



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 performed 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, Statkraft 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

¹A 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.

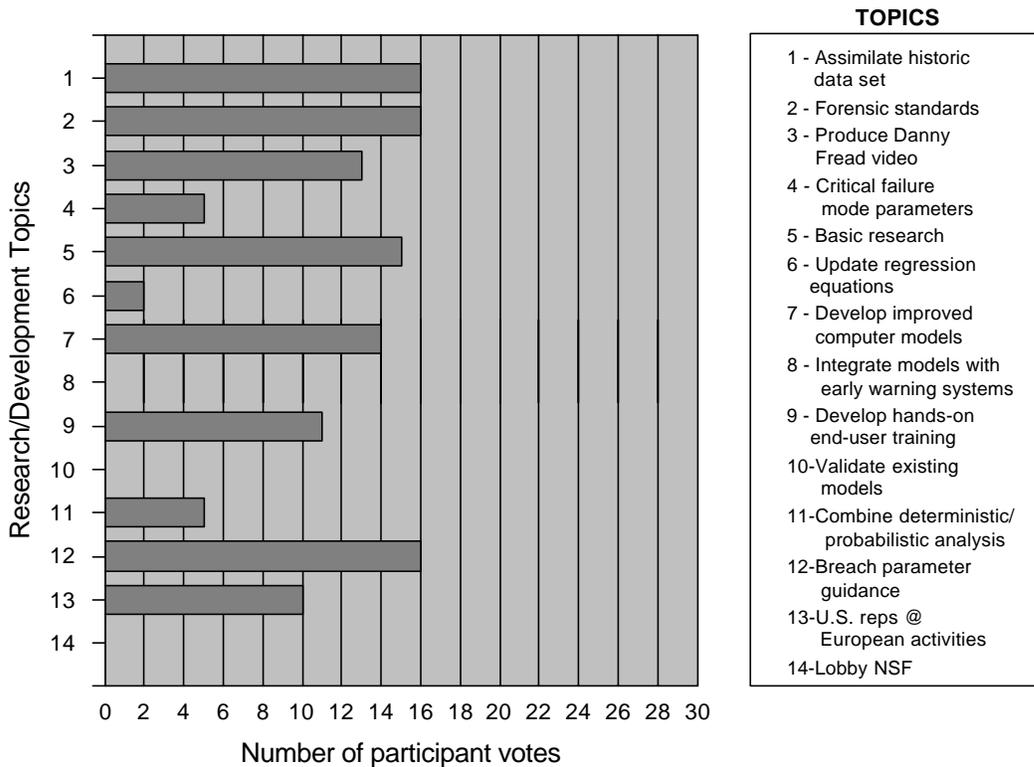


Figure 1. Probability of success

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.

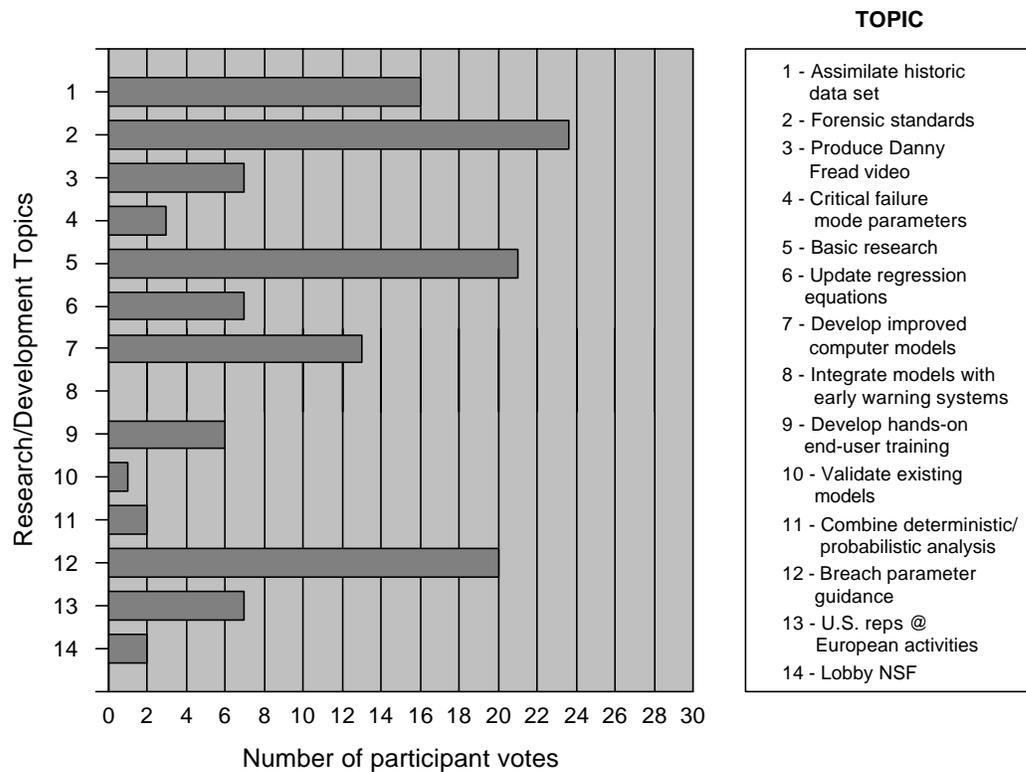


Figure 2. Value of research topic

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

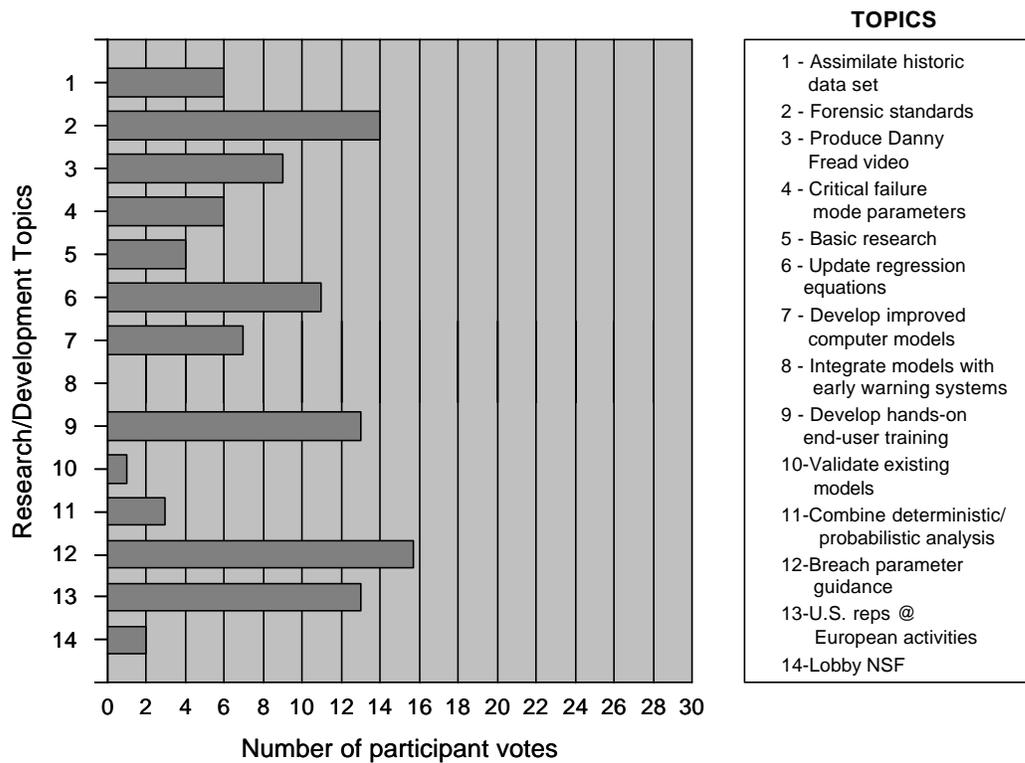


Figure 3. Cost of research topic (the more votes the lower the cost).

TABLE 4 – RESEARCH TOPICS RANKED BY COST

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

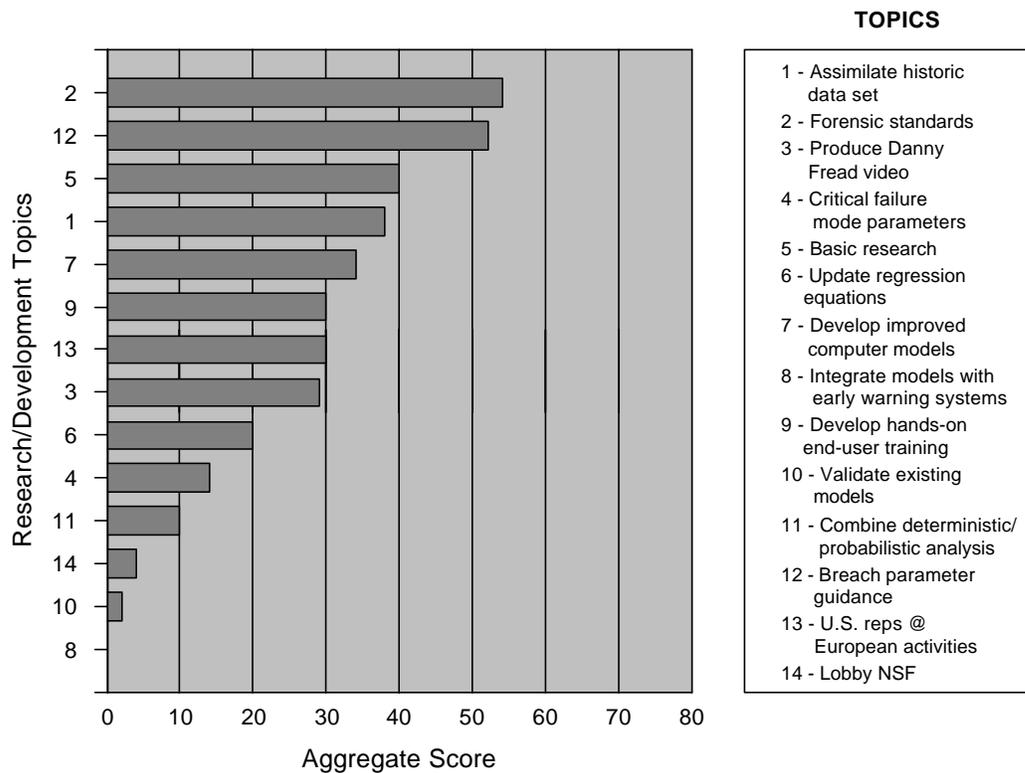


Figure 4. Research topic ranked by aggregate score of probability of success, value, and cost.

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

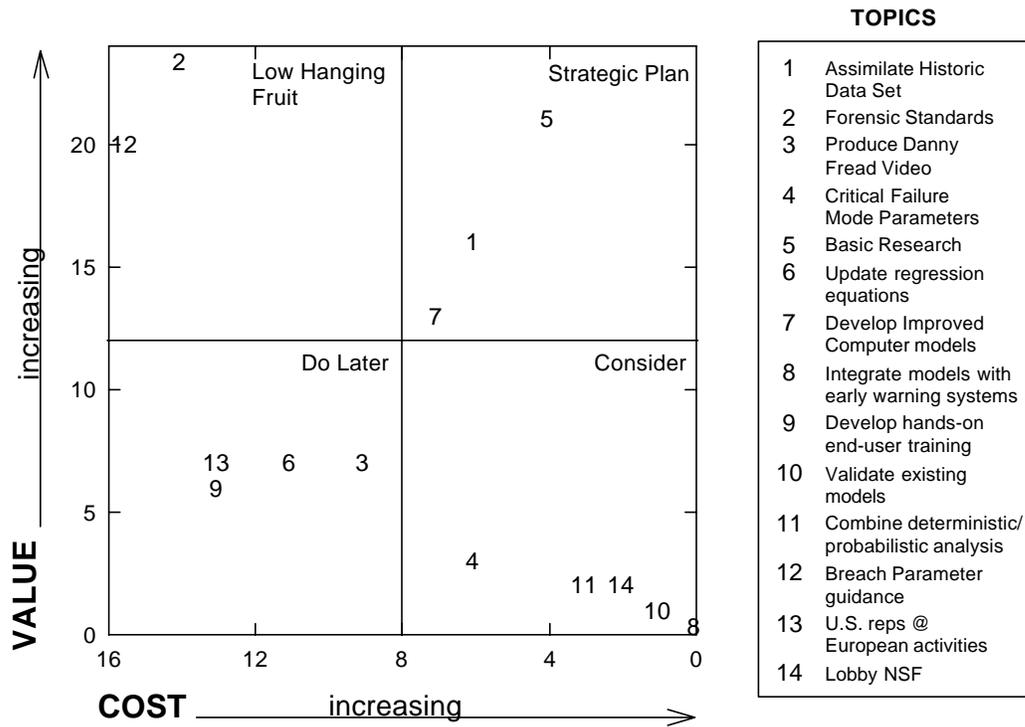


Figure 5. Decision quadrant.

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
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Break

ii. Some Existing Capabilities and Future Directions for Dam-Breach Modeling/ Flood Routing	(Danny Fread)	1015
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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 Fiegler)	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

B-1

Human and Economic Consequences of Dam Failure

by
Wayne Graham, June 26, 2001

<u>Dam</u>	<u>Date and Time of Failure</u>	<u>Dam Height (ft)</u>	<u>Volume Released (ac-ft)</u>	<u>Deaths</u>	<u>Economic Damage</u>
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 at 2 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		1hr 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: earthfill
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

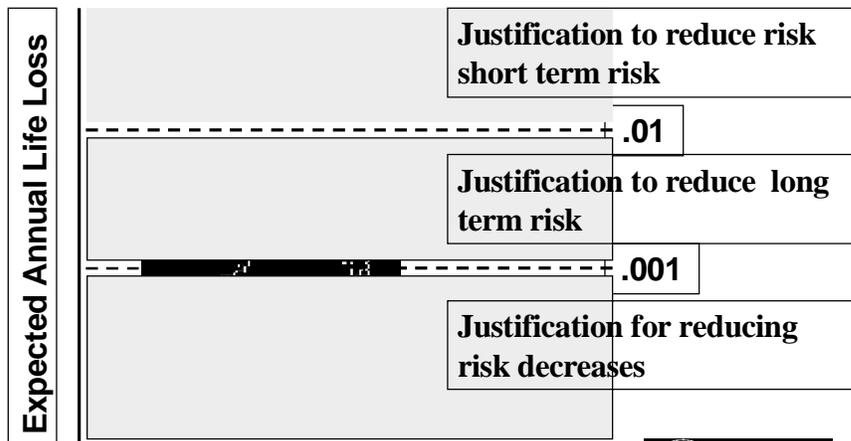


Risk Management Tools

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



Public Protection Guidelines



Life Loss



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

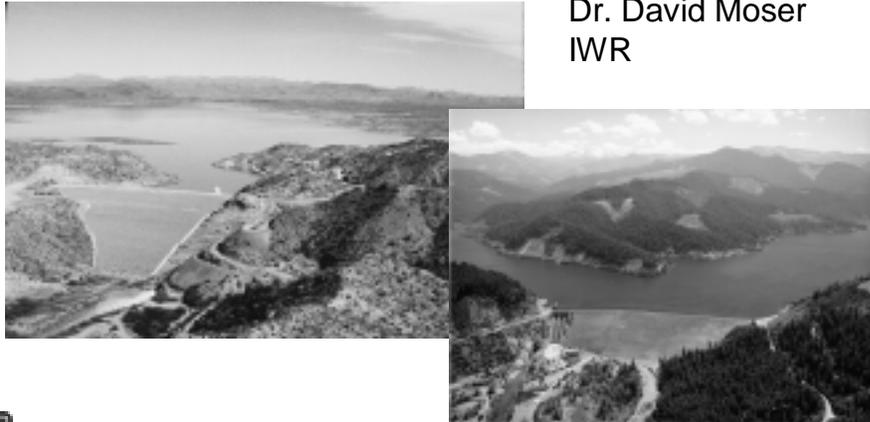
- Focus research needs on areas that help decision makers reach decisions
- Strive for balance in development of tools (breach formation, routing, consequence assessment)
- Ensure that tools can be cost effectively applied to a wide variety of structures



B-3

Dam Safety Decisions: Current USACE Practice

Dr. David Moser
IWR



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



Background

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



Standards

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



Design Events

Flood

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

Seismic

- ◆ Maximum Credible Earthquake (MOE)
- ◆ Operating Basis Earthquake (OBE)

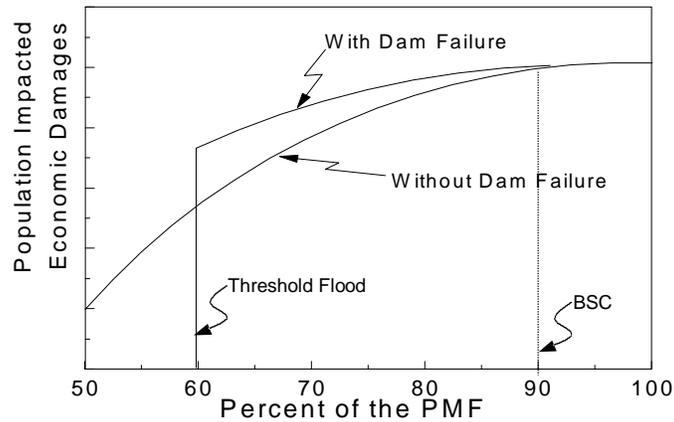


Base Safety Condition

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



Establishing the Base Safety Condition



Why Change from PMF?

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



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



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



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



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



Risk Analysis for Dam Safety

- ◆ R&D Program Initiated in 1999

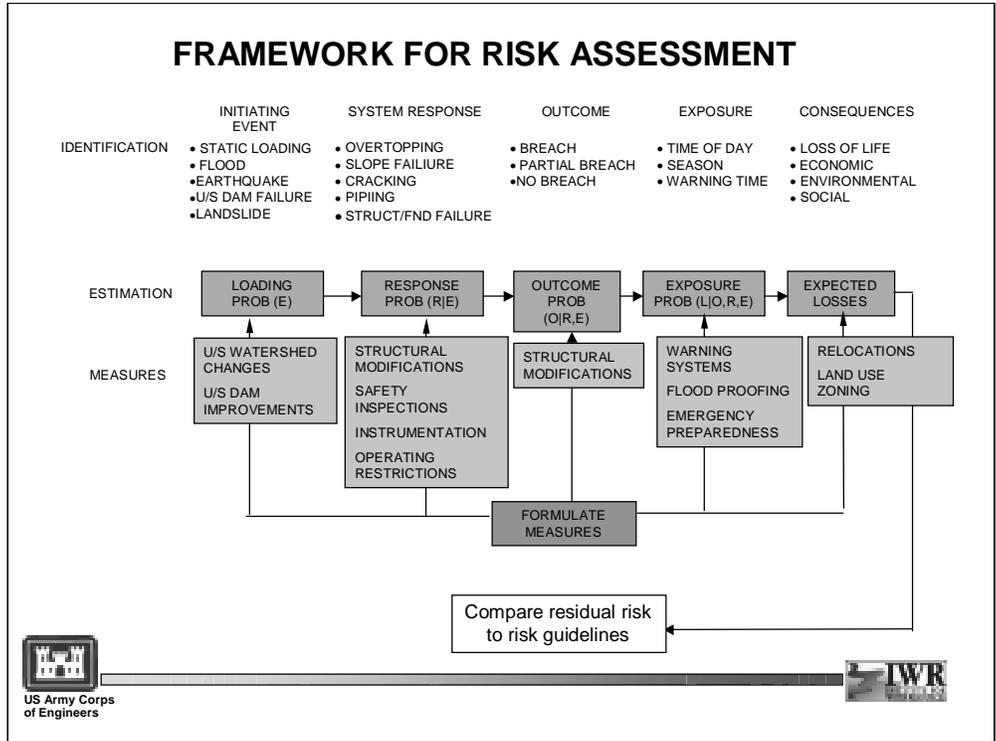
- ◆ Objective

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



Risk Analysis





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

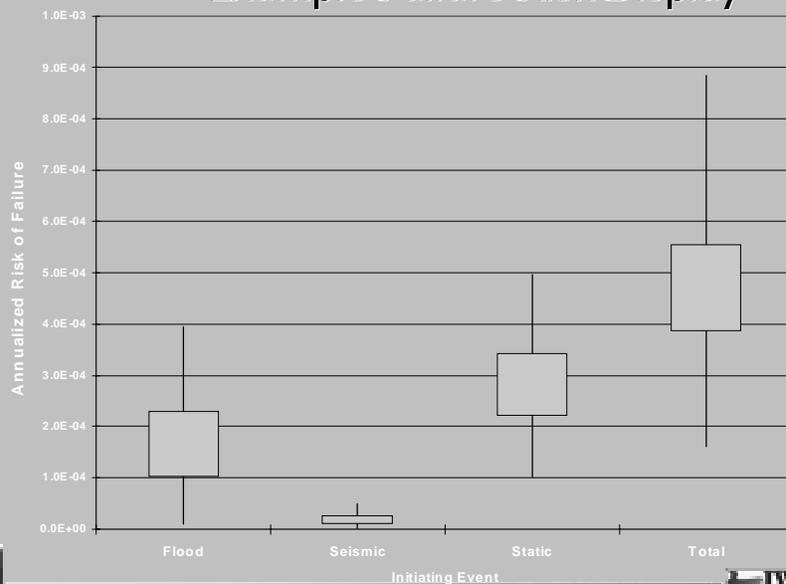


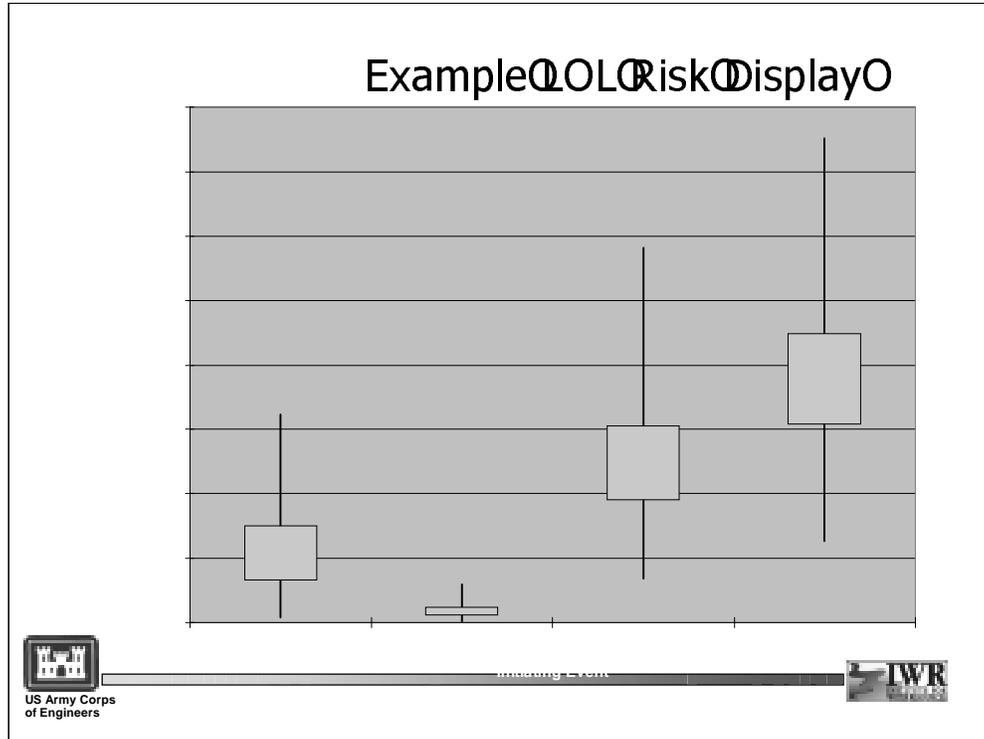
Risk Analysis R&D Focus

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



Example Failure Risk Display





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



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



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



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



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



B-4

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

by Tony L. Wahl¹

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 is 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 $\pm 75\%$ 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.

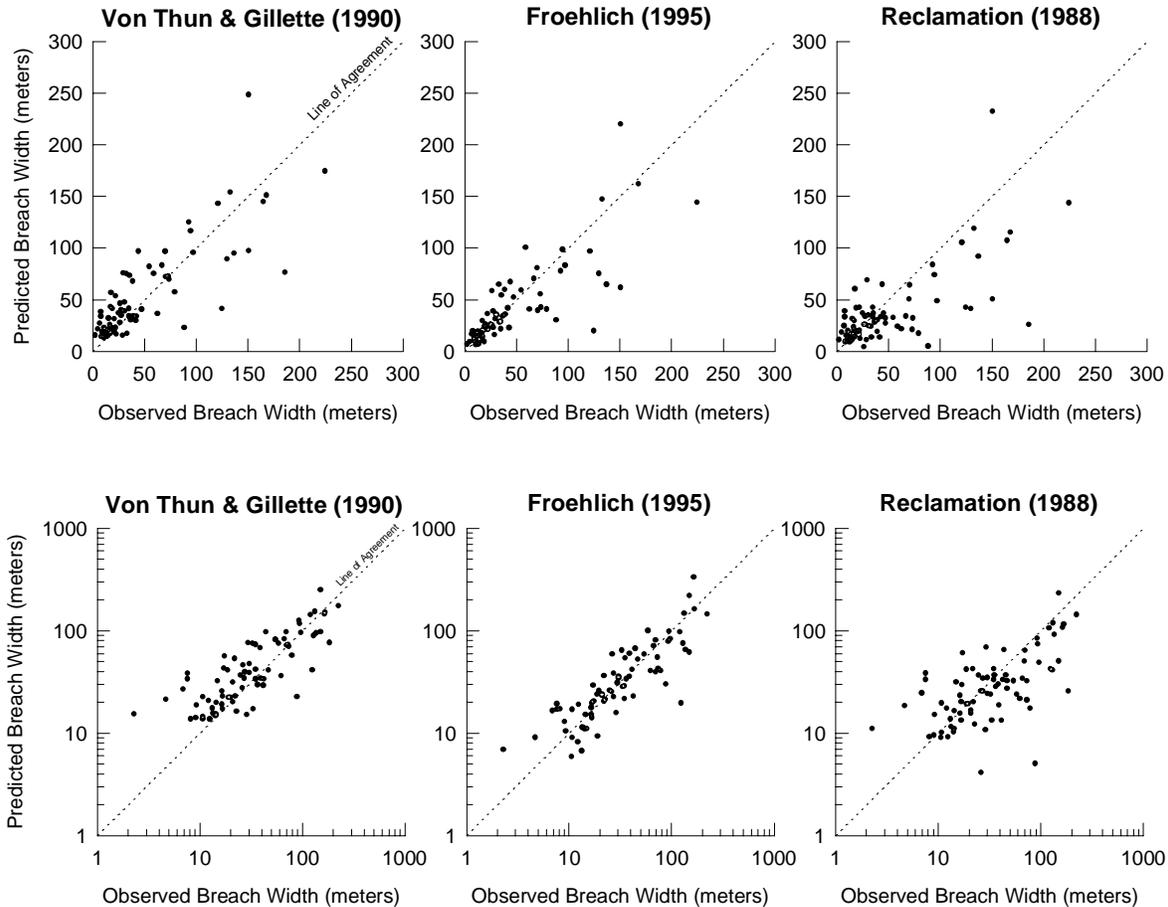


Figure 1. — Predicted and observed breach widths (Wahl 1998), plotted arithmetically (top) and on log-log scales (bottom).

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, m³, m³/s). Failure times are computed in hours.

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

Equation	Number of Case Studies		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
	Before outlier exclusion	After outlier exclusion			
PEAK FLOW EQUATIONS (continued)					
<u>Singh and Snorrason (1984)</u>					
$Q_p = 13.4(h_d)^{1.89}$	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
<u>MacDonald and Langridge-Monopolis (1984)</u>					
$Q_p = 1.154(V_w \cdot h_w)^{0.412}$	37	36	+0.13	±0.70	0.15 — 3.7
$Q_p = 3.85(V_w \cdot h_w)^{0.411}$ <i>envelope equation</i>	37	36	+0.64	±0.70	0.05 — 1.1
<u>Costa (1985)</u>					
$Q_p = 1.122(S)^{0.57}$ <i>envelope equation</i>	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
$Q_p = 2.634(S \cdot h_d)^{0.44}$ <i>envelope equation</i>	31	30	+0.64	±0.72	0.04 — 1.22
<u>Evans (1986)</u>					
$Q_p = 0.72(V_w)^{0.53}$	39	39	+0.29	±0.93	0.06 — 4.4
<u>Froehlich (1995a)</u>					
$Q_p = 0.607(V_w^{0.295} h_w^{1.24})$	32	31	-0.04	±0.32	0.53 — 2.3
<u>Walder and O'Connor (1997)</u>					
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.

\bar{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, m³/s

S = reservoir storage, m³

t_f = failure time, hours

V_{er} = volume of embankment material eroded, m³

V_w = volume of water stored above breach invert at time of failure, m³

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.

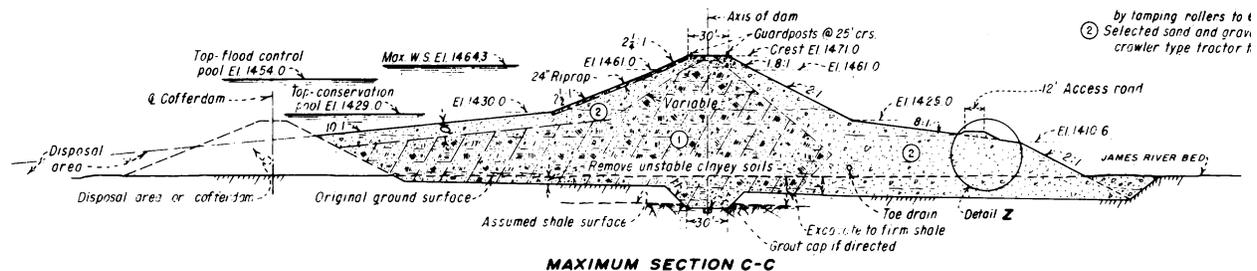


Figure 3. — Cross-section through Jamestown Dam.

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 yd³).

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 ft³/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 $Q_p = f(S \cdot h_d)$	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 $Q_p = f(S \cdot h_d)$ (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.

Initial elev. of piping failure, ft →	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)	
	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 $\pm 1/3$ 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 $\pm 1/3$ 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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B-5

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-break modeling and the associated flood routing.

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

Finally, future research/development directions in dam-break prediction capabilities are presented. These are judged to offer the most efficient and effective means of improving practical dam-break modeling and dam-break 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 ($0.5 \leq b_{av}/H_d \leq 8$) 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 after the downstream face of the dam has eroded away

YEAR	DAM - BREACH OUTFLOW			DAM-BREACH Flood Routing
	Numerical	Parametric Analytical	Physical/ Numerical Erosion/Collapse/Hydraulics	
1969-70	$Q=f(a, t_0, S_0, Q_0)$ Fread, Dissertation	$Q=f(B, a, t_0, H_0, S_0)$ Fread	----	----
1977	NWS DAMBRK Last version: 1991	----	----	NWS DAMBRK Last Version: 1991 1-D
1981	----	$Q=3 \text{ IB}, \left(\frac{C}{t_f + C_1 \sqrt{H_d}}\right)^3$ $C=23.4 \text{ S}/H_d$	----	NWS SMPDBK Last Version: 1991 Curves from DAMBRK
1981-88	----	$Q=f(H_d, V_d, \dots)$ 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/Seil (Bechtleer/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 (1971,1977,1988,1993) 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

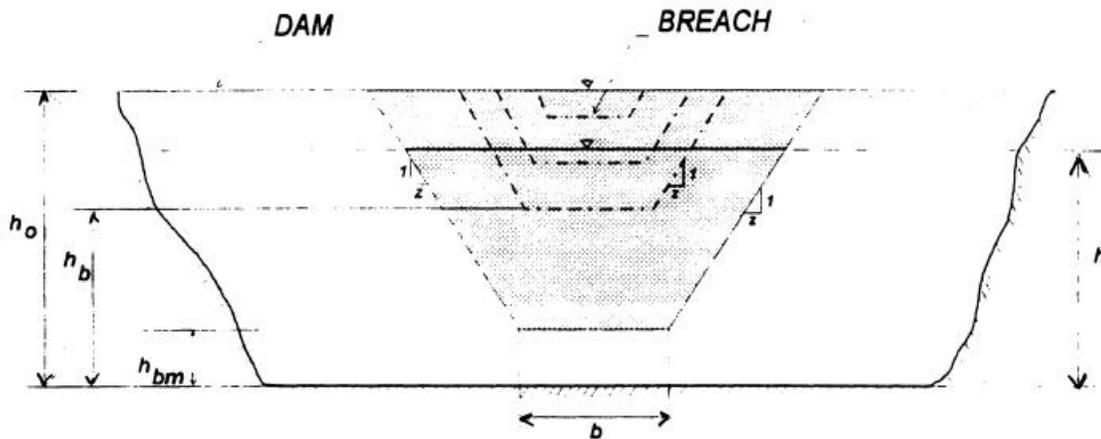


Figure 1. Front View of Dam Showing Formation of Breach.

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:

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

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:

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

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 $1 < r < 4$. The linear rate is usually assumed, although the nonlinear 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:

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

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(1987,1995). 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:

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

$$t_f = 0.3V_r^{0.53}/H^{0.9} \quad (5)$$

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 (1981,1984) 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.

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

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, 1982; Evans, 1986; Costa, 1988; Froehlich, 1995. An example of this type of equation follows:

$$Q_p = a V_r^b H_d^c \quad (7)$$

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 $a = 40.1$, $b = 0.295$, $c = 1.24$ 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 (1965) 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

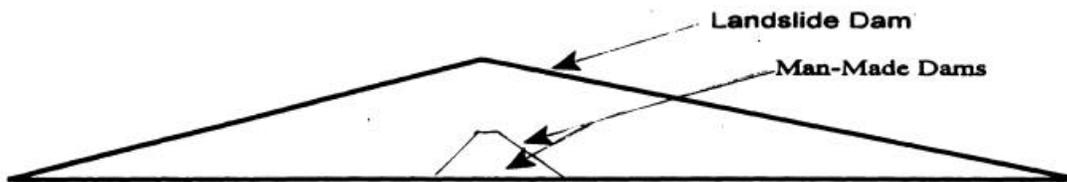


Figure 2. Comparative View of Natural Landslide Dams and Man-Made Dams.

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(γ), internal friction angle (ϕ), 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

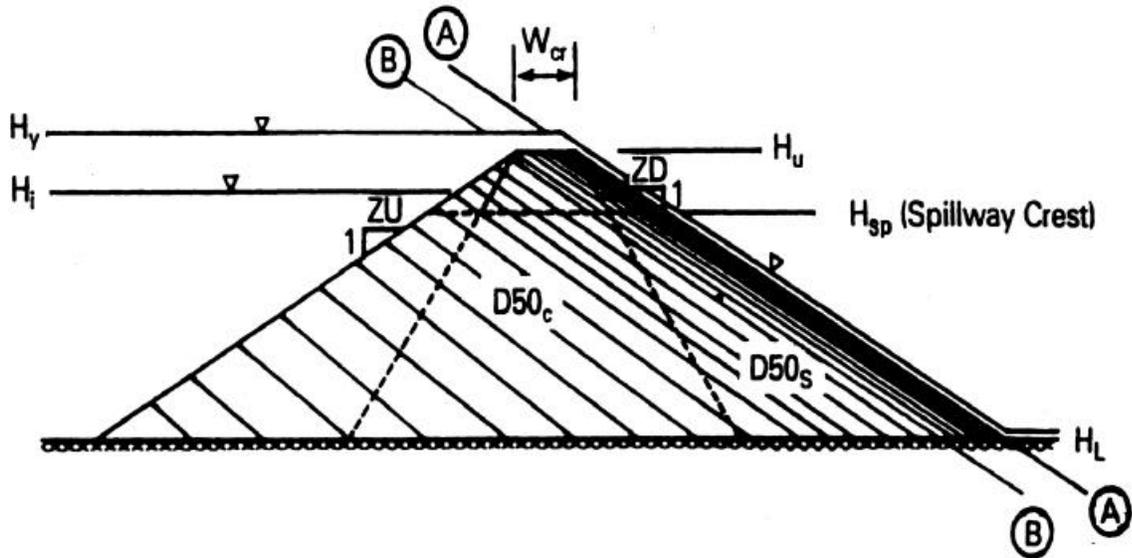


Figure 3. Side View of Dam Showing Conceptualized Overtopping Failure Sequence.

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

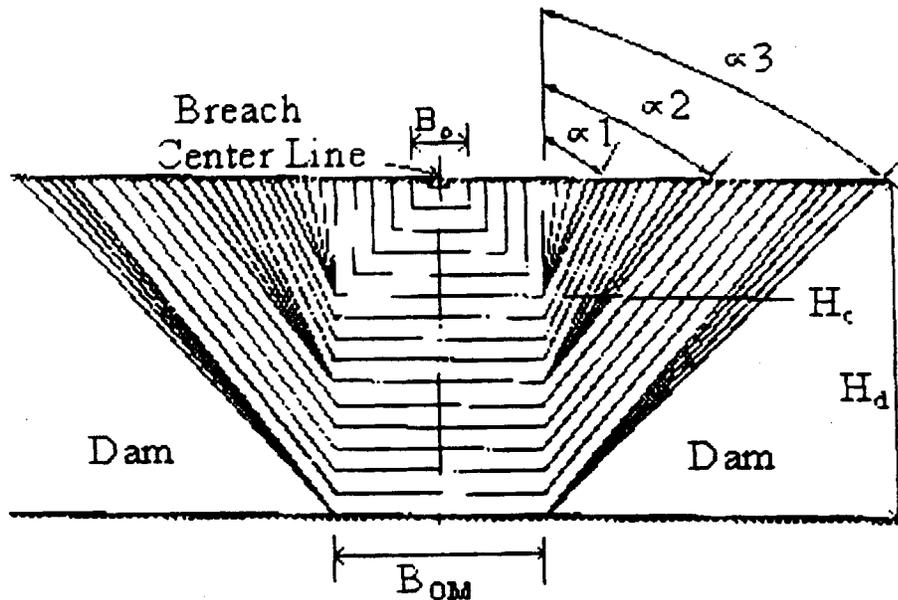


Figure 4. Front View of Dam with Breach Formation Sequence.

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 ± 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 overtopping 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, $D_{50}=0.1$ mm, $f=20$ deg, $C_h=30$ lb/ft², and $g=100$ lb/ft³. 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, $D_{50}=0.25$ mm, $f=25$ deg, $C_h=100$ lb/ ft², and $g=100$ lb/ft³. 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.

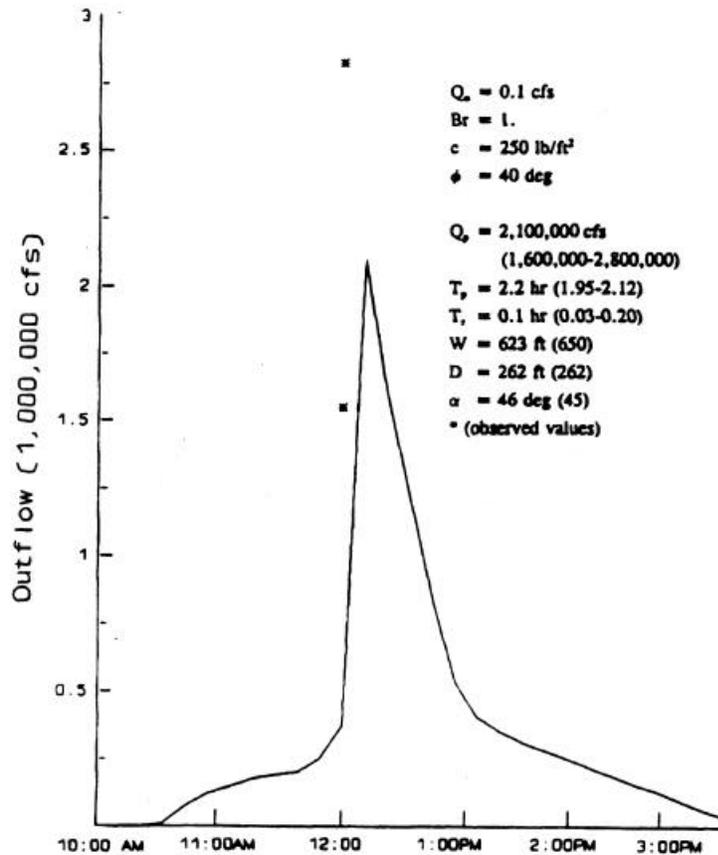


Figure 5. Teton Dam: Predicted and Observed Breach Outflow Hydrograph and Breach Properties

(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 5.6×10^{10} ft³, 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 (1981) 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, 1987) to the Mantaro landslide-formed dam using the following parameters:

upstream face slope 1:17, downstream face slope 1:8, $H_d=560$ ft, $D_{50}=11$ mm, $C_h=30$ lb/ft², $f=38$ deg, $g=100$ lb/ft³. 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.

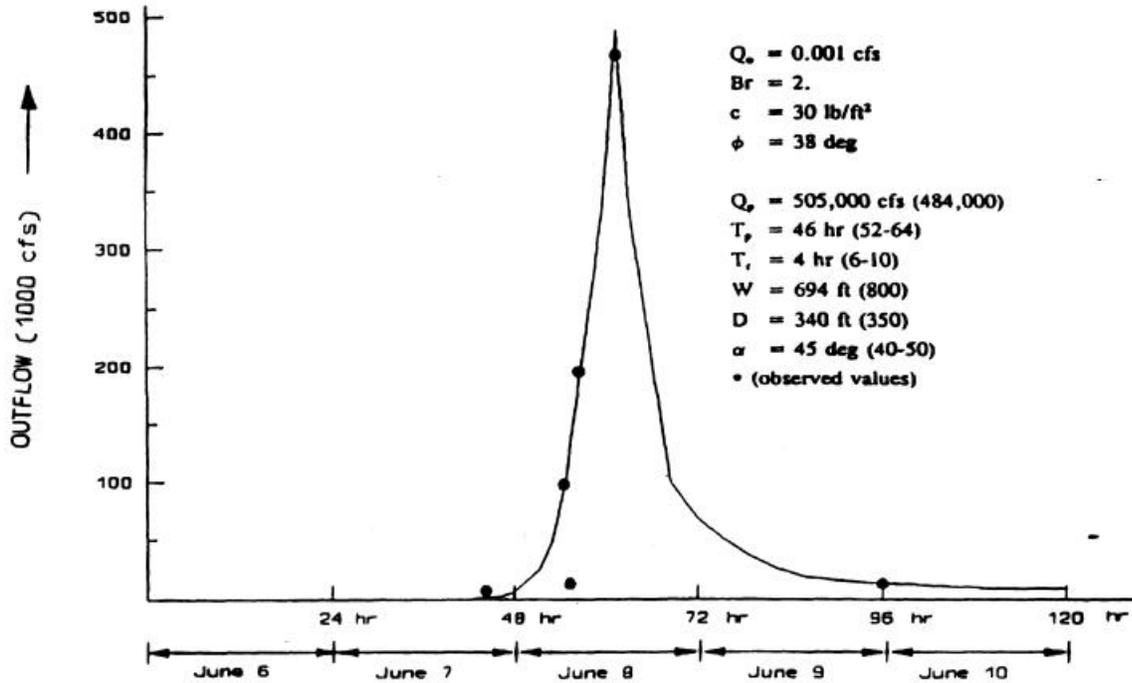


Figure 6. Mantaro Landslide Dam: Predicted and Observed Breach Hydrograph and Breach Properties.

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-break 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-break 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.

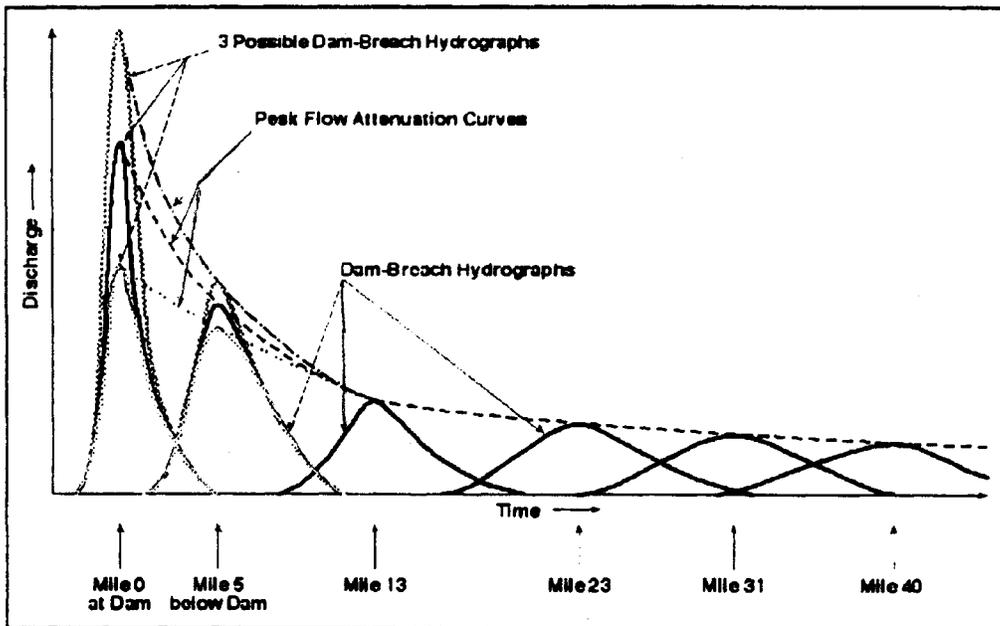


Figure 7. Dam-Break Flood Wave Attenuation Along the Routing Reach.

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, 1978) and FLDWAV (Fread, 1993). The Saint-Venant equations used in these models consists of the mass conservation equation, i.e.,

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

and the momentum conservation equation, i.e.,

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

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.

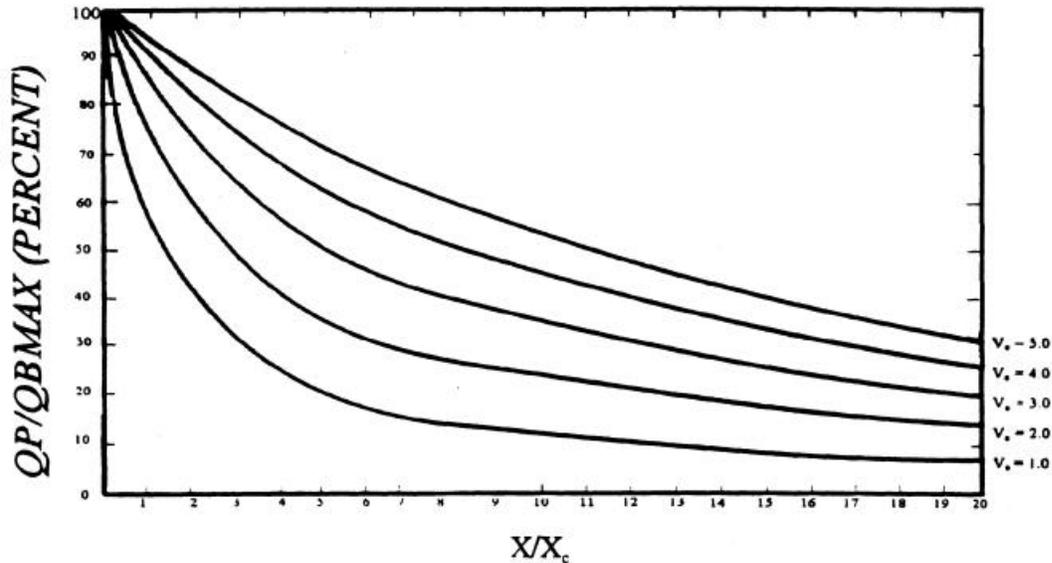


Figure 8. Routing Curves for SMPDBK Model for Froude No. = 0.25.

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:

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

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 $V_o = V_r / (c X_c)$ 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, 1987) 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 ($S_0 > 0.003$ ft/ft). The spatially distributed Muskingum-Cunge routing equation applicable to each Δx_i subreach for each Δt^j time step is as follows:

$$Q_{i+1}^{j+1} = C_1 Q_i^{j+1} + C_2 Q_i^j + C_3 Q_{i+1}^j + C_4 \quad (11)$$

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

$$C_0 = 2K(1-X) + \Delta t^j \quad (12)$$

$$C_1 = (\Delta t^j - 2KX) / C_0 \quad (13)$$

$$C_2 = (\Delta t^j + 2KX) / C_0 \quad (14)$$

$$C_3 = [2K(1-X) - \Delta t^j] / C_0 \quad (15)$$

$$C_4 = q_i \Delta x_i \Delta t^j / C_0 \quad (16)$$

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 Δx_i subreach of the river, and q_i the lateral inflow (+) or outflow (-) along the Δx_i subreach.

K and X are computed as follows:

$$K = \overline{Dx_i}/c \quad (17)$$

$$X = 0.5[1-D/(k'\overline{Dx_i})] \quad (18)$$

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 $\overline{Dx_i}$ reach and over the $\overline{Dt^j}$ time step. The coefficients (C_0, C_1, C_2, C_3, C_4) are functions of $\overline{Dx_i}$ and $\overline{Dt^j}$ (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.,

$$Q = m/nAR^{2/3}S^{1/2} \quad (19)$$

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

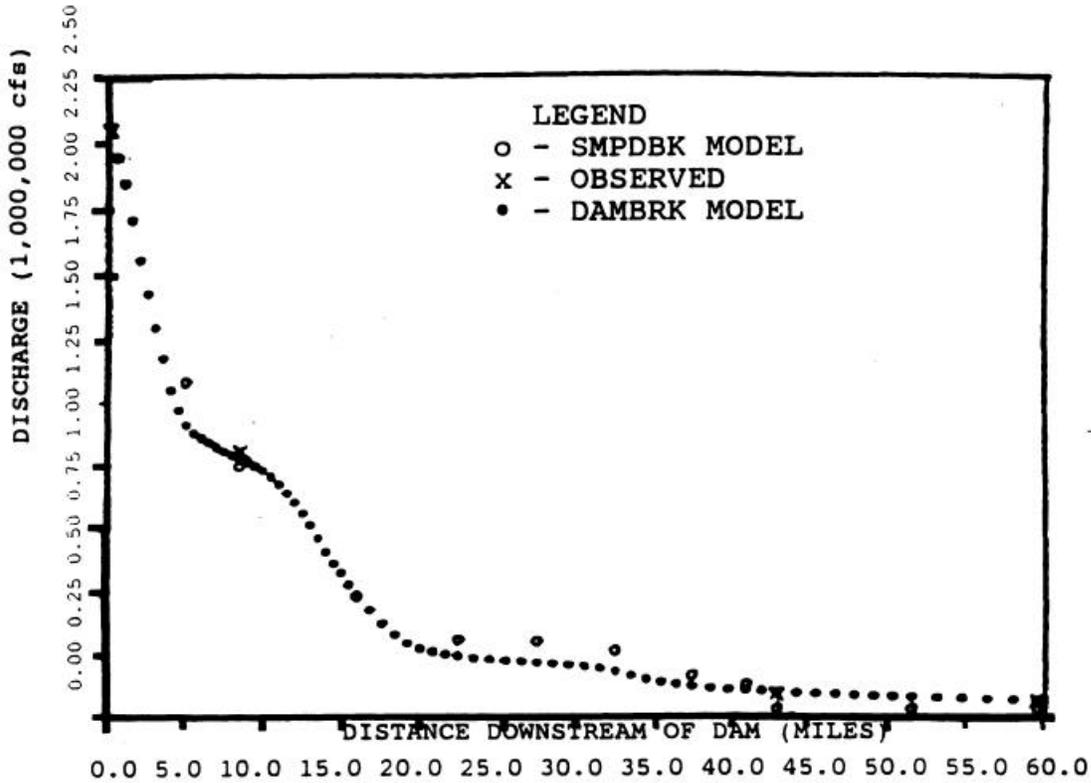


Figure 9. Profile of peak discharge downstream of Teton.

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, $b=170$ 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 $S_0 < 0.004/t_f^{0.89}$; the error increased as the bottom slope became more mild and as the time of failure (t_f) became smaller.

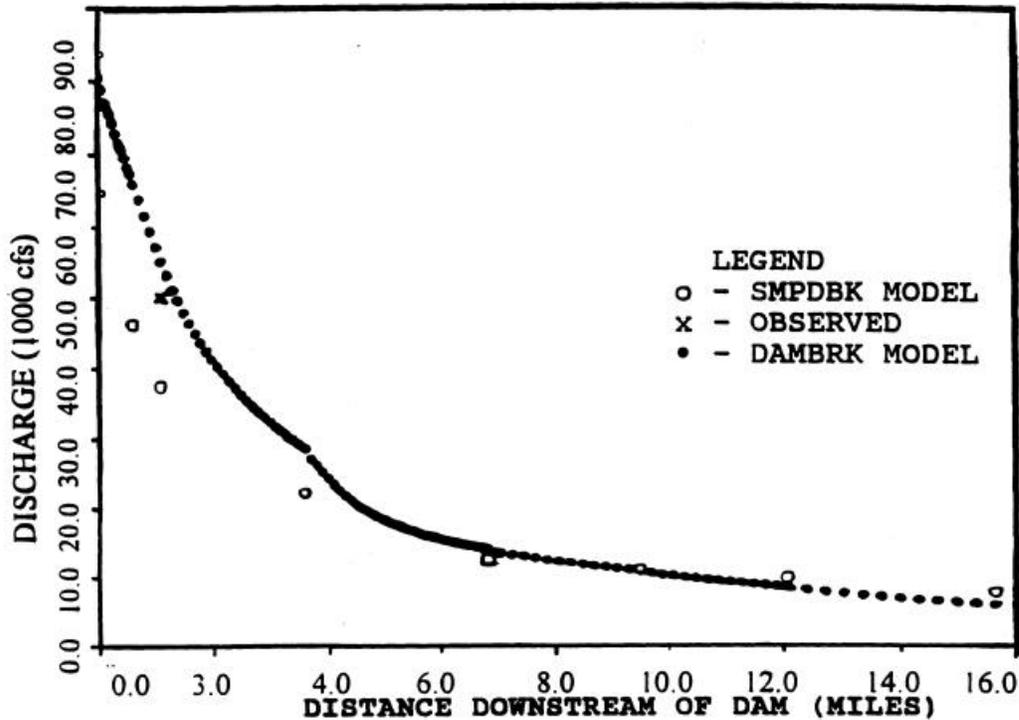


Figure 10. Profile of peak discharge downstream of Buffalo Creek

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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B-6

**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 Centre.

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 the seminar 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.

MAP OF THE KYRÖNJOKI RIVER BASIN

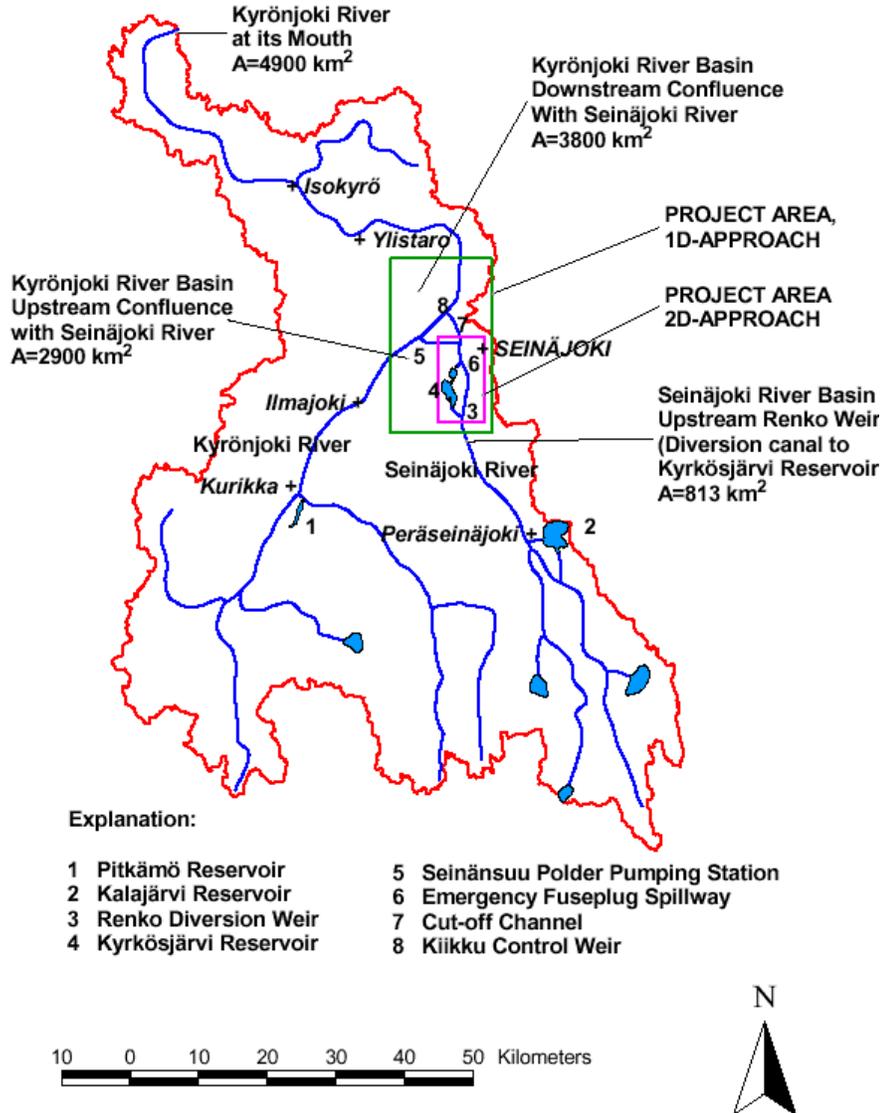


Figure 1. Map of the Kyrönjoki River basin

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 through 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 $15.8 \cdot 10^6 \text{ m}^3$ at the flood HW-level 81.25 m and $22.3 \cdot 10^6 \text{ m}^3$ 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 through 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 m³/s and at low water level differences, at least 25 m³/s. The discharge from the reservoir through the turbines of the hydropower plant can be 21 m³/s and through a discharge valve 2 m³/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.

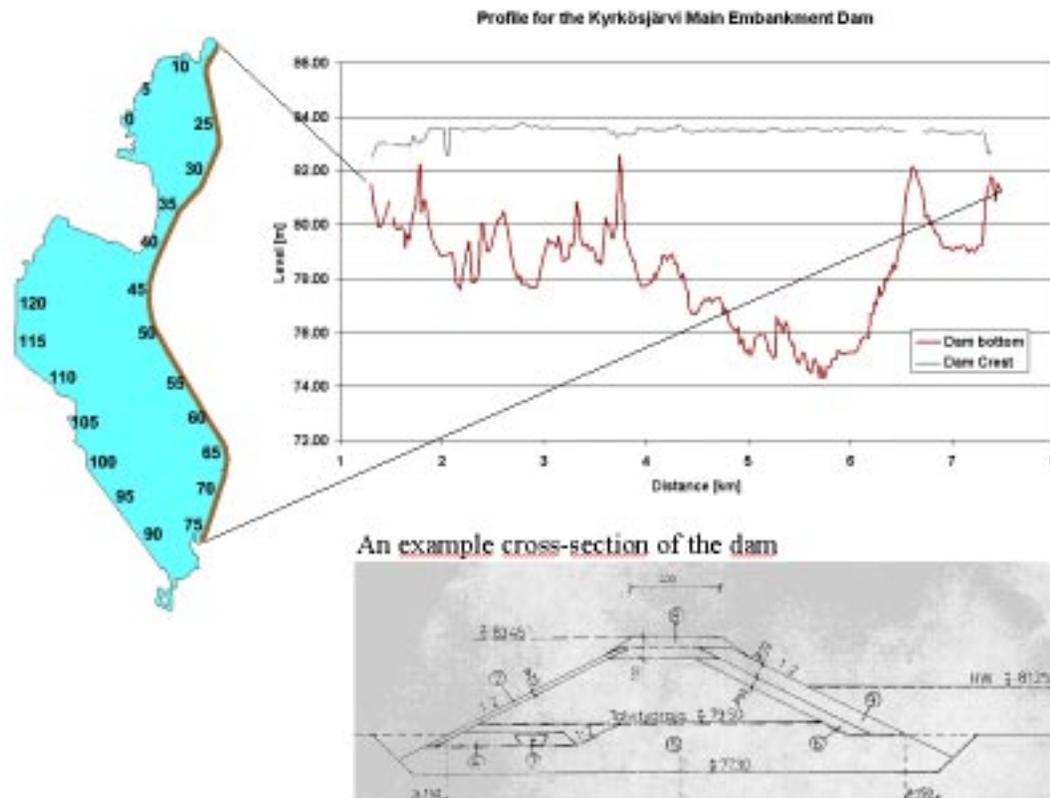


Figure 2. Kyrkösjärvi Embankment Dam

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.

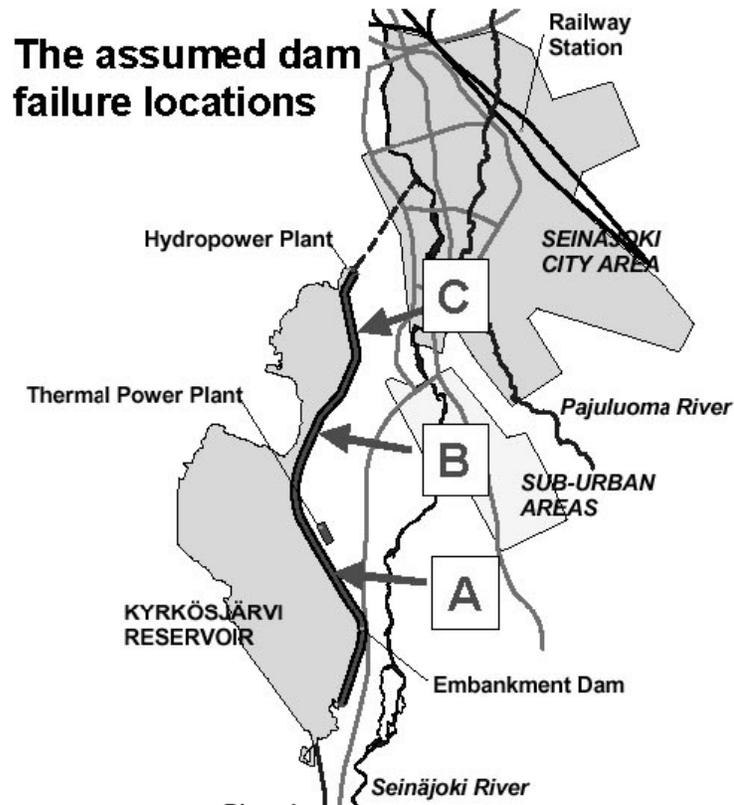


Figure 3. The locations of the assumed dam breach sites.

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.

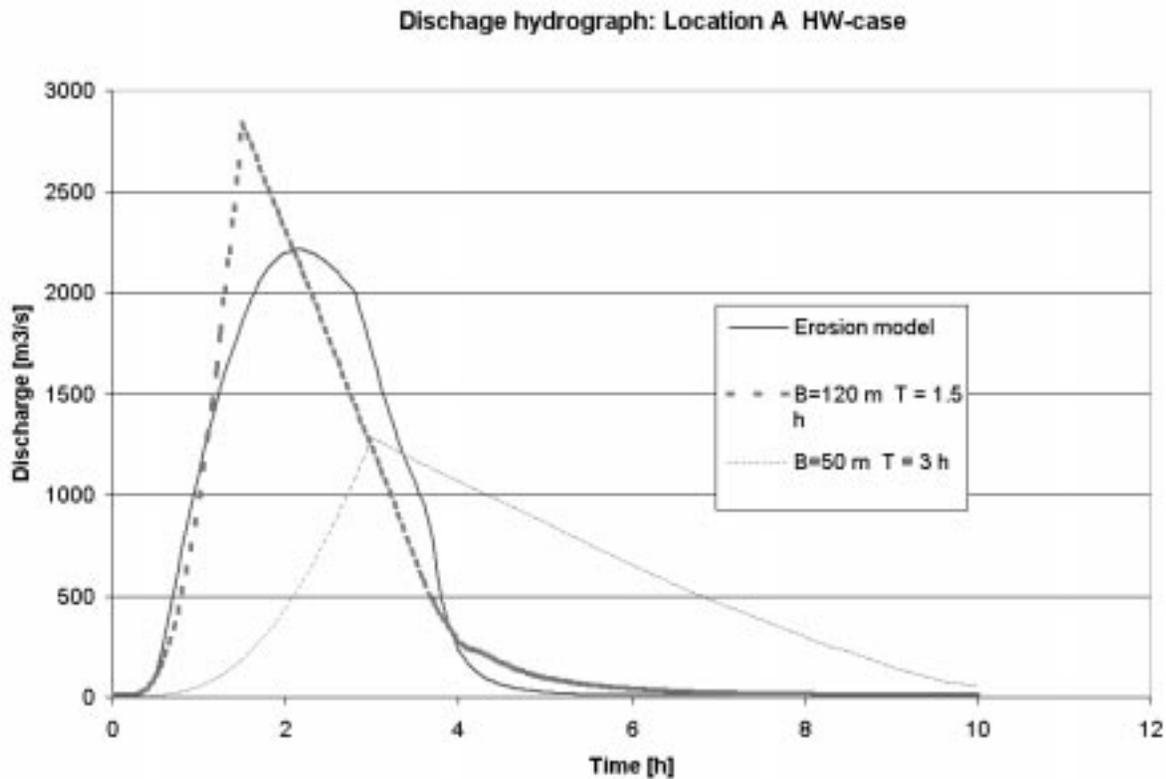


Figure 4. Breach hydrographs for the location A in the $HQ_{1/100}$ -case

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 were 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 gridding 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.

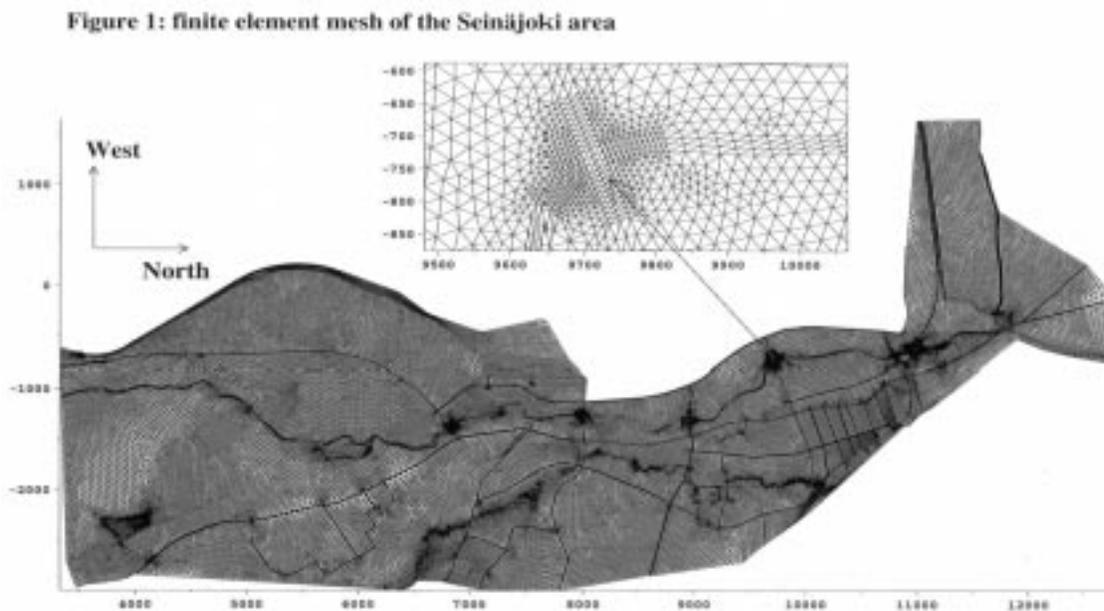


Figure 5. The Telemac-2D finite element mesh used in the Kyrkösjärvi simulations by EDF.

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 m ³ /s + breach hydrograph for max reservoir level, roughness varies between Strickler 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 | HQ _{1/100} 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 created 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 km² 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.

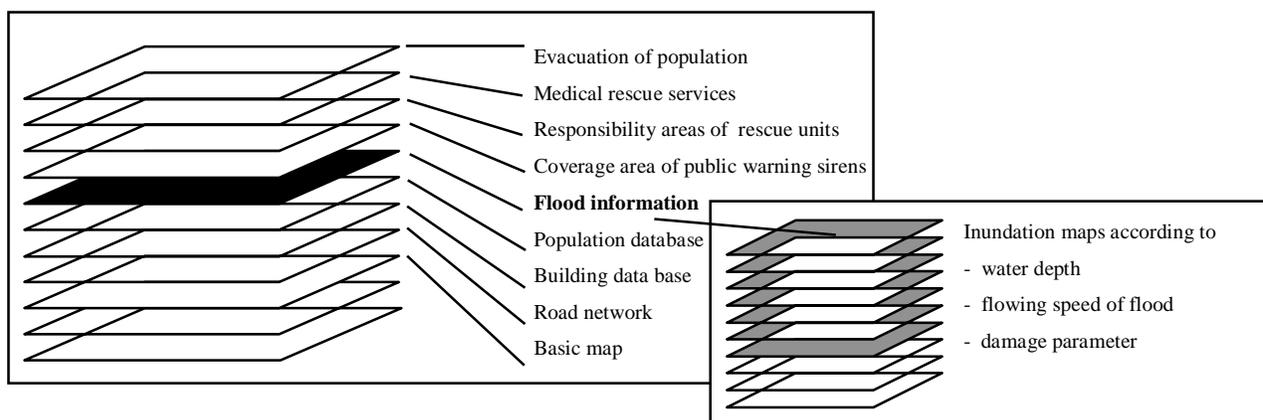


Figure 6 .Usage principles of material needed in rescue operations

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. CONCLUSIONS

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

FEMA/USDA WORKSHOP OKLAHOMA CITY JUNE 26, 2001

**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 than 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-foot 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 BASED 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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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:

$$B = 9.5K_o(V_s H)^{0.25}$$

$$T_f = 0.59(V_s^{0.47})/(H^{0.91})$$

where:

B = average breach width (ft)

T_f = time of failure (hrs)

K_o = 0.7 for piping and 1.0 for overtopping failure

V_s = storage volume (ac-ft)

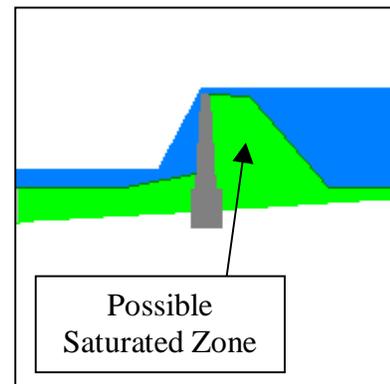
H = height (ft) of water over breach bottom

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



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

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.

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