The National Dam Safety Program

Research Needs Workshop: Embankment Dam Failure Analysis

FEMA

Preface

One of the activities authorized by the Dam Safety and Security Act of 2002 is research to enhance the Nation's ability to assure that adequate dam safety programs and practices are in place throughout the United States. The Act of 2002 states that the Director of the Federal Emergency Management Agency (FEMA), in cooperation with the National Dam Safety Review Board (Review Board), shall carry out a program of technical and archival research to develop and support:

- improved techniques, historical experience, and equipment for rapid and effective dam construction, rehabilitation, and inspection;
- devices for continued monitoring of the safety of dams;
- development and maintenance of information resources systems needed to support managing the safety of dams; and
- initiatives to guide the formulation of effective policy and advance improvements in dam safety engineering, security, and management.

With the funding authorized by the Congress, the goal of the Review Board and the Dam Safety Research Work Group (Work Group) is to encourage research in those areas expected to make significant contributions to improving the safety and security of dams throughout the United States. The Work Group (formerly the Research Subcommittee of the Interagency Committee on Dam Safety) met initially in February 1998. To identify and prioritize research needs, the Subcommittee sponsored a workshop on Research Needs in Dam Safety in Washington D.C. in April 1999. Representatives of state and federal agencies, academia, and private industry attended the workshop. Seventeen broad area topics related to the research needs of the dam safety community were identified.

To more fully develop the research needs identified, the Research Subcommittee subsequently sponsored a series of nine workshops. Each workshop addressed a broad research topic (listed below) identified in the initial workshop. Experts attending the workshops included international representatives as well as representatives of state, federal, and private organizations within the United States.

- Impacts of Plants and Animals on Earthen Dams
- Risk Assessment for Dams
- Spillway Gates
- Seepage through Embankment Dams
- Embankment Dam Failure Analysis
- Hydrologic Issues for Dams
- Dam Spillways
- Seismic Issues for Dams
- Dam Outlet Works

In April 2003, the Work Group developed a 5-year Strategic Plan that prioritizes research needs based on the results of the research workshops. The 5-year Strategic Plan ensures that priority will be given to those projects that demonstrate a high degree of collaboration and expertise, and the likelihood of producing products that will contribute to the safety of dams in the United States. As part of the Strategic Plan, the Work Group developed criteria for evaluating the research needs identified in the research workshops. Scoring criteria was broken down into three broad evaluation areas: value, technical scope, and product. The framework adopted by the Work Group involved the use of a "decision quadrant" to enable the National Dam Safety Program to move research along to produce easily developed, timely, and useful products in the near-term and to develop more difficult, but useful, research over a 5-year timeframe. The decision quadrant format also makes it possible to revisit research each year and to revise research priorities based on current needs and knowledge gained from ongoing research and other developments.

Based on the research workshops, research topics have been proposed and pursued. Several topics have progressed to products of use to the dam safety community, such as technical manuals and guidelines. For future research, it is the goal of the Work Group to expand dam safety research to other institutions and professionals performing research in this field.

The proceedings from the research workshops present a comprehensive and detailed discussion and analysis of the research topics addressed by the experts participating in the workshops. The participants at all of the research workshops are to be commended for their diligent and highly professional efforts on behalf of the National Dam Safety Program.

Acknowledgments

The National Dam Safety Program research needs workshop on Embankment Dam Failure Analysis was held on June 26-28, 2001, in Oklahoma City, Oklahoma.

The Department of Homeland Security, Federal Emergency Management Agency, would like to acknowledge the contributions of the Agricultural Research Service and the Natural Resources Conservation Service of the U.S. Department of Agriculture in organizing the workshop and developing these workshop proceedings. A complete list of workshop facilitators, presenters, and participants is included in the proceedings.

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1
OVERVIEW

This workshop is part of a series of workshops being sponsored by the Federal Emergency Management Agency (FEMA) and administered by the ARS/NRCS of the USDA. The workshop was a 3-day workshop on "Issues, Resolutions, and Research Needs Related to Embankment Dam Failure Analysis," held in Oklahoma City, OK, June 26-28th, 2001. The product of this workshop is the written report documenting the results of the workshop. The report will be included in FEMA's National Dam Safety Program Act Report Series.

The workshop consisted of convening and facilitating a group of experts with respect to dam safety associated with embankment dam failure analysis. The objectives of this work were:

1. To document, in the form of a final report, a state of practice concerning embankment dam failure analysis;

2. To identify short-term (immediate) and long-term research needs of the federal and non-federal dam safety community; and

3. To recommend a course of action to address these needs.

By research needs we understood the interest of the National Dam Safety Program to encompass both short-term (i.e. immediate) and long-term research including areas of development and technology transfer. These may include such areas as the following: a vision for the future of computer modeling of embankment breaching processes and flood routing, basic research of embankment overtopping and breach processes, and tools to conduct forensic studies. There were 14 areas of research identified and prioritized by workshop participants. The workshop was a successful undertaking that produced open communication among a wide range of experts in the field and identified research and development opportunities that could significantly improve the state-of-the-practice in the field.

2
OUTLINE OF THE WORKSHOP AGENDA

A group of 35 individuals were assembled for a three-day workshop on Issues, Resolutions, and Research Needs Related to Dam Failure Analyses. The group consisted of invited experts, facilitators, and the FEMA Project Officer for the workshop. The workshop participants were selected to provide broad representation of individuals in the topic area. Participants included 15 representatives of 7 different U.S. federal agencies, 5 representatives from 5 different state dam safety agencies, 9 representatives of 8 different consulting companies, 3 university professors, and 1 representative from a hydropower organization. The group included individuals from 15 different U.S. states and 4 other countries (Canada, United Kingdom, Norway, and Finland).

The first day and a half was devoted to exchange of information through presentations by the participants and discussions of embankment dam failures.
Presentations included:

1. Classification and case histories, including the human and economic consequences, of dam failure.

2. Overview of presently used tools for assessing risk, time to failure, dam failure processes, outflow hydrograph, and flood routing.

3. State assessment criteria, experience, and case examples.

A tour of the ARS Hydraulic Unit research facilities at Lake Carl Blackwell, OK was conducted in the afternoon of the second day. The tour included research projects covering:

1. Apparatus and procedure for measuring erodibility of cohesive materials in concentrated flow environments (i.e. earthen spillways, streambeds, streambanks, and embankments).

2. Riffle-pool rock chutes model for a specific application of stream stabilization on Sugar Creek, OK.

3. Performance studies of vegetated and bare earth on steep channels.

4. Embankment breach discharge model study.

5. Large-scale embankment breach failure study.

The first half of the third day was devoted to presentations and discussions on research and new technology related to risk assessment, embankment dam failure, and flood routing. The afternoon of the third day was devoted to discussions prioritizing research needs.

3
SUMMARY OF WORKSHOP PRESENTATIONS

The broad scope of the workshop presentations demonstrates the wide range of perspectives represented and the importance of the subject to the various entities involved. The material presented covered the spectrum from addressing concerns with developing solutions for specific immediate problems to identification of knowledge deficiencies that impede development of generalized tools, and from concerns related to the breach process itself to those related to the impacts of the resulting floodwave downstream. However, the presenters did an excellent job of focusing on the goals of the workshop and, together, these presentations present a relatively clear picture of the present state of the science in this area. This section provides a very brief overview of the material presented in the workshop, those presentations that included papers are referred by number to appendix B of this report.

1.1 Dam Failures

3.1.1 Classification and Case Histories of Dam Failures – Martin McCann, National Performance of Dams Program

This presentation focused on an overview of the National Performance of Dams Program (NPDP). The NPDP acts as a public library of dam performance. The NPDP has several priorities: facilitating reporting of dam performance, providing access to basic information, data compilation and presentation, and research. Dr McCann gave an incident summary for the last 10 years related to total number of incidents, type of dam, type of incident, hazard classification, and height of dam. Dr. McCann also discussed the challenges in data collection and archiving of dam incidents/failures. The information is a resource to support dam engineering, dam safety, and public policy.

3.1.2 Human and Economic Consequences of Dam Failure- Wayne Graham, USBR (B-1)

Mr. Graham's presentation focused on 13 dam failures in the U.S. Included was every U.S. dam failure that caused more than 50 fatalities. The presentation included a discussion of dam characteristics, cause of dam failure, dam failure warning (if any), evacuation, and human and economic losses. Loss of life from dam failure can vary widely. In 1889, the 72-foot high earthfill South Fork Dam near Johnstown, PA. failed, killing about 2,200 people. This can be contrasted to the period, 1985 to 1994, when hundreds of smaller dams failed in the U.S. and less than 2% of these failures caused fatalities. Many of the dam failure images in Mr. Graham's presentation are proprietary and not in the public domain. As such, dam failure images used in the presentation are not included in these proceedings.

1.2 Present Practice for Predicting Dam Failures

1.2.1 Will a Dam Failure Occur?- Risk Assessment USBR Perspective. – Bruce Muller, USBR (B-2)

Mr. Muller presented that the Bureau of Reclamation is developing a program to: 1) quantify the risk of storing water, 2) monitor aspects of performance that indicate potential for some form of failure mode to develop, and 3) take action to reduce the likelihood of dam failure. The USBR risk management responsibility comes out of the Dam Safety Act of 1978 which authorizes the Department of Interior to construct, restore, operate, maintain, new or modified features of their dams for safety purposes.

1.2.2 Will a Dam Failure Occur?- Risk Assessment USACE Perspective. – David Moser, USACE (B-3)

Mr. Moser discussed why the U.S. Army Corps of Engineers is interested in risk assessment and what their objectives are. The Corps of Engineers has approximately 570 dams, 64% of their dams are over 30 years old and 28% are over 50 years old. Approximately 10% of these dams are categorized as hydrologically or seismically deficient based on present Corps Criteria. The cost to fix these deficiencies is several billions of dollars. The Corps traditional approach to handling risk assessment has been meeting standards and criteria (i.e. design based on Probable Maximum Flood). Because of the current interest around the world, the Corps Major Rehabilitation Program, and a need for consistency with other agencies the Corps has a renewed interest in risk analysis for dam safety. Their objective is to develop methodologies, frameworks, and software tools necessary for the USACE to proactively manage the overall level of human and economic risk from their inventory of dams.

1.2.3 Methods Based on Case Study Database. – Tony Wahl, USBR (B-4)

Mr. Wahl focused his discussion on embankment dam breach parameter predictions based on case studies and the uncertainty of these parameters. This discussion was based on an evaluation of a database of 108 dam failures. The breach parameters evaluated were breach width, failure time, and peak outflow. The uncertainty of breach parameter predictions is very large. Four equations were evaluated for breach width, five equations were evaluated for failure time, and 13 equations were evaluated for peak outflow. There is room for improvement in determining these breach parameters and the uncertainty.

1.2.4 Directions for Dam-Breach Modeling/Flood Routing – Danny Fread (Retired National Weather Service). (B-5)

Dr. Fread concentrated his discussion on models he has been involved with at the National Weather Service as well as other types of models that are used in dam-breach prediction and flood routing. The dam-breach modeling involves the development of the breach as well as the peak outflow that would be used in flood routing downstream.

He also discussed research needs to improve these models such as; 1) prototype embankment experiments; 2) Manning's n and debris effects; and 3) risk or probabilistic approaches to dam failures.

1.2.5 RESCDAM-Project – Mikko Huokuna, Finnish Environment Institute (B-6)

Mr. Huokuna's presentation focused on Finland's dams and reservoirs and the RESCDAM project. Finland's dams and reservoirs have been constructed mainly for flood control, hydroelectric power production, water supply, recreation, and fish culture, as well as storing waste detrimental to health or the environment. At present, there are 55 large dams in Finland and based on Finnish dam safety legislation 36 dams require a rescue action plan. The RESCDAM project is meant to improve the dam safety sector. The activities of the RESCDAM project embrace risk analysis, dam-break flood analysis, and rescue action improvement. Recommendations for further research based on the dam break hazard analysis of the RESCDAM project include determination of breach formation, determination of roughness coefficients for the discharge channel, and the effect of debris and urban areas on floodwave propagation.

1.2.6 Hazard Classification – Alton Davis, Engineering Consultants Inc. (B-7)

Mr. Davis presented and discussed "FEMA Guidelines for Dam Safety: Hazard Potential Classification System for Dams." The FEMA guidelines specified three hazard potential classifications: 1) low hazard potential, 2) significant hazard potential, and 3) high hazard potential. The definitions of each hazard potential and selection criteria were provided in the presentation. Factors such as loss of human life, economic losses, lifeline disruption, and environmental damage affect classification.

1.3 Current Practice

3.3.1-5 State assessment Criteria, Experience and Case Examples –
John Ritchey, Dam Safety Section State of New Jersey (B-8)
Ed Fiegle, Dam Safety Section State of Georgia (B-9)
Matt Lindon, Dam Safety Section State of Utah (B-10)
David Gutierrez, Dam Safety Section State of California
Cecil Bearden, Dam Safety Section State of Oklahoma.

These presentations focused on state assessment criteria, experience and case examples in relationship to dam failure analysis. This not only included their states but also information related to states in their region. Current assessment criteria practiced at the state level for dam failure analyses is variable. Several states conduct in house assessments, some states require the dam owner to hire a licensed professional, and some allow the dam owner to conduct the assessments. The analyses are preformed for the purpose of determining hazard classifications, spillway design floods, flood zoning, and for establishing inundation areas for use in Emergency Action Plans. The methods accepted for dam failure analyses vary from state to state. Typical models that are used for conducting dam failure analysis and downstream flood routing are HEC-1, HEC-RAS, DAMBRK, FLDWAV, NWS Simplified DAMBRK, NRCS's TR-61, WSP2 Hydraulics, and the TR-66 Simplified Dam Breach Routing Procedure. There were several research needs mentioned in these presentations including; 1) establishment of a forensic team, 2) refinement of breach parameters, 3) training on present technology, 4) aids for determining Manning's n values, and 5) refining and understanding actual failure processes.

3.3.6-9 Federal Assessment Criteria, Experience, and Case Examples-
Wayne Graham, USBR (B-12)
Bill Irwin, NRCS (B-13)
James Evans and Michael Davis, FERC (B-14)

These presentations focused on the federal assessment criteria, experience, and case examples. The USBR, NRCS, and FERC each have a portfolio of dams that they have ownership of, partnership in, or regulatory responsibility over. These agencies conduct embankment dam failure analysis and inundation mapping to assign hazard classification, develop evacuation plans, assess risk, and evaluate rehabilitation needs. The criteria are agency specific with a recognized need for inter-agency coordination. There are several recognized uncertainties that require more investigation including; 1) failure analysis for different types of failure (i.e. overtopping, piping, and seismic), 2) breach characteristics, 3) Manning's roughness characteristics, 4) allowable overtopping, 5) consequences of failure, and 5) quantifying risk.

3.3.10-13 Owners and Consultants Assessment Criteria, Experience, and Case Examples-
Derek Sakamoto, BC Hydro (B-15)
Ellen Faulkner, Mead & Hunt Inc. (B-16)
Catalino Cecilio, Catalino B. Cecilio Consultants (B-17)
John Rutledge, Freeze and Nichols.

Mr. Sakamoto, representing BC Hydro, presented an overview of the Inundation Consequences Program for assessing the consequences resulting from a potential dam breach. The key focus of this program is to provide an improved tool for safety management planning. This program will provide decision makers with realistic characterizations of the various situations. It will provide investigators with the ability to determine effects of parameters such as dam breach scenarios and temporal variations related to flood wave propagation. This will also provide a powerful communication tool.

Engineering consultants throughout the United States perform dam safety assessments, which must be responsive to the needs of dam owners and to the requirements of state and federal regulatory agencies. The purpose of these studies is hazard classification, emergency action plan, or design flood assessment. Each dam failure study involves identification of a critical, plausible mode of failure and the selection of specific parameters, which define the severity of failure. These parameters include ultimate dimensions of the breach, time required to attain dimensions, and the depth of overtopping required to initiate failure. The choice of these parameters is influenced by what is reasonable to the engineer and also acceptable to the regulatory agency. The models used for dam safety assessment are based on what the regulatory agencies consider acceptable.

3.4 Research and New Technology

3.4.1 Risk Assessment Research- David Bowles, Utah State University

Dr. Bowles discussed the ASDSO/FEMA Specialty Workshop on Risk Assessment for Dams and some requirements for failure modes analyses for use in Risk Assessment. The workshop scope was to assess state of practice of risk assessment, technology transfer/training, and risk assessment needs. The outcomes of the workshop followed four major application areas in current risk assessment practice: failure modes identification, index prioritization, portfolio risk assessment, and detailed quantitative risk assessment.

Dr. Bowles also discussed requirements for failure mode analysis for use in risk assessment which included: understanding how the dam will perform under various stresses, improving capability of predicting failure, incorporation of uncertainties, and application over a range of site specific cases.

3.4.2 Research at CSU Related to Design Flood Impacts on Evaluating Dam Failure Mechanisms - Steve Abt, Colorado State University (B-18)

Dr. Abt's presentation focused on current dam safety research efforts being conducted at Colorado State University. The research at CSU has focused on dam embankment protection including: hydraulic design of stepped spillways (i.e. roller compacted concrete), hydraulic analysis of articulated concrete blocks, and design criteria for rounded rock riprap.

3.4.3 Limited Overtopping, Embankment Breach, and Discharge - Darrel Temple and Greg Hanson, USDA-ARS (B-19)

Mr. Temple and Dr. Hanson discussed research being conducted by the ARS Plant Science and Water Conservation Laboratory on overtopping of vegetated embankments. This research includes limited overtopping of grassed embankments, breach processes, and breach discharge. Long duration flow tests were conducted on steep vegetated and non-vegetated slopes. The embankment overtopping breach tests have been conducted on soil materials ranging from non-plastic sandy material to a plastic clay material. The vegetal cover and soil materials have a major impact on the timing of breach processes.

3.4.4 Dam Break Routing - Michael Gee, USACE (B-20)

Dr. Gee gave an overview of HEC models for dam break flood routing. The USACE Hydrologic Engineering Center provides a program of research, training, and technical assistance for hydrologic engineering and planning analysis. The future additions to the suite of HEC models includes dam and levee breaching (i.e. overtopping, and piping). The HEC-RAS 3.1 release will be available fall of 2001. Dr. Gee presented some examples of floodwave routing through a river system and the graphic output from HEC-RAS computations.

3.4.5 Overview of CADAM and Research - Mark Morris, HR Wallingford. (B-21)

Mr. Morris provided an overview of the CADAM Concerted Action Project and the IMPACT research project. Both of these projects have been funded by the European Commission. The CADAM project ran between Feb 98 and Jan 2000 with the aim of reviewing dambreak modeling codes and practice, from basics to application. The topics covered included analysis and modeling of flood wave propagation, breaching of embankments, and dambreak sediment effects. The program of study was such that the performance of modeling codes were evaluated against progressively more complex conditions.

The IMPACT project focuses research in a number of key areas that were identified during the CADAM project as contributing to uncertainty in dambreak and extreme flood predictions. Research areas include embankment breach, flood propagation, and sediment movement.

3.4.6 Embankment Breach Research - Kjetil Arne Vaskinn, Statkraaft Groner. (B-22)

Mr. Vaskinn discussed embankment breach research in Norway. The issue of dam safety has become more and more important in Norway during the last years and much money has been spent to increase the safety level of dams. Dam break analysis is performed in Norway to assess the consequences of dambreak and is a motivating factor for the dam safety work. Norway has started a new research project focusing on improving the knowledge in this field. The objectives of this project are to improve the knowledge of rock fill dams exposed to leakage and to gain knowledge on the development of a breach. There is overlap between the Norway project and that of IMPACT (discussed by Mr. Morris) so they will be coordinating their research efforts.

4
DAM FAILURE ANALYSIS RESEARCH AND DEVELOPMENT TOPICS

Process

Potential research and development ideas were compiled in a brainstorming session with the workshop participants divided into several small groups. The ideas from all the groups were then listed on flip charts and posted on the wall. As a group, the participants grouped and merged the ideas where possible. After all the topics were listed, the participants were asked to cast votes using three different criteria: Probability of Success, Value, and Cost. The aggregate score for each topic is based on the arithmetic sum of votes that topic received in each of the three voting categories.

Each participant was given three sets of 10 colored stickers with which to vote. Each participant was allowed to cast more than one vote per listed topic as long as the participant's total number of votes in each category did not exceed 10. The entire list of research and development topics and the results of the voting are shown below in Table 1.

TABLE 1 – RESEARCH AND DEVELOPMENT TOPICS AND VOTE TOTALS

TOPIC NUMBER	RESEARCH / DEVELOPMENT TOPIC(S)	NUMBER OF VOTES		
		Probability of Success	Value	Cost[1]
1	Update, Revise, and Disseminate the historic data set / database of dam failures. The data set should include failure information, flood information, and embankment properties.	16	16	6
2	Develop forensic guidelines and standards for dam safety experts to use when reporting dam failures or dam incidents. Create a forensic team that would be able to collect and disseminate valuable forensic data.	16	24	14
3	Produce an expert-level video of Danny Fread along the lines of the previous ICODS videos from Jim Mitchell, Don Deere, etc.	13	7	9
4	Identify critical parameters for different types of failure modes.	5	3	6

TOPIC NUMBER	RESEARCH / DEVELOPMENT TOPIC(S)	NUMBER OF VOTES		
		Probability of Success	Value	Cost[1]
5	Perform basic physical research to model different dam parameters such as soil properties, scaling effects, etc. with the intent to verify the ability to model actual dam failure characteristics and extend dam failure knowledge using scale models.	15	21	4
6	Update the regression equations used to develop the input data used in dam breach and flood routing models.	2	7	11
7	Develop better computer-based predictive models. This would preferably build upon existing technology rather than developing new software.	14	13	7
8	Develop a process that would be able to integrate dam breach and flood routing information into an early warning system.	0	0	0
9	Make available hands-on end-user training for breach and flood routing modeling that is available to government agencies and regulators, public entities (such as dam owners), and private consultants.	11	6	13
10	Validate and test existing dam breach and flood routing models using available dam failure information.	0	1	1
11	Develop a method to combine deterministic and probabilistic dam failure analyses including the probability of occurrence and probable breach location.	5	2	3
12	Using physical research data, develop guidance for the selection of breach parameters used during breach modeling.	16	20	16
13	Send U.S. representatives to cooperate with EU dam failure analysis activities.	10	7	13
14	Lobby the NSF to fund basic dam failure research.	0	2	2

[1]*A higher number in the cost category indicates a lower cost.*

The break down of the individual topics by probability of success is shown if Figure 1. and Table 2.

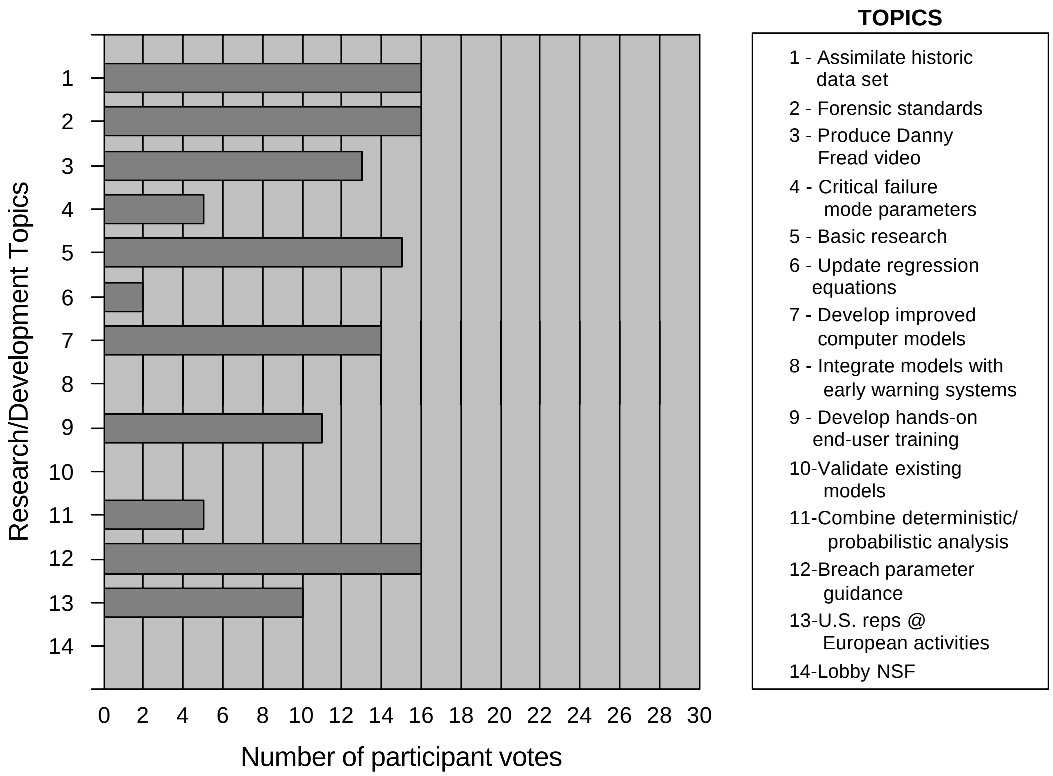


Figure 1. Probability of success

TABLE 2 – RESEARCH TOPICS RANKED BY PROBABILITY OF SUCCESS

TOPIC NUMBER	RESEARCH / DEVELOPMENT TOPIC(S)	NUMBER OF VOTES
1	Update, Revise, and Disseminate the historic data set / database. The data set should include failure information, flood information, and embankment properties.	16
2	Develop forensic guidelines and standards for dam safety experts to use when reporting dam failures or dam incidents. Create a forensic team that would be able to collect and disseminate valuable forensic data.	16
12	Using physical research data, develop guidance for the selection of breach parameters used during breach modeling.	16

TOPIC NUMBER	RESEARCH / DEVELOPMENT TOPIC(S)	NUMBER OF VOTES
5	Perform basic physical research to model different dam parameters such as soil properties, scaling effects, etc. with the intent to verify the ability to model actual dam failure characteristics and extend dam failure knowledge using scale models.	15
7	Develop better computer-based predictive models. Preferably build upon existing technology rather than developing new software.	14
3	Produce an expert-level video of Danny Fread along the lines of the previous ICODS videos from Jim Mitchell, Don Deere, etc.	13
9	Make available hands-on end-user training for breach and flood routing modeling that is available to government agencies and regulators, public entities (such as dam owners), and private consultants.	11
13	Send U.S. representatives to cooperate with EU dam failure analysis activities.	10
4	Identify critical parameters for different types of failure modes	5
11	Develop a method to combine deterministic and probabilistic dam failure analyses including the probability of occurrence and probable breach location.	5
6	Update the regression equations used to develop the input data used in dam breach and flood routing models.	2
8	Develop a process that would be able to integrate dam breach and flood routing information into an early warning system.	0
10	Validate and test existing dam breach and flood routing models using available dam failure information.	0
14	Lobby the NSF to fund basic dam failure research.	0

The break down of the individual topics by value of the item is shown if Figure 2. and Table 3.

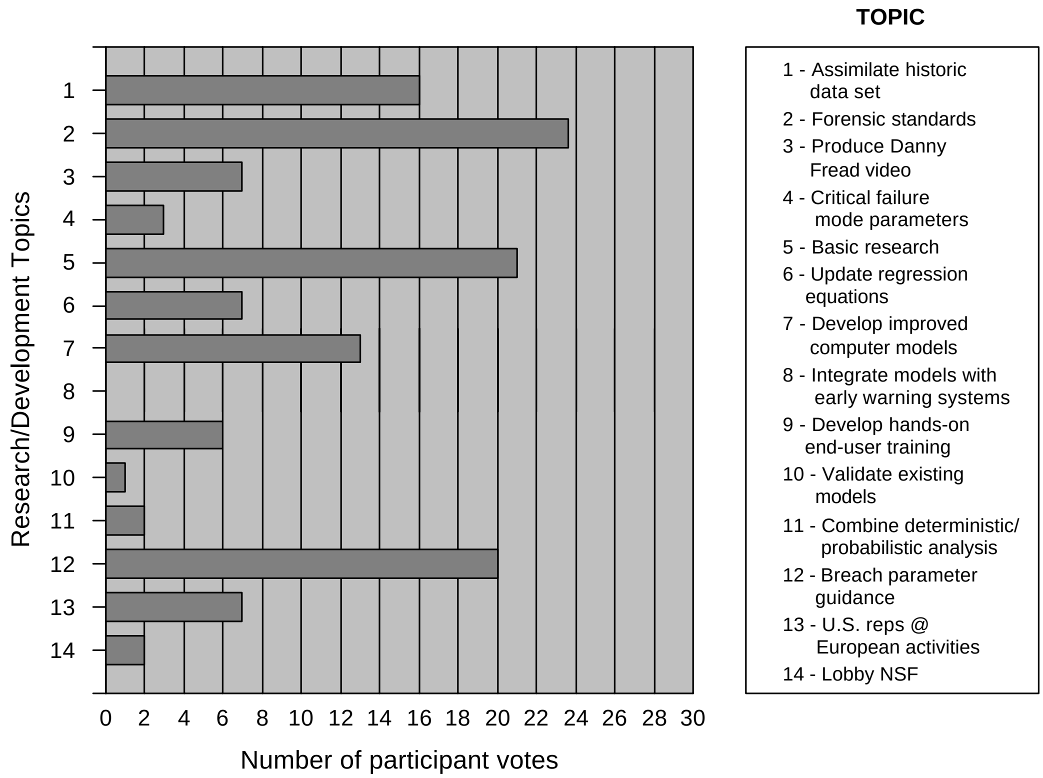


Figure 2. Value of research topic

TABLE 3 – RESEARCH TOPICS RANKED BY VALUE

TOPIC NUMBER	RESEARCH / DEVELOPMENT TOPIC(S)	NUMBER OF VOTES
2	Develop forensic guidelines and standards for dam safety experts to use when reporting dam failures or dam incidents. Create a forensic team that would be able to collect and disseminate valuable forensic data.	24
5	Perform basic physical research to model different dam parameters such as soil properties, scaling effects, etc. with the intent to verify the ability to model actual dam failure characteristics and extend dam failure knowledge using scale models.	21

TOPIC NUMBER	RESEARCH / DEVELOPMENT TOPIC(S)	NUMBER OF VOTES
12	Using physical research data, develop guidance for the selection of breach parameters used during breach modeling.	20
1	Update, Revise, and Disseminate the historic data set / database. The data set should include failure information, flood information, and embankment properties.	16
7	Develop better computer-based predictive models. Preferably build upon existing technology rather than developing new software.	13
3	Produce an expert-level video of Danny Fread along the lines of the previous ICODS videos from Jim Mitchell, Don Deere, etc.	7
6	Update the regression equations used to develop the input data used in dam breach and flood routing models.	7
13	Send U.S. representatives to cooperate with EU dam failure analysis activities.	7
9	Make available hands-on end-user training for breach and flood routing modeling that is available to government agencies and regulators, public entities (such as dam owners), and private consultants.	6
4	Identify critical parameters for different types of failure modes	3
11	Develop a method to combine deterministic and probabilistic dam failure analyses including the probability of occurrence and probable breach location.	2
14	Lobby the NSF to fund basic dam failure research.	2
10	Validate and test existing dam breach and flood routing models using available dam failure information.	1

TOPIC NUMBER	RESEARCH / DEVELOPMENT TOPIC(S)	NUMBER OF VOTES
8	Develop a process that would be able to integrate dam breach and flood routing information into an early warning system.	0

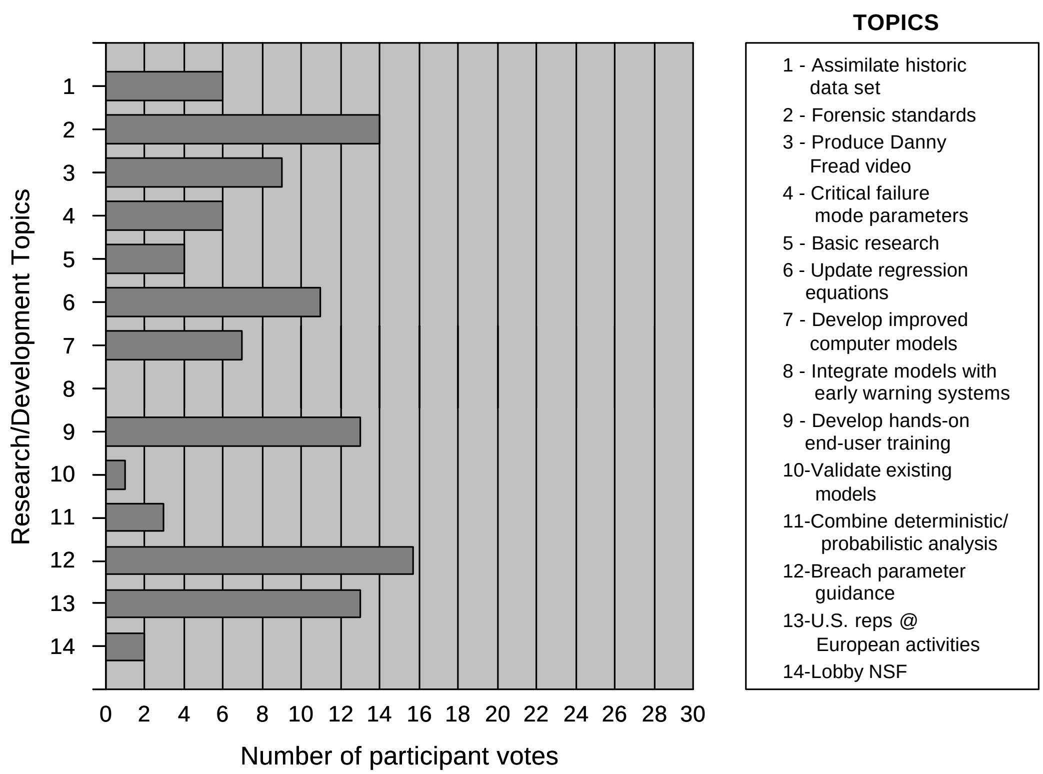


Figure 3. Cost of research topic (the more votes the lower the cost).

TABLE 4 – RESEARCH TOPICS RANKED BY COST

TOPIC NUMBER	RESEARCH / DEVELOPMENT TOPIC(S)	NUMBER OF VOTES

TOPIC NUMBER	RESEARCH / DEVELOPMENT TOPIC(S)	NUMBER OF VOTES
12	Using physical research data, develop guidance for the selection of breach parameters used during breach modeling.	16
2	Develop forensic guidelines and standards for dam safety experts to use when reporting dam failures or dam incidents. Create a forensic team that would be able to collect and disseminate valuable forensic data.	14
9	Make available hands-on end-user training for breach and flood routing modeling that is available to government agencies and regulators, public entities (such as dam owners), and private consultants.	13
13	Send U.S. representatives to cooperate with EU dam failure analysis activities.	13
6	Update the regression equations used to develop the input data used in dam breach and flood routing models.	11
3	Produce an expert-level video of Danny Fread along the lines of the previous ICODS videos from Jim Mitchell, Don Deere, etc.	9
7	Develop better computer-based predictive models. Preferably build upon existing technology rather than developing new software.	7
1	Update, Revise, and Disseminate the historic data set / database. The data set should include failure information, flood information, and embankment properties.	6
4	Identify critical parameters for different types of failure modes	6
5	Perform basic physical research to model different dam parameters such as soil properties, scaling effects, etc. with the intent to verify the ability to model actual dam failure characteristics and extend dam failure knowledge using scale models.	4
11	Develop a method to combine deterministic and probabilistic dam failure analyses including the probability of occurrence and probable breach location.	3

TOPIC NUMBER	RESEARCH / DEVELOPMENT TOPIC(S)	NUMBER OF VOTES
14	Lobby the NSF to fund basic dam failure research.	2
10	Validate and test existing dam breach and flood routing models using available dam failure information.	1
8	Develop a process that would be able to integrate dam breach and flood routing information into an early warning system.	0

Prioritization of Research Topics

After the votes were tabulated, each research topic was ranked according to the aggregate total of votes cast. The rank of each topic in Table 5 and Figure 4 is a reflection of the combination of value, cost, and probability of success, based on equal weighting, as determined by the participants. Based on all the input by the participants, it is the author's opinion that the following topics were the leading research and development ideas identified in the workshop.

1. Develop forensic guidelines and standards for dam safety representatives and experts to use when reporting dam failures or dam incidents. Create a forensic team that would be able to collect and disseminate valuable forensic data. (Topic #2)

2. Using physical research data, develop guidance for the selection of breach parameters used during breach modeling. (Topic #12)

3. Perform basic physical research to model different dam parameters such as soil properties, scaling effects, etc. with the intent to verify the ability to model actual dam failure characteristics and extend dam failure knowledge using scale models. (Topic #5)

4. Update, revise, and disseminate information in the historic data set / database. The data set should include failure information, flood information, and embankment properties. (Topic #1)

5. Develop better computer-based predictive models. Preferably these models would build upon existing technology rather than developing new software. (Topic #7)

6. Make available hands-on end-user training for breach and flood routing modeling which would be available to government agencies and regulators, public entities (such as dam owners), and private consultants. (Topic #9)

7. Record an expert-level video of Danny Fread along the lines of the previous ICODS videos from Jim Mitchell, Don Deere, etc. (Topic #3)

8. Send U.S. representatives to cooperate with EU dam failure analysis activities. (Topic #13)

The participants ranked the previous eight topics the highest overall when the three different criteria were averaged. The listing of the top 8 here is purely an arbitrary cut-off by the author.

Overall, there were fewer votes cast for cost than for the other two ranking criteria. This is probably due to the fact that cost is more difficult to estimate than the value or probability of success. Because of this, the topics above may be in a slightly different order if cost is not considered as a ranking criterion.

It is interesting to note that only fourteen topics were identified during the workshop. Previous workshops on different subjects identified a substantial number of topics, and then their ranking method narrowed their priority list down to a manageable number. This is not necessarily an indication that there is less to accomplish in the area of dam failure analysis, it is more an indication that this particular workshop attempted to combine many tasks into one research topic. It is the author's opinion that many of the identified priority items can be broken down into several distinct sub-topics, and doing so may make it easier to cooperatively address the research needs listed here.

In-order to identify short-term research versus long-term research items the votes cast for cost were plotted against value for each of the 14 research topics and the plot was broken into 4 quadrants (Figure 5). The upper left quadrant corresponded to those items that the participants deemed were of high value and low cost to accomplish. They were therefore labeled low hanging fruit and could be looked upon as short-term research items. Items 2 and 12 fell into this quadrant, which were the top two in the overall score. The upper right were items that based on relative comparison were high value but also high cost. This quadrant was labeled 'strategic plan' indicating that the items falling in this quadrant would be long-term research items. Items 1, 5, and 7 fell into this quadrant. These items were also ranked 3 – 5 in the overall scoring. The lower left quadrant was labeled 'do later' and based on relative comparisons contained research items that were low cost and low value. Items 3, 6, 9 and 13 fell into this quadrant, 3, 9 and 13 were also ranked 6 – 8 in the overall ranking. The lower right quadrant was labeled 'consider' and based on relative comparisons contained research items that were low cost and low value. Research items 4, 8, 10, 11, and 14 fell into this quadrant. This comparison may be found useful in determining the most effective use of limited resources.

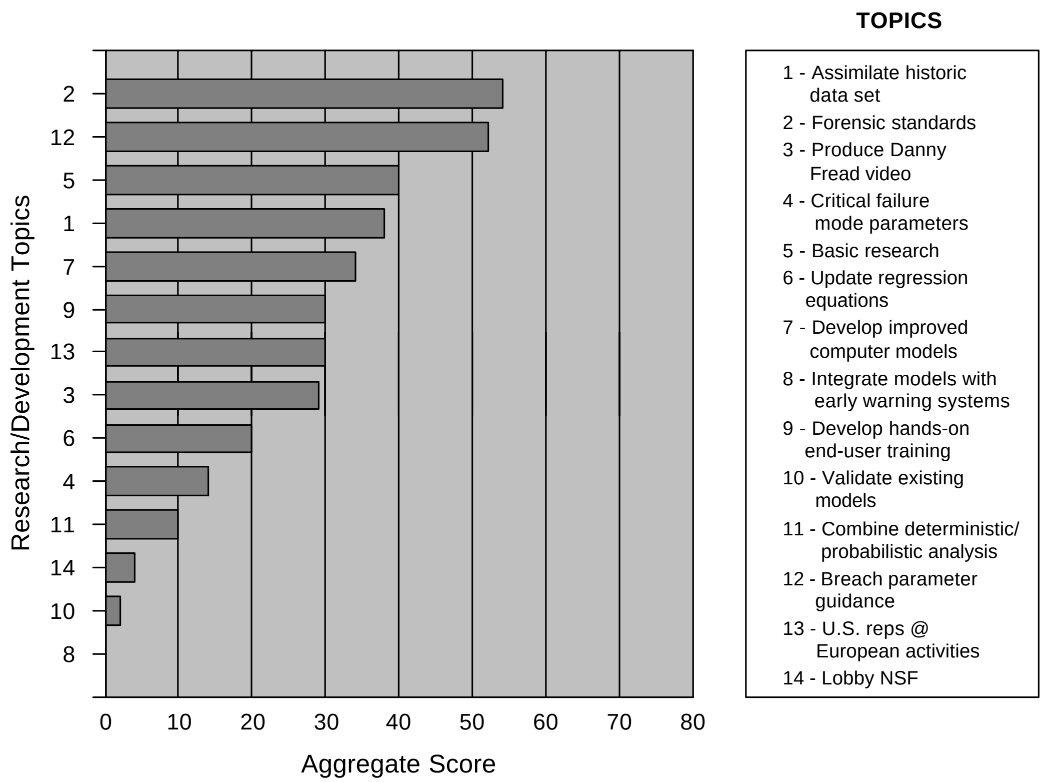


Figure 4. Research topic ranked by aggregate score of probability of success, value, and cost.

TABLE 5 – RESEARCH TOPICS RANKED BY AGGREGATE SCORE

TOPIC NUMBER	RESEARCH / DEVELOPMENT TOPIC(S)	AGGREGATE SCORE	RANK
2	Develop forensic guidelines and standards for dam safety experts to use when reporting dam failures or dam incidents. Create a forensic team that would be able to collect and disseminate valuable forensic data.	54	1
12	Using physical research data, develop guidance for the selection of breach parameters used during breach modeling.	52	2
5	Perform basic physical research to model different dam parameters such as soil properties, scaling effects, etc. with the intent to verify the ability to model actual dam failure characteristics and extend dam failure knowledge using scale models.	40	3

TOPIC NUMBER	RESEARCH / DEVELOPMENT TOPIC(S)	AGGREGATE SCORE	RANK
1	Update, Revise, and Disseminate the historic data set / database. The data set should include failure information, flood information, and embankment properties.	38	4
7	Develop better computer-based predictive models. Preferably build upon existing technology rather than developing new software.	34	5
9	Make available hands-on end-user training for breach and flood routing modeling that is available to government agencies and regulators, public entities (such as dam owners), and private consultants.	30	6
13	Send U.S. representatives to cooperate with EU dam failure analysis activities.	30	7
3	Record an expert-level video of Danny Frease along the lines of the ICODS videos from Jim Mitchell, Don Deer, etc.	29	8
6	Update the regression equations used to develop the input data used in dam breach and flood routing models.	20	9
4	Identify critical parameters for different types of failure modes	14	10
11	Develop a method to combine deterministic and probabilistic dam failure analyses including the probability of occurrence and probable breach location.	10	11
14	Lobby the NSF to fund basic dam failure research.	4	12
10	Validate and test existing dam breach and flood routing models using available dam failure information.	2	13
8	Develop a process that would be able to integrate dam breach and flood routing information into an early warning system.	0	14

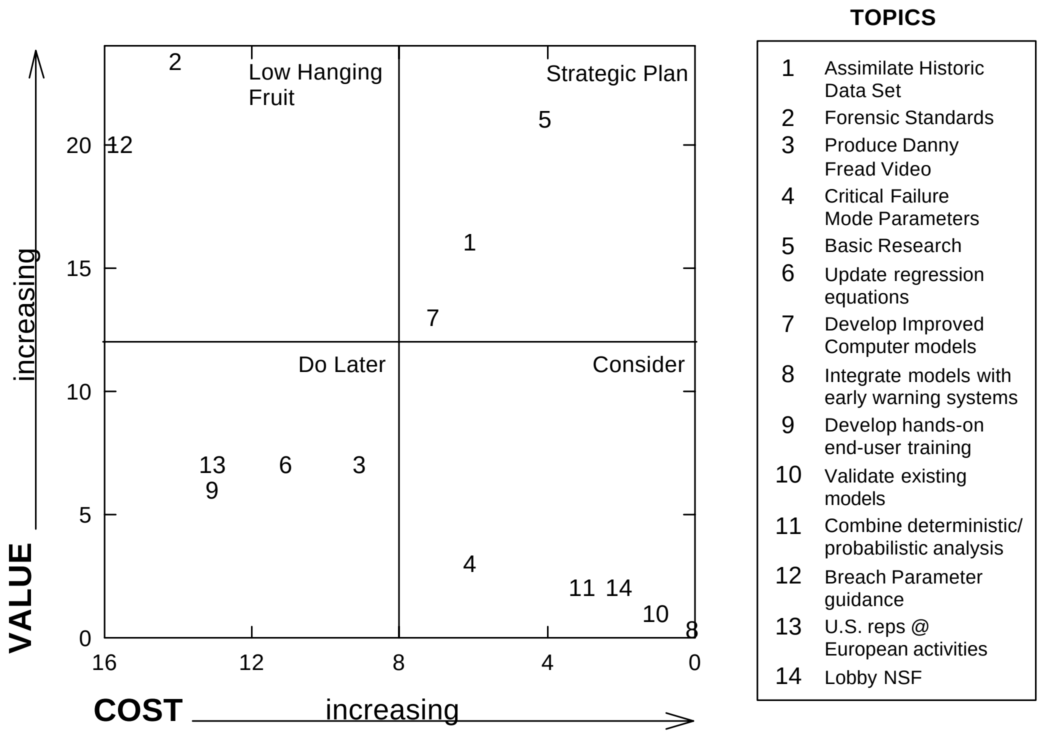


Figure 5. Decision quadrant.

APPENDICES

A
AGENDA

AGENDA FOR WORKSHOP ON ISSUES, RESOLUTIONS, AND RESEARCH NEEDS RELATED TO DAM FAILURE ANALYSES

TUESDAY, June 26

Morning

Introduction to Workshop	**Presenter**	
Introductions	(Darrel Temple)	0730
Purpose (where workshop fits into scheme of workshops.)	(Gene Zeizel)	0745
Dam Failures		
Classification & Case Histories of Dam Failures	(Martin McCann)	0800
Human and Economic Consequences of Dam Failure	(Wayne Graham)	0830
Present Practice for Predicting Dam Failures		
Overview of Presently Used tools		
a. Will a Dam Failure Occur?		
i. Risk Assessment – USBR Perspective	(Bruce Muller)	0850
ii. Risk Assessment – USACE Perspective	(David Moser)	0910
b. Time to Failure, Dam Failure Processes, Prediction of Dam Failure Discharge; Peak Discharge and Outflow Hydrograph.		
i. Methods Based on Case Study Database.	(Tony Wahl)	0930
Break		1000
ii. Some Existing Capabilities and Future Directions for Dam-Breach Modeling/ Flood Routing	(Danny Fread)	1015
c. Ultimate Use of Peak Discharge and Outflow Hydrograph.		
i. RESCDAM-project	(Mikko Huokuna)	1045
ii. Hazard Classification	(Al Davis, ICODS)	1115
Lunch Break		1145
Afternoon		
Current Practice		
State Assessment Criteria, Experience, and Case Example		
New Jersey	(John Ritchey)	1300
Georgia	(Ed Fiegle)	1320
Utah	(Matt Lindon)	1340
California	(David Gutierrez)	1400
Oklahoma	(Cecil Bearden)	1420
Break		1440
Federal Assessment Criteria, Experience, and Case Example		
Bureau of Reclamation	(Wayne Graham)	1500
NRCS	(Bill Irwin)	1520
FERC	(James Evans and Michael Davis)	1540

WEDNESDAY, June 27

Morning

Current Practice (cont.)

Private Experience and Case Example		
Owners		
BC Hydro	(Derek Sakamoto)	0740
Consultants		
Mead &Hunt Inc.	(Ellen Faulkner)	0810
Catalino B. Cecilio Consult.	(Catlino Cecilio)	0830
Freeze & Nichols	(John Rutledge)	0850
Break		0910-0930
Group Discussions	(Nate Snorteland)	0930
Lunch Break		1200

Afternoon

Tour of ARS Hydraulic Laboratory		1300-1500

THURSDAY, June 28

Morning

Research And New Technology

Risk Assessment Research	(David Bowles)	0800
Overtopping and Breach Research		
Research at CSU Related to Design Flood Impacts on Evaluating Dam Failure Mechanisms	(Steve Abt)	0830
Limited Overtopping, Embankment Breach and Discharge	(Temple and Hanson)	0900
Break		1000
Dam Break Routing	(Michael Gee)	1020
Overview of CADAM and Research	(Mark Morris)	1050
Embankment Breach Research	(Kjetil Arne Vaskinn)	1120
Lunch Break		1150

Afternoon

Group Discussions	(Nate Snorteland)	1330-1600

B
PRESENTATIONS

Human and Economic Consequences of Dam Failure

by

Wayne Graham, June 26, 2001

Dam	Date and Time of Failure	Dam Height (ft)	Volume Released (ac-ft)	Deaths	Economic Damage
Williamsburg Dam, MA (Mill River Dam)	May 16, 1874 at 7:20 a.m.	43	307	138	
South Fork Dam, PA (Johnstown Dam)	May 31, 1889 at 3:10 p.m.	72	11,500	2,209	
Walnut Grove Dam, AZ	February 22, 1890 at 2 a.m.	110	60,000	about 85	
Austin Dam, PA	September 30, 1911 at2 p.m.	50	850	78	
St. Francis Dam, CA	March 12-13, 1928 at midnight	188	38,000	420	$14 m
Castlewood Dam, CO	August 2-3, 1933 at midnight	70	5,000	2	$2 m
Baldwin Hills Dam, CA	December 14, 1963 at 3:38 p.m.	66	700	5	$11 m
Buffalo Creek, WV (Coal Waste Dam)	February 26, 1972 at 8 a.m.	46	404	125	$50 m
Black Hills Flood, SD (Canyon Lake Dam)	June 9, 1972 at about 11 p.m.	20	700	???	$160 m (All flooding)
Teton Dam, ID	June 5, 1976 at 11:57 a.m.	305	250,000	11	$400 m
Kelly Barnes Dam, GA	November 6, 1977 at 1:20 a.m.	40	630	39	$3 m
Lawn Lake Dam, CO	July 15, 1982 at 5:30 a.m.	26	674	3	$31 m
Timber Lake Dam, VA	June 22, 1995 at 11 p.m.	33	1,449	2	$0 m

Dam name: Williamsburg Dam (Mill River Dam)

Location: on east branch Mill River, 3 miles north of Williamsburg, MA

Dam Characteristics:

Dam type: earthfill with masonry core wall
Dam height: 43 feet - at time of failure, water 4 feet below crest
Dam crest length: 600 feet
Reservoir volume: 307 acre-feet
Spillway: 33 feet wide

History of Dam:

Purpose: Increase water supply to mill operators
Dam completed: 1865, just months after civil war.
Dam failed: Saturday May 16, 1874 (9 years old) (20 minutes after initial slide, entire dam failed)
Failure cause: Seepage carried away fill, embankment sliding, then collapse of masonry core wall (internal erosion)

Details on Detection of Failure/Deciding to warn: After observing large slide, gatekeeper (Cheney) rode 3 miles on horseback to Williamsburg. Another person living near dam ran 2 miles in 15 minutes after seeing the top of the dam break away.

Details on dissemination of warnings and technologies used: The gatekeeper (who had not seen the large reservoir outflow) got to Williamsburg at about the time the dam broke. He conferred with reservoir officials and changed his horse. Some overheard the conversation and a milkman (Graves) traveled by horse and warned mills downstream. Many people received either no warning or only a few minutes of warning.

Details on response to the warning:

Description of flooding resulting from dam failure: 20 to 40 foot high floodwave crumpled brass, silk, and button mills, crushed boarding houses, farmhouses and barns.

The losses included: 138 dead, 750 people homeless

Location	mileage	flood arrived	dead	
Dam	0	7:20?		
Williamsburg	3	7:40	57	flood 300 feet wide
Skinnerville	4		4	
Haydenville	5	7:45	27	
Leeds	7	8:05	50	
Florence	10	8:35	0	

All 138 fatalities occurred in the first 7 miles downstream from the dam.
Prepared by Wayne Graham

Dam name: South Fork (Johnstown)

Location: On South Fork Little Conemaugh River

Dam Characteristics:

Dam type: earthfill
Dam height: 72 feet
Dam crest length: feet
Reservoir volume: 11,500 acre-feet
Spillway:

History of Dam:

Purpose: Originally for supplying water to canal system; at time of failure was owned by South Fork Hunting and Fishing Club of Pittsburgh.
Dam completed: 1853
Dam failed: May 31, 1889 about 3:10 pm (about 36 years old)
Failure cause: overtopping during an approximate 25-year storm (Drainage area of about 48 sq. mi.)

Details on Detection of Failure/Deciding to warn:

People were at dam trying to prevent dam failure. Between 11:30 and noon the resident engineer, on horseback, reached the town of South Fork (2 miles from dam) with a warning. Word was telegraphed to Johnstown that dam was in danger.

Details on dissemination of warnings and technologies used:
Warnings were not widely disseminated.

Details on response to the warning:
- Little attention paid to warnings due to false alarms in prior years.
- At time of failure, Johnstown was inundated by up to 10 feet of floodwater.

Description of flooding resulting from dam failure: Floodwater reached Johnstown, mile 14, about 1 hour after failure. Large number of buildings destroyed.

The losses included: about 2,209 fatalities; 20,000 people at risk.

All, or nearly all, of the fatalities occurred in the first 14 miles downstream from South Fork Dam.

Prepared by Wayne Graham

Dam name: Walnut Grove Dam

Location: On the Hassayampa River, about 40 miles south of Prescott, AZ

Dam Characteristics:

Dam type: Rockfill
Dam height: 110 feet
Dam crest length: 400 feet
Reservoir volume: 60,000 acre-feet?
Spillway: 6 feet by 26 feet

History of Dam:

Purpose: Irrigation and gold placer mining. Dam completed: October 1887
Dam failed: 2 a.m. February 22, 1890 (2 years old)
Failure cause: Overtopped (inadequate spillway cap and poor construction workmanship). The dam withstood 3 feet of overtopping for 6 hours before failing.

Details on Detection of Failure/Deciding to warn:

11 hours before dam failure an employee was directed by the superintendent of the water storage company to ride by horseback and warn people at a construction camp for a lower dam about 15 miles downstream from Walnut Grove Dam.

Details on dissemination of warnings and technologies used:

The rider on horseback never reached the lower camp.

Details on response to the warning:

The majority of the 150 or more inhabitants of the (Fools Gulch) camp were calmly sleeping in their tents. When the roar of the approaching water became audible, it was almost too late for escape up the hillsides, yet many reached safety by scrambling up the hillside through cactus and rocks.

Description of flooding resulting from dam failure:

Floodwaters reached depths of 50 to 90 feet in the canyon downstream from the dam.

The losses included:

70 to 100 fatalities

Prepared by Wayne Graham

Dam name: Austin Dam

Location: On Freeman Run, about 1.5 miles upstream from Austin, Pennsylvania. The dam is located in western PA., about 130 miles northeast of Pittsburgh.

Dam Characteristics:

Dam type: Concrete gravity
Dam height: Between 43 and 50 feet
Dam crest length: 544 feet
Reservoir volume: Between 550 and 850 acre-feet
Spillway: 50 feet long and 2.5 feet deep

History of Dam:

Dam completed: November 1909
Partial failure: January 1910; part of dam moved 18 inches at base and 34 inches at the top.
Dam failed: 2pm or 2:20 pm, September 30, 1911 (2 years old)
Failure cause: Weakness of the foundation, or of the bond between the foundation and concrete.

Details on Detection of Failure/Deciding to warn:

Harry Davis, boarding in a house on the mountain slope near the dam phoned the Austin operators at whose warning the paper mill whistle sounded - about 2 pm. The phone operators warned others but many ignored the warnings.

Details on dissemination of warnings and technologies used:

The mill whistle had blown twice earlier in the day as false signals had been received from telephone company employees who had been repairing telephone lines. The two false alarms were the cause of many people losing their lives as many people assumed the whistle (sounded to warn of dam failure) was another false alarm. Warnings were issued to people in Costello, about 5 miles downstream from the dam. (A person riding a bicycle traveled from the south side of Austin to Costello to spread the warning).

Details on response to the warning:

Description of flooding resulting from dam failure:

The water traveled from the dam to the town of Austin, a distance of 1.5 miles, in either 11 minutes or in up to 20 to 30 minutes. This results in a travel time of between 3 and 8 miles per hour.

The losses included:

At least 78 fatalities, all in the first 2 miles downstream from the dam, i.e. in the Austin area. (About 3 or 4 percent of Austin's 2300 population)
Prepared by Wayne Graham

Dam name: Saint Francis Dam

Location: north of Los Angeles, CA

Dam Characteristics:

Dam type: Concrete Gravity
Dam height: 188 feet
Dam crest length: ?? feet
Reservoir volume: 38,000 acre-feet
Spillway:

History of Dam:

Purpose: LA Water Supply
Dam completed:
Dam failed: About midnight March 12, 1928 (2 years old)
Failure cause: Foundation failure at abutment

Details on Detection of Failure/Deciding to warn:

No detection before failure. Ventura County Sheriffs office informed at 1:20 am.

Details on dissemination of warnings and technologies used:

Once people learned of failure, telephone operators called local police, highway patrol and phone company customers. Warning spread by word of mouth, phone, siren and law enforcement in motor vehicles.

Details on response to the warning:

Description of flooding resulting from dam failure:

Flooding was severe through a 54 miles reach from dam to ocean. The leading edge of the flooding moved at 18 mph near dam and 6 mph nearer the ocean.

The losses included: 420 fatalities. About 3,000 people were at risk. Damage total of about $13.5 million includes death claims.

Photos from USGS library and Ventura County Museum of History and Art.

Photos:

Site	mileage	flood arrived	dead
big pile (power plant)	1.5	5 minutes	> 11 out of 50
Cal Edison const. camp 17		lhr 20mm	89 of 150
Santa Paula	38.5	3 hours	yes

Prepared by Wayne Graham

Dam name: Castlewood Dam

Location: near Franktown, Colorado (about 35 miles upstream from Denver.

Dam Characteristics:

Dam type: rockfill
Dam height: 70 feet
Dam crest length: 600 feet
Reservoir volume: 3430 acre-feet at spillway crest; 5000 acre-feet at elevation of failure.
Spillway: Central overflow, 100 feet long, 4 feet deep.

History of Dam:

Purpose: irrigation
Dam completed: 1890
Dam failed: about midnight August 3, 1933
Failure cause: overtopping

Details on Detection of Failure/Deciding to warn:

The dam begins failing due to overtopping about• midnight. Caretaker lives nearby but phone not working. Drives 12 miles to use phone. At 2:30 a.m. caretaker uses phone to initiate warning process.

Details on dissemination of warnings and technologies used:

Residents in upstream areas probably received no official warning. Flooding occurred in Denver between about 5:30 a.m. and 8:00 a.m. Warnings by police and firemen preceded the arrival of floodwaters.

Details on response to the warning:

Many people evacuated. A newspaper reported, "A stampede of 5,000, man clad in nightclothes, fled from the lowlands." People also drove to the banks of Cherry Creek to view the flood.

Description of flooding resulting from dam failure:

In the Denver area, flooding caused significant damage. The flood depth and velocity, however, were not great enough to destroy (move or collapse) buildings.

The losses included:

2 fatalities occurred. A woman was thrown into Cherry Creek while viewing the flood on horseback and a man stepped into a deep hole while wading toward high ground. $1.7 million in. damage.

Prepared by Wayne Graham

Dam name: Baldwin Hills Dam

Location: Los Angeles, California. The dam was located about midway between downtown L.A., and LAX (L.A. International Airport).

Dam Characteristics:

Dam type: earthfill
Dam height: 65.5 feet. Water depth of 59 feet when break occurred.
Dam crest length: not determined
Reservoir volume: about 700 acre-feet at time of failure.
Spillway: of f stream storage at top of hill. No spillway?

History of Dam:

Purpose: water supply
Dam completed: 1950
Dam failed: Saturday, December 14, 1963 at 3:38 p.m.
Failure cause: displacement in the foundation

Details on Detection of Failure/Deciding to warn:

11:15 a.m.: crack discovered in dam
12:20 p.m.: reservoir draining begins
1:30 p.m.: LA Dept of Water and Power notifies police
1:45 p.m.: decision made to evacuate
2:20 p.m.: evacuation begins
3:38 p.m.: dam fails

Details or dissemination of warnings and technologies used:

Warnings disseminated by police in patrol cars, motorcycle and helicopter. This event was covered by radio and television.

Details on response to the warning:

Many people evacuated but “some people were not taking the warnings seriously.

Description of flooding resulting from dam failure:

Flooding extended about 2 miles from dam. Affected area was about 1 square mile which contained about 16,500 people. The fatalities occurred about I mile downstream from the dam.

The losses included:

5 fatalities; they all resided in condo complex that was flooded but not destroyed. 41 homes destroyed; 986 houses and 100 apt. buildings damaged, 3000 automobiles damaged. Damage reported to be $11.3 million.

Prepared by Wayne Graham

Dam name: Buffalo Creek Coal Waste Dam

Location: near Saunders, West Virginia

Dam Characteristics:

Dam type: coal waste
Dam height: 46 feet
Dam crest length: feet
Reservoir volume: 404 acre-feet
Spillway: small pipe

History of Dam:

Purpose: improve water quality, dispose of coal waste
Dam completed: continually changing
Dam failed: February 26, 1972 about 8 a.m. (0 years old)
Failure cause: Slumping of dam face during 2-year rain.

Details on Detection of Failure/Deciding to warn: Owner reps were on site monitoring conditions prior to dam failure. "At least two dam owner officials urged the Logan County Sheriff's force to refrain from a massive alert and exodus."

Details on dissemination of warnings and technologies used:
Company officials issued no warnings. The senior dam safety official on the site dismissed two deputy sheriffs (at about 6:30 a.m.) who had been called to the scene to aid evacuation.

Details on response to the warning: Resident's reaction to the meager warnings that were issued were dampened due to at least 4 previous false alarms.

Description of flooding resulting from dam failure:

Wave traveled downstream through the 15-mile long valley at 5mph. Over 1,000 homes either destroyed or damaged.

The losses included: 125 deaths; 4,000 people homeless

All of the fatalities occurred in the first 15 miles downstream from the dam.

Damage total of $50 million.

Prepared by Wayne Graham

Dam name: Black Hills Flash Flood (Canyon Lake Dam)

Location: Rapid City, South Dakota

Dam Characteristics:

Dam type: earthfill
Dam height: 20+? feet
Dam crest length: 500 feet
Reservoir volume: about 700 acre-feet of water released
Spillway: Capacity of 3,200 cfs

History of Dam:

Purpose: Recreational lake in city park
Dam completed: 1933
Dam failed: June 9, 1972 Reports varied between 10:45 and 11:30 (39-years old when failed)
Failure cause: Overtopping

Details on Detection of Failure/Deciding to warn:

There were no dam failure warnings and virtually no flood warnings in Rapid City. The 10pm TV news wrap-up indicated that the magnitude and seriousness of the flood was not realized at that time. At 10:30 pm, in simultaneous TV and radio broadcast, people in low-lying areas were urged to evacuate.

Details on dissemination of warnings and technologies used:

The initial warnings did not carry a sense of urgency because of the complete lack of knowledge concerning the incredible amount of rain that was falling.

Details on response to the warning:

Description of flooding resulting from darn failure:
Water started flowing over Canyon Lake Dam at 10 am or earlier. The dam failed at 10:45 pm (or as late as 11:30 pm)

Peak inflow was about 43,000 cfs
Peak outflow was about 50,000 cfs
Flood in Rapid City covered an area up to 0.5 miles wide.

The losses included:

236 fatalities with 17,000 at risk. Of the fatalities: 35 occurred in first 3 miles above dam; 165 below dam; 36 elsewhere Incremental fatalities resulting from dam failure: ???
3,000 injured. Flooding, including that from dam failure, destroyed or caused major damage to over 4,000 permanent residences and mobile homes. Damage total (failure plus non failure): $160 million.

Prepared by Wayne Graham

Dam name: Teton Dam

Location: near Wilford, Idaho

Dam Characteristics:

Dam type: earthf ill
Dam height: 305 feet (275 depth at failure)
Dam crest length: feet
Reservoir volume: 250,000 acre-feet released
Spillway: water never reached spillway

History of Dam:

Purpose: irrigation
Dam completed: under final construction/first filling
Dam failed: Saturday June 5, 1976 at 11:57 a.m.; first filling
Failure cause: Piping of dam core in foundation key trench.

Details on Detection of Failure/Deciding to warn:

12:30 am and 7am: dam unattended.
7am to 8am: Survey crew discovers turbid leakage
9:30 am: PCE considers alerting residents but decides emergency situation is not imminent and is concerned about causing panic.
10 am: larger leak, flowing turbid water
10:30 to 10:45: PCE notifies sheriff's offices and advises them to alert citizens.

Details on dissemination of warnings and technologies used:
police, radio, television, telephone, neighbor word of mouth. (Included live commercial radio broadcasts from reporters in aircraft and at Teton Dam)

Details on response to the warning:
Why were there 800 injured?

Description of flooding resulting from dam failure:
Over 3,700 houses destroyed or damaged. 150 to 200 sq. mi. flooded

The losses included:
11 fatalities (6 from drowning, 3 heart failure, 1 accidental gun shot and 1 suicide) with about 25,000 people at risk. 800 injuries. Damage total of $400 million from USGS Open File Report 77-765.

Photos:

Sugar City	12 mi.	1pm	15 ft depth	(0 dead)
Rexburg	15 mi.	1:40pm	6 to 8 ft.	(2 deaths)

Prepared by Wayne Graham

Dam name: Kelly Barnes Dam

Location: on Toccoa Creek, near Toccoa Falls, Georgia.

Dam Characteristics:

Dam type: earthfill
Dam height: about 40 feet
Dam crest length: 400 feet
Reservoir volume: 630 acre-feet at time of failure
Spillway:

History of Dam:

Purpose: Originally for hydropower. Hydropower abandoned in 1957 and then used for recreation.
Dam completed: 1899. Enlarged/modified in 1937 and after 1945. Dam failed: Sunday, November 6, 1977 at 1:20 a.m.
Failure cause: Saturation due to heavy rain caused downstream slope failure.

Details on Detection of Failure/Deciding to warn:

Two volunteer firemen examined the dam around 10:30 p.m. and radiod that dam was solid and that there was no need for concern or alarm.

Details on dissemination of warnings and technologies used:

With concern over rising water, not dam failure, 1 or 2 families were warned by volunteer firemen just minutes before dam failure.

Details on response to the warning:

Most people were not warned. It would have been horrible conditions for evacuation - dark, rainy and cold.

Description of flooding resulting from dam failure:

Flood reached depths of 8 to 10 feet in populated floodplain.

The losses included:

39 fatalities, all within 2 miles of the dam. 9 houses, 18 house trailers, 2 college buildings demolished. 4 houses and 5 college buildings damaged. Damage total of $2.8 million.

Prepared by Wayne Graham

Dam name: Lawn Lake Dam

Location: In Rocky Mountain National Park, Colorado

Dam Characteristics:

Dam type: Earthfill
Dam height: 26 feet
Dam crest length: about 500 feet
Reservoir volume: 674 acre-feet released

History of Dam:

Purpose: irrigation
Dam completed: 1903
Dam failed: Thursday, July 15, 1982 at about 5:30 a.m.
Failure cause: Piping

Details on Detection of Failure/Deciding to warn:

The dam failure was observed by anyone able to take action until the leading edge of the flood had traveled about 4.5 miles downstream from Lawn 'Lake Dam. A trash collector heard loud noises and observed mud and debris on road. He used an emergency telephone which the National Park Service had at various locations within the park. The NPS and local government officials then began to warn and evacuate people located near the watercourse.

Details on dissemination of warnings and technologies used:

NPS Rangers and local police and sheriff used automobiles and went through area to warn. Local radio station was also broadcasting information on the flood and its movement.

Details on response to the warning:

Most people were taking the warnings seriously as the "Big Thompson" flood of 1976 which occurred nearby and killed about 140 was still in their memory. Three people died; 1 received no warning and the other 2 a weak warning not mentioning dam failure.

Description of flooding resulting from dam failure:

Flood covered an area about 13 miles long with first 7 miles in Rocky Mtn. N.P. Flood plain was generally narrow. Some buildings destroyed. Main street of Estes Park flooded.

The losses included:

3 fatalities. Damages totaled $31 million.

Prepared by Wayne Graham

Dam name: Timber Lake Dam

Location: near Lynchburg, Virginia

Dam Characteristics:

Dam type: earthfill
Dam height: 33 feet
Dam crest length: about 500 feet
Reservoir volume: 1449 acre-feet
Spillway: ungated

History of Dam:

Purpose: Real estate development
Dam completed: 1926
Dam failed: About 11 p.m., Thursday, June 22, 1995
Failure cause: Overtopping

Details on Detection of Failure/Deciding to warn:

Heavy rains in the 4.36 square mile drainage basin above dam prompted the maintenance director for the homeowners association to reach dam. Due to flooded roads he did not get to the dam before it failed.

Details on dissemination of warnings and technologies used:

There were no dam failure warnings issued for area downstream from the dam.

Details on response to the warning:

No dam failure warnings were issued. However, local volunteer firefighters were at a 4 lane divided highway about 1 mile downstream from Timber Lake Dam to search 3 cars that had stalled prior to the dam failure. The sudden surge of about 4 feet (at this location) caused by the dam failure caused the death of one firefighter.

Description of flooding resulting from dam failure:

Aside from flooded roads, very little damage occurred.

The losses included: 2 fatalities. The firefighter died in the search and rescue that started before dam failure and a woman died as she was driving on a road that crossed the dam failure floodplain. Aside from dam reconstruction, little economic damage.

Prepared by Wayne Graham

B-2

Will a Dam Failure Occur? Assessing Failure in a Risk-based Context

Bruce C. Muller, Jr.
U.S. Bureau of Reclamation
Dam Safety Office

Could we predict it today?

If we can't predict dam failures, what can we do?

- Recognize the risks associated with storing water
- Monitor those aspects of performance that would be indicative of a developing failure
- Take action to reduce risk where warranted

Reclamation's Risk Management Responsibility

Reclamation Safety of Dams Act of 1978:

To authorize the Secretary of the Interior to construct, restore, operate, and maintain new or modified features and existing Federal Reclamation dams for safety of dams purposes.

Reclamation's Risk Management Responsibility

Reclamation Safety of Dams Act of 1978 Sec. 2.

"In order to preserve the structural safety of Bureau of Reclamation dams and related facilities the Secretary of the Interior is authorized to perform such modifications as he determines to be **reasonably required**."

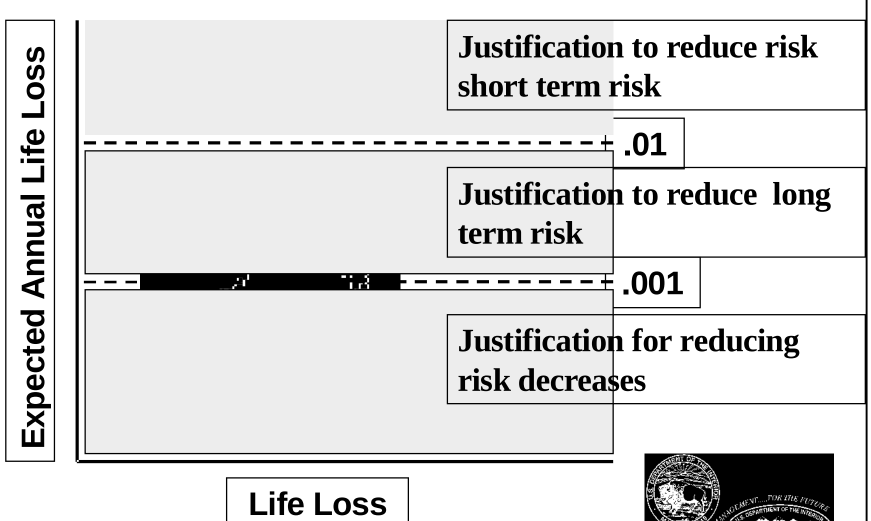
Reclamation's View of Risk

- Risk = p[load] x p[adverse response] x consequence
- **Loads**: static, hydrologic, seismic, operations
- **Adverse Response**: loss of storage, uncontrolled release, failure
- **Consequence**: **life loss**, economic damage, environmental damage

Risk Management Tools

- Facility reviews
- Performance monitoring
- Issue evaluation
  - Technical analysis
  - Risk analysis
- Risk reduction actions
- Public Protection Guidelines

Public Protection Guidelines



Roles of Dam Failure Analyses

- Identifying the extent of an adverse response to a loading condition
- Defining the outflow hydrograph
- Estimating the consequences of the outflow hydrograph

Important Issues

- Validation of technical models
- Addressing uncertainty
- Scalability
- Cost effectiveness
- Balancing unknowns

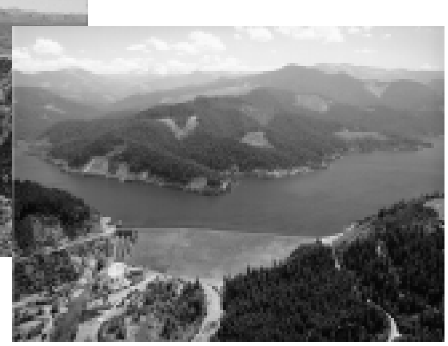
Challenges

- Focus research needs on areas that help decision makers reach decisions
- Strive for balance in development of tools (breach formation, routing, consequence assessment)
- Ensure that tools can be cost effectively applied to a wide variety of structures

B-3

Dam Safety Decisions: Current USACE Practice

Dr. David Moser
IWR



US Army Corps
of Engineers

Background

- Corps has approximately 570 dams
  - 64% over 30 years old
  - 28% over 50 years old
  - 65 categorized as hydrologically or seismically deficient, based on current criteria
- Cost to fix these deficiencies ranges between $ 1.3 and $ 6.5 billion

US Army Corps
of Engineers

Background

- Traditional Corps approach to risk and reliability issues
  - Standards
  - Criteria

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Standards

- Meet standard ➨ structure is safe against design event
- Make design event LARGE
- Simplifies Design Problem

US Army Corps
of Engineers

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Design Events

Flood

- Pre-1985, Probable Maximum Flood (PMF)
- Since 1985, Base Safety Condition (BSC)

Seismic

- Maximum Credible Earthquake (MCE)
- Operating Basis Earthquake (OBE)

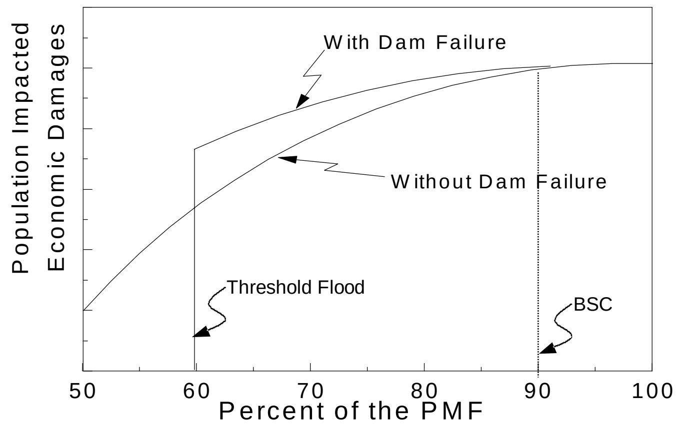
US Army Corps of Engineers

Base Safety Condition

- Design event at or above which dam failure does not increase downstream hazard

US Army Corps of Engineers

Establishing the Base Safety Condition



US Army Corps of Engineers

Why Change from PMF?

- Moving Target of PMF
- Unease with "Conservative" Assumptions
  - "Redundant Redundancies"
- Cost of Modifying Dams for Revised PMF

US Army Corps of Engineers

Why Not Change from MCE?

- MCE more likely than PMF
- PMF without failure already catastrophic
- Failure from seismic event apt to be "sunny day"
  - Little warning

US Army Corps
of Engineers

Why Not Risk Analysis for Dam Safety?

- Difficulty in quantifying likelihood of failure
- Focus on quantifying consequences
  - Engineering-economic system
- Criteria consistent with traditional engineering ethics
  - Provide safety so that dam does not impose added risk compared to natural state

US Army Corps
of Engineers

Current Dam Safety R&D

- Reasons for Corps renewed interest in risk analysis
  - Expanded use of risk analysis around world
  - Corps Major Rehabilitation Program
  - Corps policy not consistent with USBR

US Army Corps of Engineers

Major Rehabilitation Program

- Initiated in 1992
- Required risk analysis and adopted economic investment decision
  - Evaluation requirements for static risk (e.g. seepage) different than hydrologic and seismic

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Risk Analysis for Dam Safety

- R&D Program Initiated in 1999
- ObjectiveO

  Develop methodologies, frameworks and software tools necessary for the USACE to proactively manage the overall level of human and economic risk from our inventory of dams

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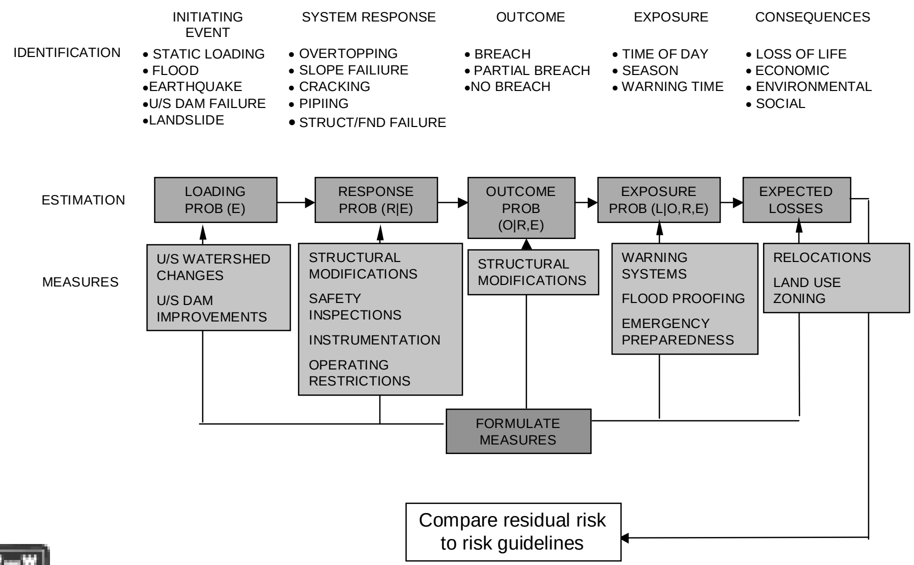
Risk Analysis

US Army Corps
of Engineers

FRAMEWORK FOR RISK ASSESSMENT



US Army Corps of Engineers

Risk Analysis R&D Focus

- Analysis for site-specific evaluation
- Analysis for dam inventory prioritization
- Technical risk assessment tools
- Risk management decision guidelines development
- Methods to field evaluation within USACE organizational structure

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Risk Analysis R&D Focus

- Analysis for site-specific evaluation
  - Technical procedures to quantify likelihoods and consequences
  - Development decision guidelines
  - Field demonstrations
    - Test procedures and expose field to approach

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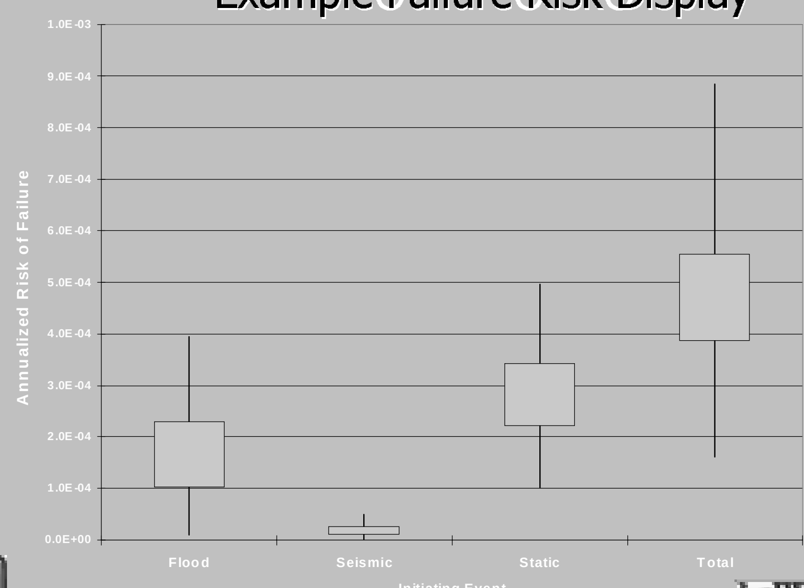
Example Failure Risk Display



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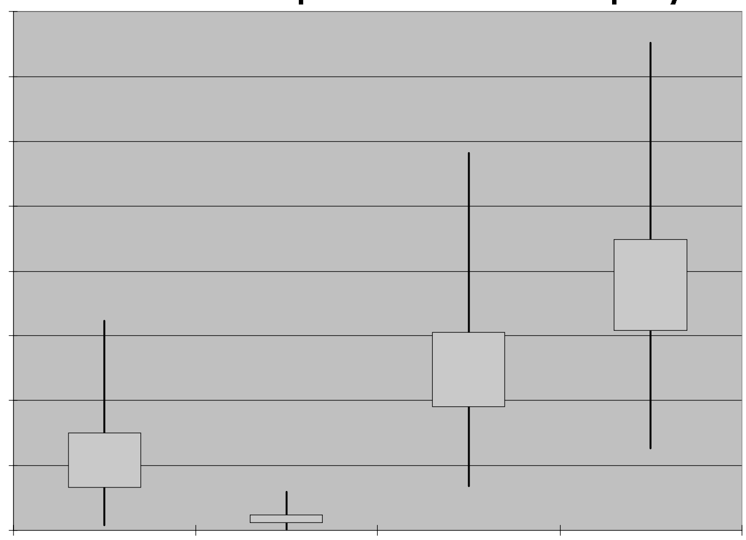
Example LOL Risk Display



Initiating Event

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Risk Analysis R&D Focus

- Analysis for dam inventory prioritization
  - Methodology for District, Division and Nationwide Portfolio (Inventory) Risk Analyses
  - Level of detail and data requirements consistent with mostly available
  - Integrate update into periodic inspections
  - Support additional investigations

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of Engineers

Risk Analysis R&D Focus

- Technical risk assessment tools
  - Probabilistic models for
    - Rate and extent of erosion in soil- and rock- lined spillways
    - Quantifying hydrologic loading uncertainty
    - Estimating extreme floods
    - Quantifying seepage & piping in embankment dams, levees, and soil foundations
    - Quantifying failure of gates and operating equipment
    - Quantifying failure mechanisms of concrete dams
    - Estimating uncertainties for breaching parameters of embankment dams.
    - Quantifying uplift uncertainties in rock foundations

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Risk Analysis R&D

- Lessons learned so far
  - The future use of risk analysis for dam safety evaluations seems to be accepted.
  - The study costs in the same ballpark as most major rehabilitation studies.
  - The analysis in the demonstration could have been improved with more time and money

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Risk Analysis R&D

❖ Methodological Problems
- Quantifying population at risk
- Subjective probability assessment used in estimating system response probabilities
- Need to use more refined models for quantifying earthquake and static risks and system responses
- Quantifying and using distributions for uncertainties

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Risk Analysis R&D

❖ Major R&D Needs:
- Improving methods for predicting the loss of life
- Special purpose software tools
- Improving the estimates in loadings, frequencies and uncertainties of large floods
- Quantifying earthquake system response uncertainties
- Quantifying static failure probabilities

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B-4

The Uncertainty of Embankment Dam Breach Parameter Predictions Based on Dam Failure Case Studies

by Tony L. Wahl[1]

Introduction

Risk assessment studies considering the failure of embankment dams often make use of breach parameter prediction methods that have been developed from analysis of historic dam failures. Similarly, predictions of peak breach outflow can also be made using relations developed from case study data. This paper presents an analysis of the uncertainty of many of these breach parameter and peak flow prediction methods, making use of a previously compiled database (Wahl 1998) of 108 dam failures. Subsets of this database were used to develop many of the relations examined.

The paper begins with a brief discussion of breach parameters and prediction methods. The uncertainty analysis of the various methods is next presented, and finally, a case study is offered to illustrate the application of several breach parameter prediction methods and the uncertainty analysis to a risk assessment recently performed by the Bureau of Reclamation for Jamestown Dam, on the James River in east-central North Dakota.

Breach Parameters

Dam break flood routing models (e.g., DAMBRK, FLDWAV) simulate the outflow from a reservoir and through the downstream valley resulting from a developing breach in a dam. These models focus their computational effort on the routing of the breach outflow hydrograph. The development of the breach is not simulated in any physical sense, but rather is idealized as a parametric process, defined by the shape of the breach, its final size, and the time required for its development (often called the failure time). Breaches in embankment dams are usually assumed to be trapezoidal, so the shape and size of the breach are defined by a base width and side slope angle, or more simply by an average breach width.

The failure time is a critical parameter affecting the outflow hydrograph and the consequences of dam failure, especially when populations at risk are located close to a dam so that available warning and evacuation time dramatically affects predictions of loss of life. For the purpose of routing a dam-break flood wave, breach development begins when a breach has reached the point at which the volume of the reservoir is compromised and failure becomes imminent. During the breach development phase, outflow from the dam increases rapidly. The breach development time ends when the breach reaches its final size; in some cases this may also correspond to the time of peak outflow through the breach, but for relatively small reservoirs the peak outflow may occur before the breach is fully developed. This breach development time as described above is the parameter predicted by most failure time prediction equations.

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The breach development time does not include the potentially long preceding period described as the breach initiation phase (Wahl 1998), which can also be important when considering available warning and evacuation time. This is the first phase of an overtopping failure, during which flow overtops a dam and may erode the downstream face, but does not create a breach through the dam that compromises the reservoir volume; if the overtopping flow were quickly stopped during the breach initiation phase, the reservoir would not fail. In an overtopping failure, the length of the breach initiation phase is important, because breach initiation can potentially be observed and may thus trigger warning and evacuation. Unfortunately, there are few tools available for predicting the length of the breach initiation phase.

During a seepage-erosion (piping) failure the delineation between breach initiation and breach development phases is less apparent. In some cases, seepage-erosion failures can take a great deal of time to develop. In contrast to the overtopping case, the loading that causes a seepage-erosion failure cannot normally be removed quickly, and the process does not take place in full view, except that the outflow from a developing pipe can be observed and measured. One useful way to view seepage-erosion failures is to consider three possible conditions:

(1) normal seepage outflow, with clear water and low flow rates;

(2) initiation of a seepage-erosion failure with cloudy seepage water that indicates a developing pipe, but flow rates are still low and not rapidly increasing. Corrective actions might still be possible that would heal the developing pipe and prevent failure.

(3) active development phase of a seepage-erosion failure in which erosion is dramatic and flow rates are rapidly increasing. Failure can no longer be prevented.

Only the length of the last phase is important when determining the breach hydrograph from a dam, but both the breach initiation and breach development phases may be important when considering warning and evacuation time. Again, as with the overtopping failure, there are few tools available for estimating the length of the breach initiation phase.

Predicting Breach Parameters

To carry out a dam break routing simulation, breach parameters must be estimated and provided as inputs to the dam-break and flood-routing simulation model. Several methods are available for estimating breach parameters; a summary of the available methods was provided by Wahl (1998). The simplest methods (Johnson and Illes 1976; Singh and Snorrason 1984; Reclamation 1988) predict the average breach width as a linear function of either the height of the dam or the depth of water stored behind the dam at the time of failure. Slightly more sophisticated methods predict more specific breach parameters, such as breach base width, side slope angles, and failure time, as functions of one or more dam and reservoir parameters, such as storage volume, depth of water at failure, depth of breach, etc. All of these methods are based on regression analyses of data collected from actual dam failures. The database of dam failures used to develop these relations is relatively lacking in data from failures of large dams, with about 75 percent of the cases having a height less than 15 meters, or 50 ft (Wahl 1998).

Physically-based simulation models are available to aid in the prediction of breach parameters. Although none are widely used, the most notable is the National Weather Service BREACH model (Fread 1988). These models simulate the hydraulic and erosion processes associated with flow over an overtopping dam or through a developing piping channel. Through such a simulation, an estimate of the breach parameters may be developed for use in a dam-break flood routing model, or the outflow hydrograph at the dam can be predicted directly. The primary weakness of the NWS-BREACH model and other similar models is the fact that they do not adequately model the headcut-type erosion processes that dominate the breaching of cohesive-soil embankments (e.g., Hahn et al. 2000). Recent work by the Agricultural Research Service (e.g., Temple and Moore 1994) on headcut erosion in earth spillways has shown that headcut erosion is best modeled with methods based on energy dissipation.

Predicting Peak Outflow

In addition to prediction of breach parameters, many investigators have proposed simplified methods for predicting peak outflow from a breached dam. These methods are valuable for reconnaissance-level work and for checking the reasonability of dam-break outflow hydrographs developed from estimated breach parameters. This paper considers the relations by:

- Kirkpatrick (1977)
- SCS (1981)
- Hagen (1982)
- Reclamation (1982)
- Singh and Snorrason (1984)
- MacDonald and Langridge-Monopolis (1984)
- Costa (1985)
- Evans (1986)
- Froehlich (1995a)
- Walder and O'Connor (1997)

All of these methods except Walder and O'Connor are straightforward regression relations that predict peak outflow as a function of various dam and/or reservoir parameters, with the relations developed from analyses of case study data from real dam failures. In contrast, Walder and O'Connor's method is based upon an analysis of numerical simulations of idealized cases spanning a range of dam and reservoir configurations and erosion scenarios. An important parameter in their method is an assumed vertical erosion rate of the breach; for reconnaissance-level estimating purposes they suggest that a range of reasonable values is 10 to 100 m/hr, based on analysis of case study data. The method makes a distinction between so-called large-reservoir/fast-erosion and small-reservoir/slow-erosion cases. In large-reservoir cases the peak outflow occurs when the breach reaches its maximum depth, before there has been any significant drawdown of the reservoir. The peak outflow in this case in insensitive to the erosion rate. In the small-reservoir case there is significant drawdown of the reservoir as the breach develops, and thus the peak outflow occurs before the breach erodes to its maximum depth. Peak outflows for small-reservoir cases are dependent on the vertical erosion rate and can be dramatically smaller than for large-reservoir cases. The determination of whether a specific situation is a large-reservoir or small-reservoir case is based on a dimensionless parameter incorporating the embankment erosion rate, reservoir size, and change in reservoir level during the failure. Thus, so-called large-reservoir/fast-erosion cases can occur even with what might be considered “small” reservoirs and vice versa. This refinement is not present in any of the other peak flow prediction methods.

Developing Uncertainty Estimates

In a typical risk assessment study, a variety of loading and failure scenarios are analyzed. This allows the study to incorporate variability in antecedent conditions and the probabilities associated with different loading conditions and failure scenarios. The uncertainty of key parameters (e.g., material properties) is sometimes considered by creating scenarios in which analyses are carried out with different parameter values and a probability of occurrence assigned to each value of the parameter. Although the uncertainty of breach parameter predictions is often very large, there have previously been no quantitative assessments of this uncertainty, and thus breach parameter uncertainty has not been incorporated into most risk assessment studies. In some studies, variations in thresholds of failure (e.g., overtopping depth to initiate breach) have been incorporated, usually through a voting process in which study team members and technical experts use engineering judgment to assign probabilities to different failure thresholds.

It is worthwhile to consider breach parameter prediction uncertainty in the risk assessment process because the uncertainty of breach parameter predictions is likely to be significantly greater than all other factors, and could thus dramatically influence the outcome. For example Wahl (1998) used many of the available relations to predict breach parameters for 108 documented case studies and plot the predictions against the observed values. Prediction errors of ±75% were not uncommon for breach width, and prediction errors for failure time often exceeded 1 order of magnitude. Most relations used to predict failure time are conservatively designed to underpredict the reported time more often than they overpredict, but overprediction errors of more than one-half order of magnitude did occur several times.

The first question that must be addressed in an uncertainty analysis of breach parameter predictions is how to express the results. The case study datasets used to develop most breach parameter prediction equations include data from a wide range of dam sizes, and thus, regressions in log-log space have been commonly used. Figure 1 shows the observed and predicted breach widths as computed by Wahl (1998) in both arithmetically-scaled and log-log plots. In the arithmetic plots, it would be difficult to draw in upper and lower bound lines to define an uncertainty band. In the log-log plots data are scattered approximately evenly above and below the lines of perfect prediction, suggesting that uncertainties would best be expressed as a number of log cycles on either side of the predicted value. This is the approach taken in the analysis that follows.

The other notable feature of the plots in Figure 1 is the presence of a few significant outliers. The source of these outliers is believed to be the variable quality of the case study observations, the potential for misapplication of some of the prediction equations due to lack of detailed knowledge of each case study, and inherent variability in the data due to the variety of factors that influence dam breach mechanics. Thus, before determining uncertainties, an outlier-exclusion algorithm was applied (Rousseeuw 1998). The algorithm has the advantage that it is, itself, insensitive to the effects of outliers.

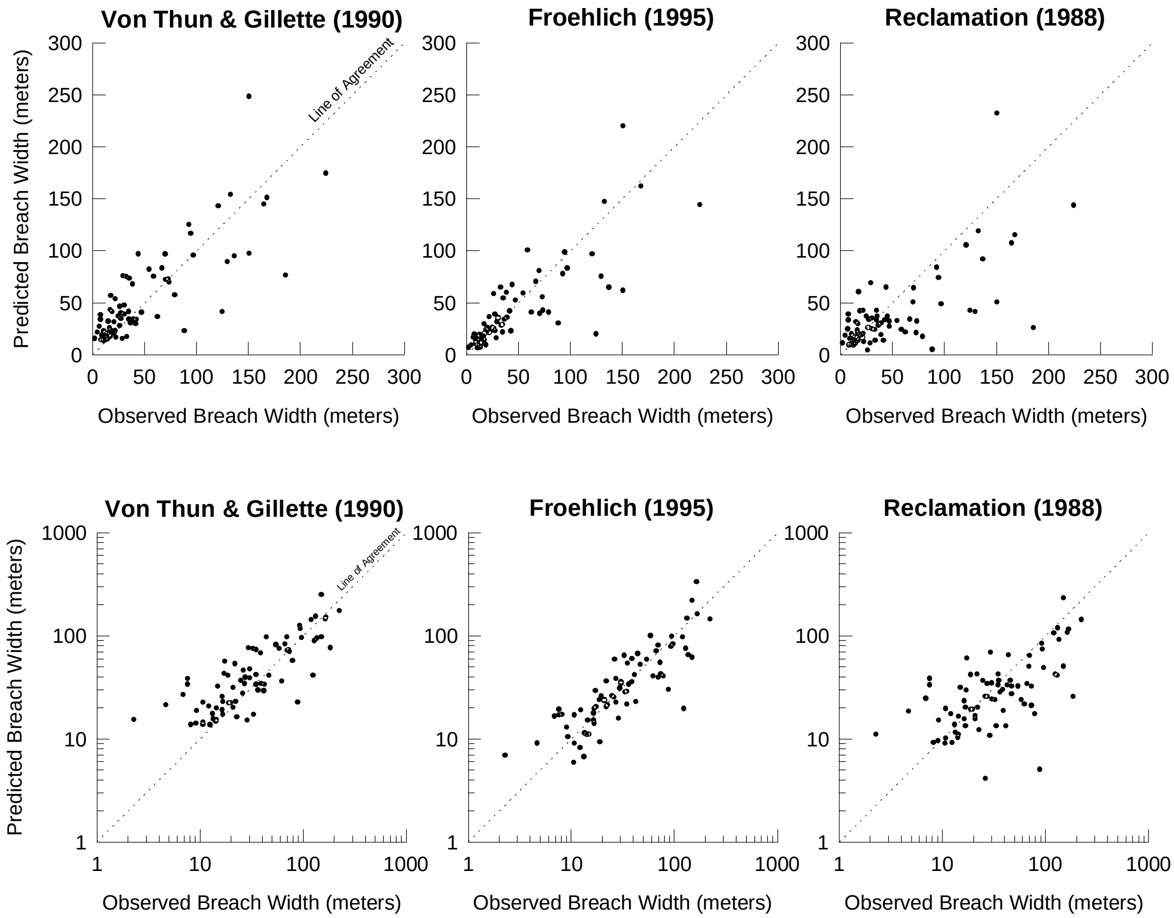

Figure 1. — Predicted and observed breach widths (Wahl 1998), plotted arithmetically (top) and on log-log scales (bottom).

The uncertainty analysis was performed using the database presented in Wahl (1998), with data on 108 case studies of actual embankment dam failures, collected from numerous sources in the literature. The majority of the available breach parameter and peak flow prediction equations were applied to this database of dam failures, and the predicted values were compared to the observed values. Computation of breach parameters or peak flows was straightforward in most cases. A notable exception was the peak flow prediction method of Walder and O'Connor (1997), which requires that the reservoir be classified as a large- or small-reservoir case. In addition, in the case of the small-reservoir situation, an average vertical erosion rate of the breach must be estimated. The Walder and O'Connor method was applied only to those dams that could be clearly identified as large-reservoir (in which case peak outflow is insensitive to the vertical erosion rate) or small-reservoir with an associated estimate of the vertical erosion rate obtained from observed breach heights and failure times. Two other facts should be noted:

- No prediction equation could be applied to all 108 dam failure cases, due to lack of required input data for the specific equation or the lack of an observed value of the parameter of interest. Most of the breach width equations could be tested against about 70 to 80 cases, the failure time equations were tested against 30 to 40 cases, and the peak flow prediction equations were generally tested against about 30 to 40 cases.

- The testing made use of the same data used to originally develop the equations, but each equation was also tested against additional cases. This should provide a fair indication of the ability of each equation to predict breach parameters for future dam failures.

A step-by-step description of the uncertainty analysis method follows:

(1) Plot predicted vs. observed values on log-log scales.

(2) Compute individual prediction errors in terms of the number of log cycles separating the predicted and observed value, $e_i = \log(\hat{x}) - \log(x) = \log(\hat{x}/x)$, where e_i is the prediction error, $\hat{x}$ is the predicted value and x is the observed value.

(3) Apply the outlier-exclusion algorithm to the series of prediction errors computed in step (2). The algorithm is described by Rousseeuw (1998).

    (a) Determine T, the median of the e_i values. T is the estimator of location.

    (b) Compute the absolute values of the deviations from the median, and determine the median of these absolute deviations (MAD).

    (c) Compute an estimator of scale, S=1.483*(MAD). The 1.483 factor makes S comparable to the standard deviation, which is the usual scale parameter of a normal distribution.

    (d) Use S and T to compute a Z-score for each observation, $Z_i=(e_i-T)/S$, where the e_i's are the observed prediction errors, expressed as a number of log cycles.

    (e) Reject any observations for which $|Z_i|>2.5$

    This method rejects at the 98.7% probability level if the samples are from a perfect normal distribution.

(4) Compute the mean, $\bar{e}$, and the standard deviation, S_e, of the remaining prediction errors. If the mean value is negative, it indicates that the prediction equation underestimated the observed values, and if positive the equation overestimated the observed values. Significant over or underestimation should be expected, since many of the breach parameter prediction equations are intended to be conservative or provide envelope estimates, e.g., maximum reasonable breach width, fastest possible failure time, etc.

(5) Using the values of $\bar{e}$ and S_e, one can express a confidence band around the predicted value of a parameter as $\{\hat{x}\cdot 10^{-\bar{e}-2S_e}, \hat{x}\cdot 10^{-\bar{e}+2S_e}\}$, where $\hat{x}$ is the predicted value. The use of $\pm 2S_e$ gives approximately a 95 percent confidence band.

Table 1 summarizes the results. The first column identifies the particular method being analyzed, the next two columns show the number of case studies used to test the method, and the next two columns give the prediction error and the width of the uncertainty band. The rightmost column shows the range of the prediction interval around a hypothetical predicted value of 1.0. The values in this column can be used as multipliers to obtain the prediction interval for a specific case.

Table 1. – Uncertainty estimates of breach parameter and peak flow prediction equations. All equations use metric units (meters, m^3, m^3/s). Failure times are computed in hours.

Equation	Number of Case Studies: Before outlier exclusion	Number of Case Studies: After outlier exclusion	Mean Prediction Error, $\bar{e}$ (log cycles)	Width of Uncertainty Band, $\pm 2S_e$ (log cycles)	Prediction interval around a hypothetical predicted value of 1.0
BREACH WIDTH EQUATIONS					
USBR (1988) $\bar{B} = 3(h_w)$	80	70	-0.09	±0.43	0.45 — 3.3
MacDonald and Langridge-Monopolis (1984) $V_{er} = 0.0261(V_w \cdot h_w)^{0.769}$ *earthfill* $V_{er} = 0.00348(V_w \cdot h_w)^{0.852}$ *non-earthfill (e.g., rockfill)*	60	58	-0.01	±0.82	0.15 — 6.8
Von Thun and Gillette (1990) $\bar{B} = 2.5h_w + C_b$ *where C_b is a function of reservoir size*	78	70	+0.09	±0.35	0.37 — 1.8
Froehlich (1995b) $\bar{B} = 0.1803 K_o V_w^{0.32} h_b^{0.19}$ *where K_o = 1.4 for overtopping, 1.0 for piping*	77	75	+0.01	±0.39	0.40 — 2.4
FAILURE TIME EQUATIONS					
MacDonald and Langridge-Monopolis (1984) $t_f = 0.0179(V_{er})^{0.364}$	37	35	-0.21	±0.83	0.24 — 11.
Von Thun and Gillette (1990) $t_f = 0.015(h_w)$ *highly erodible* $t_f = 0.020(h_w) + 0.25$ *erosion resistant*	36	34	-0.64	±0.95	0.49 — 40.
Von Thun and Gillette (1990) $t_f = \bar{B}/(4h_w)$ *highly erodible* $t_f = \bar{B}/(4h_w + 61)$ *erosion resistant*	36	35	-0.38	±0.84	0.35 — 17.
Froehlich (1995b) $t_f = 0.00254(V_w)^{0.53} h_b^{-0.9}$	34	33	-0.22	±0.64	0.38 — 7.3
USBR (1988) $t_f = 0.011(\bar{B})$	40	39	-0.40	±1.02	0.24 — 27.
PEAK FLOW EQUATIONS					
Kirkpatrick (1977) $Q_p = 1.268(h_w + 0.3)^{2.5}$	38	34	-0.14	±0.69	0.28 — 6.8
SCS (1981) $Q_p = 16.6(h_w)^{1.85}$	38	32	+0.13	±0.50	0.23 — 2.4
Hagen (1982) $Q_p = 0.54(S \cdot h_d)^{0.5}$	31	30	+0.43	±0.75	0.07 — 2.1
Reclamation (1982) $Q_p = 19.1(h_w)^{1.85}$ *envelope equation*	38	32	+0.19	±0.50	0.20 — 2.1

Equation	Number of Case Studies: Before outlier exclusion	Number of Case Studies: After outlier exclusion	Mean Prediction Error, $\overline{e}$ (log cycles)	Width of Uncertainty Band, $\pm 2S_e$ (log cycles)	Prediction interval around a hypothetical predicted value of 1.0
PEAK FLOW EQUATIONS *(continued)*					
Singh and Snorrason (1984)					
$Q_p = 13.4(h_d)^{1.89}$	38	28	+0.19	±0.46	0.23 — 1.9
$Q_p = 1.776(S)^{0.47}$	35	34	+0.17	±0.90	0.08 — 5.4
MacDonald and Langridge-Monopolis (1984)					
$Q_p = 1.154(V_w \cdot h_w)^{0.412}$	37	36	+0.13	±0.70	0.15 — 3.7
$Q_p = 3.85(V_w \cdot h_w)^{0.411}$ *envelope equation*	37	36	+0.64	±0.70	0.05 — 1.1
Costa (1985)					
$Q_p = 1.122(S)^{0.57}$ *envelope equation*	35	35	+0.69	±1.02	0.02 — 2.1
$Q_p = 0.981(S \cdot h_d)^{0.42}$	31	30	+0.05	±0.72	0.17 — 4.7
$Q_p = 2.634(S \cdot h_d)^{0.44}$ *envelope equation*	31	30	+0.64	±0.72	0.04 — 1.22
Evans (1986)					
$Q_p = 0.72(V_w)^{0.53}$	39	39	+0.29	±0.93	0.06 — 4.4
Froehlich (1995a)					
$Q_p = 0.607(V_w^{0.295} h_w^{1.24})$	32	31	-0.04	±0.32	0.53 — 2.3
Walder and O'Connor (1997)					
Q_p estimated using method based on relative erodibility of dam and size of reservoir	22	21	+0.13	±0.68	0.16 — 3.6

Notes: Where multiple equations are shown for application to different types of dams (e.g., highly erodible vs. erosion resistant), a single prediction uncertainty was analyzed, with the *system* of equations viewed as a single algorithm. The only exception is the pair of peak flow prediction equations offered by Singh and Snorrason (1984), which are alternative and independent methods for predicting peak outflow.

Definitions of Symbols for Equations Shown in Column 1.

$\overline{B}$ = average breach width, meters

C_b = offset factor in the Von Thun and Gillette breach width equation, varies from 6.1 m to 54.9 m as a function of reservoir storage

h_b = height of breach, m

h_d = height of dam, m

h_w = depth of water above breach invert at time of failure, meters

K_o = overtopping multiplier for Froehlich breach width equation, 1.4 for overtopping, 1.0 for piping

Q_p = peak breach outflow, m^3/s

S = reservoir storage, m^3

t_f = failure time, hours

V_{er} = volume of embankment material eroded, m^3

V_w = volume of water stored above breach invert at time of failure, m^3

Summary of Uncertainty Analysis Results

The four methods for predicting breach width all had absolute mean prediction errors less than one-tenth of an order of magnitude, indicating that on average their predictions are on-target. The uncertainty bands were similar (±0.3 to ±0.4 log cycles) for all of the equations except the MacDonald and Langridge-Monopolis equation, which had an uncertainty of ±0.82 log cycles.

The five methods for predicting failure time all underpredict the failure time on average, by amounts ranging from about one-fifth to two-thirds of an order of magnitude. This is consistent with the previous observation that these equations are designed to conservatively predict fast breaches, which will cause large peak outflows. The uncertainty bands on all of the failure time equations are very large, ranging from about ±0.6 to ±1 order of magnitude, with the Froehlich (1995b) equation having the smallest uncertainty.

Most of the peak flow prediction equations tend to overpredict observed peak flows, with most of the "envelope" equations overpredicting by about two-thirds to three-quarters of an order of magnitude. The uncertainty bands on the peak flow prediction equations are about ±0.5 to ±1 order of magnitude, except the Froehlich (1995a) relation which has an uncertainty of ±0.32 orders of magnitude. In fact, the Froehlich equation has both the best prediction error and uncertainty of all the peak flow prediction equations.

Application to Jamestown Dam

To illustrate the application of the uncertainty analysis results, a case study is presented. In January 2001 the Bureau of Reclamation conducted a risk assessment study for Jamestown Dam (Figure 2), a feature of the Pick-Sloan Missouri Basin Program, located on the James River immediately upstream from Jamestown, North Dakota. For this risk assessment, two potential static failure modes were considered:

- Seepage erosion and piping of foundation materials
- Seepage erosion and piping of embankment materials

No distinction between these two failure modes was made in the breach parameter analysis, since most methods used to predict breach parameters lack the refinement needed to consider the differences in breach morphology for these two failure modes.

Figure 2. — Jamestown Dam and reservoir.

The potential for failure and the downstream consequences from failure increase significantly at higher reservoir levels, although the likelihood of occurrence of high reservoir levels is low. The reservoir rarely exceeds its top-of-joint-use elevation, and has never exceeded elevation 1445.9 ft. Four potential reservoir water surface elevations at failure were considered in the study:

- Top of joint use, elev. 1432.67 ft, reservoir capacity of about 37,000 ac-ft
- Elev. 1440.0 ft, reservoir capacity of about 85,000 ac-ft
- Top of flood space, elev. 1454 ft, reservoir capacity of about 221,000 ac-ft
- Maximum design water surface, elev. 1464.3 ft, storage of about 380,000 ac-ft

Breach parameters were predicted using most of the methods discussed earlier in this paper, and also by modeling with the National Weather Service BREACH model (NWS-BREACH).

Dam Description

Jamestown Dam is located on the James River about 1.5 miles upstream from the city of Jamestown, North Dakota. It was constructed by the Bureau of Reclamation from 1952 to 1954. The facilities are operated by Reclamation to provide flood control, municipal water supply, fish and wildlife benefits and recreation.

The dam is a zoned-earthfill structure with a structural height of 111 ft and a height of 81 ft above the original streambed. The crest length is 1,418 ft at elevation 1471 ft and the crest width is 30 ft. The design includes a central compacted zone 1 impervious material, and upstream and downstream zone 2 of sand and gravel, shown in Figure 3. The upstream slope is protected with riprap and bedding above elevation 1430 ft. A toe drain consisting of sewer pipe laid with open joints is located in the downstream zone 2 along most of the embankment.

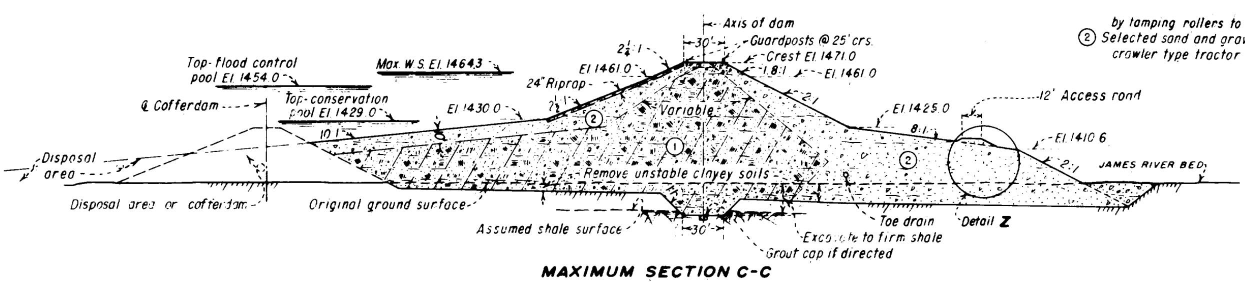

Figure 3. — Cross-section through Jamestown Dam.

The abutments are composed of Pierre Shale capped with glacial till. The main portion of the dam is founded on a thick section of alluvial deposits. The spillway and outlet works are founded on Pierre Shale. Beneath the dam a cutoff trench was excavated to the shale on both abutments, however, between the abutments, foundation excavation extended to a maximum depth of 25 ft, and did not provide a positive cutoff of the thick alluvium. The alluvium beneath the dam is more than 120 ft thick in the channel area.

There is a toe drain within the downstream embankment near the foundation level, and a fairly wide embankment section to help control seepage beneath the dam, since a positive cutoff was not constructed. The original design recognized that additional work might be required to control seepage and uplift pressures, depending on performance of the dam during first filling. In general, performance of the dam has been adequate, but, reservoir water surface elevations have never exceeded 1445.9 ft, well below the spillway crest. Based on observations of increasing pressures in the foundation during high reservoir elevations and significant boil activity downstream from the dam, eight relief wells were installed along the downstream toe in 1995 and 1996. To increase the seepage protection, a filter blanket was constructed in low areas downstream from the dam in 1998.

Results — Breach Parameter Estimates

Breach parameter predictions were computed for the four reservoir conditions listed previously: top of joint use; elev. 1440.0; top of flood space; and maximum design water surface elevation. Predictions were made for average breach width, volume of eroded material, and failure time. Side slope angles were not predicted because equations for predicting breach side slope angles are rare in the literature; Froehlich (1987) offered an equation, but in his later paper (1995b) he suggested simply assuming side slopes of 0.9:1 (horizontal:vertical) for piping failures. Von Thun and Gillette (1990) suggested using side slopes of 1:1, except for cases of dams with very thick zones of cohesive materials where side slopes of 0.5:1 or 0.33:1 might be more appropriate.

After computing breach parameters using the several available equations, the results were reviewed and engineering judgment applied to develop a single predicted value and an uncertainty band to be provided to the risk assessment study team. These recommended values are shown at the bottom of each column in the tables that follow.

Breach Width

Predictions of average breach width are summarized in Table 2. The table also lists the predictions of the volume of eroded embankment material made using the MacDonald and Langridge-Monopolis equation, and the corresponding estimate of average breach width.

Table 2. — Predictions of average breach width for Jamestown Dam.

BREACH WIDTHS ***B*, feet**	Top of joint use (elev. 1432.67 ft)		Elev. 1440.0 ft		Top of flood space (elev. 1454.0 ft)		Maximum design water surface (elev. 1464.3 ft)	
	Prediction	95% Prediction Interval	Prediction	95% Prediction Interval	Prediction	95% Prediction Interval	Prediction	95% Prediction Interval
Reclamation, 1988	128	58 — 422	150	68 — 495	192	86 — 634	223	100 — 736
Von Thun and Gillette, 1990	287	106 — 516	305	113 — 549	340	126 — 612	366	135 — 659
Froehlich, 1995b	307	123 — 737	401	160 — 962	544	218 — 1307	648	259 — 1554^
MacDonald and Langridge-Monopolis, 1984 (Volume of erosion, yd^3)	*191,000*	*29,000 — 1,296,000*	*408,000*	*61,000 — 2,775,000*	*1,029,000*	*154,000 — 6,995,000*	*1,751,000*	*263,000 — 11,904,000*
(Equivalent breach width, ft)	281	42 — 1,908^	601	90 — 4,090^	1,515^	227 — 10,300^	2,578^	387 — 17,528^
Recommended values	290	110 — 600	400	150 — 1000	540	200 — 1300	650	250 — 1418

* Recommend breach side slopes for all scenarios are 0.9 horizontal to 1.0 vertical.
^ Exceeds actual embankment length.

The uncertainty analysis described earlier showed that the Reclamation equation tends to underestimate the observed breach width, so it is not surprising that it yielded the smallest values. The Von Thun and Gillette equation and the Froehlich equation produced comparable results for the top-of-joint-use scenario, in which reservoir storage is relatively small. For the two scenarios with greater reservoir storage, the Froehlich equation predicts significantly larger breach widths. This is not surprising, since the Froehlich equation relates breach width to an exponential function of both the reservoir storage and reservoir depth. The Von Thun and Gillette equation accounts for reservoir storage only through the C_b offset parameter, but C_b is a constant for all reservoirs larger than 10,000 ac-ft, as was the case for all four of these scenarios.

Using the MacDonald and Langridge-Monopolis equation, the estimate of eroded embankment volume and associated breach width for the top-of-joint-use scenario is also comparable to the other equations. However, for the two large-volume scenarios, the predictions are much larger than any of the other equations, and in fact are unreasonable because they exceed the dimensions of the dam (1,418 ft long; volume of 763,000 yd^3).

The prediction intervals developed through the uncertainty analysis are sobering, as the ranges vary from small notches through the dam to complete washout of the embankment. Even for the top-of-joint-use case, the upper bound for the Froehlich and Von Thun/Gillette equations is equivalent to about half the length of the embankment.

Failure Time

Failure time predictions are summarized in Table 3. All of the equations indicate increasing failure times as the reservoir storage increases, except the second Von Thun and Gillette relation, which predicts a slight decrease in failure time for the large-storage scenarios. For both Von Thun and Gillette relations, the dam was assumed to be in the erosion resistant category.

Table 3. — Failure time predictions for Jamestown Dam.

FAILURE TIMES ***t_f* hours**	Top of joint use (elev. 1432.67 ft)		Elev. 1440.0 ft		Top of flood space (elev. 1454.0 ft)		Maximum design water surface (elev. 1464.3 ft)	
	Prediction	95% Prediction Interval	Prediction	95% Prediction Interval	Prediction	95% Prediction Interval	Prediction	95% Prediction Interval
MacDonald and Langridge-Monopolis, 1984	1.36	0.33 — 14.9	1.79	0.43 — 19.7	2.45*	0.59 — 26.9	2.45*	0.59 — 26.9
Von Thun and Gillette, 1990 $t_f = f(h_w)$ …erosion resistant	0.51	0.25 — 20.4	0.55	0.27 — 22.2	0.64	0.31 — 25.6	0.70	0.34 — 28.1
Von Thun and Gillette, 1990 $t_f = f(B, h_w)$ …erosion resistant	1.68	0.59 — 28.6	1.53	0.53 — 25.9	1.33	0.47 — 22.6	1.23	0.43 — 20.9
Froehlich, 1995b	1.63	0.62 — 11.9	2.53	0.96 — 18.4	4.19	1.59 — 30.6	5.59	2.12 — 40.8
Reclamation, 1988	0.43	0.10 — 11.6	0.50	0.12 — 13.6	0.64	0.15 — 17.4	0.75	0.18 — 20.2
Recommended values	1.5	0.25 — 12	1.75	0.25 — 14	3.0	0.3 — 17	4.0	0.33 — 20

* The MacDonald and Langridge-Monopolis equation is based on the prediction of eroded volume, shown previously in Table 2. Because the predicted volumes exceeded the total embankment volume in the two large-storage scenarios, the total embankment volume was used in the failure time equation. Thus, the results are identical to the top-of-joint-use case.

The predicted failure times exhibit wide variation, and the recommended values shown at the bottom of the table are based on much judgment. The uncertainty analysis showed that all of the failure time equations tend to conservatively underestimate actual failure times, especially the Von Thun and Gillette and Reclamation equations. Thus, the recommended values are generally a compromise between the results obtained from the MacDonald and Langridge-Monopolis and Froehlich relations. Despite this fact, some very fast failures are documented in the literature, and this possibility is reflected in the prediction intervals determined from the uncertainty analysis.

Results — Peak Outflow Estimates

Peak outflow estimates are shown in Table 4, sorted in order of increasing peak outflow for the top-of-joint-use scenario. The lowest peak flow predictions come from those equations that are based solely on dam height or depth of water in the reservoir. The highest peak flows are predicted by those equations that incorporate a significant dependence on reservoir storage. Some of the predicted peak flows and the upper bounds of the prediction limits would be the largest dam-break outflows ever recorded, exceeding the 2.3 million ft^3/s peak outflow from the Teton Dam failure. (Storage in Teton Dam was 289,000 ac-ft at failure). The length of Jamestown Reservoir (about 30 miles) may help to attenuate some of the large peak outflows predicted by the storage-sensitive equations, since there will be an appreciable routing effect in the reservoir itself that is probably not accounted for in the peak flow prediction equations.

Table 4. — Predictions of peak breach outflow for Jamestown Dam.

PEAK OUTFLOWS **Q_p, ft^3/s**	Top of joint use (elev. 1432.67 ft)		Elev. 1440.0 ft		Top of flood space (elev. 1454.0 ft)		Maximum design water surface (elev. 1464.3 ft)	
	Prediction	95% Prediction Interval	Prediction	95% Prediction Interval	Prediction	95% Prediction Interval	Prediction	95% Prediction Interval
Kirkpatrick, 1977	28,900	8,100 — 196,600	42,600	11,900 — 289,900	78,200	21,900 — 531,700	112,900	31,600 — 768,000
SCS, 1981	67,500	15,500 — 162,000	90,500	20,800 — 217,200	142,900	32,900 — 342,900	188,300	43,300 — 451,900
Reclamation, 1982, envelope	77,700	15,500 — 163,100	104,100	20,800 — 218,600	164,400	32,900 — 345,200	216,600	43,300 — 455,000
Froehlich, 1995a	**93,800**	**49,700 — 215,700**	**145,900**	**77,300 — 335,600**	**262,700**	**139,200 — 604,200**	**370,900**	**196,600 — 853,100**
MacDonald and Langridge-Monopolis, 1984	167,800	25,200 — 620,900	252,400	37,900 — 933,700	414,100	62,100 — 1,532,000	550,600	82,600 — 2,037,000
Singh/Snorrason, 1984 $Q_p = f(h_d)$	202,700	46,600 — 385,200	202,700	46,600 — 385,200	202,700	46,600 — 385,200	202,700	46,600 — 385,200
Walder and O'Connor, 1997	**211,700**	**33,900 — 755,600**	**279,300**	**44,700 — 997,200**	**430,200**	**68,800 — 1,536,000**	**558,600**	**89,400 — 1,994,000**
Costa, 1985 $Q_p = f(S{*}h_d)$	219,500	37,300 — 1,032,000	311,200	52,900 — 1,463,000	464,900	79,000 — 2,185,000	583,800	99,200 — 2,744,000
Singh/Snorrason, 1984 $Q_p = f(S)$	249,600	20,000 — 1,348,000	369,000	29,500 — 1,993,000	578,200	46,300 — 3,122,000	746,000	59,700 — 4,028,000
Evans, 1986	291,600	17,500 — 1,283,000	453,100	27,200 — 1,994,000	751,800	45,100 — 3,308,000	1,002,000	60,100 — 4,409,000
MacDonald and Langridge-Monopolis, 1984 (envelope equation)	548,700	27,400 — 603,500	824,300	41,200 — 906,700	1,351,000	67,600 — 1,486,000	1,795,000	89,800 — 1,975,000
Hagen, 1982	640,100	44,800 — 1,344,000	970,000	67,900 — 2,038,000	1,564,000	109,500 — 3,285,000	2,051,000	143,600 — 4,308,000
Costa, 1985 $Q_p = f(S{*}h_d)$ (envelope)	894,100	35,800 — 1,091,000	1,289,000	51,600 — 1,573,000	1,963,000	78,500 — 2,395,000	2,492,000	99,700 — 3,040,000
Costa, 1985 $Q_p = f(S)$	920,000	18,400 — 1,932,000	1,478,000	29,600 — 3,104,000	2,548,000	51,000 — 5,351,000	3,470,000	69,400 — 7,288,000

The equation offered by Froehlich (1995a) clearly had the best prediction performance in the uncertainty analysis, and is thus highlighted in the table. This equation had the smallest mean prediction error and narrowest prediction interval by a significant margin.

The results for the Walder and O'Connor method are also highlighted. As discussed earlier, this is the only method that considers the differences between the so-called large-reservoir/fast-erosion and small-reservoir/slow-erosion cases. Jamestown Dam proves to be a large-reservoir/fast-erosion case when analyzed by this method (regardless of the assumed vertical erosion rate of the breach—within reasonable limits), so the peak outflow will occur when the breach reaches its maximum size, before significant drawdown of the reservoir has occurred. Despite the refinement of considering large- vs. small-reservoir behavior, the Walder and O'Connor method was found to have uncertainty similar to most of the other peak flow prediction methods (about ±0.75 log cycles). However, amongst the 22 case studies that the method could be applied to, only four proved to be large-reservoir/fast-erosion cases. Of these, the method overpredicted the peak outflow in three cases, and dramatically underpredicted in one case (Goose Creek Dam, South Carolina, failed 1916 by overtopping). Closer examination showed some contradictions in the data reported in the literature for this case. On balance, it appears that the Walder and O'Connor method may provide reasonable estimates of the upper limit on peak outflow for large-reservoir/fast-erosion cases.

For the Jamestown Dam case, results from the Froehlich method can be considered the best estimate of peak breach outflow, and the results from the Walder and O'Connor method provide an upper bound estimate.

NWS-BREACH Simulations

Several simulations runs were made using the National Weather Service BREACH model (Fread 1988). The model requires input data related to reservoir bathymetry, dam geometry, the tailwater channel, embankment materials, and initial conditions for the simulated piping failure. Detailed information on embankment material properties was not available at the time that the simulations were run, so material properties were assumed to be similar to those of Teton Dam. A Teton Dam input data file is distributed with the model.

The results of the simulations are very sensitive to the elevation at which the piping failure is assumed to develop. In all cases analyzed, the maximum outflow occurred just prior to the crest of the dam collapsing into the pipe; after the collapse of the crest, a large volume of material partially blocks the pipe and the outflow becomes weir-controlled until the material can be removed. Thus, the largest peak outflows and largest breach sizes are obtained if the failure is initiated at the base of the dam, assumed to be elev. 1390.0 ft. This produces the maximum amount of head on the developing pipe, and allows it to grow to the largest possible size before the collapse occurs. Table 5 shows summary results of the simulations. For each of the four initial reservoir elevations a simulation was run with the pipe initiating at elev. 1390.0 ft, and a second simulation was run with the pipe initiating about midway up the height of the dam.

Table 5. — Results of NWS-BREACH simulations of seepage-erosion failures of Jamestown Dam.

	Top of joint use (elev. 1432.67 ft)		Elev. 1440.0 ft		Top of flood space (elev. 1454.0 ft)		Maximum design water surface (elev. 1464.3 ft)	
Initial elev. of piping failure, ft →	1390.0	1411.0	1390.0	1415.0	1390.0	1420.0	1390.0	1430.0
Peak outflow, ft^3/s	80,400	16,400	131,800	24,050	242,100	52,400	284,200	54,100
t_p, Time-to-peak outflow, hrs (from first significant increased flow through the breach)	3.9	2.1	4.0	1.8	4.0	1.4	3.6	1.1
Breach width at t_p, ft	51.6	21.4	63.2	24.8	81.0	33.7	81.0	34.2

There is obviously wide variation in the results depending on the assumed initial conditions for the elevation of the seepage failure. The peak outflows and breach widths tend toward the low end of the range of predictions made using the regression equations based on case study data. The predicted failure times are within the range of the previous predictions, and significantly longer than the very short (0.5 to 0.75 hr) failure times predicted by the Reclamation (1988) equation and the first Von Thun and Gillette equation.

Refinement of the material properties and other input data provided to the NWS-BREACH model might significantly change these results.

Conclusions

This paper has presented a quantitative analysis of the uncertainty of various regression-based methods for predicting embankment dam breach parameters and peak breach outflows. The uncertainties of predictions of breach width, failure time, and peak outflow are large for all methods, and thus it may be worthwhile to incorporate uncertainty analysis results into future risk assessment studies when predicting breach parameters using these methods. Predictions of breach width generally have an uncertainty of about ±1/3 order of magnitude, predictions of failure time have uncertainties approaching ±1 order of magnitude, and predictions of peak flow have uncertainties of about ±0.5 to ±1 order of magnitude, except the Froehlich peak flow equation, which has an uncertainty of about ±1/3 order of magnitude.

The uncertainty analysis made use of a database of information on the failure of 108 dams compiled from numerous sources in the literature (Wahl 1998). For those wishing to make use of this database, it is available in electronic form (Lotus 1-2-3, Microsoft Excel, and Microsoft Access) on the Internet at http://www.usbr.gov/wrrl/twahl/damfailuredatabase.zip.

The case study presented for Jamestown Dam showed that significant engineering judgment must be exercised in the interpretation of predictions obtained from the regression-based methods. The results from use of the physically-based NWS-BREACH model were reassuring because they fell within the range of values obtained from the regression-based methods, but at the same time they also helped to show that even physically-based methods can be highly sensitive to the analysts assumptions regarding breach morphology and the location of initial breach development. The NWS-BREACH simulations revealed the possibility for limiting failure mechanics that were not considered in the regression-based methods.

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SOME EXISTING CAPABILITIES AND FUTURE DIRECTIONS FOR DAM-BREACH MODELING/FLOOD ROUTING

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Abstract: Dam-breach modeling and the routing of the unsteady breach outflow through the downstream river/valley are important tasks for many Federal, state, local agencies, consultants etc., which are charged with or assist those so charged with dam design, operation, regulation, and/or public safety. A brief historical summary is provided which covers some of the relevant procedures for prediction of dam-breach outflows and their extent of flooding in the downstream river/valley.

Dam-breach modeling can be conveniently categorized as parametric-based or physically-based. The former utilizes key parameters: average breach width (b_{av}) and breach formation time (t_f) to represent the breach formation in earthen dams, and thus compute the breach outflow hydrograph using a numerical time-step solution procedure or a single analytical equation. Statistics on observed values for b_{av} and t_f have been presented. Also, various regression equations have been developed to compute peak-breach discharge using only the reservoir volume (V_r) and the dam height (H_d) or some combination thereof. Physically-based breach models use principles of hydraulics, sediment erosion, and soil stability to construct time-stepping numerical solutions of the breach formation process and the breach outflow hydrograph.

Flood routing is essential for assessing the extent of downstream flooding due to dam-breach outflows because of the extreme amount of peak attenuation the unsteady breach outflow experiences during its propagation through the river/valley. Dam-breach flood routing models have utilized (1) numerical time-step solutions of the complete one-dimensional Saint-Venant equations of unsteady flow; (2) breach peak-flow routing attenuation curves coupled with the Manning equation to compute peak-flow depths; and (3) a simplified Muskingum-Cunge flow routing and Manning equation depth computation. The latter two routing procedures incur additional error compared to the Saint-Venant routing. Finally, future research/development directions for dam-breach prediction are presented.

1. Introduction

A breach is the opening formed in a dam as it fails. The actual

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breach failure mechanics are not well understood for either earthen or concrete dams. Prior to about 1970, most attempts to predict downstream flooding due to dam failures, assumed that the dam failed completely and instantaneously. The assumptions of instantaneous and complete breaches were used for reasons of convenience when applying certain mathematical techniques for analyzing dam–break flood waves. These assumptions are somewhat appropriate for concrete arch dams with reservoir storage volumes greater than about one–half million acre–ft, but they are not appropriate for either earthen dams or concrete gravity dams.

Dam-break modeling and the associated routing of the outflow hydrograph (flood wave) through the downstream river/valley is of continuing concern to many Federal, state, local, and international agencies, the private sector, and academia. Such predictive capabilities are of concern to these entities since they are charged with or assist those charged with responsibilities for dam design, operation, regulation, and/or public safety. This paper presents a perspective on the present capabilities to accomplish dam-breach modeling and the associated flood routing.

Dam–breach models may be conveniently categorized as parametric models or physically–based models. A summary of a relevant portion of the history of dam-breach modeling and dam-breach flood routing capabilities is presented in Table 1. A brief description of both numerical and analytical dam–breach parametric models, dam-breach physically-based models, and dam-breach flood routing models are presented herein.

Finally, future research/development directions in dam-breach prediction capabilities are presented. These are judged to offer the most efficient and effective means of improving practical dam-breach modeling and dam-breach flood routing capabilities.

2. Numerical Parametric Breach Models

Earthen dams which exceedingly outnumber all other types of dams do not tend to completely fail, nor do they fail instantaneously. The fully formed breach in earthen dams tends to have an average width (b_{av}) in the range $(0.5 \le b_{av}/H_d \le 8)$ where H_d is the height of the dam. The middle portion of the range for b_{av} is supported by the summary report of Johnson and Illes (1976) and the upper range by the report of Singh and Snorrason (1982). Breach widths for earthen dams are therefore usually much less than the total length of the dam as measured across the valley. Also, the breach requires a finite interval of time (t_f) for its formation through erosion of the dam materials by the escaping water. The breach formation time is the duration of time between the first breaching of the upstream face of the dam until the breach is fully formed. For overtopping failures the beginning of breach formation is <u>after</u> the downstream face of the dam has eroded away

YEAR	DAM - BREACH OUTFLOW Parametric Numerical	Analytical	Physical/ Numerical Erosion/Collapse/Hydraulics	DAM-BREACH Flood Routing
1969-70	$Q=f(a, t_f, S_a, Q_i)$ Fread, Dissertation	$Q=f(B_r, a, t_f, H_d, S_a)$ Fread	----	----
1977	NWS DAMBRK Last version: 1991	----	----	NWS DAMBRK Last Version : 1991 1 - D
1981	----	$Q=3.1B_r\left(\frac{C}{t_f+C/\sqrt{H_d}}\right)^3$ $C=23.4\ S_a/H_d$	----	NWS SMPDBK Last Version:1991 Curves from DAMBRK
1981-88	----	$Q=f(H_d, V_R, \ldots)$ Hagen, McDonald, Evans, Costa 1982 1984 1986 1988	Ponce/Tsivoglu Erosion/Hydraulic 1981 1-D	----
1983-84	----	----	NWS BREACH Last Version: 1991	----
1987	----	Statistical Data on Parameters Froelich, 1987,1995	----	----
1988	----	Singh/Quiroga	BEED (Singh/Quiroga) 1-DSed (Macchione)	BEED (Mushingum-Cunge)
1993	---	----	2-D/Sed (Bechteler/Broich)	----
1995	NWS FLDWAV same as DAMBRK	----	----	NWS FLDWAV 1- D Multi-River

Table 1. History of Dam-Breach Modeling

and the resulting crevasse has progressed back across the width of the dam crest to reach the upstream face. This portion of the failure process could be thought of as the "initiation time" which is quite distinct from the breach "formation time" or time of failure (t_f). Time of failure *(t_f)* for overtopping initiated failures may be in the range of a few minutes to usually less than an hour, depending on the height of the dam, the type of materials, and the magnitude and duration of the overtopping flow of the escaping water.

Poorly constructed coal-waste dumps (dams) which impound water tend to fail within a few minutes, and have average breach widths in the upper range of the earthen dams mentioned above. Also, average breach widths are considerably larger for reservoirs with very large storages which sustain a fairly constant reservoir elevation during the breach formation time; such a slowly changing reservoir elevation enables the breach to erode to the bottom of the dam and then erode horizontally creating a wider breach before the peak discharge is attained.

Piping failures occur when initial breach formation takes place at some point below the top of the dam due to erosion of an internal channel through the dam by the escaping water. Breach formation times are usually considerably longer for piping than overtopping failures since the upstream face is slowly being eroded in the early phase of the piping development. As the erosion proceeds, a larger and larger opening is formed; this is eventually hastened by caving-in of the top portion of the dam.

Fread (1971,1977,l988,l993) used a parametric approach to describe and mathematically model the dynamic breach–forming process. The mathematical model combined the reservoir level–pool routing equation with a critical–flow, weir equation in which the weir or breach was time dependent whose shape and size were controlled by specified parameters. The numerical time–stepping solution of these equations produced a discharge hydrograph of breach outflow including the maximum (peak) discharge. The parametric description of the dynamic breach is shown in Figure 1.
The breach is assumed to form over a finite interval of time (t_f) and has a final (terminal) breach size of b determined by the breach side–slope parameter *(z)* and the average breach width parameter b_{av}. Such a parametric representation of the breach is

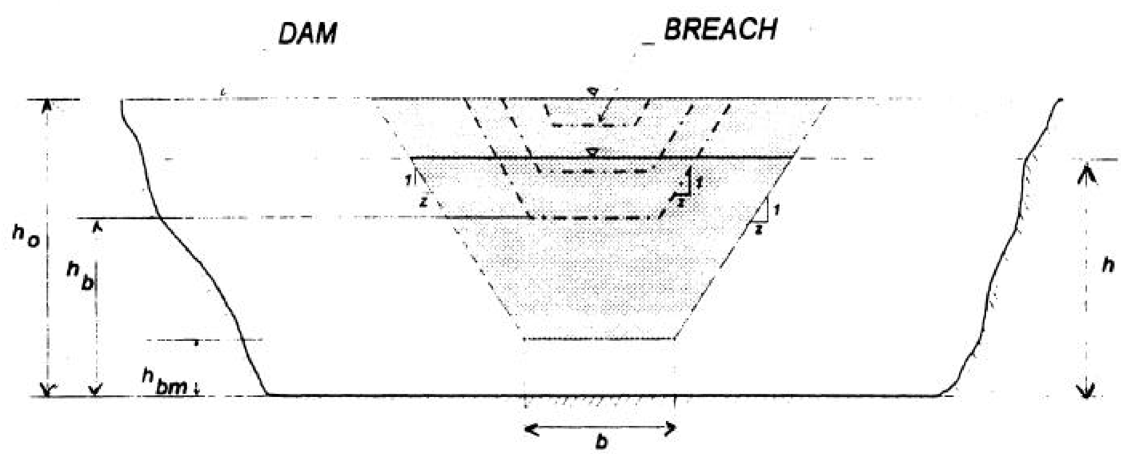

Figure 1. Front View of Dam Showing Formation of Breach.

utilized for reasons of simplicity, generality, wide applicability, and the uncertainty in the actual failure mechanism. This approach to the breach description follows that first used by Fread and Harbaugh (1973) and later in the NWS DAMBRK Model (Fread;1977, 1988). The shape parameter *(z)* identifies the side slope of the breach, i.e., 1 vertical: z horizontal. The range of *z* values is from 0 to somewhat larger than unity. The value of *z* depends on the angle of repose of the compacted, wetted materials through which the breach develops. Rectangular, triangular, or trapezoidal shapes may be specified by using various combinations of values for *z* and the terminal breach bottom width *(b),* e.g., $z=0$ and $b>0$ produces a rectangle and $z>0$ and $b=0$ yields a triangular-shaped breach. The terminal width *b* is related to the average width of the breach (b_{av}) by the following:

$$b = b_{av} - zH_d \qquad (1)$$

in which H_d is the height of the dam. The bottom elevation of the breach (h_b) is simulated as a function of time (t_f) according to the following:

$$h_b = h_d - (h_d - h_{bm})(t_b/t_f)^r \quad 0 \leq t_b \leq t_f \tag{2}$$

in which h_d is the elevation of the top of the dam. The model assumes the instantaneous breach bottom width (b_i) starts at a point (see Figure 1) and enlarges at a linear or nonlinear rate over the failure time (t_f) until the terminal bottom width (b) is attained and the breach bottom elevation (h_b) has eroded to a specified final elevation (h_{bm}). The final elevation of the breach bottom (h_{bm}) is usually, but not necessarily, the bottom of the reservoir or outlet channel bottom, t_b, is the time since beginning of breach formation, and r is a parameter specifying the degree of nonlinearity, e.g., $r=1$ is a linear formation rate, while $r=2$ is a nonlinear quadratic rate; the range for r is $1 \leq r \leq 4$. The linear rate is usually assumed, although the non-linear rate is more realistic especially for piping failures; however, its value is not well identified. The instantaneous bottom width (b_i) of the breach is given by the following:

$$b_i = b\ (t_b/t_f)^r \quad t_b \leq t_f \tag{3}$$

During the simulation of a dam failure, the actual breach formation commences when the reservoir water surface elevation (h) exceeds a specified value, h_f. This enables the simulation of an overtopping of a dam in which the breach does not form until a sufficient amount of water is flowing over the top of the dam. A piping failure may also be simulated by specifying the initial centerline elevation of the pipe-breach.

2.1 Statistically-Based Breach Predictors

Some statistically derived predictors for b_{av} and t_f. have been presented in the literature, i.e, MacDonald and Langridge-Monopolis(1984) and Froehlich(1987,l995). Using this data of the properties of 63 breaches of dams ranging in height from 15 to 285 ft, with 6 of the dams greater than 100 ft, the following predictive equations are obtained:

$$b_{av} = 9.5 k_o (V_r\ H)^{0.25} \tag{4}$$

$$t_f = 0.3 V_r^{0.53} / H^{0.9} \tag{5}$$

in which b_{av} is average breach width (ft), t_f is time of failure (hrs), k_o = 0.7 for piping and 1.0 for overtopping, V_r is volume(acre–ft) and H is the height (ft) of water over the breach bottom (H is usually about the height of the dam, H_d). Standard error of estimate for b_{av} is ±56 percent of b_{av}, and the standard error of estimate for t_f is ±74 percent of t_f.

3. Analytical Parametric Breach Models

A single analytical equation was also developed to predict the peak outflow from a breached dam. Fread (1981,l984) developed such an equation which was a critical component of the NWS SMPDBK (Simplified Dam-Break) model. This equation accounted for the hydraulic processes of dam-breach outflows, i.e, the simultaneous lowering of the reservoir elevation as the breach forms by the escaping reservoir outflow while using the basic breach parameters (b_{av}, t_f), i.e.

$$Q_p=3.1b_{av}[C/(t_f + C/H_d^{0.5})]^3 \qquad (6)$$

in which Q_p is the peak breach outflow in cfs, b_{av} is the average breach width in feet, t_f is the breach formation time in hours, H_d is the height of the dam in feet, and $C=23.4\ S_a/b_{av}$ in which S_a is the reservoir surface area (acres) somewhat above the elevation of the top of the dam. Recently, a similar but considerably more complicated approach was reported by Walder and O'Connor(1997).

Another analytical (single equation) approach for earthen dam–breaches relies on a statistical regression approach that relates the observed (estimated) peak dam–breach discharge to some measure of the impounded reservoir water volume: depth, volume, or some combination thereof, e.g., Hagen,l982; Evans,l986; Costa, 1988; Froehlich,l995. An example of this type of equation follows:

$$Q_p=aV_r^bH_d^c \qquad (7)$$

in which V_r is the reservoir volume, H_d is the height of the dam, and a, b, c are regression coefficients, e.g., Froehlich(1995) quantifies these as $a=40.1$, $b=O.295$, $c=1.24$ in which the units for Q_p, V_r, and H_d are cfs, acre-ft and ft, respectively. This approach is expedient but generally only provides an order of magnitude prediction of dam-breach peak discharge. It does not reflect the true hydraulics, but instead mixes the failure-erosion process and the hydraulic processes, while ignoring the important components of time–dependent erosion, weir flow, and reservoir routing.

4. Physically-Based Breach Erosion Models

Another means of determining the breach properties is the use of physically-based breach erosion models. Cristofano(l965) modeled the partial, time-dependent breach formation in earthen dams; however, this procedure required critical assumptions and specification of unknown critical parameter values. Also, Harris and Wagner(1967) used a sediment transport relation to determine the time for breach formation, but this procedure required specification of breach size and shape in addition to two critical parameters for the sediment transport computation; then, Ponce and Tsivoglou(1981) presented a rather computationally complex breach erosion model which coupled the Meyer–Peter and Muller sediment transport equation to the one–dimensional differential equations of unsteady flow (Saint-Venant equations) and the sediment conservation equation. They compared the model's predictions with observations of a breached landslide–formed dam on the Mantaro River in Peru. The results were substantially affected by the judicious selection of the breach channel hydraulic friction factor (Manning n), an empirical breach width–flow parameter, and an empirical coefficient in the sediment transport equation.

Another physically-based breach erosion model (BREACH) for earthen dams was developed (Fread;1984,1987) which utilizes principles of soil mechanics, hydraulics, and sediment transport. This model substantially differed from the previously mentioned models. It predicted the breach characteristics (size, shape, time of formation) and the discharge hydrograph emanating from a breached earthen dam which was man–made or naturally formed by a landslide; the typical scale and geometrical variances are illustrated in Figure 2. The model was developed by coupling the

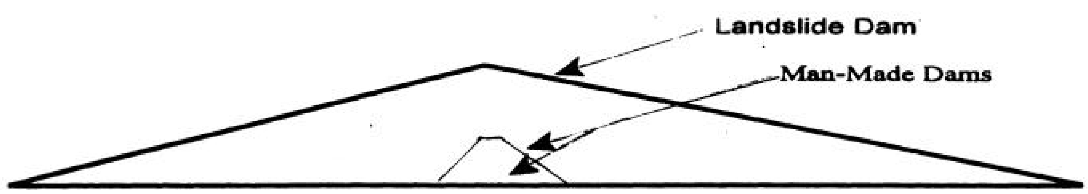

Figure 2. Comparative View of Natural Landslide Dams and Man-Made Dams.

conservation of mass of the reservoir inflow, spillway outflow, and breach outflow with the sediment transport capacity (computed along an erosion-formed breach channel. The bottom slope of the breach channel was assumed to be the downstream face of the dam as shown in Figure 3. The growth of the breach channel, conceptually modeled as shown in Figure 4, was dependent on the dam's material properties *(D_{50}* size, unit weight(g), internal friction angle (f), and cohesive strength (C_h)).

The model considered the possible existence of the following complexities: (1) core material properties which differ from those of the outer portions of the dam; (2) formation of an eroded ditch along the downstream face of the dam prior to the actual breach formation by the overtopping water; (3) the downstream face of the dam could have a grass cover or be composed of a material such as rip–rap or cobble stones of larger grain size than the major portion of the dam; (4) enlargement of the breach through the mechanism of one or more sudden structural

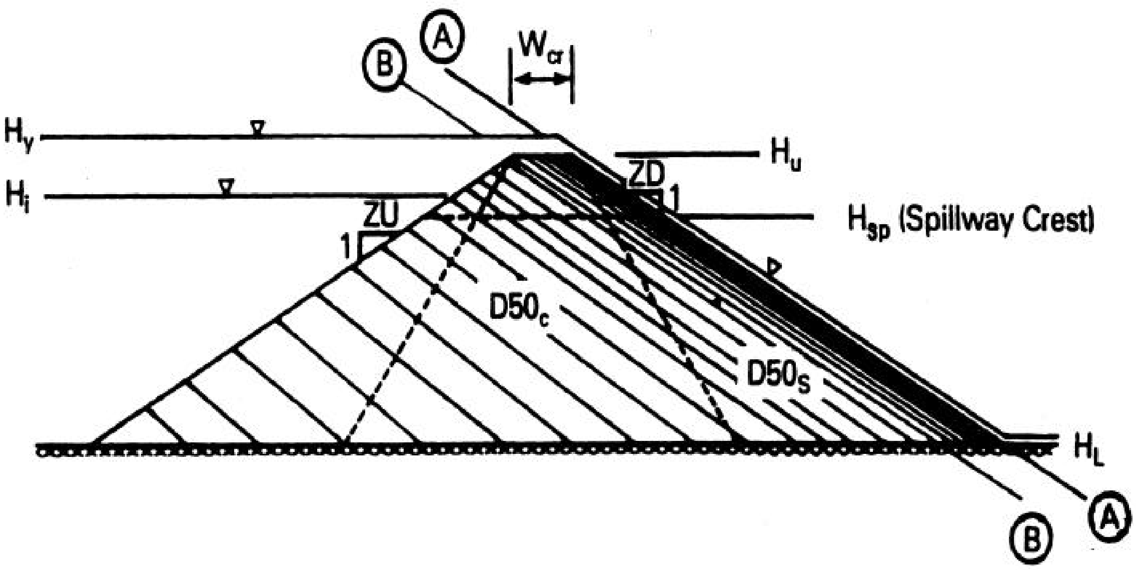

Figure 3. Side View of Dam Showing Conceptualized Overtopping Failure Sequence.

collapses of the breaching portion of the dam due to the hydrostatic pressure force exceeding the resisting shear and cohesive forces; (5) enlargement of the breach width by collapse of the breach sides according to slope stability theory as shown in Figure 4; and (6) the capability for initiation of the breach

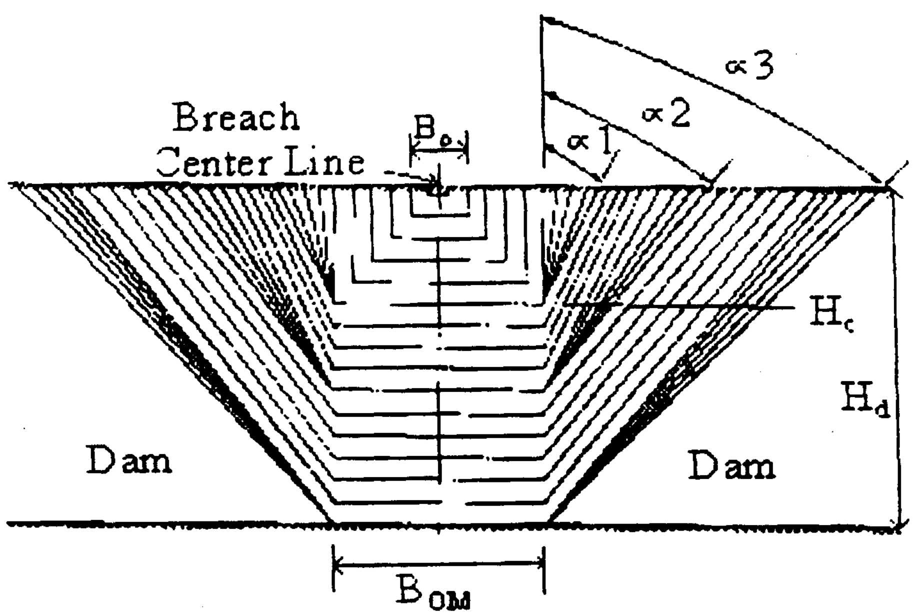

Figure 4. Front View of Dam with Breach Formation Sequence.

via piping with subsequent progression to a free—surface breach flow. The outflow hydrograph was obtained through a computationally efficient time-stepping iterative solution. This breach erosion model was *not* subject to numerical stability/ convergence difficulties experienced by the more complex model of Ponce and Tsivoglou (1981). The model's predictions were favorably compared with observations of a piping failure of the large man-made Teton Dam in Idaho, the piping failure of the small man–made Lawn Lake Dam in Colorado, and an overtopping activated breach of a large landslide–formed dam in Peru. Model sensitivity to numerical parameters was minimal. A variation of $\pm$ 30 percent in the internal friction angle and a ± 100 percent variation in the cohesion parameter resulted in less than ± 20 percent variation in the simulated breach properties and peak breach outflow. However, it was somewhat sensitive to the extent of grass cover when simulating man–made dams in which over topping flows could or could not initiate the failure of the dam.

A brief description of three breach simulations follows:

(1) Teton Dam. The BREACH model was applied to the piping initiated failure (Fread;1984,1987) of the earthfill Teton Dam which breached in June 1976, releasing an estimated peak discharge (Q_p) of 2.2 million cfs having a range of 1.6 to 2.6 million cfs. The material properties of the breach were as follows: H_d=262.5 ft, D_{50}=0.lmm, f=20 deg, C_h=3O lb/ft^2, and g=100 lb/ft^3. The downstream face of the dam had a slope of 1: 4 and upstream face slope was 1:2. An initial piping failure of 0.01 ft located at 160 ft above the bottom of the dam commenced the simulation. The simulated breach hydrograph is shown in Figure 5. The computed final breach top width *(W)* of 645 ft compared well with the observed value of 650 ft. The computed side slope of the breach was 1:1.06 compared to 1:1.0. The computed time *(T_p)* to peak flow was 2.2 hr compared to 1.95-2.12 hr.

(2) Lawn Lake Dam. This dam was a 26 ft high earthen dam with approximately 800 acre-ft of storage, which failed July 15, 1982, by piping along a bottom drain pipe (Jarrett and Costa,1984). The BREACH model was applied (Fread,1987) with the piping breach assumed to commence within 2 ft of the bottom of the dam. The material properties of the breach were assumed as follows: H_d=26 ft, D_{50}=0.25mm, f=25 deg, C_h=l00 lb/ ft^2, and g=100 lb/ft^3. The downstream face of the dam had a slope of 1:3 and the upstream face 1:1.5. The computed outflow was 17,925 cfs, while the estimated actual outflow was 18,000 cfs. The model produced a trapezoidal-shaped breach with top and bottom dimensions of 132 and 68 ft, respectively. The actual breach dimensions were 97 and 55 ft, respectively. The mean observed breach width was about 32 percent smaller than the mean breach width produced by the model.

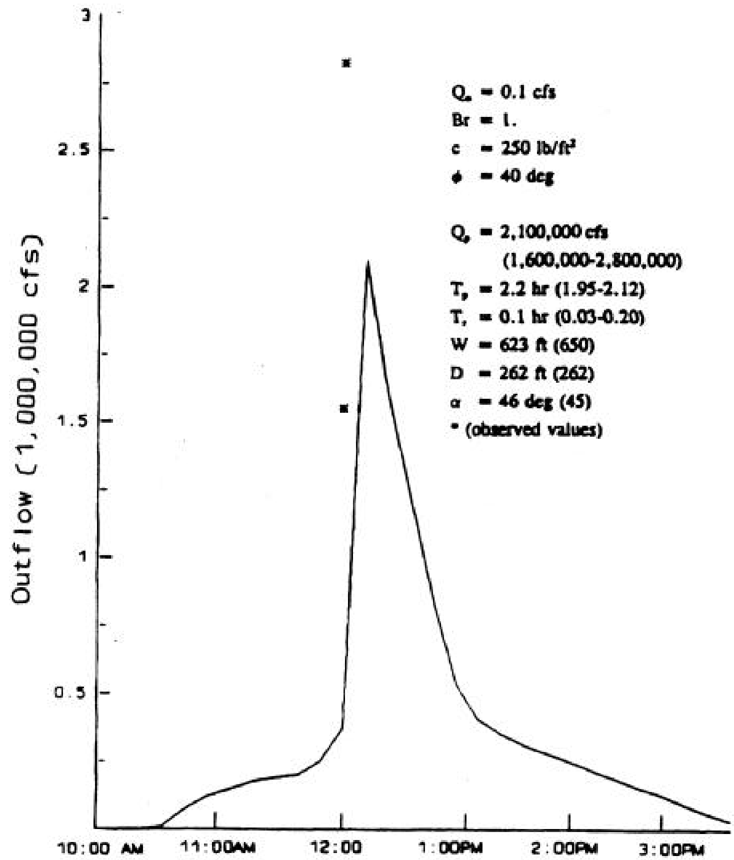

Figure 5. Teton Dam: Predicted and Observed Breach Outflow Hydrograph and Breach Properties

(3) Mantaro Landslide Dam. A massive landslide occurred in the valley of the Mantaro River in the mountainous area of central Peru on April 25, 1974. The slide, with a volume of approximately 5.6 x 10^{10}ft, dammed the Mantaro River and formed a lake which reached a depth of about 560 ft before overtopping during the period June 6-8, 1974 (Lee and Duncan,1975). The overtopping flow very gradually eroded a small channel along the approximately 1 mile long downstream face of the slide during the first two days of overtopping. Then a dramatic increase in the breach channel occurred during the next 6–10 hours resulting in a final trapezoidal-shaped breach channel approximately 350 ft deep, a top width of some 800 ft, and side slopes of about 1:1. The peak flow was estimated at 353,000 cfs as reported by Lee and Duncan (1975), although Ponce and Tsivoglou(l98l) later reported an estimated value of 484,000 cfs. The breach did not erode down to the original river bed; this caused a rather large lake about 200 ft deep to remain after the breaching had subsided some 24 hours after the peak had occurred. The landslide material was mostly a mixture of silty sand with some clay resulting in a D_{50} size of about 11mm with some material ranging in size up to 3 ft boulders. The BREACH model was applied (Fread;1984,l987) to the Mantaro landslide-formed dam using the following parameters:

upstream face slope 1:17, downstream face slope 1:8, *Hd*=560 *ft*, D_{50}=llmm, C_h=*30* lb/ft^2, f=38 deg, g=100 lb/ft^3. The initial breach depth was assumed to be 0.35 ft. The computed breach outflow is shown in Figure 6 along with the estimated actual values. The timing of the peak outflow and its magnitude are very similar as are the dimensions of the gorge eroded through the dam shown by the values of *D, W,* and a in Figure 6. Of particular interest, the BREACH model produced a depth of breach of 352 ft which compared to the observed depth of 350 ft.

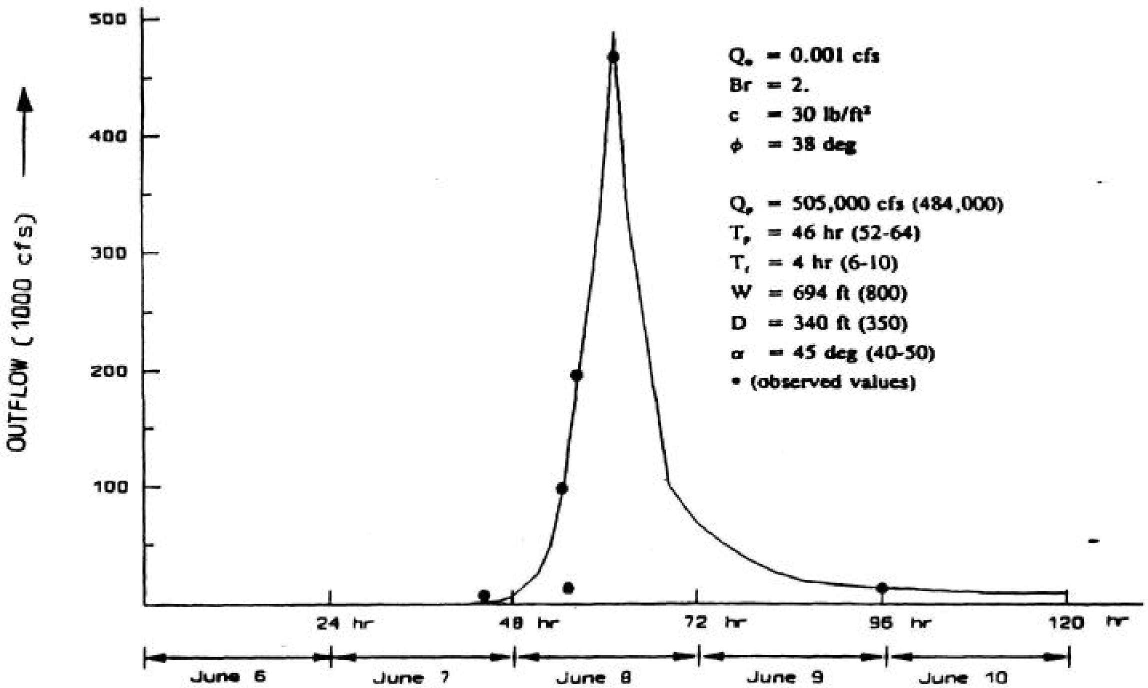

Figure 6. Mantaro Landslide Dam: Predicted and Observed Breach Hydrograph and Breach Properties.

Other physically-based breach erosion models include the following: (1) the BEED model (Singh and Quiroga,1988) which is similar to the BREACH model except it considers the effect of saturated soil in the collapse of the breach sides and it routes the breach outflow hydrograph through the downstream valley using a simple diffusion routing technique (Muskingum-Cunge) which neglects backwater effects and can produce significant errors in routing a dam-breach hydrograph when the channel/valley slope is less than 0.003 ft/ft; (2) a numerical model (Macchione and Sirangelo,1988) based on the coupling of the one-dimensional unsteady flow (Saint-Venant) equations with the continuity equation for sediment transport and the Meyer-Peter and Muller sediment transport equation; and (3) a numerical model (Bechteler and Broich,1993) based on the coupling of the two-dimensional unsteady flow equations with the sediment continuity equation and the Meyer–Peter and Muller equation.

4. Flood Routing

Flood waves produced by the breaching (failure) of a dam are known as dam–breach flood waves. They are much larger in peak magnitude, considerably more sharp–peaked, and generally of much shorter duration with flow acceleration components of a far greater significance than flood waves produced by precipitation runoff. The prediction of the time of occurrence and extent of flooding in the downstream valley is known as flood routing. The dam-breach wave is modified (attenuated, lagged, and distorted) as it flows (is routed) through the downstream valley due to the effects of valley storage, frictional resistance to flow, flood flow acceleration components, flow losses, and downstream channel constrictions and/or flow control structures. Modifications to the dam-break flood wave are manifested as attenuation (reduction) of the flood peak magnitude, spreading-out or dispersion of the temporal varying flood–wave volume, and changes in the celerity (propagation speed) or travel time of the flood wave. If the downstream valley contains significant storage volume such as a wide floodplain, the flood wave can be extensively attenuated (see Figure 7) and its propagation speed greatly reduced. Even when the downstream valley approaches that of a relatively narrow uniform rectangular–shaped section, there is appreciable attenuation of the flood peak and reduction in the wave celerity as the wave progresses through the valley.

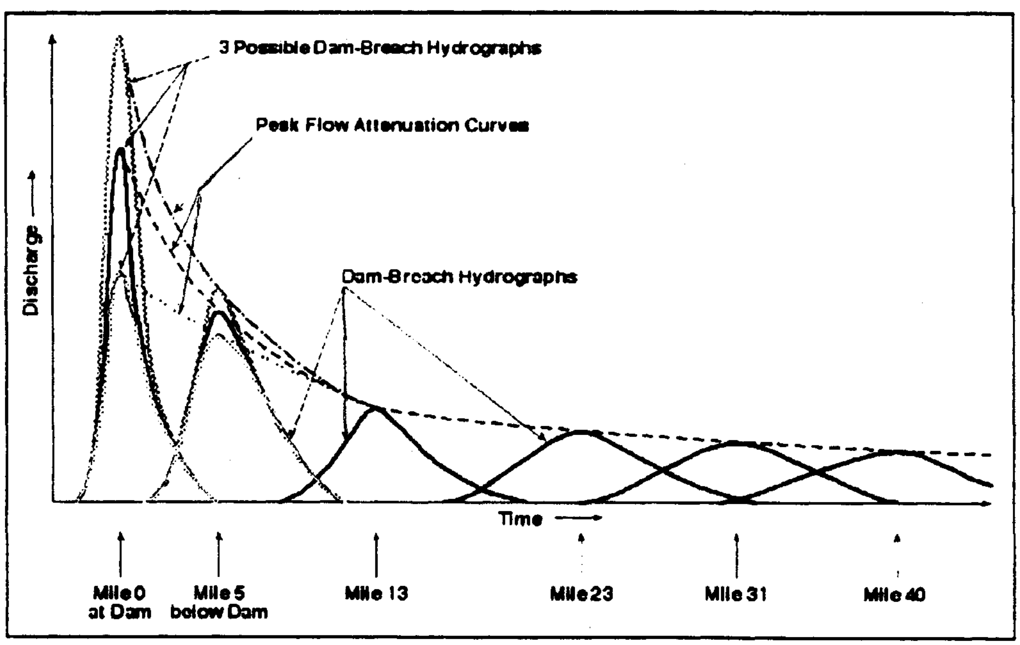

Figure 7. Dam-Break Flood Wave Attenuation Along the Routing Reach.

5.1 Flood Routing with Saint-Venant Equations

Dam–breach flood waves have been routed using simulation models based on numerical solutions of the one–dimensional Saint-Venant equations of unsteady flow, e.g., DAMBRK (Fread,1977,l978) and FLDWAV (Fread, 1993). The Saint-Venant equations used in these models consists of the mass conservation equation, i.e.,

$$\delta Q/\delta x+s_c\delta(A+A_o)/\delta t - q=0 \qquad (8)$$

and the momentum conservation equation, i.e.,

$$s_m\delta Q/\delta t + \beta(\delta Q^2/A)/\delta x+gA(\delta h/\delta x+S_f+S_e)-qn_x=0 \qquad (9)$$

where h is the water–surface elevation, A is the active cross–sectional area of flow, A_o is the inactive ***(off-channel storage)*** cross–sectional area, s_c and s_m are depth-weighted sinuosity coefficients which correct for the departure of a sinuous in–bank channel from the x-axis of the valley floodplain, x is the longitudinal mean flow–path distance measured along the center of the river/valley watercourse (river channel and floodplain), t is time, q is the lateral inflow or outflow per lineal distance along the river/valley (inflow is positive and outflow is negative), β is the momentum coefficient for nonuniform velocity distribution within the cross section, g is the gravity acceleration constant, S_f is the boundary friction slope, S_e is the expansion–contraction (large eddy loss) slope, and n_x is the velocity component of the lateral flow along the x–axis.

5.2 Peak Flow Routing Attenuation Curves

Another flood routing technique SMPDBK (Fread and Wetmore,1984; Fread, et al.,1991) has been used when the river/valley downstream from a breached dam is uncomplicated by unsteady backwater effects, levee overtopping, or large tributaries. SMPDBK determines the peak flow, depth, and time of occurrence at selected locations downstream of a breached dam. SMPDBK first computes the peak outflow at the dam, based on the reservoir size and the temporal and geometrical description of the breach. The SMPDBK uses an analytical time-dependent broad-crested weir equation, Eq.(6), to determine the maximum breach outflow (Q_p) in cfs and the user is required to supply the values of four variables for this equation. These variables are: (1) the surface area (A_s, acres) of the reservoir; (2) the depth (H_d, ft) to which the breach erodes; (3) the time (t_f, hrs) required for breach formation; and (4) the width (b_{av}, ft) of the breach, and (5) the spillway flow and overtopping crest flow (Q_o) which is estimated to occur simultaneously with the breach peak outflow. The computed flood wave and channel properties are used in conjunction with special dimensionless routing curves (see Figure 8) to determine how the peak flow will be diminished as it moves downstream.

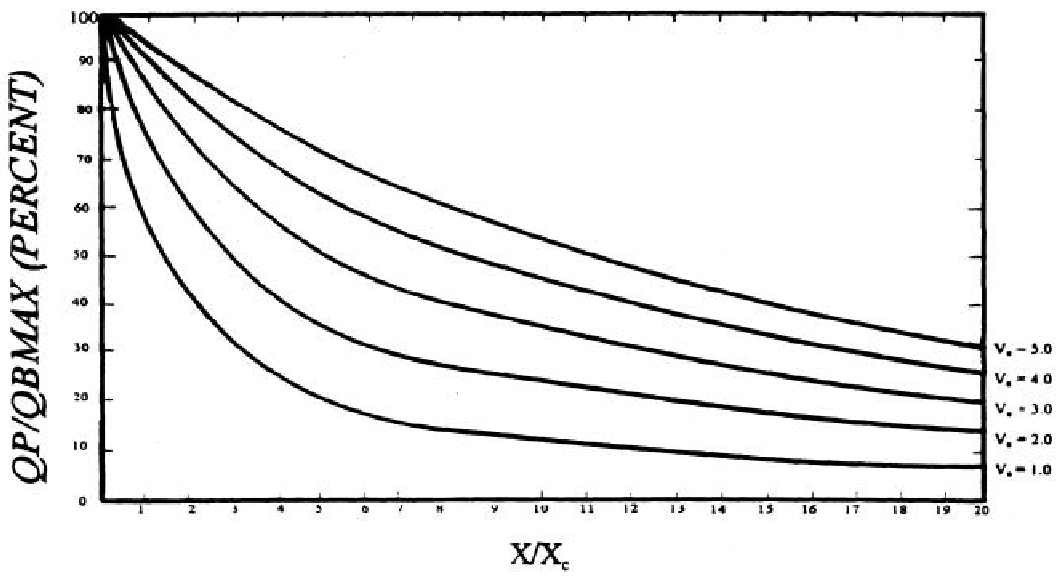

Figure 8. Routing Curves for SMPDBK Model for Froude No. = 0.25.

The dimensionless routing curves were developed from numerous executions of the NWS DAMBRK model and they are grouped into families based on the Froude number associated with the flood wave peak, and have as their *X*–abcissa the ratio of downstream distance (from the dam to a selected cross–section where Q_p and other properties of the flood wave are desired) to a distance parameter (X_c). The *Y*-ordinate of the curves used in predicting peak downstream flows is the ratio of the peak flow (Q_p) at the selected cross section to the computed peak flow at the dam, QBMAX. The distinguishing characteristic of each member of a family is the ratio (V_o) of the volume in the reservoir to the average flow volume in the downstream channel from the dam to the selected section. To specify the distance in dimensionless form, the distance parameter (X_c) in feet is computed as follows:

$$X_c = 6V_r/[(1+4(0.5)^{m-1}A_d] \qquad (10)$$

in which V_r *is* the reservoir volume (acre–ft), *m* is a cross-sectional shape factor for the routing reach, and A_d, is the average cross–section area in the routing reach at a depth of H_d. The volume parameter (V_o) is $V_o=V_r/(\ _cX_c)$ in which $_c$ represents the average cross-sectional area in the routing reach at the average maximum depth produced by the routed flow. The Froude Number (F_c) *is* $F_c=V_c/(gD_c)$ where V_c and D_c, are the average velocity and hydraulic depth, respectively, within the routing reach. Further details on the computation of the dimensionless parameters can be found elsewhere (Wetmore and Fread,1984; Fread,et al.,1991).

The SMPDBK model then computes the depth produced by the peak flow using the Manning equation based on the channel geometry, slope, and roughness at the selected downstream locations. The model also computes the time required for the peak to reach each forecast location and, if a flood depth is entered for the point, the time at which that depth is reached, as well as when the flood wave recedes below that depth, thus providing a time frame for evacuation and possible fortification on which a preparedness plan may be based. The SMPDBK model neglects backwater effects created by any downstream dams or bridge embankments, the presence of which may substantially reduce the model’s accuracy. However, its speed and ease of use, together with its small computational requirements, make it an attractive tool for use in cases where limited time and resources preclude the use of the DAMBRK or FLDWAV models. In such instances, planners, designers, emergency managers, and consulting engineers responsible for predicting the potential effects of a dam failure may employ the model where backwater effects are not significant.

The SMPDBK model was compared with the DAMBRK model in several theoretical applications (Fread, et al.,1991) and several hypothetical dam failures (Westphal and Thompson,l987) where the effects of backwater, downstream dams/bridges, levee overtopping, or significant downstream tributaries were negligible. The average differences between the two models were less than 10 percent for predicted flows, travel times, and depths.

5.3 Numerical Routing with Muskingum-Cunge Equation

Another simple routing model (Muskingum-Cunge with variable coefficients) may be used for routing dam-breach floods through downstream river/valleys with moderate to steep bottom slopes (S_o>0.003 ft/ft). The spatially distributed Muskingum-Cunge routing equation applicable to each Dx_i subreach for each Dt^j time step is as follows:

$$Q_{i+1}^{j+1}=C_1Q_i^{j+1} +C_2Q_i^j +C_3Q_{i+1}^j +C_4 \qquad (11)$$

The coefficients C_0, C_1, and C_2 are positive values whose sum must equal unity; they are defined as

$$C_0 = 2\ K(1–X) + Dt^j \qquad (12)$$

$$C_1 = (Dt^j-2KX)/C_0 \qquad (13)$$

$$C_2 = (Dt^j+2KX)/C_0 \qquad (14)$$

$$C_3= [2K(1–X) – Dt]/C_0 \qquad (15)$$

$$C_4= q_i\ Dx_i Dt^j/C_0 \qquad (16)$$

in which K is a storage constant having dimensions of time, X is a weighting factor expressing the relative importance of inflow and outflow on the storage in the Dx_i subreach of the river, and q_i the lateral inflow (+) or outflow (-) along the Dx_i subreach.

K and X are computed as follows:

$$K = \Delta x_i / c \qquad (17)$$

$$X = 0.5[1 - D/(k' S \Delta x_i)] \qquad (18)$$

in which c is the kinematic wave celerity $c=k'Q/A$, Q is discharge, S is the energy slope approximated by evaluating S_f for the initial flow condition, D is the hydraulic depth A/B where A is the cross-sectional area and B is the wetted top width associated with Q, and k' is the kinematic wave ratio, i.e., $k'=5/3-2/3\ A(dB/dh)/B^2$. The bar indicates the variable is averaged over the Δx_i reach and over the Δt^j time step. The coefficients (C_0, C_1, C_2, C_3, C_4) are functions of Δx_i and Δt^j (the independent parameters), and D, c, and k' (the dependent variables) are also functions of water–surface elevations (h).

These water–surface elevations may be obtained from a steady, uniform flow formula such as the Manning equation, i.e.,

$$Q = m/n A R^{2/3} S^{1/2} \qquad (19)$$

in which n is the Manning roughness coefficient, A is the cross-sectional area, R is the hydraulic radius given by A/P in which P is the wetted perimeter of the cross section, S is the energy slope as defined previously, and m is a units conversion factor (1.49 for U.S. and 1.0 for SI).

5.4 Testing of DAMBRK and SMPDBK

The DAMBRK and SMPDBK models have been tested on several historical floods due to breached dams to determine their ability to reconstitute observed downstream peak stages, discharges, and travel times. Among the floods that have been used in the testing are: 1976 Teton Dam Flood, 1972 Buffalo Creek (coal-waste dam) Flood, 1889 Johnstown Dam Flood, 1977 Toccoa (Kelly Barnes) Dam Flood, the 1997 Laurel Run Dam Flood and others. Some of the results from the Teton and Buffalo Creek dam-breach tests follow.

The Teton Dam, a 300 ft high earthen dam with 230,000 acre-ft of stored water and maximum 262.5 ft water depth, failed on June 5, 1976, killing 11 people making 25,000 homeless and inflicting about $400 million in damages to the downstream Teton-Snake River Valley. The following observations were reported (Ray, et al.,1976): the approximate development of the breach, description of the reservoir storage, downstream cross–sections and estimates of Manning's n approximately every five miles, estimated peak discharge measurements of four sites, flood–peak travel times, and flood-peak elevations. The critical breach parameters were t_f=1.43 hours, b=80 ft, and z=1.04. The computed peak flow profile along the downstream valley is shown in Figure 9. Variations between computed and observed values are about 5 percent for DAMBRK and 12 percent for SMPDBK. The Buffalo Creek “coal waste” dam, a 44 ft high tailings dam with 400 acre-ft of storage failed on February 1972, resulting in

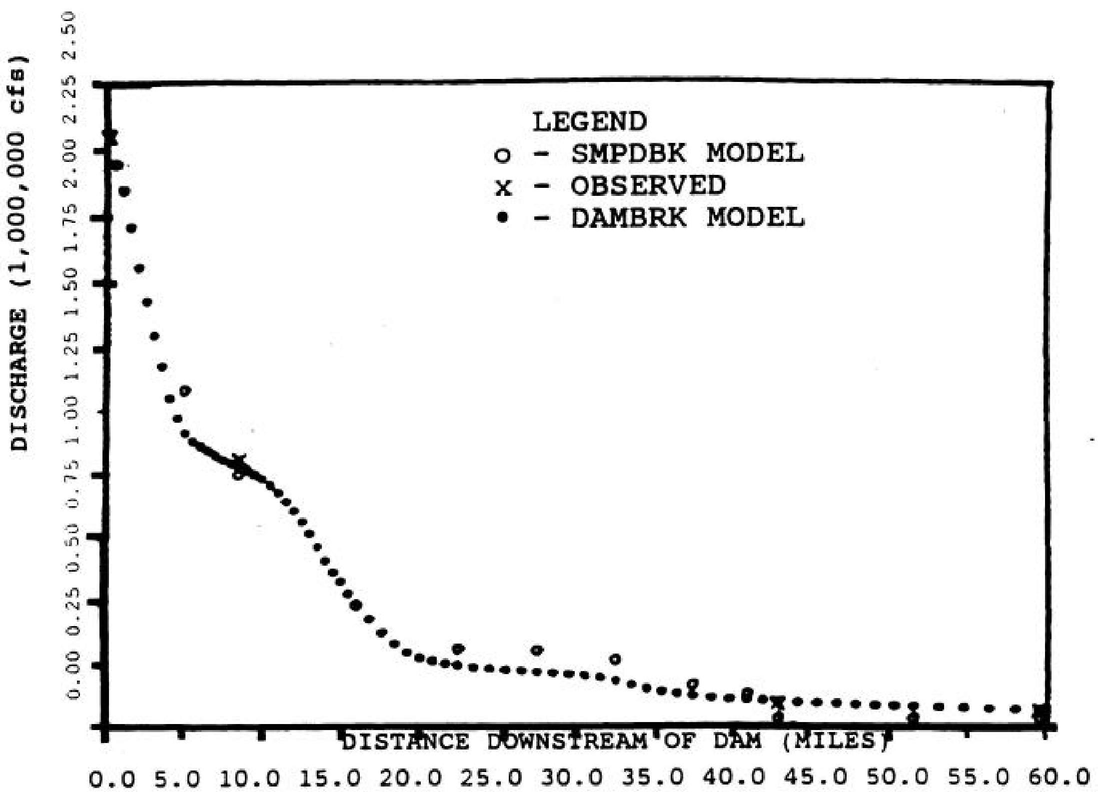

Figure 9. Profile of peak discharge downstream of Teton.

118 lives lost and over $50 million in property damage. Flood observations (Davies, et al., 1975) along with the computed flood—peak profile extending about 16 miles downstream are shown in Figure 10. Critical breach parameters were t_f=0.08 hours, b=l70 ft, and $z=2.6$. Comparison of computed and observed flows indicate an average difference of about 11 percent for both DAMBRK and SMPDBK.

The Muskingum-Cunge flood routing model was compared with the DAMBRK (Saint-Venant) model for all types of flood waves (Fread and Hsu,1993). For dam—breach waves, the routing error associated with the more simple but less accurate Muskingum-Cunge model was found to exceed 10 percent when the channel bottom slope $S_o<0.004/t_f^{0.89}$; the error increased as the bottom slope became more mild and as the time of failure (t_f) became smaller.

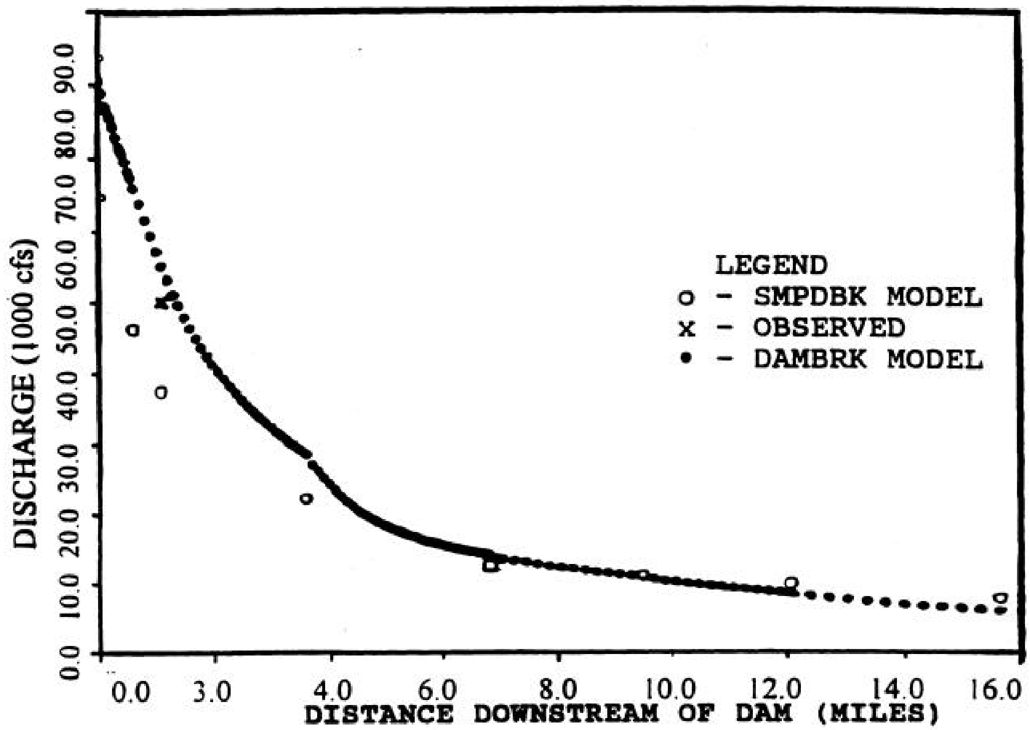

Figure 10. Profile of peak discharge downstream of Buffalo Creek

5 Future Research/Development Directions

Some possible effective future research/development directions that could improve prediction of dam-breach floods are the following: (1) use prototype physical experiments to develop breach models for embankment dams which simulate both breach "initiation time" and breach "formation time"; first, for clay embankment dams after Temple and Moore (1997), then for silt/loam embankments, sand/gravel embankments, and embankments with clay or concrete seepage-prevention cores; (2) determine the Manning n flow resistance values for dam-breach floods using both historical data from such floods and using theoretical approaches such as the component analysis used by Walton and Christianson (1980) similar to the Colebrook equation (Streeter,1966). Also, determine procedures to account for flood debris blockage effects on Manning n values and the dam effect on bridge openings (the latter could be simulated as an internal boundary equation consisting of a discharge-depth rating function which would represent increasing discharge with increasing flow depth, followed by decreasing discharge as debris blockage accumulates with increasing depth, followed by a time-dependent breach of the debris blocked bridge); (3) develop methodologies, e.g., Monte-Carlo simulation (Froehlich,1998), to produce the inherent probabilistic features of dam-breach flood predictions due to the uncertainty associated with reservoir inflows, breach formation, and downstream Manning n/debris effects.

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

An Introduction to RESCDAM project

Mikko Huokuna ed.
Finnis Environment Institute

1. INTRODUCTION

Finland's dams and reservoirs have been constructed mainly for flood control, hydroelectric power production, water supply and fish culture, as well as for storing waste detrimental to health or the environment. Some of the reservoirs play also a recreational role. At present, there are 55 large dams in Finland. According to the Finnish dam safety legislation, 36 dams require a rescue action plan. The development of rescue actions based on the risk analysis and the dam-break flood hazard analysis was found necessary by the Finnish Ministry of the Interior, the Ministry of the Agriculture and Forestry, and the organizations subordinate to these ministries. The launching of the RESCDAM project is meant to improve the dam safety sector. The project is financially supported by the European Union, the Finnish Ministry of the Agriculture and Forestry, the Finnish Ministry of the Interior and the West Finland Regional Environment Centure.

The RESCDAM project is co-ordinated by the Finnish Environment Institute. The project is being carried out in co-operation with: ENEL SpA Ricerca Polo Idraulico e Strutturale (Milan), EDF Electricite de France (Paris), the Helsinki University of Technology, the Emergency Services College (Kuopio), the Seinäjoki Rescue Centre (Seinäjoki), and the West Finland Regional Environment Centre (Vaasa, Seinäjoki). The sub-contractors of the project are the PR Water Consulting Ltd. (Helsinki) and Professor Emeritus Eero Slunga (Espoo).

The activities of the RESCDAM project embrace the risk analysis, the dam-break flood analysis and the rescue actions improvement.

The risk analysis consists of:

- Assessment of the risk of events or processes that could lead to a dam-break.
- Assessment of the risks posed by a dam-break flood causing hazards to the population, property and the environment downstream from the dam.
- Depth-velocity dam-break flood danger level curves.
- Sociological analysis on the reaction of the population during dam-break flooding and the related rescue actions and evacuation.

The dam-break flood hazard analysis consists of:

-Application of the numerical model for the simulation of dam breach formation to the example dam case.
-Application of the numerical flow models (1- and 2-d models) for dam-break flood simulation to the pilot project dam case.
-Study of the effect of different approaches to model urban areas on flood propagation.

The development of the rescue actions includes:

-Basic international investigation on how dam-break rescue actions are organised in different countries. The evaluation of the requirement for equipment, tools, training and exercises.
-Recommendations to update the existing Finnish dam-break flood rescue guidelines.
-Evaluation of the opportunities for special training and exercises taking into account the need for continuous updating activities. Acquirement of the special rescue equipment required and making arrangements for practical training and rescue exercises at the example dam.
-Drawing up rescue action plans for the example dam.

The results of the RESCDAM project formed a basis for an international seminar and workshop. The seminar and workshop took place in Seinäjoki in October 1-5, 2000. The purpose was to create a forum for the presentation of and discussion on practical experience in dam safety and emergency action planning. The papers presented during theseminar will be included in the RESCDAM final report.

The RESCDAM project started in June, 1999 and will finish in 2001. The project’s final report will be available in the form of a CD and on the net.

2. THE KYRKÖSJÄRVI RESERVOIR PILOT PROJECT

The Kyrkösjärvi Reservoir, located in the Seinäjoki City, North West Finland, is an off-river channel reservoir using the water from the river Seinäjoki, a major tributary of the river Kyrönjoki (see Figure 3.2.1.). This reservoir and its embankment dam were chosen as the area of the pilot project in the RESCDAM project. There are several reasons for this choice. The reservoir is in multi purpose use (flood control, hydropower production, water supply, cooling water for a thermal power plant and recreational use) and it is therefore very important for the local population. The area has also been recently surveyed and high accuracy digital maps and a digital terrain model are available. The reservoir is located in the City area, with urban areas at risk. The local rescue centre has in addition to the Kyrkösjärvi reservoir to deal also with other reservoirs and dams in its area and is therefore highly motivated to develop its organisation's skills.

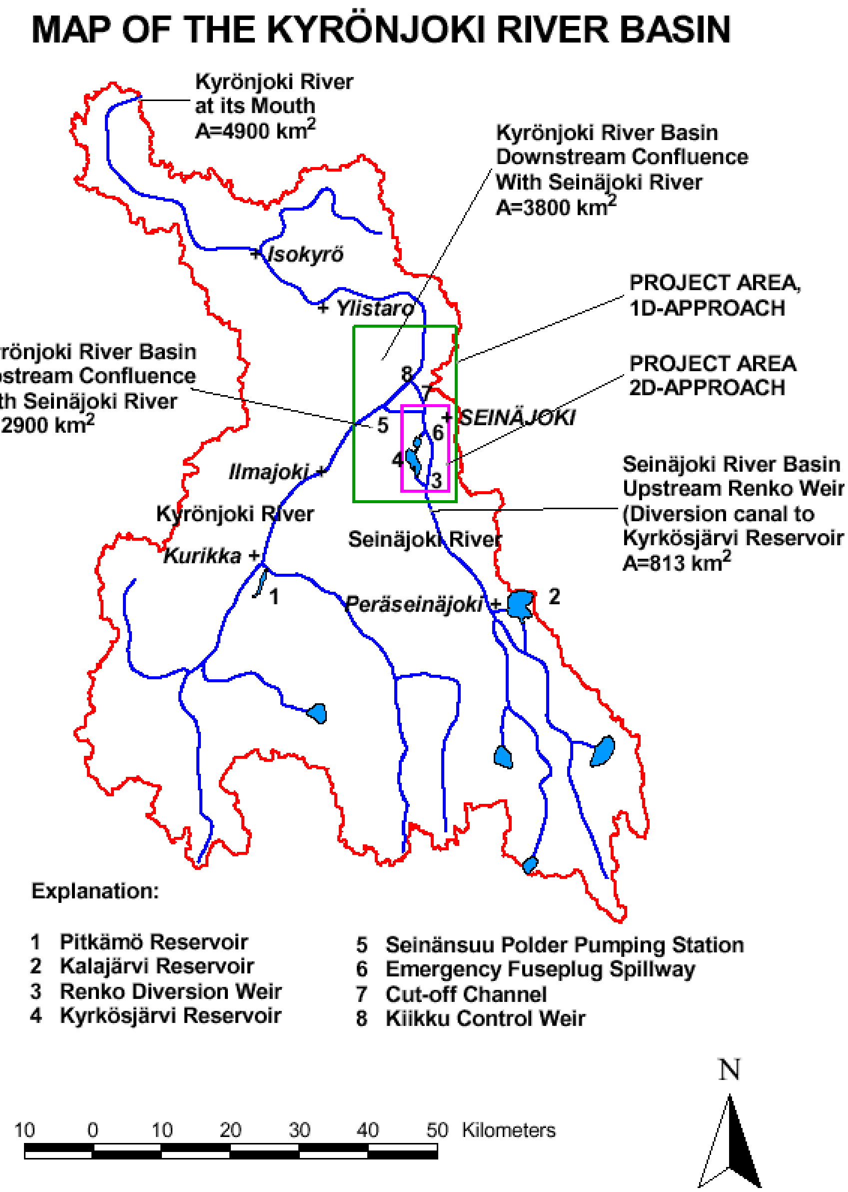

Figure 1. Map of the Kyrönjoki River basin

The dam was designed in 1977 and taken into use on February 6, 1981. The reservoir is used as flood storage for the River Seinäjoki, which flows trough the city area. There is a hydropower plant in the northern part of the lake and a thermal power plant which uses the water from the lake as cooling water, at the eastern bank of the reservoir. The volume of the reservoir is 15.8 10^6 m^3 at the flood HW-level 81.25 m and 22.3 10^6 m^3 at the emergency HW-level 82.25 m (HW + safety margin).

Seinäjoki river is a tributary of the Kyrönjoki river (drainage area 4.900 km^2).The drainage area of the Seinäjoki river upstream of the Kyrkösjärvi reservoir is 813 km^2. Water from Seinäjoki river flows to the reservoir trough a canal which begins at Renko Weir (Figure 2.). The discharge through the canal between the Renko Weir and the reservoir are at low Reservoir levels or high river levels

45 m^3/s and at low water level differences, at least 25 m^3/s. The discharge from the reservoir through the turbines of the hydropower plant can be 21 m^3/s and through a discharge valve 2 m^3/s.

Kyrkösjärvi dam is a homogeneous embankment dam (Figure 2.). The length of the dam is 12.5 km, from which one third is lower than three meters. The maximum height of the dam is about seven meters. The core material of the dam is glacial till.

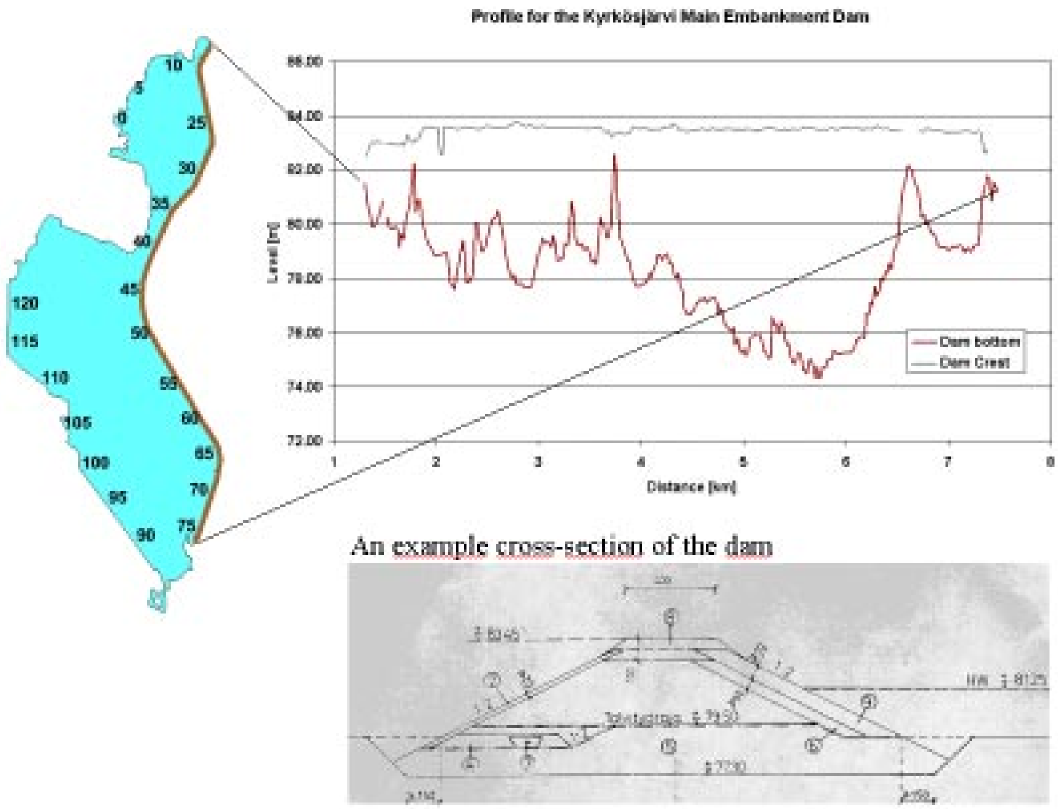

Figure 2. Kyrkösjärvi Embankment Dam

3. RISK ASSESSMENT

3.1 Public response towards dam safety issues

The sociological research on the public response towards dam safety issues was one part of the RESCDAM . It played an important role in this project by concentrating on the attitude of the public towards a risk of the Kyrkösjärvi dam-break and on the possible reactions of the people in the case of a flood caused by it. By learning the population needs in the field of security matters, the research served as one of the tools for creating the emergency action plan in the case of a Kyrkösjärvi dam-break and a better preparedness for such an accident.

The first step of the research included preparation of a questionnaire devoted to the problems of dam-breaks in general and in the case of Seinäjoki, as well as to the alarm system. The purpose of the questionnaire was to introduce the dam-break issues to the public and to learn their perception of such a risk in Seinäjoki.

At the preparation stage, it was important to construct the questions in such a way as to find out the level of public awareness on the dam-break risk problem but not to induce panic. In order to avoid unnecessary fears and stress among the public, it was decided that the questionnaire was to be accompanied by a short letter of explanation signed by the chief of the local fire brigade. It was stressed there that this questionnaire is part of a wider research and that Kyrkösjärvi is only an example dam in this study.

The questionnaire was divided into three parts concentrating on personal data, dam-safety issues and comments. One thousand copies were distributed to the households in the 2-hour dam-break flood prone area. The copy of the questionnaire was sent by post along with the explanation letter in the end of October, 1999. A free-of-charge return envelope addressed to the Finnish Environment Institute in Helsinki was also included. Until the end of December, 1999, two hundred eighty five responses were received. The results of the questionnaire analysis are available in the paper entitled "Public Response Towards Dam Safety Issues – Kyrkösjärvi Dam Pilot Project".

Several comments to this questionnaire revealed that dam-break safety issues were a new and unexpected subject to the respondents. A few persons stated that a fact that a dam could break had never crossed their minds. Nonetheless, the general risk perception among the public proved to be low.

The respondents emphasised the need for decent information on the dam safety issues. However, making a decision on the content and the amount of the information disseminated to the public creates a problem for the authorities responsible for the public information campaign. On the basis of the sociological research and the results of the international workshop on the RESCDAM project, a few conclusions were made in the subject of the public information campaign. Information presented to the public should be simple and comprehensive. It should stress the safety of a dam and simultaneously remind that the EAP has been created to make the dam even more safe. The problem of information overburden and its prolonged impact to the public should be tackled while designing the information campaign. Since people receive an enormous amount of different information daily, it is important to "pack" the information in such a way as to attract their attention. Moreover, the impact of the information received will gradually diminish with time and some people will move away or into the community. One possible solution to this problem is to create the web pages devoted to the dam-break issues and let people know regularly where to seek for the up-dated information.

On the basis of the analysis of the answers and the respondents' comments, a few recommendations for the EAP creation were made. It was recommended that a compact guide for the population on how to act in the event of a dam-break should be created. Such a guide should be in a form of a leaflet distributed to each household in the flood prone area. The other way of disseminating this information would be to include it in the local phone-book.

Other recommendation deals with the creation of an effective alarm system. In the case of a dam-break fast and effective warning of the population plays a crucial role for their safety. As the results of the questionnaire analysis reveal the traditional warning system by sirens might not be the best solution. There are a few reasons for such a statement. First, its relative ineffectiveness in the night time or when the TV or the radio is on. People fear that they might not hear the alarm sound. A second reason derives from the instruction how to act when one hears an alarm signal. Namely, people are supposed to go home where they should receive instructions by radio. The weakness of such an arrangement in the event of a dam-break is a relatively long time before the inhabitants find out what the actual reason of the alarm is. Moreover, there exists also a considerable danger that a flood might stop the energy supply which would negatively influence the effectiveness of information dissemination.

At least parts of the EAP should be presented to the local inhabitants during the public information meeting. They should be given a possibility to comment and discuss the plan. Their suggestions should be taken into consideration in the process of further development of the EAP.

3.2 The use of physical models

One part of the RESCDAM was a research carried out at the Laboratory of Water Resources at the Helsinki University of Technology. There is a separate report available on that study.

There were three goals in this part of the project: 1) human stability and manoeuvrability in flowing water, 2) permanence of buildings in flowing water and 3) roughness coefficients of forest and houses. Human stability and roughness coefficients of forest and houses were studied using physical laboratory tests. Experiments on the human stability were conducted testing full scale test persons in the 130 m long model basin equipped with towing carriage in the Helsinki University of Technology Ship Laboratory. The roughness coefficient was studied in the 50 m long fixed bed flume at the Laboratory of Water Resources using scale model forests and houses. The permanence of buildings in flowing water was based on literature.

3.3 Concept and Bases of Risk Analysis for Dams -

There is a separate report named "Concept and Bases of Risk Analysis for Dams -With an Example Application on Kyrkösjärvi Dam" by prof. Eero Slunga. In the conclusions of the report he writes:

Probabilistic risk analysis is a more rational basis for dam safety evaluation, and provides a deeper insight into the risks involved than the traditional standards-based approach. A full risk analysis provides a more comprehensive view of dam safety, in that it considers all loading conditions over the full range of loads. The analysis procedure itself should not be viewed as a replacement to traditional engineering judgement and expertise. Quite the contrary, this process depends heavily on the knowledge base of experts. Attaining an exact value of probability for dam failure is not a realistic expectation. The utility of this approach is to assess dam safety on relative basis. After having assessed the probability of failure for an existing dam, one can investigate -In relative sense - the effects that various improvement or remedial measures will have.

The concept of probabilistic risk analysis may be used for different purposes and at different levels, for example:

- at the dam design stage, to achieve a balanced design and to place the main design effort, where the uncertainties and the consequences seem the greatest;
- as a basis for decision-making when selecting among different remedial actions and upgrading for old dams within time and financial restraints;
- to relate dam engineering risk levels to acceptable risk levels established by society for other activities.

The scepticism to use the probabilistic risk analysis may result from too much emphasis on the third and most complex item above, while the benefits from applications such as the first two may be overlooked. The application example of the risk analysis of Kyrkösjärvi dam may be included in the second one of the above–mentioned items.

There is concern among practitioners that risk analyses are too subjective, in that there are no clear-cut procedures for calculating some failure probabilities, and thus there is too much reliance on expert judgement. In fact there are still many areas, where further guidance is required. Recommendations for some of the areas that need to be addressed in more detail are listed below:

- Additional refinement of quantitative analyses.
- Development of internal erosion analysis methods to be used in a risk analysis format.
- Retrospective probability of failure under static loading.
- Whether societal risk criteria should be applied on a total expected annual risk to life basis or on a specific event basis.
- The concept of average individual risk over the population risk.
- Prediction of loss of life.
- Whether upgrading of dams should have criteria applied which were as stringent as for new dams.
- Inconsistent international terminology.

4. DAM BREAK HAZARD ANALYSES

4.1 Introduction

Dam break hazard analyses (DBHA)provides information about consequences of a possible dam break for risk estimation and rescue planning. Numerical models are used in DBHA to determine the flow through a dam breach and to simulate flood propagation in the downstream valley.

Kyrkösjärvi reservoir and its embankment dam, located in the Seinäjoki City, North West Finland, were chosen as the area of the pilot project in the RESCDAM project. A numerical model for the simulation of dam breach formation was used to the determination of dam breach discharges. Propagation of flood in different breach scenarios were calculated with 1-d flow model and two 2-d models. Also the effect of different approaches to model urban areas on flood propagation were studied in the project.

The 2-d flood calculations and the study on the effect of urban areas on flood propagation were made by the partners, EDF Electricite de France (Paris) and ENEL SpA Ricerca Polo Idraulico e Strutturale (Milan). According to the preliminary sensitivity analyses the most dangerous breach location (location A) and the $HQ_{1/100}$ flood condition and mean flow condition (MQ) were chosen for DBHA calculations made by the partners EDF and ENEL. The analyses for two other breach locations (location B and C) were decided to be done with 1d-flow model. It was also decided that 1d-model is used to produce a downstream boundary condition for 2d-models. Later it was observed that the original modelling area is too small and simulations were extended to the northern area of the Seinäjoki city (area behind the railway). The simulations were first made with 1d-model by FEI and FEI provided a rating curve for the flow over the railway for the partners. Because in the simulations the flow is divided to several streams and the modelling of that area is extremely difficult it was decided to calculate the most important cases with 2d-model to the whole area. That work was done by FEI by using Telemac-2d model.

Geometry input data for the DBHA was attained from a accurate digital terrain model which was available for the project. The results of DBHA were provided as flood maps, tables, water level hydrographs and animations for rescue planning. GIS system was used to produce the flood maps and the information about buildings and people living under flood risk.

4.2 Hydrological analysis

The Hydrological Analysis in RESCDAM project are based on a HBV-hydrological model.The Kyrönjoki watershed model is a semi-distributed model with 22 sub-basins. Each sub-basin has separate precipitation, temperature and potential evaporation as input. The Kyrönjoki model has a flood-area-model which simulates water exchange between river and flood plains. Flood-area-model is simulated with shorter time steps than the main model. At every reach of river with embankments and weir the model calculates water level in river and discharge through the weir over the embankment into the flood plain. This part of the model is important during flood. This simple hydraulic model is verified against the results from a complete hydraulic model to keep up the accuracy of the simulation.

The Kyrönjoki watershed model is a conceptual model used for operational forecasting in the Finnish Environment Institute (Vehviläinen 1994). The watershed model is based on a conceptual distributed runoff model, water balance model for lake, river routing model (Muskingum and cascade reservoirs) and flood area models. The input variables for the model are daily precipitation, temperature and potential evaporation (Class A pan).

As input for Kyrkösjärvi dam-break simulation three flood situations have been simulated. Floods with return period of 20, 100 and 10 000 year have been created or determined with the operational hydrological catchment model. The method used to determine 10 000 year flood is based on precipitation with return period of 10 000 year.

More detailed information on the Hydrological analysis of Kyrkösjärvi reservoir and is given in a separate report in RESCDAM project.

4.3 Determination of the Breach Hydrographs

Determination of flow through a dam breach has lot of uncertainties. In the RESCDAM project a numerical model for erosion of a embankment dam (Huokuna 1999) was used for the determination of the discharge hydrograph. Hydrographs were also calculated by using a method in which the breach opening is increasing linearly when the duration of the breach formation and the width of the final breach opening are given. That method to calculate breach formation is available in DYX.10 flow model and it is also available in many other models, like DAMBRK model.

In the studies the breach is assumed to happen at three locations at two different hydrological conditions. The assumed dam breach locations are presented in the Figure 3. The hydrological conditions are the $HQ_{1/100}$ flood and the mean flow.

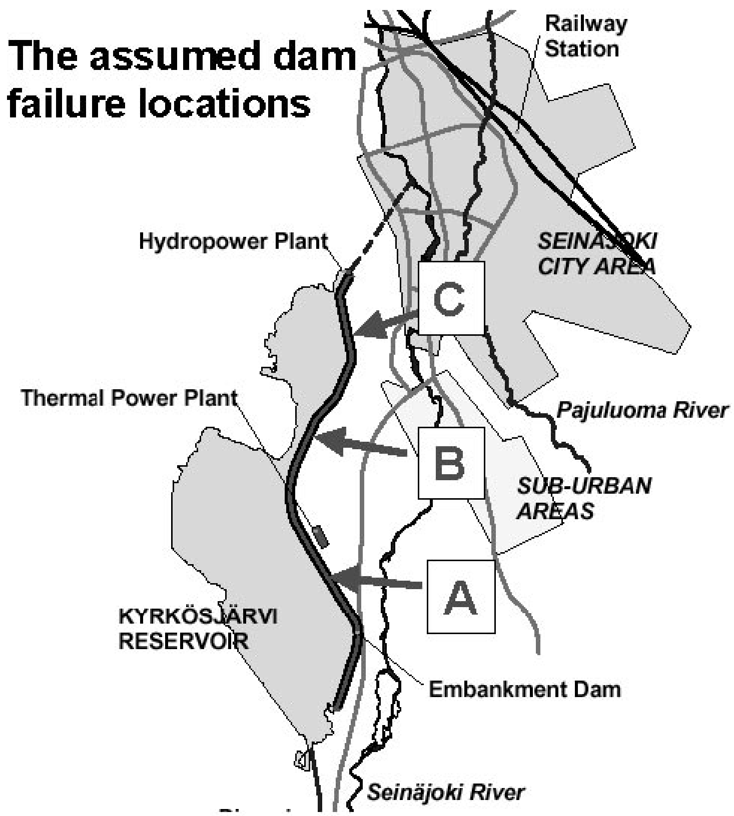

Figure 3. The locations of the assumed dam breach sites.

The location A in the Figure 3. is the most dangerous location for a possible dam break at Kyrkösjärvi dam. The dam is highest at this location (km 5.7 at Figure 2) and because of the topography of the valley downstream of the dam, the Seinäjoki downtown area could be badly flooded if the dam breaks at location A.

The breach hydrographs for the location A and $HQ_{1/100}$-flood case are presented in the Figure 4. The hydrograph calculated by the erosion model was used in the calculations made by ENEL and EDF.

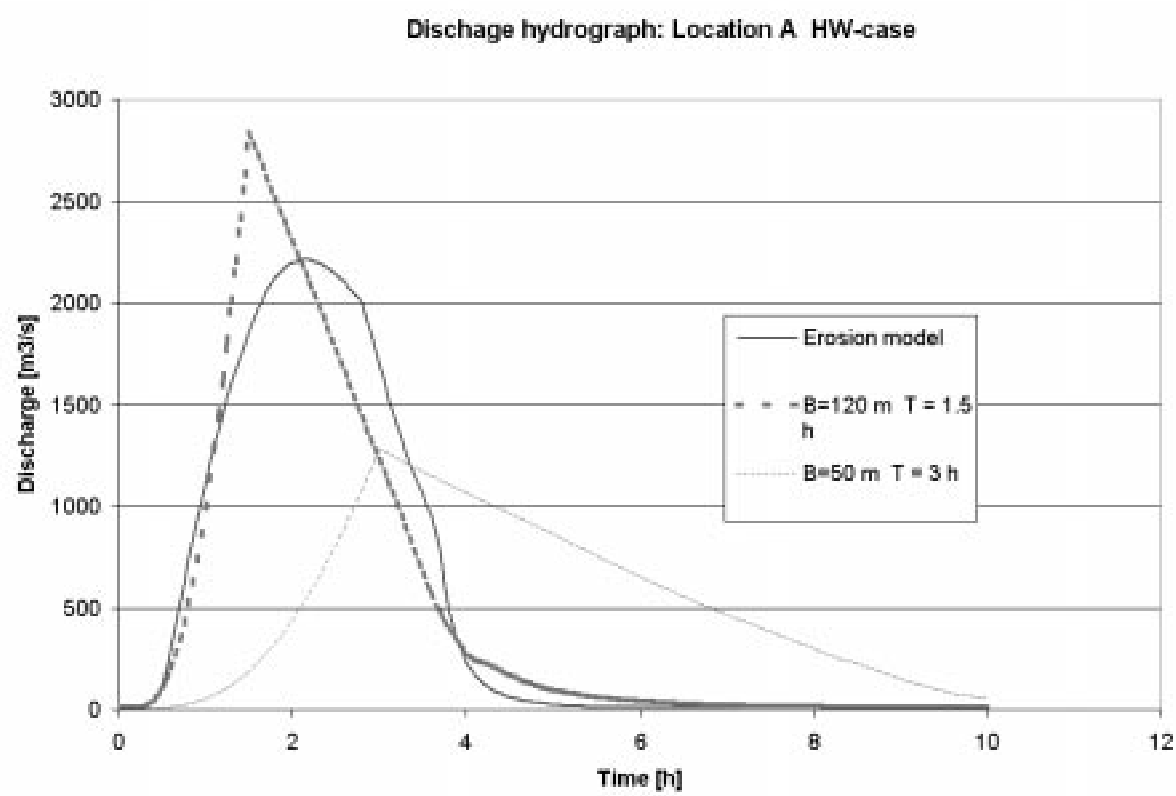

Figure 4. Breach hydrographs for the location A in the $HQ_{1/100}$ –case

4.4 1-D Flow Modelling

The 1-d modelling for the dam break hazard analysis of the Kyrkösjärvi reservoir has been done by using DYX.10-flow model. The model is based on four point implicit difference scheme developed by Danny L. Fread for DAMBRK model. During 1980's Fread's algorithm was developed further in Finland for river networks.

The 1-d flow model for the Kyrkösjärvi DBHA covers the area from Renko dam (upstream of Kyrkösjärvi Reservoir) to Kylänpää (about 30 km downstream of the reservoir). The cross-sections used in the model were taken either from a terrain model or they were measured cross-sections. There were 735 cross-sections 22 reaches and 35 junctions in the Kyrkösjärvi 1-d DBHA model (breach location A). Some of the reaches were fictive channels (flood plains, connecting channels etc.)

The following cases where studied with the 1-d model:

- -Breach location A Base flow MQ
- -Breach location A Base flow HQ $_{1/100}$
- -Breach location C Base flow MQ
- -Breach location C Base flow HQ $_{1/100}$

A constant roughness coefficient (Manning n=0.060) was used in all simulation cases. 1-d model was also used to run sensitivity analyses of the effect of the size of the breach hydrograph on the water level downstream of the dam. A more detailed description of the 1-d flow modelling in the RESCDAM-project is given in a separate report.

4.5 2-D Flow Modelling; Electricite de France

Electricite de France (EDF) used Telemac-2D model, a 2-dimensional finite element model, in the simulation. There were 81161 elements and 41086 points used in the Kyrkösjärvi model. The finite element mesh is shown in Figure 5. The mesh size ranges from 8 m to 20 m approximately. The river beds, the bridges, the roads, and the railways been highly refined to model accurately the propagation. The boundary conditions are solid boundaries everywhere except at the breach, at the entrance of river Seinäjoki, and at the output zone. In regular areas such as roads, reservoir dykes, and river beds, regular griding has been used. For other features such as railways, embankments, constraint lines have been imposed. All these features are quite visible on the mesh. A separate report is by EDF is available RESCDAM project**.**

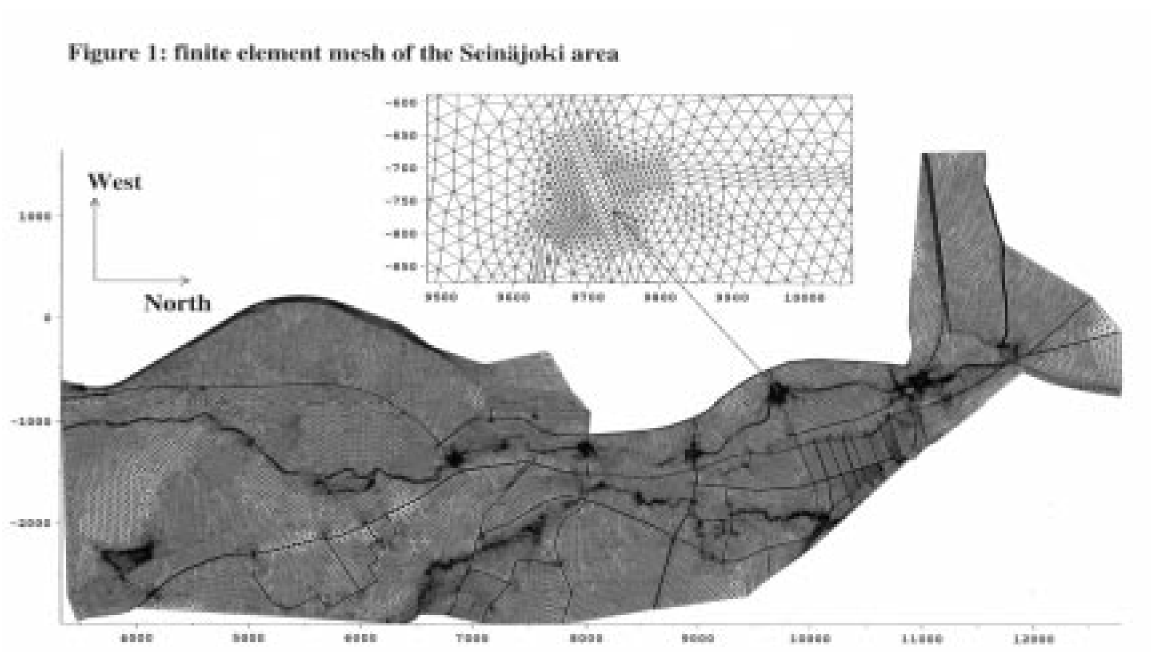

Figure 5. The Telemac-2D finite element mesh used in the Kyrkösjärvi simulations by EDF.

4.6 2-D Flow Modelling; ENEL SpA Ricerca Polo Idraulico e Strutturale

ENEL used FLOOD-2D model, a 2-dimensional finite volume model, in the simulation. FLOOD2D has been developed by Enel.Hydro Ricerca Polo Idraulico e Strutturale. The model is based on the integration of the Saint Venant equations for two-dimensional flow and it neglects the convective terms in the momentum conservation equations. The model requires as basic input only the natural ground topography and the estimated Manning's friction factors.

The two dimensional model was applied to the area below the dam including the City area of Seinäjoki. Two sets of topography were used:

1) Rectangular mesh # 10m without buildings and
2) Rectangular mesh # 10m with buildings.

The model has a total number of 446.961 grid points. Depending on the base flow condition approximately 55.000 - 70.000 cells of the model became wetted during the computations. There is a separate report about calculations in RESCDAM project by ENEL.

4.7 Urban Areas And Floating Debris

During RESCDAM project the partners EDF and ENEL developed the methods to calculate flood wave propagation on urban area. ENEL used the geometry approach to calculate flood propagation on urban area. Houses were taken into account in the model geometry. EDF used the porosity approach to model flood propagation on urban area.

There is a separate paper by EDF presenting the results of porosity approach on calculation results (Modelling urban areas in dam-break flood-wave numerical simulations). That paper was presented in the RESCDAM Seminar.

ENEL applied the geometry method for the whole calculation area. In the FLOOD2D-model he geometry was presented by a grid consisting of rectangles of 10 m times 10 m. ENEL made the calculations for MQ and $HQ_{1/100}$-cases (breach location A) by using two geometry data sets. One with buildings and another without buildings. Comparison of those results are given in the final report of RESCDAM. Generally the calculated water levels were higher in the cases in which buildings were taken in to account (maximum difference about 0.5 m). Also the propagation of a flood wave was generally a bit slower when the buildings were taken in to account. However the effect on propagation speed was not very large. The effect of buildings on damage hazard parameter (flow velocity x depth) was not very large in the case of Seinäjoki DBHA and can not be seen to be very important for the planning of emergency actions.

The effect of urban areas and floating debris in dam-break modelling is presented also in a paper by Peter Reiter (Considerations on urban areas and floating debris in dam-break flood modelling), who presented the paper in the RESCDAM seminar.

4.8 Analyses Of The Results

The following flow calculations were decided to be done by the partners EDF and ENEL:

RUN 1 Base flow in Seinäjoki River 150 m3/s + breach hydrograph for max reservoir level, roughness varies between Stricler 15 (Manning's n=0.06666) and 40 (Manning's n=0.025).

RUN 2 Base flow and breach hydrograph as in RUN 1, roughness is constant for the entire modelling area with Strickler 15 (Manning's n=0.06666).

RUN 3 MQ base flow + breach hydrograph, constant Manning n varying in the entire modelling area according to landforms and vegetation and according to the experience of EDF and ENEL.

RUN 4 HQ1/100 base flow + breach hydrograph, other conditions as in RUN 3.

RUN 5 Conditions of RUN 3 modified according to the partners choice of modelling buildings (EDF: porosity, ENEL: geometry)

RUN 6 Conditions of RUN 4 modified according to the partners choice of modelling buildings (EDF: porosity, ENEL: geometry)

The breach location was assumed to be the location A for all these calculations. The computer runs RUN 5 and RUN 6 were done for the whole area only by ENEL, while EDF used a smaller example area.

In the RESCDAM project the meaning of using the different models to simulate the same case was not to compare the computational algorithms. The meaning was to get an idea how much the results may differ depending on the models and the modellers using their own approaches. The comparison of different models and solution algorithms have been done recently in the CADAM-project.

The partners get the land use data in the 10 m x 10 m grid which was derived from the terrain model data. The computation area was divided to 6 land use areas and the modellers used their own judgement for choosing the friction factors for different areas.

In an separate report the results of calculations made by ENEL and EDF are compared together with the results of 1-d simulations. The water level comparison is done on different locations downstream of the dam. The progression of the dam break flood is also compared on maps.

According to the comparison the results calculated by EDF and ENEL seems to be relatively close to each other. There is more difference between the results of the 2-dimensional models and the results of the 1-dimensional model. The 1-dimensional simulations were made only for constant Manning’s n (n=0.06) and this is explanation for some differences. However, in the case of very complicated topography, like in the Seinäjoki case, the use of 2-dimensional model seem to be more reasonable. The use of 1-dimensional model needs a lot of experience because the cross-sections have to be put on right locations. The use of 2-dimensional models are more straightforward.

4.9 DBHA Results For Rescue Actions

The first DBHA results for emergency action planning where based on 1-d model results and results by EDF and ENEL and only the breach location A was considered. Later the calculations for the locations A, B and C was made by FEI using 2-dimensional Telemac-2d model. The original final element mesh greated by EDF was extended to the area north from the railway station when the terrain model data for that area was available. ..Those results were the final results used for emergency action planning of the Kyrkösjärvi Reservoir. There is a separate report available in RESCDAM on 2-d calculations made by the Finnish Environment Institute.

The results of DBHA for rescue actions consists of inundation maps, water depth maps, hazard parameter maps as well as water level and velocity hydrographs and tables. In the RESCDAM project the results were transferred to GIS-system and different results could be analysed together with the database information of buildings and inhabitants. That information was used to get damage and LOL-estimations.

5. DEVELOPMENT OF RESCUE ACTIONS

Developing of rescue services concerning waterbody dams can be compared to the corresponding planning obligation of nuclear power plants. These types of accidents are very unlikely, but if an accident does occur consequences can be very serious.

The main area in the preparation of an emergency action plan of a dam must be in organising the warning, alerting and evacuation activities. Consequences of a dam failure as well as conditions following the accident are so difficult for rescuing, that evacuation before the arrival of flood must be the main approach. In addition to the warning and evacuation of population also the automatical dam monitoring and notification of a dam break must furthermore be developed. In the seminar of RESCDAM project it was very unambiguously stated, that the risk of loosing human lives is influenced strongly by the time of the notification of a dam failure. If the failure is not noticed early enough, the benefit gained from public warning sirens is lost and people do not have enough time to escape from the flood area.

Failure risk of a dam should be taken into account also in laws controlling building construction. Assembly rooms, hospitals, maintenance institution and corrective institutions should not be built in the danger area of the dam, because the evacuation of such buildings is very problematic in accident situation. Buildings in the danger area should be built so that dam failure will not endanger people living in the buildings.

When preparing for a dam failure, it is especially important to consider human behaviour in crises situations. Studies of this topic show that people do not always believe in the reality of warnings. Home is “the sanctuary” for people and leaving home is difficult. In the planning and implementation of rescue operations the compliance of population with the warning and evacuation instructions must always be ensured with vehicles with loudspeaker system rotating in the danger area and rescue units going from house to house.

Compliance with warnings and instructions given by authorities can be facilitated with advance bulletin that is distributed to people in the danger area in advance. In Finland this kind of advance informing has primarily been recommended, but not requested. However advance informing has significant meaning in the success of rescue operations and thus it should be determined.

The hazard risk assessment of the dam with inundation maps about the flood situation after the failure prepared along with the assessment are almost merely the basis for the emergency action plan prepared by rescue services. The needs of rescue services must be noticed when presenting the flood information and inundation maps. The availability of maps in digital and paper form should be further developed. Digital maps were developed during RESCDAM project. These maps can be applied to all dams and results and reactions were very positive.

5.1 Emergency Action Plan For The Kyrkösjärvi Dam

Emergency action plan of Kyrkösjärvi reservoir is based on a dam-break flood analysis (Chapter 4). The dam failure may in the worst case cause a flood that covers over 10 km^2 of population centre and over 1300 buildings there. The flood will wet app. 420 00 square metres of built floor area in buildings with 0-2 floors. As a whole there is nearly 800 000 square metres of built floor area in the flood area, about half of which will stay below the water level. Dam failure will affect directly or indirectly lives of many thousands of people. Flood will significantly damage the distribution of electricity and energy, water system, road network, sewage and entrepreneurship and servicing in the city of Seinäjoki. Situation is then catastrophic in Seinäjoki and the resources of the city of Seinäjoki are not adequate considering the situation. Danger of losing human lives depends mainly on, how fast the failure is noticed.

The emergency action plan of the Kyrkösjärvi dam is mainly prepared according to the existing Finnish Dam Safety Code of Practice. However among other things the planning of warning of population is emphasised so that evacuation is really materialised. Respectively more attention is paid among other things to instructions of emergency response centre, medical rescue services and informing as well as to the organising of the maintenance of evacuated population. One of the purposes of the preparation of the plan was also to facilitate the maintenance and updating of the plan.

Sufficient guarantee to the success of rescue operation must be taken into account when preparing the emergency action plan. The basis for the planning should be the worst possible accident situation. In the emergency action plan of Kyrkösjärvi the dam failure will occur during natural flood in the waterbody. Dam will fail without warning in the worst possible place. Also possible other failure situation was considered.

Preparation of emergency action plan is almost entirely based on the flood information from the hazard risk assessment of the dam. Flood information must be prepared in such a form that rescue services is able to interpret and process it to it’s own use. During the project inundation maps produced with MicroStation- and TeleMac-software from 3D-terrain model were transferred to MapInfo- software used by rescue authorities. Actual plans and maps of rescue services were then prepared with MapInfo-software.

Levels according to geographic co-ordinates were prepared from digital inundation maps. These levels can be used together with different kind of map material and plans prepared by rescue authorities. There is an example of the use of inundation maps in Graph 5.

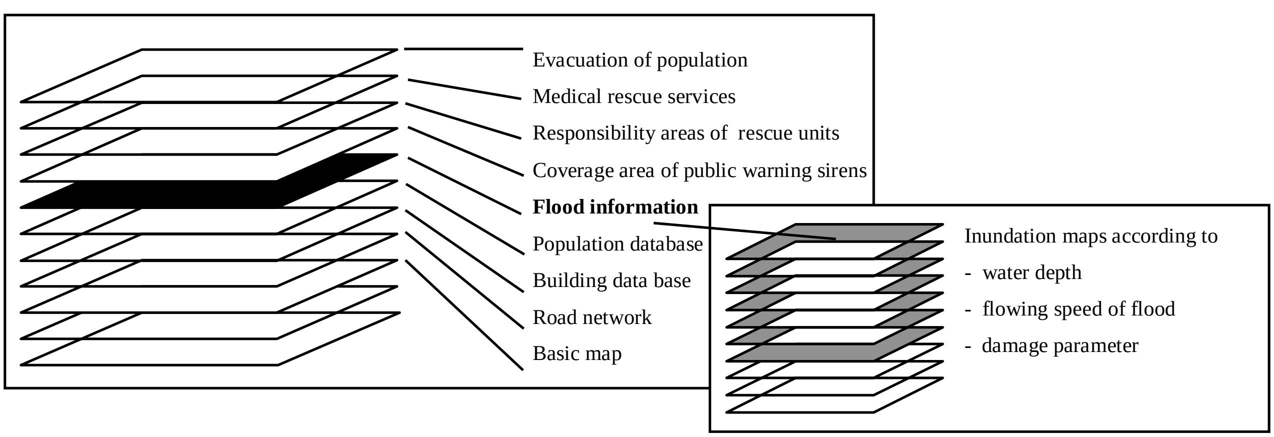

Figure 6 **.**Usage principles of material needed in rescue operations

Emergency action plans prepared earlier in Finland are mainly prepared on paper maps. MapInfo-software (or some other software processing geographical information = GIS) enables the processing of information. Several different kinds of database combined to co-ordinates can be used to help planning. This kind of database is for example population, road network and building register as well as different maps. These considerable improve the quality of planning and facilitate the preparing and updating of plans.

Earlier digital map material was not available in planning of rescue services. The results of RESCDAM project are very significant in this area.

5.2 Recommendations to update the Finnish Dam Safety Code of Practice

Acts, decrees and instructions concerning dam safety and emergency action plans of the dams today are quite sufficient and they form a good basis for the maintenance of dam safety. Recommendations for the development of the Finnish Dam Safety Code of Practice presented in Appendix XX do not change the present planning practice very much. The recommended changes have a great influence on the practical implementation of dam safety.

Recommended changes to the safety monitoring of the dam would influence among other things the periodicity of monitoring. At the moment the risk factor to people, property and environment do not have much influence the content of monitoring program. This means that the periodicity of every P-dam is almost the same according to the code. In the future the operational conditions of rescue services could influence the monitoring program of the dam. Recommendations include some changes to the content of periodic inspections and repairing of monitored deficiencies.

The most significant change influencing the dam safety concerns the informing. At the moment informing about the dam failure risk and about the prepared emergency action plan is directed to population in the danger area. The word “should” gives the dam owner and authorities a lot of possibilities and has normally lead to a situation that there is no informing at all. During RESCDAM project it was observed that advance informing has a great meaning when warning the population. In the recommendation it is presented that population in the danger area must be informed about the emergency action plan and about the risk of a dam failure.

The renovation of the rescue services act and decree as well as the regulations and instructions passed based on the act and decree influence the most on the dam safety code. These parts of the recommendation are mainly about updating the code.

6. CONLUSIONS

In the RESCDAM project the main focus was put on the development of rescue actions based on the risk assessment and dam-break hazard analysis. The experience and achievements (developments) of the International Commission on Large Dams was taken into account in the project. The developments in this field from the USA, Canada, Australia, Norway and Sweden were also considered while referring to the state of art in risk analysis and using the best available practice in calculating the risk of the project example dam – the Kyrkösjärvi dam.

For the Kyrkösjärvi dam, the detailed study was performed to calculate its risk as good as reasonably practical. The calculations were divided into two parts. One dealt with the probability of a failure and the other one with the consequences of such a failure. The detailed study of the risk identification takes into account all the characteristics of a dam and its foundation as well as the history of the dam's behaviour during its use. The detailed study is recommended to trace all the possible hazards which can lead to any kind of a dam failure. The failure in this study was defined in terms of a complete breach followed by a significant release of water from the reservoir. The detailed risk analysis including the effects of a dam failure provides a tool for the decision-makers while selecting among different remedial actions and upgrading for all dams within time and financial restraints. It provided also information and basis for the emergency action plan of the dam in question.

On the basis of the project findings the following recommendations for the particular areas of development were made:

- Additional refinement of quantitative analyses.
- Development of internal erosion analysis methods to be used in a risk analysis format.
- Retrospective probability of failure under static loading.
- Whether societal risk criteria should be applied on a total expected annual risk to life basis or on a specific event basis.
- The concept of average individual risk over the population risk.
- Prediction of loss of life.
- Whether upgrading of dams should have criteria applied which were as stringent as for new dams.
- Inconsistent international terminology.

Dam break hazard analyses (DBHA)provides information about consequences of a possible dam break for risk estimation and rescue planning. Numerical models are used in DBHA to determine the flow through a dam breach and to simulate flood propagation in the downstream valley. In the RESCDAM-project several modelling approaches have been used in the flow modelling. The results shows that with careful modelling and accurate data the results of different modelling approaches may be relatively close each other. However, there is a lot of uncertainties in the modelling and specially in the one dimensional modelling where the modeller can effect dramatically on the results by selecting the locations of cross-sections carelessly.

The determination of flow hydrographs through the dam breach opening is crucial for the results of DBHA. In the RESCDAM project a numerical erosion model for the breach of a embankment dam has been used to define the flow hydrographs. There is a lot of uncertainties in the determination of breach hydrograph and sensitivity analyses have to be committed to ensure the results. The debris flow, clogging of bridges and other structures and erosion of flooded areas are also causing uncertainties in the flood simulation and that uncertainty has to be taken into account.

In the RESCDAM project special methods have been tested to model the flow in urban areas. The results of EDF, which used porosity approach, and ENEL, which used geometry approach, are promising and they gives good basis for further development.

The results of DBHA for rescue actions consists of inundation maps, water depth maps, hazard parameter maps as well as water level and velocity hydrographs and tables. It is important that the results of DBHA are presented in the way that they can be used efficiently in the dam break risk estimation and rescue planning. The use of GIS is essential in that purpose.

Some recommendations for further research topics based on the DBHA of the RESCDAM project:

- -determination of breach formation
- -determination of roughness coefficients
- -the effect of debris flow and urban areas in DBHA

After the RESCDAM project is completed, it is planned to organise an emergency exercise of the Kyrkösjärvi emergency action plan. In connection to this happening the public will receive more information about actions during the possible dam break flood.

After the exercise the improved version of the emergency plan should be presented to the public and an information bulletin including instructions how to behave in the case of a flood caused by a dam failure should be distributed to the population in the flood prone area.

If all the above mentioned actions are completed, it will be possible to perform a new sociological research to study with the help of a new questionnaire the impact of the information given to the public on the potential behaviour patterns in the case of a flood caused by a dam failure. After this study it can be studied/checked what impact these changes have/might have on the estimated loss of life in the case of a Kyrkösjärvi dam failure.

Referenses:

The text in this paper is based on RESCDAM final report which will be available in July 2001. The report,and the references used in the text, will be available thru internet and in the form of a CD.

B-7

GUIDELINES FOR ASSIGNING HAZARD POTENTIAL CLASSIFICATIONS TO DAMS

by

Alton P. Davis, Jr., P.E. - Independent Consultant

presented at the

FEMA/USDA WORKSHOP ON ISSUES, RESOLUTIONS, AND RESEARCH NEEDS RELATED TO DAM FAILURE

“FEMA GUIDELINES FOR DAM SAFETY:

HAZARD POTENTIAL CLASSIFICATION SYSTEM

FOR DAMS”

FEMA Mitigation Directorate: FEMA No. 333
October 1998

HAZARD POTENTIAL CLASSIFICATIONS

- Low Hazard Potential
- Significant Hazard Potential
- High Hazard Potential

LOW HAZARD POTENTIAL

"Dams assigned the Low Hazard Potential classification are those where failure or mis-operation results in no probable loss of human life and low economic losses, low environmental damage, and no significant disruption of lifeline facilities. Losses are principally limited to the owner's property."

SIGNIFICANT HAZARD POTENTIAL

"Dams assigned the Significant Hazard Potential classification are those dams where failure or mis-operation results in no probable loss of human life but can cause economic loss, environmental damage, disruption of lifeline facilities, or can impact other concerns."

HIGH HAZARD POTENTIAL

"Dams assigned the High Hazard Potential classification are those where failure or mis-operation will probably cause loss of one or more human lives."

HIGH HAZARD POTENTIAL

- Loss of One or More Human Lives
- Probable
  - Likely to Occur
  - Reasonable / Realistic Scenario
- Temporary Occupancy
- High Use Areas

CONSEQUENCE BASED SYSTEM

- Adverse Impacts
- Incremental Impacts
- Immediate Impacts
- Current Conditions
- No Allowance for Evacuation

SELECTING HAZARD POTENTIAL CLASSIFICATION

- Presumptive (Phase 1)
- Incremental Hazard Assessment (Phase 2)
- Risk Based Assessment (Phase 3 - Refinement)

PRESUMPTIVE

- Obvious
- Readily Available Information
- Maps
- Site Reconnaissance

INCREMENTAL HAZARD ASSESSMENT

- Detailed Dam Break Studies
- FEMA Publication No. 94

  "Federal Guidelines For Dam Safety: Selecting & Accommodating Inflow Design Floods for Dams." October 1998

- Defining Incremental Impacts

RISK BASED ASSESSMENT

- Refinement
- Limited to Loss of Human Life Issues
- Tools, Procedures, Knowledge, Experience
- No Set Procedure Currently Accepted
- Proposed Approach in Draft Guideline

GUIDELINE GOALS

• Repeatable Classification

• Better Understanding by Public

• Standard Terminology

• Periodic Review of Classification

• Record of Decision

FACTORS AFFECTING CLASSIFICATION

• Loss of Human Life

• Economic Losses

• Lifeline Disruption

• Environmental Damage

LIFE LOSS CONSIDERATIONS

- Designated Day Use / Recreation Areas
- Non-Permanent Structures
- Overnight Recreation Facilities
- Roads / Highways
- Permanent Structures

- Occasional Downstream Recreationist
- Immediate Life Loss
- No Evacuation

Designated Day Use and Recreation Areas

- Golf Courses
- Boating, Rafting, and Kayaking River Sections
- Swimming, Wading, and Beach Areas
- Special Regulation Fisheries: Gold Medal, Wild Trout, Catch and Release
- Parks and Picnic Areas
- Sporting Events
- Scenic Attractions

Permanent Structures

- Single family homes on fixed (masonry) foundations
- Mobile homes on temporary foundations or single family homes on stilts
- Public buildings such as prisons, hospitals, and schools
- Motels
- Houses of worship
- Condo and apartment complexes
- Commercial and industrial facilities
- Emergency response facilities such as fire, police and public works

Mis-Operation

- Mis-operation of a dam or its appurtenant works is the sudden accidental and/or non-scheduled operation of a water retaining element of a dam that releases stored water to the downstream channel in an uncontrolled manner. Mis-operation also includes the deliberate release of floodwater because of an emergency situation, but without the issuance of a timely evacuation warning to the downstream interests (Ref. 12 Nigeria, Ref. 13 Dominican Republic). Mis-operation also includes the inability to operate a gate in an emergency, a condition that could lead to overtopping of the dam and potential breach. Mis-operation does not include structural failure of the dam.

Upstream Damage Potential

- It is unlikely that loss of human life will occur in the reservoir area due to dam failure. A possible exception would be during a sunny day breach event when boaters or swimmers could be drawn into the breach. These possibilities are covered under the concept of the occasional hiker or fisherman as outlined in FEMA Publication No. 333, and are not considered to represent probable loss of human life for purposes of assigning hazard potential classifications. If overnight sleeping on boats at mariners is allowed, the potential for loss of life should be evaluated in accordance with Appendix E.

Lifeline Disruption

- ASCE defines lifelines as transportation systems [highways, airports, rail lines, waterways, ports and harbor facilities] and utility systems [electric power plants, gas and liquid fuel pipelines, telecommunication systems, water supply and waste water treatment facilities].
- For the purpose of this guideline, lifeline facilities are categorized in two groups: “Easy to Restore” and “Difficult to Restore”. Easy to restore lifeline facilities are those that generally can be returned to service in seven days or less or for which there are alternative resources or routes available. Difficult to restore lifeline facilities are those that will take more then seven days to recover operation or for which there are no alternative resources available.

Lifeline Disruption

Easy to Restore in Seven Days or Less

- Transportation Infrastructure
- Emergency Shelters
- Fuel Supplies
- Radio and Telephone Centers
- Municipal Services Facilities
- Fiber Optic/Phone Trunk Lines
- Water and Gas Pipelines
- Emergency Response Services
- Evacuation Routes

Lifeline Disruption

Difficult to Restore in Seven Days or Less

- Potable Water Treatment Facilities
- Wastewater Treatment Facilities
- Power Generation Facilities
- Navigation Facilities
- Communication Facilities
- Fire and Police
- Medical Facilities
- Railroads
- Levies/Flood Control Dams
- Power Transmission Lines

ECONOMIC LOSSES

- Direct Physical Property Damage
  - Cleanup Costs
  - Repair Costs
  - Replacement Costs
- Exclude Owner Economic Losses
- Include Loss of Business Income
  - Commercial
  - Recreation
  - Replacement Water Supply
- Dollar Breakpoint ($1,000,000 Incremental 2001 $)

Economics Losses

- Residential structures
- Industrial buildings
- Commercial and Public buildings
- Railroads
- Main highways
- Bridges on main highways and on Township and County roads
- Disruption of utilities (electric, sewer, municipal and agricultural water supply)
- Economic loss due to lost recreation or damage to recreational facilities upstream and downstream of the dam
- Loss of commercial navigation
- Agricultural land and buildings
- Costs of alternative transportation or routings

ENVIRONMENTAL DAMAGE

- Habitat and Wetlands
- Toxic and Radiological Waste
- Mine Waste
- Animal Waste

Other Concerns

- National security issues (dams upstream of military facilities)
- Non-jurisdictional dams (No federal or state oversight)
- Native American sites
- Archeological and historic sites
- Facilities not easily evacuated (Assisted living establishments, prisons, hospitals)

RISK BASED ASSESSMENT

- The purpose of this procedure is to differentiate between High Hazard Potential and **not** High Hazard potential. Using the procedures outlined in this Appendix, if the calculated probable loss of human life exceeds 0.33, the dam should be classified as High Hazard Potential.

RISK BASED ASSESSMENT

- For estimating incremental life loss only
- Presumptive and incremental hazard methods inadequate
- When 2-feet incremental flooding criteria inadequate
- Use when human occupancy is seasonal
- Based on empirical life loss data
- Use to differentiate between High Hazard Potential and Not High Hazard Potential

RISK BASED ASSESSMENT

1. Assume failure scenario
2. Define incrementally impacted areas
3. Select time sequence (season, day of week, time of day, etc.)
4. Estimate number of people at risk for time sequence
5. Select empirical fatality rates
6. Compute probability of zero fatalities
7. Determine (time sequence factor)*(zero fatality probability)
8. Add values for all time sequences in 7 above
9. Compute 1.0 minus total time sequence values
10. If result in Step 9 is >0.33, then classify as High Hazard Potential

RISK BASSED ASSESSMENT

- **Reference:**

"A Procedure for Estimating Loss of Life Caused by Dam Failure"

Department of the Interior, Bureau of Reclamation,
DSO-99-06 September 1999

by Wayne J. Graham, P.E.

GUIDELINE PROCESS

- Final Draft to ICODS February 6, 2001
- Review and Comment by ICODS
- Peer Review by ASDSO, USCOLD, and ASCE
- FEMA Issue as Guideline

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* Developed final draft

B-8

Embankment Dam Failure Analysis
State Assessment Criteria, Issues and Experience
Northeastern United States

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Dam Safety Section

The Northeast Region of the Association of State Dam Safety Officials includes the states of Connecticut, Delaware, Maine, Maryland, Massachusetts, New Hampshire, New Jersey, New York, Pennsylvania, Rhode Island, and Vermont.

Dams within the region vary in size with a few large dams and many small dams. Regardless of size, many of the dams in the region are in close proximity to developed areas. It is not uncommon to find 15-foot dams that are rated as high hazard structures. Some states within this region find themselves regulating detention basins due to the provisions of their state dam safety laws. With developers trying to maximize profit, we often find 15 to 20 foot high embankment dams in the middle of residential developments.

Additionally, many of the embankment dams were constructed over 100 years ago using combinations of cyclopean concrete, masonry, concrete core walls and earth. This sometimes presents unique conditions for modeling dam failures.

State Assessment Criteria (Current Practices)

For the purpose of preparing this paper, a survey of states within the Northeast Region was conducted to determine current practices in performing dam failure analysis. Although response to the survey was low, those that responded are representative of the procedures which are used in the Northeast Region.

Generally within the Northeastern States, it is a requirement that the dam owner obtain the services of a licensed professional engineer to undertake a dam failure analysis. Analyses are performed for the purpose of determining hazard classifications, spillway design floods and for establishing inundation areas for use in Emergency Action Plans. Occasionally, state engineers will perform their own dam failure analysis. New Jersey for example will perform dam failure analysis on dams owned by the State Divisions of Parks and Forestry and Fish and Wildlife when undertaking preliminary engineering or establishing inundation areas for Emergency Action Plans.

It is common that dam failure analysis be performed for all proposed dam structures in order to establish a hazard classification for the proposed dam. Additionally, dam failure analysis is required for all high and significant hazard dams in order to establish the inundation maps for the required Emergency Action Plan. Dam failure analysis may be required to be performed on low hazard dams on a case by case basis. Generally, when an inspection report identifies that development has occurred downstream of a dam that may increase the hazard classification, a state dam safety office may require that a dam failure analysis be performed to identify the inundation areas and therefore assign an appropriate hazard classification. Changes within a watershed downstream or upstream of a high or significant hazard dam may warrant revisiting the dam failure analysis in order to refine inundation limits. Generally, this would be identified to be necessary as part of a formal inspection being undertaken on the dam.

The most common method of undertaking a dam failure analysis is to utilize the US Army Corp of Engineers Flood Hydrograph Package (HEC-1) to establish dam breach discharges. For the purpose of establishing downstream flooding limits, output data from the HEC-1 is utilized to develop a back water analysis using the US Army Corps of Engineers River Analysis System (HEC-RAS) to establish water surface elevations. It is estimated that approximately 90% of all dam failure analysis being completed in the region is done with this method.

Many of the states accept the National Weather Service's Dam Break Flood Forecasting Model (DAMBRK). However, due to the sensitivity of the DAMBRK model and the manipulation of the input necessary to get the program to run (particularly on small dams), some states reportedly try to avoid using this model except on large dams. Some states reported using the Flood Wave Model (FLDWAV). No state reported any difficulties with the FLDWAV model, however, it was the general consensus that limited information and training has been made available for the FLDWAV model. Other models that were reported to be accepted by the states were the NWS Simplified DAMBRK Model, the NRCS's TR-61, WSP2 Hydraulics, and the TR-66, Simplified Dam Breach Routing Procedure.

Pennsylvania reported that they have compared the NWS DAMBRK model and the HEC-1 model on specific projects in the past. The results showed that the two models give similar outflows, but they have noticed that the NWS model attenuates the downstream flood results quicker than that of the HEC-1 model.

For breach parameters, it is recommended that the engineers performing the analysis utilize a range of breach parameters such as those recommended by the Federal Energy Regulatory Commission (FERC). In order to achieve results that are conservative, it is recommended that the upper level of the average breach width and that the lower end of the range of breach times be used so that the resultant breach wave is a worse case scenario. The breach should be assumed to be at the location where the dam height is the greatest and the breach should occur at the peak of the design storm event. Engineers are encouraged to perform sensitivity analysis on their breach parameters to determine the reasonableness of their assumptions.

Maryland also recommends that the equations developed by Froelich in 1987 (revised 1995) for determining average breach width and time of failure be used and the results compared with the results of the breach analysis using the recommended range of breach widths and times.

Froelich Equations:

$B = 9.5Ko(VsH)^{0.25}$ $\qquad$ $T_f = 0.59(Vs^{0.47})/(H^{0.91})$

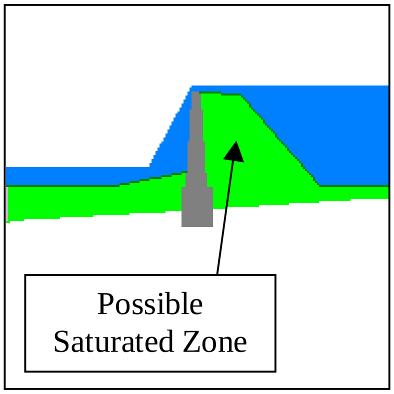
where:

B = average breach width (ft)
T_f = time of failure (hrs)
Ko = 0.7 for piping and 1.0 for overtopping failure
Vs = storage volume (ac-ft)
H = height (ft) of water over breach bottom

Issues with Dam Failure Analysis

Core walls and concrete or masonry faces

In the 1920's and 1930's, many dams were built with a concrete or masonry core walls or with concrete or masonry downstream or upstream faces. These walls within earthen dams have been an issue of discussion when it comes to determine breach parameters to be used in a dam breach analysis.



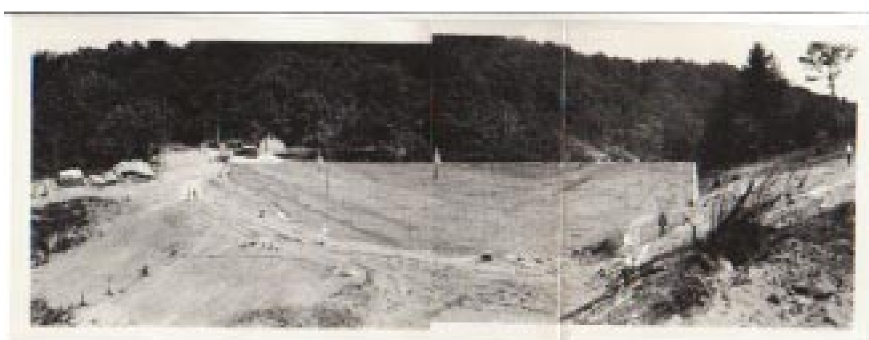
When a dam with a core wall overtops, the downstream face of the dam will erode away. However, the top of the dam will only be able to erode down to the elevation of the top of the core wall. This will leave the core wall to provide the structural stability in the dam. The remaining embankment material behind the core wall may be saturated and would likely "flow" if the core wall were to fail. Generally core walls were designed as an impervious barrier to reduce seepage and were not designed to provide structural stability. Since the downstream fill material has eroded away and the fill material behind the structurally questionable core wall is saturated, it could be recommended that breach parameters similar to those of a concrete gravity dam be used with a very fast to nearly instantaneous time of failure. The width should be established based upon the procedures used to construct the core wall (monolithic vs. continuous pour). It could be recommended that the downstream face of the dam had eroded away on the rising limb of the hydrograph and that the near instantaneous failure of the concrete core wall would occur at the peak of the design storm.

A similar situation exists in dams that were constructed with a masonry face on either the upstream or downstream side of the dam or used as a core wall in the dam. In cases where the masonry wall is on the downstream face, one could expect if the top layer of masonry were to fail and erosion of the earth portion of the dam commence, the masonry would unravel as the earth eroded and typical earth dam breach parameters could be used. A masonry wall used in the dam as a core wall and on the upstream face of a dam could be expected to act similar to the core wall situation presented above. In the case with the wall on the upstream face, it would be more likely to be a near instantaneous since there would be no remaining earth fill material upstream or downstream of the wall. It couldn't be expected to fail typical of a designed masonry dam since the wall was not designed to stand alone as a masonry dam would have been.

Overtopping of the Washington Forge Pond Dam in August 2000. Dam has masonry downstream face.

Core wall example: West Branch Reservoir Dam, Bridgewater, New Jersey

The West Branch Reservoir Dam is a 39 foot high, 330 foot long, high hazard dam constructed in 1929. The dam is an earthen dam with a concrete core wall. The core wall is 18 inches wide at the top with a 1H:20V batter on both sides resulting in a base width of approximately 5.5 feet resting on bedrock (not keyed). The core wall in this case was constructed as a continuous pour over 6 days.



Construction of core wall at West Branch Reservoir Dam

On August 27, 1971, the West Branch Reservoir Dam was overtopped as a result of Hurricane Doria. As a result of the overtopping, the downstream fill material was

washed away exposing the concrete core wall. Fortunately the dam did not fail however, a quick failure of this dam could have been catastrophic. The exposed area of the core wall was 22 feet deep and 48 feet wide.

Many have credited the core wall with saving the dam from failure. And they may have rightfully done so. Without the core, the earth fill most likely would have continued to erode to the lakeside of the dam resulting in a breach of the embankment. However, one has to question that if the core wall had not been able to withstand the water pressure, would the failure been more catastrophic than a failure of an earth dam without a core wall? This is an important point to consider in developing the inundation maps for a dam with a core wall.

The consultant for the Army Corps of Engineers performed a dam failure analysis of this dam as part of the Phase 1 inspection report. The consultant use a trapezoidal shaped breach with 45-degree side slopes, 190 feet wide at the base (original reservoir floor elevation). Six hours was chosen as the time for the breach to form to its maximum size. The start of breaching was modeled to begin when the dam first overtops.

Masonry Wall Example: Edison Pond Dam, Sparta, New Jersey

The Edison Pond Dam is a small dam in northern New Jersey. The dam is an earthen dam approximately 15 feet in height and a portion of the dam possesses a masonry wall along the upstream face of the dam. There is no history on the construction of this dam. The dam was in a serious state of disrepair. In August 2000, a storm dropped between 14 and 18 inches of rain on the Edison Pond Dam watershed resulting in the failure of the dam. It is uncertain whether the dam failed as a result of piping or overtopping or a combination of both. The earth material downstream of the masonry wall was eroded in a very narrow breach (3 to 4 foot channel through the embankment) and the masonry wall was undermined leading one to believe that piping may have attributed to the failure. There was no clear indication that overtopping occurred, however, documentation indicates that the normal flow was known to flow over the crest of the dam at this location at times when the principal spillway was clogged by beavers. The wall, however, did not fail allowing for a slow release of the lake storage. The downstream dam survived minimal overtopping during the storm, however, a total failure of the

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).

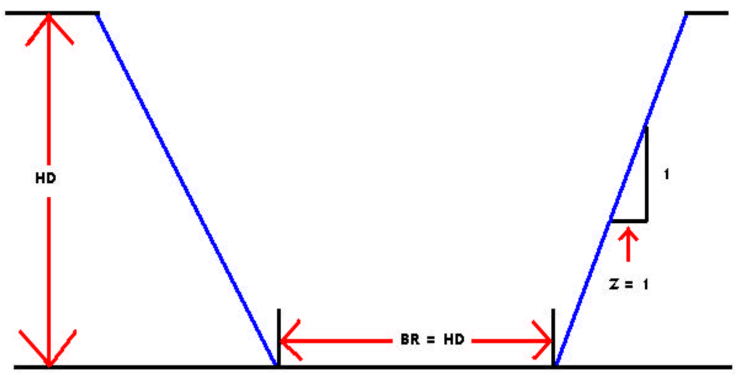
The breach parameters recommended in the Georgia Safe Dams Program's Engineering Guidelines mirror the Breach Parameters recommended by FERC and have been modified by our field observations of numerous dam failures.

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

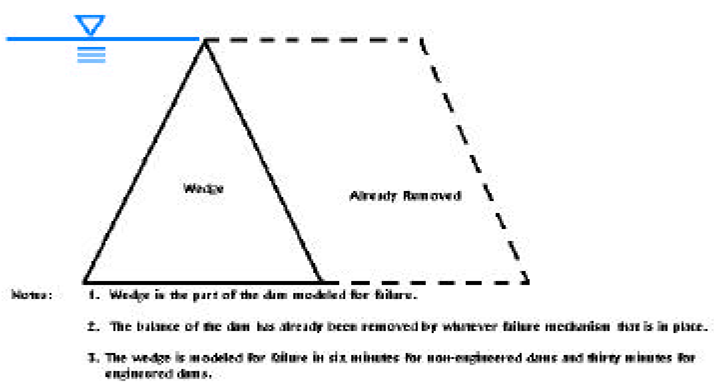
Table 2 – Breach Parameters
Definitions

- 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.

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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.

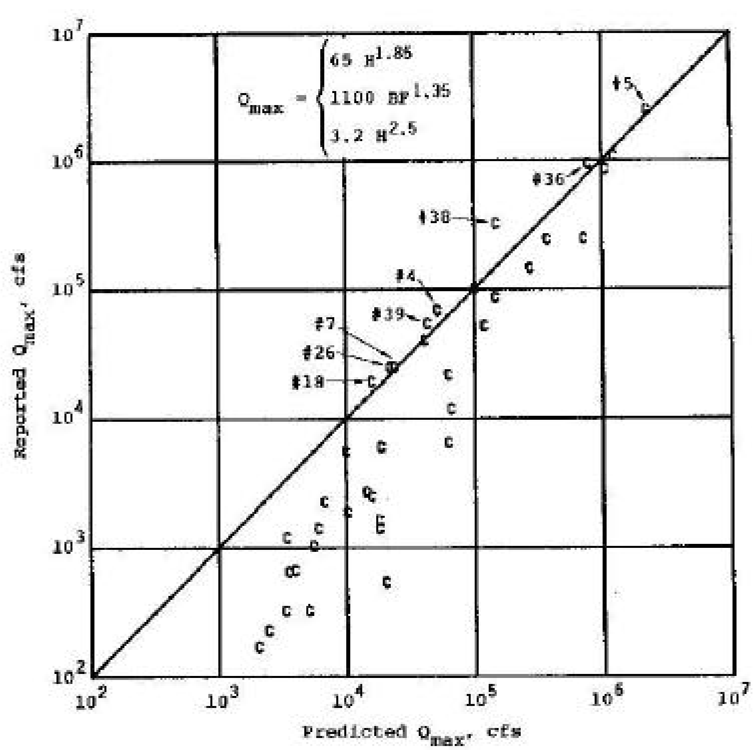


Figure 2 – Comparison of Predicted vs. Reported Qmax for 39 site data set

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.

B-13

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

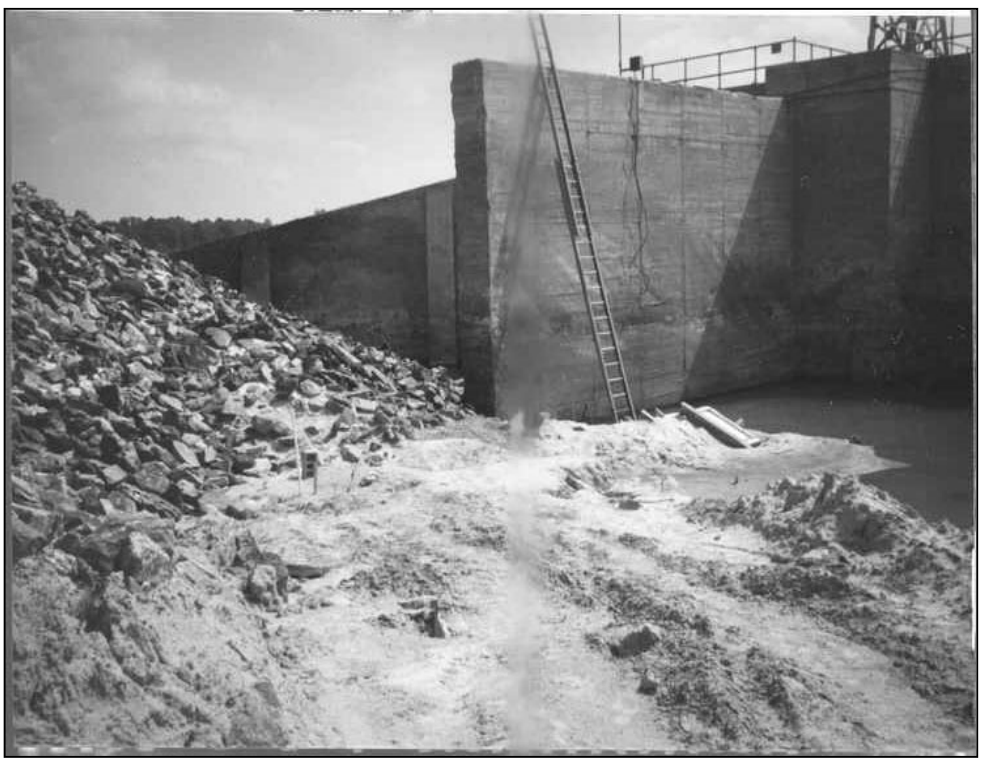
- **End of Construction--FS > 1.3**
- **Sudden Draw Down From Maximum Pool------------------------------FS > 1.1**
- **Sudden Draw Down From Spillway Crest or Top of Gates----------FS > 1.2**
- **Steady Seepage with Maximum Storage Pool---------------------------FS > 1.5**
- **Steady Seepage With Surcharge 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.

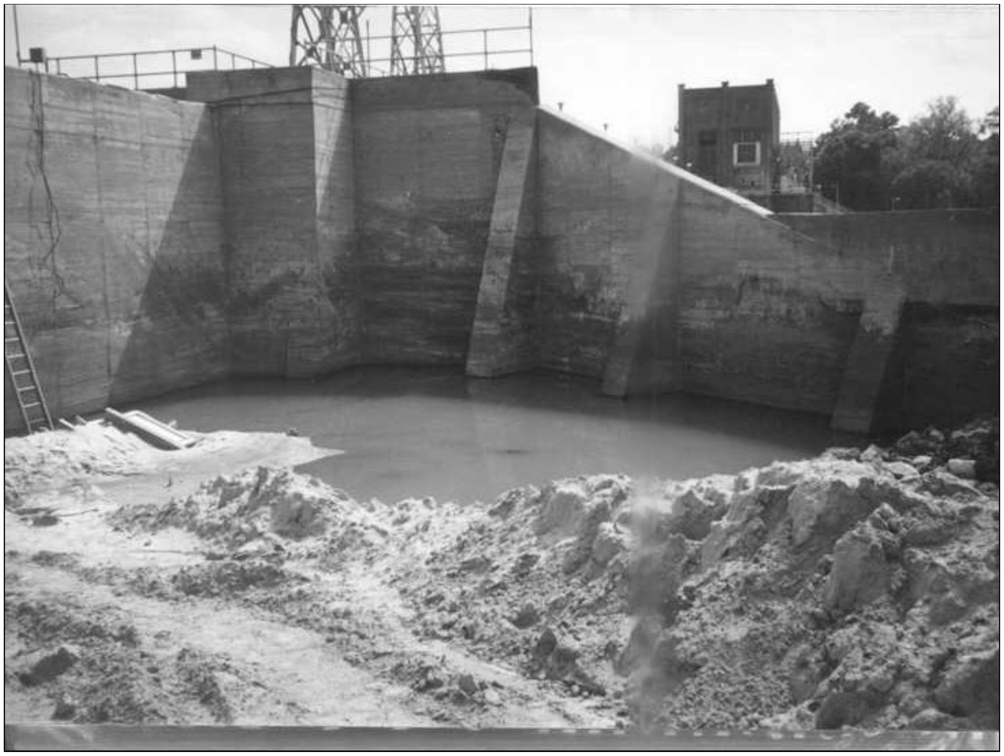
Stability Analyses Programs to Determine the Factors of Safety for the Various Loading Conditions

- **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

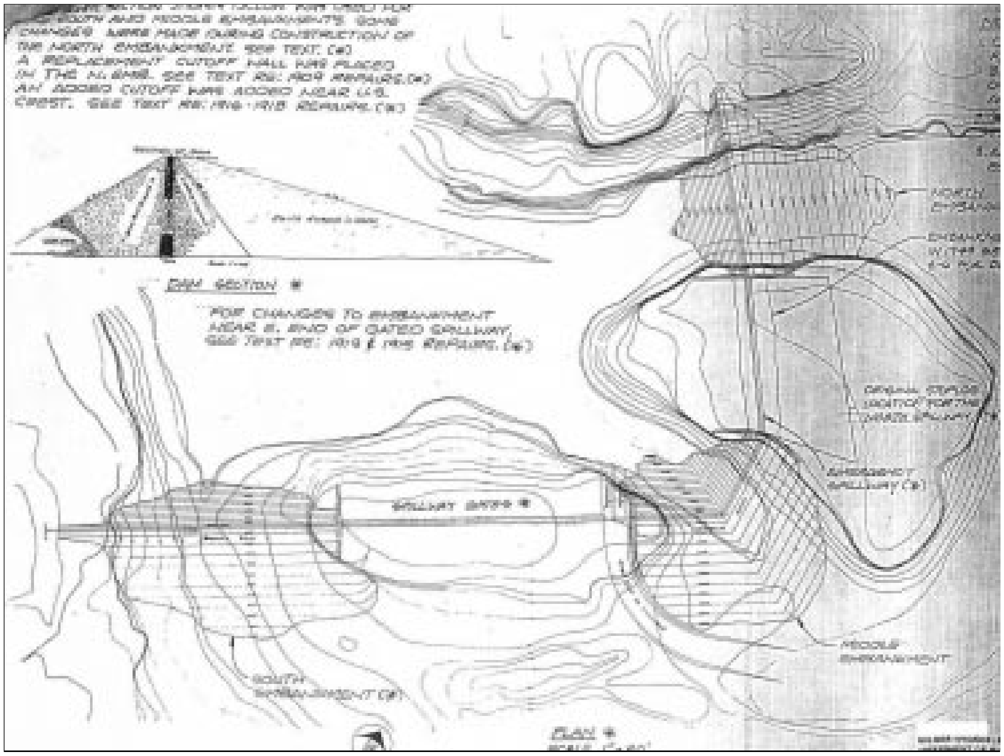
- **Reasons to Prevent Overtopping of an Embankment Even When Covered With a Good Growth of Grass**



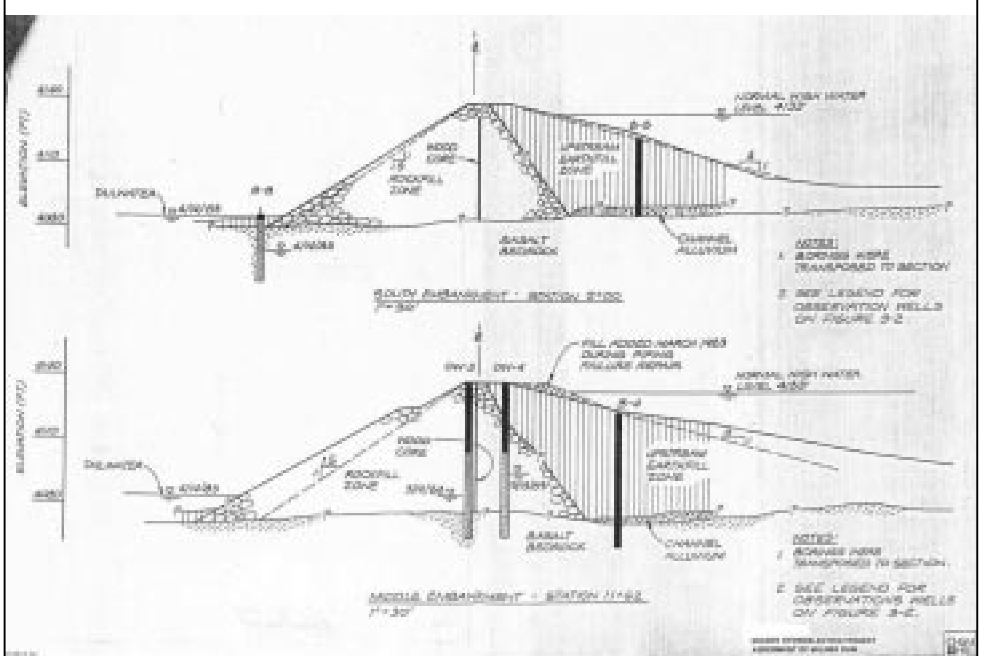


Milner Dam

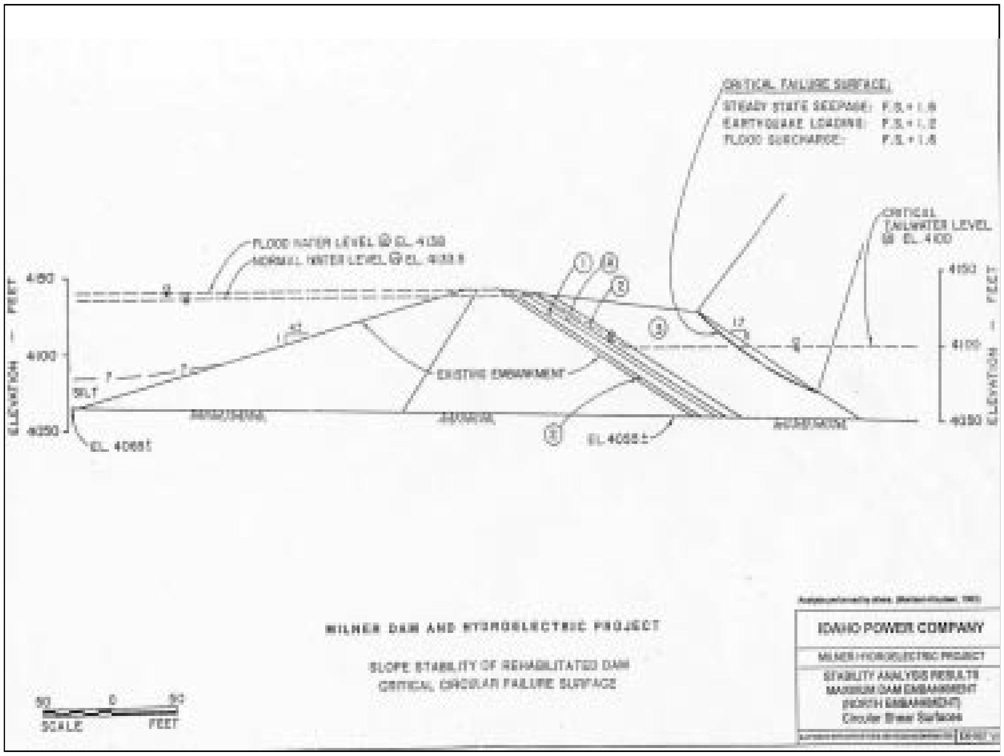
- **Control of Seepage Flow Through the Embankment, Foundation, And Abutments to Prevent Internal Erosion (Piping)**
- **The Slopes Must be Stable During all Conditions of Reservoir Operations**
- **Do Not Permit Unacceptable Deformations Under Earthquake Conditions**

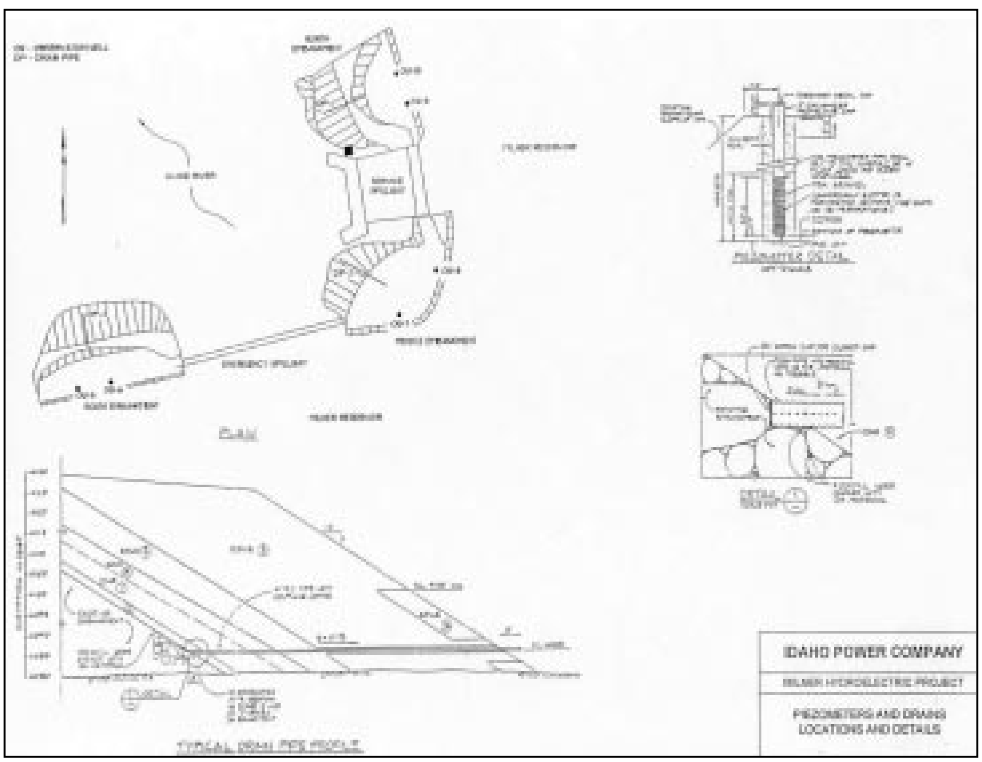


DAM SECTION
NORTH EMBANKMENT
EMERGENCY SPILLWAY
SPILLWAY GATES
MIDDLE EMBANKMENT
SOUTH EMBANKMENT
PLAN



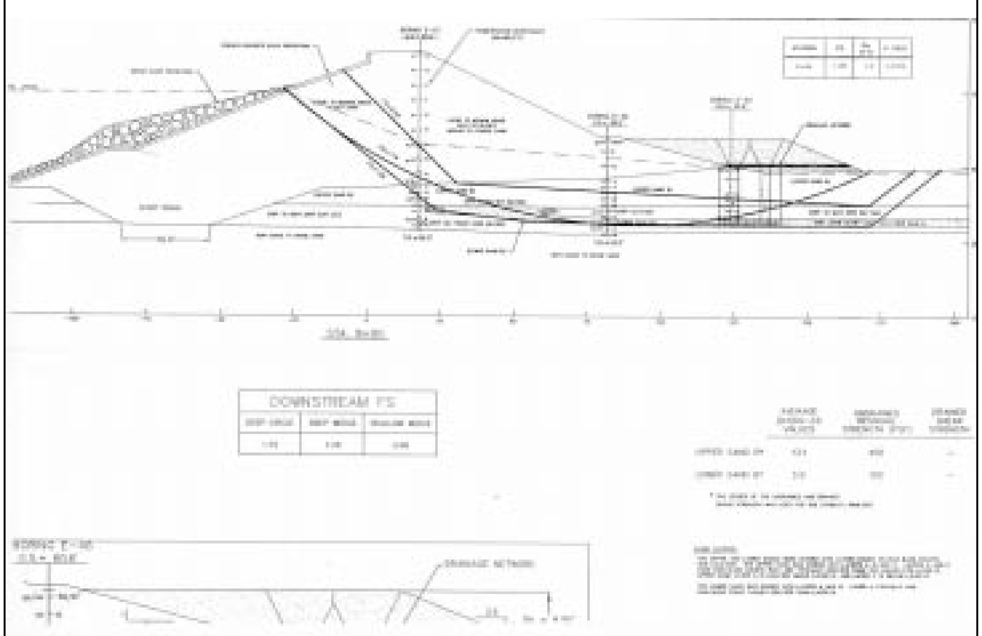
ELEVATION (FT)
NORMAL HIGH WATER
TAILWATER
ROCKFILL ZONE
CORE
UPSTREAM EARTHFILL ZONE
BASALT BEDROCK
CHANNEL ALLUVIUM
NOTES:
SOUTH EMBANKMENT
MIDDLE EMBANKMENT

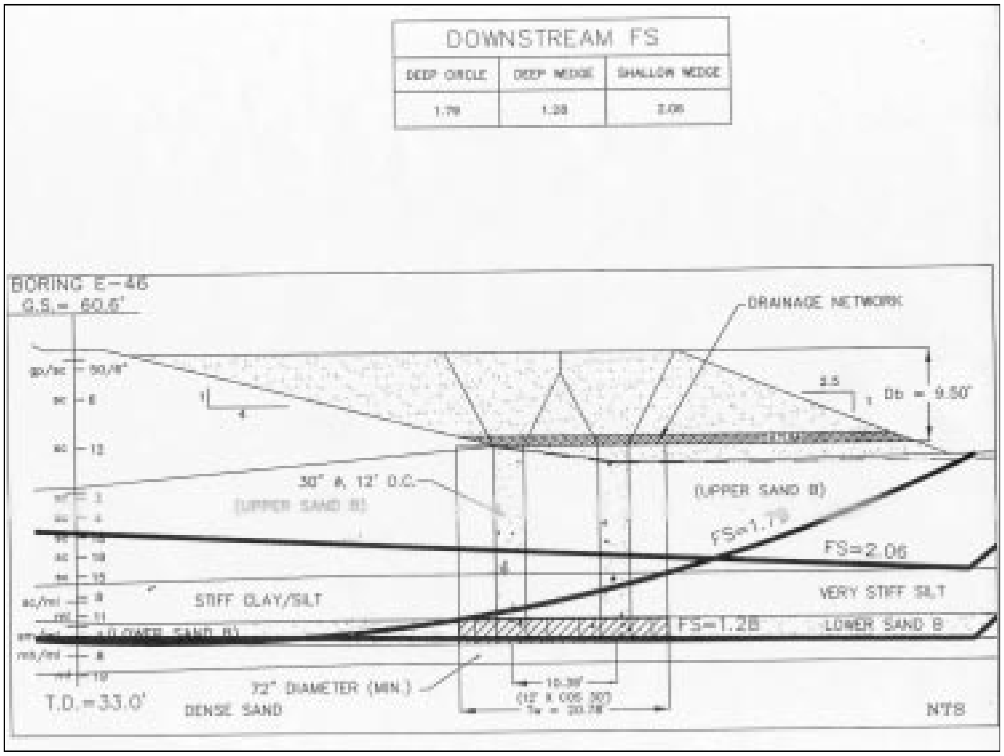

CRITICAL FAILURE SURFACE:
STEADY STATE SEEPAGE:
EARTHQUAKE LOADING:
FLOOD SURCHARGE:
CRITICAL TAILWATER LEVEL
@ EL. 4100
FLOOD WATER LEVEL
NORMAL WATER LEVEL
EXISTING EMBANKMENT
SILT
ELEVATION — FEET
SCALE FEET
MILNER DAM AND HYDROELECTRIC PROJECT
SLOPE STABILITY OF REHABILITATED DAM
CRITICAL CIRCULAR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


PLAN
PIEZOMETER DETAIL
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**



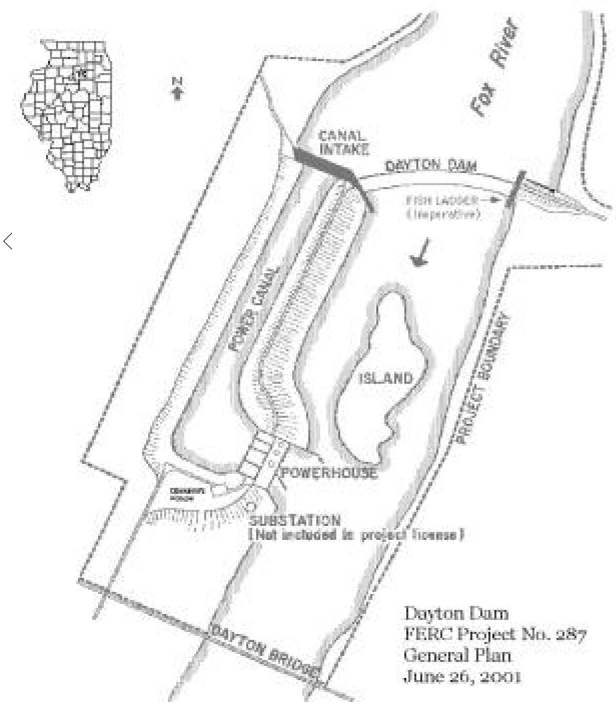

DOWNSTREAM FS
DEEP CIRCLE
DEEP WEDGE
SHALLOW WEDGE
1.79
1.28
2.06
BORING E-46
G.S.= 60.6'
DRAINAGE NETWORK
2.5
1
Db = 9.50'
1
4
30" Ø, 12' O.C.
(UPPER SAND B)
(UPPER SAND B)
FS=1.79
FS=2.06
STIFF CLAY/SILT
VERY STIFF SILT
FS=1.28
LOWER SAND B
(LOWER SAND B)
72" DIAMETER (MIN.)
10.39'
(12' X COS 30')
Tw = 20.78'
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



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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2/12/02
4

FERC-CRO June 2001

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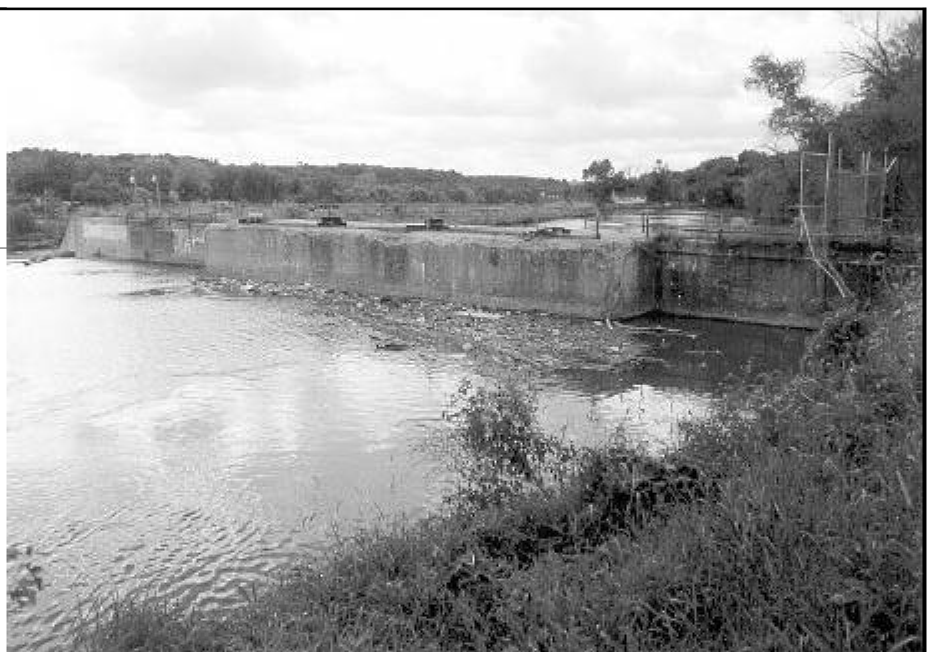

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7

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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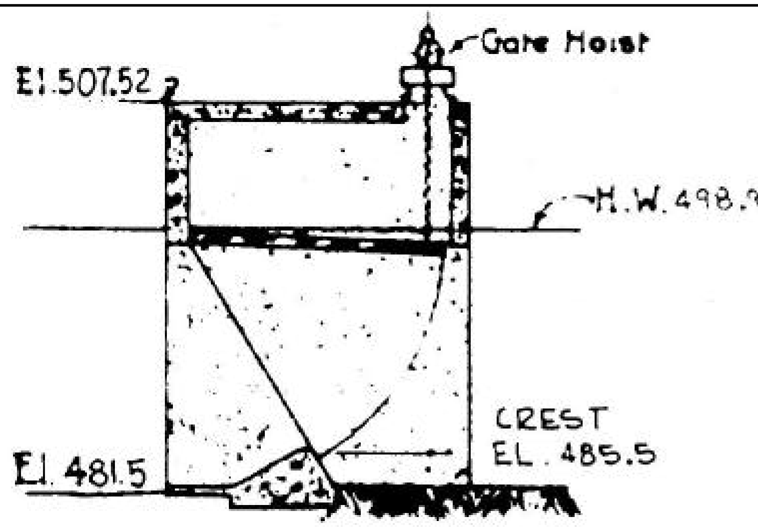

Sect. C–C
Thru Canal
Head Works

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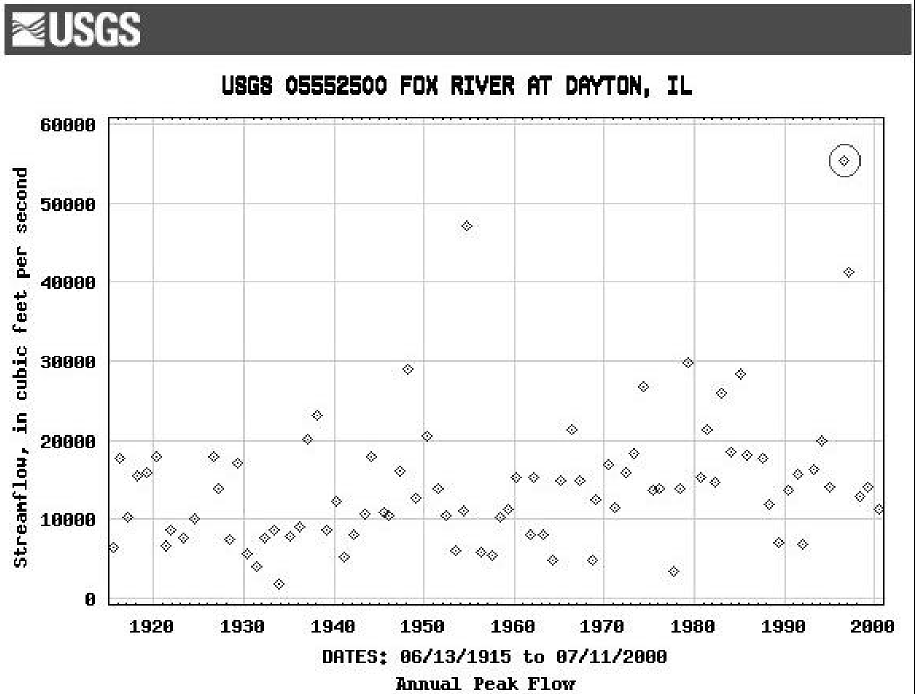
10



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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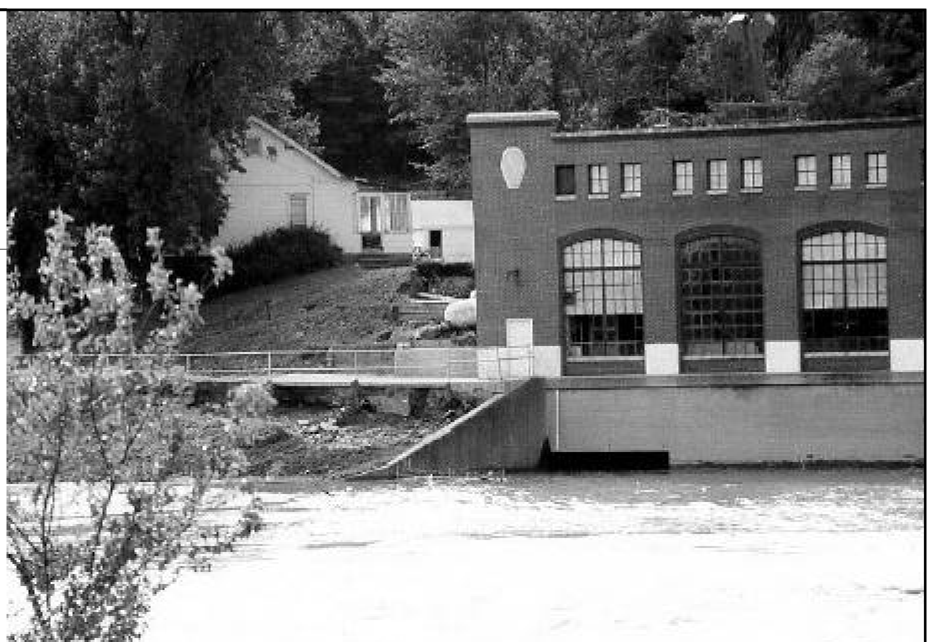

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July 22, 1996 Inspection

- 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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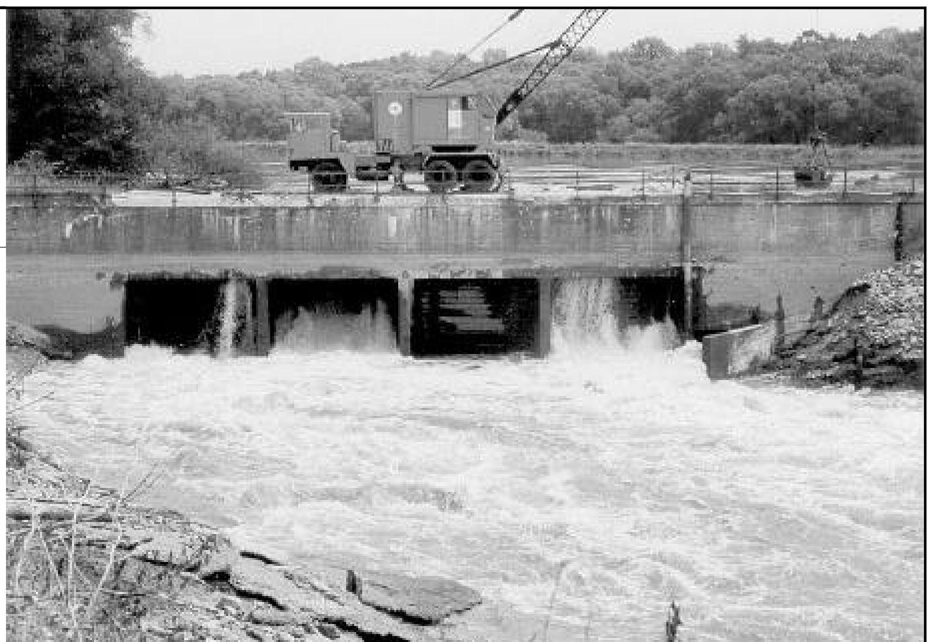

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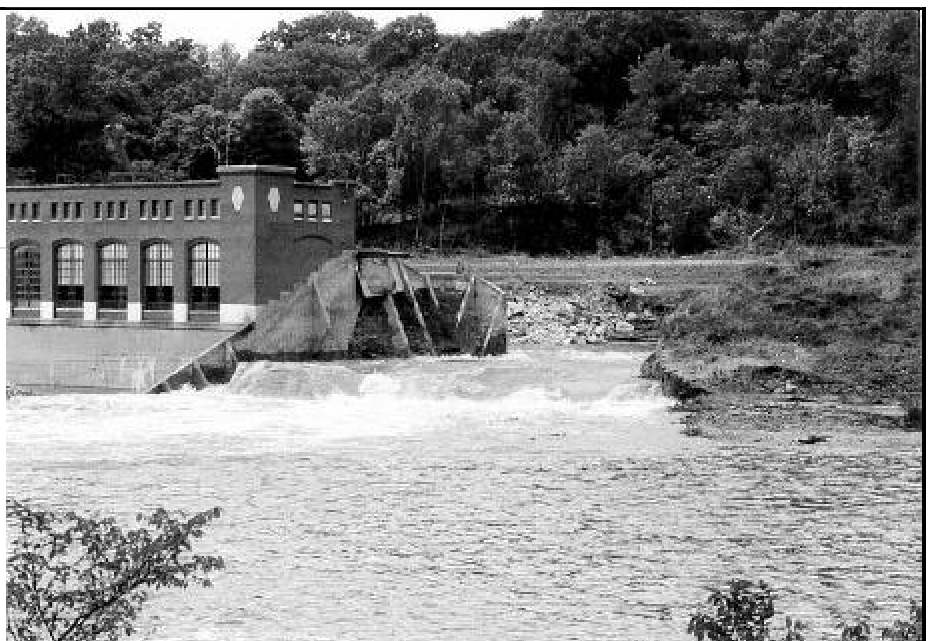

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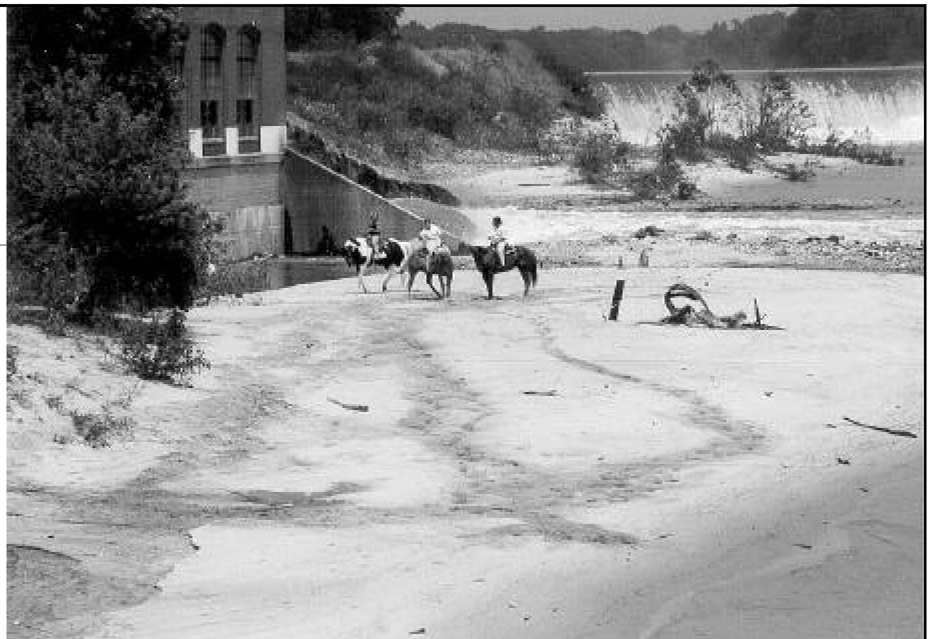

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September 30, 1996 Inspection

- 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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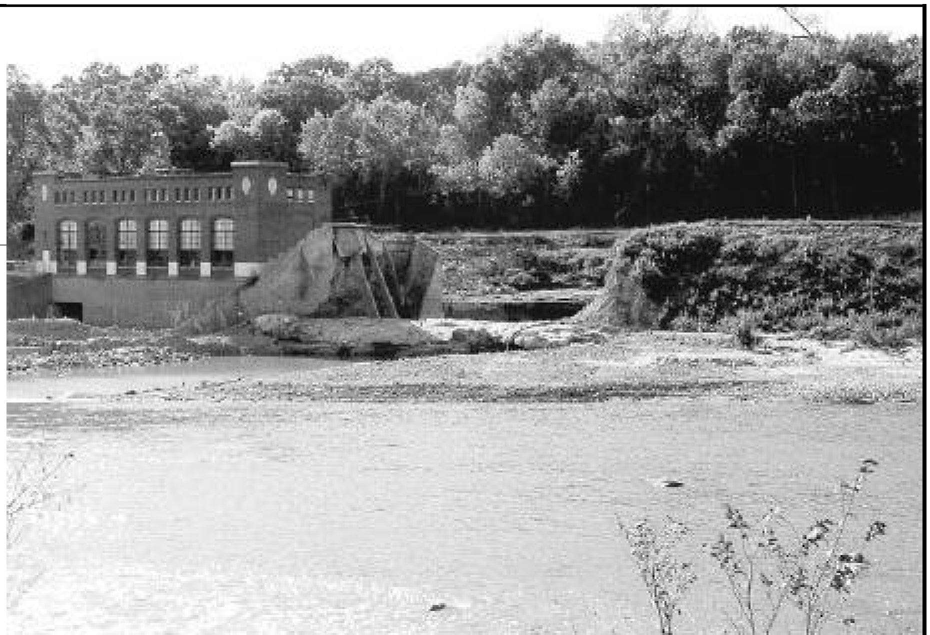

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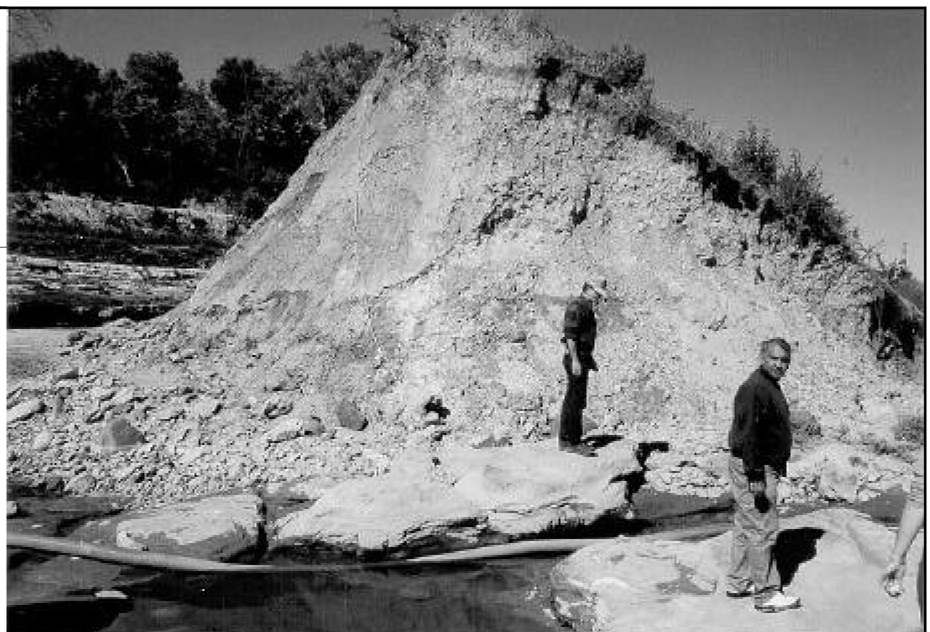

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Investigation and Evaluation

- 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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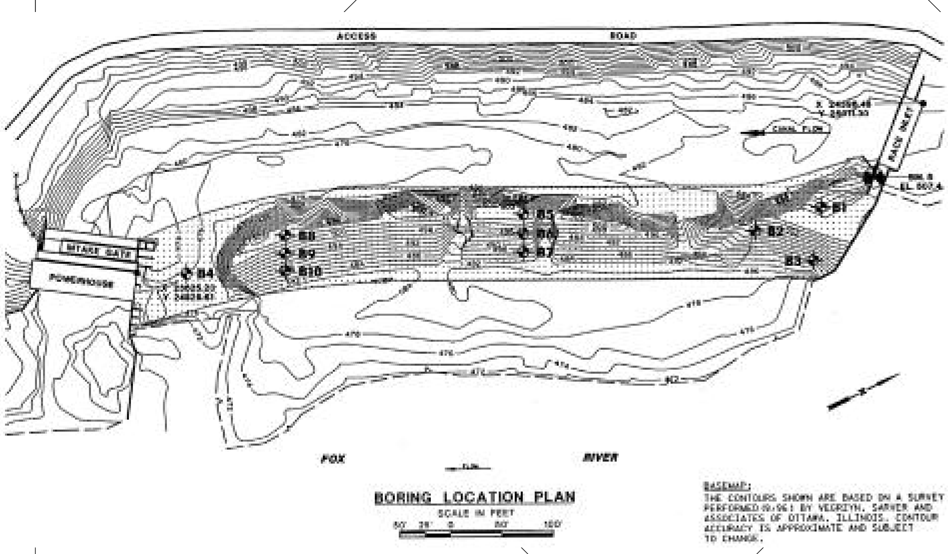

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Reconstruction of the Canal Embankment

- 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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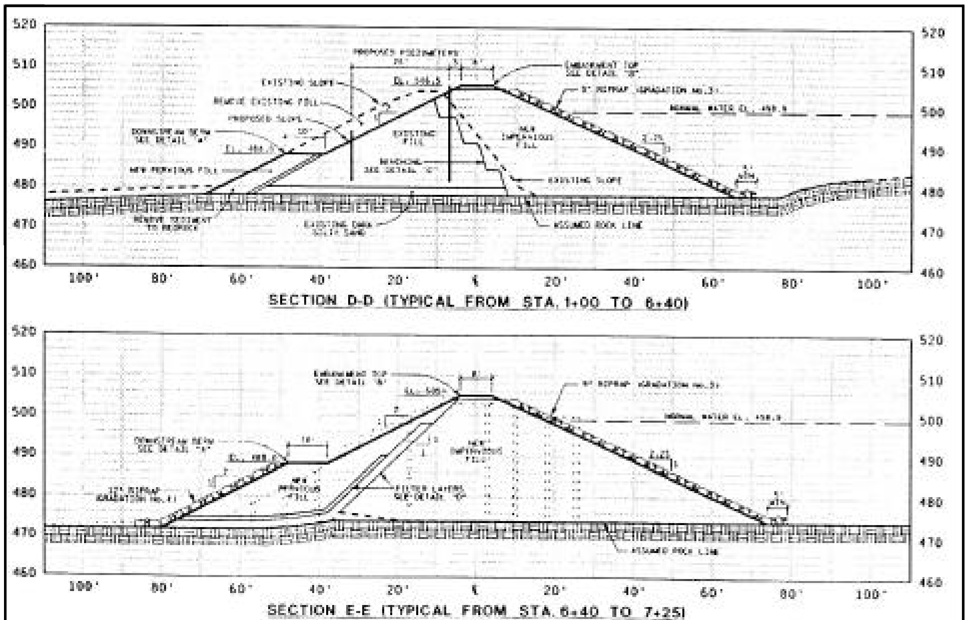

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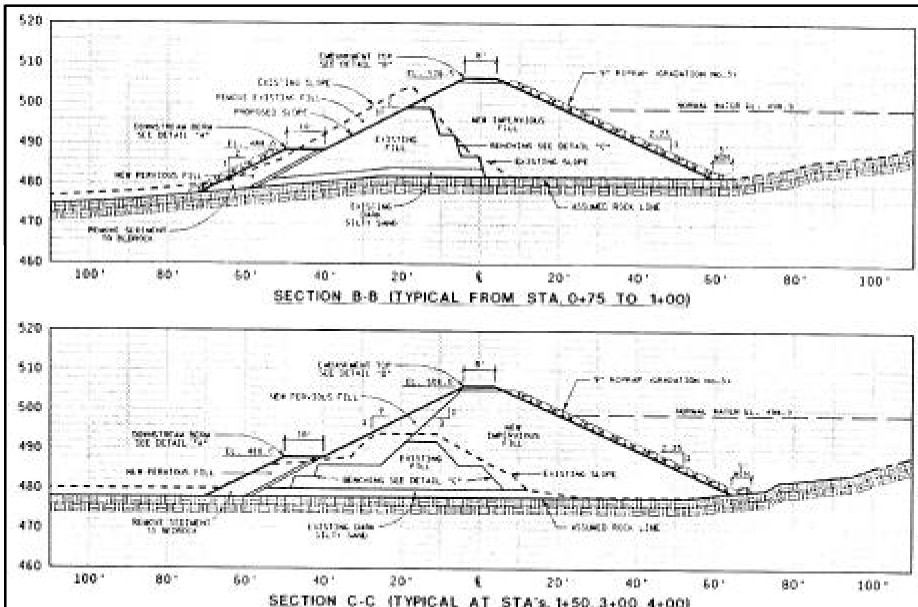

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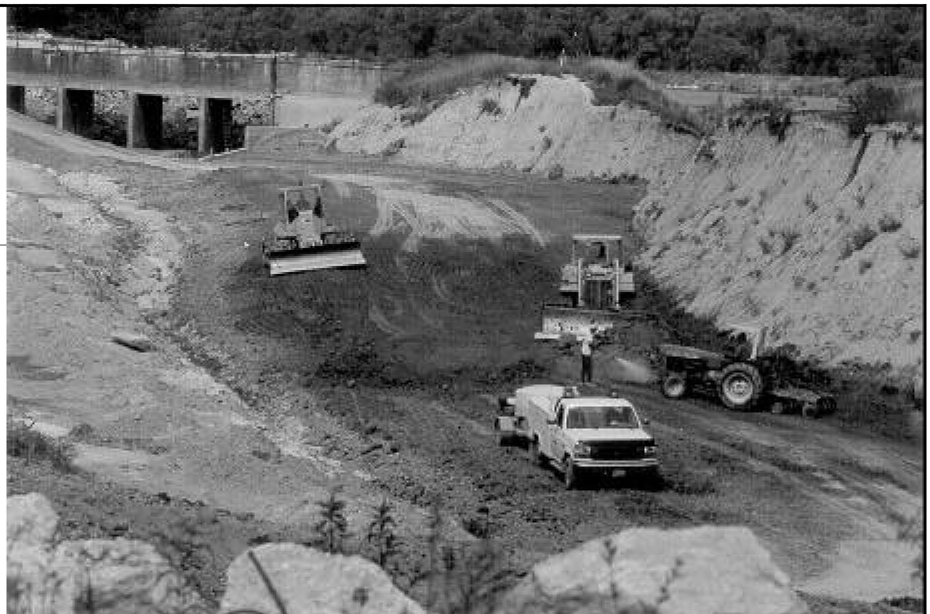
August 27, 1997 Inspection

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

FERC-CRO June 2001 2/12/02 36



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September 30, 1997 Inspection

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

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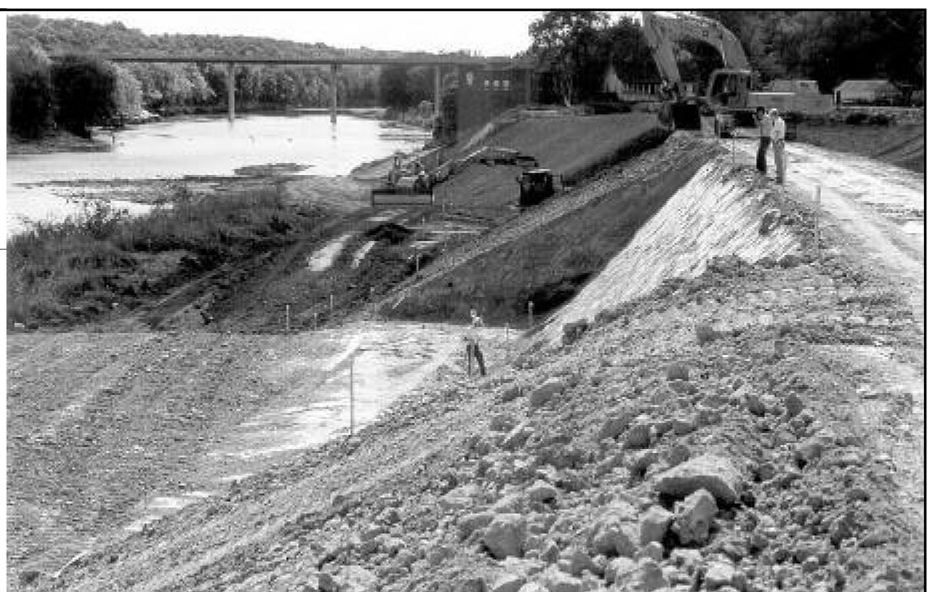

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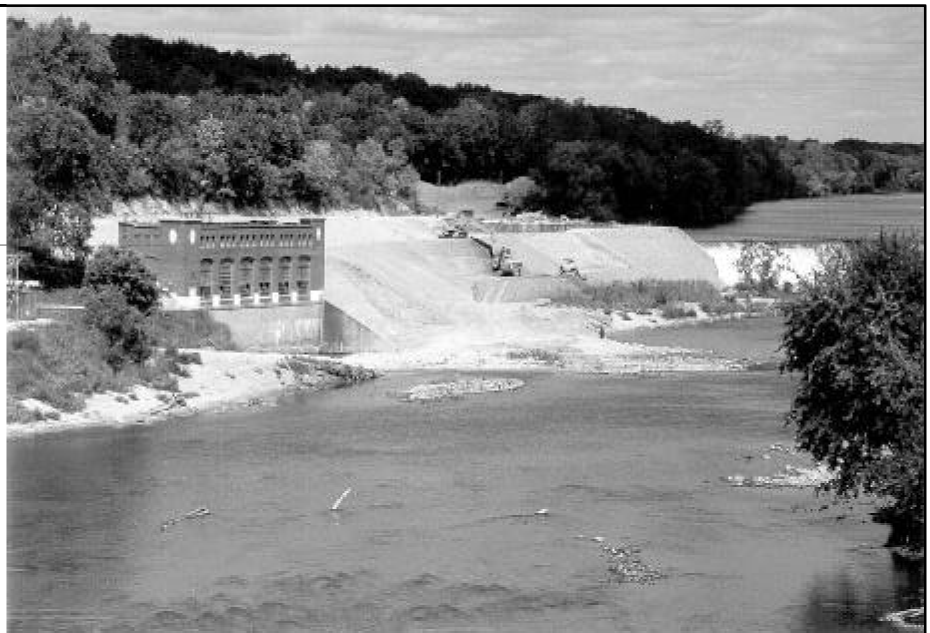

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Final Inspection – January 15, 1998.

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

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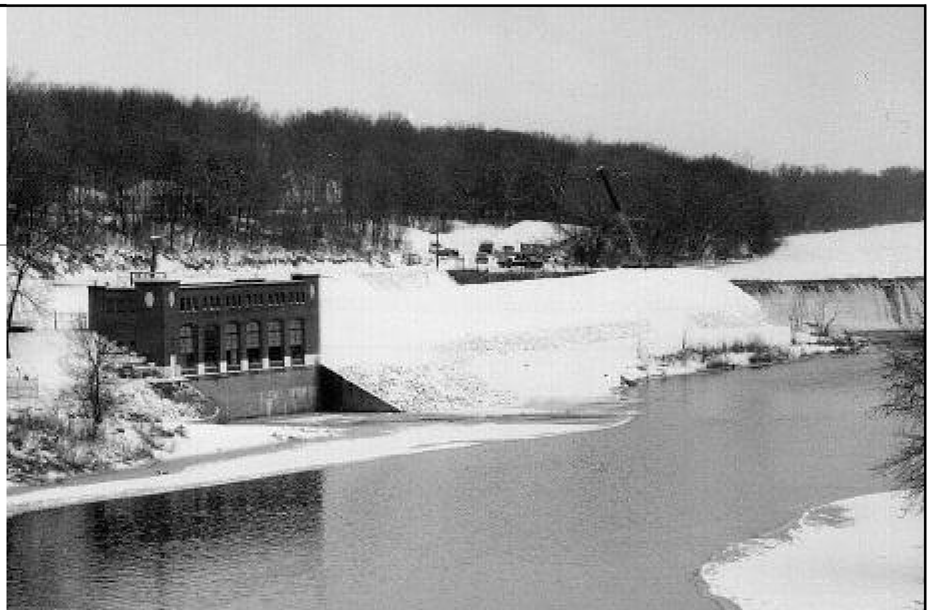

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Presentation for FEMA / USDA Workshop – 27 June 2001:
Workshop on Issues, Resolutions, and Research Needs Related to Dam Failure Analysis

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.

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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.

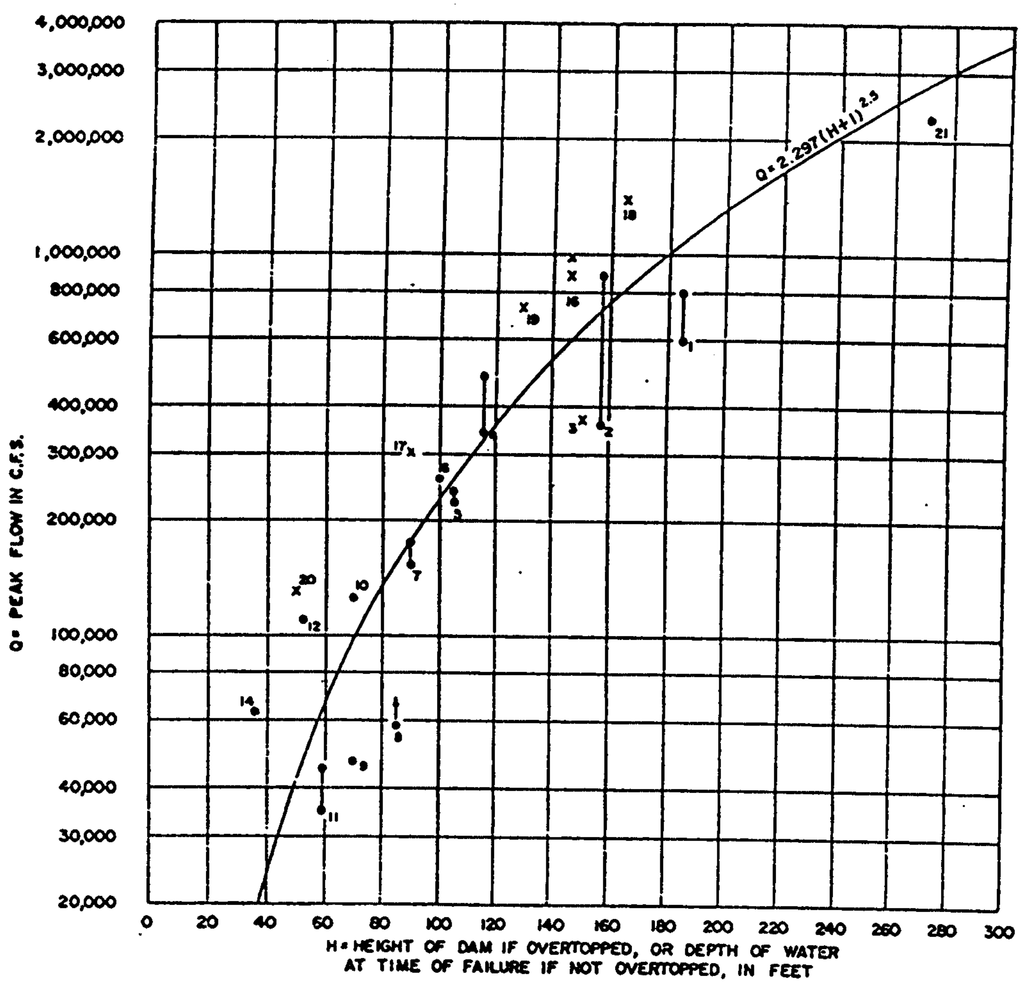
2. State of California, Senate bill No. 1632, Chapter 314, An Act to Amend Section 8589.5 of the Government Code, relating to dam safety, and declaring the urgency thereof, to take effect immediately, Approved by Governor May 31, 1974, Filed with Secretary of State May 31, 1974.

3. Cristofano, E. A., "Method of Computing Erosion Rate for Failure of Earthfill Dams, Bureau of Reclamation, Denver, Colorado, 1965.

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 x 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 = O	Masonry, Gravity Timber Crib, Buttress
	¼ $\leq$ 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*

B-17

Current Dam Safety Research Efforts

Presented By:
Steven Abt

Hydraulic Design of Stepped Spillways

James Ruff
and
Jason Ward

Project Background

- Continuation of Dam Safety Research
  - Cooperative agreement between CSU & USBR
  - spillway overtopping flows
  - near prototype scale test facility
- Stepped Spillway Phase
  - Start of construction July 1999
  - Two summers of testing
  - Data analysis and report 2000-2001

Overtopping Facility

- Near-prototype scale
- 2H:1V Slope
- 100 ft concrete chute
- 50ft height
- 10ft wide
  - reduced to 4 ft
- 5 ft deep
  - 7 ft extended height
- Horsetooth water supply
  - approx. 120 cfs max

Objective

Collect data on the characteristics of stepped spillway flow and develop a hydraulic design procedure.

Experimental Program

- Air concentration data
- Velocity data
- Visual Observations
  - Range of discharges & locations
  - Two step heights

Test Series

Stepped Spillway Tests

- Horizontal Steps
- Constructed of lumber and plywood

Smooth Spillway Tests

- Steps removed
- Comparison data

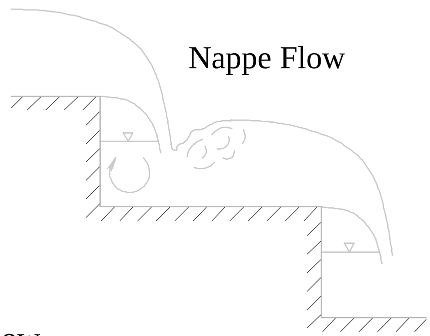
- First Series
  - 25 two-foot steps
  - 4 ft tread length
  - 2 ft riser height

- Second Series
  - 50 two-foot steps
  - 2 ft tread length
  - 1 ft riser height

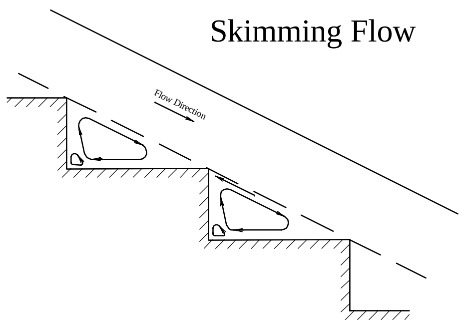
- Third Series
  - Steps removed
  - Comparison data

Flow Classification

Nappe Flow



Skimming Flow



Observations

h = 2.0 ft

Transition
Q = 40 cfs
Window # 4

Nappe
Q = 20 cfs
Window # 4

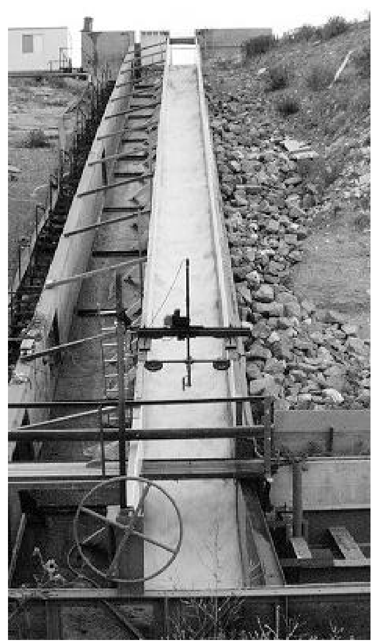
Skimming
Q = 60 cfs
Window # 3

Observations, cont'd

h = 1.0 ft

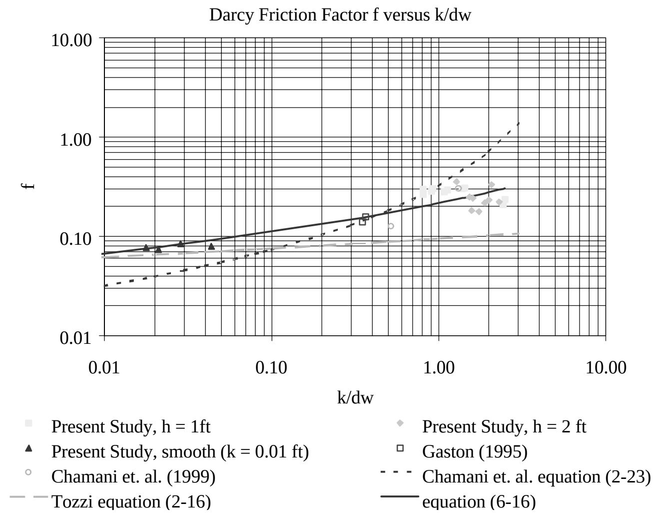
Nappe
Q = 7.1 cfs
Window # 2

Skimming
Q = 21 cfs
Window # 4



h = 1.0 ft
Q = 60 cfs

Friction Factor

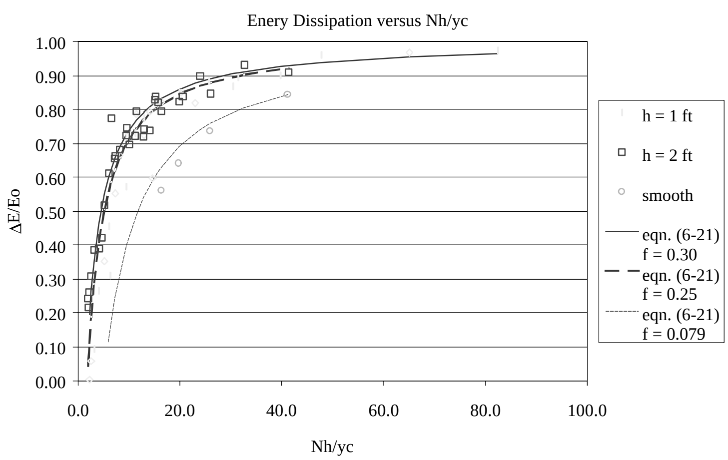


$$f = \frac{8 g d_w S_f}{U_{avg}^2}$$

$$\frac{1}{\sqrt{f}} = 2.15 + 0.85 \log\left(\frac{d_w}{k}\right)$$

k = roughness height
dw = clear water depth

Energy Dissipation, cont'd



Energy Dissipation

$$\frac{\Delta E}{E_o} = \frac{E_o - E_1}{E_o}$$

$$\frac{\Delta E}{E_o} 1 - \frac{\left(\frac{f}{8\sin\theta}\right)^{\frac{1}{3}} \cos\theta + \frac{1}{2}\left(\frac{f}{8\sin\theta}\right)^{-\frac{2}{3}}}{\frac{Nh}{y_c} + \frac{3}{2}}$$

N = number of steps
h = step height
yc = critical depth

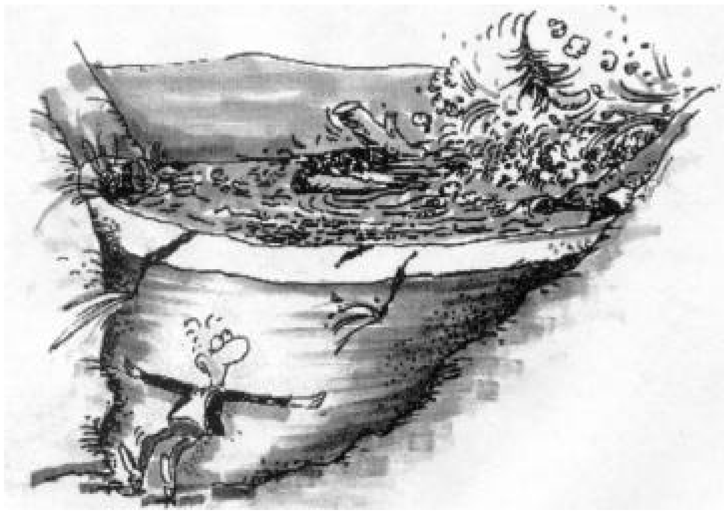
Hydraulic Design Procedure

- Assume given information
  - Total discharge, Q
  - spillway width, b
  - spillway height, H
  - spillway slope, $\theta = 26.6^{o}$
  - select step height, h (1.0 ft or 2.0 ft)
- Design Charts:
  - friction factor $f = f(H, q)$ versus Nh/y_c
  - bulking coefficient $\varepsilon = f(H,q)$ versus Nh/y_c
- Water surface profile computation with f
  - d_w, U_{avg}
- Compute energy dissipation

Hydraulic Analysis of Articulated Concrete Blocks

Christopher Thornton, Steven Abt, Chad Lipscomb and Michael Robeson

Recent Developments in the Research and Development of Articulating Concrete Blocks for Embankment Overtopping Protection

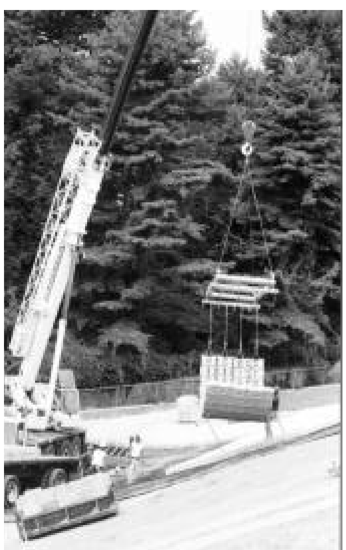


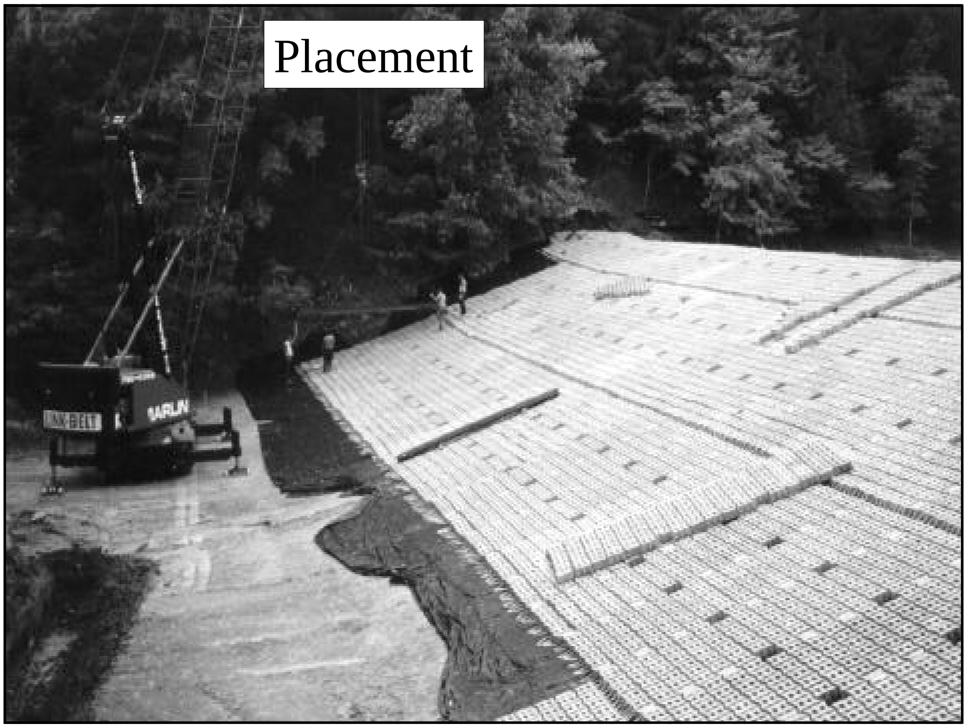
Purpose

- To evaluate the performance of commercially available embankment protection systems under various hydraulic conditions
- To develop design criteria for ACB systems
- To determine the effect of a drainage medium under ACB systems

ACB Mats

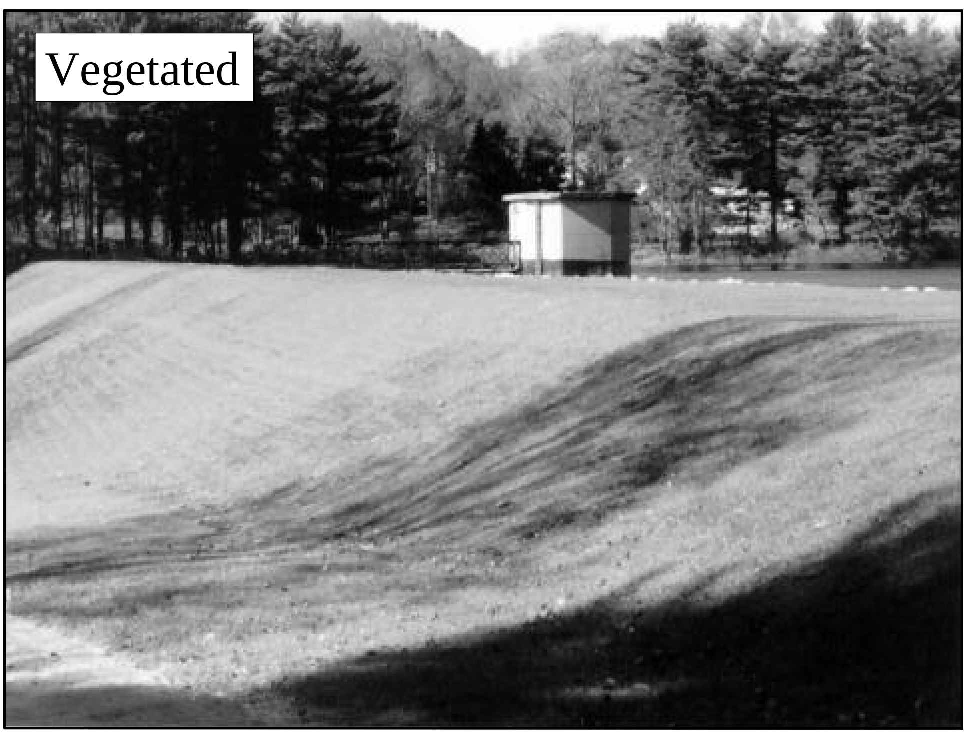
Placement



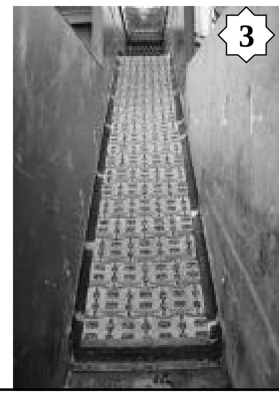

Placement
LINK-BELT
SARLIN


Un-Vegetated


Vegetated


Vegetated

Block Overtopping Tests
Flume Setup



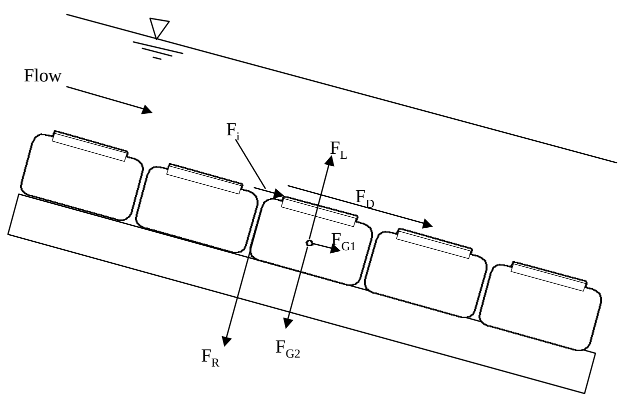
Overtopping Testing

Overtopping Testing

Threshold Levels

Threshold Levels

Force Balance



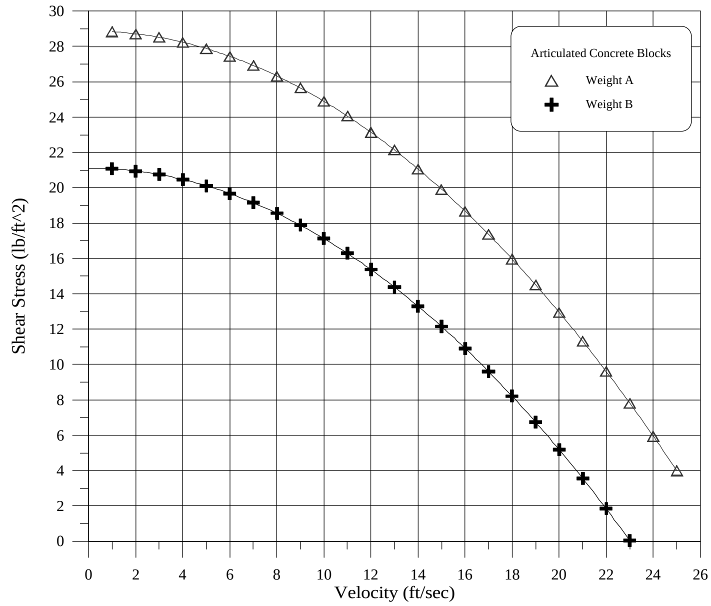
Destabilizing Forces

F_i = Impact force (projecting block)
F_L = Lift Force
F_D = Drag Force
F_{G1} = Gravity force parallel to slope

Stabilizing Forces

F_R = Interblock resistance
F_{G2} = Gravity force normal to slope

Design Curves



Results

- Drainage layer has pronounced effect on system performance
- At high flows, velocity appears to be dominant force
- Performance values consistent between overtopping and channelized test protocols

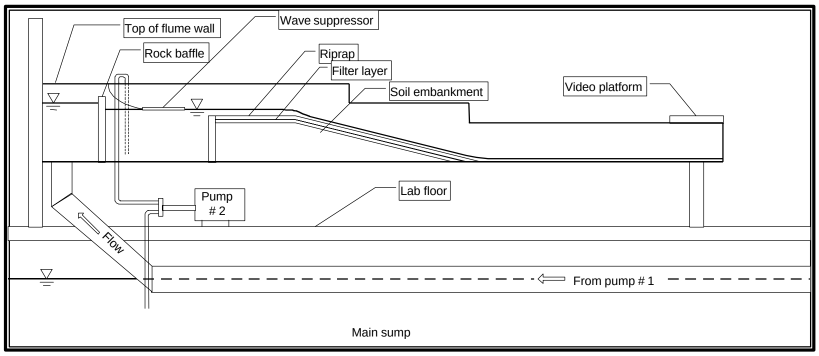
Design Criteria
for
Rounded Rock Riprap

Steven Abt
and
Humberto Gallegos

Purpose

- Develop design criteria for rounded/angular rock riprap in overtopping flow
- Expand the database of rounded rock riprap to include higher embankment slopes
- Increase the understanding of the behavior of rounded rock riprap in overtopping flow

Flume Setup



Flume Setup

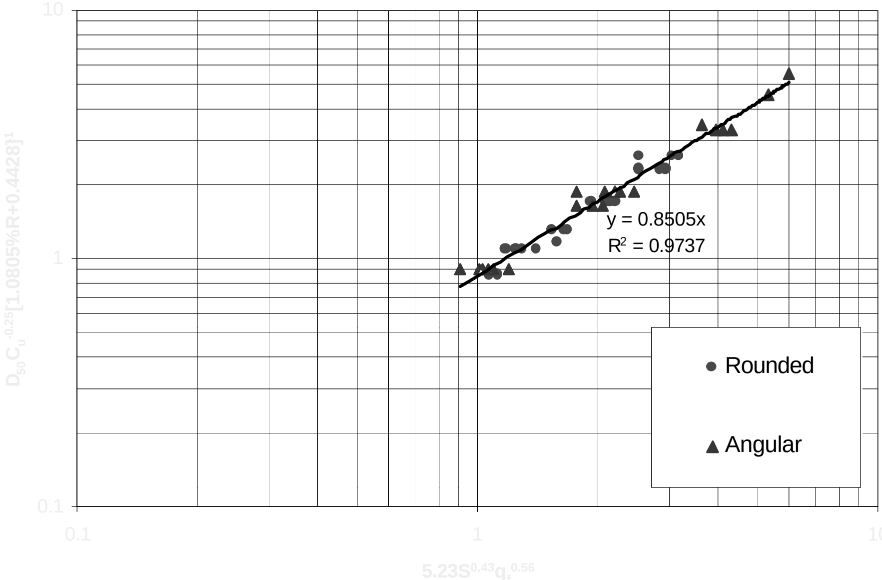
Testing Matrix

Test #	D_{50}	D_{50}	C_u	% Rounded	S
	(cm)	(in)			decimal
1	5.87	2.31	1.21	79	0.35
2	3.23	1.27	1.32	60	0.35
3	9.91	3.90	1.24	92	0.35
4	5.87	2.31	1.21	79	0.40
5	3.23	1.27	1.32	60	0.40
6	9.91	3.90	1.24	92	0.40
7	3.23	1.27	1.32	60	0.45
8	5.87	2.31	1.21	79	0.45
9	9.91	3.90	1.24	92	0.45

Cumulative Database

			D_{50}	D_{50}	C_u	Rounded	S	q_f	q_f	Mode Of
			(cm)	(in)		%	decimal	$m^3/s/m$	$ft^3/s/ft$	Failure
Current Study	1	Round	5.87	2.31	1.21	79	0.35	0.042	0.455	Exposure
	2	Round	3.23	1.27	1.32	60	0.35	0.014	0.154	Catastrophic
	3	Round	9.91	3.90	1.24	92	0.35	0.086	0.925	Exposure
	4	Round	5.87	2.31	1.21	79	0.40	0.021	0.225	Channel
	5	Round	3.23	1.27	1.32	60	0.40	0.014	0.157	Channel
	6	Round	9.91	3.90	1.24	92	0.40	0.051	0.552	Exposure
	7	Round	3.23	1.27	1.32	60	0.45	0.012	0.131	Exposure
	8	Round	5.87	2.31	1.21	79	0.45	0.036	0.394	Catastrophic
	9	Round	9.91	3.90	1.24	92	0.45	0.065	0.702	Exposure
Ullmann (2000)	10	Round	2.39	0.94	1.24	55	0.20	0.019	0.20	Channel
	11	Round	2.39	0.94	1.24	55	0.30	0.016	0.18	Channel
	12	Round	3.23	1.27	1.32	60	0.20	0.020	0.22	Channel
	13	Round	3.23	1.27	1.32	60	0.25	0.021	0.22	Channel
	14	Round	3.23	1.27	1.32	60	0.30	0.019	0.21	Channel
	15	Round	4.50	1.77	1.33	75	0.20	0.035	0.38	Channel
	16	Round	4.50	1.77	1.33	75	0.25	0.035	0.38	Channel
	17	Round	4.50	1.77	1.33	75	0.30	0.029	0.32	Channel
	18	Round	5.87	2.31	1.21	79	0.20	0.053	0.58	Catastrophic
	19	Round	5.87	2.31	1.21	79	0.20	0.049	0.53	Catastrophic
	20	Round	5.87	2.31	1.21	79	0.25	0.052	0.57	Catastrophic
	21	Round	9.91	3.90	1.24	92	0.30	0.096	1.04	Catastrophic
Abt et al. (1988 a)	22	Round	10.42	4.10	2.12	95	0.20	0.087	0.96	Unknown
	23	Round	10.42	4.10	2.12	95	0.20	0.087	0.96	Unknown
	24	Round	10.42	4.10	2.12	95	0.10	0.183	1.98	Unknown
	25	Round	10.42	4.10	2.12	95	0.10	0.192	2.08	Unknown
	26	Round	5.21	2.05	2.14	95	0.10	0.063	0.69	Unknown
Abt and Johnson (1991)	1	Angular	2.59	1.02	1.75	0	0.02	0.102	1.11	Unknown
	2	Angular	2.59	1.02	1.75	0	0.01	0.138	1.50	Unknown
	3	Angular	2.59	1.02	1.75	0	0.10	0.023	0.26	Unknown
	4	Angular	2.59	1.02	1.75	0	0.10	0.031	0.34	Unknown
	5	Angular	2.59	1.02	1.75	0	0.10	0.028	0.31	Unknown
	6	Angular	2.59	1.02	1.75	0	0.10	0.039	0.42	Unknown
	7	Angular	5.05	2.00	2.40	0	0.10	0.078	0.85	Unknown
	8	Angular	5.05	2.00	2.40	0	0.10	0.092	1.00	Unknown
	9	Angular	5.05	2.00	2.40	0	0.10	0.102	1.11	Unknown
	10	Angular	5.59	2.20	2.09	0	0.10	0.103	1.12	Unknown
	11	Angular	5.59	2.20	2.09	0	0.10	0.115	1.25	Unknown
	12	Angular	5.59	2.20	2.09	0	0.10	0.116	1.25	Unknown
	13	Angular	5.59	2.20	2.09	0	0.08	0.166	1.81	Unknown
	14	Angular	5.59	2.20	2.09	0	0.02	0.416	4.53	Unknown
	15	Angular	5.59	2.20	2.09	0	0.20	0.046	0.50	Unknown
	16	Angular	10.16	4.00	2.30	0	0.10	0.232	2.51	Unknown
	17	Angular	10.16	4.00	2.30	0	0.10	0.348	3.79	Unknown
	18	Angular	10.16	4.00	2.30	0	0.10	0.379	4.12	Unknown
	19	Angular	10.41	4.10	2.16	0	0.20	0.166	1.81	Unknown
	20	Angular	12.95	5.10	1.62	0	0.20	0.327	3.56	Unknown
	21	Angular	15.75	6.20	1.69	0	0.20	0.407	4.43	Unknown

Analysis



Results

$$D_{50} = 6.58S^{0.43}q_f^{0.56}C_u^{0.25}(1.0805\%R=0.4428)$$

•Embankment Slopes:	10 to 45 %
•Median Rock Sizes:	D_{50} = 2.4 to 15.3 cm
•Rounded Rock:	55 to 95 %
•Riprap Layer Thickness:	1.5 to 3 D_{50}
•Coefficient of Uniformity:	1.2 to 4.0

B-18

Limited Overtopping, Embankment Breach and Discharge

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

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ABSTRACT

Over 10,000 flood control reservoirs constructed with the assistance of the USDA provide almost $1 billion in benefits each year. Sixty-two percent of these 10,000 structures will reach age 50 by 2020. As these structures age additional trapped sediment may reduce the flood control capacity of the reservoir, population increases and changes in land in the upstream watershed may result in increased runoff, population encroachment on the downstream channels may result in structures that were designed to protect agricultural land now being depended upon to protect lives and homes, and many state dam safety regulatory requirements have also been increased since the original construction as a result of federal legislation and/or state laws. Because of this, public safety requires that this aging infrastructure be re-evaluated and, in some cases, rehabilitated. A key aspect of this re-evaluation is prediction of the performance of existing hydraulic structures and channels during extreme flood events that may exceed original design conditions. This includes prediction of allowable overtopping, rate of embankment breach and failure, and resulting discharge.

INTRODUCTION

The drought of the 1930's, followed by flooding in the 1940's, made the U.S. agriculture community keenly aware of the need to keep the water and soil in place. Following World War II, numerous management practices to control erosion and reduce flooding were implemented with the assistance of the USDA. Included were the upland flood control structures constructed under PL-534, PL-566, Pilot, and RC&D watershed programs. Approximately $14 billion was invested in more than 10,000 structures that presently provide on the order of $1 billion in benefits annually. These flood control structures have become an integral part or the nation's transportation and communications infrastructure through their protection of roadways, pipelines, etc.

As these structures continue to age, additional trapped sediment may reduce the flood control capacity of many of these reservoirs. Population increases and changes in land use have modified the hydrologic properties of the watersheds upstream of some of these structures, resulting in increased runoff of water and/or sediment from a given storm. Increasing population and encroachment on the downstream channels have resulted in structures that were designed to protect agricultural land now being depended upon to protect lives and homes. Many state dam safety regulatory requirements have also been increased since the original construction as a result of federal legislation and/or state laws. Essentially all of the state dam safety laws were written or significantly revised after dam safety concerns were raised in the 1970's following the failure of Teton and Tacoa Falls Dams. Over 70% of USDA-assisted projects were in place by that time. Conflicts between the design of the older dams and the new dam safety rules are inevitable. Public safety requires that the aging infrastructure that includes these dams be re-evaluated and, in some cases, the dams rehabilitated and/or modified if they are to continue to serve public needs.

The Hydraulic Engineering Research Unit of the ARS Plant Science and Water Conservation Research Laboratory is conducting research to address the problems associated with rehabilitation of the watershed flood control structures and channels, and identified as research objectives. Key identified knowledge deficiencies related to rehabilitation of watershed flood control structures and channels may be expressed in the form of research objectives as: 1) determination of the extent of overtopping that may be sustained by a vegetated earth embankment, such as a dam, without resulting in embankment breach, and 2) quantification of the processes associated with breach such that timing, rate, and geometry of breach may be predicted, and 3) quantification of the discharge hydrograph and peak discharge as a result of an embankment breach. The results of this research will be incorporated into evaluation tools and software, design criteria, and management practices that will allow the continued service and increased benefit of the nation's agricultural watershed flood control infrastructure.

EARTH EMBANKMENT EROSION RESEARCH

Although the detailed data on embankment overtopping have been very limited, substantial data have been gathered from vegetated spillways, which have experienced flood flows. Analyses of these data, combined with laboratory tests and analyses, have led to the development of a procedure for evaluation of earth spillway performance (NRCS, 1997). The model used in this procedure divides the erosion process into three phases. These phases are: 1) the failure of the vegetal cover, if any, and the development of concentrated flow, 2) erosion in the area of concentrated flow leading to the formation of a vertical or near vertical headcut, and 3) the upstream advance of the headcut leading to breach which may also be accompanied by further widening and deepening. The three phases describing progressive spillway erosion have also been observed for erosion of overtopped earth embankments when the embankment material exhibits even a small amount of cohesion (Hanson et al 2001). Therefore, even though caution is appropriate in attempting to extend this model directly to prediction of embankment breach, the breakdown of the process into these same three phases would be appropriate. Because of the short distance through the crest or an embankment dam, the concept of allowable overtopping is practically limited to the first two phases.

Tests have been conducted to evaluate the effectiveness of un-reinforced vegetation for overtopping protection and the applicability, on steep slopes, of the analysis tools of the first two phases of the three phase spillway model (Temple and Hanson, 1998; Hanson and Temple, 2001). It was found that the vegetation could provide substantial protection and that the relations used for phase 1 and phase 2 erosion of spillways could be effectively applied to the steeper embankment slopes. Differences observed were associated primarily with the reduced flow depth on the steeper slopes. This reduction in flow depth reduced the interaction of the vegetal elements with the turbulent flow field as a result of the turbulent scales being less than the length of the individual elements. However the effect of this on the flow resistance or protective action of the grass appeared to be minor. It was also observed that the decrease in flow depth emphasized the effects of discontinuities in the cover or surface. The importance of this effect on predicting breach or time to breach is shown by the curves of Figure 1 (reproduced from Temple and Hanson, 2001).

The equations used in the development of the curves of Figure 1are those documented in NRCS (1997) for the limits of phase 1, vegetal, failure. All curves are based on computations for a 3:1 embankment slope. Curve a represents a very high quality bermudagrass cover over a soil having a plasticity index of 15. Curve b is for that same condition except that the cover or surface exhibits minor discontinuities. A minor discontinuity is one that is large enough for the turbulent flow to directly impact the erodible material, but small enough that the flow does not concentrate within the discontinuity. This would normally imply a maximum dimension of the discontinuity parallel to flow on the order of flow depth and/or stem length. Curve d is for the same condition except that the discontinuity is large enough to allow the flow to fully concentrate, thereby negating any protective effect of the vegetal cover. Curve c is added to illustrate the relative importance of the material erodibility. Conditions for curve c are the same as for curve b except that the plasticity index of the material is reduced to zero to represent a highly erodible condition. The effect is substantially less than that indicated by adding discontinuities to the cover or surface. In all cases the curves represent failure on the slope and do not address the effects of the impacts of high velocity flow on toe or berm areas. Figure 1 illustrates that un-reinforced vegetation may be effective in providing overtopping protection, but attention must be given to maintenance.

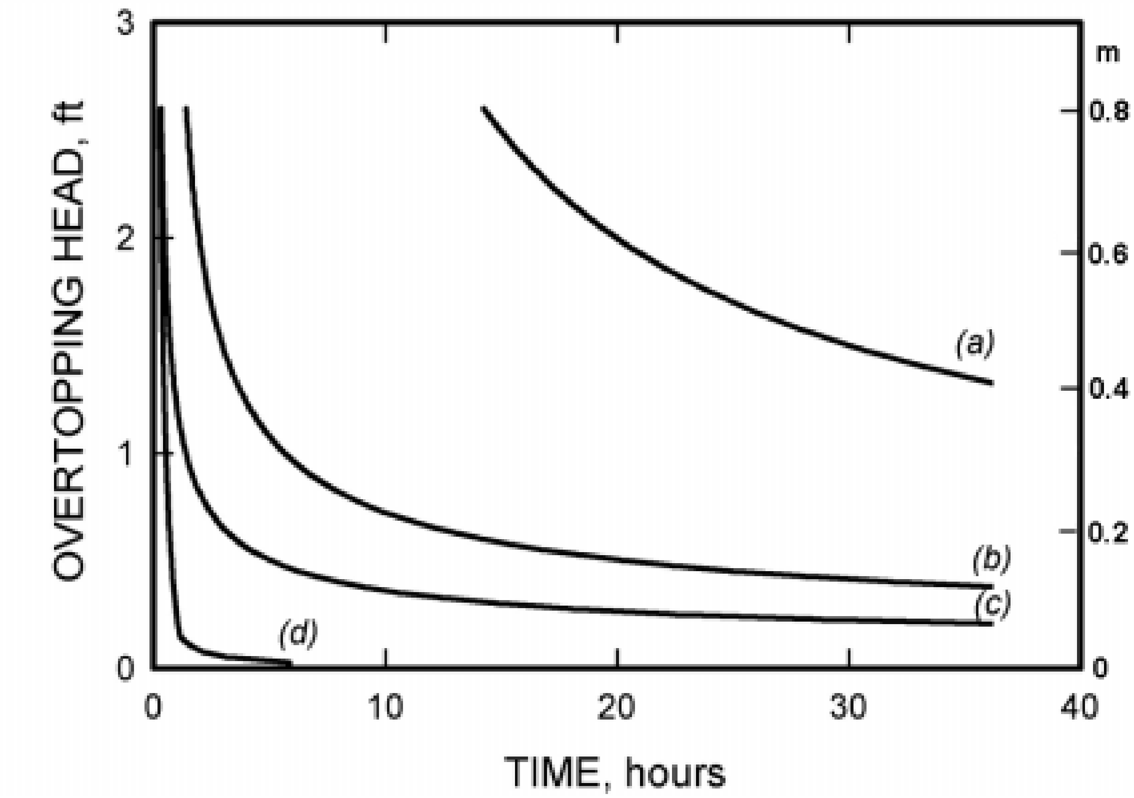

Figure 1. Potential allowable embankment overtopping based on the point of vegetal cover failure for: (a) a good cover of bermudagrass and a material plasticity index of 15; (b) a grass cover with minor surface discontinuities and a material plasticity index of 15; (c) a grass cover with minor surface discontinuities and a material plasticity index of 0; and (d) a grass cover with major discontinuities and a material plasticity index of 15.

For homogeneous earth embankments, phase 2, concentrated flow erosion, will usually represent only a very brief portion of the hydrograph. The combination high stresses and low flow depth on the steep embankment slope means that once the flow becomes concentrated in the developing discontinuity, erosion to the point of development of a vertical or near vertical headcut is normally quite rapid. This phase received some attention in the research conducted on steep slopes (Hanson and Temple 2001) and embankment overtopping (Hanson et al 2001). These tests verified that phase 2 is typically very brief and that the relations used in the spillway model are adequate. An important point that was brought out in Hanson and Temple 2001 is that erodibility of any given soil may vary several orders of magnitude depending on compaction density, and moisture content (Figure 2); indicating that proper measurement of erodibility is essential in predicting embankment performance. Erodibility is typically defined by two soil parameters, critical stress τ_c and the detachment coefficient k_d. The erosion is assumed not to begin until the effective hydraulic stress τ_e exceeds τ_c. Once the critical stress is exceeded the rate of erosion, $d\varepsilon/dt$ is assumed to occur at a linear rate described by the excess stress equation:

$$\frac{d\varepsilon}{dt} = k_d(\tau_e - \tau_c) \qquad [1]$$

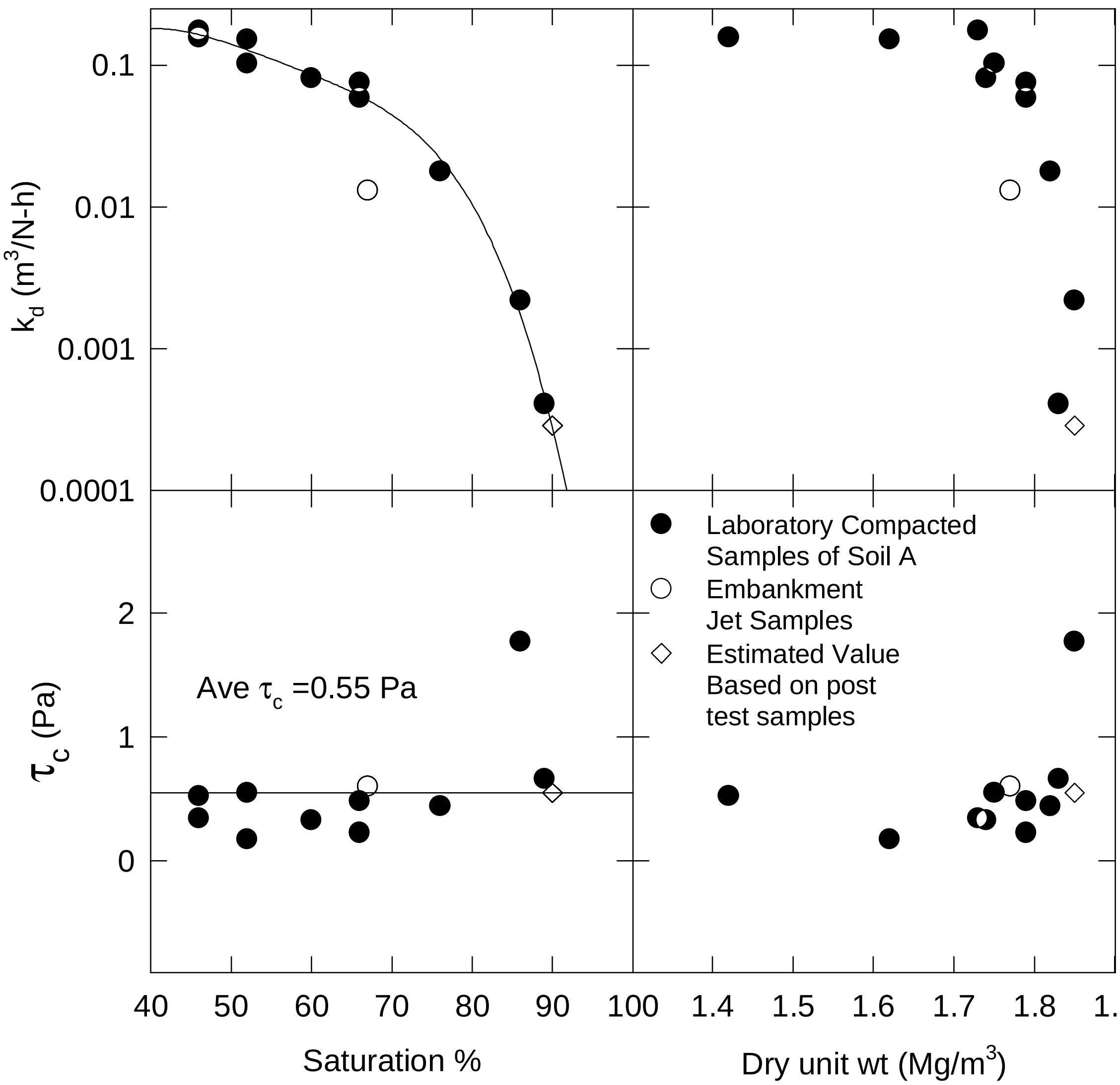

Figure 2. Relationship of a) k_d and saturation, b) k_d and dry unit weight, c) τ_c and saturation, and d) τ_c and dry unit weight for laboratory jet tests, and embankment jet tests.

Phase 3, headcut deepening and advance, is a critical part of the breaching process. The purpose of the spillway model is the determination of the potential for breach to occur. Although this is an important consideration for overtopped embankments, the time of breach and the outflow from the breach are also important considerations. This means that a two-dimensional model (width of eroded area not considered) is not adequate, and erosion following the initial breach needs to be considered. This will require the addition of a model component to track headcut width during breach development and the quantification of at least two additional phases. These additional phases are the downward erosion of the crest of the vertical following submergence of the headcut and the widening of the headcut following complete local removal of the embankment in the vicinity of the breach. Research presently underway includes breaching of embankments such as that shown in Figure 3, and will assist in quantifying the action that occurs during these additional phases. Laboratory tests confirm that material properties may have a major impact on the rate of headcut advance, and therefore time to breach and breach rate. The headcut erodibility index based relations used in the spillway erosion model are semi-empirical and were developed to cover a broad range of geologic conditions. They were also developed without consideration of such things as pore water changes with position of the headcut. Therefore, as discussed by Hanson et al 2001, it should be possible to either refine the relations for application to embankment conditions or to replace this approach with an alternate. Work is continuing in this area. The focus to date has been on homogeneous embankments, but plans are being made to expand the testing program to include other types.

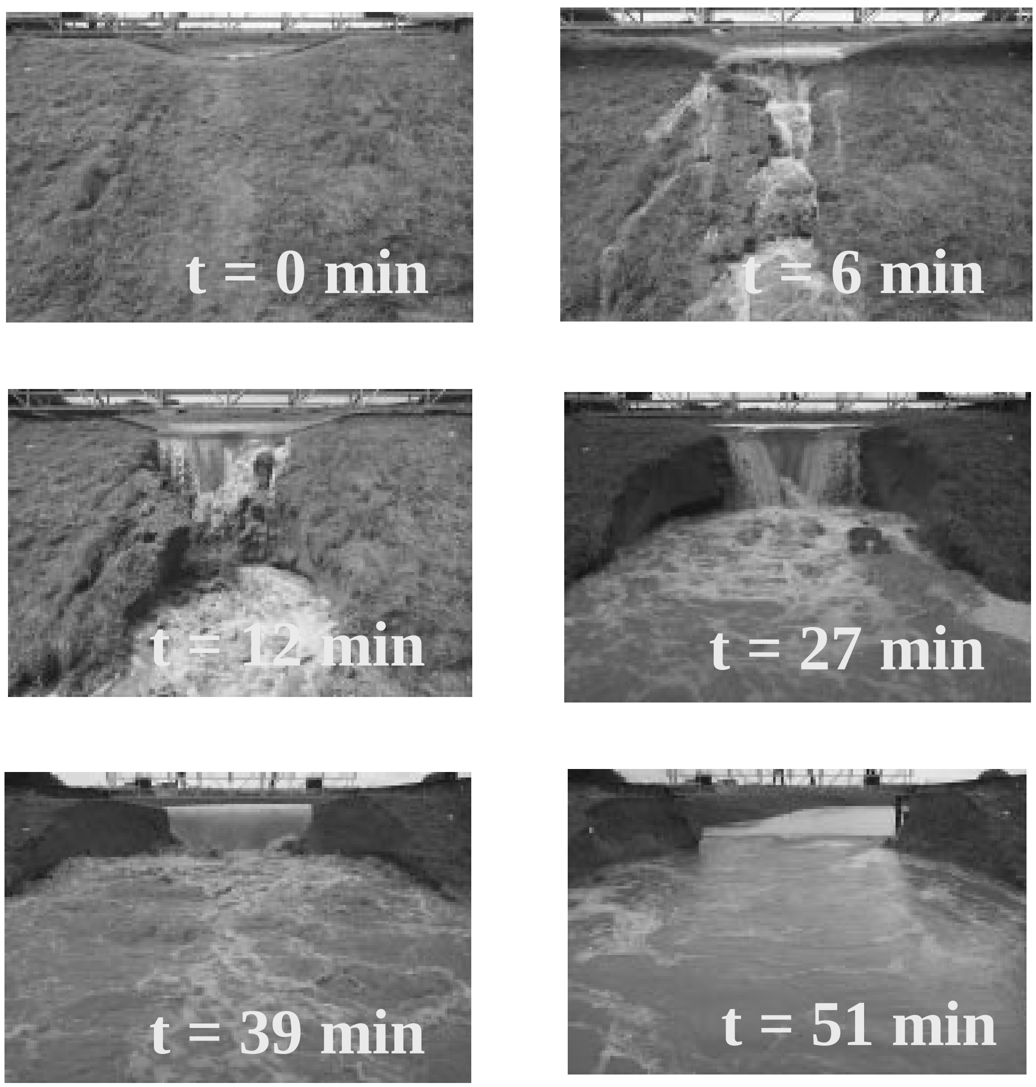

Figure 3. Time series of an embankment breach test of a homogeneous non-plastic sandy soil conducted at the ARS Hydraulic Laboratory, Stillwater, OK.

SUMMARY

Research efforts at the ARS Plant Science and Water Conservation Research Laboratory have resulted in an increased understanding of the erosion processes applicable to an overtopped earth embankment. Advances in predicting performance of vegetated earth spillways form a point of beginning for quantifying the breach process for embankments in a fashion that includes prediction of the extent of overtopping that may occur without breach, and the time of breach when breach does occur. However, the present spillway model is not considered adequate for this application.

Additional research is being conducted to allow existing erosion models to be refined and extended. With respect to the earth spillway erosion model discussed, this involves refinement of existing headcut erosion components and development of additional components to address the latter stages of breach development, breach widening and breach discharge prediction. Research presently underway will contribute to development of these components.

Research on the ability of un-reinforced vegetation to protect embankment faces has shown that grass can substantially increase the time to breach. However, taking advantage of this capability will require that attention be given to maintenance of the cover and to protecting areas of concentrated attack such as the slope toe.

Although the research described in this paper focuses on the performance of smaller dams of the type constructed with the assistance of the United States Department of Agriculture, the results may also be used to better understand the response of larger earth dams and will compliment results of research on breach of large dams such as that being carried out under the CADAM project (European Commission, 1998a, 1998b, 1999a, 1999b, 2000). This report discusses the approach being used in USDA research, some of the key underlying physical processes that must be considered, and the progress being made.

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B-19

HEC Models for Dam Break Flood Routing

by

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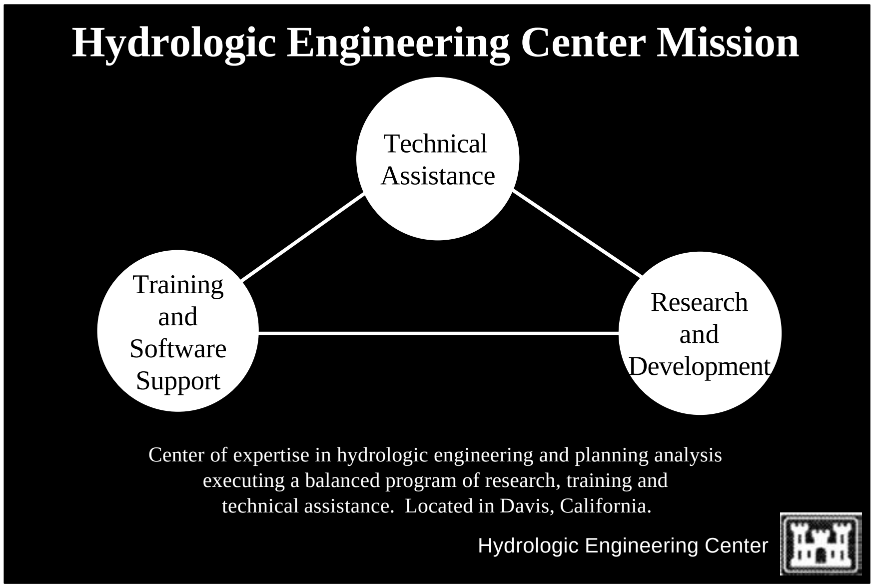


HEC Products

- Hydrologic engineering software; Corps, Public, International
- Technical Methods and Guidance
- Technical Assistance
- Prototype studies, Research and Applications
- Training Courses, Workshops and Seminars

Some HEC History

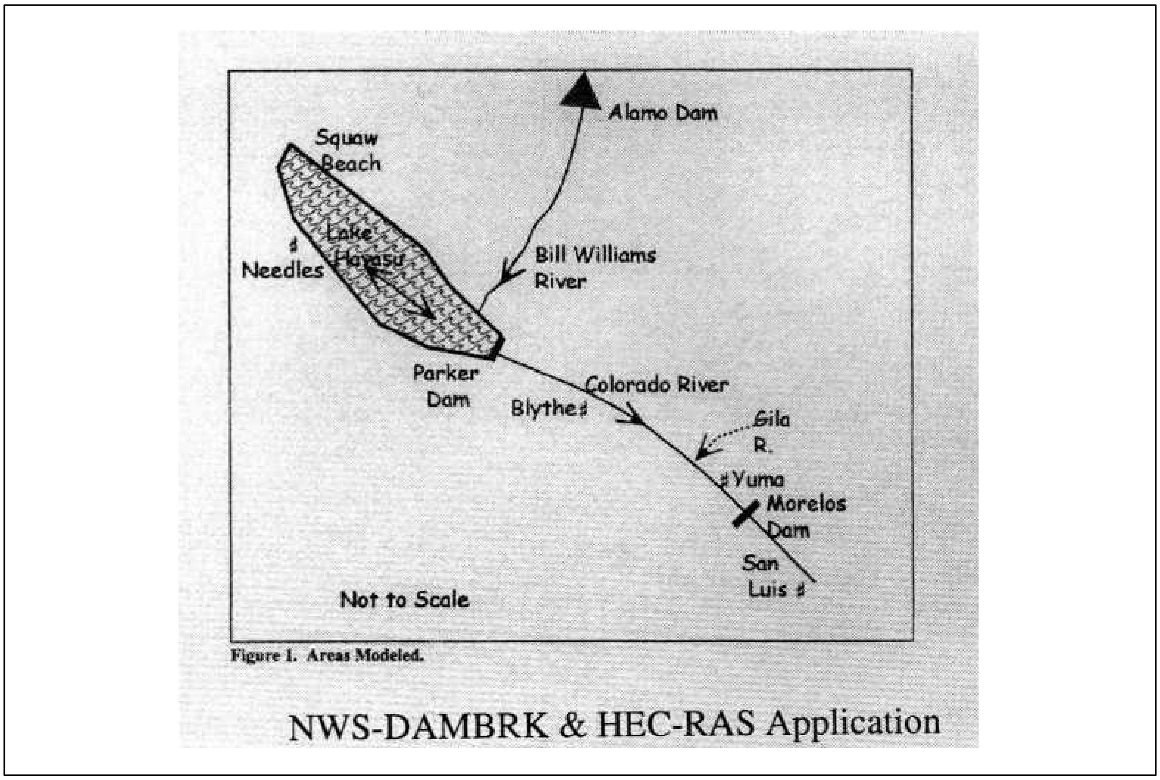
- **80's** - Simplified techniques, test routing methods
- **90's**
  - NexGen HEC-RAS development for 1-D steady flow
  - UNET (Mississippi Basin Modeling System for forecasting)
  - R & U (Alamo Dam, used combination of DAMBRK & HEC-RAS
- **2000's**
  - HEC-RAS (unsteady flow)
  - CWMS (Corp Water Management System)

References

In the early '80's HEC looked at using the TVA explicit model for unsteady flow applications. At that time, a geometric pre-processing program (GEDA) was developed to compute data tables of geometric properties for USTFLO from HEC-2 format cross-sections. This capability was later expanded to prepare DAMBRK and DWOPER geometric data from HEC-2 cross sections. Research was conducted regarding selection of appropriate flood routing procedures (HEC, 1980a) and generalized solutions to dam break flood routing (HEC, 1980b).

UNET (HEC, 2001) has been routinely utilized throughout the Corps for 1-D unsteady flow modeling for at least the last fifteen years. Many features have been developed by Dr. Barkau for local needs (HEC, 1998). Of particular interest and continuing research are issues related to calibration - both hydrology related (what is the real flow hydrograph) and hydraulics related (what is the appropriate roughness function for the observed stage hydrograph and input flow hydrograph). Major developments to UNET (levee breach connections to off-channel storage areas, etc.) were prompted by large floods in the Mississippi-Missouri system in '93.

Current HEC work involves incorporation of the UNET unsteady flow equation solver into HEC-RAS. This allows the more complete geometric description of the river used by RAS to be used as well as RAS' graphical displays and data editing capabilities. RAS unsteady flow modeling will support the Corps Water Management System (HEC, 2000b).



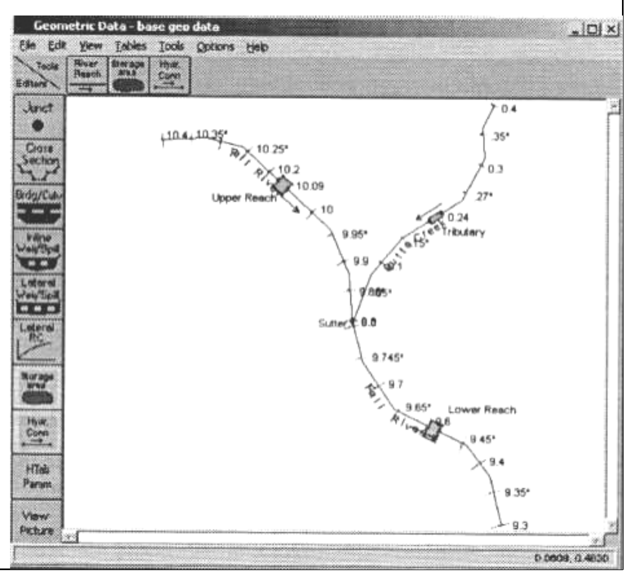
Schematic of the Alamo Dam study area. A DAMBRK model had been developed of this system by the Seattle Dist. Of the Corps (for the L.A. Dist.). This model was used to evaluate additional failure scenarios. The DAMBRK cross sections were converted into RAS sections so that overbank depths and velocities could be computed. RAS was run as a steady flow model using peak flows at each section computed by DAMBRK (RAC, 1999).

Steps to Develop RAS Data

- Start a New Project
- Enter Geometric data
- Enter Flow and Boundary data
- Establish a Plan and Run
- Evaluate model results
- Adjust model, as necessary

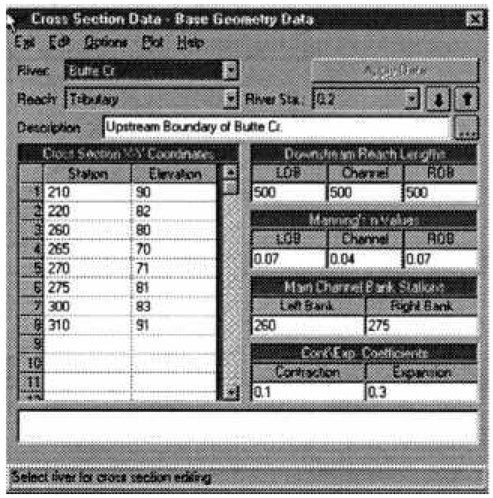
HEC-RAS Geometric Data

- River
- Reach
- Junctions
- River Stations
  - Cross section data entry



Cross-section Data Editor

- Option: Add Section
- River-Reach-Station
- Input section data
  - Station/Elevation Data
  - Reach lengths
  - Manning's *n*
  - Bank Stations
  - Contract/Expand Coef.



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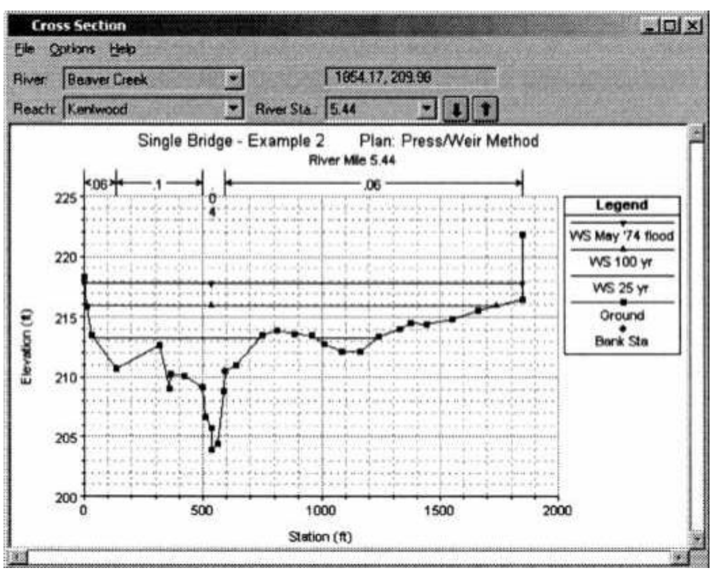
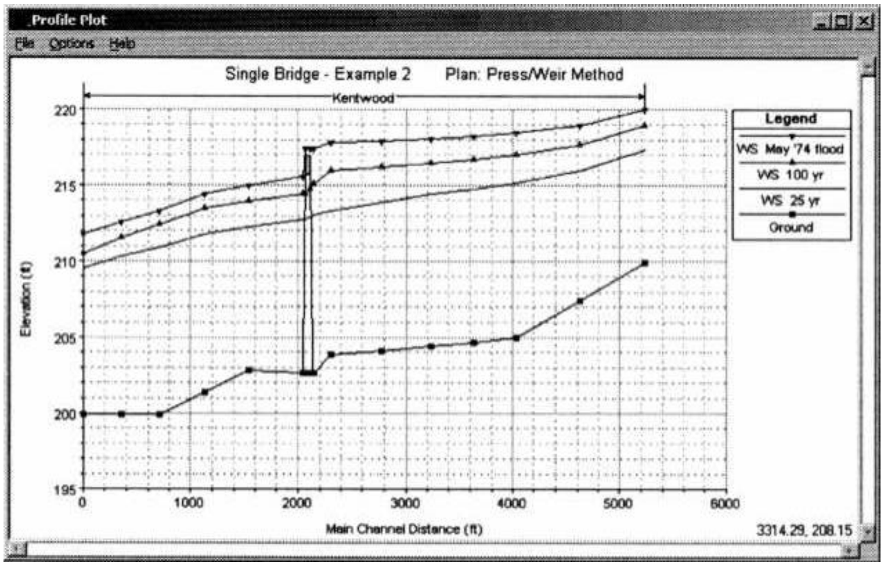
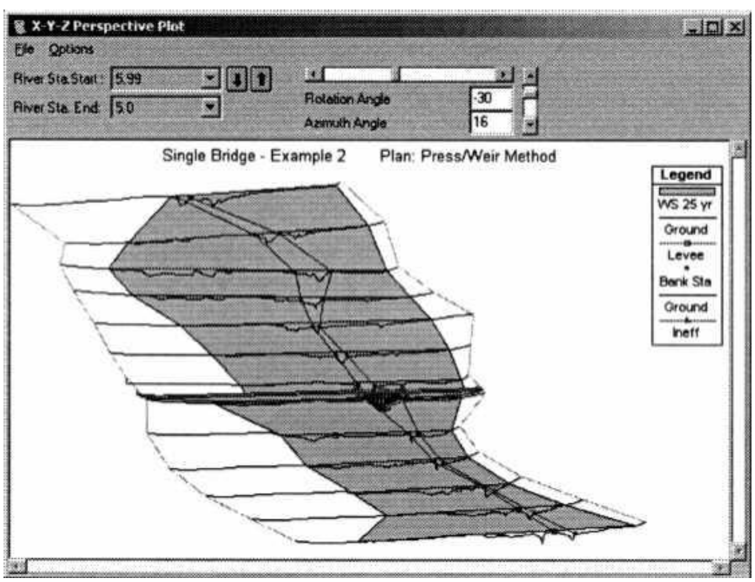
Cross-section Plot



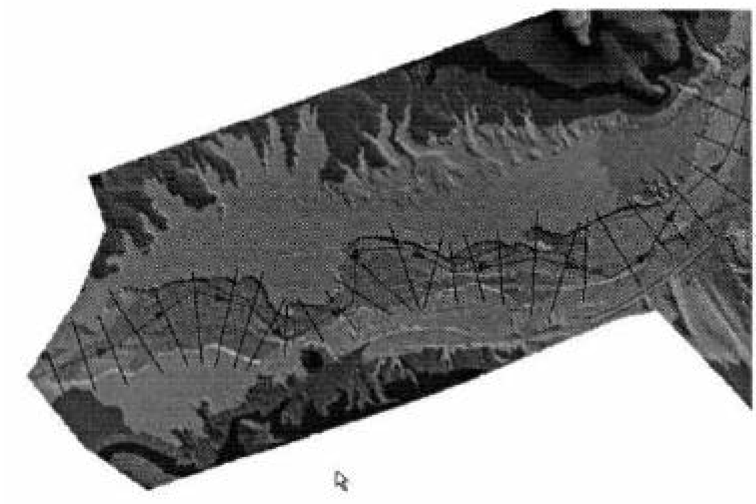
Profile Plot



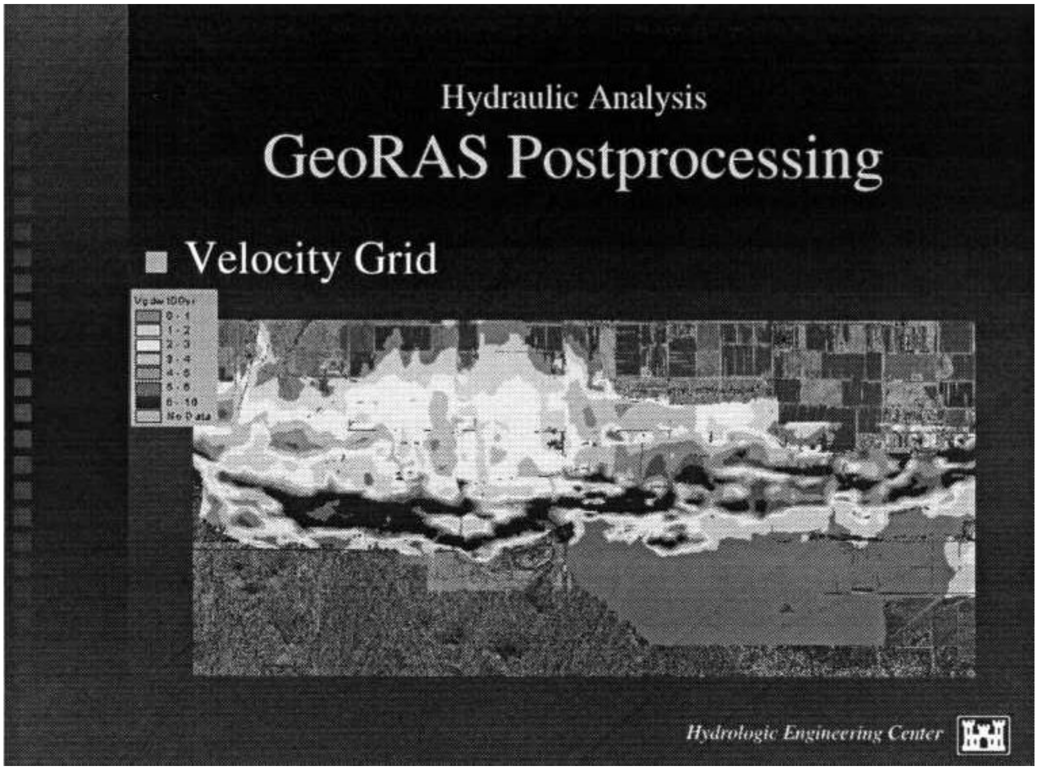
XYZ Perspective Plot



Use of GIS/DEM data



RAS cross section data can be developed from a digital terrain model using HEC-GeoRAS, which is an extension to ArcView (HEC, 2000a). This figure illustrates how the stream centerline and cross section strike lines are chosen by the user. This example is Las Vegas Wash.



Results of HEC-RAS computations can be viewed using the GIS/DEM data representation that was used to construct input data. The results that can be displayed (mapped) include traditional inundated areas for flows modeled as well as depth and velocity distributions. Ongoing work to extend HEC-RAS for sediment transport analysis will utilize this information to compute and display transverse distributions of bed shear stress and stream power (based upon the local grain size). This example is the Salt River near Phoenix, AZ.

RAS Unsteady Flow

- Overview
- New Geometric Features for RAS 3.0
- Geometric pre-processor
- Boundary and initial conditions
- Unsteady flow simulation manager
- Post-processor
- Additional graphics/tables to view results

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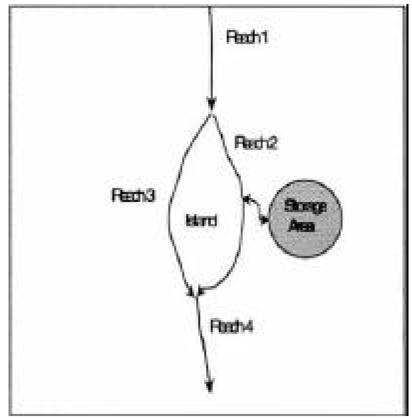
Overview

- Common geometry and hydraulic computations for steady & unsteady flow
- Using the UNET equation solver (Dr. Robert Barkau)
- Can handle simple dendritic streams to complex networks
- Able to handle a wide variety of hydraulic structures
- Extremely fast matrix solver

New Geometric Features for HEC-RAS

- Existing Geometric Features all work for unsteady flow (XS, bridges, Culverts, inline weirs/spillways)
- Lateral Weirs/Spillways
- Storage Areas
- Hydraulic Connections (weirs, gated spillways, and culverts)



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All of the existing hydraulic analysis features in the previous steady flow versions of HEC-RAS work within the new unsteady flow computation. The following new features were added to work with unsteady flow, but they also work in the steady flow simulation:

- Lateral weirs/gated spillways.
- Storage areas: used to model areas of ponded water.
- Hydraulic connections: use to exchange water between storage areas, storage areas and a river reach, and between different river reaches.

Pre-processing Geometry

- For unsteady flow, geometry is pre-processed into tables and rating curves
  - **Cross sections** are processed into tables of area, conveyance, and storage
  - **Bridges and culverts** are processed into a family of rating curves for each structure
  - **Weirs and gated structures** are calculated on the fly during unsteady flow calculations
  - Pre-processor **results can be viewed** in graphs and tables

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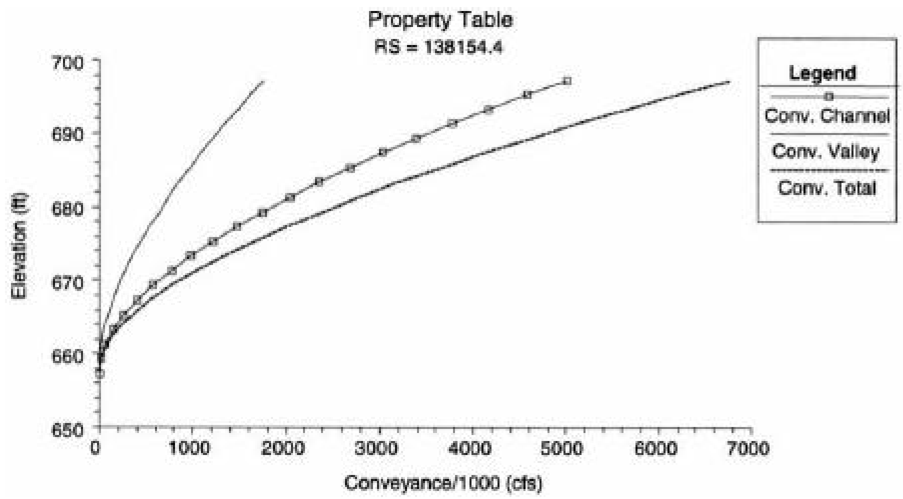
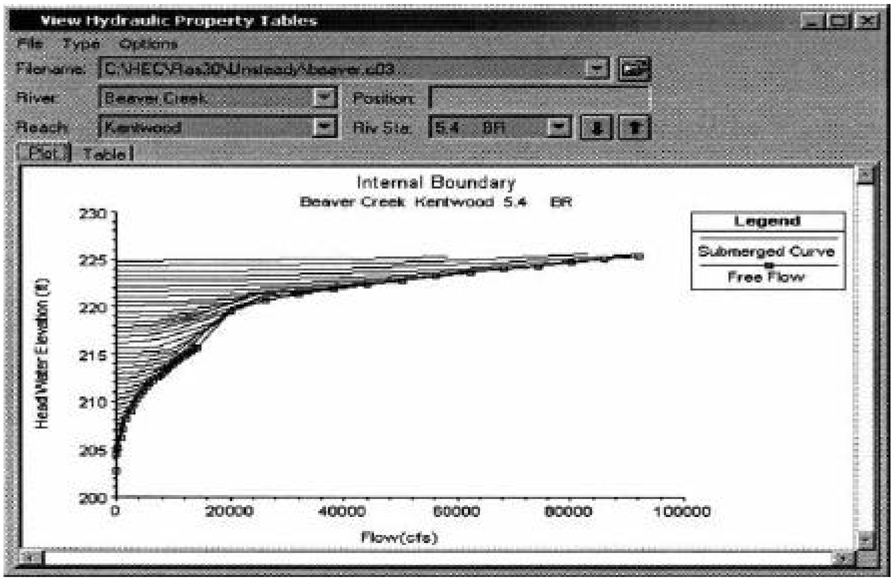
The pre-processor is used to process the geometric data into a series of hydraulic properties tables and rating curves. This is done in order to speed up the unsteady flow calculations. Instead of calculating hydraulic variables for each cross-section during each iteration, the program interpolates the hydraulic variables from the tables. **The pre-processor must be executed at least once, but then only needs to be re-executed if something in the geometric data has changed.**

Cross Section Properties Plot



Cross sections are processed into tables of elevation versus hydraulic properties of areas, conveyances, and storage. Each table contains a minimum of 21 points (a zero point at the invert and 20 computed values). The user is required to set an interval to be used for spacing the points in the cross section tables. The interval can be the same for all cross sections or it can vary from cross section to cross section. This interval is very important, in that it will define the limits of the table that is built for each cross section. On one hand, the interval must be large enough to encompass the full range of stages that may be incurred during the unsteady flow simulations. On the other hand, if the interval is too large, the tables will not have enough detail to accurately depict changes in area, conveyance, and storage with respect to elevation.

Bridge Hydraulic Properties Plot



Hydraulic structures, such as bridges and culverts, are converted into families of rating curves that describe the structure as a function of tailwater, flow and headwater. The user can set several parameters that can be used in defining the curves.

Boundary and Initial Conditions

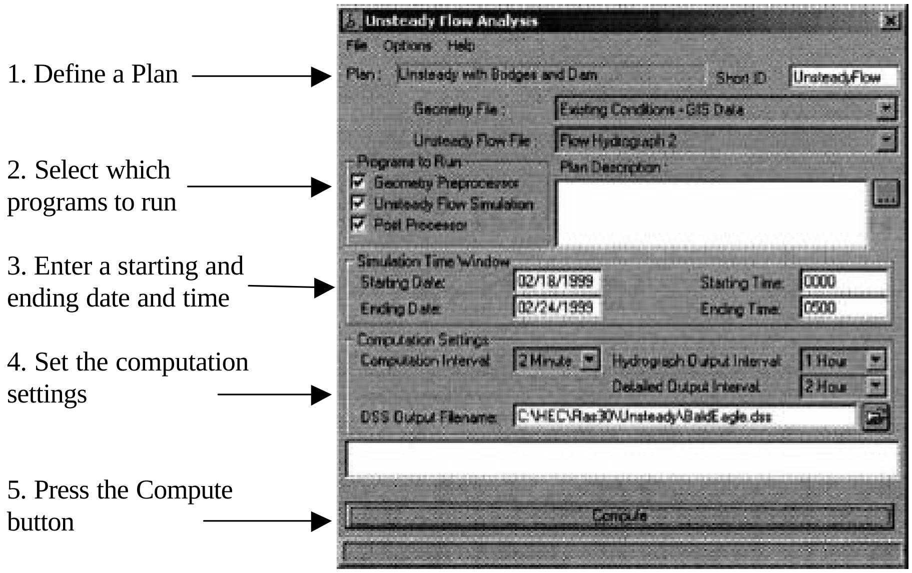
- Boundary conditions must be established at all ends of the river system:
  - Flow hydrograph
  - Stage hydrograph
  - Flow and stage hydrograph
  - Rating curve
  - Normal depth

The user is required to enter boundary conditions at all of the external boundaries of the system, as well as any desired internal locations, and set the initial flow and storage area conditions in the system at the beginning of the simulation period.

Boundary and Initial Conditions

- Interior boundary conditions can also be defined within the river system:
  - Lateral inflow to a node
  - Uniform lateral inflow across a reach
  - Ground water inflow
  - Time series of gate openings
  - Elevation controlled gate
  - Observed internal stage and/or flow hydrograph

Unsteady Flow Simulation Manager



Once all of the geometry and unsteady flow data have been entered, the user can begin performing the unsteady flow calculations. To run the simulation, go to the HEC-RAS main window and select **Unsteady Flow Analysis** from the **Run** menu.

The unsteady flow computations within HEC-RAS are performed by a modified version of UNET (HEC, 2001). The unsteady flow simulation is actually a three step process. First a program called RDSS (Read DSS data) runs. This software reads data from a HEC-DSS file and converts it into the user specified computation interval. Next, the UNET program runs. This software reads the hydraulic properties tables computed by the preprocessor, as well as the boundary conditions and flow data from the interface and the RDSS program. The program then performs the unsteady flow calculations. The final step is a program called TABLE. This software takes the results from the UNET unsteady flow run and writes them to a HEC-DSS file.

Post-processing Results

- Used to compute detailed hydraulic information for a set of user-specified times and an overall maximum water surface profile.
- Computed stages and flows are passed to the steady flow program for the computation of detailed hydraulic results

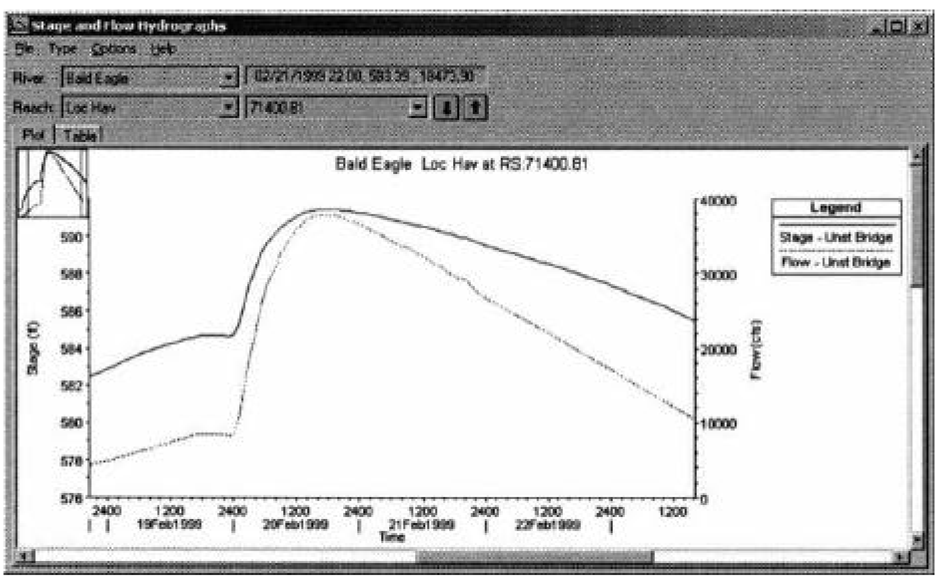
The Post Processor is used to compute detailed hydraulic information for a set of user specified time lines during the unsteady flow simulation period. In general, the UNET program only computes stage and flow hydrographs at user specified locations. **If the Post Processor is not run, then the user will only be able to view the stage and flow hydrographs and no other output from HEC-RAS**. By running the Post Processor, the user will have all of the available plots and tables for unsteady flow that HEC-RAS normally produces for steady flow.

When the Post-Processor runs, the program reads from HEC-DSS the maximum water surface profile (stages and flows) and the instantaneous profiles. These computed stages and flow are sent to the HEC-RAS steady flow computation program SNET. Because the stages are already computed, the SNET program does not need to calculate a stage, but it does calculate all of the hydraulic variables that are normally computed. This consists of over two hundred hydraulic variables that are computed at each cross section for each flow and stage.

Viewing Unsteady Flow Results

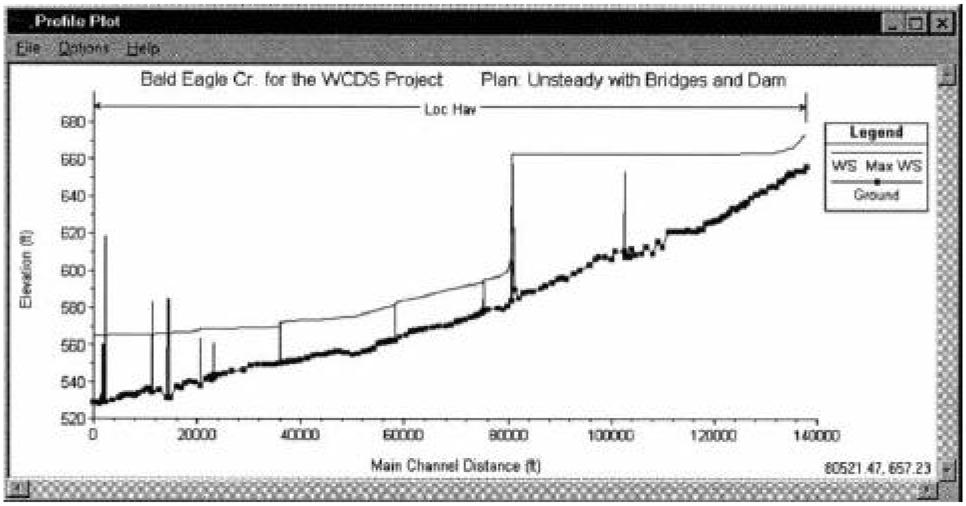
- All of the output that was available for steady flow computations is available for unsteady flow (cross sections, profile, and perspective plots and tables).
- Stage and flow hydrographs
- Time series tables
- Animation of cross section, profile and perspective graphs

Stage and Flow Hydrographs



The stage and flow hydrograph plotter allows the user to plot flow hydrographs, stage hydrographs, or both simultaneously. Additionally, if the user has observed hydrograph data, that can also be plotted at the same time. The plot can be printed or sent to the windows clipboard for use in other software.

Animation of Profile Plot



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The HEC-RAS software has the ability to animate the cross section profile. When the user selects Animation, the plot steps thought the computed results in a timed sequence. The user can control the speed of the animation, as well as step through individual time steps.

Application of HEC-RAS to a Dam Break Situation

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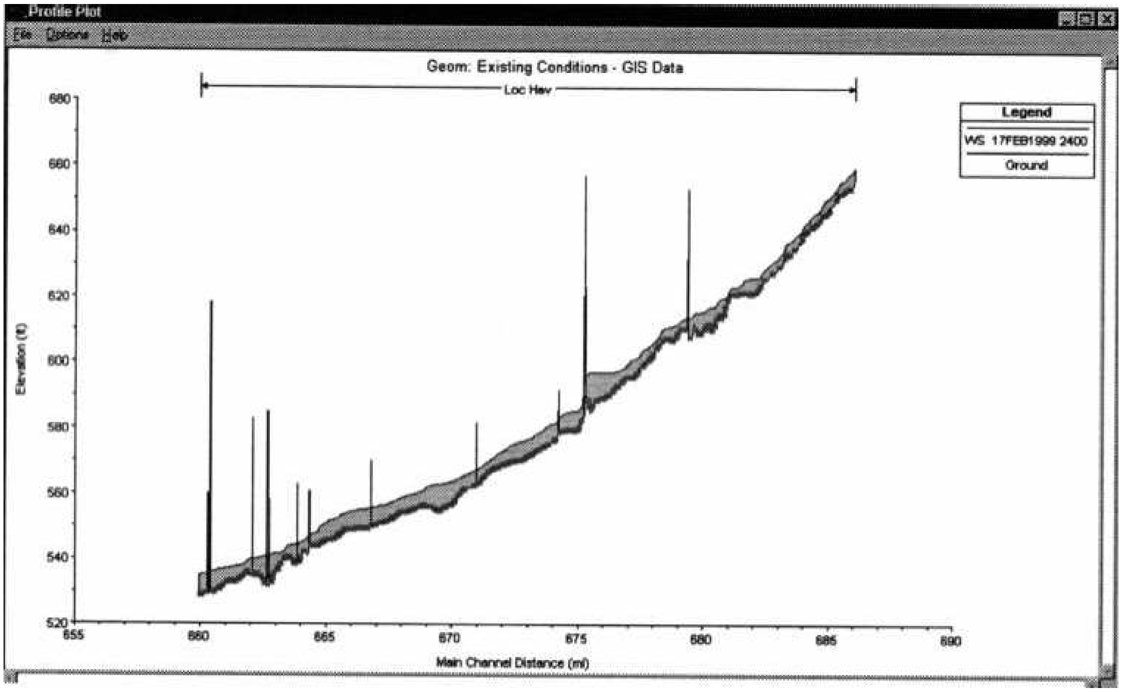
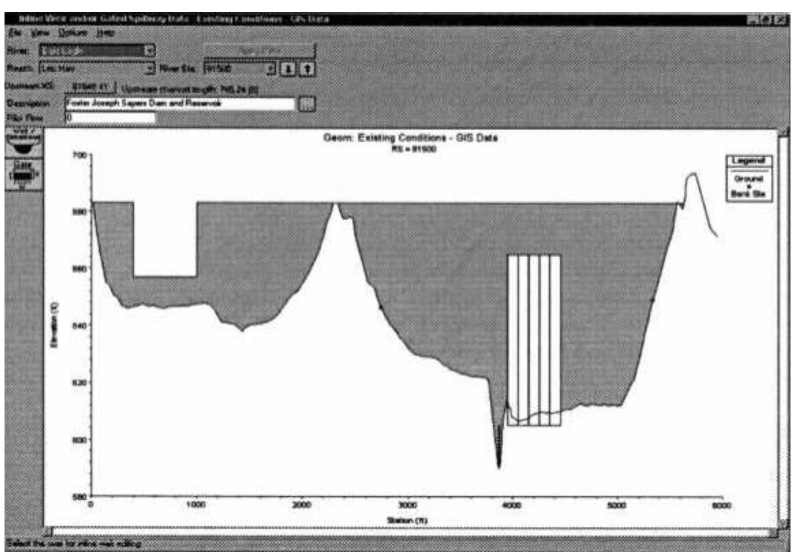
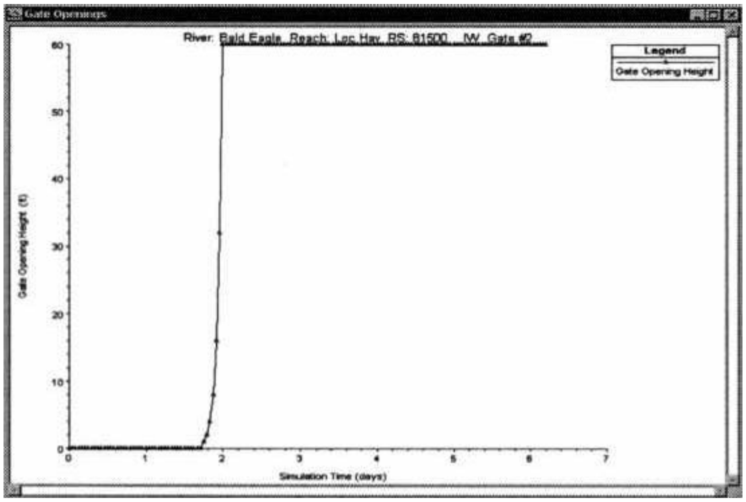
HEC-RAS (or UNET) can be used to simulate the unsteady routing of flood hydrographs resulting from breaching of dams or levees. The user has many methods available within the programs to generate the hydrographs. In this example, a gate operation is used to mimic the failure of a dam embankment. UNET has the ability to compute flows through levee and embankment breaches.



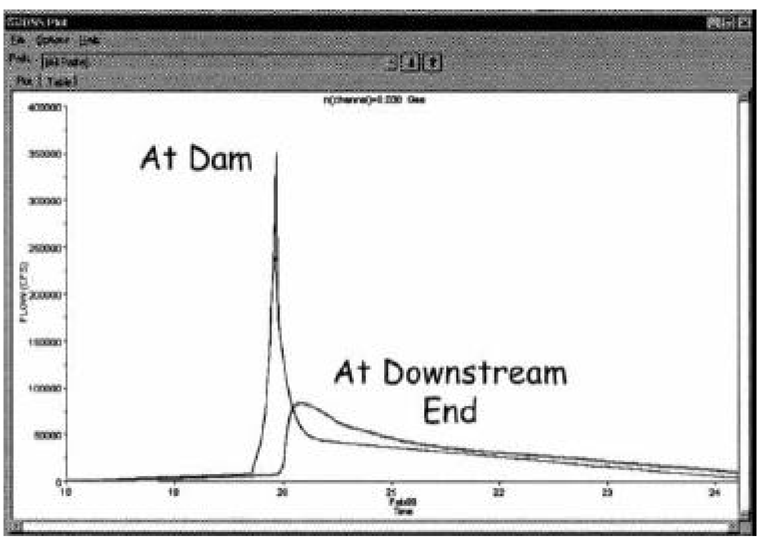
Gate Section



Gate Operation



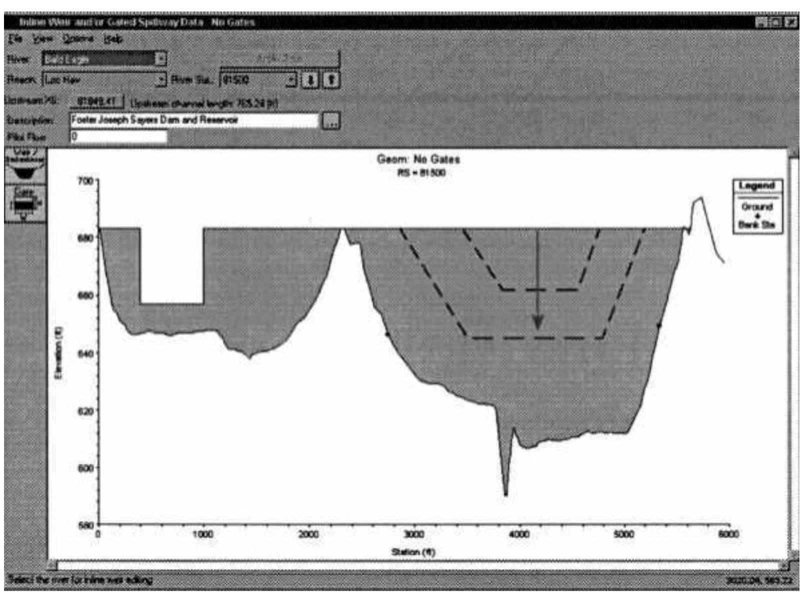
Computed Hydrographs



Future Work

- Dam & Levee Breaching
  - Overtopping
    - Initiation via
      - Water surface elevation
      - Clock (simulation) time
    - Growth rate
      - Linear
      - Exponential
    - Use weir equations with submergence

Weir Type Breach



Future Work (cont.)

- Dam & Levee Breaching
  - Piping
    - Initiation via
      - Water surface elevation
      - Clock (simulation) time
    - Progression
      - Box until top collapses (when top elev. > W.S.)
    - Orifice flow transitioning to weir flow

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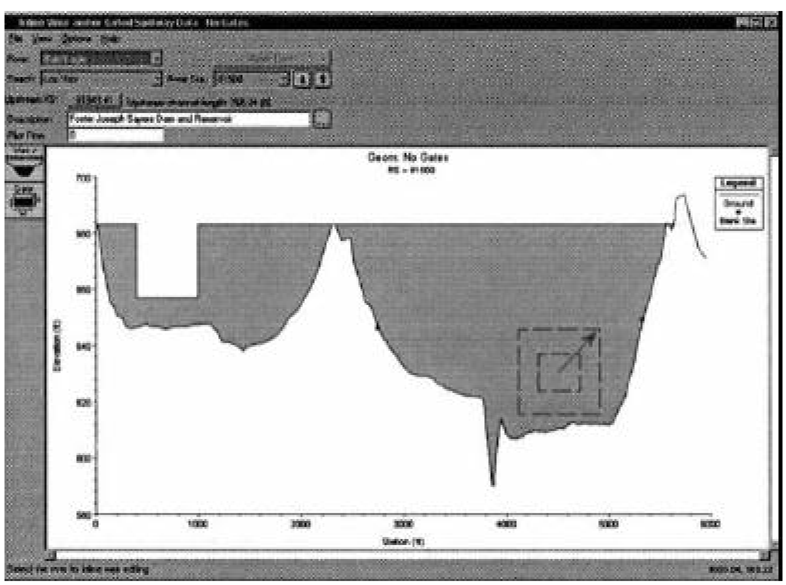
Piping Type Breach



Product Availability

- Internal testing (Teton, MBMS etc.) this summer
- General release of HEC-RAS 3.1 - Fall of 2001
- Same breaching algorithms to be used in HEC-HMS

Hydrologic Engineering Center

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B-20

CADAM & IMPACT: European Research Projects Investigating Dambreak Modelling and Extreme Flood Processes

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

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SYNOPSIS

This paper provides an overview of the CADAM Concerted Action project, which was completed in January 2000, and an introduction to the IMPACT research project which will commence November / December 2001. Both projects have been funded by the European Commission, with the IMPACT project addressing key research issues identified during the CADAM concerted action project.

The *Concerted Action project on Dambreak Modelling* (*CADAM)* involved participants from 10 different countries across Europe and ran between Feb 98 and. Jan 2000. The aim of the project was to review dambreak modelling codes and practice from first principles through to application, to try and identify modelling best practice, effectiveness of codes and research needs. Topics covered included the analysis and modelling of flood wave propagation, breaching of embankments and dambreak sediment effects. The programme of study was such that the performance of modelling codes were compared against progressively more complex conditions from simple flume tests through physical models of real valleys and finally to a real dam-break test case (the Malpasset failure). The study conclusions are presented in a final project report, published by both the EC and the IAHR. This paper provides a brief summary of the key issues identified.

The IMPACT project *(Investigation of Extreme Flood Processes & Uncertainty)* focuses research in a number of key areas that were identified during the CADAM project as contributing greatly to uncertainty in dambreak and extreme flood predictions. Research areas include embankment breach (formation and location), flood propagation (infrastructure interaction and urban flooding) and sediment movement (near and far zones with respect to embankment failure). The uncertainty associated with current predictive models and following project research will be demonstrated through application to case study material. Implications of prediction uncertainty for end users with applications such as asset management and emergency planning will also be investigated.

CADAM – AN INTRODUCTION

The first legislation in Europe for dam-break risk analysis was presented in France in 1968, following the 1959 Malpasset dam-break that was responsible for more than 400 injuries. Since then, and especially more recently, many European countries have established legal requirements. However the techniques applied when undertaking the specified work can vary greatly. The perception of risks related to natural or industrial disasters has also evolved, leading to public demand for higher standards of safety and risk assessment studies. Considering the relatively high mean population density within Europe, a dam-break incident could result in considerable injury and damage; efficient emergency planning is therefore essential to avoid or minimise potential impacts.

Dam-break analyses therefore play an essential role when considering reservoir safety, both for developing emergency plans for existing structures and in focussing planning issues for new ones. The rapid and continuing development of computing power and techniques during the last 15 years has allowed significant advances in the numerical modelling techniques that may be applied to dam-break analysis.

CADAM was funded by the European Commission as a Concerted Action Programme that ran for a period of two years from February 1998. Under these terms, funding was provided only to pay for travel and subsistence costs for meetings, and for project co-ordination. All work undertaken during the study was therefore achieved through the integration of existing university and national research projects. HR Wallingford co-ordinated the project, with additional financial support from the DETR.

The project continued work started by the IAHR Working Group (established by Alain Petitjean following the IAHR Congress in 1995) and had the following aims:

- The exchange of dam-break modelling information between participants, with a special emphasis on the links between Universities, Research Organisations and Industry.
- To promote the comparison of numerical dam-break models and modelling procedures with analytical, experimental and field data.
- To promote the comparison and validation of software packages developed or used by the participants.
- To define and promote co-operative research.

These aims were pursued through a number of objectives:

- To establish needs of industry, considering a means of identifying dam owners, operators, inspectors etc. throughout Europe.
- To link research with industry needs - encourage participation; distribute newsletters to dam owners and other interested parties.

- To create a database of test cases (analytical, experimental, real life) available for reference.
- To establish the state-of-the-art guidelines and current best practices for dam-break modelling within the technical scope of the Concerted Action. This leads towards establishing recommended European standard methods, procedures and practices for dam-break assessments.
- To determine future RTD requirements.

CONCERTED ACTION PROGRAMME

The project involved participants from over 10 different countries across Europe. All member states were encouraged to participate, with attendance at the programme workshops open to all and to expert meetings by invitation. Also, links with other experts around the world were welcomed to ensure that state-of-the-art techniques and practices were considered. The programme of meetings planned for the presentation, discussion and dissemination of results and information were as follows:

Meeting 1 Wallingford, UK. 2/3rd March 98(Expert Meeting)
A review of test cases and modelling work undertaken by the group up to the start of CADAM, followed by a review of test cases considered during the previous 6 months. Typical test cases included flood wave propagation around bends, over obstructions and spreading on a flat surface (physical modelling undertaken in laboratory flumes).

Meeting 2 Munich, Germany 8/9th October 98(Open Workshop)
Presentations and discussion on the current state of the art in breach formation modelling and sediment transport during dam-break events.

Meeting 3 Milan, Italy May6/7th 99(Expert Meeting)
Comparison and analysis of numerical model performance against a physical model of a real valley (Toce River, Italy) plus an update on breach modelling research.

Meeting 4 Zaragoza, Spain Nov 18/19th 99(Symposium)
Comparison and analysis of numerical model performance against a real failure test case (Malpasset failure) plus a presentation of the results and conclusions drawn from the work of the *Concerted Action* over the two-year study period.

MODELLING COMPARISONS

The programme of tests progressed from simple conditions to test the basic numerical stability of modelling codes, through to a real dambreak test case – the Malpasset failure. The aim of the programme was to progressively increase the complexity of the modelling, and in doing so to try and identify which models performed best under which conditions. Both breach models and flood routing models were considered during the project.

Flood Routing Analysis

Numerical Models The models applied in CADAM ranged from commercially available software to codes developed ‘in-house’ by the various participants. Participants ranged from ‘End User’ organisations such as ENEL (Italy), EDF (France) and Vattenfall (Sweden) to consultancy companies and universities undertaking research in this field. Many of the European participants codes were 2D codes based on depth averaged Saint Venant shallow water equations, but applying different numerical schemes utilising different orders of accuracy and source term implementations. Codes more familiar to the UK market included DAMBRK and ISIS (1D model - implicit finite difference Preissmann Scheme).

Analytical Tests Initial test cases were relatively simple, with analytical solutions against which the numerical modelling results could be compared. These tests included:

- Flume with vertical sides, varying bed level and width. No flow – water at rest.
- Flume with (submerged) rectangular shaped bump. Steady flow conditions.
- Dam-break flow along horizontal, rectangular flume with a dry bed. No friction used.
- Dam-break flow along horizontal, rectangular flume with a wet bed. No friction used.
- Dam-break flow along horizontal, rectangular flume with a dry bed. Friction used.

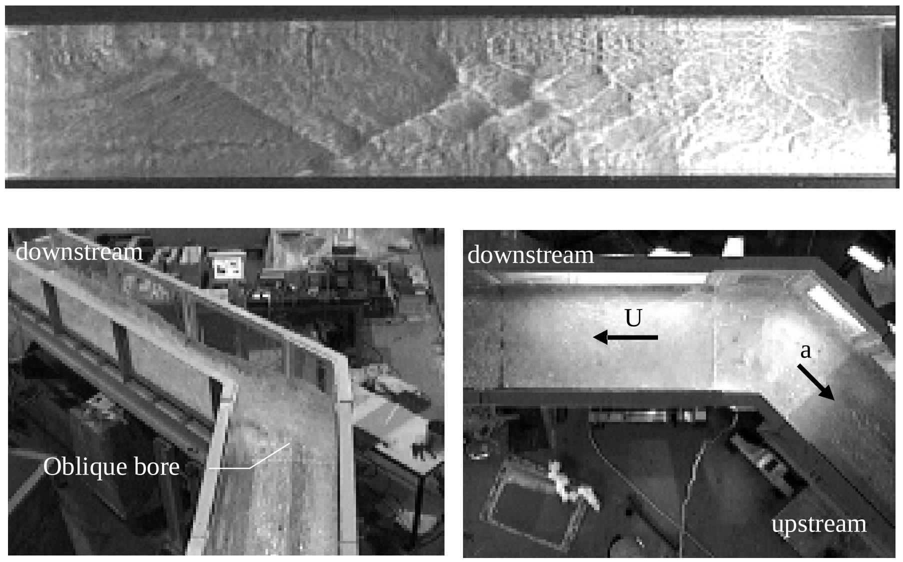
These tests were designed to create and expose numerical 'difficulties' including shock waves, dry fronts, source terms, numerical diffusion and sonic points. Results were presented and discussed at the 2nd IAHR Working Group meeting held in Lisbon, Nov. 96 (EDF, 1997).

Flume Tests Following the analytical tests, a series of more complex tests were devised for which physical models provided data (Fig 1). The aim was to check the ability of the numerical codes to handle firstly, specific 2D features, and then important source terms. These tests were:

- Dam-break wave along a rectangular flume with 90° bend to the left.
- Dam-break wave along a rectangular flume with a symmetrical channel constriction.
- Dam-break wave along a rectangular flume expanding onto a wider channel (asymmetrical).
- Dam-break wave along a rectangular flume with 45° bend to the left.

- Dam-break wave along a rectangular flume with a triangular (weir type) obstruction to flow.

The first three test cases were presented and discussed at the 3rd IAHR working Group meeting in Brussels (UCL, June 97) and the remaining two at the 1st CADAM meeting in Wallingford (CADAM, March 98).



Photos courtesy of Sandra Soares, Université Catholique de Louvain, Belgium

Fig. 1 Shock waves generated from 'dambreak' flow around a 45° bend.

'<u>Real Valley' Physical Model</u> A model of the Toce River in Italy was used for the analysis of model performance against 'real valley' conditions (Fig 2). The advantage of comparing the numerical models against a physical model, at this stage in the project, was that the model data would not include any effects from sediment or debris that might mask features of numerical model performance.

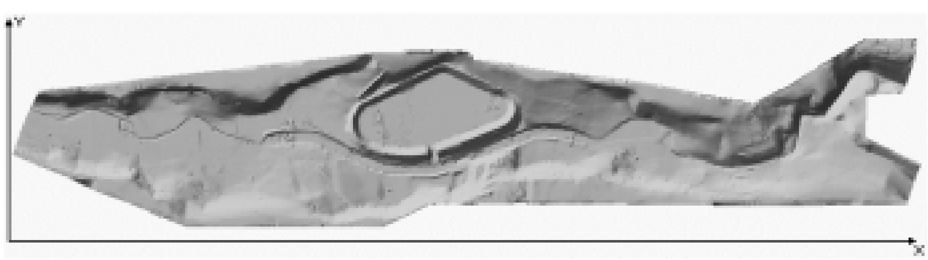


Fig. 2 Digital plan model showing the Toce River model

The model was provided by ENEL and, at a scale of 1:100, represented a 5km stretch of the Toce River, downstream of a large reservoir. An automated valve controlled flow in the model such that a flood hydrograph simulating partial or total dam failure could be simulated. Features within the downstream valley included a storage reservoir, barrage, bridges and villages (Fig 3).

Photos courtesy of Prof JM Hiver, Université Libre de Bruxelles, Belgium

Fig. 3 Bridge structure on the Toce model and a dambreak flow simulation

<u>Real Failure Test Case – The Malpasset Failure</u> The Malpasset Dam failure was selected as the real case study for the project since:

- The data was readily available through EDF (France)
- It offered a different data set to the commonly used Teton failure
- In addition to field observations for peak flood levels there were also timings for the failure of three power supply centres
- Data from a physical model study undertaken by EDF in 1964 (Scale 1:400) was also available

Modelling focused on the first 15km of valley downstream of the dam for which there was field data to compare against model predictions. This stretch of valley included features such as steep sided valleys, side valleys / tributaries and bridge / road crossings.

<u>Breach Analysis</u>

One of the four CADAM meetings was devoted to breach formation and sediment and debris effects. A comparison of the performance of breach models was undertaken using two test cases. The first test case was based on physical modelling work performed at the Federal Armed Forces University in Munich. The simulation tested was for a homogeneous embankment represented by a physical model approximately 30cm high. The second test case was based on data from the Finnish Environment Institute, derived from past collaborative research work undertaken with the Chinese. This work analysed the failure of an embankment dam some 5.6m high (Loukola et al, 1993).

SELECTED RESEARCH FINDINGS

The following sections highlight some selected issues identified during the CADAM project:

Flood Routing

It was not possible to uniquely define a single best model or single best approach for dambreak modelling within the scope of the study since the various models and approaches performed differently under varying test conditions. Equally, a more in-depth analysis of the significant quantities of test data collected is now required to understand some of the performance features identified. It was possible, however, to identify some recurring features and issues that should be considered when defining best practice for dambreak modelling. These include (in no particular order):

Wave Arrival time The speed of propagation of the flood wave is an important component of dambreak modelling since it allows emergency planners to identify when inundation of a particular area may be expected. It was found that 1D and 2D models failed to reproduce this accurately and that 1D models consistently under predicted the time (i.e. flood wave propagated too quickly) and 2D models consistently over predicted this time (i.e. flood wave propagated too slowly).

Figure 4 shows wave travel times for one set of test data. The 1D models (left) show a scatter of results, probably due to the range of numerical methods applied. Results shown spread across the observed data. Later tests showed a tendency to under predict the wave speed. Many of the 2D models used similar numerical methods perhaps resulting in the tight clustering of data, however the results here (and repeated later) show a consistent over prediction of wave speed (right).

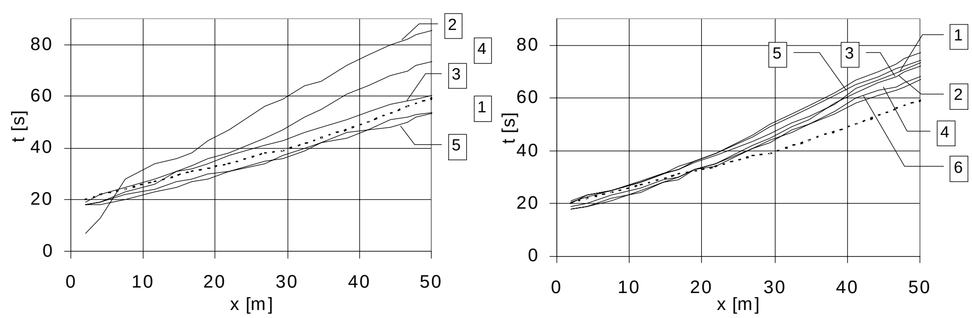


Fig. 4 Summary of flood wave travel times for 1D models (left) and 2D models (right)

Flood wave speed is poorly modelled – 1D models over predict wave speed, 2D models under predict wave speed.

<u>Use of 1D or 2D Models</u> It was found that the 1D models performed well in comparison with the 2D models for many of the test cases considered. It is clear, however, that there are instances where a 2D model predicts conditions more effectively than a 1D model. In these situations a 2D model should be used or the 1D model should be constructed to allow for 2D effects. These situations are where flow is predominantly 2 dimensional and include flow spreading across large flat areas (coastal plains, valley confluences etc), dead storage areas within valleys and highly meandering valleys. Simulation of these features using a 1D model will require experienced identification of flow features, reduction of flow cross section and addition of headloss along the channel.

A promising development that may offer a significant increase in model accuracy from a 1D model but without the heavy data processing requirements of a 2D model, is the use of a 'patched' model. This is where areas of 2D flow may be modelled using a 2D approach 'patched' within a 1D model (Fig 5). This technique requires further development and validation, but seems to offer significant potential.

In relation to the additional effort required for 2D modelling, 1D models perform well but cannot be relied upon to simulate truly 2D flow conditions. An experienced modeller is required to apply a 1D model correctly to simulate some 2D flow conditions.

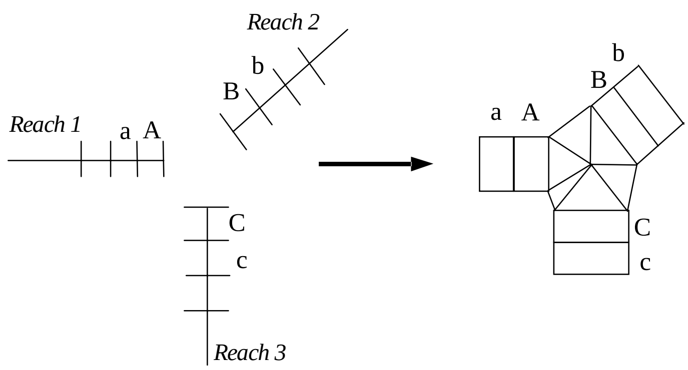


Fig. 5 2D patches within a 1D model to improve model accuracy whilst limiting processing requirements

<u>Modeller Assumptions</u> It was clear just from the test cases undertaken (and also supported by an independent study undertaken by the USBR (Graham, 1998)) that the assumptions made by modellers in setting up their models, can significantly affect the results produced. Graham (1998) deliberately gave identical topographic and structure data to two dambreak modellers and asked them to undertake independent dambreak studies for the same

site. The results varied significantly, and particularly in terms of flood wave arrival times. Variations in breach formation, valley roughness and simulation of structures contributed to the differences.

Modelling assumptions can significantly affect the model results. Different modellers may produce different results for an identical study. Care should be taken to ensure only experienced modellers are used and that all aspects and assumptions made are considered.

<u>Debris and Sediment Effects</u> It is unusual to find debris and sediment effects considered in detail for dambreak studies but it is clear from case studies and ongoing research that the movement of sediment and debris under dambreak conditions can be extreme and will significantly affect topography, which in turn affects potential flood levels. Case studies in the US have shown bed level variations in the order of 5 to 10m.

Debris and sediment effects can have a significant impact on flood water levels and should be considered during a dambreak study. These effects offer a significant source of error in flood prediction.

<u>Mesh Convergence</u> The definition of a model grid in 2D models, or the spacing of cross sections in 1D models, can significantly affect the predicted results. Models should be checked as a matter of routine to ensure that the grid spacing is appropriate for the conditions modelled and that further refinement does not significantly change the modelling results.

Mesh or section spacing should be routinely checked when modelling

<u>Breach Modelling</u>
Existing models are very limited in their ability to reliably predict discharge and the time of formation of breaches. Figure 6 below shows a typical scatter of modelling results found for the CADAM test cases. Models comprised a range of university and commercial codes, including the NWS BREACH code.

It is also clear that there is little guidance available on failure mechanisms of structures, which adds to the uncertainty of conditions assumed by modellers.

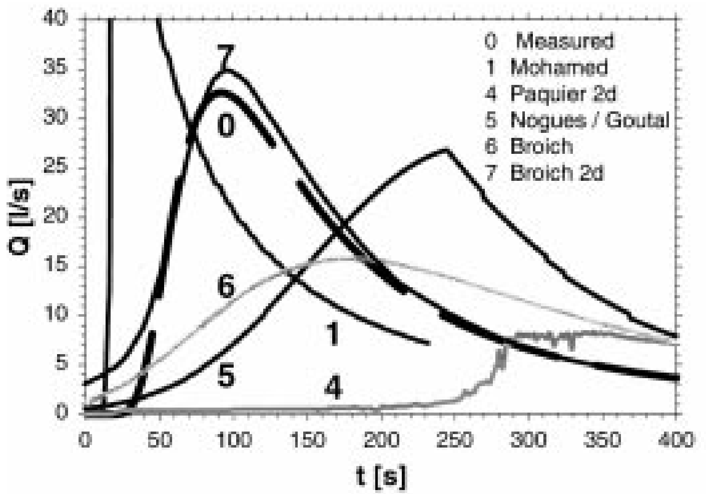


Fig. 6 Typical scatter of model results trying to predict breach formation

There are no existing breach models that can reliably predict breach formation through embankments. Discharge prediction may be within an order of magnitude, whilst the time of breach formation is even worse. Prediction of breach formation time due to a piping failure is not yet possible.

The NWS BREACH model is only calibrated against a very limited data set. The author (Danny Fread) confirmed that it is based on approximately 5 data sets.

Existing breach models should be used with caution and as an indicative tool only. A range of parameters and conditions should be modelled to assess model performance and results generated.

There is a clear need to develop more reliable predictive tools that are based on a combination of soil mechanics and hydraulic theory.

End User Needs

Throughout CADAM, the project focused on the practical needs of end users. Attempts were made to quantify a number of issues, both by end users and academic researchers alike. The initial response to the question of what accuracy models could offer and what was required from end users was limited. Without agreement on such issues it is impossible to determine whether existing modelling tools are sufficient or not! This perhaps reflects the current uncertainty of end users with regards to legislation and appropriate safety measures and of modeller's appreciation of processes and data accuracy. It was suggested that the level of modelling accuracy should be appropriate for the site in question (i.e. more detailed for urban areas). Water level prediction should be appropriate to the mapping required, and the mapping should be at a scale sufficient for emergency planning use (i.e. to identify flood levels in relation to individual properties). This suggests an inundation mapping scale of approximately 1:5000 for developed areas.

> Inundation maps should be undertaken at a scale appropriate for use in emergency planning. For urban areas it is suggested that this should be at a scale of 1:5000 or greater. Modelling accuracy should be consistent with the detail of mapping required (i.e. for the end user of the data)

Some Additional Points on DAMBRK_UK and BREACH

During the project, work undertaken by HR Wallingford identified a number of potential problems with the DAMBRK_UK and BREACH software packages.

Under certain conditions, it was found that the DAMBRK_UK package created artificial flow volume during the running of a simulation. For the limited conditions investigated this volume error was found to be as high as +13% (Mohamed (1998)). This error tended to be on the positive side, meaning that the flood levels predicted would be pessimistic. It may be assumed that similar errors exist in the original DAMBRK code. It was noted that model performance varied between DAMBRK, FLDWAV (released to replace DAMBRK) and BOSS DAMBRK. A detailed investigation into the magnitude and implications of these errors has not yet been undertaken.

Similarly, problems were also found with the BREACH software package. Under some conditions, predicted flood hydrographs were found to vary significantly with only minor modifications to input parameters. This erratic behaviour was discovered when considering the differences between piping and overtopping failure, by tending the piping location towards the crest of the dam. Erratic performance was also confirmed by a number of other CADAM members.

Figure 7 shows a plot of flood hydrographs generated by BREACH for an overtopping failure and a piping failure located just 3cm below the crest. Logic dictates that these hydrographs should be very similar however the results show a significant difference in both the volume of the hydrograph as well as the timing.

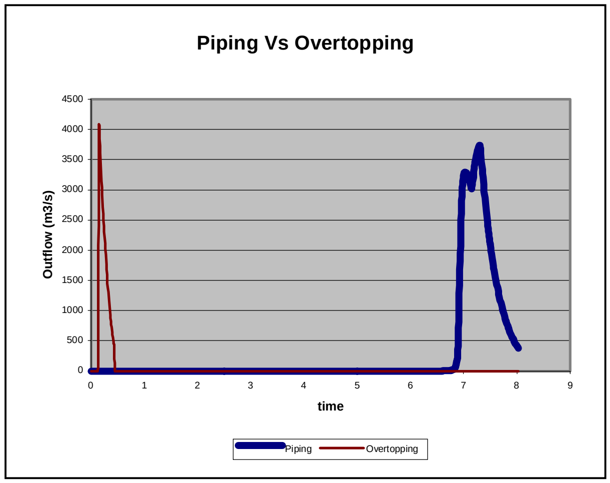


Fig. 7 Different outflow hydrographs produced by breach for an overtopping failure and a piping failure located 3cm below the crest.

CADAM CONCLUSIONS

The CADAM project has reviewed dambreak modelling codes and practice and identified a range of issues relating to model performance and accuracy. A number of these issues have been outlined above. When considering all aspects contributing to a dambreak study it was found that breach formation prediction, debris and sediment effects and modeller assumptions contribute greatly to potential prediction errors.

Full details of all findings and conclusions may be found in the project report which has been published by both the EC and the IAHR, and which may also be found on the project website at:

www.hrwallingford.co.uk/projects/CADAM

BEYOND CADAM

Following completion of the CADAM project, it was a logical extension of the work to review the recommendations and develop a programme of research work aimed at addressing the key issues. Working within the European Commission 5th Framework Research Programme, a major research proposal was developed by a new consortium of organisations, some of whom had worked on the CADAM project and others whom joined the team to provide additional and more varied expertise. This proposal was named SECURE (Safety Evaluation of Man Made Water Control Structures in Europe)

Funding of European research is undertaken on a competitive basis with a finite volume of money with which to fund projects. The original proposal initially failed to receive funding and required considerable reworking twice before funding was (informally) agreed. During this process the extent of the proposed research was significantly reduced. However, the final proposal, named IMPACT (Investigation of Extreme Flood Processes & Uncertainty), is now subject to contract negotiation with the European Commission and research work should commence towards the end of 2001.

The following sections are drawn from the European Commission discussion documents and provide an overview of the proposed work. Whilst the work programme is not yet final, it is unlikely to change significantly from the work described here.

THE IMPACT PROJECT - OVERALL AIM

The problem to be solved in the IMPACT project is to provide means of assessing and reducing the risks from the catastrophic failure of dam and flood defence structures.

THE EUROPEAN NEED FOR IMPACT

In the EU, the asset value of dam and flood defence structures amounts to many billions of EURO. These structures include, for example, dams, weirs, sluices, flood embankments, dikes, tailings dams etc. Several incidents and accidents have occurred which have caused loss of human life, environmental and economic damages. For example, in May 1999 a dam failed in Southern Germany causing four deaths and over 1 Billion EURO of damage. In Spain in 1982, Tous dam failed when still under construction with the result of 8 casualties, 100,000 evacuated people and economic losses worth 1500 MEuro. In 1997, also in Spain, a dam failed on the Guadiamar river, not far from Sevilla, causing immense ecological damage from polluted sediments released into the river valley during the failure. The dam failure at Malpasset (French riviera) in 1959 caused more than 400 casualties.

The risk posed by a structure in any area is a combination of the hazard created by the structure (e.g. flooding) and the vulnerability of the potential impact area to that hazard (e.g. loss of life, economic loss, environmental damage). To manage and minimise this risk effectively it is necessary to be able to identify the hazards and vulnerability in a consistent and reliable manner. Good knowledge of the potential behaviour of the structure is important for its proper operation and maintenance in emergency situations such as high floods. In addition, prior knowledge of the potential consequences of failure of a dam or flood defence structure is essential for effective contingency planning to ensure public safety in such an emergency.

In many areas related to structure failure our current understanding and ability to predict conditions is limited, so making the management of risk difficult. This project aims to advance the risk management process by improving knowledge of, and predictive tools for, the underlying processes that occur during and after failure. By both improving knowledge of the underlying processes and quantifying probability / uncertainty associated with these processes, the effect of these processes within the risk management system may be demonstrated and subsequently built into consideration the risk management process to improve reliability and safety. Many of the 'underlying processes' proposed for research were highlighted during the recent CADAM European Project as areas requiring further research.

A common problem integral to the failure process is that of sediment and debris movement. The sudden release of water from a control structure brings with it intensive scour in the flow downstream. Close to the structure, the flow is extremely destructive; it can scour aggressively material from the riverbed and floor of the valley, changing completely the shape of the valley, or even diverting the river from its natural course. The flow will uproot vegetation and trees, demolish buildings and bridges and wash away animals, cars, caravans etc. The floating debris can be transported for substantial distances whilst the heavier material is deposited or trapped once the flow velocity attenuates. At a different level, it is the erosion of material from an embankment or dam that occurs during breaching and hence dictates the rate at which flood water may be released, and the location at which this may occur.

The IMPACT project is of relevance to broad communities of user organisations, some of which are Partners in the IMPACT project team. The IMPACT project is organised in several complementary and interdependent themes to deliver the objectives of the research.

IMPACT: SCIENTIFIC AND TECHNICAL OBJECTIVES

The objectives of the IMPACT project are represented schematically in Figure 8 below. Specific objectives are therefore to:

1. Advance scientific knowledge and understanding, and develop predictive modelling tools in three key areas associated with the assessment of the risks posed by dam and flood defence structures:
    a. the movement of sediment (and hence potential pollutants) generated by a failure

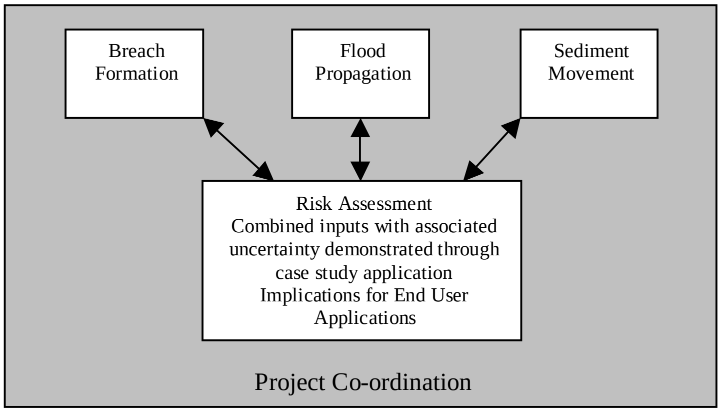


Fig. 8 Structure of the IMPACT work programme

b. the mechanisms for the breaching of embankments (dams or flood control dykes) and factors determining breach location
c. the simulation of catastrophic inundation of valleys and urban areas following the failure of a structure

2. Advance the understanding of risk and uncertainty associated with the above factors and combine these factors through a single system to demonstrate the risk / uncertainty associated with application of the end data (i.e. asset management, emergency planning etc.)

These objectives will be undertaken with careful reference to past and ongoing research projects related to these topics, including the CADAM and RESCDAM projects.

An important subsidiary objective of the IMPACT Partners is to ensure end-user relevance, acceptance and implementation of the outputs of the research. To this end, the IMPACT project Partners will develop the methodologies using demonstration sites and applications wherever appropriate. The project will include:

- breaching of large scale test embankments (6m high embankments) to investigate breach formation mechanisms and the relationship between prototype and laboratory simulation
- field assessment of sediment movement following large scale embankment failure
- simulation of catastrophic flooding through the streets of a European city
- a combined assessment of extreme flood conditions and prediction uncertainty for a real or virtual site comprising dam / flood defence and urban area.

IMPACT: BENEFITS OF THE RESEARCH PROJECT
The successful completion of the IMPACT project is expected to lead to:

- improved scientific knowledge and understanding of extreme and aggressive flood flows following the catastrophic failure of a water control structure
- Specific scientific knowledge and understanding relating to breach formation through dams and flood defence structures, movement of sediment under extreme flood conditions and the simulation of flooding in urban areas
- improved understanding of the risk associated with the potential failure of dams and flood defence structures ultimately leading to reduced risks of failure and hence a reduction in long-term costs
- improved understanding of the uncertainty associated with the prediction of extreme flood conditions and processes
- improved public safety through emergency planning and community disaster preparedness in the event of a failure
- enhanced prospects for EU-based consultancies in the International Water and Hydropower markets

IMPACT: OVERVIEW OF WORK PROGRAMME
As shown in Figure 8, the IMPACT project has been structured according to 5 *Theme Areas*. These *Theme Areas* are:

Theme 1	Project Integration, Co-ordination and Delivery
Theme 2	Breach Formation
Theme 3	Flood Propagation
Theme 4	Sediment Movement
Theme 5	Combined Risk Assessment & Uncertainty

The objectives and proposed work for each of these Theme Areas is presented in more detail below:

<u>Theme 1</u> <u>Project Integration, Co-ordination and Delivery</u>
The IMPACT project involves 9 organisations drawn from 8 European countries and thus will require careful attention to the management of the research to ensure that it delivers it outputs. The project integration, co-ordination and delivery is a core management function of the project Co-ordinator. The project integration will be achieved through facilitation of communication between each of the project themes and the researchers engaged in the work packages. There will be a regular meeting of the Theme leaders approximately every four to six months. Where possible, these meetings will be scheduled with other project meetings to minimise the travel costs. Full team meetings will be held at project workshops, of which four are scheduled during the 36-month period. These workshops will provide opportunities for representatives of all the research teams to discuss

their findings and future approaches. The Co-ordinator (M Morris: HR Wallingford) will use the project workshops to review progress and define the detailed work programme for the coming months.

The Co-ordinator will establish a project Internet site with public and private areas. The public area will give information on the definitive project outputs, whilst the private site with an FTP area will be the main vehicle for electronic communication of data, software and results between the IMPACT project team members. An Internet based email database will be established to allow any interested parties to register their email address for receipt of project newsletters, meeting details etc. The Co-ordinator will take final responsibility for the documentation and reporting of the project. Project team members will be encouraged to publish the results of the research in refereed scientific journals and conferences as appropriate. Public outputs from the project will be recorded and made available through the public area of the project Internet site.

<u>Theme 2</u>   <u>Breach Formation</u>

The problem to be solved in this theme is the lack of quantitative understanding of the modes and mechanisms involved in the failure of dams and flood defence structures. Without such understanding, the rate of outflow from a failed structure cannot be assessed and hence the risks posed by the structure cannot be assessed with confidence. The approach to the research proposed in the IMPACT project is a combination of experimental and theoretical investigations, leading to a new modelling procedure for the failure of embankments. Three components of failure modes will be investigated, internal erosion, overtopping and slope stability during the breach enlargement. The methods will be validated as far as possible against data from the physical experiments as well as actual failures. The large-scale experimental facilities available to the IMPACT partners will allow the factors that govern the initiation and growth of breaches to be studied under controlled conditions. However, the issue of scaling from the laboratory to the prototype scale must be addressed. A novel part of the experimental programme is the rare opportunity to include field tests at large scale. A test site has been identified in Norway located between existing dams where a 6m high embankment may be constructed and then tested to failure using controlled flow released from the upper dam. Five failure tests are planned. The location and test programme means that no damage to infrastructure will occur, also with minimal environmental effect. Individual work packages within this *Theme Area* include:

- Breach formation processes – controlled failure of 6m high embankments to identify key processes
- Breach formation processes – laboratory physical modelling of embankment failure to identify key processes

- Model development and comparison - development and comparison of breach model performance through use of a common modelling framework
- Breach location – development of a methodology / prototype tool for a risk based approach to identifying breach location

Theme 3 Flood Propagation

The problem to be solved in this theme is to produce reliable modelling methods for the propagation of catastrophic floods generated by the failure of a water-control structure (often called the *dam-break* problem). The intensity of the flood will depend upon the initial difference in depth between the impounded level behind the control structure and the land level on the other side. Hence the research in this theme will concentrate upon the dam-break flood problem but the techniques will also be applicable to the failure of flood embankments. The overall objectives for this theme are:

- To identify dam-break flow behaviour in complex valleys, around infrastructure and in urban areas, and the destructive potential of these catastrophic flood waves.
- To compare different modelling techniques & identify best approach, including assessment of accuracy (in relation to practical use of software).
- To adapt existing and develop new modelling techniques for the specific features of floods induced by failure of man made structures.
- To develop guidelines for an appropriate strategy as regards modelling techniques, for a reliable and accurate prediction of flooded areas.

The approach to be adopted is to:

- compare different mathematical modelling techniques
- identify the best approaches, including assessment of implementation of the methods in industrial software packages
- check the accuracy and appropriateness of the recommended methods by validation of the models against the results from physical experimentation.
- validate the different modelling techniques adopted, both existing and newly developed against field data obtained from actual catastrophic flood events.

The research has been organised into two work-packages each subdivided into several distinct tasks; for each task there is a Technical Co-ordinator and a team of Partners involved in the activities. The two work-packages are:

- Urban flood propagation
- Flood propagation in natural topographies

Theme 4 Sediment Movement

The problem to be solved in this theme is to improve the predictions of the motion of sediments in association with catastrophic floods. The nature of the problem is different form that in normal flood flows in that the quantity and size of sediment will be much greater in the catastrophic flood flow. This is an important issue for an accurate prediction of the downstream consequences:

- the river bottom elevation can vary by tens of meters
- or the river can be diverted from its natural bed (as for the Saguenay river tributary – the Lake Ha!Ha! damn failure, Canada, in 1997),

with the associated impact on the flooded areas. The approach adopted in the IMPACT project is to combine physical experiments designed to improve our physical understanding of these cases with the development and testing of mathematical modelling methods for simulation of these flows. The IMPACT Partners will use the extensive experimental facilities available to them in undertaking the experimental programme. An output of the research will be a set of well-documented experimental investigations of flows with transported sediment, which could serve the international research community as benchmarks for future theoretical developments outside the scope of the current IMPACT project.

The research is divided into two work-packages that address:

- near field sediment flow in dam-break conditions
- geomorphological changes in a valley induced by dam-break flows (far field)

Theme 5 Combined Risk Assessment and Uncertainty

Themes 2-4 outline proposed research into processes that are currently poorly understood or poorly simulated by predictive models. An important aspect of any process that contributes towards an overall risk assessment (i.e. prediction of flood risk) is an understanding of any uncertainty that may be associated with prediction of that particular process. For example, a flood level may be predicted to reach 20m and an emergency plan developed to cope with this. However, if the uncertainty associated with this prediction is ±2m, then different measures may be taken to manage this event. The problem to be solved in this theme, is to quantify the uncertainty associated with each process contributing to the risk assessment and to demonstrate the significance of this for the end application. This may be in the form of uncertainty associated with flood level prediction, flood location, flood timing or flood volume – depending upon the particular application of data.

Uncertainty will be quantified by working closely with the fundamental research being undertaken in Themes 2-4 and demonstrated through a number of case study applications. The procedure for combining the uncertainty associated with different data will depend upon the process itself. This may require multiple model simulations or combination through spreadsheet and / or GIS systems as appropriate.

Having identified process uncertainty and the effect that particular processes may have on the end application, it will also be possible to identify the importance (with respect to the accuracy of a risk assessment) that each process has and hence the effort that should be applied within the risk management process to achieve best value for money.

IMPACT CONCLUSIONS

Subject to final contract negotiations with the European Commission, the IMPACT project should commence towards the end of 2001 and run for a period of 3 years. The research findings from this project should enhance understanding of extreme flood processes and simulation including breach formation, flood routing and sediment movement. It is intended that research undertaken for the IMPACT project shall remain focussed upon the needs of end users, and active participation by representatives from industry worldwide shall be sought. Technical knowledge relating to the specific extreme event processes will be presented but also analysed in the context of end user applications with the aim of demonstrating not only what is known, but also the uncertainty related to that knowledge and how this might influence direct applications.

For more information on this project, and to sign up to the project email list, visit the project website (from November 2001) at:

www.hrwallingford.co.uk/projects/IMPACT

or contact the project Co-ordinator directly on:

m.morris@hrwallingford.co.uk

ACKNOWLEDGEMENTS

The members of CADAM are grateful for the financial support offered by the European Community to structure this concerted action programme. Without funding for research time or facilities, however, the project relies on contributions made by the group members, which is greatly appreciated. The teams who undertook modelling comprise:

Université Catholique de Louvain (Belgium), CEMAGREF (France), EDF/LNH (France), Université de Bordeaux (France), INSA Rouen (France), Federal Armed Forces University Munich (Germany), ENEL

(Italy), Politechnika Gdanska (Poland), Universidade Tecnica de Lisboa (Portugal), Universidad de Zaragoza (Spain), Universidad Santiago de Compostela (Spain), Vattenfall Utveckling AB (Sweden), University of Leeds (UK), HR Wallingford (UK).

Special thanks should also go to the following organisations that have undertaken, or made available data from, physical modelling work:

Université Catholique de Louvain (Belgium), Université Libre de Bruxelles (Belgium), LNEC (Portugal), ENEL (Italy), Federal Armed Forces University Munich (Germany), The Finnish Environment Institute.

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Issues, Resolutions, And Research Needs Related To Dam Failure Analysis
USDA/FEMA Workshop
Oklahoma City June 26 - 28, 2001

Embankment Breach Research in Norway

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1. INTRODUCTION

The modern dam building in Norway started around the turn of the century, when we started to exploit our hydropower resources. Hydropower is today one of Norway's major natural resources. The development of the resource has resulted in construction of many reservoirs. 2500 dams are controlled by the Norwegian Water Resources and Energy Directorate (NVE). NVE is the dam-safety authority in Norway. The dams controlled by NVE are higher than 4 m or have a reservoir capacity exceed 0,5 Mill.(m^3).

In the beginning most of the dams was masonry or concrete dams. After 1950, large embankment dams began to dominate the scene.

A water reservoir behind a dam represents an enormous energy potential, which might cause catastrophic damage in case of a dam failure. The dams therefore pose a risk to the downstream area. To manage and minimize this risk effectively it is necessary to be able to identify the hazards and vulnerability in a consistent and reliable manner. Good knowledge of the behavior of the structure is important for the maintenance and proper operation. In addition, prior knowledge of the potential consequences of failure of a dam or flood defense structure is essential for effective contingency planning to ensure public safety.

The issue of dam safety has become more and more important in Norway during the last years and much money has been spent to increase the safety level. The dam owner is responsible for the safety of his dams. He has to follow the requirement and guidelines from NVE:

- [1] Statkraft Grøner is one of the major consulting firms in Norway with 300 employee in 1998 and an annual turnover 42 mill. US dollar. The company has a high expertise in the field of research and development, working closely with academic research groups and the hydropower industry. The company is fully owned by Statkraft SF, the largest Hydropower Company in Norway. Statkraft SF Operates 55 power plants and has ownership in 36 more. The average annual production for Statkraft in Norway is 36 TWh (30% of Norway's total). Statkraft SF owns 113 water reservoirs with a capacity of 33,7 billion m3 (40% of Norway's total storage capacity)

1. Contingency planning for abnormal situations
2. Safety revisions
3. Load recording
4. Damage and accident reporting
5. Risk analysis
6. Discussions of the failure probability and studies on impact of failure

To make sure that the dam-safety work is done in a proper way, NVE has made several guidelines. These include "*Guidelines for simulation of dam-break*"(Backe et al 1999). All the dam-break simulations in Norway are made according to these guidelines.

The system for revisions of dams, developed by NVE, has been operation for several years. The experience so far is good. From time to time the result from a safety revision implies that the dam-owners have to put in a lot of money to fulfill the requirements. In most cases this is done without any discussions. Sometimes, however, there is a discussion between the owner of a dam and NVE based on different understanding of the guidelines and a lack of common understanding of the basic mechanism in how the strength and stability of the dam can be improved. The focus for discussion is now the new guidelines for dam-safety, not yet put into operation.

The most frequent theme for discussion is whether or not a dam satisfies the requirement to:

- Stability when exposed to normal loads.
- Stability with extreme loads e.g. major leakage and resistance against erosion in case of overtopping.

Stability of rock filled dams is determined mostly as a function of the shear strength of the rock filling. Stability in case of extreme loads is also dependent of the shear strength, but in these cases there is a big uncertainty in the loads.

The regulations require that the dams can resist a certain leakage through the core. An ongoing project (Cost efficient rehabilitation of dams) also put the focus on the breaching mechanism in the case of a major leakage.

Dam break analysis is performed to assess the consequences of dambreak and is a motivating factor for the dam safety work. The routines used today to in Norway give a too simplified description of the development in the breach in our rock-filled dams. The materials and the way the dam is constructed are not taken into consideration. Due to this most of the work done on the dam to improve the security will have no visible influence on the development of the dambreak and on the downstream consequences of the break, when performing the dambreak simulation. This is not logical and gives not incentives to the dam safety work. The result of this can also be a wrong classification of a dam.

Based on these experiences Norway has started a new project with the main objective of improving the knowledge in this field.

Parallel to the planning of a Norwegian project there, European research institutions have been working for establishing a common within the filed of dam-safety. The project is called IMPACT and is presented in detail by Mark Morris. *The problem to be solved in the IMPACT project is to provide means of assessing and reducing the risks from the catastrophic failure of dam and flood defense structures* (quote from the application to EU for funding of IMPACT).

2. MAJOR OBJECTIVES

The scope of the project is to improve the knowledge of, and to develop predictive tools for the underlying processes that occur during and failure. By doing so the proper decisions can be taken for improving the dam safety taking into account the technology and economy.

The objectives of the project are:

- To improve the knowledge on the behavior of rock filled dams exposed to leakage.
- To get knowledge on the developent of a breach.

This knowledge will be used to:

- Develop simulation tools that will be used in the planning of dam safety work.
- Develop new criteria for design of dams
- Develop criteria for stability and failure mechanics of dams.

3. PROJECT PLAN.

The Norwegian project consists of 4 sub-projects:

1. Shear strengths and permeability of rock-fillings
2. Stability of the supporting fill and dam-toe in rockfill dams exposed to heavy leakage
3. Breach formation in embankment-dams (rock-filled dams)
4. Breach formation in concrete dams

A rockfill dam is defined as an embankment dam comprising more than 50% by volume of fill obtained from rock quarry or rock excavation. (Konov, 2001)

The IMPACT-project project consist of 4 themes:

1 Breach Formation
2 Flood Propagation
3 Sediment Movement
4 Combined Risk Assessment and Uncertainty

Sub-project 3 in the Norwegian project and theme 2 in IMPACT has the same objectives. Some of the problems that will be solved in sub-project 1 and 2, will give information that is relevant for IMPACT. Through coordination of these to major projects we hope to improve the knowledge about embankment dams

Mark Morris has presented the details of IMPACT. In the following chapters a short description of the Norwegian project will be given.

3.1 Shear strengths and permeability of rock-fillings

Through the process of reevaluation of rock-filled dams the question of the shear-strength has been asked. Very few dams have a documentation of the shear-strength of the rock-fillings based on test of the rock materials. In most cases the planning is based on experience from similar dam-constructions and geological conditions.

The main question to answer in the project will be what the correct or best parameters to describe the materials are. Physical test and experience will be used. Some tests have been done at some of the largest rockfill-dams in Norway. This knowledge will be used to correlate the shear-strength from single tests of the rock material to roughness, shape of the grains, pressure strength.

Permeability or hydraulic conductivity is important for the leakage through the supporting fill and for the erosion during a dam failure. Existing knowledge and data on this topic will be collected through a literature review. Test of the permeability in the large-scale test dam will be performed.

3.2 Stability of the supporting fill and dam-toe in rock-filled dams exposed to heavy leakage

Sub-project no 2 will focus on developing of tools or routines for assesment of the stability of a dam exposed to leakage through or over the core.

The objectives of the tests are:

1. To find the connection between the drainage capacity through a rock-filled dam, the size of the stones, layout of the filling/dam-toe etc. This information will be used setup of new guidelines for assessment of old dams and for layout and dimensioning of new dams in general and specifically the dam-toe.
2. To increase the knowledge of the permeability of rock-fillings in general.
3. To find and verify the connection between different scaling. (1:10, 1:5 and prototype)

A computer simulation program will be developed to analyze the flow through a rock filling. Criteria that tells when a rock-filled dam will collapse either due to erosion of the individual stones or a major break along shear flater through the supporting fill, will be developed.

The simulating program will be tested on physical models in large scale

3.3 Breach formation in rock-filled dams

The objective of this sub-project is to improve the understanding of the breach formation process that occurs in and through embankments, with a special focus on rockfill dams.

Breach formation covers factors that will lead to an uncontrolled release of water from the structure. The most common modes of failure for an embankment are from water overtopping the crest or internal erosion also called piping. The ability to predict the location and rate of development of a breach through a flood embankment or dam is limited.

The most commonly applied approach is the deterministic BREACH model developed in the late 1970's at the US National Weather Service (USNWS). Several parametric relationships based upon analysis of actual failures of dams are also in use. In Norway the relationship developed by Froelich (Backe D et al.1999) is used.

Most of the tests and analysis have been of homogeneous structures of non-cohesive material. The failure of multi-element structures incorporating an impervious core remains poorly understood.

Experimental tests will be undertaken to support the theoretical development of models of the failure modes and rates of failure. These tests will be made as part of the IMPACT project. Investigation will be made of the factors contributing to breach location and analysis of the likely probability of failure resulting from these factors.

The tests will be completed through "large scale" tests. Based on the results from these tests a simulation program will be developed for simulation of the breach formation in these kinds of dams.

3.4 Breach formation in concrete dams

This subproject will focus on the breach formation in concrete dams. Finite element methods will be used. The project will take advantage of the experience of concrete technology for simulation of failure of concrete dams.

There exist several very advanced simulation programs that can be used e.g. ABAQUS. This program was developed to help engineers to design the off-shore platforms that are used for oil-drilling in the North See.

4. PHYSICAL/SCALE MODELING

There will be several test of physical modes in the in the laboratory and in the field (large-scale test). This test are necessary in order to find the answers in sub-project 1, 2 and 3 and also for the questions asked in theme 2 of IMPACT.

4.1 Tests in the laboratory.

There will be laboratory test both in Norway and in UK (Wallingford). The tests in UK will be undertaken to examine the different aspects of breach formation. This part of the laboratory modeling will use embankments approximately 0.75m in height and the experiments will investigate:

- overtopping failures for water flowing over the crest of an embankment
- piping failures where fine material is progressively eroded through the body of the embankment

Initial tests will cover homogeneous non-cohesive material – an idealized embankment. Tests will then progressively tend towards real embankment designs through the analysis of cohesive material and composite structures. Tests will measure flow-rate, the hydraulic heads, and evolution of the crest erosion and piping to establish the erosion rates of the material.

The main issues of the Norwegian laboratory tests are to find out any possible problem with the field tests. Several tests of the dam-toe will be made. Focus will be on the following:

1. Scaling effects
2. Size of the materials (stones)
3. Grain size distribution of material
4. The shape (layout) of the dam-toe
5. The importance of the downstream-water level.

In the laboratory we will use to test-flumes: one in the scale 1:5, the second one in scale 1:10. According to the plans there will be 13 different tests of the dam-toe. These are shown in table 1.

Table 1. Tests in the laboratory

Test no		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**
Scale	1:10	*		*	*	*	*						*	*
	1:5		*					*	*	*?	*?	*?		
Size of the stones	D_{50} [mm]	500	500	100	300	100 0	200 0	100	100 0	500	500	500	500	500
	Sizing	E	E	E	E	E	E	E	E	V	E	V	E	E
Shape	Slope	1,5	1,5	1,5	1,5	1,5	1,5	1,5	1,5	1,5	3,0	3,0	1,5	3,0
	Water level	L	L	L	L	L	L	L	L	L	L	L	H	H
Reproducibilityt														
Scale		*1	*1	*2		*3		*2	*3					
Size of the stones		*1	*2	*1	*1	*1	*1	*2	*2					
Sizing			*1							*1	*2	*2		
Slope			*1							*2	*1	*2		
H/L waterlevel			*1								*2		*1	*2

E: Uniform, V: well graded, L: low water level downstream, H: high water level downstream

*: referring to which tests can be compared. E.g.: If we want to study the scaling effects, results from test 1 and 2, 2 and 7, and 5 and 8 should be compared.

Key figures for the flumes are shown in table 2.

	Scale 1:10	**Scale 1:5**
Width (meter)	2,20	4,0
Length (meter)	10,0	10,0
Depth (meter)	0,75	1,43
Maximum discharge (l/s)	320	600

We will also make some test with the focus of planning of the field test.

All of the tests in table 1 are used to evaluate the stability of the stones in the outflow area.

Test no 1,2,3,5,7, and 8 will be used to assess if there are some scaling effects. Tests #1 to #8 are designed to help in evaluating the effect of the different sizes of the stones in the outflow area.

The results here will also be used to compare with data from earlier tests and projects:

- "Safety analysis of rock-filed dams", Dam safety project 1992.
- " Safe remedies for leaking embankment dams", ICOLD, Rio de Janeiro, 1982.
- "Flow through and stability problems in rockfill dams exposed to exceptional loads", Vienna 1991.
- " The risk for internal erosion in rock-filled dams and the calculation of turbulent coefficient of permeability", Norwegian research Council, 1991.
- "Extreme situations", Short course in dam safety, 1992.

The test #2, 9, 10 and #11 are identical except from the differences in the grain size distribution of the building material. Two of them are well graded. The other two have a uniform grain size distribution. These tests will be used to assess the difference in stability with the same characteristic size of the stones, but different grain size distribution.

Table 3 mixing of material

Table 3.Size of material in prototype

Test	**Size of stones (mm)**				**mixing**	
no.	**d_{max}**	**d_{60}**	**d_{50}**	**d_{10}**	**E/V**	**C_u**
1	750	600	500	240	E	2,5
2	750	600	500	240	E	2,5
3	150	120	100	48	E	2,5
4	400	360	300	144	E	2,5
5	1500	1200	1000	480	E	2,5
6	***1800***	***1600***	***1500***	***640***	E	2,5
7	150	120	100	48	E	2,5
8	1500	1200	1000	480	E	2,5
9	750	600	500	120	V	5
10	750	600	500	240	E	2,5
11	750	600	500	120	V	5
12	750	600	500	240	E	2,5
13	750	600	500	240	E	2,5

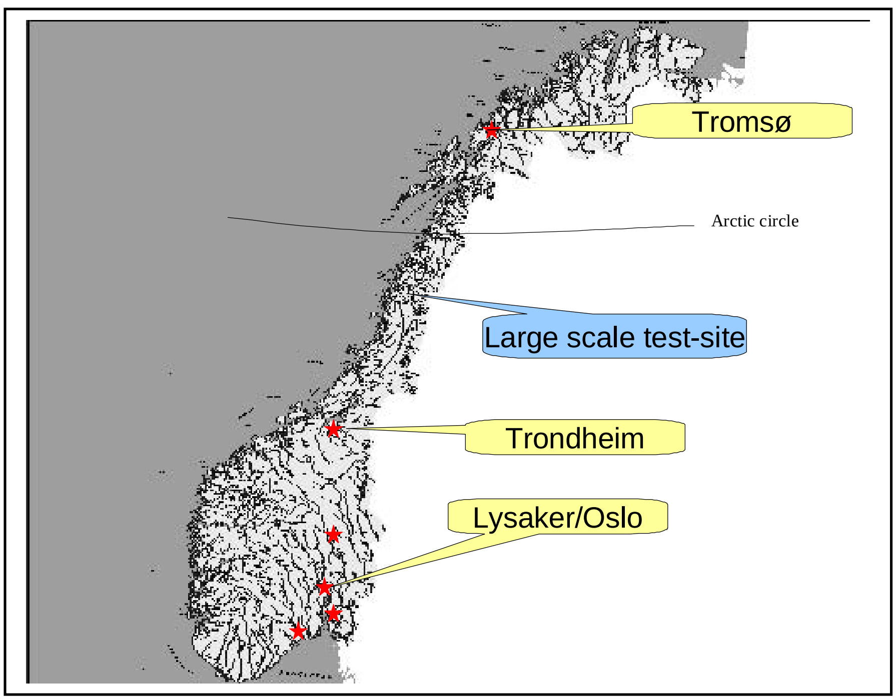
Tests #2 , #9, # 10 and #11is identical except for the slope of the downstream side: two of them have the slope of 1:3, the two other have a slope equal to 1:1,5. Results from these tests will be used to assess the stability of the toe, due to different slopes.

Test #2 and #12 are identical, the same is #10 and #13 except for difference in the water-level downstream of the dam. These tests will be used to compare the effect on the stability due to different level down stream of the dam.

The following data will be recorded in the tests:
- The pressure line in the filling
- Water-level upstream and downstream of the dam
- The discharge through the dam (measurd downstream of the dam)
- The water-level at the downstream edge of the dam_toe
- The grain size distribution curve.
- The porosity in a test volume
- Picture from the test
- Video recording of the test.

4.2 The field tests

The field test will be made downstream of one of the largest reservoir in Norway (The lake Røssvatnet). Statkraft SF is the owner of this reservoir and is an active partner in the project. The test site is in northern Norway, close to the Arctic Circle. Figure 1



The dam on this reservoir has just been revised. As a result of the safety revision Statkraft SF has made safety improvements on the dam. An overview of the area downstream of the dam is shown in figure 2. Figure 3 shows cross-section of the test-site and also a longitudinal section along the river. So far the focus in the project has been on the laboratory tests. The results from these test will give information and knowledge that will be used for the detailed planning

We are going to run two different kind of tests:
- tests of the stability of the dam-toe

- breach tests

A local contracting firm will be responsible for the building of the dam according to our specifications.

The release of water from the upstream-reservoir has to be done in close cooperation with the dam-owner, Statkraft SF.

The gates at Røssvassdammen have a total capacity up to 500m^3/s. The gates are new and the operation of them is easy and flexible. The high capacity through the gates gives us the opportunity to simulate breaching in a large reservoir (slow reduction in the water level in the reservoir as a function of time) and a small reservoir.

Prior to the tests we will establish a measurement station for discharge. The capacity of the gate as a function of the opening is known. By releasing a known discharge through the gates and record the corresponding water level a stage-discharge relationship will be established.

Exact measurement of the discharge through or over the dam is important.

There might be a minor price to pay for the release of water, because it normally would have been use for hydropower production. Negotiation with Statkraft SF is going on now.

The following data will be recorded in the tests:

- The pressure line in the filling
- Water-level upstream and downstream of the dam
- The discharge through the dam (measurd downstream of the dam)
- The water-level at the downstream edge of the dam_toe
- Picture from the test
- Video recording of the test.
- The development of the breach

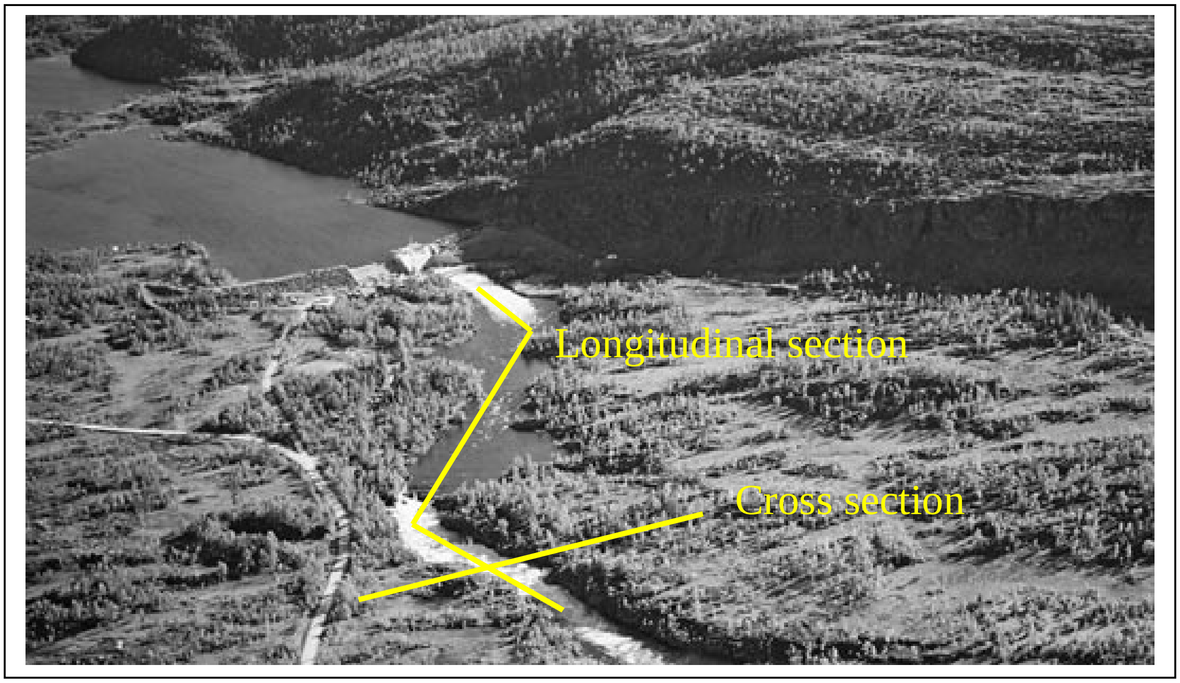


Figure 2 Overview of the test-area.

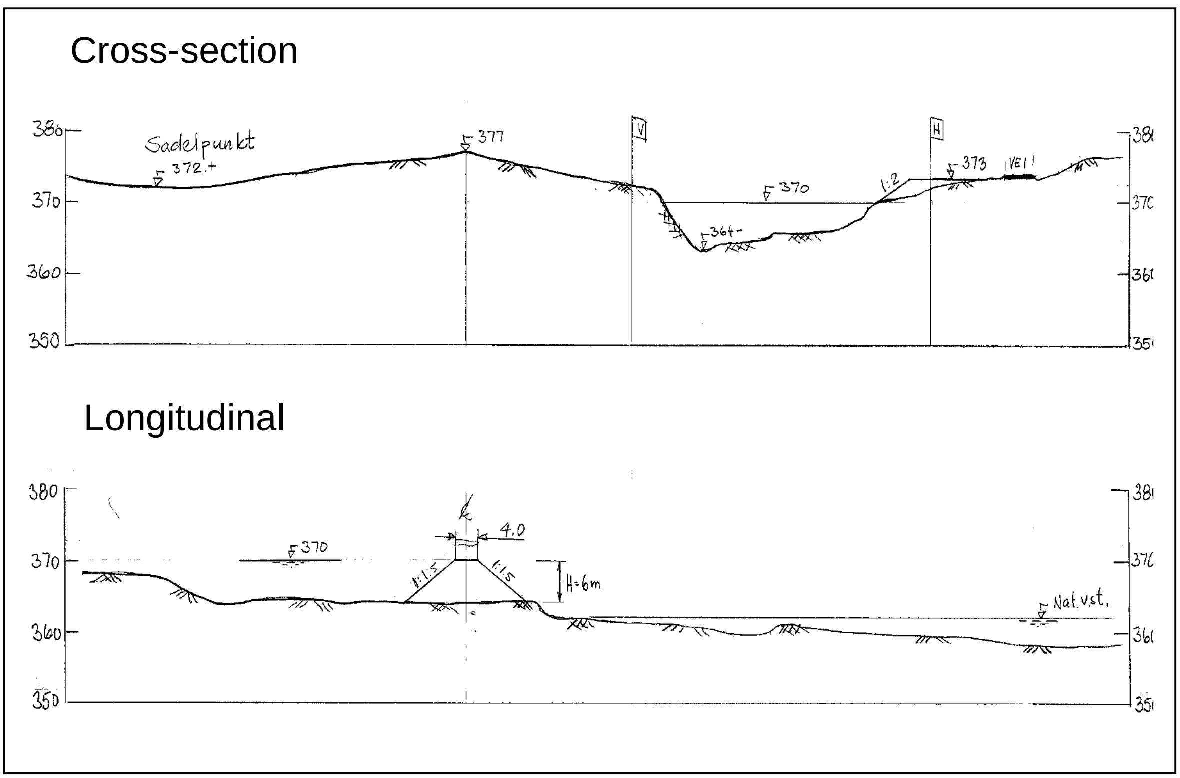


Figure 3. Cross-section and longitudinal section

5. BUDGET AND TIME SCHEDULE

The whole Norwegian project will run for 3 years with the startup the spring 2001 and with a total budget of 7 mill Norwegian Kroner. This close to 900 000 Euros.

6. PARTNERS

The Statkraft Grøner AS is the leader of the Norwegian project and also partner in IMPACT. The other Norwegian companies involved are:

- Norconsult AS
- NGI (Norwegian Geotechnical Institute)
- SINTEF Energy Research
- NTNU (Norwegian University for Science and Technology)

There is established advisory group or steering committee for the project in Norway. This group is made up of the major dam owners in Norway, NVE and the Norwegian Electricity Association EBL.

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