# Federal Guidelines for Inundation Mapping of Flood Risks Associated with Dam Incidents and Failures

First Edition

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**FEMA**

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| ACER | Assistant Commissioner – Engineering and Research |
| --- | --- |
| ARS | Agricultural Research Service |
| ASDSO | Association of State Dam Safety Officials |
| Att-Kin | Attenuation-Kinematic Flood Routing Method |
| DAMBRK | Dam Break Forecasting Model |
| DAMS | Structure Site Analysis Program |
| DEM | Digital Elevation Models |
| DHS | Department of Homeland Security |
| DSAT | Dams Sector Analysis Tool |
| DSS-WISE | Numerical Simulation-Based Decision Support System for Water Infrastructural Security |
| DTM | Digital Terrain Model |
| DWOPER | Dynamic Wave Operational Model |
| EAP | Emergency Action Plan |
| FERC | Federal Energy Regulatory Commission |
| FEMA | Federal Emergency Management Agency |
| FLO-2D[©] | Flow Two-Dimensional Program |
| FLDWAV | Flood Wave Dynamic Model |
| GeoDamBREACH | Geospatial Dam Breach Rapid Emergency Action Plan, Consequences and Hazards |
| HAZUS | Hazards United States Multi-Hazard |
| HEC-RAS | Hydrologic Engineering Center River Analysis System |
| HMR | Hydrometrological Report |
| HMS | Hydrologic Modeling System |
| ICODS | Interagency Committee on Dam Safety |
| IDF | Inflow Design Flood |
| IFSAR | Interferometric Synthetic Aperture Radar |
| Lidar | Light Detection and Ranging |
| NCCHE | National Center for Computational Hydroscience and Engineering |
| NDSP | National Dam Safety Program |
| NDSRB | National Dam Safety Review Board |
| NED | National Elevation Dataset |
| NEM | National Engineering Manual |
| NFIP | National Flood Insurance Program |
| NID | National Inventory of Dams |
| NOAA | National Oceanic and Atmospheric Administration |

| | |
|---|---|
| NPDP | National Performance of Dams Program |
| NRCS | Natural Resources Conservation Service |
| NWS | National Weather Service |
| PL | Public Law |
| PMF | Probable Maximum Flood |
| PMP | Probable Maximum Precipitation |
| RAMPP | Risk Assessment, Mapping, and Planning Partners a Joint Venture of URS Corporation, Dewberry and ESP |
| RCC | Roller-compacted Concrete |
| Risk MAP | Risk Mapping, Assessment, and Planning |
| SCS | Soil Conservation Service |
| SDF | Spillway Design Flood |
| SIMBA | Simplified Breach Analysis |
| SITES | Water Resource Site Analysis Computer Program |
| SMPDBK | Simplified Dam Break |
| TIGER | Topographically Integrated Geographic Encoding and Referencing |
| TIN | Triangulated Irregular Network |
| TM | Technical Memorandum |
| TR | Technical Release |
| USACE | U.S. Army Corps of Engineers |
| USBR | U.S. Bureau of Reclamation |
| USDA | U.S. Department of Agriculture |
| USDOI | U.S. Department of the Interior |
| USGS | U.S. Geological Survey |
| WinDAM | Windows Dam Analysis Modules |

The *Federal Guidelines for Inundation Mapping of Flood Risks Associated with Dam Incidents and Failures* is presented by the Federal Emergency Management Agency (FEMA) as part of the National Dam Safety Program (NDSP), a partnership of States, Federal agencies, and other stakeholders formed to encourage individual and community responsibility for dam safety. As part of the NDSP, States are responsible for regulating non-Federal dams and do so autonomously from the Federal government and other States. This document provides information for Federal and State agencies, local governments, dam owners, and emergency management officials to use for reducing flood hazards and the resulting potential for economic damage and loss of life. This document is intended as a resource for developing State-specific guidelines for dam safety and as a reference manual for dam safety professionals to map dam breach inundation zones. FEMA developed the Geospatial Dam Break, Emergency Action Planning, Consequences and Hazards (Geo-DamBREACH) toolset to support the application of this document. The Geo-DamBREACH toolset will be available for download from fema.gov beginning in 2013.

Dam breach inundation zone mapping is a foundational element used for a wide variety of purposes, including designing dams, classifying hazard potential, emergency management planning, evaluating consequences, and communicating risk. Because each State is autonomous within the NDSP, developing a single national standard for dam breach modeling and mapping is not realistic. Therefore, this document presents a variety of information that individual States can incorporate specific to their needs and as allowed by their enabling legislation.

This document was developed to provide end users with information and guidance that can be applied across different geographic regions and to various site-specific conditions. It presents a mix of research, reference material, and recommendations. Specific recommendations are provided for procedures where standardization across the United States is needed and possible. Recommendations in this document include using a tiered study approach for modeling and mapping, standardizing Emergency Action Plans (EAPs) and risk communication mapping products, and organizing dam breach modeling data through a standardized database structure.

The recommendation to use a tiered study approach for dam breach modeling and mapping is included to support the increased development of breach inundation zone mapping and associated EAPs where studies are constrained by limited funds, as is commonly the case. The premise of the tiered study approach is that having a simplified dam breach inundation zone and EAP is better than having none, and having simplified breach information will help steer limited funding to the highest priority dams in need of more detailed evaluations.

Also included in this document are specific recommendations for standardizing the EAP and risk communication mapping products used by local emergency management officials, first responders, and local elected officials. This guidance recommends standardizing the database structure for organizing dam breach modeling data to simplify database maintenance and allow data to be used for a variety of dam safety products.

# SECTION 1  INTRODUCTION

There are no available Federal agency guidelines specifically developed for non-Federal entities that provide both current modeling and geographic information system (GIS)-based mapping standards for dam breach inundation mapping efforts that can be used directly by States to regulate the preparation of consistent dam breach inundation studies. The models and maps created as part of a dam breach inundation study can be used for hazard potential classification, in developing Emergency Action Plans (EAPs), and to conduct dam breach consequence and hazard mitigation studies.

## 1.1  PURPOSE

The purpose of this document is to provide dam safety professionals with guidance on how to prepare dam breach inundation modeling studies and conduct mapping that can be used for multiple purposes, including dam safety, hazard mitigation, consequence evaluation, and emergency management including developing EAPs. This guidance is intended to provide a consistent approach that can be applied across the country.  If adopted by individual States, the standardized methods and approaches presented in this document can be leveraged for a variety of dam safety products and across jurisdictional boundaries to strengthen dam safety across the United States.

This document is not intended to establish dam safety policy or provide requirements and specifications, but rather to function as a compilation of the best technical resources and practices available for inundation modeling and mapping. It is also not intended to provide a complete manual of all procedures available for dam breach analysis and mapping. Topics include conducting breach assessments, methods for estimating breach parameters and generating breach hydrographs, downstream inundation modeling, and generation of consistent inundation maps. This guidance document is intended to aid in the areas of dam safety, hazard mitigation planning, consequence and loss estimation, and emergency management.

## 1.2  POTENTIAL USES OF INUNDATION MAPPING

Inundation maps can have a variety of uses including EAPs, mitigation planning, emergency response, and consequence assessment. Each use has unique information requirements and may be used in different manners ranging from multi-year office-based planning efforts by mitigation planners and dam safety officials to field-based emergency responders responding to a developing or imminent dam breach.

**Emergency Action Plans:** An EAP is a formal document that identifies potential emergency conditions at a dam and specifies preplanned actions to be followed to minimize property damage and loss of life. The EAP specifies actions the dam owner, in coordination with emergency management authorities, should take to respond to incidents or emergencies related to the dam. It presents procedures and information to assist the dam owner in issuing early warning and notification messages to responsible downstream emergency management authorities. The EAP includes inundation maps to assist the dam owner and emergency management authorities with identifying critical infrastructure and population-at-risk sites that may require protective

measures and warning and evacuation planning. The EAP must clearly delineate the responsibilities of all those involved in managing the incident and how those responsibilities should be coordinated.

**Emergency Response:** Emergency response embodies the actions taken during and in the immediate aftermath of an incident to save and sustain lives, meet basic human needs, and reduce the loss of property and the effect on critical infrastructure and the environment. In the case of dam failures and incidents, this would be the response by the dam owner, local community emergency management, and first responders such as fire and police departments to minimize the consequences of an imminent or actual dam failure or incident. Actions may include warning and evacuating the population at risk. It is important for dam owners to coordinate with the appropriate emergency management authorities and provide information obtained through dam inundation studies and mapping projects to assist the evacuation planning process.

**Hazard Mitigation Planning:** Mitigation is the proactive effort to reduce loss of life and property by lessening the effect of disasters. This is achieved through identifying potential hazards and the risks they pose in a given area, identifying mitigation alternatives to reduce the risk, and risk analysis of mitigation alternatives. The result is the selection of proactive measures, both structural and non-structural, that will reduce economic losses and potential loss of life when implemented. Inundation maps can provide hazard mitigation planners information about at-risk population and structures; such information can then be used to identify actions to reduce their vulnerability. Actions might include setting up a reverse 911 system to provide advanced flood warning and relocating critical infrastructure and facilities out of the inundation zone.

**Dam Failure Consequence Assessment:** Dam breach consequence assessment includes identifying and quantifying the potential consequences of a dam failure or incident. While hazard mitigation planning focuses on specific projects to reduce flood risk, consequence assessment focuses on the economic and social impacts of a potential disaster and the organizational and government actions needed in the aftermath of a dam breach to respond and recover. Data compiled for a consequence assessment can also be used in risk assessments. Both consequence assessment and risk assessments require the type of information captured on dam breach inundation maps.

## 1.3    SUMMARY OF PROJECT APPROACH

To create this document in cooperation with interested end users and partners, a two-phased approach was employed. During the first phase, research was conducted to obtain published dam breach and inundation mapping guidelines prepared by, or in use by, established State dam safety programs from all 50 States. The published guidelines were reviewed to extract details of their dam breach and inundation mapping guidelines, including referenced external guidelines. Since many States reference the dam breach guidelines of Federal agencies, the available published documents of all Interagency Committee on Dam Safety (ICODS) participants, as well as various dam safety organizations including the Association of State Dam Safety Officials (ASDSO) and the Association of State Floodplain Managers (ASFPM), were included in this review.

To determine the needs of various end users, telephone interviews were conducted with representatives of a number of State dam safety programs and State Emergency Managers to solicit their ideas for this new guidance document.

The results of the first phase of research and evaluation of existing guidelines, including recommendations for the content of this guidance document, were submitted to the Federal Emergency Management Agency (FEMA) as a report titled *Draft Data Report*. The *Draft Data Report* was reviewed by a Work Group established by the National Dam Safety Review Board (NDSRB). In addition, a number of presentations were made at ASFPM and ASDSO conferences, to the NDSRB and FEMA Levee and Dam Integrated Policy Team.

The second phase of report development was preparation of a draft report and subsequent review and comment from the NDSRB Work Group and FEMA staff representing the Levee and Dam Integrated Policy Team. The content from a number of Federal publications or presentations sponsored by Federal agencies is directly incorporated into this document. The NDSRB Workgroup included dam safety professionals representing the U.S. Army Corps of Engineers (USACE), U.S. Bureau of Reclamation (USBR), Federal Energy Regulatory Commission (FERC), Natural Resources Conservation Service (NRCS), and a State Dam Safety Program representative. The review comments and input of the NDSRB was incorporated to create this consensus document. An important outcome of the review process was the identified need for modeling and mapping standards that could be applied by the States to regulate the development of consistent dam breach modeling that could be used for multiple purposes including hazard potential classification, EAP development, dam breach consequence analysis, and hazard mitigation planning.

In addition, the reviews identified the need for the use of simplified dam breach inundation mapping methods that could be applied for low-hazard potential dams or as an initial (first pass) modeling of any dam without a dam breach inundation zone map. In response to the need for simplified dam breach inundation mapping methods, FEMA commissioned the development of the Geospatial Dam Breach Rapid EAP, Consequences and Hazards (GeoDamBREACH) Toolset that provides a geographic information system (GIS)-based method to develop simplified dam breach inundation zone mapping and a semi-automated EAP report with maps. The modeling and mapping standards recommended in this document are consistent with the GeoDamBREACH output, which is consistent with FEMA standards for the development of non-regulatory products for dams.

## 1.4 ORGANIZATION OF DOCUMENT

This document presents the following Sections:

- **Section 1.0 Introduction**: Includes the purpose, potential uses of inundation maps, and a summary of the approach taken in developing this document.

- **Section 2.0 Background**: Includes general background information for users of this guidance document, including history of inundation mapping and the dam safety

program; definition of a dam; description of the dam safety program, and a general discussion about current dam safety in the United States.

- **Section 3.0 Types of Dams:** Describes the history, use, and general construction of concrete, embankment, timber, and stone dams

- **Section 4.0 Causes of Dam Failures**: Describes dam failure modes including hydrologic, geologic, structural, seismic, and human-influenced.

- **Section 5.0 Dam Classification Systems**: Describes the dam classification systems in use in the United States, including hazard potential, size, and probably loss of life.

- **Section 6.0 Dam Breach Analysis Study Approaches**: Describes event-based and risk-based approaches, and presents a tiered dam breach analysis approach.

- **Section 7.0 Hydrologic Analyses**: Describes the steps required to perform the hydrologic analyses that precede dam breach inundation mapping.

- **Section 8.0 Downstream Routing Analysis:** Describes non-hydrologic (fair weather) and hydrologic dam breach failures, as well as sequential dam failures.

- **Section 9.0 Estimating Breach Parameters:** Presents specific information regarding breach parameters and breach mechanisms.

- **Section 10.0 Analysis Tools for Dam Failure Modeling:** Provides detailed information on the history of dam breach models, a discussion of the tools available, and recommendations for selecting appropriate models.

- **Section 11.0 Dam Breach Mapping Guidance:** Presents a systematic approach to creating inundation maps for different end users and storing mapping information in an inundation database that can accessed easily for updating materials and modifying maps for different end users.

## SECTION 2 BACKGROUND

This section provides background information on the history of inundation mapping and the dam safety program (Section 2.1), presents the definition of a dam (Section 2.2), describes the National Dam Safety Program (NDSP) (Section 2.3), and describes dam safety and EAP statistics (Section 2.4).

## 2.1 HISTORY OF INUNDATION MAPPING AND THE DAM SAFETY PROGRAM

The State of California enacted a dam safety program following the 1929 failure of the St. Francis Dam located in Southern California. Subsequent dam failures across the country that caused loss of life and property resulted in additional legislation at both State and national levels (FEMA, 1979; reprinted in 2004).

Dam breach inundation mapping received national attention following two significant failure incidents in California (the failure of Baldwin Hills Dam in 1964 and the near failure of Lower Van Norman Dam in 1971), which prompted the State of California to enact statutes requiring dam owners to prepare dam failure inundation maps. Prior to the enactment of the **California statutes**, little attention was given to dam failure inundation mapping nationally (Wahl, 2010).

In 1972, Congress enacted the **National Dam Inspection Act of 1972** (33 U.S.C. § 467f) that authorized the Secretary of the Army to inspect non-Federal dams meeting the size and storage limitations of the Act. Responsibilities under the law were delegated to the USACE. Activities performed under this Act included preparation of an inventory of dams in the United States; a survey of each State and Federal agency's capabilities, practices, and regulations regarding the design, construction, operation, and maintenance of dams; development of guidelines for inspecting and evaluating dam safety; and formulating recommendations for a comprehensive national program.

While methods for predicting a dam breach have existed since the mid-1960s, the need for improvements to dam failure modeling and downstream consequence assessment was realized in the 1970s after several fatal dam failures including the Buffalo Creek Dam in West Virginia (1972), Teton Dam in Idaho (1976), and the Laurel Run Dam in Pennsylvania (1977). On April 23, 1977, a **Presidential memorandum** directed Federal agencies to review their dam safety practices, addressing many elements of dam safety. Additionally, the **Reclamation Safety of Dams Act of 1978**, Public Law (PL) 95-578, was enacted November 2, 1978, to authorize the Secretary of the Interior to construct, restore, operate, and maintain new or modified features at existing Federal reclamation dams for safety of dams purposes. This Act was amended in 1984 under PL 98-404, in 2000 under PL 106-377, in 2002 under PL 107-117, and in 2004 under PL 108-439.

In 1979, President Carter created FEMA and directed Federal agencies to adopt and implement **FEMA 93**, *Federal Guidelines for Dam Safety* (FEMA 1979; reprinted in 2004). The **ICODS** was then formed to coordinate Federal activities and work with the States to ensure implementation of dam safety practices.

Since the publication of FEMA 93 in 1979, other Federal acts have been enacted to address major aspects of dam safety, including dam failure studies and inundation mapping. These include the **Water Resources Development Act of 1986**, amended in 1996 to include the NDSP Act and the **Dam Safety and Security Act of 2002**, amended in 2006.

Several ICODS member Federal agencies have published dam safety guidelines, including dam failure analysis and inundation mapping (refer to Table 2-1). Table 2-2 outlines the major events in dam safety regulations in the United States from 1972 to present.

**Table 2-1: Availability of Dam Breach and Inundation Mapping Guidance from ICODS Agencies**

| ICODS Agencies | Dam Breach and Inundation Guidelines |
|---|:---:|
| U.S. Department of Agriculture | |
|     Natural Resources Conservation Service | **Yes** |
|     Agricultural Research Service | **Yes** |
| U.S. Department of Defense | |
|     Army Corps of Engineers | **Yes** |
| U.S. Department of Energy | No |
| U.S. Department of the Interior | |
|     Bureau of Indian Affairs | No |
|     Bureau of Land Management | No |
|     Bureau of Reclamation | **Yes** |
|     Fish and Wildlife Service | **Yes** |
|     Geological Survey | **Yes** |
|     National Park Service | No |
| U.S. Department of Labor | |
|     Mine Safety and Health Administration | **Yes** |
| Federal Energy Regulatory Commission | **Yes** |
| Department of State | |
|     International Boundary and Water Commission | No |
| U.S. Nuclear Regulatory Commission | **Yes** |
| Tennessee Valley Authority | No |
| Department of Homeland Security | |
|     Federal Emergency Management Agency | **Yes** |

**Table 2-2: Major Events in Dam Safety Regulations in the United States**

| | |
|---|---|
| **1972** | Enactment of the National Dam Inspection Act (PL 92-367) following attention to the hazards created by the February 26, 1972, failure of a mine tailings embankment at Buffalo Creek, WV. |
| **1975** | USACE issued a report, *National Program of Inspection of Dams,* which included an inventory of U.S. dams and inspection guidelines. |
| **1976** | Failure of Teton Dam in Idaho due to internal erosion caused $1 billion in losses and 14 deaths. This failure led to widespread review by Federal agencies regarding dam inspection, evaluation, and modification. |
| **1977** | Failure of the Kelly Barnes Lake Dam in Toccoa, GA, caused 39 fatalities due to the resulting flood. |
| **1979** | Executive Order 12148 from President Carter created FEMA. FEMA 93, *Federal Guidelines for Dam Safety* (FEMA 1979; reprinted in 2004), was published. FEMA Director was given authority to coordinate Federal dam safety efforts. ICODS formed. |
| **1982** | U.S. Committee on Large Dams passed a resolution urging State governments to give high priority to enacting dam safety legislation and to allocating resources to dam supervision. |
| **1985** | National Research Council issued the report *Safety of Dams: Flood and Earthquake Criteria* (NRC, 1985). |
| **1985** | Association of Dam Safety Officials (ASDSO) became active. |
| **1986** | Water Resources Development Act of 1986 (PL 99-662) was signed into law providing for creation of a National Dam Safety Review Board; this Act provides for the maintenance and periodically updating of the USACE National Inventory of Dams (NID) list. |
| **1994** | National Performance of Dams Program (NPDP) officially started. The NPDP is a cooperative effort of engineers and dam safety professionals in the United States to create an information resource on dams and their performance of dams. The objectives of the NPDP are to retrieve, archive, and disseminate information on the performance of dams. |
| **1996** | The Water Resources Development Act of 1996 (PL 104-303) was signed into law by President Clinton. The National Dam Safety Program (NDSP) Act was also established through public law (Section 215 of PL 104-303). |
| **2002** | The Dam Safety and Security Act of 2002 was signed into law, reauthorizing the NDSP for four more years with added enhancements to the 1996 Act. |
| **2006** | The Dam Safety and Security Act of 2006 reauthorized as P.L. 109-460; reauthorized NDSP through fiscal year 2011 with added amendments. |

*Source:  Adapted from NPDM, n.d.*

## 2.2    DEFINITION OF A DAM

State dam safety programs are responsible for developing legislation and associated regulations, including the definition of a dam specific to their State. The State Dam Safety Agency should be consulted for the definition of a dam and all dam safety regulations pertaining to any given State.

The National Dam Safety Act of 2006 (P.L. 109-460) defines a dam as:

 "*any artificial barrier that has the ability to impound water, wastewater, or any liquid-borne material, for the purpose of storage or control of water, that*:

*(i) is 25 feet or more in height from:*

> *(I) the natural bed of the stream channel or watercourse measured at the downstream toe of the barrier; or*

> *(II) if the barrier is not across a stream channel or watercourse, from the lowest elevation of the outside limit of the barrier; to the maximum water storage elevation; or*

*(ii) has an impounding capacity for maximum storage elevation of 50 acre-feet or more.*

*This definition does not apply to any such barrier that:*

*(i) is a levee; or*

*(ii) is a barrier described above that-*

> *(I) is 6 feet or less in height regardless of storage capacity; or(II) has a storage capacity at the maximum water storage elevation that is 15 acre-feet or less regardless of height; unless the barrier, because of the location of the barrier or another physical characteristic of the barrier, is likely to pose a significant threat to human life or property if the barrier fails."*


## 2.3    NATIONAL DAM SAFETY PROGRAM

The NDSP, first authorized by Congress in 1996, is a partnership of the States, Federal agencies, and other stakeholders to encourage individual and community responsibility for dam safety. The Department of Homeland Security's (DHS) FEMA is responsible for coordinating the activities of the NDSP.

In 2012, Congress passed the Biggert-Waters Flood Insurance Reform Act (BW12) reauthorizing the National Flood Insurance Program through 2017. BW12 includes an authorization for the FEMA flood mapping program to be carried out in coordination with a Technical Mapping Advisory Committee. The authorization includes some new requirements related to mapping flood risks related to dams:

> *[I]dentify, review, update, maintain, and publish National Flood Insurance Program rate maps with respect to—*

> *(i) all populated areas and areas of possible population growth located within the 100-year floodplain; (ii) all populated areas and areas of possible population growth located within the 500-year floodplain; (iii) areas of residual risk, including areas that are protected by levees, dams, and other flood control structures; (iv) areas that could be inundated as a result of the failure of a levee, dam, or other flood control structure; and (v) the level of protection provided by flood control structures;*

This document was written prior to the enactment of BW12, but may support coordination with the Technical Mapping Advisory Council on how to implement the new mapping requirements related to dams.

The FEMA NDSP responsibilities include:

- Providing grant assistance to support improvement of dam safety programs in the United States

- Performing dam safety research to provide technical support to the individual States

- Conducting dam safety training for individual States

FEMA coordinates input to the NDSP from other parties through the NDSRB and ICODS. The 11-person NDSRB includes a representative from FEMA, who chairs the board; four Federal agencies, each of whom serves on the ICODS; five State dam safety program representatives and one member from the private sector. FEMA also chairs the ICODS, which includes representatives from a number of Federal agencies, and provides a permanent forum for coordinating Federal dam safety and security issues.

One of the key roles of FEMA and its partners is to provide technical assistance to the individual State dam safety programs. Under the NDSP, State dam safety programs are autonomous and develop State-specific regulations and guidance. There are numerous technical publications developed by Federal agencies that are incorporated by the States in their guidance documents on dam design, operation and maintenance, breach assessments, and the preparation of EAPs. Due to the autonomous nature of the State dam safety programs, not all States require EAPs and among those that do, the guidance used to prepare breach inundation mapping for EAPs is not consistent among the States. In 2008, the NDSRB voted to require participating States to achieve a 100 percent compliance with EAP development for high-hazard potential dams in their State. The NDSRB also voted to reserve a portion of FEMA grant funds received by the States for EAP compliance activities.

FEMA, with input from the NDSRB, determined that a new guidance publication related to dam breach inundation mapping for areas downstream of dams could lead to more consistent inundation mapping efforts and improved EAPs. In January 2008, the NDSRB formed a Work Group on Emergency Action Planning for Dams (EAP Workgroup) to evaluate, recommend, and monitor activities and initiatives to increase the number of EAPs for high- and significant-hazard potential dams consistent with the NDSRB goal for improving EAP compliance. The EAP Workgroup developed the concept of simplified inundation maps (SIMS) to quickly increase the number of EAPs. This document incorporates the EAP Workgroup recommendations.

## 2.4   DAM SAFETY

According to the USACE 2010 National Inventory of Dams (NID) dataset, there are more than 84,000 structures meeting the Federal definition of a dam (refer to Section 2.2) located in the United States. The NID was first authorized by Congress as part of the National Dam Inspection Act (PL 92-367) of 1972 and required that the USACE inventory dams in the United States. The USACE NID provides statistics for regulated dams in the United States.

### 2.4.1  EAP Statistics

Of the dams included in the NID dataset, approximately 17 percent and 15 percent are classified as high- and significant-hazard potential dams, respectively (USACE, 2009). Dams are classified according to FEMA Publication 333, *Hazard Potential Classification System for Dams* (2004a) (see also Section 5 of this report, Table 5-1) using the following definitions:

- *High-hazard potential* – Dams assigned the high-hazard potential classification are those where failure or misoperation will probably cause loss of human life.

- *Significant-hazard potential* – Dams assigned the significant-hazard potential classification are those dams where failure or misoperation are not likely to result in loss of human life but may cause economic loss, environmental damage, disruption of lifeline facilities, or impact other concerns. Significant-hazard potential classification dams are often located in predominantly rural or agricultural areas but may be located in areas with population and significant infrastructure.

- *Low-hazard potential* – Dams assigned the low-hazard potential classification are those where failure or misoperation are not likely to result in loss of human life and only low economic and/or environmental losses. Losses experienced are likely limited principally to the owner's property.

All dams regulated under the NDSP should have an EAP, but the highest priority should be given to developing EAPs for dams that could result in the loss of life in the event of a breach or other failure incident. An EAP provides a plan for dam owners, emergency management professionals, first responders, and community officials and citizens to constructively act during a potential dam failure incident to avoid a dam failure and to warn and evacuate the population at risk (see also Section 1.2 and 11. 1).

States are responsible for developing enabling legislation and associated requirements for EAPs for dams under their jurisdiction. Not all States require EAPs for potential dam breach incidents. According to ASDSO[1] less than 66 percent of the high-hazard potential dams and 23 percent of the significant-hazard potential dams in the United States have EAPs as of 2011. The percentages of high-, significant-, and low-hazard potential dams with existing EAPs are presented in Figure 2-1.

### 2.4.2  Dam Statistics for the United States

Over 95 percent of the dams listed in the NID are either privately owned, public utility owned, or locally owned and under the responsibility of the individual State for which they are located. The vast majority of the dams (over 88 percent) consist of an earthen embankment. Over 93 percent of the regulated dams have a dam height less than or equal to 50 feet and 50 percent of the regulated dams have a dam height less than or equal to 25 feet. The inventory of regulated dams is aging, with 70 percent of the dams older than 43 years. By 2029, over 85 percent of the dam inventory will be older than 50 years.

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[1] www.damsafety.org

**Percentage of Dams in the United States
by Hazard Potential**
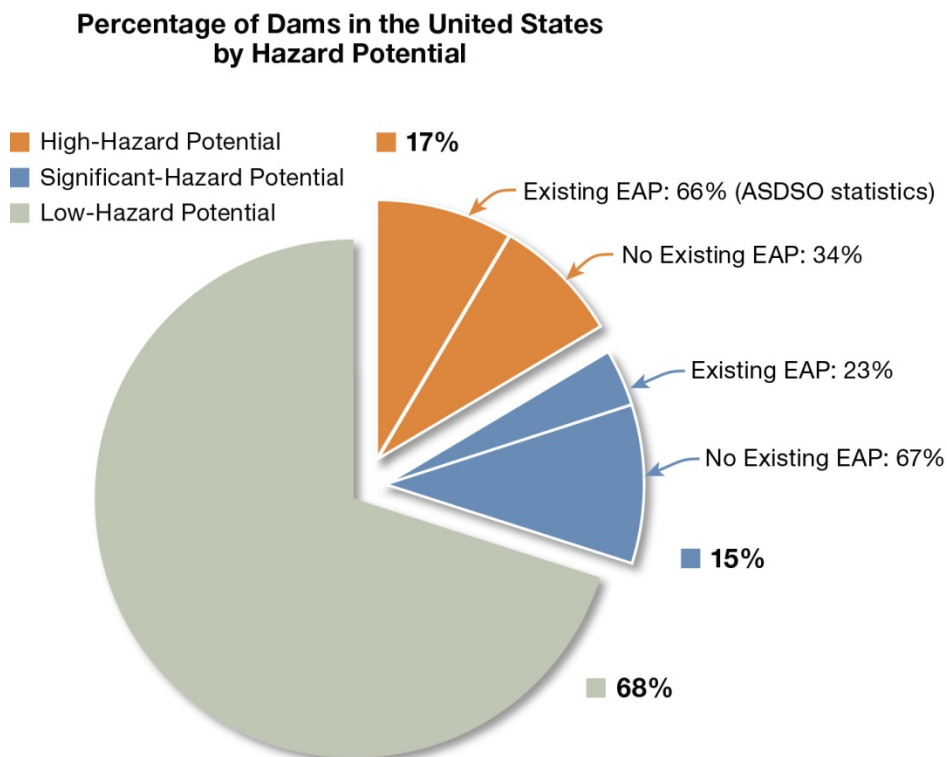


*Source: USACE, 2009*

**Figure 2-1: EAP statistics for U.S. dams**

With such a large number of aging earthen dams, it is critically important to develop breach inundation mapping studies associated with the preparation of EAPs for high- and significant-hazard potential dams where loss of life is probable due to a dam failure. In addition, it is important to identify the dam breach inundation zones downstream of low-hazard potential dams to inform the public of the unstudied and unmapped hazards of living downstream of dams and to avoid "hazard creep," where new development constructed downstream of dams increases a dam's hazard potential classification and results in the need for costly dam modifications.

### 2.4.3   Available Federal Agency Procedures and Guidelines

With few exceptions, State dam safety programs reference Federal agency procedures and guidelines for hydrologic and hydraulic modeling guidance and dam breach parameter selection. Federal agencies with relevant procedures and guidelines most referenced by the States include the NRCS, FEMA, USACE, USBR, National Weather Service (NWS), and FERC. Many of the Federal agency dam breach guidelines have not been updated in years, however, and contain outdated modeling guidance. Further, many describe guidelines specific only to the standard operation procedures for dam design or dam safety of the Federal agency that published the document.

## SECTION 3    TYPES OF DAMS

Dams may be classified by purpose, type, size, and hazard potential, the latter of which varies greatly between States and Federal agencies. This section describes the most common types of dams and the primary causes of dam failure. A discussion of the Federal system used to categorize the size and hazard potential of dams is discussed in Section 5.

There are numerous intended purposes for man-made dam structures, such as flood retarding, diversionary, irrigation and water supply, hydroelectric power generation, and recreational. Figure 3-1 shows the primary tapes of dams and purpose in the United States. Recreation, flood control, and fire protection are the three most common applications. Fire protection, as defined by the NID data dictionary, includes stock ponds and small farm ponds.
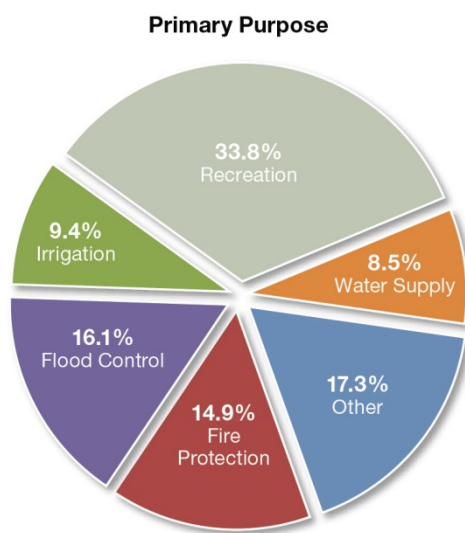
**Primary Purpose**



*Source: USACE NID, 2009*

**Figure 3-1: Percent of dams by primary purpose**

The NID classifies dams by the type of construction material used with the majority listed as either a concrete or embankment type dam. ***Concrete dams*** include arch, buttress, concrete, gravity, masonry, multi-arch, and roller-compacted concrete (RCC) and are typically constructed of concrete or other masonry components. ***Embankment dams*** are made of earthen materials and may be filled with rock, earth, or other materials resistant to erosion. In the United States, the number of earthen embankment dams far exceed any other type of dam, representing 88 percent of all dams (Figure 3-2) (USACE, 2009). Timber and stone dams, also described in this section, were constructed historically in the United States and represent a small percentage of dams.
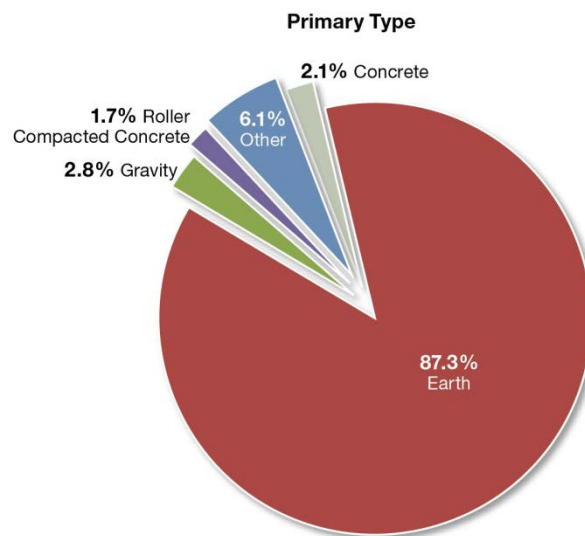
**Primary Type**



*Source: USACE NID, 2009*

**Figure 3-2: Percent of dams by primary type**

## 3.1 CONCRETE DAMS

There are several types of concrete dams ranging from conventional design styles such as gravity, arch, multi-arch, and buttress dams to newer design approaches such as RCC dams.

***Gravity dams*** typically consist of a solid concrete structure that maintains stability against design loads from the geometric shape, mass and strength of the concrete. Conventional placed mass concrete and RCC are the two general concrete construction methods for concrete gravity dams (USACE, 1995). In earlier periods of dam design, gravity dams were built of masonry materials.

Generally, gravity dams must be sized and shaped to resist overturning, sliding, and crushing at the toe. An example of a gravity dam is shown in Figure 3-3. Provided that the moment around the turning point caused by the water pressure is smaller than the moment caused by the weight of the dam, the dam will not overturn. This is the case if the resultant force of water pressure and weight falls within the base of the dam. Typically gravity dams are constructed on a straight axis, though they may be slightly angled or curved, in an arch shape. Gravity dams may be constructed of masonry materials such as stone, brick, or concrete blocks jointed with mortar; this type of construction was typical in early dam design. Additionally, gravity dams may have a hollow interior with concrete or masonry used on the outside.



*Photo retrieved from USACE Web site: (http://www.iwr.usace.army.mil/docs/MMDL/FLD/Feature.asp?ID=31)*

**Figure 3-3: An example of a gravity dam is Dworshak Dam in Clearwater Canyon, Idaho, North Fork of the Clearwater River**

Another form of concrete gravity dam is the buttress dam, which consists of a sloping upstream face that is supported by a series of concrete buttresses that bear on the foundation of the dam. Typically buttress dams require less concrete material than conventional gravity dams by reducing the uplift pressure associated with the dam (Canadian Dam Association, 2007a).

A gravity dam can be combined with an ***arch dam***, known as a gravity-arch dam, in areas with massive amounts of water flow but less material available for a purely gravity dam. A gravity-arch dam may be curved upstream so as to transmit the major part of the water load to the abutments (two-dimensional) or curved vertically and horizontally (three-dimensional). The most common location for a gravity-arch dam is in a deep canyon with steep side walls such as Hoover Dam in Nevada (Figure 3-4). Since a large portion of the hydrostatic pressure is transferred to the abutments, the strength and character of the rock should be carefully inspected. An example of a constant angle arch dam is shown in Figure 3-5.



**Figure 3-4: An example of a concrete gravity-arch dam is Hoover Dam in the Black Canyon of the Colorado River on the Nevada –Arizona border**

Arch dams are defined by the ratio of the width of the base (b) to the height (h). There are two recognized types of ***single-arch dams***: gravity and thin arch. A thin-arch dam is defined by the USBR to have a base-to-structural height ratio less than 0.2. Single-arch dams can be further segregated into two main groups, the constant-angle dam and the constant-radius dam. Constant-angle dams are the more common of the two and are constructed so that the angle subtended by the face of the dam is kept constant. In order to achieve this, the radius of the dam varies. Figure 3-5 shows an example of a constant-angle arch dam in LaPaz, Arizona. Constant-radius dams, on the other hand, are constructed so the radius of the face of the dam is kept constant at all elevations.



*Picture Retrieved from USBR Web site: (http://www.usbr.gov/projects/Facility.jsp?fac_Name=Parker+Dam)*

**Figure 3-5: An example of a constant-angle arch dam**

Multiple-arch dams are composed of two or more contiguous arches, typically with concrete supporting buttresses.

## 3.2  EMBANKMENT DAMS

Embankment dams are made from compacted earth. There are several types, as shown in Figure 3-6. The two most common types of embankment dams are rock-fill and earth-fill dams. ***Earth-fill dams*** are composed of suitable soils obtained from borrow areas or required excavation that

are spread and compacted in layers by mechanical means. Earth-fill dams may be constructed with homogenous layers (homogeneous dam) or zones of different materials of varying characteristics (zoned-earth dam). Earth-fill dams are typically trapezoidal in shape and rely on their weight to hold back the force of water, similarly to concrete gravity dams. Typical zones include a clay core and filter and drain zones.

A unique category of earth-fill embankment dams are tailings dams used by the mining industry. Tailings dams are often constructed of coarse tailings produced by the mine but may also consist of other soils obtained near the construction site. Tailings dams often rely on the stored tailings to control seepage, but otherwise include many of the same design features as conventional water storage dams.

***Rock-fill dams*** are constructed from compacted earth fill that contains a high percentage of rocks and other larger particles. The fill typically drains easily and therefore no drainage layer is required. To prevent seepage, rock-fill dams have an impervious zone on the upstream side of the dam or within the embankment. The impervious zone can be made from a variety of materials including masonry, concrete, plastic, steel pile sheets, timber, or clay. If clay is used, it is often separated from the fill by a filter to prevent erosion of the clay into the fill material.

Earth-fill dams may include a water-tight core can also be made from asphalt concrete. Dams with this type of core are called ***concrete-asphalt core embankment dams***. Most concrete-asphalt dams use rock and/or gravel as the main fill material. These types of dams are considered especially appropriate for areas susceptible to earthquakes due to the flexible nature of the asphalt core.

## 3.3    TIMBER AND STONE DAMS

Timber dams and stone dams were constructed in the past, but are not typically built in modern times. Timber and stone dams still exist and remain in operation.

***Timber dams*** are fast and easy to construct and were at one time a popular dam type. Timber dams, however, have many shortcomings. They have a short life span due to wood rot, they cannot be built to any great height, and they must be kept wet in order to preserve their water-retentive properties.

There are two common variations of the timber dam, the crib dam and the plank. ***Timber crib dams*** were constructed from long pieces of timber that were interlocked similar to a log home construction with space between the logs filled with earth or rubble. ***Timber plank dams***, on the other hand, were constructed from flat timber boards laid side-by-side to create a sloping wall (University of Vermont, 2004).

***Stone dams*** were far less common than timber dams and comprised rough stones stacked to form a wall that acted as the principal spillway (University of Vermont, 2004).
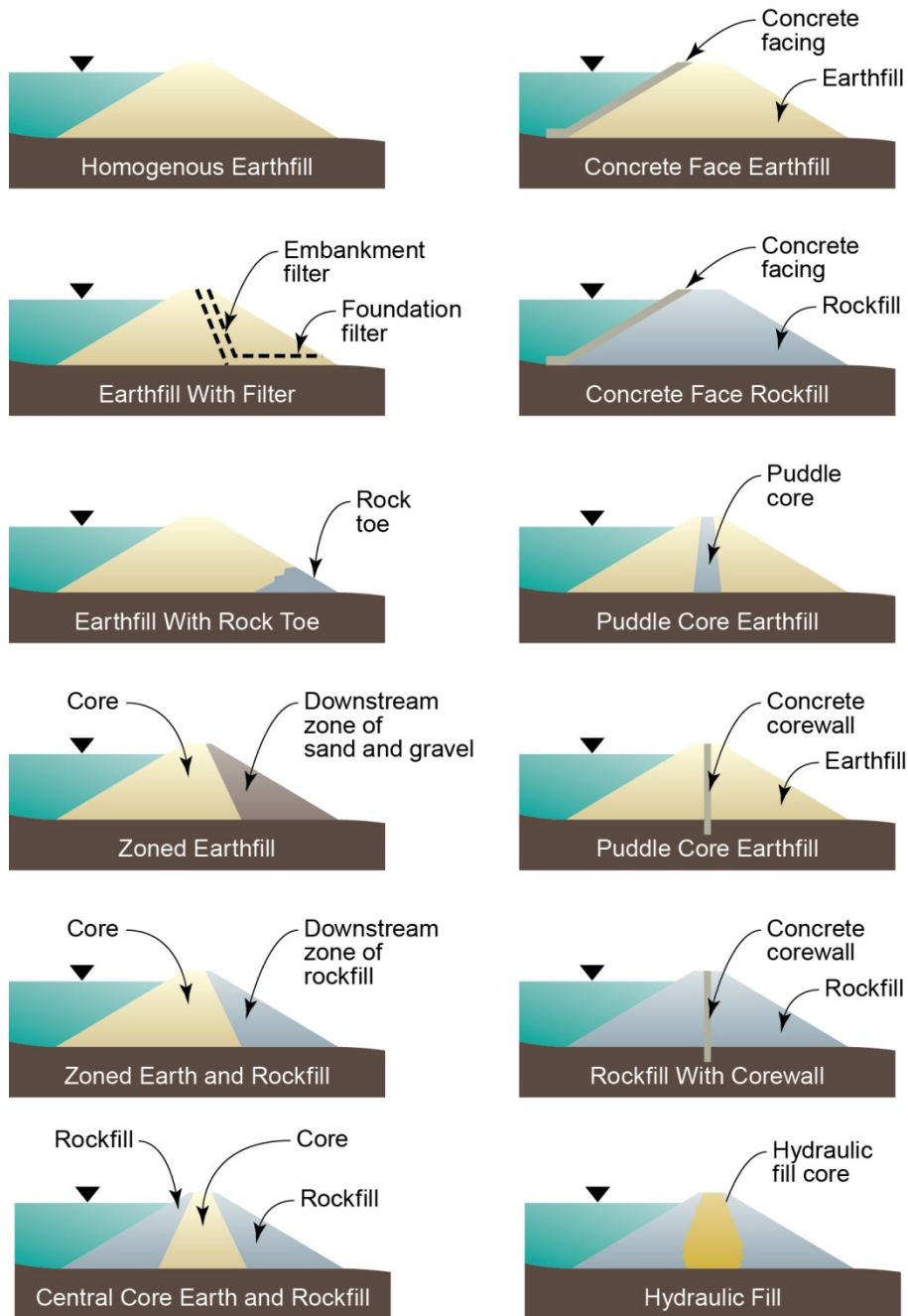
Source: Adapted from Foster et al., 1998

**Figure 3-6: Types of embankment dams**

## SECTION 4   CAUSES OF DAM FAILURES

Depending on the type of dam and site-specific conditions, a dam may be susceptible to failure from multiple causes. Additionally, the breach shape and timing of a dam failure varies depending on the type of dam. For instance, concrete gravity dams tend to have a partial breach, as one or more monolith sections formed during dam construction fail, whereas concrete arch dams tend to fail suddenly and completely (Canadian Dam Association, 2007b). In contrast, embankment dams do not usually have a complete or sudden failure, but rather tend to breach to the point where the reservoir is depleted or to where the breached materials resist erosion, such as at the dam foundation.

The most common causes of dam failure between January 1975 and January 2011are summarized in Table 4-1. Flood or overtopping was the most common cause of dam failure, followed by piping or seepage. Additional information on dam failures can be found on the ASDSO Web site at http://www.damsafety.org/news/?p=412f29c8-3fd8-4529-b5c9-8d47364c1f3e.

**Table 4-1: Causes of Dam Failure 1975-2011**

| Cause of Failure | Number of Dam Failures | Percentage of Dam Failure |
|---|---|---|
| Flood or Overtopping | 465 | 70.9% |
| Piping or Seepage | 94 | 14.3% |
| Structural | 12 | 1.8% |
| Human Related | 4 | 0.6% |
| Animal Activities | 7 | 1.1% |
| Spillway | 11 | 1.7% |
| Erosion/Slide/Instability | 13 | 2.0% |
| Unknown | 32 | 4.9% |
| Other | 18 | 2.7% |
| **Total number of dam failures** | **656** | |

*Source:  NPDM (2011)*

**Dam Failure Examples**

The following examples of dam failures illustrate that dams fail for a number of causes. These examples illustrate the need for dam breach inundation modeling and mapping to identify the flood risk and the importance of developing EAPs to mitigate the potential loss of life and other losses that can result from dam failure incidences.

*Flooding and overtopping failure:* The Kaloko Reservoir Dam in Kauai, HI, failed due to overtopping during an extreme rain event in March 2006. This earth dam was built in 1890 with a storage capacity of about 420 million gallons. The embankment had a maximum height of

about 40 feet. The dam crest was about 770 feet long and 15 feet wide. There were seven deaths reported due to this dam failure.

The southern embankment of the Lake Delhi Dam in Delhi, IA, failed on July 24, 2010, due to heavy raining and flooding. The dam failed after receiving about 10 inches of rainfall in 12 hours. Before the breach, river levels upstream of the dam had reached 24.22 feet, 10 feet above flood stage, breaking the May 2004 record of 21.66 feet.

*Piping and seepage failure*: In 1976, the failure of the Teton Dam in Idaho led to flooding in the cities of Sugar City and Rexburg (Figure 4-1). The dam failure killed 14 people and caused over $1 billion in property damage. Over 2,000,000 cubic feet per second of sediment-filled water emptied through the breach into the remaining 6 miles of the Teton River canyon, after which the flood spread out and shallowed on the Snake River Plain. Study of the dam's environment and structure placed blame for the collapse on cracks in the permeable soil (loess) used in the core and on cracks in the foundation bedrock that allowed water to seep under the dam. The combination of these flaws is believed to have allowed water to seep through the dam, which led to internal erosion, called piping, which eventually caused the dam's collapse.

*Source:*
*http://web.mst.edu/~rogersda/teton_dam/index.htm*

**Figure 4-1: Teton Dam failure, Rexburg, ID, June 1976**

*Structural failure:* The Kingston Plant coal waste dam failed in Harriman, TN, on December 22, 2008. This was a 40-acre pond used by the Tennessee Valley Authority to hold slurry generated by the coal-burning Kingston Steam Plant. The dam gave way just before 1 a.m., burying a road and railroad tracks leading to the plant. Although no one was seriously injured or hospitalized, 5.4 million cubic yards (> 1 billion gallons) of sludge damaged 12 homes and covered hundreds of acres.

*Spillway gate failure:* A spillway gate of Folsom Dam in California failed in 1995, increasing flows into the American River significantly. The spillway was repaired and the USBR carried out an investigation of the water flow patterns around the spillway using numerical modeling,

*Earthquake failure:* The Lower San Fernando Dam in California failed during an earthquake in 1971, causing the fill in the dam wall to liquefy which resulted in the collapse of the upstream part of the dam. A disastrous flood was only prevented because the reservoir level happened to be low at the time of the earthquake and no water escaped downstream.

*Poor design/construction failure:* In August 2008, the Redlands Ranch Dam located in Havasu, AZ, failed due to neglect and poor design and construction. No loss of life was reported, but 426 people were evacuated by helicopter and there was significant damage to the landscape.

*Misoperation*: On December 14, 2005, the Taum Sauk Upper Reservoir on top of Proffit Mountain near Lesterville Missouri failed due to human caused misoperation. The pump storage reservoir was constructed in 1963 as a 70- to 90-foot rockfill dam capped by a 10 foot parapet

wall, holding 1.5 billion gallons of water (4,600 acre feet) when full. The reservoir operations included maintaining the reservoir level 1 foot below the top of parapet wall, but did not include visual monitoring. The embankment breached when improperly installed and malfunctioning water level instrumentation failed to indicate the reservoir was full. The backup systems also failed to shut down the pumped inflows. There was no overflow spillway or overtopping protection to safely convey the flow to downstream areas. The reservoir overtopped, breaching the rockfill embankment in 5 to 6 minutes with an estimated peak discharge of 289,000 cfs to downstream areas. The released water severely damaged Johnson's Shut-Ins State Park (Rogers and Watkins, 2008).

**Failure Modes**

The many causes of dam failures are commonly summarized using five types of failures modes: hydrologic, geologic, structural, seismic, and human-influenced (refer to Table 4-2).

**Table 4-2: Typical Dam Failure Modes**

| Failure Mode | Examples of dam failures |
|---|---|
| **Hydrologic** | Overtopping due to: <br> • Inadequate spillway design <br> • Blocked spillway <br> • Loss of freeboard* due to embankment settlement or erosion <br> • Structural overstressing of dam components |
| | Surface erosion due to: <br> • High velocity water <br> • Wave action |
| **Geologic** | Piping and internal erosion caused by: <br> • Internal cracking, hydraulic fracture, or differential settlement <br> • Inadequate filters <br> • Outlet pipeline failure <br> • Pipes through the embankment formed by roots or animal/insect burrows |
| | Slope instability and hydraulic fracturing: <br> • Load exceeds sliding resistance at base or at joints of structure |
| **Structural** | Concrete dam: Failure of critical structural components <br> Embankment dam: Failure of the upstream or downstream face |
| **Seismic** | Earthquakes/ground movement; also liquefiable foundations or embankment materials |
| **Human influenced or caused** | Misoperation: <br> • Sudden rise in reservoir level causes flow through transverse cracks in embankment <br> • Incidents including gate failures, power interruption etc. |
| | Terrorist activities: <br> • Purposeful misoperation of the dam <br> • Impact of object that removes part of the dam crest |

*Freeboard = Vertical distance between a specified stillwater (or other) reservoir surface elevation and the top of the dam, without camber (FEMA, 2004a)

## 4.1    HYDROLOGIC FAILURE MODES

Hydrologic dam failures are induced by extreme rainfall or snowmelt events that can lead to natural floods of variable magnitude. The main causes of hydrologic dam failure include overtopping, structural overstressing, and surface erosion due to high velocity flow and wave action.

### 4.1.1   Overtopping

Overtopping occurs when the water surface elevation in the reservoir exceeds the height of the dam; water can then flow over the top crest of the dam, an abutment, or a low point in the reservoir rim (Figure 4-2). Overtopping usually results from a design inadequacy of the dam/spillway system and reservoir storage capacity to handle the resulting flooding event. A failure may also occur when a reservoir's outlet system is not functioning properly, thereby raising the water surface elevation of the dam.

Overtopping of a dam as a result of flooding is the most common failure mode for embankment dams. During a severe overtopping event, the foundation and abutments of concrete dams may also be eroded, leading to a loss of support and failure from sliding or overturning (FEMA, 2004a).

Dam failure begins when appreciable amounts of water begin flowing over or around the dam face and begin to erode the face of the dam. For embankment dams, the failure typically begins at a point on the top of the dam and expands in a generally trapezoidal shape. The water flow through the expanding breach acts as a weir; however, depending on conditions such as



*Source:* http://ga.water.usgs.gov/flood/flooding-sept09/images/miscellaneous/

**Figure 4-2: Water overtopping the Morgan Falls Dam, GA**

headwater and tailwater, various flow characteristics can be observed during a breach development including weir flow, converging flow, and channel flow.

### 4.1.2   Structural Overstressing of Dam Components

Higher loading conditions are typically found in dams where the reservoir elevation is increased due to a hydrologic event. While the dam itself may not be overtopped, the surcharge may be increased, overstressing the dam's structural components. This overstressing may then result in an overturning failure, sliding failure, or failure of specific components of the dam.

Embankment dams may be at risk when increased water surfaces result in increased pore pressures and seepage rates that exceed the design seepage control measures for the dam. Concrete dams may be at risk due to potential failure of specific components of the dam, such as overturning or slipping of a slab section (FEMA, 2004a).

### 4.1.3 Surface Erosion from High Velocity and Wave Action

Surface erosion can occur along earthen spillways, the upstream or downstream embankment slopes, or along other appurtenant structure inlet and outlet channels. Surface erosion is primarily caused by high velocity runoff, reservoir wave action, and ice action. High flow velocities may cause headcutting along spillway sides that can progress towards the spillway crest, eventually leading to a full dam beach (FEMA, 2004a).

The U.S. Department of Agriculture (USDA) Agricultural Research Service (ARS) and NRCS integrate the evaluation of erosion in earthen and vegetated auxiliary spillways of dams as part of their study through assessing the headcut erodibility in the Water Resource Site Analysis Computer Program (SITES) model and through ongoing research and development of the Windows Dam Analysis Modules (WinDAM) and SIMplified Breach Analysis (SIMBA) models. Refer to Section 8 of this document for further discussion.

## 4.2 GEOLOGIC FAILURE MODES

Geologic failure modes include piping and internal erosion as well as slope instability and hydraulic fracturing. For embankment dams, geologic failures are typically caused by long-term seepage of water stored in the reservoir; the water seeps through the dam or the foundation and abutments, weakening the embankment over time. If seepage is uncontrolled it may lead to internal erosion or piping of the embankment materials within the dam.

A geologic failure may also result from inadequate geotechnical design of the embankment and foundation, inadequate seepage controls, or increased load situations such as the rapid increase or drawdown of water level due to a hydrologic event, landslide, earthquake, or wave action.

### 4.2.1 Piping and Internal Erosion

**Piping:** Piping occurs when concentrated seepage develops within an embankment dam. The seepage slowly erodes the dam embankment or foundation leaving large voids in the soil. Typically, piping begins near the downstream toe of the dam and works its way toward the upper reservoir. As the voids become larger, erosion becomes more rapid. Once the erosion reaches the reservoir, it can enlarge and cause catastrophic dam failure.

Piping failures typically occur only in earthen dams. The failure begins when water, naturally seeping through the dam core, increases in velocity and quantity to begin eroding fine sediments out of the soil matrix. If enough material erodes, a direct piping connection may be established from the reservoir water to the dam face. Once such a piping connect is formed, it is almost impossible to stop the dam from failing. Piping failures begin at a point in the dam face and expand as a circular opening. When the circular opening reaches the top of the dam, it continues expanding as a trapezoidal shape. The water flow through the circular opening is modeled as orifice flow, but in the second stage (as trapezoidal in shape) is modeled as weir flow. Piping can also occur along conduits, outlet works, and abutments. The Teton Dam failure in 1976, shown in Figure 4-1, is a famous example of a piping failure.

**Internal Erosion:** Similar to piping, internal erosion is the occurrence of erosion where two adjacent zones interface within the embankment or at the contact between the embankment and foundation. Internal erosion is differentiated from piping in that internal erosion originates internally, whereas piping originates externally. When voids of the material into which seepage is flowing are larger than a critical size required to retain the particles, the particles of the up-gradient material can be transported into or through the adjacent material, thereby resulting in internal erosion (Canadian Dam Association, 2007c).

## 4.3    STRUCTURAL FAILURE MODES

Structural failures can occur when there is a failure of a critical dam component. Structural failures may be related to an inadequate initial design, poor construction, poor construction materials, inadequate maintenance and repair, or gradual degradation and weakening over time (FEMA, 2010). Additionally, structural failure may be inter-related with other modes of failure. For example, structural failure of the main embankment may be related to internal piping, or a critical dam component could fail due to overstressing during a flood event.

Structural failures of concrete dams can occur with the loss of the entire concrete dam structure or monolith sections. Structural failures of earth embankment dams can occur in the main embankment or appurtenant structures. Failure of an appurtenant structure such as the spillway or lake drain can lead to embankment failure. Typical warning signs include cracking and settlement.

## 4.4    SEISMIC FAILURE MODES

Earthquakes are another important cause of dam failures, especially in seismic zones of the United States. Seismic failures are generally related to either ground movement or liquefaction. Ground movements may cause a dam to shift, settle, or crack into an undesirable configuration that prevents the dam from performing as designed.

For embankment dams, two failure scenarios are typically considered in dam breach analyses: liquefaction and seismic-induced piping. Earthquakes can cause extreme stress on a dam, and liquefaction can occur when soils are loaded causing the soil to transfer from a solid to liquefied state. Soil liquefaction can cause a dam to fail almost instantaneously or it can cause slumping that exposes the dam crest to overtopping and subsequent erosion. Seismic-induced piping can occur due to internal cracking caused by ground motions of an earthquake (United States Department of the Interior [USDOI], 2011).

Failure mechanisms due to seismic activities include:

- Slope instability

- Permanent deformations

- Fissures or cracking

- Differential settling

- Rupture of principal spillway outlet pipeline

- Liquefaction (Australian National Committee on Large Dams Incorporated, 2000; Canadian Dam Association, 2007c).

Seismic failures are most likely to occur in seismic zones. The U.S. Geological Survey's (USGS) Earthquake Hazards Program provides seismic hazard maps, data, and engineering tools for areas throughout the United States at the following Web site: http://earthquake.usgs.gov/hazards/. Figure 4-3 presents the USGS 2008 National Seismic Hazard Map.
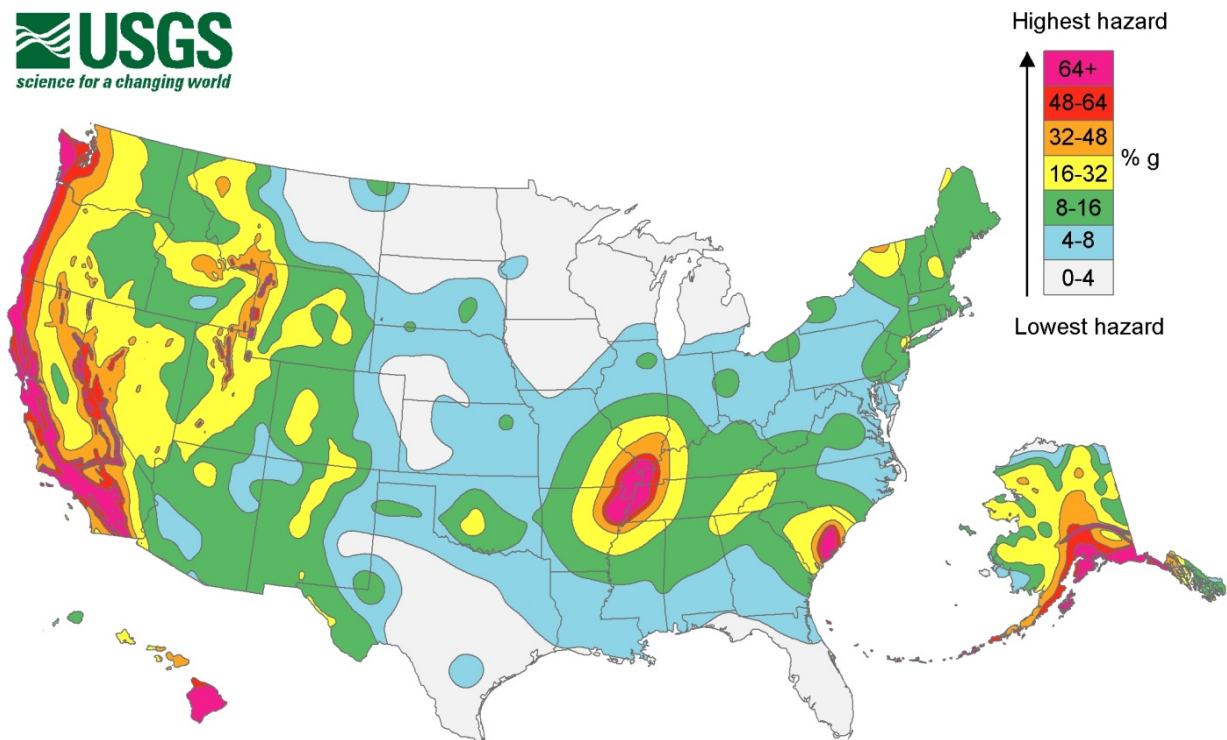


*Source: Peterson (2008)*

**Figure 4-3: USGS 2008 National Seismic Hazard Map**

## 4.5 HUMAN-INFLUENCED FAILURE MODES

Human-influenced dam failure incidents can be related to improper design or maintenance, misoperation including scheduled volume releases, or terrorist acts.

**Maintenance**: For more information regarding appropriate dam maintenance and operations, see FEMA 93, *Federal Guidelines for Dam Safety* (FEMA, 1979; reprinted in 2004).

**Misoperation**: Misoperation, as defined in FEMA 541, *Embankment Dam Failure Analysis* (FEMA, 2005), is "the sudden or 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. Misoperation 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…
[It] also includes the inability to operate a gate in an emergency, a condition that could lead to overtopping of the dam and potential breach."

**Scheduled volume releases:** The release of reservoir volume is a common practice for maintenance purposes, and to provide additional flood storage volume in a reservoir in anticipation of an extreme flooding event. The rapid release of reservoir volume in an upstream dam may result in dam overtopping at a downstream dam, resulting in dam failure. A rapid release of storage volume in a reservoir may also result in a rapid drawdown and a geologic failure. Improper releases of storage volume may result in a dam failure.

**Terrorist incidents:** Terrorist activities can range from purposeful misoperation of the dam to physical attacks on the structure itself. Two common scenarios are typically considered when analyzing human-influenced dam failure: rapid failure of spillway gates, and a lowering of the dam crest. For an embankment dam, the rapid lowering of the dam crest could subjugate the dam to overtopping and subsequent erosion (USDOI, 2011).

## SECTION 5   DAM CLASSIFICATION SYSTEMS

The hazard potential classification of a dam, along with its size (height and capacity) classification, is used by State agencies to regulate dam design and dam breach modeling. Common practice among Federal and State Dam Safety agencies is to classify a dam according to the potential consequences of a dam failure on areas located downstream of the dam.

FEMA guidance recommends a three-step rating system that defines low-, significant-, and high-hazard potential classifications depending on the potential for loss of life, economic loss, and environmental damage resulting from a hypothetical dam failure. In addition, guidance developed by the USACE incorporates size classification determined by the dam's height and storage volume.

Other hazard potential classification systems have been designated by individual State programs. In September 2010, ASDSO published a survey of the hazard potential classification systems used by each State in its report *State Dam Safety* (ASDSO, 2010). This document shows that most States have adopted a classification system similar to the FEMA three-step rating system. Some States have additional hazard potential categories such as "extreme hazard" and "very low hazard" and/or have added additional classifications to account for the size of the dam (height and capacity).

The States regulate the selection of the inflow design flood (IDF), also referred to as the spillway design flood (SDF), according to the assigned hazard potential rating and size classification. The assigned ratings establish the flood events (dam breach scenarios) used in dam breach modeling for design purposes and for use in EAPs.

At the time of this publication, there is no consistency among the States regarding which flood events (dam breach scenarios) should be modeled. Further, since State dam safety programs are autonomous and dam safety regulations are established by specific State law and may be difficult to change, specifying guidelines for the specific flood events to be used for dam breach modeling and mapping is highly unlikely to be consistently applied throughout the United States. Therefore, this document provides guidance for dam safety professionals to promote consistency in dam breach modeling and mapping.

## 5.1   FEDERAL HAZARD POTENTIAL CLASSIFICATION GUIDANCE

A number of Federal agencies provide guidance on a dam hazard potential classification. States generally follow the FEMA guidance for hazard potential classification, though a number of States have modified FEMA's hazard potential classification to comply with State-specific requirements. The following paragraphs describe hazard potential classification guidance published by Federal agencies to show its relevance to what is typically required to model and map dam breach inundation zones.

**Federal Emergency Management Agency:** FEMA provides guidance on dam hazard potential classification in their publication *Federal Guidelines for Dam Safety: Hazard Potential Classification Systems for Dams* (FEMA 333, 2004a), summarized in Table 5-1. In most situations, the investigation of the impacts of failure on downstream life and property is

sufficient to determine the appropriate hazard potential rating; however, there may be circumstances where further evaluation is appropriate. For example, the reservoir of a dam that would normally be considered to have a low-hazard potential based on insignificant flooding due to failure may be known to contain toxic sediments, such as may exist in a tailings pond. Therefore, a low-hazard potential rating may not be appropriate and instead a higher standard may be more appropriate to classify the hazard potential (FEMA 333). FEMA guidance recommends that the hazard potential rating be based on consideration of the effects of a failure or misoperation during both normal and flood flow conditions. FEMA further recommends that the hazard potential should be based on the worst-case probable scenario of failure or misoperation of the dam.

**Table 5-1: FEMA Hazard Potential Classification System**

| Hazard Potential | Loss of Human Life | Economic, Environmental, Lifeline Losses |
|---|---|---|
| Low | None expected | Low and generally limited to owner |
| Significant | None expected | Yes |
| High | Probable. One or more expected | Yes |

*Source: FEMA 333 (2004a)*

For sequential dam failure (failure of dams in a series), FEMA guidance recommends that the hazard potential classification of the upstream dam must be as high as or higher than any downstream dams that could fail as a result of the upstream dam's failure.

**Other Federal Agencies:** Other Federal agencies, specifically FERC, USBR, NRCS, and USACE, also use the high-, significant-, and low-hazard potential classifications. The basis for establishing the hazard potential classification differs, however, among these Federal agencies. The USACE system for hazard potential classification is provided in Table 5-2.

Technical Memorandum No. 11 (TM 11), *Downstream Hazard Classification Guidelines* (USBR,1988), prepared by the Assistant Commissioner – Engineering and Research (ACER), provides a similar classification system to FEMA's except that TM 11 uses the concept of "lives in jeopardy" instead of probable loss of life. Because of this difference, dams categorized according to TM 11 may be determined to have a significant-hazard potential for situations where they would not have been rated similarly using FEMA 333 criteria. The USDOI (of which USBR is a part) specifies in "Part 753: Dam Safety and Security Program" of their *Department Manual* (2004) that the department no longer uses the TM 11 guidelines, but uses instead the FEMA 333 criteria for hazard potential classification of dams.

The NRCS policy on dam classification is presented in "Part 520, Subpart C, Structure Site Analysis Program" of their *National Engineering Manual* (210-V-NEM) (NRCS,1999), which is essentially the same classification system as FEMA.

**Table 5-2: USACE Dam Hazard Potential Classification System for Civil Works Projects**

| Category[1] | Hazard Potential Classification | | |
|---|---|---|---|
| | Low | Significant | High |
| Direct Loss of Life[2] | None expected (due to rural location with no permanent structures for human habitation) | Uncertain (rural location with few residences and only transient or industrial development) | Certain (one or more extensive residential, commercial or industrial development) |
| Lifeline Losses[3] | No disruption of services – repairs are cosmetic or rapidly repairable damage | Disruption of essential facilities and access | Disruption of critical facilities and access |
| Property Losses[4] | Private agricultural lands, equipment and isolated buildings | Major public and private facilities | Extensive public and private facilities |
| Environmental Losses[5] | Minimal incremental damage | Major mitigation required | Extensive mitigation cost or impossible to mitigate |

[1] Categories are based upon project performance and do not apply to individual structures within a project.
[2] Potential for loss of life is based on inundation mapping of the area downstream of the project. Analyses of loss of life potential should take into account the extent of development and associated population at risk, time of flood wave travel, and warning time.
[3] Refer to indirect threats to life caused by the interruption of lifeline services due to project failure, or operation, i.e., direct loss of (or access to) critical medical facilities or loss of water or power supply, communications, power supply, etc.
[4] Refers to direct economic effect on the value of property damage to project facilities and downstream property. Also includes the indirect economic effect due to loss of project services, i.e., impact on navigation industry of the loss of a dam and navigation pool, or impact upon a community of the loss of water or power supply.
[5] Environmental impact downstream caused by the incremental flood wave produced by the project failure, beyond which would normally be expected for the magnitude flood event under a without project conditions.
*Source: USACE (1997)*

## 5.2    DAM SIZE CLASSIFICATION SYSTEM

A dam size classification system (Table 5-3) was developed in 1979 by USACE for implementing the National Dam Inspection Act (PL 92-367). In this system, size classification may be determined by either the dam's height or storage volume (whichever gives the larger size category). Several States have developed dam size as a component of their hazard potential classification based on the USACE guidance.

**Table 5-3: USACE Federal Dam Size Classification System**

| Category | Dam height and impoundment storage volume | |
|---|---|---|
| | Height (feet) | Storage (acre-feet) |
| Small | 25 to 40 | 50 to 1,000 |
| Intermediate | 40 to 100 | 1,000 to 50,000 |
| Large | More than 100 | More than 50,000 |

*Source: USACE (1979)*

## 5.3    PROBABLE LOSS OF LIFE

Probable loss of life is an important factor used in hazard potential classification systems and emergency action planning. Probable loss of life is often determined based on how many habitable structures and roads are located in the area that would be inundated as a result of a dam breach. Hazard potential classification systems do not typically take into account improbable (transient) loss of life, such as that of a passer-by or occasional, non-overnight recreational user of the downstream area (FEMA, 2004b).

DHS issued a document, *Dams Sector Estimating Loss of Life for Dam Failure Scenarios* (DHS, 2011)*,* that highlights the importance of consistent consequence estimation approaches to facilitate comparing results across the sector and to support developing and implementing sector-wide risk management strategies. The document discusses the strengths and limitations of several methods for estimating loss of life including DSO-99-06, *A Procedure for Estimating Loss of Life Caused by Dam Failure* (USBR, 1999); the Flood Comparison Method;[2] and LIFESim, a model developed by Utah State University.[3]

The BC Hydro Life Safety Model (LSM) is software initially developed by BC Hydro in Canada that previously had only been used to carry out dam break risk assessments for small Canadian communities (e.g., less than 3,000 people) with readily available data (FLOODsite, 2007). LSM is currently supported by HR Wallingford[©] of the United Kingdom.

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[2] The Flood Comparison Method is a risk prioritization tool developed by URS Corporation, Inc. under contract with FEMA to assist State dam safety offices in determining relative risk values for dams in their inventories. Additional information on this tool is presented in Appendix B of *Risk-Based Dam Safety Prioritization – A Method for Easily Estimating the Loss of Life from Dam Failure* (FEMA, 2008)

[3] Model development was sponsored by USACE and the Australian National Committee on Large Dams (ANCOLD), and supported by other partners including the USBR. LifeSim is a modular spatially-distributed dynamic simulation system for estimating potential life loss from natural and dam-failure floods. The model can be used for dam safety risk assessments and to explore options for improving the effectiveness of emergency planning and response by dam owners and local authorities (USACE and ANCOLD, 2005).

## SECTION 6   DAM BREACH ANALYSIS STUDY APPROACHES

The two primary dam breach study approaches used by State governments and Federal aagencies are an ***event-based approach*** and a ***risk-based approach***. The event-based approach has been traditionally the most widely used for dam breach analysis. The event-based approach is a deterministic method based on specific precipitation and non-precipitation events for the dam breach analysis and downstream inundation mapping. For the event-based approach, both a non-hydrologic "fair weather failure," also referred to as a "sunny day failure," and a specific hydrologic failure event, such as the Probable Maximum Flood (PMF), are usually established based on a dam's hazard potential classification.

In the past two decades, risk-based approaches to dam breach analysis have become more acceptable for dam safety and dam design purposes. A risk-based approach is commonly used for dam design purposes to establish the SDF or IDF for a dam. For a risk-based approach, the downstream consequences for a range of hydrologic dam failure events are evaluated.

Dam breach inundation studies are used for multiple purposes, including:

- Evaluating and establishing the hazard potential classification for a dam

- Estimating the potential for loss of life

- Evaluating dam safety risk and prioritizing dams within a dam safety portfolio

- Selecting the appropriate SDF or IDF for dam and spillway design

- Developing EAPs and exercise planning associated with dam safety permitting

- Developing breach inundation zone mapping for flood warning systems and flood evacuation planning

- Developing breach inundation zone mapping for dam breach consequence studies and for flood mitigation planning

- Developing dam breach inundation zone mapping for risk communication to inform the public of the risk living downstream of dams.

## 6.1   EVENT-BASED APPROACH

An event-based approach is a deterministic method that requires the use of a specific or series of specific precipitation and non-precipitation events for the evaluation of dam failure and downstream inundation mapping. These events include extreme rainfall and runoff events that can lead to natural floods of variable magnitude. The maximum flood for which a dam is to be designed or evaluated is often dependent on its existing hazard potential classification or size classification (refer to Section 5).

Typically, several hydrologic and non-hydrologic (fair weather) events are evaluated as part of an event-based dam safety analysis. For hydrologic failure events, an extreme flood event ranging from the 50-year event for low-hazard dams up to the PMF for high-hazard dams is selected based on the potential for loss of life due to a dam failure or for significant economic

and environmental losses. Typically, the hazard potential classification of the dam is used to select the extreme hydrologic failure event. The PMF is the flood that may be expected from the most severe combination of critical meteorological and hydrologic conditions that are reasonably possible in the drainage basin under study. The Probable Maximum Precipitation (PMP) is an estimate of the maximum possible precipitation depth over a given size catchment for a given length of time (Stedinger et al., 1996).

The greatest advantage to using an event-based approach is that it is a direct approach, is less complicated to perform and regulate, and produces more conservative breach inundation zone mapping when compared to a risk-based approach. High-hazard potential dams are typically evaluated using a full PMF, and significant- or low-hazard potential dams are evaluated on a percentage of a PMF or some more frequent storm event.

### 6.1.1   Fair Weather (Non-Hydrologic) Failure

A fair weather (Sunny Day) breach is a dam failure that occurs during fair weather (i.e., non-hydrologic or non-precipitation) conditions. A fair weather breach is analyzed by establishing an initial reservoir water level and commencing a breach analysis without additional inflow from a storm event. A fair weather breach is typically used to model piping failures for hydrologic, geologic, structural, seismic, and human-influenced failure modes.

Base flow conditions for a fair weather failure are typically ignored because of the small discharge and volume compared to that of a dam breach. As a general guidance, base flow can be ignored if the dam breach flow is two times greater than the base flow. Where base flow is considered, the discharge is typically estimated based on reported base flows through the dam's outlet works or from stream gage records. The three most common initial water level elevations for fair weather breach analyses are as follows:

- **Normal Pool Elevation (invert of the highest elevation of the primary outlet)**

    A breach at the normal pool elevation of the reservoir is used to estimate the volume and associated breach discharge that would result from a failure event during fair weather conditions. For an embankment dam, this type of event is modeled as piping/internal erosion failure, whereas for a concrete dam, this event is modeled as a monolith collapse resulting from sliding, foundation instabilities, or a seismic event.

- **Invert of Auxiliary Spillway (lowest uncontrolled spillway)**

    A breach of the dam with the reservoir water level set at the auxiliary spillway (also referred to as an emergency spillway) is common practice to simulate a breach during misoperation of the primary outlet works. Initiation of dam failure is typically the same as for the reservoir level at normal pool.

- **Top of Dam / Maximum High Pool**

  The reservoir level set to the top of the dam to represent the maximum amount of volume that may be stored in the reservoir. This condition may be selected to evaluate the most conservative non-hydrologic event. In practice, dams without adequate spillways or pump storage facilities, where the water level during non-hydrologic events is maintained at the top of dam, are unique situations subject to this conservative assumption. A breach event when the water level is at the top of dam may be modeled as a piping / internal erosion failure or as an overtopping failure with the water level just above the top of dam invert.

Various Federal agency publications provide guidance for establishing the initial water surface elevation of a reservoir during a fair weather failure event. Each of these specified elevations is used to characterize different failure modes as well as the potential volume of the reservoir at the time of failure.

Table 6-1 provides the recommended water surface elevation of a reservoir for used in dam breach modelingbased on published documents from Federal agencies and dam safety resource groups. The normal pool elevation is recommended as the default volume for the fair weather failure. States should consider a larger storage volume for dams where the primary and emergency spillway systems are considered susceptible to blockage resulting in a higher water surface elevation and volume during a non-hydrologic event.

**Table 6-1: Range of Initial Reservoir Pool Levels for a Fair Weather (Non-Hydrologic) Analysis**

| Initial Reservoir WSEL | Referenced Name in Publication | Initial Inflow to Reservoir | Failure Mode | Supporting Federal Organization | Supporting Documentation |
|---|---|---|---|---|---|
| Normal pool | Normal full reservoir[1] | Normal stream flow | None specified | Federal Emergency Management Agency (FEMA) | *Federal Guidelines for Dam Safety: Selecting and Accommodating Inflow Design Floods for Dams.* pp. 17. 2004b. |
| | Normal full reservoir | Normal stream flow | None specified | Federal Energy Regulatory Commission (FERC) | *Engineering Guidelines for the Evaluation of Hydropower Plants.* Ch 2. pp. 2-7. 1993. |
| | Normal pool elevation | Normal stream flow | Piping | Mine Safety and Health Administration (MSHA) | *Engineering and Design Manual: Coal Refuse Disposal Facilities.* 2009. |
| | Normal pool elevation[2] | Normal stream flow | Piping | National Dam Safety Review Board (NDSRB) | *Simplified Inundation Maps for Emergency Action Plans.* 2009. |
| | Top of active conservation | Normal stream flow | Piping | U.S. Department of the Interior (USDOI) | *Reduce Dam Safety Risk Modernization Blueprint / Implementation Phase 1: Launch Risk Reduction / Inundation Mapping / Modeling Subproject Report.* 2011. |
| Invert of auxiliary spillway | Seismic | Normal stream flow | Piping/ internal erosion | Natural Resources Conservation Service (NRCS) | National Engineering Manual: 210-V. 1982. |
| | Top of joint use[3] (auxiliary spillway) | Normal stream flow | Piping | USDOI | *Reduce Dam Safety Risk Modernization Blueprint / Implementation Phase 1: Launch Risk Reduction / Inundation Mapping / Modeling Subproject Report.* 2011. |
| Between normal pool and top of dam | Hydrologically induced static failure | Hydrologic event | Below top of dam (piping); above top of dam (overflow breach) | USDOI | *Reduce Dam Safety Risk Modernization Blueprint / Implementation Phase 1: Launch Risk Reduction / Inundation Mapping / Modeling Subproject Report.* 2011. |
| Top of dam | Static | Normal stream flow | Not specified | NRCS | *National Engineering Manual: 210-V.* 1982. |

**Table 6-1: Range of Initial Reservoir Pool Levels for a Fair Weather (Non-Hydrologic) Analysis**

| Initial Reservoir WSEL | Referenced Name in Publication | Initial Inflow to Reservoir | Failure Mode | Supporting Federal Organization | Supporting Documentation |
|---|---|---|---|---|---|
| | | | | | |
| Other | Seismic | Not specified | Catastrophic failure or overtopping (caused by liquefaction) and seismic-induced piping | USDOI | *Reduce Dam Safety Risk Modernization Blueprint / Implementation Phase 1: Launch Risk Reduction / Inundation Mapping / Modeling Subproject Report.* 2011. |
| | Normal Low Pool: 90% exceedance duration pool elevation | Constant inflow as required to produce the 90% exceedance duration pool | Piping | U.S. Army Corps of Engineers (USACE) | *Modeling, mapping, and Consequences Production Center Standard Operating Procedures (Final-Draft).* Unpublished draft, 2011. |
| | Normal High Pool: 10% exceedance duration pool elevation | Constant inflow as required to produce the 10% exceedance duration pool | Piping | USACE | *Modeling, mapping, and Consequences Production Center Standard Operating Procedures (Final-Draft).* 2011. |

WSEL = water surface elevation

[1] Normal reservoir level: "For a reservoir with a fixed overflow sill the lowest crest level of that sill. For a reservoir whose outflow is controlled wholly or partly by moveable gates, siphons or other means, it is the maximum level to which water may rise under normal operating conditions, exclusive of any provision for flood surcharge" (FEMA, 2004a).

[2] For small and intermediate-sized dams, it **may** be appropriate to use a single fair weather failure with the initial elevation set to the top of the dam instead of the rainy and fair weather situations. This "eliminates the need for expensive watershed and spillway studies and provides a reasonable upper limit estimate for warning and evacuation" (NDSRB, 2009).

[3] Joint use is a designation for dams with gated spillways. In these cases, the top of joint use is not the invert of the spillway but rather some elevation that places water up on the gates.

## 6.1.2 Hydrologic Failure

Hydrologic breaches that occur with extreme precipitation and runoff are termed "rainy day" or hydrologic failures. Hydrologic failures that cause dam breach events are generally analyzed based on the IDF established by the dam's hazard potential and hazard size classification, typically a PMF for high-hazard potential dams. For significant-hazard potential dams, the breach event may include a breach of the PMF and IDF that, according to State regulations,

could range from the 1-percent-annual-chance flood event (often called the 100-year flood) to a percentage of the PMF.

Many State and Federal agencies also allow the use of a risk-based approach to establish the SDF or IDF for dam design purposes. Many States require that the SDF or IDF event be one of the dam breach methods used for hazard potential classification and EAP mapping; for those cases, risk-based dam breach inundation zone mapping may be the dam breach inundation zone shown on an EAP map.

## 6.2 RISK-BASED (CONSEQUENCES-BASED) APPROACH

A risk-based approach to dam design and dam safety evaluations has been developed to account for the downstream consequences of a potential dam failure. The consequences evaluation is not based on the probability of failure, but instead on the potential loss of life or increase in economic losses caused by a potential dam failure.

A benefit of the risk-based approach is that it may demonstrate, via an incremental damage assessment, that areas located downstream of a dam may be marginally affected by the reduction in the SDF or IDF design standard for a dam. By lowering the SDF or IDF requirements, limited funding for needed rehabilitation measures can be used for more dams, resulting in an overall increase in dam safety.

A disadvantage of the risk-based approach is that by reducing the SDF or IDF to less than the full PMF based on downstream consequences, new development in the downstream breach inundation zone could alter the consequences, resulting in the need for future dam rehabilitation measures to increase spillway capacity. Effective risk communication as a component of the local development approval process can assist in reducing the occurrence of "hazard creep," an occurrence where new downstream development in a dam breach inundation zone increases the dam's hazard potential classification or SDF/IDF design requirement.

Several Federal agencies, including the USACE, USBR, FERC, and FEMA have developed risk-based assessment procedures for dam failure analysis and inundation mapping as described in the following sections.

### 6.2.1 Inflow Design Flood and the Incremental Hazard Evaluation

FEMA 94, *Federal Guidelines for Dam Safety: Selecting and Accommodating Inflow Design Floods for Dams* (2004b), and the FERC document, *Engineering Guidelines for the Evaluation of Hydropower Projects* (1993), laid the foundation for risk-based dam safety analysis. In FEMA 94, IDF is defined as "the flood flow above which the incremental increase in water surface elevation downstream due to failure of a dam or other water retaining structure is no longer considered to present an unacceptable additional downstream threat." Therefore, incremental hazard evaluation and the establishment of the IDF is, in essence, a risk-based approach.

The selection of the IDF is based on the evaluation of the magnitude of several flood events The incremental hazard evaluation begins with simulation of a dam failure during a hydrologic flooding condition, typically beginning with the PMF or percentage of the PMF as specified by

the State hazard potential classification requirements.. The same hydrologic event is then run for non-failure conditions. The water surface elevations for both the breach and non-breach events are compared to determine the increase in the water surface elevation resulting from the dam breach. If the incremental increase in downstream water surface elevation between the failure and non-failure scenarios results in an acceptable increase in consequences, ( as defined by State requirements) a smaller percentage of the PMF flood inflow or other magnitude flood is then used to repeat the process. The process is repeated until the incremental increase in consequences due to failure falls within acceptable requirements specified by the State.

Both the FEMA and FERC publications identify "acceptable consequences" of failure to be when the incremental effects (depth) of failure on downstream structures are approximately 2 feet or less; various other sources consider "acceptable consequences" to be 1 foot or less. FERC guidelines state that engineering judgment and sensitivity analyses are needed to make final decisions on the acceptability of consequences.

Once the appropriate IDF for the dam has been selected, the IDF is then routed through the dam to determine whether the flood can be safely passed without failure. Should the IDF pass safely, then no further evaluation or action is required; however, if the IDF cannot pass safely, then measures must be taken to enable the project to safely accommodate all floods up to the IDF to alleviate the incremental increase in unacceptable additional consequences a failure may have on areas downstream.

FEMA is updating the Federal guidelines for selecting IDFs for dams. The revisions will present a method for determining the IDF based on a downstream consequence assessment rather than an incremental increase of the flood flows downstream.

New FEMA guidance included in the publication *Selecting and Accommodating Inflow Design Floods for Dams* (FEMA, 2012) includes the recommendations shown in Table 6-2.

**Table 6-2: Recommended IDF Requirements for Dams Using Prescriptive Approach**

| Hazard Potential Classification | Definition of Hazard Potential Classification | Inflow Design Flood |
|---|---|---|
| High | Probable loss of life due to dam failure or misoperation | PMF[1] |
| Significant | No probable loss of human life but can cause economic loss, environmental damage, or disruption of lifeline facilities due to dam failure or misoperation | 0.1-percent-annual-chance exceedance flood (1,000-year flood)[2] |
| Low | No probable loss of human life and low economic and/or environmental losses due to dam failure or misoperation | 1-percent-annual-chance exceedance flood (100-year flood) |

PMF = Probable maximum flood

[1] Incremental consequence analysis, risk-informed decision making, or site-specific PMP studies may be used to evaluate the potential for selecting an IDF lower than the prescribed minimum. An IDF less than the 0.2-percent-annual-chance exceedance flood (500-year flood) is not recommended.

[2] Incremental consequence analysis or risk-informed decision making studies may be used to evaluate the potential for selecting an IDF lower than the prescribed minimum. An IDF less than the 1-percent-annual-chance exceedance flood (100-year flood) is not recommended.

Source: FEMA, 2012

## 6.2.2   Loss of Life / Population at Risk

It is important that consistent approaches for consequence estimation be adopted across the dam-safety sector. FEMA's *Estimating Loss of Life for Dam Failure Scenarios* (FEMA, 2011) discusses the strengths and limitations of several methods for estimating loss of life (refer to Section 5.3). This section further describes the procedure described in USBR's publication *A Procedure for Estimating Loss of Life Caused by Dam Failure* ([DSO-99-06], 1999) and companion document *Guidelines to Decision Analysis* (1986), as it is the most currently and widely used procedure for estimating loss of life resulting from dam failure.

Probable loss of life is an important factor used in hazard potential classification systems and emergency action planning. DSO-99-06 (USBR, 1999) presents a risk-based method to estimate the number of fatalities that would result from dam failure. This method was developed using data from about 40 floods, many of which were caused by dam failure. These publications outline the following seven steps to complete an analysis for loss of life:

- Step 1: Determine dam failure scenarios to evaluate

- Step 2: Determine time categories for which loss of life estimates are needed

- Step 3: Determine when dam failure warnings would be initiated

- Step 4: Determine area flooded for each dam failure scenario

- Step 5: Estimate the number of people at risk for each failure scenario and time category

- Step 6: Apply empirically based equations or methods for estimating fatalities

- Step 7: Evaluate uncertainty

The number of fatalities resulting from dam failure is most influenced by three factors: 1) the number of people occupying the dam failure floodplain, 2) the amount of warning provided to the people exposed to dangerous flooding, and 3) the severity of the flooding. Without exception, dam failures that have caused high fatality rates were those in which residences were destroyed and timely dam failure warnings were not issued. Estimating when dam failure warnings would be initiated is probably the most important part of estimating the loss of life that would result from dam failure.

For each failure scenario and time category, the population at risk must be calculated. Population at risk is defined as the number of people occupying the dam failure floodplain prior to the issuance of any warning. The method developed for estimating loss of life provides recommended fatality rates based on the flood severity, amount of warning time, and a measure of whether people understand the severity of the flooding. Recommended fatality rates for estimating loss of life may be determined based on a set of criteria that includes 15 different combinations of flood severity, warning times, and flood severity understandings. This method is also used by the NRCS to determine loss of life estimates to assign hazard potential classifications and design/evaluation requirements.

## 6.3    TIERED DAM BREACH ANALYSIS

A tiered approach to dam breach analyses can be used to establish an initial dam hazard potential classification and to produce dam breach inundation zone mapping for EAPs. The tiered dam breach analysis structure is not appropriate for use in dam design.

A tiered study approach was developed by the USDOI and is presented in their report titled *Reduce Dam Safety Risk Modernization Blueprint / Implementation Phase 1:  Launch Risk Reduction / Inundation Mapping / Modeling Subproject Report* (USDOI, 2011). The tiered dam breach analysis approach presented in this document adapts the USDOI approach and provides additional detail.

The NDRSB EAP Workgroup (2009) noted that the cost of detailed dam breach studies is consistently cited as the primary impediment to EAP development and, therefore, many States have adopted a form of simplified and conservative inundation maps for use in EAPs. The NDRSB EAP Workgroup also stated that although detailed studies often provide a more precise representation of potential flooding for a given set of assumptions, a more accurate representation of dam failure flooding is not necessarily provided.

In their effort to increase the number of EAPs for dams, a tiered approach in dam inundation modeling has gained popularity with many State and Federal dam safety programs. Unlike the event- and risk-based approaches discussed in Sections 6.1 and 6.2, the tiered approach is not used to determine the appropriate flood event to use in a dam failure analysis. Instead, the tiered approach is used to determine the appropriate level of complexity in the assessment, modeling, and mapping of a dam failure based on a dam's hazard potential, size, and the complexity of the downstream area under investigation.

The level of analysis for the tiered approach should correlate the sophistication and accuracy of the analyses with the scale and complexity of the dam and downstream area under investigation. Therefore, analysis of high-hazard potential dams located upstream of populated areas or complex floodplains should use more sophisticated modeling and additional sensitivity studies to properly assess the consequences of a dam failure; whereas, analysis of low-hazard potential dams situated upstream of sparsely populated areas may rely on more approximate methods of analyses.

In general, as the sophistication of the modeling increases, so does the level of effort, time, and cost necessary to conduct the analysis. Table 6-3 provides guidance to determine the tier level for analysis for dam failure inundation modeling and mapping. This table is arranged similarly to some State-developed tiered analysis structures, providing a logical combination of methods to perform an analysis. The dam failure analysis should be continued downstream to a point where the breach flood no longer poses a risk to life and property damage, such as the confluence with a large river or reservoir with the capacity to store the flood waters.

**Table 6-3: Tiered Approach Dam Breach Inundation Mapping for use in EAPs**

| Tier Level | Applicable to | Breach Parameter Prediction | Peak Breach Discharge Prediction | Downstream Routing of Breach Hydrograph |
|---|---|---|---|---|
| **Tier 1 – Basic level Screening and Simple Analysis** | • Low-hazard potential / small size <br> • First level screening for significant- or high-hazard dams | Empirical Equations | Simplified Models (SMPDBK, GeoDam-BREACH, or Technical Release [TR]-66) or HEC-HMS | GeoDam-BREACH, SMPDBK, DSAT,1D HEC-RAS Steady State, or HEC-HMS Hydrologic Routing |
| **Tier 2 – Intermediate** | • Significant-hazard potential / intermediate size <br> • High-hazard dams with limited population at risk | Empirical Equations | HEC-HMS or HEC-RAS Unsteady Model | HEC-RAS (Steady or Unsteady Modeling) 1-D or 2-D models |
| **Tier 3 – Advanced** | • High-hazard potential / large size dams with sufficient population at risk to justify advanced analyses | Empirical Equations, NWS BREACH, or WinDAM | HEC-RAS Unsteady Model | HEC-RAS Unsteady Model or 2-D models |

Tier 1 and 2 analyses are most appropriate for low-hazard potential / small sized and significant-hazard potential / intermediate-sized dams with a limited number of structures. More detailed surveying or modeling may be warranted for Tier 3 analyses for high-hazard potential / large-sized dams, those with a large population in the evacuation area, or those with significant downstream hydraulic complexities, such as major diversion structures, split flows, or potential for a series of dam failures such as a "domino effect" (NDSRB, 2009).

Additional guidance on the methods and models listed in Table 6-3 are provided in Sections 9 and 10 of this document.

## SECTION 7   HYDROLOGIC ANALYSES

This section provides an overview of key elements for preparing hydrologic analyses for dam breach inundation studies. The hydrology for dam breach inundation studies involves determining the peak discharge and volume for extreme hydrological and meteorological events. The flood event to be evaluated in a dam breach analysis may be a recurrence-interval-based event (i.e., 25-, 50-, 100-, 500-, 1,000-year event) or a ratio of the PMF, depending on specific State dam safety requirements.

It is not the intent of this document to require that updated hydrologic studies be performed for dam breach inundation mapping studies. Existing hydrologic studies should continue to be used as the best available technical data and the basis for breach inundation mapping unless significant changes in the dam design or watershed conditions justify a recalculation of the watershed hydrology. Typically, changes that warrant a revision of the hydrology for the dam include a change in the dam design requiring the resizing of the principal and emergency spillways or a significant change in the watershed runoff conditions.

Where existing hydrology cannot be used and new hydrology is justified, the following key factors that should be considered in the preparation of hydrologic analyses supporting dam breach analyses.

## 7.1    EXISTING AND FUTURE WATERSHED CONDITIONS LAND USE

Changing watershed characteristics, such as an increase in basin perviousness, loss of vegetative cover, loss of basin storage, and a reduction in the time floodwaters travel through the stream network system, may increase the peak discharge and runoff volume. In addition, the influence of climate change may result in an increase in the occurrence of extreme events and the magnitude of their severity. For dam design purposes both existing and a future land use conditions should be considered to establish the hydrology. However, for the preparation of dam breach inundation mapping for EAPs, the use of existing watershed conditions hydrology is standard practice.

Consideration of future conditions in the hydrology analysis for dam breach inundation zone mapping is recommended for dam design purposes and for flood risk communication purposes to inform the public of the potential existing and future risk of flooding downstream of dams. Dam breach inundation zone mapping of future watershed conditions may be used by local governments in land planning decisions to avoid new development that can increase the hazard potential classification of a dam ("hazard creep") leading to potential costly rehabilitation measures of the dam.

## 7.2    BASE FLOW CONSIDERATIONS

Base flow, which consists of the flow in the stream system that is coincident with the extreme event, should be considered in the dam breach inundation study. Base flow consists of natural flow caused by antecedent rainfall over the watershed, snowmelt runoff, and human-influenced releases of flow from upstream structures (dams and flow diversions).

Base flow should be evaluated based on historical records using stream gage records and/or historical hydrographs available from the USGS, States, or the NWS River Forecast Centers. The following are potential sources of information:

- Real-time daily stream flow data may be obtained from the USGS:
  http://waterdata.usgs.gov/nwis/rt

- The NWS's Advanced Hydrologic Prediction Service also provides real-time stream information:
  http://water.weather.gov/ahps/

- Historical stream flow data for continuous stage, discharge, and other instantaneous time-series data is available through the USGS Instantaneous Data Archive:
  http://ida.water.usgs.gov/ida/index.cfm

Snowmelt runoff is influenced by available snowpack, temperature, and the inability of runoff to infiltrate frozen ground. Rainfall on snowpack should be considered in hydrology for areas of the United States where extreme rainfall on snowpack can occur.

For fair weather non-hydrologic dam breach inundation studies, base flow is not as significant as it may be for hydrologic failure situations. As a general guidance, base flow can be ignored if the dam breach flow is two times greater than the base flow. If base flow is to be used for a non-hydrologic event, setting the base flow for non-hydrologic fair weather failures in the range starting at the mean peak stream flow discharge up to the bank full discharge is recommended. For hydrologic dam breach studies, the selection of the appropriate base flow may be significant and should be established specific to the dam and watershed location.

## 7.3    EVENT AND BREACH SIMULATION DURATION

For hydrologic evaluations involving a precipitation-runoff method, the selection of the event or storm duration is important to the determination of the extreme event peak discharge and volume. The selection of the event duration is largely influenced by the response rate of the upstream watershed to precipitation. The response rate of a watershed is influenced by many variables which vary by region, most notably the size of the watershed and the basin slope.

The event duration is typically established in intervals of 6 hours, including the 6-, 12-, 24-, 48-, and 72-hour events, with longer durations for extremely large watersheds. For specific areas of the arid or semi-arid United States, event durations less than 6 hours are justified based on historic events.

Various storm durations should be simulated to determine the critical storm duration that produces the greatest discharge. If the simulation of various storm durations is not performed, the use of a minimum storm duration of 24 hours is recommended unless historical data justifies a shorter duration, based on geographic location, or a longer duration for a large watershed where 24 hours does not represent the full contribution of watershed runoff.

When performing a dam breach analysis, the model simulation time often must be significantly longer than the precipitation event duration to ensure that the peak discharge and elevation are

adequately captured in the model without prematurely truncating the hydrograph. Longer simulation durations may be needed to model when flood waters have completely receded to determine the total duration of flooding.

## 7.4    PRECIPITATION

Precipitation considerations include the PMP and frequency-based precipitation.

### 7.4.1   Probable Maximum Precipitation

The PMP theoretically represents the largest depth of precipitation for a given duration that is physically possible over a particular drainage area at a certain time of year; in practice, this is derived over flat terrain by storm transposition and moisture adjustment to observed storm patterns.

The National Oceanic and Atmospheric Administration (NOAA) National Climatic Data Center is the primary source of meteorological data for the United States. Data for precipitation frequency and PMP estimates are available from NOAA online as well as in published reports. NOAA's NWS has published PMP guidance and studies since the late 1940s at the request of various Federal agencies and with funding provided by those agencies. Figure 7-1 illustrates the geographic coverage of the NWS Hydrometeorological Reports (HMR) series of publications.

In addition to the PMP studies published by the NWS, a number of States have published statewide updates that should be used for dam breach modeling and mapping efforts. Federal agencies and private dam owners have also commissioned site-specific PMP studies throughout the United States; however, site-specific PMP information is not readily available to the public or may be copy-right protected to the organization that commissioned the study or the author of the study.

### 7.4.2   Frequency-Based Precipitation

For recurrence-interval-based hydrology, the NWS is a primary source of statically based precipitation information, having published *Atlas 14 Precipitation-Frequency Atlas of the United States* (NWS, 2012; NOAA, 2004a, 2004b, 2006, 2009a, 2009b), as well as other related documents.

Precipitation frequency intervals are provided for the average recurrence intervals of 1, 2, 5, 10, 25, 50, 100, 200, 500, and 1,000 years. Duration is provided for 5, 10, 15, and 30 minutes; 1, 2, 3, 6, and 12 hours; and 1, 2, 4, 7, 10, 20, 30, 45, and 60 days. Current NWS document related to precipitation frequency and related studies are available at the following Web address: http://www.nws.noaa.gov/oh/hdsc/currentpf.htm.
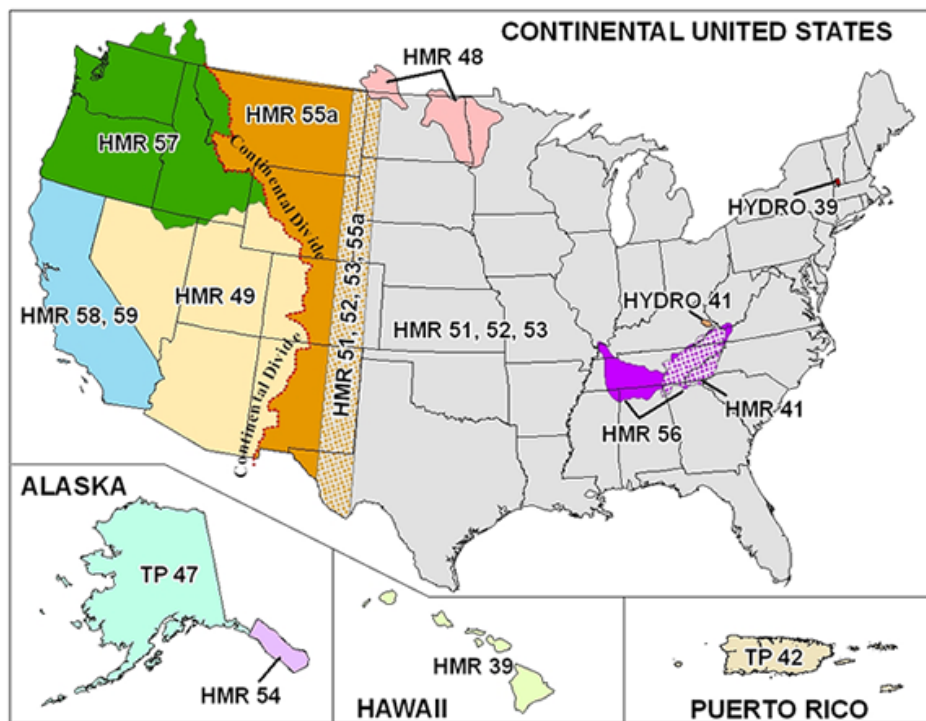
**Figure 7-1: Regions covered by different PMP documents (as of 2008)**

## 7.5    PRECIPITATION DISTRIBUTION

In addition to the key watershed physical characteristics that affect runoff, the temporal and spatial distribution of hypothetical storms in the watershed affect the peak discharge and runoff volume.

### 7.5.1    Temporal Distribution

The distribution of rainfall over the event duration must be considered. A number of rainfall distributions are referenced in hydrologic guidelines developed by Federal agencies that are used to compute hydrology for dam breach modeling. Use of these different rainfall distributions has been observed to change peak discharge values by 50 to 100 percent resulting in either an underestimation or overestimation of the magnitude of the peak discharge and flood hydrograph.

A center-peak-based distribution places the highest intensity rainfall at the center of the storm duration with varying intensities before and after the peak. An early-peak distribution shifts the maximum intensity to an earlier portion of the storm duration and conversely a late-storm distribution places the most extreme rainfall after the peak.

Typically an early-peak distribution produces a peak discharge lower than a center-peak distribution and a late-peak distribution produces a peak discharge higher than a center-peak distribution.

The selection of the appropriate temporal distribution should take into account geographic and watershed considerations. An early- or late-peak distribution may be appropriate for a very flat watershed with significant storage potential or arid areas where the runoff is immediate and flash floods occur.

Because the PMP rainfall event is hypothetical, following the guidance provided in the NWS HMR publications for temporal distribution and/or using an average center-peak distribution as the default distribution for dam breach inundation studies is recommended.

## 7.5.2   Spatial Distribution

Unless site- or State-specific PMP rainfall distribution data is available, guidance provided in the NWS HMR publications for the spatial placement of the PMP rainfall over the contributing watershed can be used.

For dam breach inundation studies, hydrologic studies are commonly performed for the watershed area draining to a dam and for the area downstream of the dam to the limit of study to calculate the downstream reach affected by a potential dam breach.

For dams in the watershed that are located on offline tributary streams to a main river, it is recommended that the NWS HMR PMP spatial distribution be evaluated two ways to determine the impact of the offline dam failure on downstream areas:

1. Spatially center the PMP event over the entire main river watershed with and without the failure of the offline dam.

2. Spatially center the PMP event over the contributing watershed of the offline dam with and without a dam failure.

This evaluation may produce results that show the largest PMF discharge for areas downstream of a dam is produced with the PMP centered over the entire main stem watershed area resulting in a small impact of the offline dam failure on downstream areas. However, when the offline dam is breached with the PMP centered over the tributary stream watershed, a lower PMF discharge may be coincident on the main stem and the influence of the dam failure may be more significant and a better indicator of the impact of a potential dam failure to downstream areas. Tailwater from the main stem channel should be considered when evaluating dam breach routing for the offline dam specifically when determining the lowest elevation of the breach.

## SECTION 8   DOWNSTREAM ROUTING ANALYSIS

Dam breach studies typically require the routing of the breach hydrograph downstream of the dam to evaluate the consequences of the hypothetical dam failure. The consequences of a hypothetical dam failure are determined by comparing the flood elevations and associated flood zone mapping for areas downstream of the dam with and without a dam failure for potential loss of life and economic losses.

The following sections provide guidance on how far downstream to consider the dam breach study and how to consider dams in series.

### 8.1   DOWNSTREAM EXTENT OF STUDY

The downstream extent or limit of study for a dam breach study should be evaluated on a site-specific basis. Dam breach modeling involves the calculation of a dam breach hydrograph and the routing of the hydrograph through the downstream channel and floodplain. As the breach hydrograph progresses downstream, attenuation and floodplain storage reduces the peak discharge and alters the shape of the hydrograph while maintaining the breach volume.

For some small dams, such as navigation dams, the small height and volume associated with the structure may not result in a significant breach inundation zone. In these cases, a breach inundation modeling and mapping effort may not be warranted.

For non-hydrologic fair weather dam failures, the dam breach peak discharge will typically be reduced as a result of attenuation and will eventually be confined to the downstream channel.

For hydrologic dam breach failures, hydrographs representing runoff upstream of the reservoir are included in the calculation of the dam breach hydrograph that is then routed to downstream areas. Lateral inflows are then added to the breach hydrograph for areas downstream of the dam to the downstream limit of study as explained below. As the dam breach hydrograph is routed through the downstream channel and floodplain, the flow is attenuated so that the contributing lateral inflows increase in percentage of the combined breach and lateral flow. Eventually, the effect of the dam breach on the flood elevations typically dissipates and the breach-non-breach flood elevations converge vertically within in a specified tolerance.

Because both non-hydrologic and hydrologic dam breach studies may be required, two different downstream limits of study may need to be determined. The following three-step guidance is provided to determine the extent of the combined downstream study reach:

1.  Perform a non-hydrologic fair weather dam breach failure using a simplified method, such as the GeoDamBREACH model, Dams Sector Analysis Tool (DSAT), or simple Hydrologic Engineering Center's River Analysis System (HEC-RAS) model, to a point downstream until the flood elevations are attenuated to the estimated capacity of the channel.

2.  Perform a hydrologic dam breach analysis to calculate the flood elevations with and without a hypothetical dam failure.

3. Establish the combined downstream limit of study as the most downstream point where habitable structures are not located in the non-hydrologic fair weather dam breach inundation zone *and* the with- and without-dam breach flood elevations for the hydrologic failure converge to a specified vertical tolerance.

The three-step guidance standardizes the determination of the downstream limit of study and ensures that habitable structures in the dam breach inundation zone are identified. By using a simplified non-hydrologic fair weather failure method such as GeoDamBREACH or DSAT, the downstream limit of study can be extended considerably downstream of the dam at little additional effort. This solves a potential problem where the downstream limit of study is set based on the absence of habitable structures in the breach inundation zone immediately downstream of the dam without knowledge that additional habitable structures may be at risk farther downstream.

The following sections provide guidance for tolerances to be used for establishing the downstream limit of study.

## 8.1.1  Non-Hydrologic Fair Weather (Sunny Day) Dam Breach Failures

Dam breach inundation studies should continue to the point where adequate floodwater disposal is reached and the breach flood no longer poses a risk to life and property damage. The downstream extent of study should be established using the following criteria:

- There are no habitable structures in the dam breach inundation zone, and anticipated future development in the floodplain is limited;

- Dam breach flood flows are contained within a large downstream reservoir;

- Dam breach flood flows are confined within the downstream channel; or

- Dam breach flood flows enter a bay or ocean.

When determining the downstream limit of study based on the absence of at-risk habitable structures, the vertical accuracy of the dam breach modeling should consider whether structures located immediately adjacent to but outside the breach zone are appropriately classified as not at risk.

With the extensive use of GIS-based dam breach modeling, the vertical accuracy of the digital elevation model (DEM) used in the calculation of the dam breach flood elevations should be used to establish a horizontal buffer beyond the breach inundation zone where surveying of the lowest adjacent grade elevations for uncertain habitable structures located in this buffer area are considered.

To establish whether the dam breach flood flows are confined to within the downstream channel when specific channel capacity information is unknown, USGS regional regression equations can be used to estimate the magnitude and frequency of the 2-year flood for estimating channel capacity for non-urban streams. The USGS National Streamflow Statistics Program provides regression equation publications by State. The National Streamflow Statistics Program software

and an accompanying database are available at
http://water.usgs.gov/osw/programs/nss/pubs.html.

The USGS also supports a Web-based GIS application called StreamStats
(http://water.usgs.gov/osw/streamstats/index.html), which provides stream flow statistics,
drainage-basin characteristics, and descriptive information for USGS data-collection stations and
user-selected ungaged sites.

In the event that the non-hydrologic fair weather failure limit of study is established solely based
on the presence of habitable structures in or out of the breach inundation zone and not channel
capacity, care must be taken to ensure that the breach inundation limit of study extends far
enough downstream. The concern is that habitable structures located immediately downstream of
the dam are located outside the dam breach inundation zone and the study limit is terminated
even though additional habitable structures exist in the breach inundation zone farther
downstream.

### 8.1.2   Hydrologic Dam Breach Failures

Hydrologic dam breach studies include the effect of basin runoff upstream and downstream of
the dam. Because of the contribution of watershed runoff downstream of a dam, the flood
elevations downstream of the dam resulting from a dam breach dam eventually converge to the
flood elevations without a dam breach during a hydrologic event.

The downstream limit of study is based on a set vertical elevation difference (or tolerance)
between the flood elevations calculated for a hydrologic event without a dam failure and the
flood elevations for the same event with a dam failure.

Establishing a small tolerance can require that the study limit extend miles downstream of the
dam potentially into downstream States, increasing the study effort beyond the financial means
of the entity commissioning the dam breach study. This section provides guidance is on avoiding
establishing limits that may financially discourage dam breach studies from being performed but
will still provide adequate information for dam safety purposes.

For hydrologic dam breach studies, the downstream limit of study should be established based on
the vertical elevation difference caused by a dam failure that produces an incremental rise in
flood elevations determined to no longer be a concern to life and property. The vertical elevation
difference is typically calculated by comparing with and without dam breach flood elevations
using hydraulic model results.

The vertical accuracy of the hydraulic model results related to the vertical accuracy of cross-
sections and the terrain model used for the mapping should be considered when determining
whether buildings and structures located in close proximity to the calculated dam breach
inundation zone are actually located in or out of the floodplain. Adding the vertical elevation
accuracy of the cross-sections or DEM, whichever is higher, to the computed dam breach water
surface elevation is recommended in order to determine whether buildings and structures are
located outside the breach inundation zone or whether accurate field surveying of the vertical
elevations of the structures is warranted.

It is recommended that the downstream limit of the study be set for situations where the vertical elevation tolerance between the flood elevations calculated for a hydrologic event *without* a dam failure and the flood elevations for the same event *with* a dam failure is 2 feet or less.

A lower threshold of 1 foot should be considered for non-rural areas or areas where a reliable dam failure warning system does not exist or the time for a dam breach flood to reach populated areas does not allow time for the population at risk to be warned and to evacuate.

Alternative methods to establish the downstream limit of study include ending the study when the dam breach flood elevation converges with the flood elevation of a set discharge or when the peak discharge travels for a specific period of time. For dam breach studies where the downstream limit of study has a large watershed area ratio compared to the area upstream of the dam, an alternative level of tolerance may be considered. Using a tolerance of 1 foot of convergence of the dam breach elevation for a specific extreme event (PMF or other extreme event specified by the State) compared to a regulatory 100-year flood elevation (or other recurrence interval-based event) may be appropriate in these situations. FEMA Flood Insurance Study results should be considered a source of peak discharge and flood elevation data for the 10-, 50-, 100-, and 500-year flood events. For dam breach studies where the peak flood discharge travels for a specific period of time (such as 24 hours), setting the downstream limit of study based on the selected flood wave travel time may be appropriate with the understanding that adequate flood warning has been provided by the NWS and local emergency management officials.

## 8.2    SEQUENTIAL DAM FAILURE

Sequential dams or dams in series (also known as domino-like or cascading dams) where the potential failure of an upstream dam or dams may negatively affect a downstream dam should be evaluated to determine the overall effects of multiple failures along the stream.

For non-hydrologic fair weather failures, the potential failure of an upstream dam should be considered in the potential non-hydrologic fair weather failure of the downstream dam. The dam breach hydrograph from the upstream dam should be routed downstream and through the storage area of the downstream dam using a dynamic storage routing procedure.

For hydrologic dam failure studies, the extreme flood event that is specified by the regulatory authority for use in the dam breach study must be applied similarly to all dams in series in a watershed. If the upstream dam has sufficient capacity to safely release the specified extreme event without failure, then the residual non breach hydrograph should be routed downstream to and through the downstream dam. In the event that the upstream dam does not have sufficient capacity to release the specified extreme event and may fail, then the breach hydrograph should be routed downstream to and through the downstream dam.

For dam design purposes, the downstream dam should be designed to accommodate the SDF or IDF specified by the regulatory authority and a dam breach hydrograph from an upstream dam in series if it is determined the upstream dam does not have sufficient capacity and cannot safely pass the same SDF or IDF event.

## SECTION 9   ESTIMATING BREACH PARAMETERS

A key element for calculating a dam breach hydrograph for a specific dam involves estimating the dam breach parameters for dam breach modeling related to the geometry and timing (e.g., width, depth, shape, and time of failure) of the breach formation.

It has been noted by several sources that the selection of breach parameters for modeling dam breaches contain the greatest uncertainty of all aspects of dam failure analysis and therefore a careful evaluation and understanding of the associated breach parameters is necessary (Wurbs, 1987; USBR, 1998; Wahl, 2004; Gee, 2008, etc.).

A number of methods are available for estimating breach parameters for use in dam breach studies. Since the selection of the breach parameters is specific to each dam, guidance is provided describing methods currently applied by dam safety professionals without recommending a standardized method.

## 9.1   BREACH PARAMETER DEFINITIONS

The following definitions are commonly accepted for use in evaluating and selecting dam breach parameters.

- **Breach formation time (also time-to-failure)** – The duration of time between the first breaching of the upstream face of the dam (breach initiation) and when the breach has reached it full geometry.

- **Breach depth (also breach height)** – The breach depth is the vertical extent of the breach measured from a specific elevation to the invert of the dam breach.

- **Breach width** – The breach width is the average of the final breach width, typically measured at the vertical center of the breach.

- **Breach side slope factor** – The breach side slope is a measure of the angle of the breach sides represented as X horizontal to 1 vertical (XH: 1V).

A dam breach usually occurs in two distinct phases starting with the breach initiation followed by the breach formation.

**Breach initiation:** During the breach initiation phase, flow through the dam is minor and the dam is not considered to have failed. It may be possible to prevent a dam breach during this phase if flow is controlled.

**Breach formation:** Breach formation (defined above) begins when the flow through the dam has increased and progressed from the upstream face to the downstream face of the dam, is uncontrolled, and will result in the failure of the dam.

## 9.2   DISCUSSION OF SELECTION OF BREACH PARAMETERS

Many factors must be considered in selecting appropriate breach parametersincluding dam type, dam dimensions, and dam materials of construction. Other pertinent information such as

historical records of seepage or foundation problems should also be considered (Gee, 2009). As discussed in Section 4 of this document, dam failures occur for a wide variety of reasons. This section contains a detailed discussion of some of the more common dam breach mechanisms for embankment and concrete dams.

## 9.2.1 Breach Mechanisms for Embankment Dams

Although breaching in embankment dams may occur for a variety of reasons, breaches in embankment dams are most often modeled as overtopping or piping failures.

### 9.2.1.1 Overtopping Failures

Overtopping failures can occur very differently depending on the composition of the dam. Perhaps the simplest overtopping failure to discuss is *failure of a cohesive soil embankment*. According to a study by Ralston (1987), a small headcut typically forms on the downstream face of a cohesive soil embankment and progresses upstream as shown in Figure 9-1.
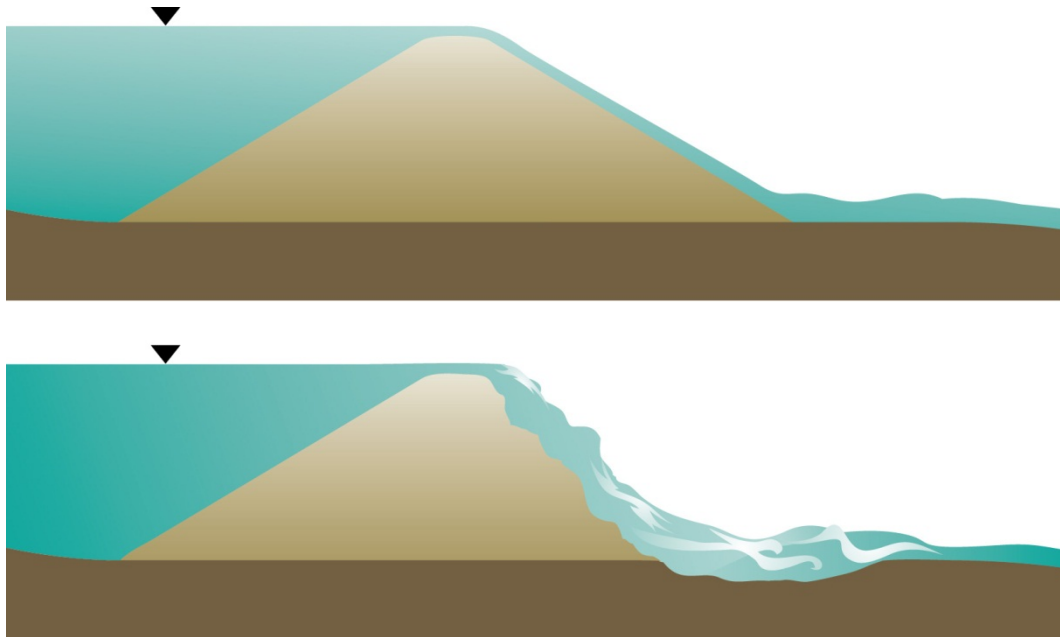
*Source: Adapted from Powledge et al. (1989)*

**Figure 9-1: Erosion on the downstream face of a cohesive soil embankment dam**

The breach is considered to begin when erosion occurs across the width of the dam crest. After the breach initiates at the top of the dam crest, it enlarges to its ultimate extent. If there is no physical reason to believe the embankment would fail at a certain location, the breach should be modeled as initiating at the maximum section typically located at the centerline of the downstream main channel. A generalized trapezoidal breach progression is illustrated in Figure 9-2.
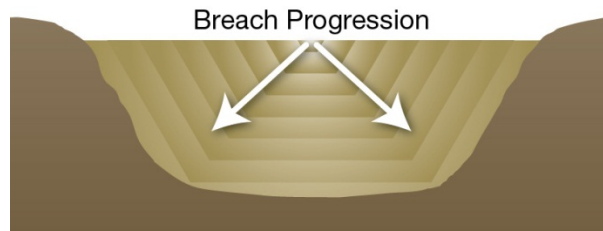
*Source: Adapted from Gee, 2009*

**Figure 9-2: Overtopping trapezoidal breach progression**

The breach may stop growing when the reservoir has emptied and there is no more water to erode the dam or the dam has completely eroded to the bottom of the reservoir or has reached bedrock (Gee, 2009). The breach progression may be modeled as either a linear progression or a sine wave progression:

- **Linear progression:** rate of erosion remains the same for the duration of erosion development)

- **Sine wave progression:** breach grows very slowly at the beginning and end of development and rapidly in between

In a study by the State of Colorado Department of Natural Resources, no significant difference were found between linear and sine wave progression models when comparing one overtopping case study in HEC-Hydrologic Modeling System (HMS) and HEC-RAS (2010). Both progressions should be evaluated and the progression with the more conservative results should be utilized.

### 9.2.1.2    Piping / Internal Erosion Failures

Piping and internal erosion occurs when concentrated seepage develops within an embankment dam. The seepage slowly erodes the dam, leaving large voids in the soil. Typically, piping begins near the downstream toe of the dam and works its way toward the upper reservoir. As the voids become larger, erosion becomes more rapid (refer also to Section 4). Water flow through the embankment will appear muddy as erosion increases. Once the erosion reaches the reservoir, the piping hole can enlarge and cause the dam crest to collapse. Figure 9-3 shows a schematic of a fully formed piping hole.

Piping failures are typically modeled in two phases, before and after the dam crest collapses. Water flow through the piping hole is modeled as orifice flow before the dam crest collapses and as weir flow after the dam crest collapses. For small dams constructed from cohesive soils, it is possible for the reservoir to completely empty before the dam crest collapses (State of Colorado Department of Natural Resources, 2010).

There are several possible options to identify the breach initiation time. For breaches associated with a hydrologic event, the initiation can be considered to begin when the reservoir water level reaches a certain elevation or after the water level has exceeded a certain elevation for a specified

duration. For fair weather breach analysis, an initiation time should be specified regardless of pool elevation (Gee, 2010).
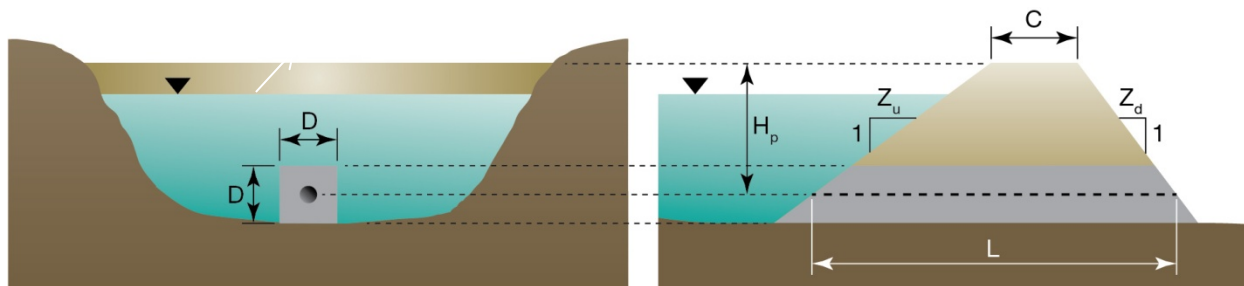


D= piping hole width/height assumed square     L = length of the pipe

$H_p$ = breach depth for piping                 Z = horizontal slope of the embankment

C = dam top width

*Source: Adapted from State of Colorado Department of Natural Resources, Division of Water Resources (2010)*

**Figure 9-3: Schematic of piping hole**

## 9.2.2   Breach Mechanisms for Concrete Dams

Concrete dam failures are typically modeled as structural failures. As such, there are different failure mechanisms dependent on the type of dam. This section focuses on failure of concrete gravity dams and concrete arch dams. Refer to Section 3.1 for general information regarding these dam types.

**Concrete gravity dams:**  Concrete gravity dams are typically constructed from numerous concrete monoliths. For this type of dam, USACE suggests using an average breach width of multiple monoliths (2007), while FERC (1988) and NWS (Fread, 2006) suggest using an average breach width of less than or equal to half of the entire length of the dam.

USACE, FERC, and NWS all suggest using a vertical breach side slope since monoliths are typically rectangular in shape and therefore have vertical sides. Figure 9-4 illustrates the monolith failure width (B), the breach depth (H), and the vertical side slope of 0:1. The range of possible failure times for modeling purposes is 0.1 hours to 0.5 hours.
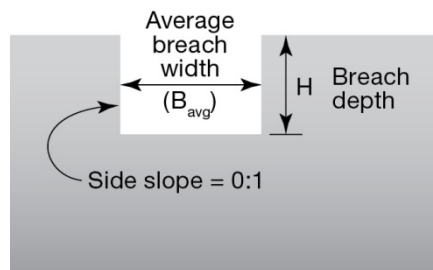


**Figure 9-4: Schematic of a concrete gravity dam breach**

**Concrete arch dams**:  As discussed in Section 3.1, the most common location for a gravity-arch dam is in a deep canyon with steep side walls. For this reason, the breach side slope is assumed to range from vertical to the slope of the valley wall. The suggested breach widths for this type of dam range from 80 percent of the entire length of the dam to the entire length of the dam (refer to Figure 9-5). The breach formation time for modeling purposes ranges from instantaneous to 0.1 hours (USACE, 1980 and 2007; FERC, 1988; Fread, 2006).



$B_{ave}$ = breach width average
H = breach depth
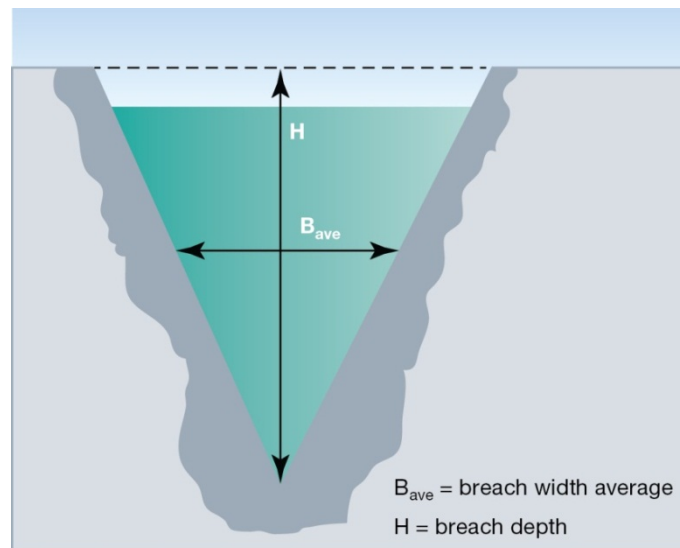
**Figure 9-5: Schematic of a concrete arch dam breach**

## 9.3    PUBLISHED BREACH PARAMETER ESTIMATION METHODS

A variety of methods are used by dam safety professionals to estimate dam breach parameters and the resultant dam breach peak discharge and timing. These methods are summarized below and described in detail in Sections 9.3.1 to 9.3.3.

- **Physically Based Erosion Methods** – These methods predict the development of an embankment breach and the resulting breach outflows using an erosion model based on principles of hydraulics, sediment transport, and soil mechanics.

- **Parametric Regression Equations** – These equations, developed from case study information, are used to estimate the time-to-failure and ultimate breach geometry. The breach can then be simulated to proceed as a time-dependent linear process with the computation breach outflows using principles of hydraulics.

- **Predictor Regression Equations** – These equations estimate the dam breach peak discharge empirically based on case study data of peak discharge and hydrograph shape.

### 9.3.1    Physically Based Erosion Models

Since the 1960s there have been numerous developments of physically-based, numerical dam breach models. In 1965, the first breach model was proposed (Cristofano, 1965), pioneering the development by others of physically based models BRDAM (1977), Dam Break Forecasting

Model (DAMBRK) (1977), Breach Erosion of Earth-Fill Dams and Flood Routing (BEED) (1985), and BREACH (NWS, 1988).

Currently, the NWS BREACH model is a well-known and commonly applied physically based model developed by a Federal agency. The NWS BREACH model was developed to more realistically simulate breaches initiated by overtopping or piping in an embankment dam. A modified form of the Meyer-Peter and Muller sediment transport equation is used in this model. The NWS BREACH model is described in detail in Section 10.3.3.

The NRCS SITES and WinDAM models are other Federal-sponsored models. The NRCS SITES model was developed for estimating headcut erosion of earthen spillways. It is an integrated design program for the hydraulic and hydrologic analysis of dams and has the ability to analyze the stability and integrity of vegetated earthen spillways.

A number of commercial models are also available and are described throughout Section 10 of this document.

### 9.3.2   Parametric Regression Equations

Parametric regression equations are empirically derived using case study information to estimate the time-to-failure and ultimate breach geometry, then simulate breach growth as a time-dependent linear process to compute breach outflows using principles of hydraulics. Numerous equations to predict breach parameters have been developed based on analyses of case studies. Table 9-1, adapted from DSO-98-004 (USBR,1998), provides the most common parametric regression equations developed based on information from case studies of historic dam failures available at the time of this publication. Refer to DSO-98-004 for the units of measure attributed to the equations in Table 9-1; the units of measure vary by equation in metric or English units.

**Table 9-1: Published Parametric Regression Equations for**
**Predicting Breach Parameters**

| Reference | Number of Studies | Relations Proposed (S.I. units, meters, m$^3$/s, hours) |
|---|---|---|
| Johnson and Illes (1976) | | $0.5h_d$  $B$  $3h_d$   for earth-fill dams |
| Singh and Snorrason (1982, 1984) | 20 | $2h_d$  $B$  $5h_d$<br>$0.15$m  *dovtop*  $0.61$m<br>$0.25$hr  $t_f$  $1.0$hr |
| MacDonald and Langridge-Monopolis (1984) | 42 | Earth-fill dams:<br>$V_{er} = 0.0261(V_{out}{}^*h_w)^{0.769}$                      [best-fit]<br>$T_f = 0.0179(V_{er})^{0.364}$                  [upper envelope]<br><br>Non-Earth-fill dams:<br>$V_{er} = 0.00348(V_{out}{}^*h_w)^{0.852}$                    [best-fit] |
| FERC (1987) | | $B$ is normally 2-4 times $h_d$<br>$B$ can range from 1-5 times $h_d$<br>$Z = 0.25$ to $1.0$              [engineered, compacted dams]<br>$Z = 1$ to $2$              [non-engineered, slag or refuse dams]<br>$t_f = 0.1$-$1$ hours                 [engineered, compacted earth dam]<br>$t_f = 0.1$-$0.5$ hours             [non-engineered, poorly compacted] |

| Reference | Number of Studies | Relations Proposed (S.I. units, meters, m³/s, hours) |
|---|---|---|
| Reclamation (1988) | | $B = (3)h_w$<br>$t_f = (0.011)B$ |
| Singh and Scarlatos (1988) | 52 | Breach geometry and time of failure tenancies<br>$B_{top}/B_{bottom}$ averages 1.29 |
| Von Thun and Gillette (1990) | 57 | $B, Z, t_f$ guidance (see discussion) |
| Dewey and Gillette (1993) | 57 | Breach initiation model; $B, Z, t_f$ guidance |
| Froehlich (1995) | 63 | $\overline{BR} = 0.1803\, K_o V_w^{0.32} h_b^{0.19}$<br>$t_f = 0.00254\, V_w^{0.53}\, h_b^{(-0.90)}$<br>$K_o = 1.4$ for overtopping; 1.0 otherwise |
| Froehlich (2008) | 74 | $\overline{BR} = 8.239\, K_o V_w^{0.32} h_b^{0.04}$<br>$t_f = 3.664\,(V_{w/g} H_b^2)^{0.5}$<br>$K_o = 1.3$ for overtopping; 1.0 otherwise |

These empirical regression equations were developed to predict the average breach width, breach depth, and time-of-failure or formation time. Wahl (2010) suggests that one of the main advantages of using empirical parametric regression equations is that the user can exhibit some control over the breach parameters used in the model, and thus account for site-specific factors.

The following discussion extending to the end of Section 9.3.2 provides historical information about model development. Much of the discussion is included from DSO-98-004 (USBR, 1998) with some text editing.

Johnson and Illes (1976) published a classification of failure configurations for earth-fill, gravity, and arch dams. The breach shape for earthen dams was described as varying from triangular to trapezoidal as the breach progressed. Singh and Snorrason (1982) conducted a study of 20 dam failures and noted that breach width was generally between two and five times the dam height. They also found that the breach formation time was generally 15 minutes to 1 hour and the maximum overtopping depth prior to failure (for overtopping failures) ranged from approximately 0.5 foot to 2 feet.

Based on 42 dam failure case studies, MacDonald and Langridge-Monopolis (1984) proposed a "breach formation factor," defined as the product of the volume of breach outflow and the depth of water above the dam. They related this factor to the volume of material eroded from the dam's embankment. The amount of water to pass through the breach is not known before breach analysis occurs; however, the entire volume of the reservoir can be used as a starting estimate (Gee, 2009). Based on their study, MacDonald and Langridge-Monopolis also concluded that the breach can be assumed to be trapezoidal with a side slope of 0.5H:1V. The study further presented an envelope equation for the breach formation time for earth-fill dams.

FERC presents breach parameter estimates in their *Engineering Guidelines for the Evaluation of Hydropower Projects* (1987, revised 1993). Breach parameter values are proposed for the breach width, side slope, and time-to-failure. Parameter suggestions are dependent on the type of dam (i.e., arch, buttress, masonry, gravity, monoliths, earth-fill, rock-fill, timber, crib, slag, and

refuse). For example, the proposed breach width for an arch dam is the entire length of the crest, while the proposed width for an earthen dam is a range between the height of the dam and five times the height. The suggested time-to-failure ranges from less than 0.1 hours for arch dams to 0.1 to 1 hour for earthen dams (engineered and compacted). Chapter 2 of the FERC guidance recommends a sensitivity analysis be performed to justify selection of breach parameters.

The USBR Simplified Dam-Break (SMPDBK) model (1988) provides a method for selecting ultimate breach width and time of failure. The USBR specifies that these parameters are to be used in hazard potential classification studies and are intended to provide conservative, "upper bound" values. The proposed breach width for earthen dams is three times the breach depth, defined as the distance between the initial reservoir water level and the breach bottom elevation. The recommended time for the breach to develop in hours is 0.011 times the breach width in meters.

Based on a study of 52 earthen embankment dam breaches, Singh and Scarlatos (1988) determined that the ratio of top and bottom breach widths ranged from 1.06 to 1.74, with an average value of 1.29. They also noted that the majority of breach formation times were less than 1.5 hours and most were less than 3 hours.

In 1995, Froehlich developed equations for breach width and breach formation time based on a total of 63 case studies. Froehlich suggests using a breach side slope factors of 1.4 for overtopping failures and 0.9 for other failure modes . Froehlich provided further guidance regarding breach parameters in 2008 based on data collected from 74 embankment dam failures. Froehlich proposes a mathematical expression for final breach peak flow, breach width, side slope of a trapezoidal breach, and formation time. The findings of the statistical analysis were applied in a Monte Carlo simulation to estimate the degree of uncertainty (Froehlich, 2008).

### 9.3.3   Predictor Regression Equations

Predictor regression equations are empirically developed equations used to estimate peak discharge based on actual case study data. These equations are used as a prediction method to determine a reasonable outflow hydrograph shape.

Table 9-2 presents the empirical relationships developed by various authors for predicting peak breach discharge. The equations presented in Table 9-2 were adapted from DSO-98-004 (USBR, 1998). These equations are based on case study data used to develop empirical equations relating peak breach outflow to dam height and/or reservoir storage volume. The predictor regression equations provide an alternative method of computing the dam breach discharge; they can be used instead of determining breach parameters and then using a hydrologic-hydraulic model to compute the breach hydrograph.

**Table 9-2: Published Predictor Regression Equations for Prediction of Peak Breach Flow**

| Reference | Case Studies | Relations Proposed | Notes |
|---|---|---|---|
| Babb and Mermel (1968) | >600 incidents | | Many cases not well documented |
| Kirkpatrick (1977) | 16 (plus 5 hypothetical failures) | $Q_p = f(h_w)$ | |
| Soil Conservation Service (SCS) (1981) | 13 | $Q_p = f(h_w)$ | |
| Hagen (1982) | 6 | $Q_p = f(h_w{}^*S)$ | |
| Reclamation (1982) | 21 | $Q_p = f(h_w)$ | |
| Graham (1983) | 6 | | Dams with large storage-to-height ratios |
| Singh and Snorrason (1982, 1984) | 20 real failures and 8 simulated failures | | $Q_p$ relations based on simulations |
| Graham (undated) | 19 | $Q_p = f(h_w, S)$ | |
| MacDonald and Langridge-Monopolis (1984) | 42 | $V_{er} = f(V_{out}{}^*h_w)$ $t_f = f(V_{er})$ $Q_p = f(V_{out}{}^*h_w)$ | |
| Costa (1985) | 31 constructed dams | $Q_p = f(hd)$ $Q_p = f(S)$ $Q_p = f(hd{}^*S)$ | |
| Evans (1986) | | $Q_p = f(V_w)$ | |
| FERC (1987) | | Guidance for $B$, $Z$, $t_f$ | |
| Reclamation (1988) | | $B$, $t_f$ guidance | |
| Singh and Scarlatos (1988) | 52 | Guidance for $B$, $Z$, $t_f$ | |
| Von Thun and Gillette (1990) | 57 | $Z$ guidance $B = f(h_w, S)$ $t_f = f(h_w,$ erosion resistance) | |
| Froehlich (1995b) | 63 | $B$, $Z$, $t_f$ relations | |
| Froehlich (1995a) | 22 | $Q_p = f(V_w, h_w)$ | |
| Froehlich (2008) | 74 | $Q_p = 3.1 B_{avg} H_w^{1.5} \left( \dfrac{\gamma}{\gamma + T_f \sqrt{H_w}} \right)^3$ | |

## 9.4  SUMMARY OF TYPICALLY USED BREACH PARAMETERS

The selection of breach parameters for a dam is specific to the dam and therefore guidance is not provided for one method or set of breach parameters. The following guidance presents a summary of breach parameters or range of parameters covering both overtopping and piping breach situations referenced in State and Federal dam safety guidelines and used for dam breach modeling (Table 9-3).

**Table 9-3: Typical Breach Parameters or Range of Parameters**

| Earth-Fill Dams | |
|---|---|
| Average breach width | ½ to 5 times the dam height |
| Side slope of breach | 0:1 to 1:1 |
| Breach formation time | 0.1 to 4 hours |
| **Concrete Gravity Dams** | |
| Breach width | A multiple of monolith widths |
| Side slope of breach | 0:1 |
| Breach formation time | 0.1 to 0.5 hours |
| **Concrete Arch Dams** | |
| Breach width | Entire dam width |
| Side slope of breach | 0:1 to valley wall slope |
| Breach formation time | Nearly instantaneous, $\leq 0.1$ hour |

Dam breach parameter selection guidance published in Chapter 2, Appendix II-A of FERC's *Engineering Guidelines for the Evaluation of Hydropower Projects* (FERC, 1993) is widely referenced as an acceptable method by regulating authorities and is provided in the following table. Refer to pages 2-A-10 and 2-A-11 of that document for comments on using Table 9-4.

**Table 9-4: FERC Suggested Breach Parameters**

| Parameter | Value | Type of Dam |
|---|---|---|
| Average width of breach (BR) | BR = Crest length | Arch |
| | BR = Multiple slabs | Buttress |
| | BR = Width of 1 or more | Masonry, Gravity monoliths |
| | Usually $\overline{BR} \leq 0.5$ W | |
| | $HD \leq \overline{BR} \leq 5HD$ ............................... | Earth-fill, rock-fill |
| | (usually between 2HD to 4HD)........ | Timber Crib |
| | $\overline{BR} \geq 0.8$ x Crest Length ................... | Slag, Refuse |
| Horizontal component of side slope of breach (Z) | $0 \leq Z \leq$ slope of valley walls ............ | Arch |
| | Z = O   ........................................... | Masonry, Gravity Timber Crib, Buttress |
| | $\frac{1}{4} \leq Z \leq 1$ ........................................ | Earthen (engineered and compacted) |
| | $1 \leq Z \leq 2$ ........................................ | Slag, Refuse (non-engineered) |

| Parameter | Value | Type of Dam |
|---|---|---|
| Time-to-failure (TFH) (in hours) | TFH ≤ 0.1 ......................................... | Arch |
| | 0.1 ≤     TFH ≤ 0.3 .......................... | Masonry, Gravity, Buttress |
| | 0.1 ≤     TFH ≤ 1.0 .......................... | Earthen (engineered, and compacted), Timber Crib |
| | 0.1 ≤     TFH ≤ 0.5 .......................... | Earthen (non-engineered, poor construction) |
| | 0.1 ≤     TFH ≤ 0.3 .......................... | Slag, Refuse |

Definitions:

HD    =    Height of dam

Z      =    Horizontal component of side slope of breach

$\overline{BR}$    =    Average width of breach

TFH    =    Time to fully form the breach

W     =    Crest length

*Source: FERC, 1993*

## 9.5    UNCERTAINTY OF PREDICTED RESULTS

Progress has been made over the past decade in the field of dam failure analysis. The use of sophisticated GIS and computer resources has made it easier to integrate flow information to predict the reservoir outflow hydrograph. However, predicting the reservoir outflow hydrograph remains a great source of uncertainty, especially for embankment dams in which dam failure is usually a complex progressive process that is difficult to model (Wahl, 2010). Since the scale of estimated consequences associated with a dam failure can be sensitive to the choice of breach parameters, careful consideration should be given to the selection of the proper method(s) of determining breach parameters and the uncertainty associated, not only with the parameters themselves, but of the overall result of the breach modeling efforts.

Numerous regression equations, summarized in Tables 9-1, 9-2,9-4 and 10-3, have been developed for peak discharge and breach parameters. The available equations vary widely depending on the analyst and the types of dam failures studied. Regression equations i.e. parametric and predictors, suffer from a lack of well-documented case study data as well as a high level of uncertainty in the data used to develop the equations. Approximately 75 percent of the dams for which historical data was available and used deriving the equations are less than 50 feet in height; therefore, these equations may not be representative of dams greater than 50 feet in height. According to Wahl (2010), the best methods of breach width prediction are empirically derived parametric equations (e.g., USBR [1988]; Von Thun & Gillette [1990], and Froehlich (1995). These methods were found to have uncertainties of about ± one-third of an order of magnitude.

In general, predictions of the side slope of dam breach openings have a high uncertainty, although this is of secondary importance since breach outflows are relatively insensitive to the selection of side slopes. Observed breach openings are generally vertical, but researchers believe

that a sloped side slope occurs during the breaching event and that the side slope continues to erode after the breach to form the observed vertical condition.

Predictions of breach formation time also have a very high uncertainty due to a lack of reliable case study data; many dams fail without eyewitnesses, and the problem of distinguishing between breach initiation and breach formation phases has likely tainted much of the data (USBR, 1998). Time-of-failure predictions based on empirically derived parametric equations also have great uncertainty, on the order of magnitude of approximately ± two-thirds, with Froehlich (1995b) having the lowest related uncertainty (Wahl, 2010).

Analyzing and predicting the flow of impounded mine tailings is difficult. Many factors affect the flow of tailings, including moisture content, degree of consolidation, viscosity, grain size and shape, amount of supernatant water, and solid/liquid interaction.

## 9.6    PERFORMING A SENSITIVITY ANALYSIS TO SELECT FINAL BREACH PARAMETERS

With a wide range of methods available to assign breach parameters that can result in a wide range of results for breach width and breach formation time, a sensitivity analysis should be performed for the breach parameters prior to selecting the final breach parameters.

The sensitivity analysis should not be restricted to identifying the impact of varying the breach parameters on the peak discharge, breach discharge, and breach hydrograph at the dam. It should also identify the effect of breach parameters on the calculated water surface elevations at locations of interest downstream of the dam. While a model may indicate that the stage and outflow at the dam vary greatly depending on the selected breach parameters, the sensitivity of the stage, flow, and travel time to an area of interest downstream of the dam may be smaller due to flood attenuations and floodplain hydraulics.

Significant engineering judgment must be exercised in interpreting breach parameter and/or breach peak flow results. The sensitivity analysis could involve using several widely used predictor equations to establish breach parameters. The final determination of the parameters can include a comparison to parameters considered acceptable to the Federal or State regulating authority approving the breach inundation study.

# SECTION 10 ANALYSIS TOOLS FOR DAM FAILURE MODELING

Models for prediction of a dam breach have existed since the mid-1960s. However, the need for further development of dam breach models was realized in the 1970s as a result of several fatal dam failures, The sections that follow document the history of dam breach modeling and outline the current state of dam breach modeling tools. Recommendations on the selection of modeling software are provided.

## 10.1 HISTORY OF DAM BREACH MODELING

In 1977, the NWS developed **DAMBRK**, a model to analyze the dam breach process and route peak breach outflows to determine inundation depths downstream of the dam. Between 1977 and the mid-1990s, a series of regression relations were developed to predict breach parameters and peak discharge from breached embankment dams. Statistical regression analyses such as MacDonald and Langridge-Monopolis (1984), USBR (1988), Von Thun and Gillette (1990), Dewey and Gillette (1993), and Froehlich (1995, 2008) were developed for use with empirical methods of evaluating dam failure and breach parameters. The breach modeling process was further advanced with the prediction of the reservoir outflow hydrograph and the routing of the hydrograph downstream through the use of two types of breach models: physical and empirical.

Physical models are based on physical laws and empirical relations governing flow and erosion. However, these types of models are not widely used in dam breach assessment because of lack of data to estimate breach erosion. The most notable model is the **NWS BREACH** (1985), a physically based mathematical model used to predict the breach characteristics and the discharge hydrograph emanating from a breached earthen dam. The model was developed by coupling the conservation of mass of the reservoir inflow, spillway outflow, and the breach outflow within the sediment transport capacity of the unsteady uniform flow along an erosion-formed breached channel (Fread, 2001; Wahl, 2004).

Conversely, empirical models, also known as parametric models, are based on predetermined controlled input parameters for estimation of a resulting breach through regression equations. These models are based on vast study information for estimation of time-to-failure and ultimate breach geometry, which can then be used to simulate breach growth as a time-dependent linear process, computing breach outflow in a triangular or trapezoidal shape. Examples of one-dimensional empirical breach models include **NWS DAMBRK** (1988) and its successor NWS Flood Wave Dynamic Model (**FLDWAV**) (1998), HEC-1, HEC- HMS, and **HEC-RAS** (State of Colorado Department of Natural Resources, 2010; Wahl, 1997 and 2004).

HEC-1 and HEC-HMS are watershed modeling software capable of generating a breach hydrograph from predefined breach parameters (i.e., breach width, time of failure, etc.) input in the model. NWS DAMBRK/FLDWAV, an unsteady model, and HEC-RAS, capable of both steady-state and unsteady routing, are based on the St. Venant equations for flow computations, which generate a breach hydrograph and route the flow wave downstream. Both HEC-HMS and HEC-RAS are frequently updated by the USACE to include additional capabilities. HEC-RAS has the capability of interfacing with GIS for generation of inundation maps using predefined

locations downstream of the breach event coupled with a flow quantity, water surface elevation, and travel time to the location.

Simplified methods for predicting peak breach discharge were developed using regression equations by Kirkpatrick (1977), Soil Conservation Service ([SCS]1981), Hagen (1982), USBR (1982), MacDonald and Langridge-Monopolis (1984), Singh and Snorrason (1984), Costa (1985), Evans (1986), and Froehlich (1995, 2008), each of whom relate the predicted peak discharge as a function of various dam and/or reservoir parameters developed from analyses of historical dam failures (Wahl, 2004).

Simplified methods used in breach hydrograph generation and downstream routing include the SCS Technical Release (TR) No. 66, *Simplified Dam Breach Route Procedure* (1985), and the **NWS SMPDBK** (1991). TR-66 presents a method for estimating the breach hydrograph and the peak flood flow at a predefined location, associated maximum depth of flow, and the time to peak flow using a simplified Attenuation-Kinematic (Att-Kin) flood routing method (SCS, 1985).

SMPDBK is a simplified model for predicting downstream flooding produced by dam failure. This program is still capable of producing the information necessary to estimate flooded areas resulting from dam-break floodwaters while substantially reducing the amount of time, data, and expertise required to run a simulation of the more sophisticated unsteady NWS DAMBRK, now called FLDWAV. SMPDBK is capable of predicting necessary information to estimate flooded areas resulting from a dam break, but does not account for backwater effects of additional downstream inflow.

In 2012, FEMA developed the **GeoDamBREACH** toolset, which is based on the NWS SMPDBK model. GeoDamBREACH includes an automated GIS-based mapping function for producing breach inundation mapping and FEMA non-regulatory products for dams. It also includes a semi-automated EAP function.

Recent developments in dam breach modeling have been concentrated in the area of two-dimensional hydraulic modeling for dam breach flood routing and on the erosion processes of dam failure in physically based modeling. Two-dimensional models with GIS integration are now common, allowing more sophisticated analyses. Two-dimensional models—such as the Decision Support System for Water Infrastructural Security (**DSS-WISE**) developed by the National Center for Computational Hydroscience and Engineering (NCCHE) of the University of Mississippi,[1] **MIKE**© software by DHI,[2] and **FLO-2D**© by FLO-2D Software, Inc.[3]—solve either full dynamic or simplified forms of conservative or non-conservative two-dimensional shallow water equations where as one-dimensional flow uses one-dimensional, cross-section-averaged shallow water equations. Two-dimensional model strengths are highlighted in

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[1] http://www.ncche.olemiss.edu/

[2] http://mikebydhi.com/

[3] http://www.flo-2d.com/

unconfined alluvial fans where overland flow cannot be accurately modeled in a one-dimensional model *(Altinakar, 2008; Wahl et al., 2008; Wahl, 2009 and 2010)*.

Researchers are focusing on developing a new generation of physically based hydraulic routing models to estimate the erosion processes associated with dam failure; understanding these processes better will facilitate more accurate determinations of downstream dam breach inundation mapping zones. Current models rely on the user to input parametric descriptions of the breach event and the model simulates flow through the breach as it enlarges at the specified rate (Wahl, et al., 2008).

The ARS is currently conducting research for the SIMBA model. According to information provided to FEMA by the ARS, "research conducted over the past decade has led to a greater understanding of the overall breach process of overtopped embankment dams. This research has included physical model studies involving relatively small-scale embankments in the United Kingdom, relatively large-scale embankments in Norway, and intermediate-scale embankments in the United States. Supporting research has also been conducted on vegetal slope protection materials characterization and erosion processes. This increased understanding has allowed for the development of computational models that better reflect the complex physical action observed during embankment dam breach. Initially a computational research tool, SIMBA was developed to evaluate and validate the resulting algorithms used to link and quantify the complex erosive action associated with an embankment breach during overtopping. Through a collaborative effort between ARS, NRCS, and Kansas State University, the erosion technology within SIMBA has been incorporated into WinDAM B.

The **WinDAM** software is being developed in stages reflecting the progress of the underlying research. WinDAM A focused on the concept of allowable overtopping of embankments protected by grassy vegetation or riprap. WinDAM B extends WinDAM A to include the erosion and potential breach of a homogeneous embankment overtopped sufficiently to generate failure of any slope protection present. WinDAM is expected to be further expanded in the future to include non-homogenous embankments and breach initiated by internal erosion.

According to the ARS, the future of dam breach modeling will continue to rapidly advance with the integration of software to perform advanced breach analyses and modeling downstream effects with more accurate inundation mapping using a GIS platform. As advancement and technology allow for more sophisticated modeling programs to run in shorter amounts of time, the use of two-dimensional analysis will likely become more prominent.

## 10.2   OVERVIEW OF DAM BREACH MODELING

Performing a dam breach model involves prediction of the dam breach hydrograph and the routing of that hydrograph downstream. A number of modeling tools are available to perform dam breach modeling, ranging from simple methods to complex models. With advancements in GIS-based modeling, many models can interface with digital terrain data to produce automated dam breach inundation zone delineations.

Section 10.3 describes the most commonly used and currently available dam breach models. Section 10.4 provides guidance on model selection for a specific dam breach study.

Dam breach modeling can be divided into two categories, each of which has a number of models, tools, or equations, ranging from simple to advanced:

1. Tools that generate the dam breach peak discharge and/or hydrograph only; and

2. Tools that develop a breach hydrograph and perform downstream flood routing using a one- or two-dimensional hydraulic model.

Simplified numerical models typically relate the breach hydrograph (or breach peak flow) to simple reservoir characteristics such as reservoir volume and dam height. These models may or may not include hydrologic modeling to determine the envelope maximum water depths to calculate the breach flow. Most simplified models do not consider complicated downstream conditions such as backwater effects. Additionally, reservoir routing (if present) uses level pool routing methods; in other words, the reservoir water surface is considered level during drawdown. This simplification is not applicable to all situations. The main benefit of simplified numerical models is that substantially less time is required to set up and execute these models. Table 10-1 presents a matrix of the most widely used dam breach modeling tools/models and their general capabilities.

One-dimensional models solve either full dynamic or simplified forms of one-dimensional, cross-section-averaged shallow water equations. These models are more sophisticated than simplified numerical models and do typically consider backwater effects; many one-dimensional models are capable of dynamic reservoir routing rather than level pool routing. The one-dimensional models discussed in this document also have downstream routing capabilities. One-dimensional routing is fairly sophisticated, but is best suited for modeling flow through a well-defined, confined channel. For routing over wide, flat surfaces, such as floodplains, one-dimensional models make certain assumptions (such as uniform flow velocity over a cross-section) that are not true and can have significant consequences on the accuracy of the model. Routing these situations using one-dimensional models is possible using appropriate, conservative modifications; however, another option is to use two-dimensional models that can more accurately model flow over floodplains.

Two-dimensional models use full dynamic or simplified forms of one- and two-dimensional shallow water equations to solve both one-dimensional channel flow and two-dimensional overland flow. Two-dimensional models are capable of routing flow over unconfined floodplains where flood waters are not contained within a defined channel.

**Table 10-1: Matrix of Most Widely Used Dam Breach Modeling Tools**

| Method | Computation of Peak Breach Outflow | Computation of Ultimate Breach Parameters | Breach Hydrograph Generation | Downstream Routing Capability | | | |
|---|---|---|---|---|---|---|---|
| | | | | Steady-State | Unsteady-State | 1-D Flow | 2-D Flow |
| **BREACH HYDROGRAPH GENERATION ONLY** | | | | | | | |
| Empirical Equations | ✓ | ✓ | | | | | |
| NWS BREACH | ✓ | ✓ | ✓ | | | | |
| USACE HEC-1 and HEC-HMS | ✓ | | ✓ | Without downstream hydrologic routing | ✓ | | |
| **BREACH HYDROGRAPH GENERATION AND DOWNSTREAM HYDRAULIC ROUTING** **One-Dimensional Models** | | | | | | | |
| NRCS TR-66[1] | ✓ | | ✓ | ✓ | ✓ | ✓ | |
| WinDAM | | ✓ | ✓ | | | | |
| NWS SMPDBK | ✓ | | | ✓ | | ✓ | |
| NWS FLDWAV | ✓ | ✓[2] | ✓ | | ✓ | ✓ | |
| USACE HEC-1 and HEC-HMS | ✓ | | ✓ | Downstream hydrologic routing[3] | | ✓ | |
| USACE HEC-RAS | ✓ | | ✓ | ✓ | ✓ | ✓ | |
| **Two-Dimensional Models** | | | | | | | |
| DSS-WISE | ✓ | | ✓ | | ✓ | | ✓ |
| FLO-2D© | ✓ | ✓ | ✓ | | ✓ | ✓ | ✓ |
| MIKE© FLOOD | ✓ | | ✓ | | ✓ | ✓ | ✓ |

[1] NRCS TR-66 may be used in conjunction with TR-60 to determine input breach parameters such as breach width and time to breach.

[2] NWS FLDWAV program is embedded with NWS BREACH to determine breach parameters internally.

[3] Routing of breach hydrography downstream using a hydrologic routing method.

FEMA provides a list of nationally accepted hydraulic models based on their memorandum titled *Policy for Accepting Numerical Models for Use in the NFIP* (FEMA, 2004c). The purpose of this memorandum is to clarify the procedures to follow for accepting numerical models for flood hazard mapping and adding them to the "Numerical Models Meeting the Minimum Requirement of the NFIP" list. The list of the FEMA-accepted hydraulic models includes both one-

dimensional and two-dimensional steady and unsteady-flow models and may be found at the flowing Web site: http://www.fema.gov/national-flood-insurance-program-flood-hazard-mapping/numerical-models-meeting-minimum-requirement-0. Although acceptable for flood hazard mapping, these nationally accepted models may not be appropriate for use in a dam breach study.

It is common practice to first estimate breach parameters through empirical equations and then to use another model to define the breach hydrograph. The breach hydrograph is then routed downstream using a one-dimensional or two-dimensional hydraulic model.

## 10.3   DAM BREACH HYDROGRAPH AND PEAK OUTFLOW GENERATION TOOLS

The most common methods for either breach hydrograph generation or dam breach peak outflow computation are discussed in this section. These models/methods do not include the capability of a hydraulic routing of the breach hydrograph downstream. The NWS BREACH model is no longer supported by the NWS. The applicability, strengths, limitations, and governing equations applicable to available methods for breach hydrograph generation are summarized in Table 10-2.

**Table 10-2: Methods for Breach Hydrograph Generation Only**

| Method | Applicability | Strengths | Limitations | Governing Equations |
|--------|---------------|-----------|-------------|---------------------|
| Empirical Equations | Earthen dam failure | Fast and simple to use | Large potential level of error; suitable for Tier 1 studies | Empirical relationships derived based on analysis of historical dam failures |
| | | Minimal input data | | |
| WinDAM B | Earthen dam failure overtopping breach | Models headcutting and downstream tailwater effects, and evaluates erosion in spillways using SITES technology | Limited to homogeneous embankments with simple embankment geometry. Level pool routing may not be applicable to some reservoirs. | Stress-based, energy-based headcutting equations available as analysis options. Integrity analysis for vegetation equations for riprap surface protection are based on threshold concepts for rock on steep slopes. The outflow hydrograph is obtained through a time-stepped solution. |
| | | Physically based model using erosion and sediment transport principals | | |
| NWS-BREACH | Since 2005, the model source code has not been supported by the NWS. | | | |
| USACE HEC-HMS | Concrete and earthen dam failure | Ease of program use | Level pool routing is not applicable to some reservoirs. | Continuity equation and an analytical or empirical relationship between reservoir/reach storage and discharge |
| | | Inherently stable | | Level pool routing |

### 10.3.1 Empirical Equations for Estimating Peak Discharge

The relationships between dam breach parameters and the regression equations were developed from analyses of historical dam failures. Wahl (2004) conducted a literature review of breach parameter equations, including 16 peak breach outflow equations, which are regression relations that predict peak outflow as a function of various dam and/or reservoir parameters. Table 10-3 presents the level of error for each of the 16 empirical prediction equations evaluated in his study. These empirical equations are typically used for Tier 1 reconnaissance level studies and are rarely used in detailed breach assessments.

In 2010, Wahl discussed the application of empirical methods to calculate breach flow and concluded that the MacDonald and Langridge-Monopolis (1984), USBR (1988), Von Thun and Gillette (1990), and Froehlich (1995, 2008) methods are the most commonly used, empirically derived equations for predicting peak breach flow.

**Table 10-3: Empirical Equations for Estimating Peak Discharge and Associated Error**

| Reference | Equation | Number of case Studies | | Mean prediction error (local cycles) | Width of uncertainty band, $\pm 2S_e$ | Prediction interval around hypothetical predicted value of 1.0 |
| --- | --- | --- | --- | --- | --- | --- |
| | | Before outlier exclusion | After outlier exclusion | | | |
| **Peak flow equations** | | | | | | |
| Kirkpatrick (1977) | $Q_p = 12.68(h_w + 0.3)^{25}$ | 38 | 34 | −0.14 | ±0.69 | 0.28–6.8 |
| SCS (1981) | $Q_p = 16.6(h_w)^{1.85}$ | 38 | 32 | +0.13 | ±0.50 | 0.23–2.4 |
| Hagen (1982) | $Q_p = 0.54(S - h_d)^{0.5}$ | 31 | 30 | +0.43 | ±0.75 | 0.07–2.1 |
| Bureau of Reclamation (1982) | $Q_p = 19.1(h_w)^{1.85}$ envelope eq. | 38 | 32 | +0.19 | ±0.50 | 0.20–2.1 |
| Singh and Snorrason (1984) | $Q_p = 13.4(h_d)^{1.89}$ | 38 | 28 | +0.19 | ±0.46 | 0.23–1.9 |
| Singh and Snorrason (1984) | $Q_p = 1.776(S)^{0.47}$ | 35 | 34 | +0.17 | ±0.90 | 0.08–5.4 |
| MacDonald and Langridge-Monopolis (1984) | $Q_p = 1.154(V_w h_w)^{0.412}$ | 37 | 36 | +0.13 | ±0.70 | 0.15–3.7 |
| MacDonald and Langridge-Monopolis (1984) | $Q_p = 3.85(V_w h_w)^{0.411}$ envelope eq. | 37 | 36 | +0.64 | ±0.70 | 0.05–1.1 |
| Costa (1985) | $Q_p = 1.122(S)^{0.57}$ | 35 | 35 | +0.69 | ±1.02 | 0.02–2.1 |
| Costa (1985) | $Q_p = 0.981(S - h_d)^{0.42}$ | 31 | 30 | +0.05 | ±0.72 | 0.17–4.7 |
| Costa (1985) | $Q_p = 2.634(S - h_d)^{0.44}$ | 31 | 30 | +0.64 | ±0.72 | 0.04–1.22 |
| Evans (1986) | $Q_p = 0.72(V_w)^{0.53}$ | 39 | 39 | +0.29 | ±0.93 | 0.06–4.4 |
| Froechlich (1995) | $Q_p = 0.607(V_w^{0.295} h_w^{124})$ | 32 | 31 | −0.04 | ±032 | 0.53–2.3 |

| Reference | Equation | Number of case Studies | | Mean prediction error (local cycles) | Width of uncertainty band, $\pm 2S_e$ | Prediction interval around hypothetical predicted value of 1.0 |
|---|---|---|---|---|---|---|
| | | Before outlier exclusion | After outlier exclusion | | | |
| Walder and O'Connor (1997) | $Q_p$ Estimated by computational and graphical method using relative erodibility of dam and volume of reservoir | 22 | 21 | +0.13 | ±0.68 | 0.16–3.6 |

*Source: Wahl, (2004)*

## 10.3.2  WinDAM B

The ARS recently developed WinDam B in cooperation with the NRCS and Kansas State University, which expands on the capabilities of WinDam A. A description of the model is included in Section 10.1 of this document.

## 10.3.3  NWS BREACH Model

The NWS BREACH model was the first widely applied and most well-known, physically based model to predict the breach characteristics and the discharge hydrograph emanating from a breached earthen dam. Since 2005, the NWS has not supported code development; however, given the model's significance in dam breach studies and its ongoing use for some dam breach studies, a description of the model is included in this document.The model was initially developed in 1987 with updates in 1988, 1991, and 2005. The BREACH program is no longer supported by the NWS and is not available for download on the NWS Web site. It is still used because it is known to more accurately predict breach progression than other available methods and perhaps because it has not yet been replaced by another freely available, non-proprietary program that performs the same function.

BREACH couples the conservation of mass of the reservoir inflow, spillway outflow, and breach outflow with the sediment transport capacity of the unsteady uniform flow along an erosion-formed breach. The growth of the breach, as shown in Figure 10-1, is dependent on the dam's material properties and the assumed location of the downstream face of the dam. Sediment transport equations are used in the model to compute the rate of erosion and size of a breach based on supplied soil characteristics of the dam material and the inflow hydrograph. Enlargement of the breach is further evaluated by a sudden collapse due to excess hydrostatic pressure and breach width expansion by slope stability (Gee, 2010). The outflow hydrograph is obtained through a time-stepping solution.
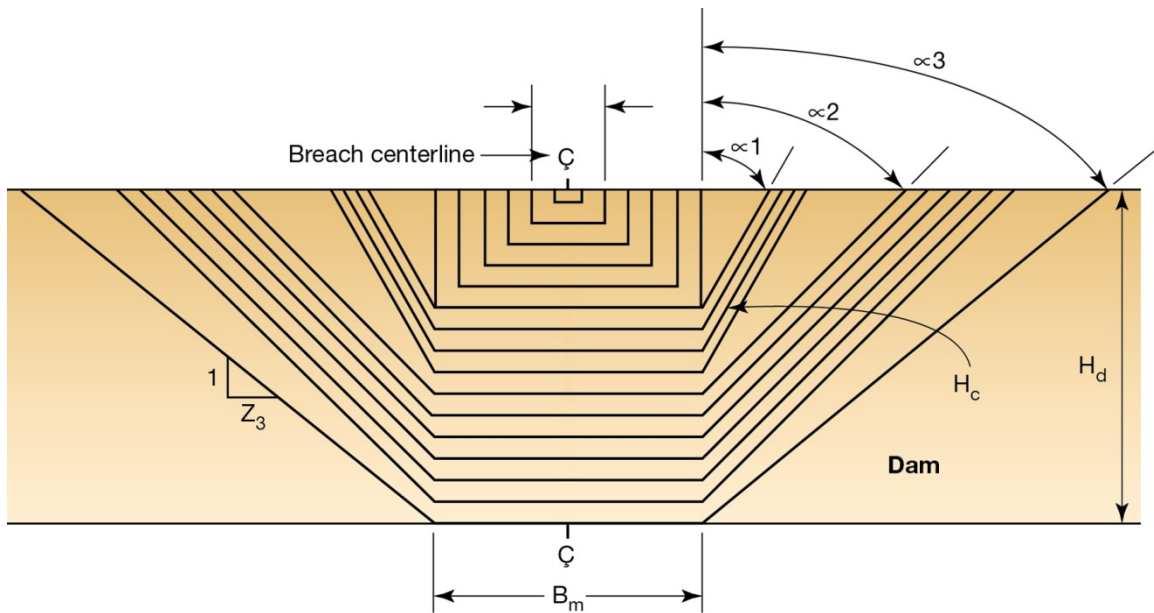
**Figure 10-1: NWS breach development scheme (Source: Gee, 2010)**

As documented in the BREACH Manual developed by Fread in 1991, the BREACH model considers the possible existence of the following complexities:

- Core material having properties that differ from those of the outer portions of the dam

- The necessity of forming an eroded ditch along the downstream face of the dam prior to the actual breach formation by the overtopping water

- The downstream face of the dam having a grass cover or being composed of a material of larger grain size than the outer portion of the dam

- Enlargement of the breach through the mechanism of one or more sudden structural collapses due to the hydrostatic pressure force exceeding the resisting shear and cohesive forces

- Enlargement of the breach width by slope stability theory

- Initiation of the breach via piping with subsequent progression to a free surface breach flow

- Erosion transport for either non-cohesive (granular) materials or cohesive (clay) materials

Wahl (2004) suggests that the BREACH model is constrained, as other similar models, in that it does not adequately model headcutting erosion processes that dominate the breaching of cohesive soil embankments. Another limitation of the BREACH model is that the breach hydrograph prediction is simulated without incorporating downstream effects, such as tailwater and dynamic effects on the flow within the upstream reservoir, because it uses level pool reservoir routing. This program may be used in conjunction with other programs to simulate downstream dynamic effects using the breach parameter results (i.e., breach width and

development time) as input into a separate flood routing model that can determine the breach hydrograph itself, while accounting for dynamic water-level effects of the reservoir and downstream tailwater effects (Fread, 1988; Wahl, 2010).

The NWS DAMBRK and FLDWAV software contain a BREACH subprogram that simulates piping and overtopping failures in earthen dams when users provide the typical dam and reservoir characteristics, thus generating breach parameters.

### 10.3.4 USACE HEC-HMS Program

HEC-HMS is a hydrologic modeling program typically used to conduct hydrologic simulations of the precipitation-runoff process of dendritic drainage basins. The program can also be used to perform dam failure analysis. HEC-HMS was developed by the USACE in 1992 to replace the HEC-1 program. The program has been updated several times since its initial release and the most current version of the HEC-HMS program can be found at the USACE's HEC's Web site at the following location: http://www.hec.usace.army.mil/software/hec-hms/.The following paragraphs have been adapted from HMS user support documents developed by the USACE.

In HEC-HMS, the user identifies ultimate breach parameters (i.e., breadth width, side slopes, time-of-failure) for dam breach simulations. Because the user defines the ultimate breach parameters, both earthen and concrete dam breaches may be simulated.

A dam breach simulation in HEC-HMS may be computed through two breach methods: overtopping or piping. For overtopping, the failure is simulated at a point on the top of the dam and expands in a trapezoidal shape until it reaches the maximum size input into the program. The piping dam breach function of HEC-HMS is used to simulate failures caused by piping inside an earthen dam. The failure begins with the water naturally seeping through the dam core until it increases in velocity and quantity enough to begin eroding fine sediments out of the soil matrix. The piping failure uses many of the same user-input parameters as the dam overtopping breach; however, it also requires the initial piping elevation and piping coefficient. The time growth curve may be specified in HEC-HMS as either linear, non-linear (sine wave), or user specified.

Similar to the precursor program HEC-1, HEC-HMS uses a level pool routing procedure for the upstream reservoir to estimate the breach hydrograph. The reservoir is represented as either a controlled or uncontrolled water body with the assumption of level pool and a monotonically increasing storage-outflow function. Hydrologic routing employs the continuity equation and an analytical or empirical relationship between reservoir/reach storage and the discharge. Output results from HEC-HMS include a resulting breach hydrograph that must be used in conjunction with other software, such as HEC-RAS, for downstream routing of the generated flood wave.

The main advantage of using HEC-HMS to simulate a dam failure is the ease of program use. The program does not suffer from the instability issues of its counterpart HEC-RAS. A major difference between HEC-HMS and HEC-RAS, , is that HEC-HMS uses level pool routing whereas HEC-RAS uses dynamic pool routing (full St. Venant equations of conservation of mass and conservation of momentum) for reservoir drawdown. However, dynamic routing requires detailed bathymetric data for the reservoir, which are frequently difficult and expensive to

obtain. Level pool routing, on the other hand, only requires a simple stage-storage curve for estimating reservoir drawdown. Goodell et al. (2009) argued that dynamic routing is generally a more accurate method for estimating reservoir drawdown. However, level-pool routing is often an adequate method for drawdown computation. This is especially true for small reservoirs that are roughly equal in length and width and do not have a considerably long fetch length.

## 10.4 BREACH HYDROGRAPH GENERATION AND DOWNSTREAM HYDRAULIC ROUTING TOOLS

This section addresses the most common tools that can perform both breach hydrograph generation and downstream routing of the associated breach hydrograph. These models are divided into one-dimensional and two-dimensional models.

### 10.4.1 Advantages and Limitations of One-Dimensional and Two-Dimensional Models

Hydraulic modeling of a dam breach has traditionally been completed using one-dimensional flow equations. One-dimensional models solve either fully dynamic or simplified forms of conservative or non-conservative forms of one-dimensional, cross-section-averaged shallow water equations. One-dimensional models provide reliable results for many situations; however, one-dimensional models in unconfined floodplains do not accurately represent the breach flood wave moving downstream. Two-dimensional models capable of solving two-dimensional, shallow water equations are more widely used. With recent advancements in the speed of computing capability, the use of two-dimensional models for dam failure studies has grown.

One-dimensional models are best suited to geographic regions with moderate to steep slopes whereby floodwaters are contained within a relatively narrow floodplain and generally flow in the direction of a single stream line without major or frequent divergence of flow. Confined floodplains are found in most parts of the United States and can range from deep entrenched rivers located within mountain ranges to wide coastal plain rivers. Confined floodplains typically present the engineer with a one-dimensional problem for which the one-dimensional assumption of many hydraulic models can yield meaningful results.

One-dimensional models only provide a depth and discharge at computational cross-sections along the river. Although this process may be appropriate for confined alluvial floodplains/channels, errors in the unsteady simulation may be introduced in unconfined flat areas. Two-dimensional models, and coupled one- and two-dimensional models, have the capability to route both channel flow (one-dimensional) and overland flow on flat terrain (two-dimensional).

When a flood wave enters an unconfined floodplain, one-dimensional routing is no longer a valid assumption. Geographic regions with flat to mild slopes, areas of depressed terrain, poorly defined flow paths, alluvial fans, and fluvial areas typically exhibit unconfined floodplains whereby floodwaters are not contained within a well-defined floodplain and generally flow in multiple directions, often with frequently diverging and converging flows. Unconfined floodplains are highly unpredictable and can exhibit both deep and shallow flooding with

significant lateral differences in water surface elevations. Unconfined floodplains are found in areas ranging from flat swampy areas and coastal regions to mountain valleys.

In flat areas, the results of a dam break are likely to be largely influenced by the location of the breach because the flat terrain has the potential to allow dam break floodwaters to flow in many directions without being confined to a river valley.

Areas below a dam may exhibit both confined and unconfined floodplains. An example of this would be a dam located along a mountain river that flows out into a large valley via an alluvial fan. The mountain river may exhibit a steep, deeply entrenched confined floodplain until it reaches the alluvial fan upon which the slope of the river decreases suddenly and becomes a very unpredictable unconfined floodplain as it diverges and spreads out across a wide alluvial fan and continues into a wide fluvial floodplain. Some models incorporate both one-dimensional and two-dimensional capabilities to be used in different situations.

### 10.4.2  One-Dimensional Models

Table 10-4 lists the most widely used one-dimensional hydraulic models for dam breach simulation and downstream hydraulic routing of the flood wave. Table 10-4 also provides a summary of the application, strengths, limitations and governing equations, of each model.

#### *10.4.2.1    NRCS TR-66 Simplified Dam Breach Routing*

TR-66 (1985) is a computer-based program developed by the USDA's SCS, now known as the NRCS. This technical release presents a method for estimating peak flood flow and the travel time of the flood wave moving downstream in the floodplain. The TR-66 program and User's Manual is available from the following NRCS Web site (under "Other Models"): http://www.wsi.nrcs.usda.gov/products/W2Q/H&H/Tools_Models/tool.

Although TR-66 is available on the NRCS Web site, it is no longer technically supported by NRCS and has not been updated since 1985. However, TR-66 remains in use across the United States.

The peak breach flow in the TR-66 program is determined by methods that are process based (using scientific procedures for erosion, sediment transport, and hydraulics) or based on empirical relationships derived from analysis of recorded dam failure data. The breach hydrograph, as all hydrographs, is completely defined by its peak discharge, its total volume, and its shape. Regarding shape, the method postulates that the breach hydrograph is a continuous decaying function of time, either triangular or curvilinear (exponential) in shape. The user must select an applicable breach hydrograph for a given situation based on the anticipated flow regime in the valley immediately below the dam.

**Table 10-4: Summary of One-Dimensional Models**

| Model | Application | Strengths | Limitations | Governing Equations |
|---|---|---|---|---|
| NRCS TR-66 | Dams with small height and storage<br>Reconnaissance-level work<br>Value of the parameter k* is less than or equal to 1.0 | Fast and easy to use | Provided but no longer supported by NRCS | ***Peak breach***<br>Process-based methods or empirical relationships<br>***Hydraulic routing***<br>A simplified form of the Att-Kin routing method |
| NRCS SITES | Analysis of erosion in an earthen and vegetative spillway<br>To determine discharge capacity of the principal and auxiliary spillway | | Does not consider full erosion/failure of an embankment dam | |
| WinDAM B | Analysis of erosion in an earthen and vegetative spillway to determine the discharge capacity of the principal and auxiliary spillway<br>Analyzes overtopping erosional breach using physical parameters | Erosion estimation based on geotechnical input parameters and condition of vegetation | Does not consider breach flow through erosion/failure of the auxiliary spillway | ***Routing***<br>Does not route breach hydrograph downstream and uses level pool routing for dam breach simulation. |
| NWS SMPDBK | For use in emergency situations | Fast and easy to use | Not a nationally accepted, FEMA-supported hydraulic model<br>Neglects backwater effects | ***Routing***<br>Dimensionless curves distinguished by the ratio of the volume in the reservoir to the average flow volume in the downstream channel governed by the Froude number<br>***Travel time of the peak flow***<br>Kinematic wave (steady-state) velocity |

**Table 10-4: Summary of One-Dimensional Models**

| Model | Application | Strengths | Limitations | Governing Equations |
|---|---|---|---|---|
| NWS FLDWAV | Breach analysis for fair weather piping/internal erosion and overtopping breaches<br><br>Can analyze flows in mixed-flow regimes in a system of interconnected waterways | Considers effects of downstream obstructions such as backwater effects | Calibration is time consuming<br><br>Not adequate for all complex river conditions | ***Routing***<br>One-dimensional St. Venant equations<br>User selects implicit dynamic wave, explicit dynamic wave, implicit diffusion wave, or level pool solutions of the St. Venant equations of unsteady flow |
| USACE HEC-1 | | | No longer supported by USACE; replaced by HEC-HMS | ***Routing***<br>Steady-state theory<br>***Dam-break Simulation***<br>Level pool routing |
| USACE HEC-RAS | Recommended for detailed analysis and routing of the breach hydrograph | Considers effects of downstream obstructions such as backwater effects<br>Output data can be input into GIS to produce inundation maps<br>Allows dynamic reservoir routing | Labor intensive and time consuming<br>Instability problems may arise | Governing equations vary depending on the assigned function. HEC-RAS can perform four functions:<br>Steady flow routing<br>Unsteady-flow routing<br>Movable boundary flow for sediment transport analysis<br>Water quality analysis |
| FEMA Geo-DamBREACH Toolset | Simplified method to be used in initial analysis and non-regulatory studies | Simple and quick to use | Only conducts fair weather breach analysis | Uses principles established in the NWS SMPDBK<br>***Routing***<br>Dimensionless curves distinguished by the ratio of the volume in the reservoir to the average flow volume in the downstream channel governed by the Froude number<br>***Travel time of the peak flow***<br>Kinematic wave (steady-state) velocity |

If the downstream valley is expected to be supercritical flow, the TR-66 User's Manual suggests a triangular hydrograph shape. If subcritical or critical flow is expected, the appropriate hydrograph shape is curvilinear. The rationale is that if the flow in the reach immediately below the breached dam is subcritical, the tailwater will submerge the breach at some outflow, thereby retarding the total flow from the breach.

Hydraulic routing of the breach flood wave downstream is completed using a simplified form of the Att-Kin method, which was developed by the NRCS in the late 1970s. The Att-Kin method divides routing into two sequential steps: the first step provides reservoir attenuation, and the second step provides pure kinematic translation. The Att-Kin method routes an inflow hydrograph through the reach using a storage indication method to solve the integral form of the conservation of mass equation. The method then positions the peak in time and distorts the storage-routed hydrograph using a kinematic model. The kinematic routing solves the differential form of the conservation of mass equation using a single-valued flow area-discharge relationship. The inflow hydrograph and kinematic model are completed in series as a linear combination of the storage and kinematic models until the outflow hydrograph satisfies the conservation of mass equation at the time to peak of the outflow hydrograph. The storage routing provides attenuation but does not describe translation; the kinematic routing provides translation and distortion but does not attenuate the peak (SCS, 1985; Comer et. al., 1982).

In the simplified Att-Kin method, the storage-discharge curve is normally used to represent the momentum equation. However, the TR-66 Manual notes that caution should be applied to use of the simplified Att-Kin form when the exponent in the discharge-valley storage relationship exceeds its standard specified limits. The User's Manual states the accuracy of the method has been evaluated through a comparison test of its results with data generated under slightly modified conditions by the NWS DAMBRK model. Even though limited in scope, the test confirmed the predicted tendency of the simplified Att-Kin model, consistent with the dry-bed assumption, of a higher than normal rate of attenuation of the peak discharge with distance. Confirmation of suspected questionable behavior suggested limiting the method's application to situations in which the value of the parameter k* [coefficient in the discharge-valley storage relationship] is smaller than or equal to 1.0. In the domain beyond that limit, the method may be used with caution, that is, with the understanding that predicted values are good only for qualitative assessments of potential hazards.

Several studies have been conducted comparing the Att-Kin and modified Att-Kin routing procedures with the Muskingum-Cunge method. Ponce, et al. (1996) and Merkel (2002) suggest that the Muskingum-Cunge method is the most reliable hydrologic channel routing method, showing stability, convergence, and consistency (i.e., grid independent) when used within its recommended parameter ranges, because it simulates the diffusion wave model, not the kinematic wave model (Ponce and Simons, 1977).

Later forms of hydrologic/hydraulic models developed by the NRCS, such as the TR-20 computer program, incorporated a modified form of the Att-Kin method for valley floor flood routing. In more recent developments, the Muskingum-Cunge flood routing procedure has replaced the modified Att-Kin method in NRCS technical releases. However, TR-66 has not been updated to include a modified flood routing procedure.

### 10.4.2.2    SITES and WinDAM B Models

The Water Resources Site Analysis Program (SITES) model, developed as a collaborative effort by the ARS, NRCS, and Kansas State University, is an earthen/vegetated auxiliary spillway

erosion prediction model for dams. The SITES model is not a dam failure model tool. It is intended to be used to evaluate the erosion potential of materials in the auxiliary spillway, but does not produce a breach outflow hydrograph.Without consideration of the potential for embankment failure, the SITES model is typically used for dam assessments and dam design and not for dam breach modeling. The most current version of the SITES model is available for download at the following NRCS Web site: http://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national//?&cid=stelprdb1042517.

The WinDAM B model was developed to evaluate dams for overtopping and breach. A description of the WinDAM B model is included in Section 10.1 of this document. The WinDAM B model is available for download at the following NRCS Web site: http://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/water/manage/?cid=stelprdb1045404#downloadSoftware.

### 10.4.2.3    NWS SMPDBK Simplified Dam Break Model

The SMPDBK model was developed by NWS in 1984, and last updated for public use in 1991, to predict downstream flooding produced by a dam failure. The model is internally supported by the NWS and has been converted to a GIS-based version called Geo-SMPDBK for use at the NWS River Forecast Centers. This model produces the basic information needed to determine flood inundation areas of a dam failure while substantially reducing the amount of time, data, computer facilities, and technical expertise required to employ more sophisticated unsteady-flow routing models, such as FLDWAV. Although no longer technically supported for public use by the NWS, the SMPDBK program may be downloaded from the following location: http://www.rivermechanics.net/downloads.htm. The following discussion on the NWS SMPDBK model is adapted from NWS user support documents. The model consists of three major steps:

1. Approximation of the downstream channel as a prismatic channel

2. Calculation of the peak outflow discharge and stage produced by time-dependent, rectangular-shaped breach of the dam using a temporal and geometrical decision of the breach and reservoir volume

3. Calculation of the dimensionless routing parameters used with dimensionless graphs to route the peak outflow and time to peak at selected downstream locations

The SMPDBK model can easily be used on a personal computer with a minimal amount of data. SMPDBK can produce approximate flood forecasts after adding only the dam height, reservoir storage volume, and depth vs. width area for one cross-section of the downstream river valley. Should additional information be entered, the model utilizes the information to enhance the accuracy of the forecast (Wetmore et al., 1991). Input for the simplified model may include:

- River characteristics
  - ➢ Cross-sectional information
  - ➢ Slope

> ➢ Manning's n for the river valley

- Reservoir storage volume at the time of failure

- Breach parameters

  > ➢ Final breach width

  > ➢ Final breach depth

  > ➢ Breach formation time

  > ➢ The distance from the dam to the downstream points of interest

The model allows for the investigation of partial and complete failures occurring over a finite interval of time. The breach geometry analyzed may be either trapezoidal or rectangular in shape. Failures due to overtopping of the dam and/or failures due to piping/internal erosion may be analyzed by specifying appropriate parameters for the elevation of the breach formation.

SMPDBK uses the same basic components as the DAMBRK model (the predecessor of FLDWAV), but neglects the effects of off-channel storage, concerning itself with only peak flows, stage, and travel times. Backwater effects for downstream constrictions, such as bridges and dams, are also neglected in computations. Routing is achieved by employing dimensionless curves distinguished by the ratio of the volume in the reservoir to the average flow volume in the downstream channel governed by the Froude number developed as the flood wave moves downstream. The travel time of the peak flow is computed using the kinematic wave (steady-state) velocity, which is a known function of the average flow velocity throughout the routing reach.

A study completed by Wetmore and Fread (1981) suggests that the simplified model generally produces errors of less than 10 percent. However, understanding the model's limitations is important. In a 1991 study, Fread et al. discussed the following limitations:

> First, as with all dam-break flood routing models, the validity of the SMPDBK model's prediction depends upon the accuracy of the required input data, whether these data are supplied by the user or provided as default "most probable" values by the model. Secondly, because the model assumes normal, steady flow at the peak, the backwater effects created by downstream channel constrictions such as bridges with their embankments or dams cannot be taken into account. Under these conditions, the model will predict peak flood elevations upstream of the constriction that may be substantially lower than those actually encountered, while peak flood elevations downstream of the constriction may be somewhat over-predicted.

The SMPDBK program may be combined with an external interface to automate the modeling process for GIS overlay and map production. The external interface saves the model data to a properly formatted input file for SMPDBK and then launches the executable. The executable automatically reads the results and creates a water surface elevation data set that can be used for automated floodplain delineation.

### 10.4.2.4    *FEMA Geospatial Dam Break, Emergency Action Planning, Consequences, and Hazards Toolset*

FEMA, with the support of the NWS, recently developed Geospatial Dam Break, Emergency Action Planning, Consequences, and Hazards (GeoDamBREACH) toolset to provide dam owners with a simplified tool to produce dam breach EAPs and to create FEMA Risk Mapping, Assessment, and Planning (Risk MAP) products for dams. By simplifying the development of consistent EAPs, FEMA's goal is to reduce the cost of a basic EAP, thereby increasing the number of EAPs produced through State Dam Safety Programs. In the process, FEMA hopes to leverage available breach inundation maps for risk communication purposes through Risk MAP.

The GeoDamBREACH toolset includes a user-friendly preprocessor interface to a semi-automated EAP report generator based on the NRCS fillable form EAP template in an effort to reduce the cost of producing a basic EAP. GeoDamBREACH also provides tools to assist dam owners reduce the cost of developing standardized EAP maps by providing an automated paneling scheme and map panel creation and annotation software. Dam breach inundation zone model results are aligned to the EAP report and maps, further reducing costs.

The GeoDamBREACH toolset includes an automated GIS-based simplified dam breach inundation mapping tool based on the NWS SMPDBK program and the option to import a dam breach inundation polygon developed using another program. GeoDamBREACH also includes an automated loss-of-life tool, based on the methodology described in *Estimating Loss of Life for Dam Failure Scenarios* (DHS, 2011) as a module of the toolset.

To support FEMA's Risk MAP program, GeoDamBREACH automated the creation of Risk MAP non-regulatory product datasets, including FEMA's Hazards United States Multi-Hazard (HAZUS) compatible inundation depth grids, arrival time grids, de-flood time grids, and velocity grids. The non-regulatory products are designed to provide local communities with digital information they can use for risk communication purposes and for use in dam breach consequence assessments and enhanced hazard mitigation studies.

### 10.4.2.5    *NWS FLDWV Model*

The NWS FLDWAV program is a generalized flood routing program for the solution of fully dynamic equations of motion for one-dimensional flow. FLDWAV was first released in November 1998 and replaced the NWS generalized flood routing programs, DAMBRK (released in 1988) and DWOPER (released in 1984). The NWS is in the process of transitioning from FLDWAV to HEC-RAS "because it will not be cost effective to continue supporting two very similar hydraulic models (FLDWAV and HEC-RAS) in operations" (NWS, 2009b).

The following discussion on FLDWAV is adapted from the NWS user support documents. The FLDWAV model and user support documents are available for download at the following location: http://www.rivermechanics.net/downloads.htm.

FLDWAV has all of the features found in the SMPBDK model with expanded capabilities. The program is capable of performing a breach analysis for fair weather piping/internal erosion failures as well as overtopping breaches. In addition, the program has the ability to analyze flows

in mixed-flow regimes in a system of interconnected waterways by routing the outflow hydrograph hydraulically through downstream river/valley system using an expanded form of the one-dimensional St. Venant equation. The program considers the effects of downstream dams, bridges, levees, tributaries, off-channel storage areas, river sinuosity, and backwater. Another advanced feature of FLDWAV is that it can automatically provide linearly interpolated cross-sections at a user-specified spatial resolution to increase the spatial frequency at which solutions are obtained.

To produce acceptable forecasts using FLDWAV, the model should be calibrated to observed stages at gages within the study area. The program includes an automatic calibration feature. FLDWAV may be used for Newtonian (water) or non-Newtonian (mud/debris) fluids. Non-Newtonian fluids are modeled using a technique that determines the friction slope of mud/debris flows based on a semi-empirical rheological power-law equation and a wave-front tracking technique (Ming and Fread, 1997).

Additional capabilities of FLDWAV include: 1) the ability to dynamically model dam failures as well as flows that are affected by bridge constrictions; 2) the ability to simulate flows that overtop and crevasse levees located along either or both sides of a main stem and/or its principal tributaries; and 3) the ability to handle flows in the subcritical and/or supercritical flow regime. User-specified dam characteristics and a description of the reservoir (cross-sections or storage-elevation curve) enable FLDWAV to compute the breach outflow hydrograph. The breach outflow is computed by the principles of soil mechanics, hydraulics, and sediment transport to simulate the erosion and bank collapse processes for the breach. Reservoir inflow, storage, and spillway characteristics, along with the geometrical and material properties of the dam (median grain size, cohesion, internal friction angle, porosity, and unit weight) are used to predict the breach hydrograph.

FLDWAV assumes the breach develops over a finite interval of time and will have a final size determined by a terminal bottom width parameter and various shapes depending on the breach side slope. Such a parametric representation of the breach is used in FLDWAV for simplicity, generality, wide applicability, and the uncertainty in the actual failure mechanism. The model assumes the breach bottom width starts at a point either on top of the dam for overtopping failure or at a specified point for piping/internal erosion failure. The starting point enlarges at a linear or nonlinear rate over the failure time until the terminal bottom width is attained and the breach bottom has eroded to the terminal elevation (Fread and Lewis, 1998). The governing equations of the model are the expanded one-dimensional equations of unsteady flow derived by St. Venant. A four-point, implicit finite-difference numerical solution of the complete one-dimensional St. Venant equations of unsteady flow and appropriate external and internal boundary equations form the basis for the floodplains simulated by FLDWAV. The boundary conditions supported by FLDWAV include dams, bridges, weirs, waterfalls, and other manmade and natural flow controls. In addition, FLDWAV allows the user to select implicit dynamic wave, explicit dynamic wave, implicit diffusion wave, or level pool solutions of the St. Venant equations of unsteady flow.

The information necessary to execute an analysis in FLDWAV includes an upstream stage or discharge hydrograph; a downstream boundary condition; downstream cross-section geometry; information about downstream hydraulic structures (dams, bridges, levees); hydraulic roughness coefficients along the waterway; and the initial flow depth and quantity (base flow) at each cross-section location. Given this information, FLDWAV simultaneously solves for depth and flow along the routing reach for each time interval during the specified simulation period (Sylvestre and Sylvestre, n.d.).

In a 2006 NWS study, the FLDWAV program capabilities were evaluated relative to other widely used hydraulic models, such as the USACE's HEC-RAS. This evaluation noted that in some instances "FLDWAV modeling capabilities are not adequate for all complex conditions that exist in rivers, that the model lacks the tools for calibration, and that model calibration is a very time consuming, labor intensive and highly inefficient process." The NWS study group recommended integrating HEC-RAS into the NWS operational forecasting environment and the NWS Office of Hydrologic Development set a goal to fully transition away from the use of FLDWAV and only include HEC-RAS in the first release of the Community Hydrologic Prediction System. This decision was based on the following assumptions: (1) HEC-RAS can provide equivalent functionality capable of replacing existing FLDWAV and Dynamic Wave Operational (DWOPER) models (with relatively minor HEC-RAS enhancements), and (2) through collaboration with HEC, maintaining and enhancing HEC-RAS alone will be more economical than independently maintaining HEC-RAS, DWOPER, and FLDWAV (Reed et al., n.d.). A full transition from FLDWAV to HEC-RAS has not been completed by the NWS to date; however, the industry standard for dam breach analysis has moved away from the FLDWAV program to other one- and two-dimensional hydraulic routing programs.

### 10.4.2.6    USACE HEC-1 Program

The USACE's HEC-1 program was first developed in 1968 and last updated in 1998, after which it was replaced by the hydrologic modeling software HEC-HMS, developed in 1992. Although, the HEC-1 has been superseded, some Federal documentation still references the use of the HEC-1 model for dam failure analysis, primarily because most of these documents pre-date the common application of the USACE HEC-HMS or HEC-RAS programs for dam failure simulation. For this reason, a discussion of HEC-1 is included within this document. The HEC-1 Program V. 4.1 and User's Manual can be found under the USACE's legacy software section located at the following Web site:
http://www.hec.usace.army.mil/software/legacysoftware/hec1/hec1-download.htm.

The program includes a dam safety analysis capability that uses simplified hydraulic techniques to estimate the potential for and consequences of dam overtopping or structural failures on downstream areas in a floodplain. A dam failure analysis has two main components: the reservoir component and the dam safety simulation component. The reservoir component is employed in a stream network model to simulate a dam failure. Most of the modeling effort is characterizing the inflows to the dam under investigation, specifying the characteristics of the dam failure, and routing the dam failure hydrograph to a desired location in the downstream

floodplain. The dam safety simulation differs from reservoir routing in that the elevation-outflow relation is computed by determining the flow over the top of the dam (dam overtopping) and/or through the dam breach (piping/internal erosion), as well as through other reservoir outlet works. The elevation-outflow characteristics are then combined with the level pool storage routing to simulate a dam failure.

A dam breach is simulated in the HEC-1 program using the methodology incorporated by Fread in the NWS DAMBRK program (Fread, 1979). Structural failures are modeled by assuming certain geometrical shapes for the dam breach. The outflow from a dam breach may be reduced by backwater from downstream constrictions or other flow resistances. HEC-1 allows a tailwater rating curve or a single cross-section (and a calculated normal-depth rating curve) to be used to reflect such flow resistance. Submergence effects are calculated in the same manner as in DAMBRK. The dam-break simulation assumes that the reservoir pool remains level and routes the flood wave downstream using steady-state theory (USACE, 1998).

### 10.4.2.7    USACE HEC-RAS Program

The USACE HEC-RAS program released in 1995 is a one-dimensional steady- and unsteady-flow modeling program. The current version of the program can perform four functions: (1) steady-flow routing, (2) unsteady-flow routing, (3) movable-boundary flow for sediment transport analysis, and (4) water quality analysis.

The following discussion on HEC-RAS is adapted from user support documents developed by the USACE. HEC-RAS and use documentation are available for download at the following Web site: http://www.hec.usace.army.mil/software/hec-ras/hecras-download.html.

The steady-flow component of the modeling system uses a standard step method intended for the solution of water surface profiles for steady, gradually varied flow. The basic computations are based on the one-dimensional energy equation in which energy losses are evaluated by friction and contraction/expansion of the channel. The momentum equation may be used when the water surface profile is rapidly varied in conditions such as a mixed flow regime. The system can handle a full network of channels, a dendritic system, or a single river reach. The steady-flow component is capable of modeling subcritical, supercritical, and mixed flow regime water surface profiles. To perform a steady-state analysis for routing a resulting breach flow downstream in HEC-RAS, an upstream boundary condition must be provided in the model. This boundary condition is the peak outflow generated from the breach hydrograph that has been determined externally, in such forms as HEC-HMS, NWS BREACH, or an empirical equation.

The unsteady component of the HEC-RAS modeling simulates one-dimensional unsteady flow and can perform subcritical, supercritical or mixed flow regime computations. The governing equations for unsteady flow are the conservation of mass (continuity) and momentum equations derived from the full equations of motion (St. Venant equations). Upstream boundary conditions typically consist of an inflow hydrograph from the upstream watershed into a defined reservoir. For a dam breach analysis, the reservoir outflow is dynamically routed downstream.

Failure modes integrated into the HEC-RAS model include overtopping and piping. Additional failure modes may be approximated with variations to one of those two methods. Overtopping failures start at the top of the dam while a piping failure can start at a specified elevation/location and grow to the maximum specified extents. Breach parameters, such as breach width, depth, side slopes, and development time are estimated external to the model. Values for the breach size and development time are needed to produce a reliable estimate of the outflow hydrographs and resulting downstream inundation areas.

In HEC-RAS, both steady-state and unsteady-flow analysis use the same set of geometric data. This geometric data includes the reservoir storage volume, dam and downstream channel characteristics, cross-sectional data, etc. Differences in results between these two routing methods are a result of the computation procedures and inclusion of flow attenuation in unsteady-flow routing. The ASPFM has noted a generally small computational difference of 0.1 to 1 foot between steady and unsteady-flow analysis based on hypothetical event analysis (Altinakar, 2008). Further suggesting that while the difference between the two methods can be outside of this specified range, these differences do not necessarily mean that unsteady flow is more accurate than steady flow. The ASPFM has identified three key features between the steady-state and unsteady flow that provide computation differences:

1. **Losses**: Steady-flow losses computations use absolute differences in velocity head at adjacent cross-sections multiplied by an expansion or contraction coefficient, whereas unsteady-flow loss computations are computed by the momentum equation.

2. **Friction Slope**: Average friction slope between cross-sections is determined by averaging the conveyance method for steady flow. For unsteady flow, the average friction slope between cross-sections is computed directly from a simple average of the computed friction slopes.

3. **Discharge**: Steady-flow computations compute losses through downstream obstructions, such as culverts and bridges, directly from the obstruction geometry and the type of flow conditions through the structure. In unsteady flow, a family of curves is developed for defining the headwater-tailwater-discharge relationships through each obstruction for a full range of flow.

HEC-RAS can perform inundation mapping of water surface profile results directly using the RAS Mapper or the external HEC-GeoRAS tool. Using the HEC-RAS geometry and computed water surface profiles, RAS Mapper creates an inundation depth and floodplain boundary dataset. Additional geospatial data can be generated for analysis of velocity, shear stress, stream power, ice thickness, and floodway encroachment data. HEC-GeoRAS is a set of GIS tools that prepare the geometric date for import into HEC-RAS and generate the flood inundation data from the HEC-RAS output. Figure 10-2 shows typical HEC-RAS model output.
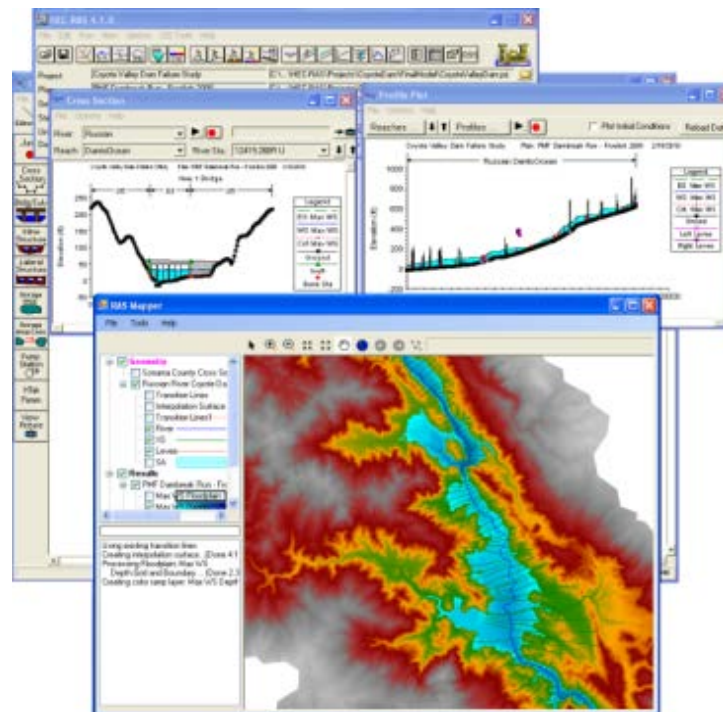
*Source: UASCE (2012)*

**Figure 10-2: Example of HEC-RAS modeling program output showing cross sections, flood profile, and floodplain map.**

## 10.4.3  Two-Dimensional Models

One-dimensional models include mathematical simplifications related to the assumption that flood depth remains uniform over the entire cross-section. This assumption is not accurate for wide and flat floodplain areas. Two-dimensional models, use full dynamic or simplified forms of one- and two-dimensional shallow water equations to solve both one-dimensional channel flow and two-dimensional overland flow and are more appropriate for flat and wide floodplain areas..

Several two-dimensional flow models are available for hydraulic modeling and to route dam breach flood waves through downstream channels and floodplains. Currently, the most commonly used two-dimensional models for dam breach studies include the DHI MIKE$^{©}$ software and FLO-2D$^{©}$ software. Recently, DHS and the USACE incorporated a simplified version of the two-dimensional model DSS-WISE into DSAT.

Proprietary two-dimensional models developed by or supported by Aquavelo, HR Wallingford$^{©}$, DHI$^{©}$, and XP-SWMM$^{©}$ are currently in limited use; however, these models are expected to become more widely used for dam breach modeling.

The strengths, limitations, governing equations, of the most widely used two-dimensional dam breach models are summarized in Table 10-5.

**Table 10-5: Summary of Widely Used Two-Dimensional Dam Breach Models**

| Model | Applicability | Strengths | Limitations | Governing Equations |
|---|---|---|---|---|
| DSS-WISE | Available as a component of DHS DSAT | Two-dimensional routing | DSS-WISE version in DSAT has default national datasets and limited user interactive capability | Full shallow water equations |
| FLO-2D© | Detailed analysis and routing of the breach hydrograph<br>Confined and unconfined floodplains, alluvial fans, urban areas, and coastal areas | Two-dimensional routing | May not be appropriate for areas where water has ponded or if the water surface is very flat (FLO-2D, 2009) | Dynamic wave momentum equation<br>Continuity equation<br>Momentum equation |
| MIKE FLOOD© and MIKE 21© | Detailed analysis and routing of the breach hydrograph<br>Confined and unconfined floodplains, alluvial fans, urban areas, and coastal areas | Option between one-dimensional and two-dimensional routing | | Varies depending on which integrated model is used (MOUSE, MIKE 11, or MIKE 21) |
| XP2D | Detailed analysis and routing of the breach hydrograph with confined and unconfined floodplains, alluvial fans, urban areas and coastal areas | Fully integrated one- and two-dimensional routing | | Full dynamic one-dimensional St. Venant's equation; full dynamic two-dimensional shallow water equations |

## 10.4.3.1  DSS-WISE

DSS-WISE was developed by the NCCHE at the University of Mississippi with funding from the DHS Science and Technology Directorate under the South East Region Research Initiative Program and was monitored by the Oak Ridge National Laboratory.

DSS-WISE is a product of the CCHE2D-FLOOD model previously developed by the NCCHE. Accessible via DSAT, a streamlined version of the full the DSS-WISE model requires a dam owner to provide minimum information about a target dam and produces a dam breach inundation zone within minutes and without human intervention. This is achievable because the DSS-WISE tool runs on a server that contains default data required to produce a dam breach model, including data from the NID, the National Bridge Inventory, 10-meter-resolution DEM data for the entire United States, and the National Land Cover Dataset 2006 (MRLC, 2006).

### 10.4.3.2    FLO-2D Program

The FLO-2D[©] software program, provided by FLO-2D Software, Inc., a privately owned corporation, is a dynamic flood routing model that simulates channel flow, unconfined overland flow, and street flow. This software can simulate dam failure and subsequent flow in confined channels, unconfined floodplains, and urban areas in two dimensions (FLO-2D, n.d.). The most recent version of this program is available from the FLO-2D Web site:http://www.flo-2d.com/products/flo-2d/.

FLO-2D can simulate the following dam breach conditions: overtopping and the development of a breach channel, piping failure, piping and roof collapse and development of a breach channel, breach channel enlargements through side slope slumping, and breach enlargement by wedge collapse. The user has the option to specify ultimate breach parameters or initial breach elevation; if not provided, the program will determine breach parameters. The breach erosion component of the code is based on the NWS BREACH model with revisions (FLO-2D, 2009).

Flood routing is computed in two dimensions and the model uses volume conservation, the continuity equation, and the full dynamic wave momentum equation (FLO-2D, 2009). Routing is achieved by computing the solution over uniform, square grid elements. The discharge across each element boundary is calculated in eight potential directions. FLO-2D also has options to simulate sediment transport, sediment flows, and loss of storage due to buildings and flow obstructions (FLO-2D, 2009).

### 10.4.3.3    MIKE FLOOD and MIKE 21

MIKE FLOOD[©], by DHI Software (2007), is a one-dimensional and two-dimensional flood simulation software, enabling modeling of rivers, floodplains, floods in streets, drainage networks, coastal areas, dam and levee breaches, or any combination of the above. The program is a compilation of three models: MOUSE, MIKE 11, and MIKE 21. MOUSE is a one-dimensional program designed for urban environments; MIKE 11 is optimal for one-dimensional river modeling; and MIKE 21 provides two-dimensional modeling capabilities. MIKE FLOOD allows the user to dynamically link between these programs. More information about MIKE FLOOD and MIKE 21 can be found on the DHI Web site: http://www.dhisoftware.com/Products/WaterResources/MIKEFLOOD.aspx.

This MIKE FLOOD program allows the user to reduce computational efforts by choosing the optimal model for different scenarios. For example, if a river can be modeled with one-dimensional code except in a few places, the user could choose to use MIKE 11 (with one-dimensional code) for the majority of the river and link to MIKE 21 (with two-dimensional code) only when appropriate. This facilitates a fast run time (associated with the one-dimensional code) without losing the ability to appropriately model areas that would be best modeled using two-dimensional routing.

### 10.4.3.4    XP-SWMM

XPSWMM 2D / XPStorm 2D$^{©}$ is an integrated one- and two-dimensional hydraulic flood modeling software package for the comprehensive analysis of storm water, sanitary, or combined systems, and river systems. It simulates natural rainfall-runoff processes and the hydraulic performance of drainage systems, allowing an integrated analysis of flow and pollutant transport in engineered and natural systems, including ponds, rivers, lakes, overland floodplains, and the interaction with groundwater.

The one-dimensional hydraulic engine solves the complete St. Venant (dynamic flow) equations for gradually varied, one-dimensional, unsteady flow throughout the drainage network. The calculation accurately models backwater effects, flow reversal, surcharging, pressure flow, tidal outfalls, and interconnected ponds. The model allows for looped networks, multiple outfalls, and pumps and accounts for storage in conduits.

The two-dimensional solution algorithm solves the full two-dimensional, depth-averaged, momentum and continuity equations for free-surface flow. The scheme includes the viscosity or sub-grid-scale turbulence term. It is specifically orientated towards establishing flow patterns in urban areas, floodplains, rivers, coastal waters, and estuaries where the flow patterns are essentially two-dimensional in nature and cannot be represented, or would be awkward to represent, using a one-dimensional network model.

The computational procedure used for the two-dimensional hydraulic analysis is an alternating direction implicit finite difference method based on the work of Stelling (1984). The method involves two stages, each with two steps. Each step involves solving a tri-diagonal matrix. The program also allows for direct rainfall in grid analyses along with transmission losses. XPSWMM 2D / XPStorm 2D couples the one-dimensional network flow with two-dimensional overland flow to accurately model interaction between flood waters and drainage systems, including underground pipes and natural channels.

Inundation resulting from dam breach may be modeled using various approaches. A breach outflow hydrograph may be used as direct input to the one-/two-dimensional model, should the user have an existing hydrograph or generate a hydrograph from another program. The breach hydrograph can also be developed within the program and used in conjunction with a one-dimensional node and real-time controls to define the breach development and timing to trigger flow from a storage area. The breach flows can be defined from a head/flow relationship boundary condition to discharge directly to the two-dimensional mesh. The breach flow can be simulated using a Dynamic Elevation Shape to alter the two-dimensional grid based on a trigger setting.

## 10.5    RECOMMENDATIONS FOR SELECTING MODELING SOFTWARE

The selection of an appropriate model for computing a dam breach is dependent on type of results needed, the level of effort that can be expended, and the potential for loss of life and economic damages that can result from a dam failure.

For dams in rural areas where the potential for loss of life is low, a tier 1 level study using simplified methods may be appropriate. For areas where a potential dam breach can result in the loss of life an intermediate tier 2 level or advanced tier 3 should be performed. The intermediate tier 2 level study may be used for areas where more detailed calculations are justified because of

the potential for loss of life. Advanced tier 3 level studies may be needed to develop dam breach inundation zone mapping for urbanized areas and for unconfined floodplains.

Section 10.6 summarizes guidance on a balanced tiered approach to dam breach modeling cross referenced to the most commonly used one- and two-dimensional model used for dam breach studies. Regulatory authorities should be consulted for allowable use of propriety non-Federal and two-dimensional models not listed in Table 10-6.

**Table 10-6: Recommended Model Types for Various Levels of Dam Breach Modeling**

| Tier Level | Applicable to | Peak Breach Discharge Prediction | Downstream Routing of Breach Hydrograph |
|---|---|---|---|
| **Tier 1-Screening and Simple Analysis (basic method)** | Low-hazard potential / small-size or first-level screening for significant or high-hazard dams | Regression equations, NWS SMPDBK, GeoDamBREACH or TR-66, HEC-HMS or DSAT | GeoDamBREACH, SMPDBK, HEC-RAS Steady State, HEC-HMS Hydrologic Routing, or DSAT |
| **Tier 2-Intermediate** | Significant-hazard potential / intermediate-size or high-hazard dams with limited population at risk | HEC-HMS, HEC-RAS Unsteady Model, DSAT or WinDAM | HEC-RAS (Steady or Unsteady Modeling) or Two-Dimensional Model for unconfined floodplains |
| **Tier 3-Advanced** | High-hazard potential/ large-size dams with sufficient population at risk to justify advanced analyses | HEC-HMS, HEC-RAS Unsteady Model, or WinDAM | HEC-RAS Unsteady Model or Two-Dimensional Model |

## 10.6    Terrain Data for Modeling

The USDOI in its report *Inundation Mapping/Modeling Subproject Report, Implementation Phase 1: Launch Risk Reduction*, dated March, 2011, recommends that as a general rule, larger dam breach flows, in terms of peak breach discharge and volume, do not require as detailed terrain data as would smaller flood flows. Smaller flood flow requires a more detailed definition of drainage paths, which creates a need for more accurate terrain data.

Before performing an inundation study, available terrain data sources should be carefully evaluated to identify the best-available data by considering numerous factors, including date, accuracy, and spatial extent of data to name a few. For instance, the selection of the level of detail and accuracy of the terrain data to be used for a dam breach study should consider the level of detail of the hydrologic and hydraulic modeling used for the study. A tier 1 simple and basic study, described in section 6.3 of this guidance document, covering a rural area does not warrant the same level of detailed terrain data as tier 2 intermediate or tier 3 advanced studies for heavily populated areas. Section 9 provides further discussion on terrain data and cross-section accuracy for use in dam breach studies.

To achieve a suitable level of accuracy, it may be necessary to combine multiple terrain datasets to ensure that either the best available or an acceptable accuracy terrain data source is utilized throughout the spatial extent of the inundation. When combining terrain datasets, edge matching

must be carefully reviewed to ensure that no artificial walls or drops are inadvertently created during the merge. Combined datasets must also use consistent horizontal and vertical datum and units.

Light Detection and Ranging (lidar) and Interferometric Synthetic Aperture Radar (IFSAR) have recently become the technologies of choice in mass production of Digital Elevation Models (DEMs), Digital Terrain Models (DTMs), and Triangulated Irregular Networks (TINs) used for floodplain mapping, including dam breach flood inundation studies.

IFSAR is a developing technology capable of capturing large geographic areas during a single acquisition flight thereby making it cost effective for developing DEM for large areas. IFSAR technology is not as suited to producing a bare earth model due to the complex scattering presented by buildings and manmade features in urbanized areas, and by vegetation and forest cover, including that along stream channels in both urban and undeveloped areas.

The vertical accuracy of IFSAR (1 to 2 meters) is less accurate than the accuracy of lidar-generated DEMs (measured in centimeters). However, the accuracy of a DEM generated by IFSAR may be appropriate for use in the modeling of dam breach flood inundation areas depending on the level of detail and accuracy required.

A readily available potential source of digital terrain data includes the USGS National Elevation Dataset (NED) DEM developed using the IFSAR technology. The NED DEM resolution varies across the Nation, whereby 1-arc-second (27 meter/~100 feet) DEMs are currently available throughout the whole conterminous United States. In many parts of the United States, 1/3-arc-second (9 meter/~30 feet) DEMs are available, and 1/9-arc-second (3-meter/ ~10 feet) DEM grid cells are available in limited areas. The USGS DEMs can be obtained through the NED Web site http://ned.usgs.gov/.

Many Federal, State, and local agencies collect digital terrain data, which may be available with much greater accuracy than is typically available through the USGS DEMs. Usually, these data are limited to areas of high urbanization, but an increasing number of States now have statewide coverage. High-accuracy digital terrain data in both urban and rural locations is becoming more common.

The United States National Map Accuracy Standards provide accuracy standards for published maps, including horizontal and vertical accuracy, accuracy testing methods, accuracy labeling on published maps, labeling when a map is an enlargement of another map, and basic information for map construction as to latitude and longitude boundaries. The latest standards can be downloaded http://nationalmap.gov/standards/nmas.html.

## SECTION 11 DAM BREACH MAPPING GUIDANCE

Although some State and Federal agencies provide guidance for hydrologic and hydraulic procedures for dam breach inundation studies, few provide guidance for creating maps for an inundation area for purposes other than developing an EAP.

When digital deliverables are required by State and Federal agencies, generally only the digital file format or extension is specified. GIS-based mapping standards have been developed by the USACE and the USDOI, as follows:

> **USACE:** The USACE *Modeling, Mapping and Consequence Standard Operating Procedures* (SOP [April 2011]) provides mapping guidelines in terms of data use and documentation, modeling, and map production technology. The USACE SOP clearly defines new presentation standards for maps and flood profiles that will result in standardized dam breach inundation mapping products for all USACE dams.

> **USDOI:** The USDOI *Inundation Mapping/Modeling Subproject Report, Implementation Phase 1: Launch Risk Reduction* (USDOI, 2011) recommends the development of inundation map technical standards scaled to the magnitude of the dam and the consequences of its failure. The USDOI report defines the minimum map standards by including a list of information required on the map, guidance for rounding of numerical information such as flood depth and velocity, and guidance for optional items such as base map features.

Both the USACE and USDOI guidelines involve standards that were developed specific to the mission of those agencies; these standards may not be easily adapted to the full range of potential mapping products States and local governments may need for dam safety purposes.

The dam breach mapping guidance included in the following sections is intended for use by State and local governments. Adherence to this guidance will result in: 1) map products that look consistent across the United States, and 2) a database structure for GIS-based dam breach modeling such that the models can be archived and leveraged for a range of dam safety products. The preservation of the underlying GIS data used to create hardcopy maps can facilitate the more accurate and cost effective performance of other tasks related to dam safety including hazard mitigation planning, evacuation planning, and consequence assessment. The use of GIS has emerged as the backbone to modern day hydrologic and hydraulic modeling and is the most common method used to build and develop the basic geometry and parameters used in dam breach models.

## 11.1 POTENTIAL USES OF INUNDATION MAPPING

Inundation maps can have a variety of uses including EAPs, mitigation planning, emergency response, and consequence assessment. Each use has unique information requirements and may be used in different manners. This may range from multi-year office-based planning efforts by mitigation planners and dam safety officials to field-based emergency responders responding to a developing or imminent dam breach.

## 11.1.1 Emergency Action Plans

An EAP is a formal document that identifies potential emergency conditions at a dam and specifies preplanned actions to be followed to minimize property damage and loss of life. The EAP specifies actions the dam owner, in coordination with emergency management authorities, should take to respond to incidents or emergencies related to the dam. It presents procedures and information to assist the dam owner in issuing early warning and notification messages to responsible downstream emergency management authorities.

The EAP also includes inundation maps to assist the dam owner and emergency management authorities with identifying critical infrastructure and population-at-risk sites that may require protective measures and warning and evacuation planning. The EAP must clearly delineate the responsibilities of all those involved in managing the incident and how those responsibilities should be coordinated.

Since EAP maps are intended to be used in an emergency, it is critical for these maps to be easily reproducible without loss of critical information. For this reason, maps should be standard sizes such as 8½×11 inches and 11×17 inches. Further, the maps should be created so that they reproduce well in black and white.

## 11.1.2 Emergency Response

Emergency response embodies the actions taken in the immediate aftermath of an incident to save and sustain lives, meet basic human needs, and reduce the loss of property and the effect on critical infrastructure and the environment. In the case of dam failures and incidents, this would be the response by the dam owner, local community emergency management, and first responders such as fire and police departments to minimize the consequences of an imminent or actual dam failure or incident. Actions may include warning and evacuating the population at risk. Evacuation planning and implementation is typically the responsibility of either State or local emergency management authorities. It is important for dam owners to coordinate with the appropriate emergency management authorities and provide information obtained through dam inundation studies to assist the evacuation planning process.

Given the short warning times typically encountered with dam failures and incidents, dam emergency evacuation plans should be developed before the occurrence of an incident. It is recommended that plans be based on a worst case scenario and address the following elements, including identifying the roles and responsibilities for all action items:

- Identification of critical facilities and sheltering

- Initiating emergency warning systems (who is responsible and what is the method)

- Specific evacuation procedures, including flood wave travel time considerations (for example, evacuation of special needs populations and lifting evacuation orders)

- Distance and routes to high ground

- Traffic control measures and traffic routes

- Potential effect of weather or dam releases on evacuation routes (for example, identify whether portions of the evacuation route may be flooded before the dam incident occurs)

- Vertical evacuation/sheltering-in-place

- Emergency transportation

- Safety and security measures for the dam perimeter and affected areas

- Re-entry into affected areas

Although the EAP does not need to include the actual evacuation plan, it should indicate who is responsible for an evacuation, and what plan will be followed. In addition, inundation maps developed by the dam owner must be included in the EAP and shared with emergency management authorities. These maps may help in developing the warning and evacuation plans. Finally, dam owners should include procedures in the EAP for ensuring that emergency management authorities are provided with timely and accurate information on dam conditions during an incident. This will assist those agencies in making the appropriate decisions regarding evacuations.

### 11.1.3  Hazard Mitigation Planning

Mitigation is the proactive effort to reduce loss of life and property by lessening the effect of disasters. This is achieved through identifying potential hazards and the risks they pose in a given area, identifying mitigation alternatives to reduce the risk, and risk analysis of mitigation alternatives. The result is the selection of proactive measures, both structural and non-structural, that will reduce economic losses and potential loss of life when implemented.

In the case of dam failures and incidents, hazard mitigation planning involves identifying the population at risk and identifying actions to reduce their vulnerability. Actions might include setting up a reverse 911 system to provide advanced flood warning and relocating critical infrastructure and facilities out of the inundation zone.

Hazard mitigation planners need digital data that defines the dam breach hazard. Information needed includes the breach inundation zone boundary, depth of flooding, velocity, and timing. Further, digital products compatible with FEMA's HAZUS software allows mitigation planners to use dam breach information to estimate potential damages for benefit-cost analyses, and compare mitigation alternatives and their cost effectiveness.

### 11.1.4  Dam Breach Consequence Assessment

Dam breach consequence assessment includes identifying and quantifying the potential consequences of a dam failure or incident. While hazard mitigation planning focuses on specific projects to reduce flood risk, consequence assessment focuses on the economic and social impacts of a potential disaster and the organizational and government actions needed in the aftermath of a dam breach to respond and recover. Data compiled for a consequence assessment can also be used in risk assessments.

Consequence assessment requires the same basic data as used in hazard mitigation planning, with the addition of data related to communicating the hazard to community elected officials and the public. Advanced mapping products that allow state-of-the-art visualization is key to communicating the hazards and consequences of a potential dam failure.

## 11.2 END-USER PRODUCTS

Potential end users may have different goals and deliverables but all have a common need for a dam breach inundation map. If the inundation modeling and mapping is preserved in a consistent database structure, data can be leveraged to produce a wide range of mapping products (see Section 11.1) allowing the end user the ability to customize the deliverable according to their specific needs.

### 11.2.1 Inundation Map

Inundation maps can incorporate elements beneficial to dam safety officials, emergency responders, and mitigation planners. The maps can be used to facilitate communication during an event while at the same time convey relevant information regarding at-risk areas useful for effective long-term mitigation planning. For example, an inundation map may highlight the most vulnerable population areas. Such information is useful for mitigation planners, who may be able to minimize future flood damage via infrastructure projects and rezoning/relocation efforts.

### 11.2.2 Dam Inundation Database

This document presents an approach to creating a dam inundation database to store and preserve digital data associated with dam breach inundation and mapping from which all users can easily extract information required for a specific purpose. A dam inundation database can enable the preservation of digital data and provide a common approach for the consistent storage and presentation of data. For instance, the database can be developed to drive the labeling and symbology of inundation maps developed using GIS technology. A universal approach to database development will support not only the creation of hardcopy maps, but also the interactive viewing and analysis of data in the digital environment.

The inundation database can accompany the hardcopy map. The database can be as basic as containing only information shown on the maps, to a comprehensive data system that provides enhanced information to users for a wide spectrum of applications. Such a database would yield significant future savings when updating maps since the base data would be available in a workable digital format that could be used to quickly recreate and update hardcopy maps.

The guidance in this document provides the structure consistent with the FEMA non-regulatory database standards with a dual purpose of map storage as well as serving as a clearinghouse of inundation map data including supplemental information. Because the inundation database can include inundation areas and data for multiple failure events, land use, DEMs, population density and an array of other features not shown on the inundation map, the database is thus a valuable asset for hazard mitigation planners and emergency responders, providing them information that

can be accessed in both a digital or hardcopy environment. Details of the inundation database are provided in Section 11.3.4.

## 11.3   MAPPING GUIDANCE

The inundation delineation provides the most important dataset for inundation mapping, the inundation polygon. A polygon refers to a type of layer in a GIS. Once a dam breach simulation is completed, the data can be stored in GIS to provide many potential benefits to users including the potential for improved accuracy and efficiency. Many GIS tools, including the USACE HEC-GeoRAS ArcMap extension and RAS Mapper, provide tools to automatically delineate floodplains or inundations as a post-processing function of HEC-RAS. Many hydraulic models delineate inundation boundaries, potentially eliminating manual GIS processing. However, some of these tools have limitations since they do not contain some of the advanced geoprocessing functions available in more advanced GIS applications. For instance, some two-dimensional models delineate the inundation at a resolution equal to the input grid cell or mesh dimensions. Because the input grids or meshes for two-dimensional models are frequently on the order of several hundred feet, the resulting inundation boundaries can be very approximate and sometimes rectilinear-looking, as shown in Figure 11-1. Depending on the intended map scale, rectilinear-looking inundation boundaries may not be a problem; however, more refined delineations generated by smoothing the rectilinear-looking inundation boundaries are often desirable, particularly if working within a digital environment that allows users to zoom to scales typically not used for hardcopy maps. Care must be applied in the decision to smooth rectilinear-looking inundation boundaries as it can hide the accuracy represented by the modeling and can imply a level of accuracy greater than what actually exists.
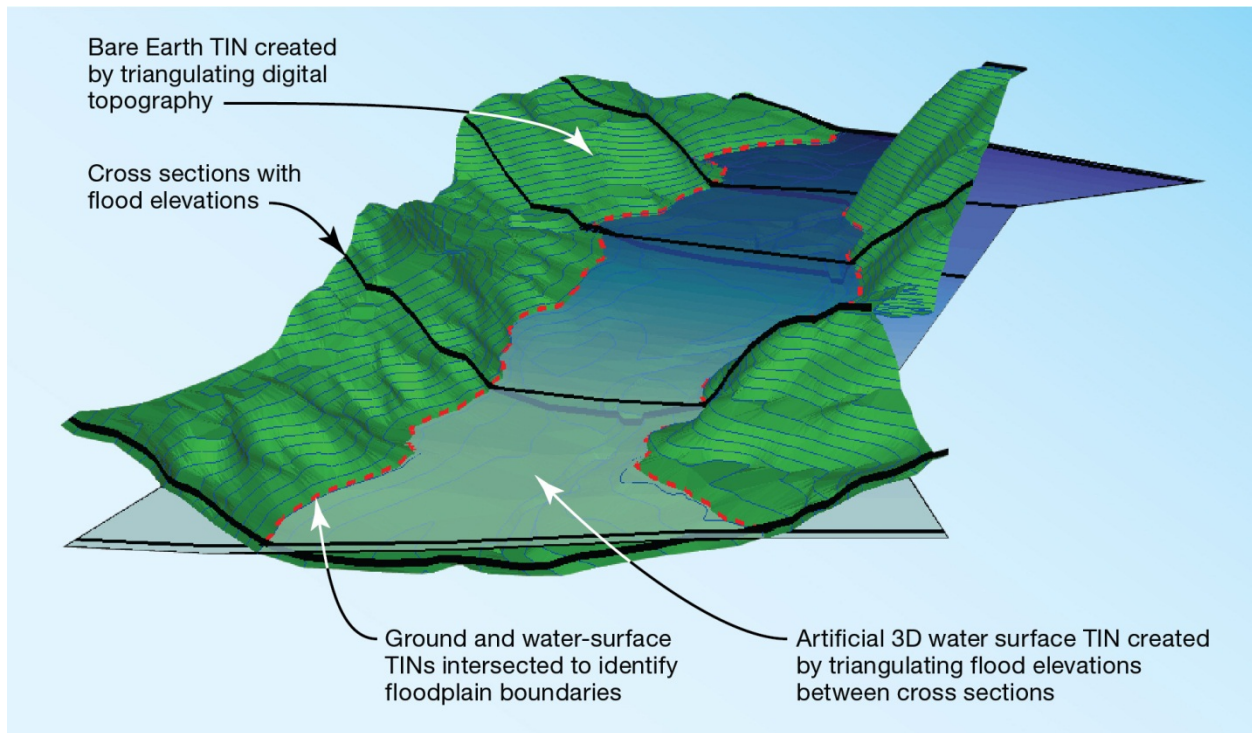
**Figure 11-1: Example of an inundation delineation directly from a two-dimensional model (red lines on the graphic represent cross-section locations)**

Although some two-dimensional applications may provide tools for smoothing, transfer of the two-dimensional model elevations to GIS is sometimes necessary to perform a more detailed delineation on finer DEMs or TINs using the procedures presented in this section. The procedures in this section use advanced geoprocessing techniques built into most specialized GIS applications with three-dimensional and spatial analysis functionality. The selection of the terrain dataset accuracy limits the potential accuracy of an inundation model and similarly can limit the accuracy of the inundation delineation; users must therefore be careful to evaluate and understand the accuracy of a selected terrain dataset.

Regardless of what models are used to determine inundation elevations, GIS can be used to delineate flood boundaries with a level of detail and accuracy that is impractical for manual methods. Automatic delineations compare the inundation model results and ground elevation data. Areas where the modeled flood elevations exceed the ground elevations are mapped as inundation areas. Automated inundation delineations can be performed using either regular (DEM) or irregular (TIN) terrain datasets. Figure 11-2 illustrates the principle of automated inundation delineation within GIS using TIN methodologies.

Bare Earth TIN created by triangulating digital topography

Cross sections with flood elevations

Ground and water-surface TINs intersected to identify floodplain boundaries

Artificial 3D water surface TIN created by triangulating flood elevations between cross sections

**Figure 11-2: Automated inundation delineation methods using irregular (TIN) elevation data**

The basic steps to perform an inundation delineation using raster methods (DEMs) are highlighted in Figure 11-3. Note that the finer the "Ground" and "Water Surface" grid resolutions are, the smoother the inundation delineation will be. Further smoothing can be performed once a vector inundation polygon has been created; the polygon can be either generalized or smoothed depending on specifications or preferences.



Create a ground surface DEM [Ground]

Create a DEM of water surface from model results [Water Surface]

Perform calculation [Water Surface] – [Ground] = [Water Depth]

Perform raster evaluation [Water Depth] > 0 feet = [Boolean] (true/false)

Convert all TRUE values to vector polygons and optionally smooth to create inundation polygons
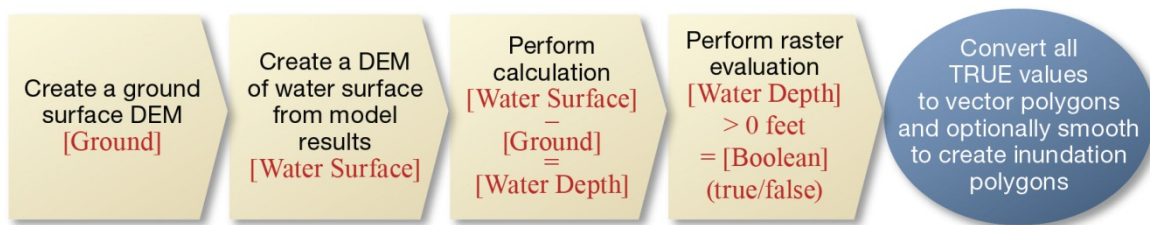
**Figure 11-3: Automated inundation delineation workflow method using raster methodologies (DEMs)**

## 11.3.1 Data Sources

Information to develop a dam breach inundation map can be obtained from various agencies including the USGS, USDA, U.S. Census, Web GIS Services, and various State and local agencies. Geospatial information may include topography, imagery, land use datasets, inventory

of infrastructure elements, demographic features, and more. Online sources of these data are shown in Table 11-1.

**Table 11-1: Online Agency Data Sources**

| Agency | Link | Data Available |
|---|---|---|
| U.S. Geological Survey (USGS) | http://nhd.usgs.gov/ | • National Hydrologic Dataset |
| U.S. Department of Agriculture (USDA) | http://datagateway.nrcs.usda.gov | • National Elevation Dataset<br>• National Agricultural Imagery Program (NAIP)<br>• Land Use Land Cover |
| U.S. Census | http://www.census.gov/geo | • Total population<br>• Population density |
| FEMA Map Service Center | http://www.msc.fema.gov | • FEMA Floodplain and Databases<br>• National Flood Hazard Layer (NFHL) |

Although most searches can be conducted online, data searches can also be performed at local and jurisdictional offices, where information may already be compiled. A community's hazard mitigation plan may also include significant information related to a dam failure and should be consulted for information.

## 11.3.2 Recommended Inundation Map Elements

The following section describes elements of an inundation map that will help users create a simple yet effective and easy-to-use hardcopy map.

### 11.3.2.1 Map Collar Information

Latitude and longitude coordinates can be referenced at the corners of the neatline.[7] Other useful information displayed on the map collar[8] can be horizontal reference grid ticks (e.g., Universal Transverse Mercator or State Plane) to help orient map users to real world coordinates. Adjacent map panel numbers should also be listed along the neatline borders that correspond with adjacent map panels.

### 11.3.2.2 Base Map Data

Base map data provide the background from which inundation hazard information is overlaid and interpreted. Clear, easy-to-interpret base maps are critical for the effective use of an inundation map. There are two broad categories of base maps:

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[7] Border line that indicates the limits of an area shown on a map

[8] Any of the supporting objects or elements that help readers interpret a map. Typical map surround elements include the title, legend, north arrow, scale bar, border, source information and other text, and inset maps.

1. **Orthophotographic base maps:** Orthophotographic base maps use aerial imagery as the map background. Additional content is added to annotate key road names and key landmarks to help orient users and aid interpretation of the inundation maps. Many local communities have high-resolution orthophotographic imagery that may be obtained for a base map. Alternatively, the USGS NAIP listed in Table 11-1 is readily available for use as a base map for most of the United States.

2. **Planimetric base maps:** Planimetric base maps use vector features, such as annotated road lines, political boundaries, streamlines, landmarks, etc., against a typically white background that allow users to orient themselves and interpret the inundation maps. Many local community planning or GIS departments have suitable vector data available. Data may include road lines, property boundaries, building footprints, political boundaries, etc. In the absence of local data, the U.S. Census Bureau's Topographically Integrated Geographic Encoding and Referencing (TIGER)/Line shapefiles are a valuable source of vector base data; area names, transportation networks, and geographical divisions are available.

Additional content for a base map can be added on a case-by-case basis. Map makers should be careful to provide informative ways for users to reference the elements of the inundation maps while minimizing clutter on the map. A hybrid orthophotographic base map supplemented with planimetric data is generally more effective than an orthophotographic base map alone, especially when the resolution of the orthophotographic image is low or has deliberately been reduced to minimize digital file sizes. Landmarks, reference marks, hydrographic features, and road networks are useful vector features that are often displayed to cue map users to a given location. Map makers should use cartographic judgment when developing a map; the final product should be easy to use. Often the decision between an orthophotographic or planimetric base map comes down to individual user preferences or defined standards.

## 11.3.2.3 Inundation Polygons

The key information on an inundation map is provided by one or more inundation polygons that define the horizontal limits of the inundated area for one or more breach events. As described in the introduction to Section 11.3, the inundation polygons show the intersection of the peak water surface elevations from the dam breach model with the ground elevations from the terrain source. If multiple breach events will be shown on the inundation map, the polygon representing the event that would result in smallest inundation area should be displayed on top of those representing events with larger inundation areas. For example, an inundation resulting from a ¼-PMF event would be shown in the display order above the larger inundation resulting from a PMF event.

## 11.3.2.4 Inundation Elevations

Inundation elevations can be annotated at key locations along the inundation polygon if desired. The inundation elevations can be extracted directly from a dam breach model. Elevations are not always a critical element for an inundation map. Emergency responders are primarily interested

in the extent and depth of inundation rather than the elevation of flooding. Elevations may be important for flood warnings, however, particularly if early warnings are possible.

### 11.3.2.5    Flood Wave Arrival Time

Flood wave arrival times can be annotated at key locations along the inundation polygon if desired. The flood wave arrival time is the time (usually in minutes) from dam breach initiation until the leading edge of the inundation arrives at a specific location. This is typically determined by inspecting the stage (time versus stage) or flow (time versus flow) hydrograph plots at points of interest. Points of interest typically include road crossings, significant populations at risk, and critical infrastructure. The arrival time is often characterized by a sharp increase in the flow.

For a fair weather failure, the arrival time can be considered the first time that a notable change in the base flow is observed. This is normally defined as the time until the stage of the river or creek rises by a set depth from base flow conditions, typically 1 or 2 feet. Some agencies may use the safe channel capacity as the standard for defining arrival time.

For a hydrologic failure event, the arrival time is best determined by comparing two simulations for the same hydrologic event. The first simulation would be a non-breach hydrologic event, while the second simulation would be the exact same hydrologic event but with the dam breaching. The downstream hydrographs of both events can be overlaid to identify what time the effects of the dam breach would be first observed. The separation of the two hydrographs at the point of interest indicates the effects of the dam breach at that location. The arrival time for hydrologic events is normally defined as the time lapse from breach initiation until the differential stage for with- and without-failure simulation for the river or creek to exceed a defined depth, typically 1 or 2 feet. Figure 11-4 illustrates a simplified hydrograph plot generated using HEC-RAS for a non-breach hydrologic event and the same hydrologic event with a dam breach. The arrival time of the effects from the dam breach can clearly be seen to occur around 04:00 hours as characterized by the separation of the two hydrographs at this time.
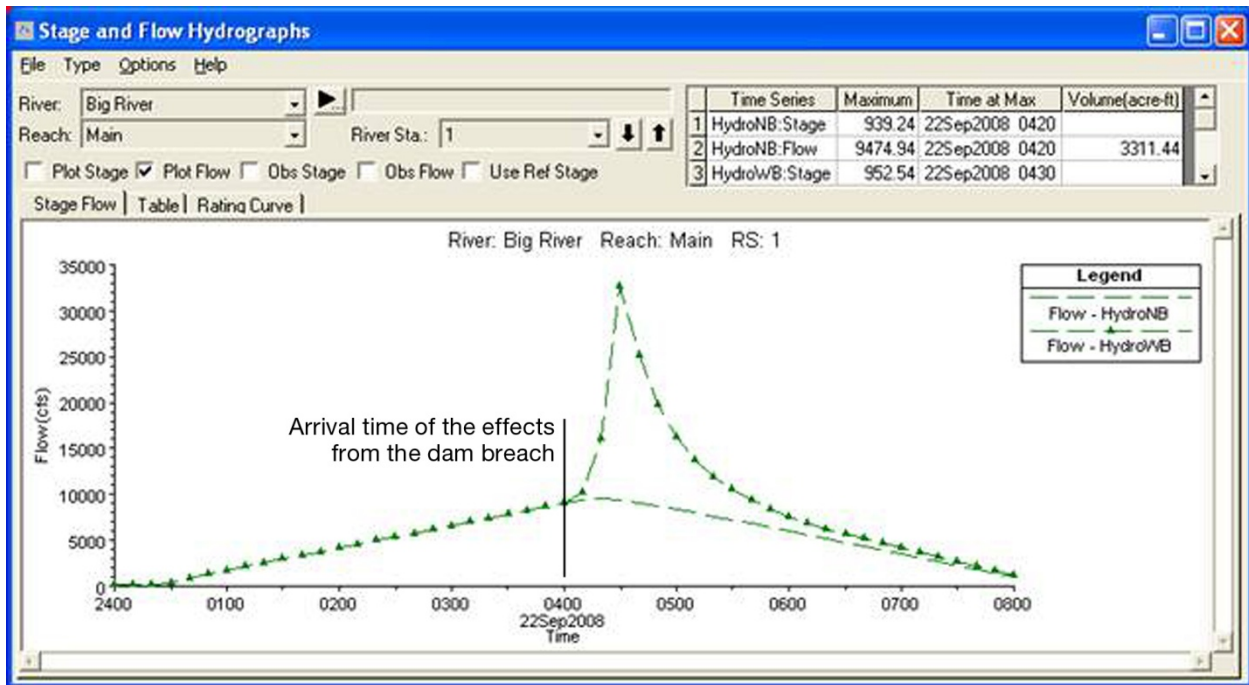
**Figure 11-4: HEC-RAS computer program output showing differences between two hydrographs generated by the same hydrologic event**

Table 11-2 shows the recommended intervals for flood arrival times that should be included on inundation mapping, although judgment should be applied when selecting mapped intervals and should be commensurate with the population at risk and map scale:

**Table 11-2: Time Intervals to Include on Inundation Maps**

| Time after Breach | Mapped Arrival Time Intervals |
| --- | --- |
| 0 – 30 minutes | 5 minutes |
| 30 – 90 minutes | 10 minutes |

For extremely large dam failures, it may be necessary to report arrival times in hours for situations where the point of interest is a considerable distance downstream of the dam and the inundation elevations due to the breach are still significant at that location. Engineers and mapping analysts should take care to annotate the map with flood arrival times that provide the most useful information. For emergency responders, arrival times should be labeled at points of interest. For mitigation planning and consequence assessment, more detailed information is often required. For instance, probable loss-of-life estimates are generally highly sensitive to flood arrival times and users may require inclusion of all time intervals identified in Table 11-2.

Inundation zones (and associated data) can be developed for many types of incidents ranging from a minor incident, such as a minor uncontrolled release of water, to a full catastrophic dam failure associated with a PMF event. As a result, the type of event or events to be shown on a

map should be carefully selected and the map should be clearly labeled. All of the data needed to create a variety of mapping products in support of the identified user need can be stored in the inundation database. The inundation database is described in more detail in Section 11.4.

### 11.3.3  Recommended Format of EAP Inundation Maps

EAP inundation maps must be developed with the anticipation of being widely used in the field by emergency responders in the event of an EAP being activated. Hardcopy maps are typically preferred by emergency responders; inundation maps should always be printed when creating an EAP to avoid relying on power and printing technology that may not be available to print the maps in an emergency situation.

**Printing Considerations:** Hardcopy maps should be uncluttered and easy to read. Only the most relevant information for field use should be provided on the maps. Although maps of larger size can be created, 8½×11 inch or 11×17 inch sizes provide a map that is easily reproduced by most modern photocopy machines. Since most EAP activations occur as a result of hydrologic events, lamination of hardcopy maps can make them more resistant to water and general wear and tear when being used in severe weather conditions.

Maps can be printed in color, allowing an emergency responder to see the most detail; however, maps should also be tested using black and white copiers to ensure the maps still communicate the same information if only black and white copiers are available. To ensure that orthophotographic base maps can be easily reproduced in black and white without overpowering overlaid features such as inundation polygons, the brightness of the base map should either be increased or alternatively, the imagery should be made approximately 50 percent transparent.

Mitigation planners may use EAP maps, but they may wish to overlay additional features such as political boundaries, population data, or zoning information. Additionally, mitigation planning discussions often require large-sized maps suitable for workshops and presentations. In these cases, digital inundation maps or hardcopy maps printed on full size (24×36 inch or larger) sheets are typically preferred.

**Title Block Information**: Every map should be linked by an index map to allow the user to know the location of the panel in relation to other map panels. The title block on each map panel should contain the following information:

- Title of the map referencing the EAP and inundation scenario

- Index map for all multi-panel map schemes

- North arrow

- Map scale bar

- Legend identifying all critical map features

- Vertical elevation datum reference (if elevations are shown)

- Date of map creation

As described above, inundation maps should be tailored to fit the end use, which is usually emergency response, mitigation planning, or consequence assessment. Table 11-3 below shows a matrix of inundation map guidance and features recommended for each use.

**Table 11-3: Inundation Map Development Guidelines**

| Primary End Use | Size | | | Format | | | Base Map Data | | | | | | | Inundation Data | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Letter - 8.5×11 inch | Tabloid - 11×17 inch | Full - 24×36 inch | Hardcopy | Hardcopy Laminated | Digital | Streets and Highways | Planimetric Data | Political Boundaries | Key Landmarks | Orthophotographic Imagery | Cross-Sections | Hydrography | Inundation Duration | Multiple Inundation Areas | Flood Depth | Water Surface Elevation | Discharge | Velocity | Arrival Time | Time-to-Peak | Danger Zones [1] |
| Emergency Response (EAP) | • | • | | | • | • | • | • | • | • | • | | | • | | • | | | | • | | |
| Mitigation Planning | | | • | • | | • | • | • | • | • | • | • | • | • | • | • | • | • | • | • | • | • |
| Consequence Assessment | | | | | | • | • | • | • | • | • | • | • | • | • | • | • | • | • | • | • | • |

(1)  Classification determined in TM 11 (USBR, 1988)

Figures 11-5 through 11-7 provide typical layout views of an EAP inundation map. All maps are printed to full page, landscape orientation, on 8½ ×11 inch, 11×17 inch, or 24×36 inch paper.

**Map Scale:** The appropriate scale of an inundation map depends on the size of the modeled inundation area and the population at risk.

- In areas with a high population at risk, a scale of 1:6,000 (1 inch = 500 feet) or larger is typically used.

- Areas with a low population at risk may be more suited to 1:12,000 (1 inch = 1,000 feet) or 1:24,000 (1 inch = 2,000 feet) for areas of very low population density.

- For very large dams with an extensive inundation area that is largely unpopulated, larger scales such as 1:63,360 (1 inch = 1 mile) or 1:126,720 (1 inch = 2 miles) may be adequate.

When mapping a large inundation area, it may be practical to use map panels of varying scales if the land use varies notably within the inundation area; this allows the use of larger scale more detailed map panel scales can be used in high risk areas.

Showing a scale bar on each map panel, in addition to or in lieu of a stated map scale, is important. Stated map panel scales (such as 1:24,000) are strongly discouraged because the stated scale can become invalid and result in misleading information if the maps are enlarged or reduced during printing and copying. Scales can be easily changed when printing or reproducing maps whether accidentally or intentionally. However, a scale bar remains valid regardless of

whether the maps are reduced or enlarged during printing and copying as long as the aspect ratio of the map does not change.
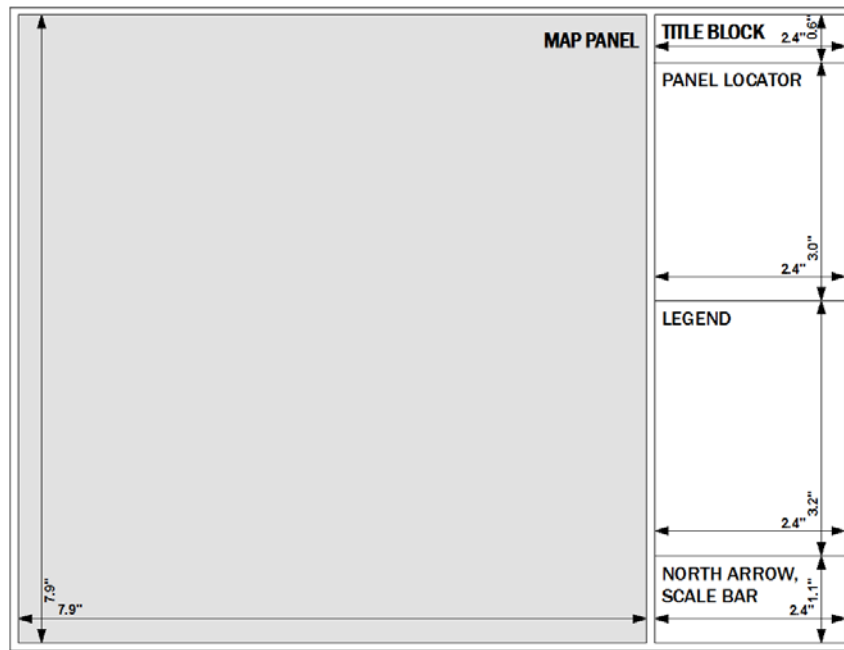


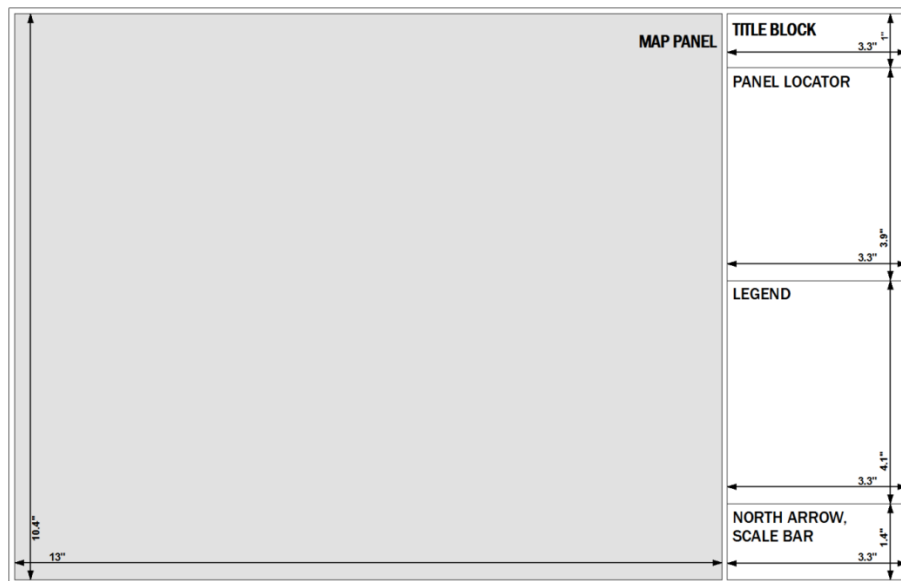**Figure 11-5: 8.5-inch × 11-inch map frame dimensions**



**Figure 11-6: 11-inch × 17-inch map frame dimensions**

**Figure 11-7: 24-inch × 36-inch map frame dimensions**

### 11.3.3.1  Map Annotation

The USDOI provides guidance for rounding numerical figures on an inundation map in *Inundation Mapping/Modeling Subproject Report, Implementation Phase 1: Launch Risk Reduction* (USDOI, 2011). The USDOI rounding recommendations are presented in Table 11-4 and provide guidance for the annotation of parameters on a hardcopy map.

**Table 11-4: Rounding Recommendations for Map Annotation**

| Item | Rounding Recommendations |
|---|---|
| Flood depth | 1 ft |
| Water surface elevation | 1 ft |
| Discharge | 100 ft$^3$/second |
| Arrival time | 5 minutes |
| Time to peak inundation | 5 minutes |
| Velocity | 1 ft/second |
| Depth x velocity | 1 ft$^2$/second |

*Source: USDOI, 2011*

Rounding requirements for hardcopy maps should not be confused with rounding requirements for the inundation database presented in Section 11.4 because an inundation database is capable of storing data to a much higher level of precision, allowing future users to make an informed decision as to the rounding accuracy presented in future inundation maps and analyses.
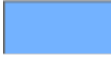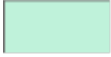
### 11.3.3.2    Map Symbology

Table 11-5 provides guidance related to feature symbology for an EAP map. Figure 11-8 illustrates an example EAP map applying the guidance from Table 11-5.

**Table 11-5: Suggested Map Symbology for EAP Maps**

| FEATURE SYMBOLOGY | | |
|---|---|---|
| **Example** | **Feature** | **Specification [Hatch Pattern] (RGB Value)** |
| BIG CREEK DAM | Dam Alignment and Text Label | Symbol Layer #1:  Line weight 1 pt., Orange (250,115,17),<br><br>Solid Horizontal Line<br><br>Symbol Layer #2:  Line weight 4 pt., Orange (250, 117, 17),<br><br>Dash/Gap Pattern: [6-9-5] (in points)<br><br>Dash/Gap Pattern Interval: 1 pt.<br><br>Font: 14 pt. Arial, Bold, Italics, Aligned Left,<br><br>0.75 pt. White Halo, CAPS |
| 0.5 miles | Distance Downstream of Dam Markers | Circle radius 0.2372 inches, Blue (190, 232, 255),<br><br>outline 0.5 pt  Black (0,0,0)<br><br>Number Font: 6.8 pt. Arial, Bold, Black<br><br>Unit Font: 4.86 pt. Arial, Bold, Black |
| *Big River* | Stream Centerline and Text Label | Line weight 1.5 pt., Blue (0, 112, 255)<br><br>Dash/Gap Pattern: [9-1-1-1-1-1-1-1-9] (in points)<br><br>Dash/Gap Pattern Interval: 4 pt.<br><br>Font: 13 pt. Times New Roman, Bold, Italics |
|  | Arrival Time Marker and Text Label | Line weight 0.8 pt, Black (0, 0, 0), Solid line<br><br>Line weight 2.6 pt, White (255, 255, 255), Solid line<br><br>Line weight 3.4 pt, Black (0, 0, 0), Solid line |

**Table 11-5: Suggested Map Symbology for EAP Maps**

| FEATURE SYMBOLOGY | | |
|---|---|---|
| **Example** | **Feature** | **Specification [Hatch Pattern] (RGB Value)** |
| | | Font: 8pt. Arial, Black, 2 pt, White Halo |
| | Downstream Inundation Area (Lowest Magnitude) | Fill Color: Blue (115, 178, 255)<br><br>Outline weight 0.4 pt, Gray (104,104,104), Solid line<br><br>Transparency: 50% |
| | Downstream Inundation Area (Greatest Magnitude) | Fill Color: Blue (168, 0, 0)<br><br>Outline weight 0.4 pt, Gray (104,104,104), Solid line<br><br>Transparency: 50% |
| | Lake/Reservoir | Line Fill Symbol Layer #1: Line weight 0.4 pt<br><br>Line Color: Blue (64,101, 235), angle 45°, offset 0.90, separation 5.00, outline 0.4 pt, Blue (64,101,235)<br><br>Line Fill Symbol Layer #2: Line weight 1.6 pt<br><br>Line Color: Blue (151, 219, 242), angle 45°, offset 0.00, separation 5.00 |
| | Inundated Structures | Fill Color: Blue (191, 242, 218)<br><br>Outline weight 0.4 pt, Gray (110, 110, 110), Solid line |
| | County Boundaries and Text Label | Symbol Layer #1: Line weight 0.4 pt, Black (255, 255, 255),<br><br>Dash/Gap Pattern: [6-1-3-1] (in points)<br><br>Dash/Gap Pattern Interval: 1 pt<br><br>Symbol Layer #2: Line weight 3 pt, Gray (170, 170, 170), Solid line<br><br>Font: 9 pt, Arial, Bold, Black, 1 pt, White Halo |

**Table 11-5: Suggested Map Symbology for EAP Maps**

| FEATURE SYMBOLOGY | | |
|---|---|---|
| **Example** | **Feature** | **Specification**<br>**[Hatch Pattern] (RGB Value)** |
|  | Corporate Limits (Cities/other) and Text Label | Line weight 1.5 pt, Black (255, 255, 255), Solid line<br><br>Font: 9 pt, Arial, Bold, Black, 1 pt, White Halo |
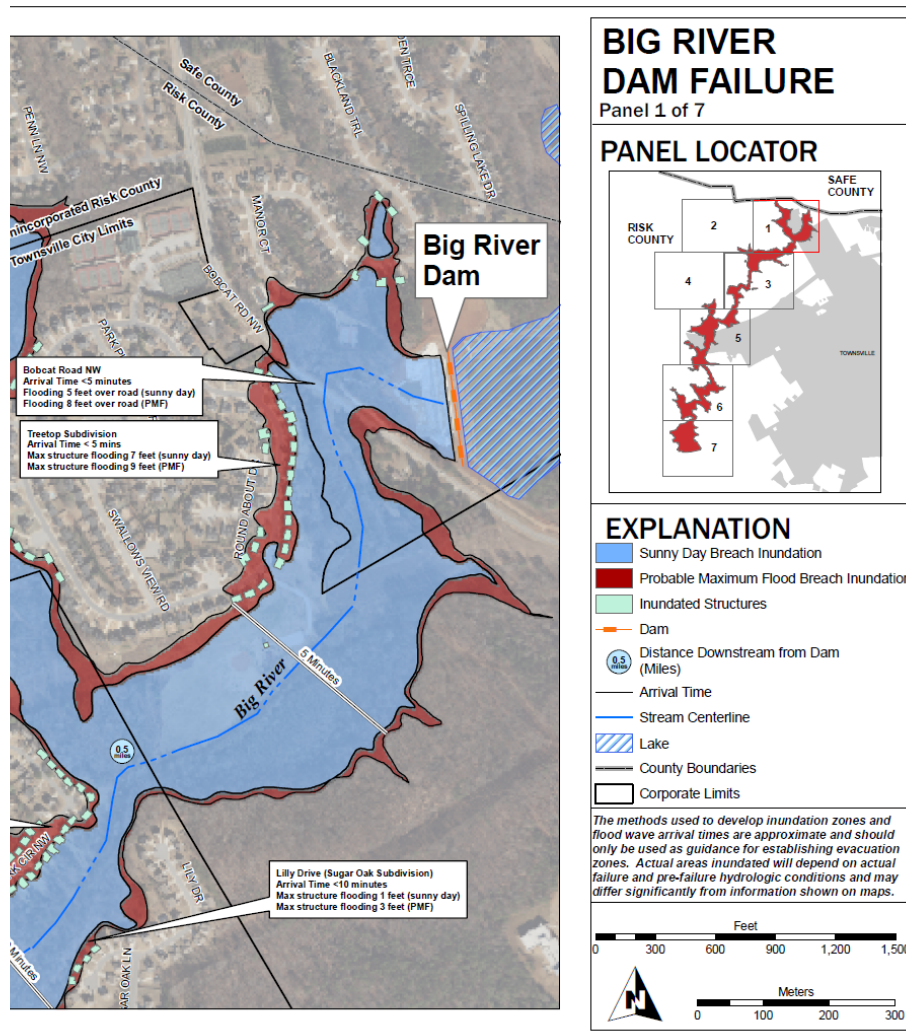
**Figure 11-8: Example EAP map applying suggested map symbology**

## 11.4   DAM BREACH INUNDATION DATABASE

The purpose of the inundation database is to provide a consistent digital format to store dam-specific digital data related to dam breach studies and mapping, preserving the data and making it readily accessible for future study and mapping updates as well as further review and analysis. The inundation database can contain information on multiple inundation events including geospatial data, tabular data, elevation data, model data, reports, and scanned hardcopy documents. FEMA's GeoDamBREACH software uses this format to store and organize all data created during a GeoDamBREACH study.

If geospatial and tabular database features are stored in the inundation database, inundation maps can be produced from the database. To maximize usefulness, electronic copies of the inundation maps can be stored in both native GIS project and portable document formats (PDF). This is useful both to provide information to mitigation planners and when it is necessary to distribute information to emergency responders quickly and efficiently. Where digital data is not available,

it is recommended that all hardcopy data be scanned to PDF and saved into the inundation database in lieu of digital data.

## 11.4.1 Database Structure

The database structure comprises a series of file folders that contain information specific to a dam. The database structure is intended to be a flexible foundation that can be customized to meet the needs of individual dam owners, and Federal, State, and local agencies. At the root of the folder structure is a folder that contains all information for that specific dam. It is recommended that this folder be labeled according to the NID identity or a unique identity assigned to the dam by a State or Federal agency as the dam name (refer to Figure 11-9, [Dam_Name]). Within this primary folder, there are several subfolders to organize digital data related to the dam. Adoption of the folder names shown in Figure 11-9 will ensure consistency across different State and local agencies and facilitate information sharing.
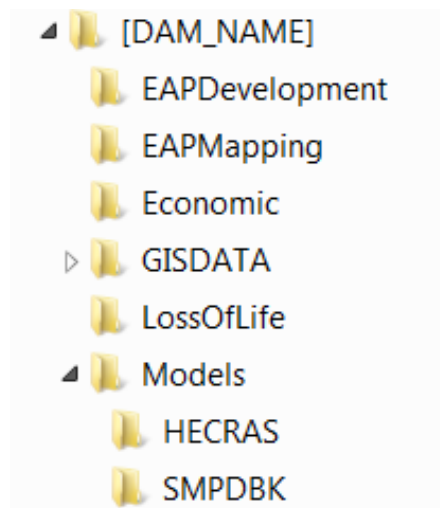


**Figure 11-9: Inundation database folder structure**

### 11.4.1.1   EAPDevelopment Folder

The *EAPDevelopment* folder can be used to store all digital data related to the development of an EAP, including digital word processing files used to generate the EAP, as well as the databases, spreadsheets, and diagrams used to populate the EAP. The folder can also be used to store scanned copies of existing or historic EAPs.

### 11.4.1.2   EAPMapping Folder

The *EAPMapping* folder can be used to store all digital data related to EAP maps, including digital GIS map documents and associated data or simple scans of hardcopy maps.

### 11.4.1.3   Economic Folder

The *Economic* folder can be used to store all economic studies associated with the potential failure of the dam, including economic losses assessments using HAZUS, the USACE HEC-

Flood Impact Assessment program, and benefit-cost analyses of potential mitigation projects among others.

### 11.4.1.4    GISData Folder

The *GISData* folder can be used to contain all geospatial data related to dam break models. FEMA has developed numerous digital dataset formats for dams as part of its Non-Regulatory Flood Risk Datasets[9] that can be used for guidance in developing the data structure and naming conventions for the following three types of data that would be stored within the *GISdata* folder:

1. **Vector data with real world coordinates and associated tabular data.** Vector data can be in the form of lines, polygons, or points and can contain combinations of text and numerical tabular information. Vector data is generally used to define the spatial extents of the inundation area and hydrographic features such as stream centerline and cross-section locations.

2. **Lookup tables that store tabular data and can be linked to spatial data using primary and secondary keys.** Lookup tables are generally used to store multiple data entries for a single vector feature where a many-to-one relationship exists. An example is a lookup table linked to individual model cross-sections that relates results from many modeling scenarios for individual cross-sections, such as fair weather, gate failure, and PMF.

3. **Raster datasets that contain regularly spaced gridded data with values (can include imagery and elevation data).** Raster datasets are a key component of the Non-Regulatory Flood Risk Datasets. Raster datasets presenting the results of a dam break inundation study are typically developed at 10-foot to 25-foot resolutions, although larger resolutions may be suitable for very large dams. These datasets allow for detailed inundation data to be stored and rapidly queried or analyzed. Figure 11-10 illustrates a raster depth grid that represents the inundation depth at any point in the inundation area using regularly spaced 10-foot grid cells.

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[9] http://pm.riskmapcds.com/riskmap_usergroups/NRPT/Shared Documents/Guidance
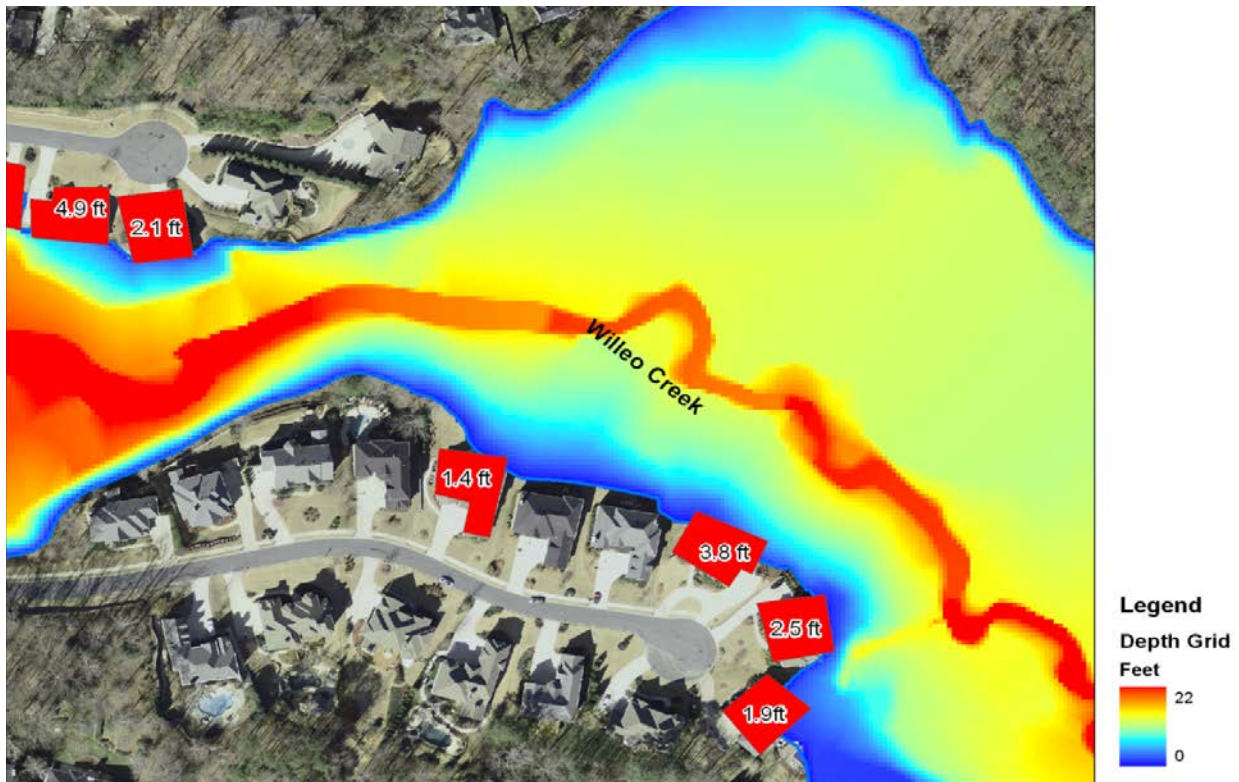
**Figure 11-10: Example of a depth grid for a dam inundation with structures overlaid and attributed with maximum flood depths extracted from the depth grid**

FEMA's Non Regulatory Flood Risk Datasets present numerous raster datasets. FEMA's GeoDamBREACH application enables users to create many flood risk datasets. FEMA's Non-Regulatory Raster Flood Risk Datasets include:

- **Depth Grid.** This raster dataset contains flood depths resulting from an inundation. Unique combinations of flooding event, release type, and the hydrologic condition of the reservoir at the time of the release are used to differentiate the depth grids.

- **Velocity Grid.** This raster dataset contains flood velocities resulting from an inundation. Unique combinations of flooding event, release type, and the hydrologic condition of the reservoir at the time of the release are used to differentiate the velocity grids.

- **Flood Wave Arrival Time Grid.** This raster dataset depicts the arrival time (in minutes) of the flood wave from an inundation. This is typically defined as 1 to 2 feet of rise in water surface elevation from the pre-scenario base flow. Unique combinations of flooding event, release type, and the hydrologic condition of the reservoir at the time of the release are used to differentiate the arrival times.

- **Time-to-Peak Grid.** This raster dataset depicts the time it takes for the peak of the inundation to reach a particular location. Time is measured in minutes. Unique combinations of flooding event, release type, and the hydrologic condition of the reservoir at the time of the release are used to differentiate the time-to-peak grids.

- **Flood Inundation Duration.** This raster dataset depicts the amount of time it takes a flood wave to arrive at, pass through, and leave a particular location. Time is measured in minutes. Unique combinations of flooding event, release type, and the hydrologic condition of the reservoir at the time of the release are used to differentiate the flood inundation duration grids.

- **Water Surface Elevation Grid.** This raster dataset depicts the peak water surface elevation from an inundation. Unique combinations of flooding event, release type, and the hydrologic condition of the reservoir at the time of the release are used to differentiate the water surface elevation grids.

Although raster datasets can sometimes be a direct output from two-dimensional hydraulic modeling software, some GIS processing is often needed to create raster datasets when using one-dimensional software. Raster datasets—including elevation-based grids, time grids, discharge grids, and velocity grids—are often created from one-dimensional vector model features and results. This is achieved by attributing model results to vector cross-section information and using raster interpolation to create a seamless raster dataset. Raster datasets such as depth grids and danger zone grids determined in accordance with TM 11, *Downstream Hazard Classification Guidelines* (USBR,1988) may be derived through raster calculations using a variety of two or more raster datasets.

Elevation for raster datasets is normally reported in feet above the North American Vertical Datum of 1988 (NAVD 88) although some communities may still use the National Geodetic Vertical Datum (NGVD) of 1929. Time for raster datasets is normally reported in minutes.

Raster datasets of critical model outputs can be invaluable for annotating individual map panels since a simple click on a grid can provide critical information such as depth, inundation duration, and arrival times as illustrated in Figure 11-11.

GIS data is typically stored in shapefile, grid, and tabular formats as well as within a File GeoDatabase or Personal Geodatabase.

### 11.4.1.5   LossOfLife Folder

The *LossOfLife* folder can be used to store all probable loss-of-life studies associated with the failure of the dam, including loss-of-life assessments derived by applying the Brown and Graham method outlined in the USBR's DSO-99-06 (USBR, 1999).

### 11.4.1.6   Models Folder

The *Models* folder can be used to store all dam break models including digital models, if available. It can also store scanned hardcopies of historic dam break models. These models are organized into subfolders that describe the type of model used.
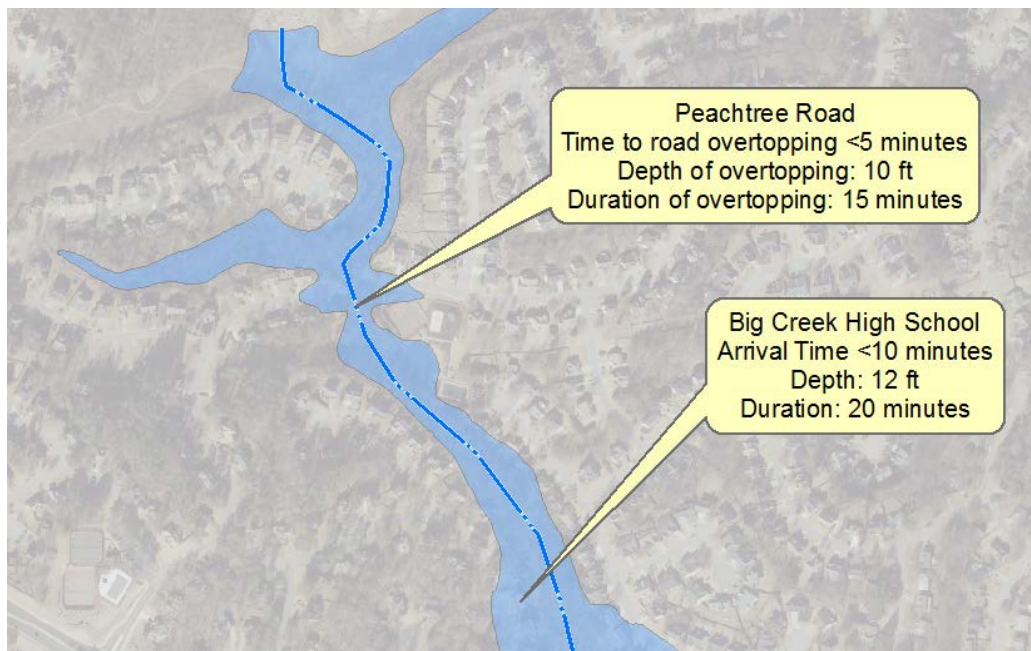
**Figure 11-11: Individual map panels annotated from raster datasets of critical model outputs**

## 11.4.2 Inundation Database Overview

Abbreviations for common breach events, such as a PMF or fair weather event, are provided in Table 11-6 and can be used to name and describe data within the inundation database. Since the list is not inclusive of all events that a dam safety official may need, additional abbreviations may be made following the event description. For example, if a breach event for an observed hydrologic event was modeled, the event abbreviation may be named 'Sep09' if the event occurred in September 2009. FEMA's Non-Regulatory Flood Risk Datasets documentation further describes how these descriptions can be used for naming of digital datasets.

## 11.4.3 Projection and Datum

All digital data within the inundation database should be of a common horizontal and vertical projection. The recommended format of the inundation database facilitates the storage and identification of projection and datum information.

Different agencies that use inundation maps may use different horizontal projections and vertical datums; therefore, it is essential to document the projection and datum of all datasets throughout the inundation mapping process. When using digital GIS data, the horizontal projection and vertical datum should be documented in the metadata and defined within the dataset. Most modern GIS applications are able to instantly re-project data into a common defined horizontal map datum as long as the datum is accurately defined within the inundation database.

**Table 11-6: Dam Breach Inundation Event Abbreviations**

| Event | Release Type | Reservoir Condition |
|---|---|---|
| **0_2** – 0.2-percent-annual-chance event<br>**01** – 1-percent-annual-chance event<br>**02** – 2-percent-annual-chance event<br>**04** – 4-percent-annual-chance event<br>**10** – 10-percent-annual-chance event<br>**PMF** – Probable Maximum Flood<br>**PMF14** – ¼ of PMF<br>**PMF13** – ⅓ of PMF<br>**PMF12** – ½ of PMF<br>**PMF34** – ¾ of PMF<br>**PMP** – Probable Maximum Precipitation<br>**PMP14** – ¼ of PMP<br>**PMP13** – ⅓ of PMP<br>**PMP12** – ½ of PMP<br>**PMP34** – ¾ of PMP<br>**FW** – Fair weather<br>**FOR** – Flood of Record (to be described in L_Scenario and metadata) | **P**iping<br><br>**O**vertop<br><br>**G**ate Failure | **F**ull<br><br>**N**ormal Pool<br><br>**A**uxiliary Spillway<br><br>**P**rimary Spillway |

Almost all Federal, State, and local agencies use the NAVD 88 for vertical elevation data, although many existing datasets have not yet been converted from the NGVD of 1929. Therefore, when processing any vertical elevation data, it is critical to identify the vertical datum and, where necessary, perform a vertical datum conversion. FEMA's *Guidelines and Specifications for Flood Hazard Mapping Partners, Appendix B* (FEMA, 2002), provides users with guidance for performing datum conversions of elevation data. When a datum conversion is performed, all documentation supporting the calculated conversion factors should be preserved for future reference and quality checks.

## 11.5    INTERACTIVE USE OF MAPS AND GIS DATA WITHIN A DIGITAL ENVIRONMENT

The use of the inundation database and other datasets within a digital GIS environment provides many potential benefits to emergency responders and mitigation planners since users are not limited to information displayed only on a hardcopy map. Useful information that may be stored in the database may include data related to the inundation including arrival times, duration, and depth for areas not annotated on a hardcopy EAP map. Users can determine these parameters for any building or point of interest within the inundation area using simple GIS queries. Additionally, base map features not shown on an EAP map, such as road centerlines, building footprints, and tax parcels, may provide additional critical information to an emergency

responder that could help them identify a road name not annotated on an EAP map, identify the name of a building and its owners, or derive information related to a tax parcel.

Digital GIS data not only supports queries of existing data, but can also be used for geoprocessing and calculations to determine additional information to assist emergency responses, planning efforts, and consequence assessments. Users may need to determine information such as danger classifications based on depth and velocity relationships to implement a classification system such as that presented in TM 11 (USBR, 1988).

## 11.5.1  Mobile GIS for Emergency Responses

In recent years, the use of mobile computing using laptop and tablet computers has increased significantly. Laptop and tablet computers have become smaller and more powerful, enabling performance similar to desktop computers while maintaining mobility. Numerous GIS applications now support mobile working whereby users can access GIS through either locally stored data or Web-based mapping systems including online map sites or cloud computing. These applications cater to a wide variety of user abilities, ranging from basic easy-to-use applications with limited functionality to highly complex and flexible applications suited to expert users. The greatest advantage of mobile GIS for an emergency response is that the user is not limited to the printed information on an EAP map, which typically shows only the most critical information to avoid confusion and cluttering of the map. With mobile GIS, users can potentially access thousands of different datasets that provide additional information that may enable a user to save a life or avoid losses during an emergency situation. Users can turn datasets on and off as needed without cluttering the digital map view.

Additionally, unlike hard copy EAP maps that are generally limited to one or two breach scenarios, an inundation database accessible by mobile GIS can potentially contain many breach scenarios and thereby allow a mobile user to rapidly customize their mobile GIS applications to reflect the most appropriate scenario.

Lastly, many modern mobile devices include a Global Positioning System that can be used to ground truth areas of concern if adequate warning is provided prior to a breach and determine if special evacuation accommodations are necessary.

Although mobile GIS is not a solution suitable for all users and situations, the use of an inundation database in combination with local and regional GIS information in the digital environment can potentially enhance a user's ability to respond when an EAP is activated and unanticipated circumstances are encountered. Mobile GIS should never completely replace hardcopy EAP maps, however, since the GIS abilities of emergency responders likely vary considerably and there is always the potential for technical complications including power failure, equipment damage, and software failure.

Mobile GIS equipment used for emergency response should be tested regularly to ensure that equipment is ready and operable with all required software and data preloaded. Special care must be taken to ensure that mobile equipment is fully charged and that backup power is available. Waterproof cases and hardware protection are essential for mobile GIS devices because

emergency responses frequently take place during times of severe weather that can damage electronic devices. Figure 11-12 illustrates a mobile device used in the field.

**Figure 11-12: Example of a mobile device**

## 11.5.2 Desktop GIS for Mitigation Planning and Consequence Assessments

Similar to mobile GIS, the greatest advantage of desktop GIS for mitigation planning is that the user is not limited to just the printed information on an EAP map. Because mitigation planners are not under the same time constraints as emergency responders, they normally have time to perform more detailed analysis that can be supported by information in an inundation database. Detailed analyses may include, among others, economic assessments, loss-of-life assessments, and critical infrastructure impact assessments. The use of an inundation database with consistently formatted GIS data can support these assessments by providing critical information for geoprocessing and queries.

**Example – Estimating potential economic loss:** An example of a mitigation planning use of the inundation database is the use of FEMA's HAZUS application to perform economic loss estimation. HAZUS is able to use inundation depth grids to estimate potential economic losses by using census data, referred to as the general building stock method, or by using the user-defined facilities method, which uses user-defined datasets including individual building footprints to more accurately estimate potential losses.

**Example – Assessing potential loss-of-life:** Another example includes the use of the inundation database for loss-of-life assessments. There are numerous studies, including the publication DSO-99-06 (USBR, 1999), that provide methods for estimating the potential loss of life from dam failures and natural floods. The methods presented in DSO-99-06 include the use of warning times that can be calculated from arrival times using arrival time grids and flood severity values queried from depth and velocity using inundation depth and velocity grids.
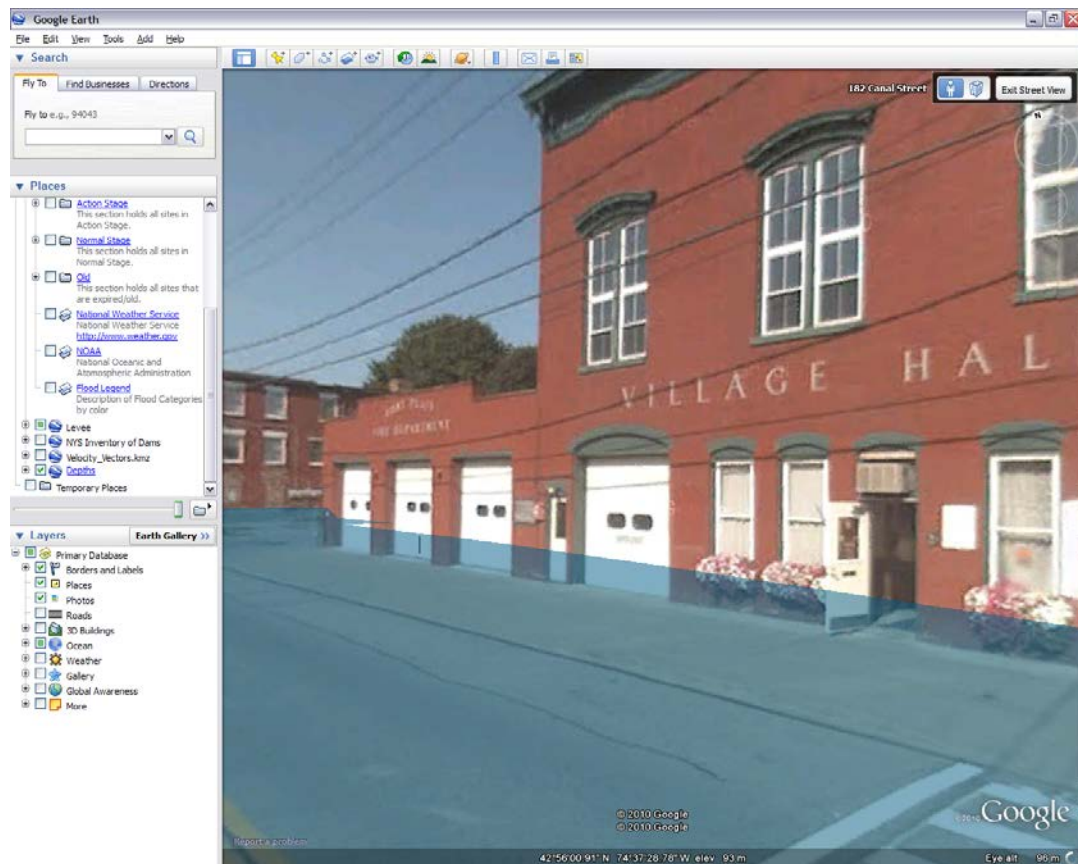
## 11.6    THREE-DIMENSIONAL VISUALIZATION OF INUNDATION ZONES

Three-dimensional visualization can be a powerful tool for communicating risk and can be easily applied to dam inundation areas. Three-dimensional visualization can provide both experienced and inexperienced users easily recognizable images of an area inundated by water that aid in understanding the magnitude and severity of the event. For example, a standard two-dimensional map typically does not indicate the depth of flooding; the use of graduated colors to represent inundation depths may not convey the true severity of the event to the untrained eye. Three-dimensional visualization can put a new perspective on flood risk with the scale of the disaster being conveyed by overlaying the inundation depths on real life structures such as buildings, bridges, and notable landmarks. Potential applications of three-dimensional visualization are training of emergency officials and public outreach to warn a community of potential dangers. Figure 11-13 illustrates three-dimensional visualization for an inundation.

### 11.6.1  Annotation of Images

Three-dimensional visualization does not always require GIS and graphic software applications. A simple yet very effective means of communicating the magnitude of an inundation in three dimensions can involve the simple annotation of known landmarks with modeled high water marks associated with a dam breach. Such annotations can help bring the severity of a potential event into context for those not familiar with the potential consequences of a dam failure. This type of visualization can be executed with a minimum of resources; all that is needed is an inundation elevation or depth, an image of a landmark, and a means of referencing the elevation or depth onto the image.

The following example illustrates an annotated digital photograph of a local landmark in the fictional city of Floodville, USA. A common word processing application was used to annotate the digital photograph of the landmark, which in this example is a bridge. Bridges are a valuable landmark for communicating the risk of dam failure since they are familiar to users and are potentially one of the critical pieces of infrastructure at greatest risk for loss of life and failure during a dam failure or incident.

**Figure 11-13: Three-dimensional visualization of an inundation**

Figure 11-14 illustrates the image annotation. The annotation of "Flood level of August 1999" is provided to give users an additional point of reference. Experienced emergency responders, planners, and dam safety officials are likely to remember historical flooding events and the devastation they caused. Historical high water marks, when compared to the high water levels anticipated for a dam failure or incident, can often put the severity of a dam failure into perspective and help users imagine the sometimes unimaginable.

Other landmarks in the inundation zone may include public buildings such as city halls, libraries, and courthouses. For buildings, it is often helpful to annotate floor levels on an inundated building such as "Sunny Day Dam Failure Flooding to the 2nd floor of Floodville Courthouse." This can be helpful for situations when evacuation of the building may pose a greater risk than simply moving occupants to a higher floor. When warning times are short, vertical evacuation may be the preferred option for facilities such as hospitals and senior living accommodations where the population at risk has limited mobility. Vertical evacuation could be considered as part of a facility's EAP after careful review of factors including the flood severity and structural stability under these conditions.
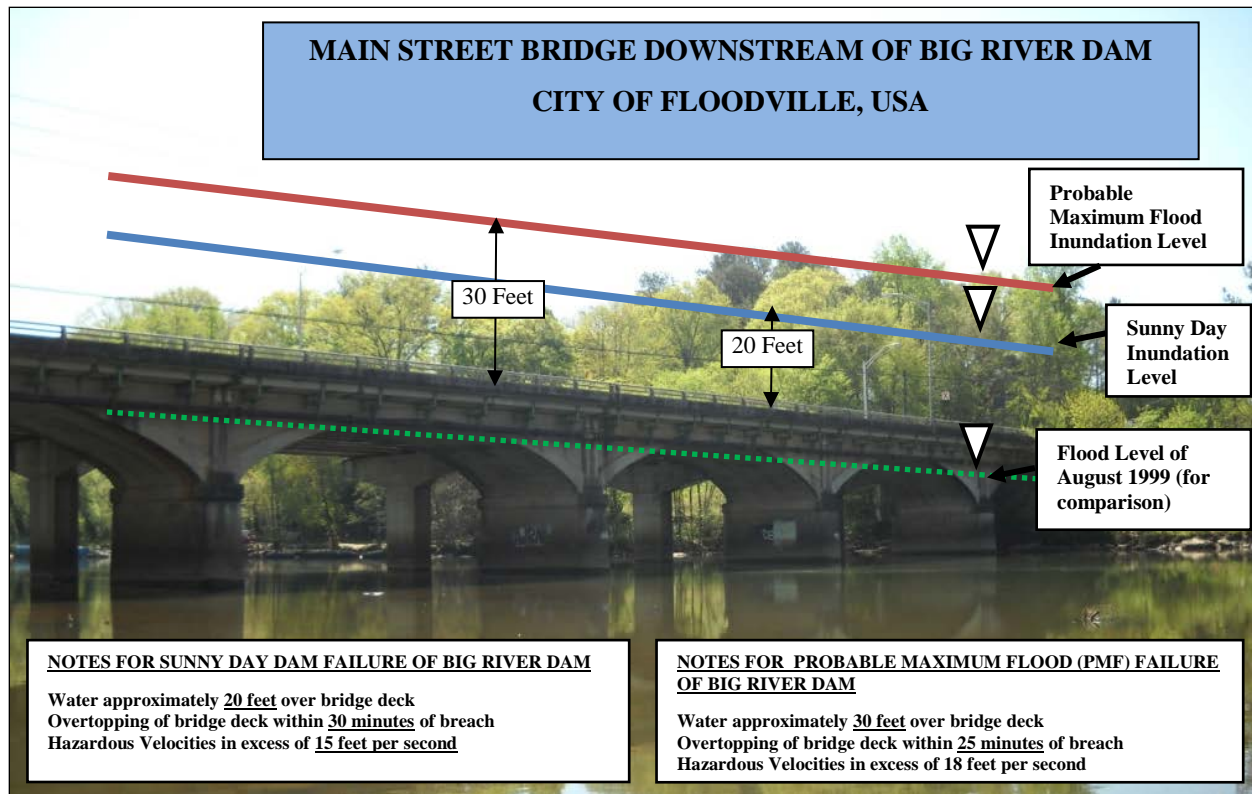
**MAIN STREET BRIDGE DOWNSTREAM OF BIG RIVER DAM**

**CITY OF FLOODVILLE, USA**

Probable Maximum Flood Inundation Level

Sunny Day Inundation Level

Flood Level of August 1999 (for comparison)

30 Feet

20 Feet

NOTES FOR SUNNY DAY DAM FAILURE OF BIG RIVER DAM

Water approximately **20 feet** over bridge deck
Overtopping of bridge deck within **30 minutes** of breach
Hazardous Velocities in excess of **15 feet per second**

NOTES FOR PROBABLE MAXIMUM FLOOD (PMF) FAILURE OF BIG RIVER DAM

Water approximately **30 feet** over bridge deck
Overtopping of bridge deck within **25 minutes** of breach
Hazardous Velocities in excess of 18 feet per second

**Figure 11-14: Image annotation**

## 11.6.2  Animation

Animation can be another valuable tool for communicating the magnitude and response time needed in the event of a dam failure. Many software applications used for dam break modeling provide varying degrees of functionality for animation. For example, HEC-RAS unsteady flow simulations allow the user to animate a flood or dam breach simulation at an individual cross-section or structure level, profile view, or X-Y-Z perspective view using HEC-RAS's built-in animation control (see Figures 11-15 and 11-16).

Two-dimensional models used for unsteady flow analysis frequently provide powerful plan view animations. Sometimes X-Y-Z perspective views and flyovers of flood or dam break inundation progression can also provide realistic insight into the magnitude and severity of a flooding event. Numerous online map applications also support the creation of animations, including flyovers.

Although animations would rarely be used in the event of an emergency unless given a large lead time, they can be invaluable tools for training staff identified within an EAP as responders. They can also be an important visualization tool for planners for use in training and in public outreach.
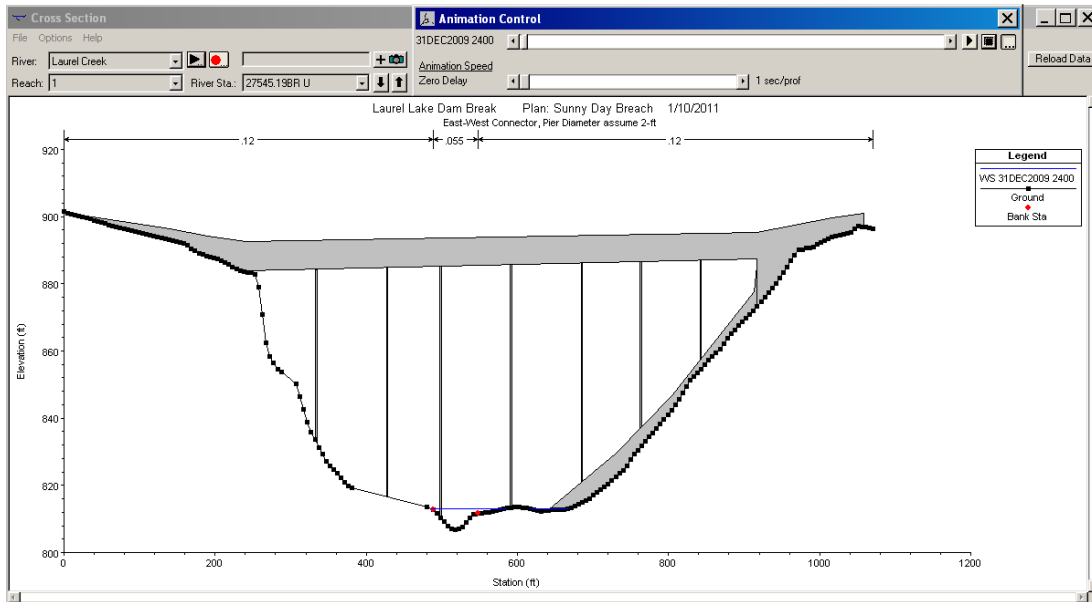
**Figure 11-15: HEC-RAS cross-section view with animation control activated**
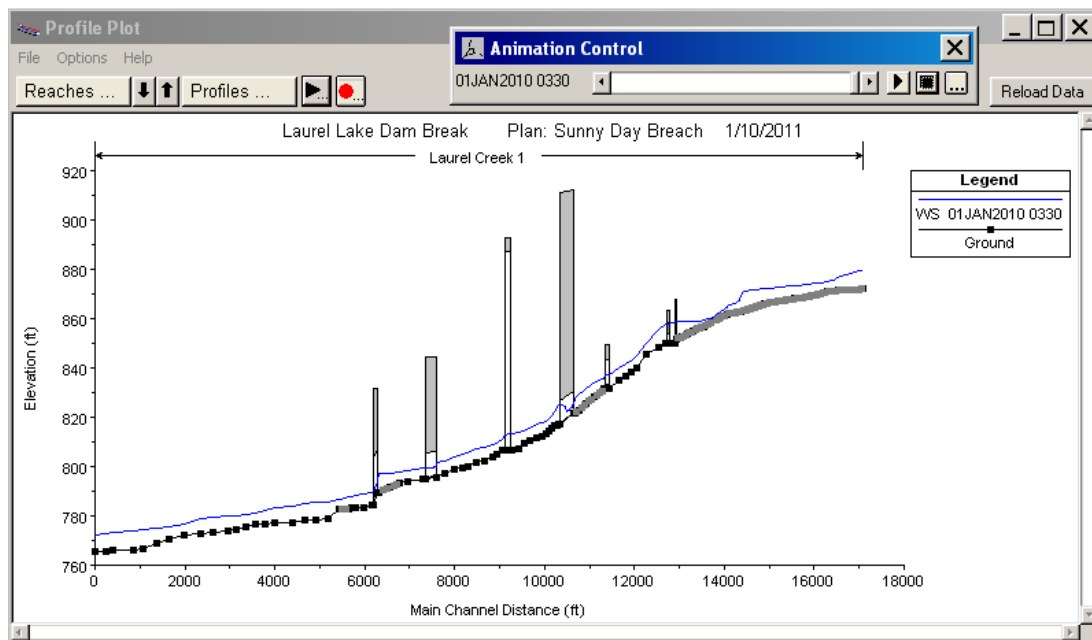


**Figure 11-16: HEC-RAS profile view with animation control activated**

**Appendix A**

**References and Resources**

## References

ASDSO (Association of Dam Safety Officials). 2010. *State Dam Safety Hazard Potential Classification*.

Australian National Committee on Large Dams Incorporated. 2000. *Guidelines on Assessment of the Consequences of Dam Failure*.

Babb and Memel. 1968. *Catalog of Dam Disaster, Failures and Accidents*. Bureau of Reclamation. Washington, DC.

Canadian Dam Association. 2007. *Technical Bulletin 9: Structural Considerations for Dam Safety*.

Canadian Dam Association. 2007a. *Technical Bulletin 1: Inundation, Consequences, and Classification for Dam Safety*.

Canadian Dam Association. 2007b. *Technical Bulletin 8: Geotechnical Considerations for Dam Safety*.

Comer, G., F. Theurer and H. Richardson. 1982. The Modified Attenuation-Kinematic (ATT-KIN) Routing Model. International Symposium on Rainfall-Runoff Modeling. Mississippi State University: Water Resources Publications.

Costa, J.E. 1985. *Floods from dam failures*. Denver: U.S. Dept. of the Interior, Geological Survey : Open-File Services Section, Western Distribution Branch.

Dewey, R.L. and D.R. Gillette. 1993. *Prediction of Embankment Dam Breaching for Hazard Assessment*. ASCE Specialty Conference on Geotechnical Practice in Dam Rehabilitation. Raleigh, NC.

DHI Software. 2007. *MIKE FLOOD: 1D-2D Modeling / User Manual*. Retrieved January 7, 2011 from http://www.hydroasia.org/jahia/webdav/site/hydroasia/shared/ Document_public/Project/Manuals/WRS/MIKE_FLOOD_UserManual.pdf.

DHS (Department of Homeland Security). 2011. *Dams Sector: Estimating Loss of Life for Dam Failure Scenarios.* September.

Evans, S. G. 1986. *The maximum discharge of outburst floods caused by the breaching of man-made and natural dams*. Canada Geotechnical Journal, 23, 385-387.

FEMA (Federal Emergency Management Agency). 1979, reprinted in 2004. *Federal Guidelines for Dam Safety* (FEMA 93). Washington, DC: FEMA.

FEMA. 2002. *Guidelines and Specifications for Flood Hazard Mapping Partners, Appendix B: Guidance for Converting to the North American Vertical Datum of 1988*. Final. February.

FEMA. 2004a. *Federal Guidelines for Dam Safety: Selecting and Accommodating Inflow Design Floods for Dams* (FEMA 94). Washington, DC: FEMA.

FEMA. 2004b. *Federal Guidelines for Dam Safety: Hazard Potential Classification System for Dams (*FEMA 333). Washington, DC: FEMA.FEMA. 2004b. *Federal Guidelines for Dam Safety: Selecting and Accommodating Inflow Design Floods for Dams*. Available at http://www.fema.gov/library/viewRecord.do?id=1828.

FEMA. 2004c. *Memorandum: Policy for Accepting Numerical Models for Use in the NFIP.*August. Available at http://www.fema.gov/library/viewRecord.do?id=3226.

FEMA. 2005. *The National Dam Safety Program Research Needs Workshop: Embankment Dam Failure Analysis* (FEMA 541). Washington, DC: FEMA.

FEMA (Federal Emergency Management Agency). 2008. *Risk-Based Dam Safety Prioritization – A Method for Easily Estimating the Loss of Life from Dam Failure, Appendix B*. March.

FEMA. 2010. *Why Dams Fail*. Retrieved January 12, 2011, from http://www.fema.gov/hazard/damfailure/why.shtm.

FERC (Federal Energy Regulatory Commission). 1988. USA Federal Regulatory Commission – Notice of Revised Emergency Action Plan Guidelines.

FERC. 1987, revised 1993. *Engineering Guidelines for the Evaluation of Hydropower Projects*.

FLO-2D. n.d. *FLO-2D: Two-Dimensional flood routing model that simulates river, alluvial fan, urban and coastal flooding*. Retrieved January 7, 2011, from http://www.flo-2d.com/products/flo-2d/.

FLO-2D. 2009. *FLO-2D Reference Manual*. Nutrioso, AZ: FLO-2D.

FLOODsite. 2007. FLOODsite Web site, BC Hydro: Life Safety Model (LSM). Sixth Framework Programme of the European Commission Integrated Project. Available online at http://www.floodsite.net/html/cd_task17-19/life_savety_model.html#1.

Foster, M.A., Fell, R., and Spannagle, M. 1998. *Analysis of embankment dam incidents*. UNICIV Report No. R-374, The University of New South Wales, School of Civil and Environmental Engineering.

Fread, D.L. 1979. *DAMBRK: The NWS Dam Break Flood Forecasing Model*. National Weather Service.

Fread, D. L. 1988."The NWS DAMBRK Model: Theoretical Background/User Documentation," National Weather Service, Silver Spring, Maryland, June 20, 1988.

Fread, D.L. 2001. *Some Existing Capabilities and Future Directions for Dam-Breach Modeling/Flood Routing*. Proceedings of the FEMA Workshop. Oklahoma City, OK.

Fread, D.L. 2006. ASDSO Advanced Technical Seminar, *Dam Failure Analysis*.

Fread, D. L., J.M. Lewis and S.M. Wiele. 1991. *The NWS Simplified DAM-BREAK Flood Forecasting Model.* Hydrologic Research Laboratory, National Weather Service, Silver Spring, MD.

Fread, D.L. and J. M. Lewis. 1998. *NWS FLDWAV Model: Theoretical Description and User Documentation.* Hydrologic Research Laboratory, Office of Hydrology, National Weather Service.

Froelich, D. 1995. *Embankment Dam Breach Parameters Revisited*. Journal of Water Resources Planning and Management, 121(1), 90-97.

Froehlich, D. 1995. *Peak Outflow from Breached Embankment Dam.* Journal of Water Resources Planning and Management Division, vol. 121, no. 1., p. 90-97.

Froehlich, D. 2008. *Embankment Breach Parameters and Their Uncertainties*. Journal of Hydraulic Engineering, 134(12), 1708-1721.

Gee, M. 2008. *Comparison of Dam Breach Parameter Estimators.* Corps of Engineers Hydrologic Engineering Center, 609. 2nd St., Davis, CA 95616.

Gee, M. 2009. *Comparison of Breach Parameter Estimators*. World Environmental and Water Resources Congress 2009: Great Rivers Proceedings.

Gee, D.M. 2010. *Use of Breach Process Models to Estimate HEC-RAS Dam Breach Parameters*. 2nd Joint Federal Interagency Conference, Las Vegas, NV.

Goodell, C. and B. Wahlin. 2009. *Dynamic and Level Pool Reservoir Drawdown: A Practical Comparison for Dam Breach Modeling*. Proceedings of the 33rd IAHR Congress. Vancouver, Canada.

Graham. 1983. "Physical and breach parameters for dams with large storage and height ratios." Unpublished internal document, USBR Denver CO.

Hagen, V. K. 1982. *Re-evaluation of design floods and dam safety*. Proceedings of the 14th Congress of International Commission on Large Dams, International Commission on Large Dams. Paris, France.

Johnson, F.A., and P. Illes. 1976. *A Classification of Dam Failures*. International Water Power and Dam Construction. December.

Kirkpatrick, G. 1977. *Evaluation Guidelines for Spillway Adequacy*. The Evaluation of Dam Safety, Engineering Foundation Conference, Pacific Grove, California, ASCE.

MacDonald, T. C. and J. Langridge-Monopolis. 1984. *Breaching Characteristics of Dam Failures*. Journal of Hydraulic Engineering, 110(5), 576-586. ASCE.

Merkel, W. H. 2002. *Muskingum-Cunge Flood Routing Procedure in NRCS Hydrologic Models*. Proceedings of the Second Federal Interagency Hydrologic Modeling Conference.

Ming, J. and D.L. Fread. 1997. *One-Dimensional Routing of Mud/Debris Flows Using NWS FLDWAV Model*. Proceedings of the First International Conference on Debris-Flow Hazards Mitigation. San Francisco, CA: USGS.

MRLC (Multi-Resolution Land Characteristics Consortium). 2006. National Land Cover Database 2006 Web Site. Available at http://www.mrlc.gov/nlcd2006.php.

MSHA (Mine Safety and Health Administration). 2009. *Engineering and Design Manual: Coal Refuse Disposal Facilities*.

NDSRBB (National Dam Safety Review Board Emergency Action Plan Workgroup). 2009. *Simplified Inundation Maps for Emergency Action Plans*.

NOAA (National Oceanic and Atmospheric Administration). 2004a. Precipitation-Frequency Atlas of the United States, Semiarid Southwest. *NOAA Atlas 14*, *Volume 1*.

NOAA. 2004b. Precipitation-Frequency Atlas of the United States, Ohio River Basin and Surrounding States. *NOAA Atlas 14, Volume 2*.

NOAA. 2006. Precipitation-Frequency Atlas of the United States, Puerto Rico and the U.S. Virgin Islands. *NOAA Atlas 14, Volume 3*.

NOAA. 2009a. Precipitation-Frequency Atlas of the United States, Hawaiian Islands. *NOAA Atlas 14, Volume 4*.

NOAA. 2009b. Precipitation-Frequency Atlas of the United States, Selected Pacific Islands. *NOAA Atlas 14, Volume 5*.

NPDM (National Performance of Dams Program). n.d. *Chronology of Major Events in Dam Safety*. Stanford University. Retrieved January 10, 2011, from http://npdp.stanford.edu/chronology.html.

NPDM. 2011. *NPDP Dam Incident Query*. Retrieved January 25, 2011, from http://npdp.stanford.edu/DamDirectory/Dam IncidentQuery/IncidentForm.jsp.

NRCS (National Resource and Conservation Service). 1982. *National Engineering Manual: 210-V*.

NRCS. 2007. *SITES 2005 Water Resource Site Analysis Computer Program: User Guide*. Retrieved January 7, 2011, from http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=18657.wba.

NRCS. 1999. *National Engineering Manual: 210-V* Amendment IN31. December.

NWS (National Weather Service). 2009. *Current NWS Probable Maximum Precipitation (PMP) Documents and Related Studies*. National Oceanic and Atmospheric Administration, National Weather Service, Hydrometeorological Design Studies Center. Retrieved January 27, 2011, from http://www.nws.noaa.gov/oh/hdsc/studies/pmp.html.

NWS. 2009. *Transition from FLDWAV to HEC-RAS; Forecast Implications and Transition Tools*. National Oceanic and Atmospheric Administration, National Weather Service, Office of Hydrologic Development. Retrieved January 7, 2011, from http://www.nws.noaa.gov/oh/hrl/hsmb/hydraulics/FLDWAV2HEC-RAS.html.

NWS. 2012. Current NWS Precipitation Frequency (PF) Documents and Related Studies: NOAA Atlas 14, All Volumes. Available at http://www.nws.noaa.gov/oh/hdsc/currentpf.htm. Site accessed November 21, 2012.

Peterson, M.D. and others. 2008. 2008 United States National Seismic Hazard Maps: U.S. Geological Survey Fact Sheet 2008-3018, 2p.

Ponce, V. M., A. K. Lohani and C. Scheyhing. 1996. *Analytical verification of Muskingum-Cunge routing*. Journal of Hydrology, 174(3-4), 235-241.

Ponce, V. M. and D. B. Simons. 1977. *Shallow wave propagation in open channel flow*. Journal of the Hydraulics Division, 103(12), 1461-1476.

Powledge, George R., D.C. Ralston, P. Miller, Y.H. Chen, P.E. Clopper, and D.M. Temple. 1989. *Mechanics of Overflow Erosion on Embankments. II: Hydraulic and Design Considerations*. Journal of Hydraulic Engineering, vol. 115, no. 8, August, p. 1056-1075.

Ralston, David C. 1987. *Mechanics of Embankment Erosion During Overflow*. Hydraulic Engineering, Proceedings of the 1987 ASCE National Conference on Hydraulic Engineering, Williamsburg, Virginia, p. 733-738.

Reed, S., Moreda, F., Gutuirrez, A., and Aschwanden, C. 2009. *Guidelines for the Transition from FLDWAV to HEC-RAS; Forecast Implications and Transition Tool*.

Rogers, J. David and Conor M. Watkins. 2008. *Overview of the Taum Sauk Pumped Storage Power Plant Upper Reservoir Failure, Reynolds County, MO.* Published in Sixth International Conference on Case Histories in Geotechnical Engineering, Arlington, VA. August 11-16, 2008.

SCS (Soil Conservation Service). 1981. *Simplified Dam Breach Routing Procedure, Technical Release No. 66 (Rev. 1).*

SCS (Soil Conservation Service). 1985. *Technical Release No. 66 (Third Edition): Simplified Dam Breach Routing Procedure.*

Stedinger, J., D. Heath and K. Thompson. 1996. *Risk Analysis for Dam Safety Evaluation: Hydrologic Risk*. U.S. Army Corps of Engineers.

Singh and Scarlatos. 1988. Analysis of Gradual Earth-Dam failure, Journal of Hydraulic Engineering, Vol 114 No. 7 1988 p2 21-42

Singh, K. P. and A. Snorrason. 1982. *Sensitivity of Outflow Peaks and Flood Stage to the Selection of Dam Breach Parameters and Simulation Models*. Journal of Hydrology, 68, 295-310.

Singh, K. P. and A. Snorrason. 1984. *Sensitivity of outflow peaks and flood stage to the selection of dam breach parameters and simulation models.* Journal of Hydrology, 68, 295-310.

State of Colorado Department of Natural Resources. 2010. *Guidelines for Dam Breach Analysis*. Division of Water Resources. Retrieved January 19, 2011, from http://water.state.co.us/DWRIPub/Documents/GuidelinesForDamBreachAnalysis021010.pdf.

Stelling, G.S. 1984/ *On the Construction of Computational Methods for Shallow Water Flow Problems* Rijkswaterstaat Communications, No. 35/1984, The Hague, The Netherlands

Sylvestre, J. and P. Sylvestre. n.d. *FLDWAV Application: Transitioning from Calibration to Operational Mode*. Silver Spring, MD: National Oceanic and Atmospheric Administration, National Weather Service, Office of Hydrologic Development, Hydrology Laboratory.

University of Vermont, Landscape Change Program. 2004. *Dams and Reservoirs*. Retrieved January 11, 2011, from http://www.uvm.edu/landscape/dating/ dams_and_ reservoirs/.

USACE (U.S. Army Corps of Engineers) and ANCOLD (Australian National Committee on Large Dams). 2005. *LifeSim-A model for Estimating Dam Failure Loss*. Draft. By Megend Aboelata and David S. Bowles Institute for Dam Safety Risk Management, Utah State University, Logan, Utah.

USACE. 1979. *Recommended Guidelines for Safety Inspection of Dams* (ER 1110-2-106). Retrieved January 25, 2011, from http://www.mde.state.md.us/programs/Water/DamSafety/TechnicalReferences/Documents/www.mde.state.md.us/assets/document/damsafety/COE/Guidelines%20for%20Safety%20Inspections%20of%20Dams.pdf.

USACE. 1980. Hydraulic Engineering Center, Flood Engineering Plans-Guidelines for Corps Dams, RD 13, June 1970

USACE (U.S. Army Corps of Engineers). 1995. *Gravity Dam Design*. Hyattsville, MD: U.S. Army Corps of Engineers Publication Depot.

USACE. 1997. *Engineering and Design Dam Safety Assurance Program* (ER 1110-2-1155). Retrieved January 12, 2011, from http://140.194.76.129/publications/eng-regs/er1110-2-1155/entire.pdf.

USACE. 1998. *HEC-1 Flood Hydrograph Package / User's Manual*. Retrieved January 7, 2011, from http://www.hec.usace.army.mil/software/legacysoftware/hec1/hec1.htm.

USACE. 2007. *Engineering & Design-Earthquake design and Evaluation of Concrete Hydraulic Structures*, EM 1110-2-6051 May 2007.

USACE. 2009. *The National Inventory of Dams.* Retrieved December 20, 2010, from http://nid.usace.army.mil/.

USACE. 2011. *Modeling, Mapping, and Consequence Production Center Standard Operating Procedures (FINAL-DRAFT) MCC Steering Committee, Headquarters*. April. Unpublished (USACE internal draft).

USACE. 2012. HEC-RAS Web site. Available online at http://www.hec.usace.army.mil/software/hec-ras/. Accessed November 20.

USBR (U.S. Bureau of Reclamation). 1982. *Guidelines for Defining Inundated Areas Downstream from Bureau of Reclamation Dams*. Reclamation Planning Instruction No. 82-11.

USBR. 1986. *Guidelines to Decision Analysis*. Issue 7 of ACER Technical Memorandum. U.S. Department of the Interior.

USBR. 1988. *ACER Technical Memorandum No. 11: Downstream Hazard Classification Guidelines.* Denver, CO: USBR.

USBR. 1998. *Prediction of embankment dam breach parameters- A literature review and needs assessment*. Written by Wahl, T. DSO-98-004 Dam Safety Office, U.S. Department of the Interior.

USBR. 1999. *A Procedure for Estimating Loss of Life Caused by Dam Failure, DSO-99-06. Sedimentation & River Hydraulics*. Dam Safety Office. September.

USDOI (U.S. Department of the Interior). 2004. *Department Manual Part 753: Dam Safety and Security Program*. Special Programs Series. Effective Date October 27. Originating Office: Bureau of Reclamation.

USDOI. 2011. *Reduce Dam Safety Risk Modernization Blueprint / Implementation Phase 1: Launch Risk Reduction / Inundation Mapping/Modeling Subproject Report*. U.S. Department of the Interior Enterprise Architecture. March, 2011.

Von Thun, J. L. and D. R. Gillette. 1990. *Guidance on Breach Parameters*, [Internal Memorandum]. Denver, CO: U.S. Department of the Interior, Bureau of Reclamation.

Wahl, T. L. 1997. *Predicting Embankment Dam breach Parameters – A Needs Assessment*. Proceedings of the U.S. Bureau of Reclamation, XXVIIth IAHR Congress. San Francisco, CA.

Wahl, T. L. 2004. *Uncertainty of Predictions of Embankment Dam Brach Parameters*. Journal of Hydraulic Engineering, 130(5), 389-397.

Wahl, T. 2009. "Evaluation of New Models for Simulating Embankment Dam Breach," Dam Safety '09, Annual Meeting of the Association of State Dam Safety Officials (ASDSO), Sept. 27-Oct. 1, 2009, Hollywood, FL.

Wahl, T. 2010. *Dam Breach Modeling- An Overview of Analysis Methods.* Published in the Joint Federal Interagency Conference on Sedimentation and Hydrologic Modeling, June 27 – July 1, 2010, Las Vegas, NV. Published by Tony L. Wahl, Hydraulic Engineer, U.S. Bureau of Reclamation, Denver, CO. Retrieved January 19, 2011, from http://www.usbr.gov/pmts/hydraulics_lab/pubs/PAP/PAP-1018.pdf.

Wahl, Tony L., Jean-Robert Courivaud, Rene Kahawita, Gregory J. Hanson, Mark W. Morris, and Jeffrey T. McClenathan. 2008. *Development of Next-Generation Embankment Dam Breach Models*. Retrieved January 19, 2011, from http://www.usbr.gov/pmts/hydraulics _lab/pubs/PAP/PAP-0978.pdf.

Walder, J. S. and J.E. O'Connor. 1997. *Methods for Predicting Peak Discharge of Floods Caused by Failure of Natural and Constructed Earth Dams*. Water Resources Research, 33(10), 12.

Wetmore, J. N. and D.L. Fread. 1981. *The NWS Simplified Dam-Break Flood Forecasting Model*. Proceedings of the 5th Canadian Hydrotechnical Conference.

Wetmore et al. 1991. *The NWS Simplified Dam-Break Flood Forecasting Model.* Retrieved January 5, 2011, from http://www.rivermechanics.net/models/ smpdbk.pdf.

Wurbs. R.A. 1987. "Dam-Breach Flood Wave Models," *Journal of Hydraulic Engineering*, Vol. 113, No. 1, p. 29-46.

## *Resources*

Kirkpatrick, G. W. 1976. *Evaluation guidelines for spillway adequacy.* Proceedings from the Engineering Foundation Conference, The Evaluation of Dam Safety (pp. 395-414). ASCE.

NRCS. 2005. *Technical Release 60: Earth Dams and Reservoirs.*

Pierce, M. W., C. I. Thornton, S. R. Abt, 2010. *Predicting Peak Outflow from Breached Embankment Dams*. ASCE Journal of Hydrologic Engineering Vol. 15(1):338-349.USBR (U.S. Bureau of Reclamation). 1976. *Design of Gravity Dams.*

USBR. 1977. *Design of Arch Dams*.

USBR. 1987. *Design of Small Dams*.

USDA (U.S. Department of Agriculture). 2001. Proceedings, FEMA Workshop, *Issues, Resolutions and Research Needs to Embankment Dam Failure Analysis,* June 26–28. Oklahoma City, OK.

USDOI (U.S. Department of the Interior). 2011. *Reduce Dam Safety Risk Modernization Blueprint / Implementation Phase 1: Launch Risk Reduction / Inundation*

*Mapping/Modeling Subproject Report*. U.S. Department of the Interior Enterprise Architecture. March, 2011.

Xu, Y. and L. M. Zhang. 2009. *Breaching Parameters for Earth and Rockfill Dams*. ASCE Journal of Geotechnical and Geoenvironmental Engineering Vol. 135(12):1957-1970.

### *Models*

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| BEED | Breach Erosion of Earth-Fill Dams and Flood Routing, |
| BRDAM | U.S. Geological Survey |
| BREACH | National Weather Service |
| CCHE2D-FLOOD | National Center for Computational Hydroscience and Engineering at the University of Mississippi |
| DAMBRK | Dam Break Forecasting Model, National Weather Service |
| DSAT | Dams Sector Analysis Tool, Department of Homeland Security and U.S. Army Corps of Engineers |
| DSS-WISE | Decision Support System for Water Infrastructural Security, National Center for Computational Hydroscience and Engineering at the University of Mississippi |
| DWOPER | Dynamic Wave Operational Models, National Weather Service |
| FIREBIRD-BREACH | Collaborative effort, under development by HR Wallingford of the UK with the Dam Safety Interest Group (DSIG) of CEATI International, Inc. |
| FLDWAV | Flood Wave Dynamic Model, National Weather Service |
| FLDVIEW | Flood Forecast Mapping Application, National Weather Service |
| FLO-2D$^{©}$ | Flow Two-Dimensional, FLO-2D Software, Inc. |
| GeoDamBREACH | Geospatial Dam Break, Emergency Action Planning, Consequences, and Hazards, Federal Emergency Management Agency |
| HEC-1 | Hydrologic Engineering Center One, U.S. Army Corps of Engineers |
| HEC-RAS | Hydrologic Engineering Center's River Analysis System, U.S. Army Corps of Engineers |
| HEC-HMS | Hydrologic Engineering Center's Hydrologic Modeling System, U.S. Army Corps of Engineers |
| HR-BREACH | European FLOODsite Project |
| MIKE 21$^{©}$ | DHI Software |
| MIKE FLOOD$^{©}$ | DHI Software |
| MOUSE$^{©}$ | DHI Software |
| LIFESim | Utah State University |

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| LSM | Life Safety Model, BC Hydro |
| SIMBA | SIMplified Breach Analysis, Natural Resources Conservation Service |
| SITES | Water Resource Site Analysis Computer Program, Natural Resources Conservation Service |
| SMPDBK | Simplified Dam Break Method, National Weather Service |
| TR-66 | Technical Release Number 66, Natural Resources Conservation Service |
| WinDAM | Windows Dam Analysis Modules, Agricultural Research Service |
| XPSWMM/XPStorm 2D | XP Solutions |

**Appendix B**

**Glossary**

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| **One-dimensional hydraulic model** | One-dimensional hydraulic modeling considers flow variations in one direction (i.e., the y-direction) at each river cross-section. |
| **Two-dimensional hydraulic model** | Two-dimensional hydraulic modeling considers flow variations in two directions (i.e., the x- and y-direction) at each river cross-section. |
| **Bathymetric survey** | Underwater survey of the reservoir floor. |
| **Breach** | An opening through a dam that allows the uncontrolled draining of a reservoir. A controlled breach is a constructed opening. An uncontrolled breach is an unintentional opening caused by discharge from the reservoir. A breach is generally associated with the partial or total failure of the dam. Often used interchangeably with "failure" in document. |
| **Breach depth** | The vertical extent of the breach measured from the dam crest down to the invert of the dam breach. Some publications cite the reservoir head on the breach, measured from the reservoir water surface to the breach invert. |
| **Breach formation time or time-to-failure** | The time of failure as used in DAMBRK 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 and the resulting crevasse has progressed back across the width of the dam crest to reach the upstream face. |
| **Breach hydrograph** | A graph showing the discharge from a dam breach over time. |
| **Breach parameter** | Parameters that define the breach geometry and formation time. Common breach parameters include: breach depth, breach height, breach side slopes and breach formation time. |
| **Breach progression** | Progression in which dam embankment material is removed from the structure due to dam failure. |
| **Breach side slope** | The breach side slope is a measure of the angle of the ultimate breach sides and is typically described as horizontal to 1 vertical (H:1V) |
| **Breach width** | The average ultimate breach width typically measured at the vertical center of the breach. |
| **Concrete dam** | A dam constructed from concrete. There are several types of concrete dams ranging from conventional design styles such as gravity, arch, multi-arch, and buttress dams to newer approaches in design such as roller compacted concrete. |
| **Concurrent inflows** | Flows expected on tributaries to the river system downstream of the dam at the same time a flood inflow occurs. |
| **Cross-section** | A section formed by cutting a plane through an object, usually perpendicular to an axis. |
| **Dam** | An artificial barrier that has the ability to impound water, wastewater, or any liquid-borne material, for the purpose of storage or control of water. |

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| **Dam failure** | Catastrophic type of failure characterized by the sudden, rapid, and uncontrolled release of impounded water or the likelihood of such an uncontrolled release. There are lesser degrees of failure and any malfunction or abnormality outside the design assumptions and parameters that adversely affect a dam's primary function of impounding water is properly considered a failure. These lesser degrees of failure can progressively lead to or heighten the risk of a catastrophic failure. They are, however, normally amenable to corrective action. |
| **Dam size classification system** | A system that categorizes dams according to the storage capacity and/or height of the dam. |
| **Deterministic methodology** | A method in which the chance of occurrence of the variable involved is ignored and the method or model used is considered to follow a definite law of certainty, and not probability. |
| **Drainage area** | The area that drains to a particular point on a river or stream. |
| **Dynamic routing** | Hydraulic flow routing based on the solution of the St. Venant equation(s) to compute the changes of discharge and stage with respect to time at various locations along a stream. |
| **Embankment dam** | Any dam constructed of excavated natural materials (includes both earth-fill and rock-fill dams). |
| **Emergency Action Plan** | A plan of action to be taken to reduce the potential for property damage and loss of life in an area affected by a dam failure or large flood. |
| **Erosion** | The wearing away of a surface (bank, streambed, embankment) by floods, waves, wind, or any other natural process. |
| **Exceedance duration elevation** | U.S. Army Corps of Engineers term to define the elevation in a reservoir that is exceeded a certain percentage of the time. For example, a 10 percent exceedance duration elevation for a particular reservoir is exceeded only 10 percent of the time. |
| **Failure mode** | A potential failure mode is a physically plausible process for dam failure resulting from an existing inadequacy or defect related to a natural foundation condition, the dam or appurtenant structures design, the construction, the materials incorporated, the operations and maintenance, or aging process, which can lead to an uncontrolled release of the reservoir. |
| **Flood** | A temporary rise in water surface elevation resulting in inundation of areas not normally covered by water. Hypothetical floods may be expressed in terms of average probability of exceedance per year such as 1-percent-chance flood, or expressed as a fraction of the probable maximum flood or other reference flood. |
| **Floodplain** | The downstream area that would be inundated or otherwise affected by the failure of a dam or by large flood flows. |
| **Flood routing** | A process of determining progressively the amplitude of a flood wave as it moves past a dam and continues downstream. |
| **Flood storage** | Storage volume in the reservoir exclusively allocated for regulation of flood inflows which is the storage in between the top of active storage( above normal pool operating level) and the top of conservation (top of dam) storage. |
| **Foundation** | The portion of the valley floor that underlies and supports the dam structure. |

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| **Freeboard** | Vertical distance between a specified stillwater reservoir surface elevation and the top of the dam, without camber. |
| **Gravity dam** | A dam constructed of concrete and/or masonry, which relies on its weight and internal strength for stability. |
| **Hazard potential** | The possible adverse incremental consequences that result from the release of water or stored contents due to failure of the dam or misoperation of the dam or appurtenances. Impacts may be for a defined area downstream of a dam from flood waters released through spillways and outlet works of the dam or waters released by partial or complete failure of the dam. There may also be impacts for an area upstream of the dam from effects of backwater flooding or landslides around the reservoir perimeter. |
| **Hazard potential classification** | A system that categorizes dams according to the degree of adverse incremental consequences of a failure or misoperation of a dam. The hazard potential classification does not reflect in any way on the current condition of the dam (i.e., safety, structural integrity, and flood routing capacity). |
| **Hydrograph, Flood** | A graphical representation of the flood discharge with respect to time for a particular point on a stream or river. |
| **Hydrologic breach** | A dam breach associated with a rain event and/or flooding. |
| **Incremental hazard evaluation** | The incremental hazard evaluation is used to determine the inflow design flood. The hazard potential is determined for incrementally larger flood flow conditions until the incremental increase in consequences due to failure is acceptable and it is apparent that a larger flood inflow would not result in an incremental increase in consequences due to failure, or up to a point where the hydrologic event is the probable maximum flood. |
| **Inflow design flood** | The flood flow above which the incremental increase in downstream water surface elevation due to failure of a dam or other water impounding structure is no longer considered to present an unacceptable threat to downstream life or property. The flood hydrograph used in the design of a dam and its appurtenant works particularly for sizing the spillway and outlet works and for determining maximum storage, height of dam, and freeboard requirements. |
| **Inundate** | To overflow, to flood. |
| **Inundation map** | A map showing areas that would be affected by flooding from an uncontrolled release of a dam's reservoir. |
| **Level pool routing** | Reservoir routing that assumes the water surface in the reservoir remains flat. |
| **Liquefaction** | A condition whereby soil undergoes continued deformation at a constant low residual stress or with low residual resistance, due to the buildup and maintenance of high pore water pressures, which reduces the effective confining pressure to a very low value. Pore pressure buildup leading to liquefaction may be due either to static or cyclic stress applications and the possibility of its occurrence will depend on the void ratio or relative density of a cohesionless soil and the confining pressure. |
| **Meteorology** | The science that deals with the atmosphere and atmospheric phenomena, the study of weather, particularly storms and the rainfall they produce. |

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| **Normal reservoir level** | For a reservoir with a fixed overflow sill the lowest crest level of that sill. For a reservoir whose outflow is controlled wholly or partly by moveable gates, siphons or other means, it is the maximum level to which water may rise under normal operating conditions, exclusive of any provision for flood surcharge. |
| **Normal reservoir pool elevation** | The normal operating water elevation, typically the same elevation as the lowest outlet (i.e., primary spillway). |
| **Normal reservoir storage** | Reservoir storage volume when the water surface elevation is at normal pool. |
| **Overtopping failure** | A hydrologic dam failure that occurs as a result of the water level in the reservoir exceeding the height of the dam. |
| **Parametric regression equation** | Equations that use case study information to estimate time-to-failure and ultimate breach geometry then simulate breach growth as a time-dependent linear process and compute breach outflows using principles of hydraulics. |
| **Peak flow** | The maximum instantaneous discharge that occurs during a flood. It is coincident with the peak of a flood hydrograph. |
| **Physically based models** | Models that predict the development of an embankment breach and the resulting breach outflows using an erosion model based on principles of hydraulics, sediment transport, and soil mechanics (e.g., NWS BREACH). |
| **Piping failure** | Dam failure caused when concentrated seepage develops within an embankment dam and erodes to form a "pipe." Piping typically occurs in two phases: formation of the "pipe" and the subsequent collapse of the dam crest. It is possible for the reservoir to drain before the dam crest collapses. |
| **Predictor regression equations** | Equations that empirically estimate peak discharge based on case study data and assume a reasonable outflow hydrograph shape. |
| **Probable loss of life** | The probable loss of life due to inundation caused by dam failure and is often determined based on how many habitable structures and roads are located in the inundated area. |
| **Probable maximum flood (PMF)** | The flood that may be expected from the most severe combination of critical meteorological and hydrologic conditions that is reasonably possible in the drainage basin under study. |
| **Probable maximum precipitation (PMP)** | Theoretically, the greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographical location during a certain time of the year. |
| **Reservoir** | A body of water impounded by a dam and in which water can be stored. |
| **Roller-compacted concrete (RCC)** | Concrete composed of water, cement and aggregate with essentially no slump. RCC generates less heat during curing than other conventional concretes. |
| **Routing** | A mathematical procedure for predicting characteristics of a flood wave (such as velocity, Froude number, height, discharge, etc.) as a function of time at one or more points along a waterway or channel. |
| **Seismic failure** | Dam failure caused by earth movements such as earthquakes. |

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| **Sensitivity analysis** | An analysis in which the relative importance of one or more of the variables thought to have an influence on the phenomenon under consideration is determined. |
| **Spillway** | A dam structure that allows water to discharge from a reservoir when the water level exceeds the top of the spillway. |
| **Spillway, Auxiliary** | See Spillway, Emergency. |
| **Spillway, Emergency** | A spillway that provides additional discharge capacity to the principal spillway's design discharge in the event of extreme weather or other emergency conditions. |
| **Spillway, Principal** | A spillway designed to pass normal flow conditions through a reservoir. |
| **Spillway, Vegetated** | A spillway that is lined with erosion-resistant vegetation. |
| **Steady flow** | All fluid flow properties such as velocity, temperature, pressure, and density are independent of time. |
| **Stone dam** | A dam constructed from rocks, boulders, or stone. |
| **Structural failure** | Dam failure caused by failure of the main embankment or appurtenant structure. |
| **Sunny day breach** | A dam breach that is not associated with a hydrologic event. |
| **Timber dam** | A dam constructed from timber. |
| **Topographic map** | A detailed graphic delineation (representation) of natural and man-made features of a region with particular emphasis on relative position and elevation. |
| **Tributary** | A stream that flows into a larger stream or body of water. |
| **Unit hydrograph** | A hydrograph with a volume of one inch of direct runoff resulting from a storm of a specified duration and areal distribution. Hydrographs from other storms of the same duration and distribution are assumed to have the same time base but with ordinates of flow in proportion to the runoff volumes. |
| **Unsteady flow** | All fluid flow properties such as velocity, temperature, pressure, and density are a function of time. |
| **Watershed** | The area drained by a river or river system. |