## Working Group on
### Dam Issues Related to Floodplain Management

Sub-group on
Modeling Tools for Dam Break Analysis
by
Mustafa Altinakar
November 11, 2008

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</tr>
</tbody>
</table>

<table>
<thead>
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</tr>
</thead>
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<table>
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<tr>
<th>Saleem Ashraf, Ph.D., P.E.</th>
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<td>MAPMOD Team</td>
<td></td>
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<td>Michael Baker Jr., Inc.</td>
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Current status of Dam Safety in the USA

In the USA, Homeland Security Presidential Directive 7 of 5/7/2007 classified dams and levees among the 17 critical infrastructure and key resource sectors\(^1\) “that require protective actions to prepare for, or mitigate against, a terrorist attack or other hazards identified.” These 17 critical sectors are summarized in Fig. 1.

Failure of dams and levees may lead to highly dynamic catastrophic floods that can cause significant loss-of-life, and bring considerable socio-economic hardship by damaging property and infrastructures. Floods, be it fluvial or due to failure of control structures (such as dams or levees), has the potential to affect one or more of these critical sectors. Moreover, pollution caused by cascading failure of hazardous chemical production and/or storage centers may also lead to environmental disasters and affect the ecosystem. Floods due to failure of dams need to be considered as an important element of flood plain management.

![Diagram of critical infrastructure sectors](image)

Fig. 1 Diagram showing the 17 critical infrastructure areas originally listed in the USA, Homeland Security Presidential Directive 7 of 5/7/2007. Floods, be it fluvial or due to failure of control structures (such as dams or levees), has the potential to affect one or more of these critical sectors.

The database of the National Dam Performance Program (NPDP, 2008a) lists a total of 1019 dam failures of varying importance since 1850 (see Fig. 2). A total of about 4,000 lives were lost

---

\(^1\) Later “Manufacturing” was added to the list as the 18th critical infrastructure and key resource sector.
during these incidents (Fig. 3). The majority of these failed dams are earthfill embankments. As shown in Fig. 4 (NPDP, 2008b), the principal cause of dam failures is the overtopping due to extreme hydrologic events. The second important cause is the piping.

US National Inventory of Dams (NID, 2008) currently lists about 80,443 dams in the USA, including those in Guam and Puerto Rico. Fig. 5 shows a pie chart regarding the ownership of these dams. As it can be seen, only about 5% of these dams are owned by the federal government. The majority of the remaining dams are privately owned and they are under the responsibility of the states. This leads to the important conclusion that the dam safety and security is not only a federal, state, or local issue. It also requires collaboration of private dam owners. This peculiar situation, where the responsibilities are divided among numerous stakeholders, requires an extremely careful approach to dam safety and security.

![Cumulative Number of Dam Failure: 1848-2005](image)

**Fig. 2** Cumulative number of dam failures on record in the NPDP database\(^2\). The dashed red line shows the cumulative number of failures excluding the many small dams that failed during the 1994 Georgia floods.

In the United States, all dams are classified into three hazard levels based on the vulnerabilities downstream and the expected impact of a failure, rather than the quality of their structure and/or the probability of their failure:

- A High Hazard (or Category I or Class C) dam is a dam whose failure may cause loss-of-life, serious damage to homes, industrial or commercial buildings, important public utilities, main highways, or railroads.

A Significant Hazard (Category II, or Class B) dam is a dam whose failure poses no threat to life, but may cause significant damage to main roads, minor roads, or cause interruption of public utilities’ services.

A Low Hazard (Category III, or Class A) dam is a dam whose failure would at most result in damage to agricultural land, farm buildings (excluding residences), or minor roads.

Fig. 3 Cumulative number of fatalities due to dam failures during the period 1850-2005 (adapted from NPDP, 2008b).

Fig. 4 Causes of all dam failures that occurred during the period 1975-2001 (adapted from NPDP, 2008b)
Although federal guidelines exist (FEMA, 2004a), hazard potential classification for dams is ultimately under the responsibility of states. Therefore, small variations should be expected from one state to the other.

Out of the 80,443 existing dams, 11,768 are classified as high hazard and 13,578 as significant hazard dams. The remaining 55,097 dams are low-hazard dams. The Table 1, lists the number of dams in each category by state based on the information contained in the NID.

In principle, all high hazard dams are required to have an Emergency Action Plan (EAP). Federal guidelines for dam safety published by FEMA (2004b) define EAP as “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 also contains inundation maps as well as the procedures and information to assist the dam owner/operator in issuing early warning and notification messages to responsible downstream emergency management authorities.

An analysis of the data in the NID reveals that 5,035 high hazard dams and 6,013 significant hazard dams do not yet have an EAP do not yet have an EAP. This is a serious situation, which requires immediate attention, especially considering potential loss-of-life.

In Figs 2.6 and 2.7, the EAP status of high hazard and significant hazard dams is plotted based on height and age of the dam. It is observed that 1,858 high-hazard dams and 2,533 significant hazard dams were built before 1940, thus have probably reached the end of their useful life. It can also be seen that a large number of high hazard dams that do not have and EAP fall into the category of small dams with a height less than 49 ft. This is in fact the category for which most of the dam failures have been observed.
Table 1  Number of dams in high- (H), significant- (S) and low-hazard (L) categories listed by state.

<table>
<thead>
<tr>
<th>State</th>
<th>Total Dams</th>
<th>Dams H</th>
<th>Dams S</th>
<th>Dams L</th>
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<td>163</td>
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<td>862</td>
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<td>DELAWARE</td>
<td>61</td>
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<td>MASSACHUSETTS</td>
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The situation is in fact probably more serious than it appears from these statistics based on the public portion of the NID. The first issue concerns the validity of the existing EAPs. The public version of the NID, unfortunately, does not list the date of establishment for the existing EAPs. Some of the existing EAPs may be very old and not reflect the true vulnerabilities downstream. The new developments at the downstream of the dam may even necessitate a reclassification of some of the dams. The second issue is the quality of the EAP. Not all the EAPs follow the standard guidelines set forth by FEMA or by the states. In some extreme cases, the EAP can be just a sheet containing a list of telephone numbers to call in case of emergency.

Fig. 6 Classification of “High Hazard” dams in the USA by height versus EAP Status (left) and by age versus EAP status (right), based on the entries in the National Inventory of Dams (9/28/2008). Legend for EAP status: Y= Yes, EAP exists; NR= EAP not required; and N= No; EAP does not exist.

Fig. 7 Classification of “Significant Hazard” dams in the USA by height versus EAP Status (left) and by age versus EAP status (right), based on the entries in the National Inventory of Dams (9/28/2008). Legend for EAP status: Y= Yes, EAP exists; NR= EAP not required; and N= No; EAP does not exist.
### Important Dates in Dam Safety Regulation in the USA

<table>
<thead>
<tr>
<th>Date</th>
<th>Significant Event</th>
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<tr>
<td>1965</td>
<td>The Federal Power Commission issued Order No. 315 defining the responsibilities of power licensees to ensure safe construction and operation of dams.</td>
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<tr>
<td>Aug 8, 1972</td>
<td>Public attention to the hazards created by water reservoirs after the February 26, 1972 failure of a mine tailings embankment at Buffalo Creek, West Virginia led to the enactment of the National Dam Inspection Act (Public Law 92-367).</td>
</tr>
<tr>
<td>Jun 5, 1976</td>
<td>Failure of Teton Dam in Idaho due to internal erosion. This failure led to widespread review by federal agencies regarding dam inspection, evaluation, and modification.</td>
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<tr>
<td>1979</td>
<td>Federal Guidelines for Dam Safety was published.</td>
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<tr>
<td>1982</td>
<td>U.S. Committee on Large Dams (USCOLD) passed a resolution urging state governments to give high priority to enacting dam safety legislation and to allocating resources to dam supervision.</td>
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<tr>
<td>1985</td>
<td>The Association of State Dam Safety Officials (ASDSO) became active. The Interagency Committee on Dam Safety (ICODS) formed.</td>
</tr>
<tr>
<td>1996</td>
<td>National Performance of Dams Program (NPDP) officially started.</td>
</tr>
<tr>
<td>Oct 12, 1996</td>
<td>The Water Resources Development Act of 1996 (Public Law 104-303) was signed into law by President Clinton. A National Dam Safety Program (NDSP) was established (Section 215 of Public Law 104-303).</td>
</tr>
<tr>
<td>Dec 2, 2002</td>
<td>Signed into law, the Dam Safety and Security Act of 2002 reauthorized the NDSP for 4 more years and added enhancements to the 1996 Act that are designed to safeguard dams against terrorist attacks.</td>
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3 Adapted from [http://npdp.stanford.edu/chronology.html](http://npdp.stanford.edu/chronology.html)

4 See [http://thomas.loc.gov/cgi-bin/bdquery/z?d104:SN00640:@@L&summ2=m&](http://thomas.loc.gov/cgi-bin/bdquery/z?d104:SN00640:@@L&summ2=m&)

5 See [http://thomas.loc.gov/cgi-bin/bdquery/z?d107:HR04727:@@L&summ2=m&](http://thomas.loc.gov/cgi-bin/bdquery/z?d107:HR04727:@@L&summ2=m&)
The Association of State Dam Safety Officials (ASDSO)

The Association of State Dam Safety Officials (ASDSO)\(^6\) is FEMA's primary partner in the National Dam Safety Program (NDSP) and serves as the official voice for state dam safety.

Responsibilities include:
- Grant Assistance to the States, which provides vital support for the improvement of the state dam safety programs that regulate most of the 79,500 dams in the United States.
- Dam Safety Research, which is a program of technical and archival research.
- Dam Safety Training, which trains state dam safety staff and inspectors.

Although state programs vary in the scope of their authority, program activities typically include:
- evaluation of existing dams,
- review of plans and specifications for dam construction and major repairs,
- periodic inspections of construction on new and existing dams, and
- review and approval of Emergency Action Plans

National Dam Safety Program (NDSP)

National Dam Safety Program (NDSP) (Section 215 of Public Law 104-303), which is led by FEMA, is a partnership of the states, federal agencies, and other stakeholders to encourage individual and community responsibility for dam safety. The principal areas activity are described in [http://www.fema.gov/plan/prevent/damfailure/ndsp.shtm](http://www.fema.gov/plan/prevent/damfailure/ndsp.shtm). These are listed below for ease of reference:
- Grant Assistance to the States
  The primary purpose of the National Dam Safety Program (NDSP) is to provide financial assistance to the states for strengthening their dam safety programs.

  The states use NDSP funds for the following types of activities:
  - Dam safety training for state personnel.
  - Increase in the number of dam inspections.
  - Increase in the submittal and testing of Emergency Action Plans.
  - More timely review and issuance of permits.
  - Improved coordination with state emergency preparedness officials.
  - Identification of dams to be repaired or removed.

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• Conduct of dam safety awareness workshops and creation of dam safety videos and other outreach materials.

• Dam Safety Research
Research is critical to the Nation's agenda for dam safety. Research funding under the National Dam Safety Program (NDSP) has addressed a cross-section of issues and needs, all in support of ultimately making dams in the United States safer.

To guide decisions on the funding of specific research projects, the National Dam Safety Review Board developed a 5-year Strategic Plan to prioritize research needs in dam safety. The goal in developing the Strategic Plan was to ensure that priority would be given to those projects demonstrating a high degree of collaboration and expertise, and the likelihood of producing products that will contribute to the safety of dams in the United States.

Over the past 6 years, research funds have been allocated to workshops in nine of the priority areas. Based on the workshop findings, research topics have been pursued and several topics have now progressed to products of use to the dam safety community, including technical manuals and guidelines. For future research, it is the goal of the Work Group to expand dam safety research to other institutions and professionals performing research in the field.

• Dam Safety Training
Since the inception of the National Dam Safety Program (NDSP) in 1979, FEMA has supported a strong, collaborative training program for dam safety professionals and dam owners. With NDSP training funds, FEMA has been able to expand existing training programs, begin new initiatives to keep pace with evolving technology, and enhance the sharing of expertise between the federal and state sectors.

Training activities conducted under the NDSP fall under one of three components:
○ national training opportunities, most of which are conducted at FEMA's Emergency Management Institute;
○ regional training conducted by the Association of State Dam Safety Officials (ASDSO) and other private vendors; and
○ local training through direct assistance to the states and self-paced training.

Where to find information on U.S. Dams?

With the National Dam Inspection Act (P.L. 92-367) of 1972, Congress authorized the U.S. Army Corps of Engineers (USACE) to inventory dams located in the United States. The Water

NID maintenance and publication program is achieved through cooperative participation of the 50 states and Puerto Rico (as facilitated by ASDSO), and 17 federal agencies, who provide information on approximately 79,000 dams currently in the NID.

The NID has public and restricted access possibilities. Public database can be accessed at the following web page:
http://crunch.tec.army.mil/nidpublic/webpages/nid.cfm
It is recommended to use Internet Explorer as web browser. Other browsers are not fully supported by the web site. The future version to be published soon is expected to remedy this shortcoming.

Where to find information on Dam Failures in the USA?

The National Performance of Dams Program (NPDP) is a cooperative effort of engineers and dam safety professionals in the U.S. 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.

The NPDP creates an information track that facilitates the evaluation and use of dam performance data to improve methods of design and rehabilitation, and the development of effective public policy.

The NPDP secretariat and archive is located at the Department of Civil and Environmental Engineering of Stanford University. The archive contains over six thousand documents. Among the library's holdings are the dam incident files collected by the U.S. Committee on Large Dams that were the basis for the 1975 and 1988 reports, entitled “Lessons From Dam Incidents”. A database tracks the documents in the archive and the dam incidents on file.

The homepage of NPDP is located at the following address:
http://npdp.stanford.edu/index.html
Review of Unsteady Flow Basics

Dam-breaks generally lead to rapidly varying unsteady flows. In order to understand the differences between different models and their underlying assumptions, a brief review of governing equations and different types of simplifications are presented in the following sub sections.

The unsteady flow in a natural river with irregular cross-sections is described by one-dimensional shallow water (or Saint-Venant) equations. Shallow water equations are derived from Navier-Stokes equations by making certain assumptions. In developing shallow water equations the vertical accelerations are neglected, the water surface variation is assumed to be small and the pressure distribution is assumed to be hydrostatic.

In non-conservative form, one dimensional shallow water equations can be written as follows:

\[
\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q
\]

\[
\frac{dQ}{dt} + \frac{\partial \left( \beta Q^2 \right)}{\partial x} + gA \left( \frac{\partial h}{\partial x} \right) = gA \left( S_b - S_f \right)
\]

where:

- \( A \) = Active (wetted) cross section area
- \( Q \) = Flow discharge
- \( q \) = Lateral discharge between cross sections
- \( \beta \) = Top width of the wetted area
- \( h \) = Water surface elevation
- \( g \) = Gravitational acceleration
- \( S_b \) = Bed slope
- \( S_e \) = Energy slope

The first equation expresses the conservation of mass and the second equation expresses the conservation of momentum. In many cases, numerical models solve only a simplified form of the shallow water equations. Let us review different types of models that can be obtained by simplifying the equation of conservation of momentum:

Kinematic wave equation:

\[
0 = gA \left( S_b - S_f \right)
\]
All time and space derivative terms in the momentum equation are neglected. The conservation of mass equation, which contains the time derivative of wetted area and the space derivative of discharge, remains valid. Kinematic wave can be considered to be a succession of uniform flows at each time step. Kinematic wave cannot take into account backwater effects. In a kinematic wave, there is no attenuation of the wave amplitude.

Diffusive wave equation:

\[ gA\left(\frac{\partial h}{\partial x}\right) = gA\left(S_b - S_f\right) \]

Diffusive wave equation includes the space derivative of the depth. Thus, it takes into account the backwater effects. The amplitude of the wave is subject to attenuation. Many models convert to solving diffusive equation instead of full-dynamic equation when mixed flow regimes are present.

Dynamic, quasi-steady wave:

\[ \frac{\partial}{\partial x}\left(\frac{\beta Q^2}{A}\right) + gA\left(\frac{\partial h}{\partial x}\right) = gA\left(S_b - S_f\right) \]

This equation now considers the inertia terms due to variation of velocity in space. It is used in some models.

Full-dynamic wave

\[ \frac{dQ}{dt} + \frac{\partial}{\partial x}\left(\frac{\beta Q^2}{A}\right) + gA\left(\frac{\partial h}{\partial x}\right) = gA\left(S_b - S_f\right) \]

This is the complete equation with all the terms present.

Unsteady flows generally lead to a hysteresis in the stage-discharge curve as shown in Fig. 8. As shown on the left side of the figure, at a given station the peak values of velocity, discharge and depth do not arrive at the same time. The right side of the figure shows the hysteresis in the stage-discharge curve.

On the same figure the stage discharge curve for uniform flow is shown as a single curve. It will be noted that the kinematic wave solution, for which all space and time derivatives are neglected, follows the same stage discharge curve as the uniform flow, which assumes that the gravity forces are balanced by the friction forces. In kinematic flow, the flow depth in the rising and falling stages is the same for the same discharge.
Consideration of space and time derivative terms leads to the hysteresis. Hysteresis can be observed already for diffusive waves. For a given discharge the stage discharge curve yield different depths in rising and falling stages.

Fig. 8 Arrival of peak values of velocity, discharge and depth in an unsteady flow and the hysteresis observed in the stage-discharge relationship.

The shallow water equations in this non-conservative form are not suitable for solving dam break type flows with mixed flow regimes and flow discontinuities. For dam break type flows, the conservative form of the equations should be used:

\[
\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q
\]

\[
\frac{dQ}{dt} + \frac{\partial \left( \frac{\beta Q^2}{A} \right)}{\partial x} + gA \left( \frac{\partial h}{\partial x} \right) = gA \left( S_h - S_f \right)
\]

The above equations can be written in the vector form as follows:

\[
\frac{\partial U}{\partial t} + \frac{\partial F(U)}{\partial x} = S(U) \quad \text{with} \quad U = \begin{bmatrix} A \\ Q \end{bmatrix} \quad F(U) = \begin{bmatrix} Q \\ Q^2/A \end{bmatrix} \quad S(U) = \begin{bmatrix} 0 \\ -gA \frac{\partial Z}{\partial x} - g \frac{OQ}{C^2 R_h A} \end{bmatrix}
\]

where:
- \( U \) = is the vector of conserved variables
- \( F(U) \) = Vector of Flux terms in x direction
- \( S(U) \) = Vector of source terms
- \( C \) = Chezy coefficient of roughness
- \( Z \) = Elevation of the water surface with respect to a reference datum
- \( R_h \) = Hydraulic Radius

Referring to Fig. 9, the conservative form of shallow water equations for two dimensional flows can be written as:
\[
\frac{\partial U}{\partial t} + \frac{\partial F(U)}{\partial x} + \frac{\partial G(U)}{\partial y} = S(U)
\]

with

\[
U = \begin{bmatrix} h \\ hu \\ hv \end{bmatrix}, \quad F = \begin{bmatrix} Q_x \\ Q_x^2 / h \\ Q_x Q_y / h \end{bmatrix}, \quad G = \begin{bmatrix} Q_y \\ Q_x Q_y / h \\ Q_y^2 / h \end{bmatrix}, \quad S = \begin{bmatrix} 0 \\ -gh(\partial Z / \partial x) - g\left(u\sqrt{u^2 + v^2} / C^2\right) \\ -gh(\partial Z / \partial y) - g\left(v\sqrt{u^2 + v^2} / C^2\right) \end{bmatrix}
\]

where:

- \( U \) = is the vector of conserved variables
- \( F(U) \) = Vector of Flux terms in x direction
- \( G(U) \) = Vector of Flux terms in y direction
- \( S(U) \) = Vector of source terms
- \( h \) = Flow Depth
- \( Q_x \) = Discharge in x direction
- \( Q_y \) = Discharge in y direction
- \( Z \) = Elevation of the water surface with respect to a reference datum
- \( u \) = Velocity in x direction
- \( v \) = Velocity in y direction
- \( C \) = Chezy coefficient of roughness

Fig. 9 Coordinate system for writing two-dimensional shallow water equations.

Generally, development of a numerical scheme for solving either 1D or 2D version of the shallow water equations boil down to developing the best way to express the flux terms to achieve an oscillation free shock capturing scheme. Treatment of source terms constitutes another important point. The numerical scheme must be well balanced; i.e. in the absence of flow, the source terms must be balanced by the hydrostatic pressure terms in order to avoid generation of spurious flows.
**Numerical Modeling of Dam Break Flows**

Numerical modeling of dam-break flows involves some challenges:

- **Mixed flow regimes**
  Dam break flows generally involve presence of mixed flow regimes (subcritical, transcritical, and supercritical) in the same computational domain. At a given location, the flow may also evolve in time from one regime into another. Handling of these types of flows require special considerations with regard to boundary conditions and the computational algorithms.

- **Flow discontinuities**
  Dam break flows may involve discontinuities in the flow such as translatory waves, standing or moving hydraulic jumps, etc. In order to capture these discontinuities, special numerical schemes, called “shock capturing schemes”, must be used based on the integral formulation of the conservative form of shallow water equations.

  These shock capturing schemes must be specially designed to avoid smearing of discontinuities and the oscillations near the edges of discontinuities. These issues are explained briefly in Fig. 10.

  ![Fig. 10 Some challenges encountered when solving shallow water equations for flows involving discontinuities.](image)

  The true discontinuous solution is the one with sharp edges. If not properly designed, numerical schemes may round the edges of the discontinuities. These types of schemes will smooth out the true waves as they travel and lead to wrong results.

  Special numerical schemes need to be used to avoid oscillations near the leading and/or trailing edges of flow discontinuities. Such oscillations may cause instabilities in the solution.

- **Wetting/drying**
  Dam break flows may involve dry conditions at the downstream of the dam. Special numerical procedures must be used to handle wet-dry contact in order to compute correctly the propagation of the wave front.

- **Complex topography**
  The use of real digitized topography leads to large source terms where the terrain changes abruptly. In order to avoid numerical instabilities, “well balanced” schemes must be used.
Convergence to non-physical (entropy violating) solutions
In order to obtain shock capturing schemes that can simulate flow discontinuities, one generally solves integral (weak) form of shallow water equations. Unfortunately, under some situation this may lead to non physical solutions, such as the one shown in Fig. 11. Various methods are available to test for these physically impossible solutions and eliminate them by imposing “entropy” constraints.

Fig. 11. Physically impossible solution with a hydraulic jump in a negative wave obtained using first order Roe’s scheme\(^7\).

In the recent years various special methods have been developed to address these difficulties, such as:
- TVD Schemes,
- Approximate Riemann Solver based approaches,
- Flux vector splitting schemes,
- Petrov-Galerking finite element schemes, etc.

Numerical solution of shallow water equations requires a discretization based on a regular or irregular grid. Generally, numerical models use one of the following numerical discretization methods listed below:
- Finite difference method
- Finite-volume method
- Finite element method
- Hybrid methods

Moreover, the solution can be obtained using an implicit or explicit scheme. In the explicit scheme, the values at the next time step are computed solely based on the known values of the variables at the current time step. This method is quite straightforward to code and does not involve solving large systems of equations. The disadvantage of this method is that it generally requires small times steps. Given a grid size, the time step must be chosen to make sure that the

\(^7\) Zoppou and Roberts (2003), J. Hydraulic Engineering, 129(1)
flow cannot propagate a distance longer than the smallest grid size. This is called Courant – Friedrichs-Levy (CFL) condition for stability of the solution. CFL condition in some cases may lead to very small time steps and, thus, require long computational times.

The implicit method, computes the values of variables at the next time step based on both current known values and future unknown values. This leads to a system of equations that must be solved simultaneously. The implicit methods generally have no restrictions on the time step to ensure the stability of the computations. One can use time steps considerably larger than those used in an explicit scheme. In practice, however, the size of the time step must be restricted due to convergence and accuracy considerations. In two-dimensional models, if the domain is large and the grid resolution is high, implicit methods may lead to the solution of very large systems of equations.

It is important to note that, even some of the widely used numerical models ignore the challenges listed at the beginning of this subsection, and prefer to use non-conservative methods, with varying degrees of success, although they are not particularly adapted for dam break type flows involving mixed flow regimes and/or flow discontinuities.

Therefore, in reviewing numerical tools available for solving dam break problems, one should look into the details of the solution methods and the underlying assumptions. The devil is in the detail, and some numerical models involve important assumptions and simplifications that may be difficult to detect without a careful study.

For example, some well widely used numerical models include special features to deal with mixed flow regimes, which in fact completely changes the nature of the equations solved. The way HEC-RAS deals with mixed flow regimes can be given as an example.

HEC-RAS normally solves the following full-dynamic unsteady momentum equation for shallow water flows:

$$\frac{dQ}{dt} + \frac{\partial}{\partial x} \left( \frac{\beta Q^2}{A} \right) + gA \left( \frac{\partial h}{\partial x} + S_f \right) = 0$$

where:

- $h$ = Water surface elevation
- $S_f$ = Friction slope
- $Q$ = Flow discharge
- $A$ = Active (wetted) cross section area
- $g$ = Gravitational acceleration
- $\beta$ = Momentum coefficient
In order to deal with the stability problems resulting from the presence of mixed flow regimes, Fread (1986), the developer of HEC-UNET (which later became the unsteady flow solver for HEC-RAS), has introduced a concept called the “Local Partial Inertia Technique (LPI)”. This method is implemented in HEC-RAS as an option for solving mixed flow regime problems. The LPI methodology consists of using a multiplier in front of the first two terms (inertia terms) which reduces their contribution as the Froude number approaches to 1 (critical flow). The modified equation is as follows:

$$
\sigma \left( \frac{dQ}{dt} + \frac{\beta Q^2}{A} \right) + gA \left( \frac{\partial h}{\partial x} + S_f \right) = 0
$$

with

$$
\sigma = \begin{cases} 
F_r - F_T^m & \text{if } F_r \leq F_T; \ m \geq 1 \\
0 & \text{if } F_r > F_T
\end{cases}
$$

where:

- $\sigma$ = LPI multiplier in front of inertial terms
- $F_r$ = Froude number threshold at which the factor is set to zero. In HEC-RAS this value can range from 1.0 to 2.0 (the default is 2.0)
- $F_T$ = Local Froude number of the flow
\[ m = \text{Exponent which changes the shape of the curve expressing the variation of } \sigma \text{ as a function of the Froude number, } F_r \text{ (see figure below)} \]

\[ g = \text{Gravitational acceleration} \]

It is important to note that the LPI is nothing but a procedure that converts the full-dynamic unsteady momentum equation into a diffusive flow equation, by getting rid of the inertia terms, as the Froude number of the flow approaches to 1 (critical flow). Namely, when the flow Froude number is greater than or equal to zero the HEC-RAS solves the diffusive wave equation shown below:

\[ gA \left( \frac{\partial h}{\partial x} + S_f \right) = 0 \]

HEC-RAS does not contain any consideration regarding the wet-dry fronts, or flow discontinuities. Any flow discontinuities are completely smeared. Due to these shortcomings, the unsteady flow option of HEC-RAS may be subject to numerical instabilities under some flow conditions.

**Brief Review of Dam Break Analysis**

Dam failure mechanisms considerably vary depending on the type of the dam. In earthen dams, failure due to overtopping and piping seem to be the principal causes (see Fig. 4). Generally, it is required to study various scenarios. In some scenarios only the failure of the dam is considered (sunny day scenario). The discharge entering the reservoir is assumed to be small and, therefore, neglected. In other types of scenarios discharge entering the reservoir (such as PMF) is also considered.

In any case, the breaching process needs to be considered for determining the flood hydrograph that will be routed either as a channel flow (1D model) or an overland flow (2D model).

In some models, the breaching process and the resulting hydrograph are calculated by a separate study using a model either based on user defined evolution of breach geometry or a physically based computation involving the erosion of the embankment material by the flow. The hydrograph obtained from this separate study is then introduced as upstream boundary condition in the flood routing model and the propagation of the flood is computed.

In other models, the computation of the breaching is already included in the model.
Review of Available Computational Tools for Dam Break Analysis

A large number of numerical models are available for computation of dam break flows. Generally the existing models can be categorized into one of the following categories:

1. Simplified numerical models aimed at determining the envelope maximum water depths based on some hydrologic modeling and/or by filling a digital map with a volume equal to a portion or totality of the water stored in the reservoir.
2. One dimensional models that solve either full dynamic or simplified forms of conservative or non conservative forms of one dimensional, cross section averaged shallow water equations.
3. Two-dimensional models that solve either full dynamic or simplified forms of conservative or non conservative forms of two dimensional shallow water equations.
4. 1D-2D coupled (or integrated) models that solve both the one-dimensional channel flow and the two-dimensional overland flow together using full dynamic or simplified forms of conservative or non conservative forms of one- and two-dimensional shallow water equations.

Until very recently, due to technological limitations, almost exclusively 1D models were used for dam break studies, even for the cases where the underlying assumptions of one-dimensional modeling are clearly violated. This was mainly dictated by various technological limitations. The main objections to the use of two-dimensional models were threefold:

- long computational times
- requirement for topographic mesh generation, which can be quite time and effort consuming, and
- lengthy input data preparation.

It can be confidently argued that the above objections to the use of two-dimensional models for flood studies are no longer valid. The recent scientific and technological developments have completely changed outlook:

- Developments in conservative numerical solutions of hyperbolic equations with shock capturing capability have permitted the development of robust numerical codes to solve shallow water equations for mixed regimes over a realistic topography.
- Developments in computer hardware (processor speed, storage capabilities, parallel and/or cluster computing, networking, etc.) allows large storage space and superior computational speeds. Modeling of even relatively large areas, say 75km by 75km, at a resolution of 50m can be accomplished in a few hours.
- Recent developments in remote sensing and measuring methods and GIS technologies have enormously facilitated the collection, treatment, storage, and retrieval of enormous amounts of geospatial data (topography, land cover, roughness, etc.) needed by the 2D numerical codes for realistic simulations.
The time has, therefore come to choose the right type of model for the type of the problem at hand and the amount of data available, rather than trying to adapt the problem at hand to the available modeling capabilities.

Experience has shown that one-dimensional models can provide reliable results even in relatively complex problems. Errors may arise where:

- the dam is very high and the initial water difference between upstream and downstream is large,
- mixed flow regimes occur in the same reach,
- there are sudden cross section alterations (contractions or expansions),
- the flood propagates on a flat terrain (see Fig. 13).

![Fig. 13 Frames showing the propagation of the flood wave due to failure of Malpasset Dam, France, in 1959. In the earlier stages, the two leftmost frames, the flood front advances in a relatively narrow valley. One dimensional modeling can be assumed to hold while the flood is in the valley. In the later stages, two rightmost frames, the flood propagates on a relative flat terrain. One observes an important lateral expansion. In this case, the underlying assumptions for one-dimensional model no longer hold. A two dimensional approach may be more appropriate to capture the correct dynamics of the flow with arrival times and flow depths.](image)

Ideally, the numerical model should provide the following information:

- Flood arrival times
- Flood depths (both the maximum depth and the evolution of the depth as a function of time)
- Flood velocities (both the maximum velocity and the evolution of the velocity as a function of time)\(^8\)
- Flood duration\(^9\)

Ideally, the numerical model should possess the following characteristics:

- User friendly GUI and easy problem set-up and running
- Ability to work with data of varying complexity

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\(^8\) This information can be required only for two-dimensional models.

\(^9\) Provided that the simulation is carried out sufficiently long time.
• Connectivity with a GIS system (exporting of the results to a GIS platform for consequence analysis).

One dimensional models have several attractive features. They are generally faster to run although they may involve some engineering judgment for the selection of cross section locations and extents. There is also considerable level of expertise accumulated in the engineering community regarding the use of 1D models.

Fig. 14 Steps for converting one-dimensional simulation results into two-dimensional flood delineation maps (adapted from FEMA, 2003). Flood elevations between adjacent sets of cross-sections are calculated by interpolation based on the topography. Flood elevations for tributaries need to be calculated by carrying out backwater calculations.

One of the shortcomings of one-dimensional models is that the results only provide a depth and a discharge at computational cross section along the river. In order to obtain a two dimensional map of the inundated area, one dimensional model results must be converted into two-dimensional maps by interpolating between 1D model cross sections based on digital elevation maps. Although it is widely used in studying fluvial floods (see Fig. 14), which are treated as steady flows, the interpolation process may introduce errors in unsteady simulations, especially over relatively flat terrain. In the case of highly transient flows, the interpolation process does not respect the mass conservation principle.

Two-dimensional models, however, directly provide a two dimensional map of the inundated area, which can be easily imported into a GIS platform for further analysis.

Recently, several coupled (or integrated) 1D-2D models have appeared. These models combine the strengths of one-dimensional and two-dimensional approaches in a single model. They are especially useful for studying levee breaching problems and/or cases where the flow overtops the channel and propagates in a large flood plain as overland flow.

It is important to note that there are no guidelines yet as to the use of unsteady flow models, be it 1D or 2D, in flood studies.
List of Available Models

Simplified Models

**SMPDBK / Simplified dambreak modeling by National Weather Service (NWS)**

The Simplified Dam-Break (SMPDBK) was developed by the National Weather Service (NWS) for predicting downstream flooding produced by a dam failure. MPDBK does not provide time-varying results for a dam break simulation. 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, or now called FLDWAV. The SMPDBK method is useful for situations where reconnaissance level results are adequate, and when data and time available to prepare the simulation are sparse. Unlike the more sophisticated versions of DAMBRK and FLDWAV, the SMPDBK method does not account for backwater effects created by natural channel constrictions of those due to such obstacles as downstream dams or bridge embankments.

![Fig. 15](http://www.xmswiki.com/wiki/index.php5?title=WMS:SMPDBK)

The input required for a SMPDBK model is a stream centerline, cross sections, and information regarding the storage and failure of the dam being modeled. WMS saves the model data to a properly formatted input file for SMPDBK and then launches the executable. The executable is the same version distributed by the NWS. When a model is successfully run, WMS will automatically read the results and create a water surface elevation data set that can be used for automated floodplain delineation as illustrated in Fig. 15. SMPDBK can be downloaded from: [http://www.rivermechanics.net/downloads.htm](http://www.rivermechanics.net/downloads.htm)
One-Dimensional Models

Below a list of one dimensional models that can be used for dam break studies are briefly presented. This list needs to be completed in the future by adding other models. More information should be provided for each model. Moreover, a table comparing merits and shortcomings of these models must be prepared.

Full Equations (FEQ) Model

The Full Equations (FEQ) computer program (Franz and Melching, 1997) simulates flow in a stream system by solving the full equations of motion for one-dimensional, subcritical unsteady flow. The effect and/or operation of structures including bridges, culverts, dams, spillways, weirs, and pumps may be simulated with the program. A companion program (FEQUTL) operates as a preprocessor to convert cross-section data into hydraulic tables for use during the unsteady computations. FEQ uses the continuity and momentum equations to determine the flow and depth throughout the stream system following the specification of initial flow and boundary conditions.

The program was initially developed in 1976 for simulation of flow through the Sanitary and Ship Canal in Chicago, Illinois. The program has been improved and modified in many versions over the years. Documentation was published by the USGS in 1997, as referenced in the previous paragraph. The program has been used for a wide variety of rivers and streams, ranging from 600 mi (966 km) of the Mississippi River to small creeks in DuPage County, Illinois.

The program is distributed by the USGS and additional information may be obtained at the USGS web site:
http://il.water.usgs.gov/proj/feq/

FLDWAV Model developed by National Weather Service

In the late 1980s, the U.S. National Weather Service (NWS) developed the FLDWAV computer program (NOAA, 2000) for unsteady flow analysis using the full equations of motion. The program performs hydraulic simulations for real-time forecasting of natural floods or dam-break events and supplies information for the design of waterway improvements and for flood inundation mapping for dambreak flood planning. The flow can be subcritical, supercritical, or mixed throughout the downstream reach. The flood being modeled can be interconnected through a river system (main stem and tributaries). Levee overtopping and breaching are
handled, along with split flow (island) situations and the modeling of mud-debris flow. Bridges can be modeled with the program, but not culverts.

Planned improvements include the addition of culvert modeling, the operation of movable gates, sediment transport, additional routing methods, and the modeling of landslide-generated waves in reservoirs.

The FLDWAV program is a combination of two popular NWS programs: the Dynamic Wave Operation Network Model (DWOPER) and the Dam-Break Forecasting Model (DAMBRK). DWOPER was developed by Danny Fread (Fread, 1982) of the NWS for use in the river forecasting program. It is a general model with many added features to allow simulation of river structures and levees. The NWS used the program to routinely provide daily stage and discharge predictions for the Lower Mississippi River prior to FLDWAV. The NWS also developed DAMBRK (Fread, 1984) specifically for simulating the failure of a dam and the resulting flood wave through the downstream valley. The model has been used in numerous dam break simulations to determine the maximum crest height to be expected and the warning time available for downstream inhabitants.

**HEC-RAS (HEC-UNET) Model**

HEC-RAS can now incorporate both steady and unsteady, one-dimensional flow computations using the same set of geometry data for either analysis. Unsteady flow computations use the full equations of motion (St. Venant equations). The unsteady flow equation solver is taken directly from the HEC-UNET program, but all other unsteady flow procedures in HEC-RAS are different from those in HEC-UNET.

The unsteady portion of the program accepts geometric input in the form of standard HEC-2 or HEC-RAS cross sections and then converts each section to tables of hydraulic properties, using a preprocessor program (HTAB) to facilitate the computations in the unsteady flow engine (UNET). The engineer must specify all cross sections, inflow hydrographs (not just peak discharge) for all tributaries, upstream and downstream boundary conditions (flow or stage hydrographs or discharge rating curves), and various coefficients. A postprocessor program is available to facilitate the output review. The postprocessor provides all the tables and plots for unsteady flow that are available for steady flow computations. Without the postprocessor, only graphical output consisting of stage and/or discharge hydrographs at all cross sections is available.

The unsteady flow portion of the program can perform subcritical, supercritical, or mixed-flow computations. Dam breaching and levee break algorithms are also included, as is the ability to model pumping stations. A maximum of ten pump groups, with each group consisting of up to
20 identical pumps, can be modeled in the unsteady flow portion of HEC-RAS. The modeling of flap-gated culverts (allowing only one-way flow) is also an option. As is the case when modeling steady flow in HEC-RAS, the unsteady analysis is limited to 500 profiles. A maximum of 6,000 cross sections may be used in the model, with up to 500 elevation-station points for each cross section.

Both steady and unsteady flow analysis in HEC-RAS begin with the same set of geometric data, but the water surface profiles for the same actual or hypothetical event are normally somewhat different. Differences are primarily due to three key features:

- For eddy or other losses, steady flow computations use the absolute difference in velocity heads at adjacent cross sections multiplied by an expansion or contraction coefficient. In unsteady flow computations, these eddy losses are computed within the momentum equation.
- Steady flow computations find the average friction slope between cross sections based on the average conveyance method (HEC-RAS default method). Unsteady flow computations use the average friction slope between cross sections directly from a simple average of the computed friction slopes. Tests using both UNET and HEC-UNET (Brunner, 2002) have shown that the unsteady flow program is more stable when using average friction slope, rather than the average conveyance method that is applied in steady flow computations.
- For a given discharge, steady flow computations compute losses through bridges, culverts, and other obstructions directly from the obstruction geometry and the type of flow conditions through the bridge (low flow, pressure, weir, momentum, or combination). 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. Unsteady flow analysis in HEC-RAS interpolates the headwater elevation for computed discharge and tailwater data for each time period. Depending on the number of discharges used to set up the family of curves, there could be differences in computed losses at obstructions between steady and unsteady state computations.

Given the computational differences between steady and unsteady state analysis, there is generally a small difference between results for a selected flood discharge for a steady flow solution compared to the unsteady flow solution. The steady flow solution is generally 0.1-1 ft (0.03-0.3 m) higher than the unsteady flow solution, but the difference can be outside this range, depending on how the engineer developed his input data and the degree of variation in flow expansion and contraction through a reach. The difference does not necessarily mean that one method is more accurate than another; it simply means that a difference may exist because the computation procedures are different between steady and unsteady flow analysis.

The unsteady flow analysis routing in HEC-RAS has been successfully applied for a variety of rivers and streams, ranging from more than 2000 mi (3200 km) of the Mississippi and Missouri
ISIS Flow/Hydrology Model by Wallingford Software

Developed by Wallingford Software, ISIS Flow is a full hydrodynamic simulator for modeling flows and levels in open channels and estuaries and is at the heart of the system. ISIS Flow is able to model complex looped and branched networks, and is designed to provide a comprehensive range of methods for simulating flood plain flows. ISIS Flow incorporates both unsteady and steady flow solvers, with options that include simple backwaters, flow routing and full unsteady simulation. The simulation engine provides a direct steady-state solver and adaptive time-stepping methods to optimize run-time and enhance model stability. ISIS Flow employs the Preissmann implicit scheme, which is popularly referred to as the 4-points Box scheme, to solve the one-dimensional gradually varied unsteady flow.

ISIS provides full interactive views of the model data and results using plan views, long sections, form based editing tools and time series plots. Results can also be reported in text and tabular formats. The software includes a wide range of diagnostic error checks and a comprehensive on-line help system.

A key strength of ISIS Flow is the ability to model a wide range of hydraulic structures including all common types of bridges, sluices, culverts, pumps and weirs. Where ever possible, standard equations or methods are incorporated into the software so that the calculation of level and discharge relationships is fully handled by the software. For structures with automatic operation such as pumps and sluices ISIS Flow allows the user to incorporate logical control rules.

Provided with ISIS Flow is an event-based hydrological module. For UK use, the well established Flood Studies Report (FSR) unit hydrograph method is available, including the recommendations of FSSR 16. In the near future this will be enhanced with the methods of the new Flood Estimation Handbook (FEH). For other countries the US Soil Conservation Service (USSCS) method is available. Regardless of the method used, the software allows the user to specify observed or predicted rainfall profiles and unit-hydrographs.

ISIS model has been used to simulate Malpasset Dam break event\textsuperscript{10} and Toce River physical dam-break model results\textsuperscript{11}.

More information can be found at the web site: 
http://www.wallingfordsoftware.com/products/isis/

\textsuperscript{10} http://www.hrwallingford.co.uk/projects/CADAM/CADAM/Zaragoza/Z11.pdf
\textsuperscript{11} http://www.hrwallingford.co.uk/projects/CADAM/CADAM/Zaragoza/Z16.pdf
Two-Dimensional and Coupled 1d-2d Models

Below a list of two-dimensional and coupled 1D-2D models that can be used for dam break studies are briefly presented. This list needs to be completed in the future by adding other models. More information should be provided for each model. Moreover, a table comparing merits and shortcomings of these models must be prepared.

MIKE FLOOD
Developed by DHI Group, Denmark, MIKE FLOOD is a commercially available integrated tool for detailed floodplain studies. It combines the two numerical hydrodynamic models MIKE 11 (1-D) and MIKE 21 (2-D) with a unified user interface and gives you the best of both worlds: Detailed spatial modelling where needed, plus the speed of 1-D calculations where appropriate. MIKE FLOOD is ideal for many types of analyses such as flooding, storm surge, dam break, embankment failure, and more.

Being a 1D-2D coupled modeling environment, MIKE FLOOD consists of both 1D and 2D solvers and the modeler can combine these as they see fit. Typically the 1D model may be used
- to represent flow in channels that may not be resolved in the 2D grid,
- to model underlying pipe and sewer networks
- to simulate hydraulic structures such as culverts, bridges, weirs etc, and
- to simulate dam or levee failures
- to route flow in longer river reaches for which a 2D models would be computationally heavy.

Whereas the 2D model is normally used
- to represent over bank flows, an application where 1D modelling may be insufficient. A 2D model has the advantage of being able to accurately represent the floodplain geometry, so that discharge, storage, and attenuation in the floodplain can be accurately simulated.
- to relieve the modeller of the burden of having to pre-define the flow paths, the 2D model will simulate flow splits based on the input topography.
- to describe the complex network of streets and pathways found in urban areas.

More information on MIKE FLOOD can be obtained from: http://www.dhigroup.com/Software/WaterResources/MIKEFLOOD.aspx

BASEMENT
Basement is a numerical simulation software developed at the Laboratory of Hydraulics, Hydrology and Glaciology (VAW) of the Swiss Federal Institute of Technology (ETH) at
Zurich, Switzerland. The purpose of the software is to provide a software tool for numerical modeling of environmental flow and natural hazard events.

More information and a copy of the program can be obtained from:
http://www.basement.ethz.ch/

**CCHE2D-FLOOD**

Developed by the National Center for Computational Hydroscience and Engineering, CCHE2D-FLOOD is a two-dimensional and 1D-2D integrated model, which is accompanied by a GIS-based decision support system for risk and vulnerability analysis including loss-of-life, urban and agricultural damage, etc. The GIS-Based decision Support System and the numerical model CCHE2D-FLOOD together form the DSS-WISE (Decision Support System for Water Infrastructural Security) modeling environment.

The numerical model CCHE2D-FLOOD solves the conservative form of the two-dimensional shallow water equations (Saint-Venant Equations) in conservative form over a complex topography defined by a regular mesh. Referring to the coordinate system depicted in Fig. 9, the shallow water equations, or Saint-Venant equations, that describe the unsteady two-dimensional flow can be written as follows:

\[
\frac{\partial \mathbf{U}}{\partial t} + \frac{\partial \mathbf{F}(\mathbf{U})}{\partial x} + \frac{\partial \mathbf{G}(\mathbf{U})}{\partial y} = \mathbf{S}(\mathbf{U})
\]

(1)

where \( \mathbf{U} \), \( \mathbf{F}(\mathbf{U}) \), \( \mathbf{G}(\mathbf{U}) \) and \( \mathbf{S}(\mathbf{U}) \) are the vectors of conserved variables, fluxes in the \( x \) and \( y \) direction, and sources, respectively. They are defined as:

\[
\mathbf{U} = \begin{bmatrix} h \\ h u \\ h v \end{bmatrix}, \quad \mathbf{F} = \begin{bmatrix} Q_x \\ Q_x^2/h \\ Q_x Q_y/h \end{bmatrix}, \quad \mathbf{G} = \begin{bmatrix} Q_y \\ Q_y Q_x/h \\ Q_y^2/h \end{bmatrix}, \quad \mathbf{S} = \begin{bmatrix} 0 \\ -gh(\partial Z/\partial x) - g(u\sqrt{u^2 + v^2})/C^2 \\ -gh(\partial Z/\partial y) - g(v\sqrt{u^2 + v^2})/C^2 \end{bmatrix}
\]

(2)

in which \( Z \) represents the water surface elevation and \( C \) the Chezy friction coefficient. Cell-centered finite-volume discretization of Eq. 1 over a rectangular control volume leads to the following explicit scheme:

\[
U_{ij}^{n+1} = U_{ij}^n - \frac{\Delta t}{\Delta x_i} \left( F_{i+1/2,j} - F_{i-1/2,j} \right) - \frac{\Delta t}{\Delta y_j} \left( G_{i,j+1/2} - G_{i,j-1/2} \right) + \Delta t S_{ij}
\]

(3)

where \( \Delta x \) and \( \Delta y \) are the cell dimensions in \( x \) and \( y \) directions, and \( \Delta t \) is the time step. The intercell fluxes are computed using a first order upwinding (Ying et al., 2004):

\[
F_{i+1/2} = \begin{bmatrix} Q_x \\ Q_x^2/h \\ Q_x Q_y/h \end{bmatrix}, \quad k = \begin{cases} 0 \text{ if } Q_x \geq 0 \\ 1 \text{ if } Q_x < 0 \end{cases} \quad \text{and} \quad G_{j+1/2} = \begin{bmatrix} Q_y \\ Q_y Q_x/h \\ Q_y^2/h \end{bmatrix}, \quad m = \begin{cases} 0 \text{ if } Q_y \geq 0 \\ 1 \text{ if } Q_y < 0 \end{cases}
\]

(4)
To avoid dry bed condition a very small water depth is maintained over the entire computational mesh. CCHE2D-FLOOD was tested and validated using analytical solutions as well as data from laboratory experiments, model tests, and past dam break events (see Ying et al., 2003a and b; Ying and Wang, 2004, and Ying et al., 2004). These tests show that the model is stable, oscillation-free, robust, and conserves mass rigorously. Figure 8 shows the application of CCHE2D-FLOOD to simulate a hypothetical complete break of the Sardis Dam on the Tallahatchie River, MS, which has been in operation since 1940. The dam has a crest length of 15,300 ft and an average height of 97 feet. Maximum storage capacity is 1,512,000 acre-feet, and the dam has a drainage area of 1,545 square miles. A DEM with a resolution of 30m was used for this 72-hour simulation.

Fig. 8  Simulation of the hypothetical sudden break of the Sardis Dam, Mississippi using CCHE2D-FLOOD. A DEM with a resolution of 30m was used for this simulation
CCHE2D-FLOOD can directly use a DEM as a computational mesh. Although the direct use of DEM as computational mesh provides several advantages, depending on the resolution, it may not be possible to capture the influence of linear terrain features such as roads, railroads, on the flow. CCHE2D-FLOOD offers a capability called cut-cell boundary, which allows projection of linear features to be projected onto the computational mesh from a GIS shape layer. Projected linear terrain feature is taken into account in the computations. Overtopping of the cut-lines is also allowed. CCHE2D-FLOOD uses a special version of cut-cell boundary method to offer the possibility of coupled 1D-2D simulations.

A preliminary version of CCHE2D-FLOOD called CCHE2D-DAMBREAK has been in use by Army Corps of Engineers-ERDC in Vicksburg since 2004. Army Corps of Engineers-Vicksburg District and National Geospatial Intelligence Agency have also began using the program in 2008. The new version of the CCHE2D-FLOOD will be available for federal and state agencies in December 2008.

CCHE2D-FLOOD has a graphical user interface comprising pre- and post-processor modules, which is developed as an add-on extension to ArcGIS software.

- The pre-processor module contains all the functionalities required for defining the problem and the associated input data. Some of the major functionalities included are:
  1. Specialized tools for manipulating the DEM file (estimating unknown reservoir bathymetry, removing the cells defining the dam from DEM, etc.)
  2. Defining computational domains that are inclined with respect to the North-South and East-West grid lines of DEM
  3. Definition of roughness coefficients by regions defined as polygons.
  4. Defining the location of multiple dams or water bodies (such as lakes). Each dam has its own information set specifying when the dam starts to break and the sequence of breaching.
  5. Projecting linear structures over the two-dimensional computational domain defined by DEM
  6. Projecting the plan-form of a one dimensional river model over the two-dimensional computational domain defined by DEM
  7. Writing out the input file to be used by the model
  8. A button to launch the model from within ArcGIS.

- The post processor module comprises of tools that are needed to import the computational results into ArcGIS environment as well as modules to analyze and assess the consequences of the flood.
  1. Tools for reading computational result files into ArcGIS as raster and/or shape files
  2. Contour plots of equal arrival times at different times
  3. Contour plots of maximum and minimum flood depths
4. Contour plots of Maximum and minimum velocities
5. Contour plots of equal escape times for population at risk (PAR)
7. Urban damage estimation with consideration of uncertainty using Monte-Carlo Method (provides maps of expected urban damage estimation in dollars and standard deviation)
8. Rural (or agricultural) damage estimation with consideration of uncertainty using Monte-Carlo Method (provides maps of expected rural damage estimation in dollars and standard deviation)
9. Tracking of infrastructure failure using network theory
10. Spatial Compromise Programming tool for multi-criteria decision making
11. Various special hazard maps (flood depths as a function of time, flood velocity vectors as a function of time, map of maximum/minimum flood depths, map of maximum/minimum flood velocities, etc.)
12. Various time series plots (flood depth, velocity as a function of time at any given location in the area of interest.

**FLO-2D: Two-Dimensional Flood Routing**

FLO-2D is a FEMA approved hydraulics model for both riverine studies and unconfined alluvial fans. It is a dynamic flood routing model that simulates channel flow, unconfined overland flow and street flow. It simulates a flood over complex topography and roughness while reporting on volume conservation, the key to accurate flood distribution. The model uses the full dynamic wave momentum equation and a central finite difference routing scheme with eight potential flow directions to predict the progression of a flood hydrograph over a system of square grid elements.

FLO-2D is a combined hydrologic and hydraulic model so there is no need to separate rainfall/runoff and channel routing. FLO-2D can be applied to simulate a diverse realm of complex flood problems including:

- River overbank flooding
- Unconfined alluvial fan and floodplain flows
- Urban flooding with street flow, flow obstruction and storage loss
- Over progression of tsunami and hurricane storm surges
- Mud and debris flows
- Watershed rainfall and runoff
- Flood insurance studies
- Flood mitigation design
FLO-2D has a number of components that enhance detailed flood routing including channel-
floodplain flow exchange, loss of storage due to buildings, flow obstructions, simulation of
hydraulic structures, levee and levee failure, sediment transport, simulation of hyperconcentrated
sediment flows (mudflows), rill and gully flow, rainfall and infiltration. More information can be
obtained from the website:
http://www.flo-2d.com/

**TELEMAC2D Model**

TELEMAC-2D is a software developed by the EDF, France. It is used to simulate free-surface
flows in two dimensions of horizontal space. At each point of the mesh, the program calculates
the depth of water and the two velocity components.

TELEMAC-2D has many fields of application, including dam breaks, flood studies.

More information can be obtained from the following website:
http://www.telemacsystem.com/gb/info/comm/telemac2d/telemac2d.html

**Models for Dam Breach Simulation and Dam-Break Hydrograph Computation**

This section will be completed in a future version.

**BREACH Model**

BREACH is a physically based mathematical model to predict the breach characteristics (size,
time of formation) and the discharge hydrograph emanating from a breached earthen dam. The
earthen dam may be man-made or naturally formed by a landslide. The model is developed by
coupling 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
breached channel. The bottom slope of the breach is assumed to be essentially that of the
downstream face of the dam. The growth of the breach channel is dependent on the dam’s
material properties (D50 size, unit weight, friction angle, cohesive strength). The model
considers the possible existence of the following complexities: 1) core material having properties
which differ from those of the outer portions of the dam; 2) the necessity of forming an eroded
ditch along the downstream face of the dam prior to the actual breach formation by the
overtopping water; 3) the downstream face of the dam can have a grass cover or be composed of
a material of larger grain size than the outer portion of the dam; 4) 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; 5) enlargement of the breach width by slope stability theory; 6) initiation of the breach via piping with subsequent progression to a free surface breach flow; and 7) erosion transport can be for either noncohesive (granular) materials or cohesive (clay) materials. The outflow hydrograph is obtained through a time-stepping iterative solution that requires only a few seconds for computation on a mainframe computer.

The model is not subject to numerical stability or convergence difficulties. The model’s predictions are compared with observations of a piping failure of the man-made Teton Dam in Idaho, the piping failure of the man-made Lawn Lake Dam in Colorado, and a breached landslide-formed dam in Peru. Also, the model has been used to predict possible downstream flooding from a potential breach of the landslide blockage of Spirit Lake in the aftermath of the eruption of Mount St. Helens in Washington. Model sensitivity to numerical parameters is minimal; however, it is sensitive to the internal friction angle of the dam’s material and the extent of grass cover when simulating man-made dams and to the cohesive strength of the material composing landslide-formed dams.

BREACH can be downloaded free-of-charge from: http://www.rivermechanics.net/downloads.htm

**BREA DA Model**

See following publication
Uncertainty Analysis and Risk Assessment in Dam Break Modelling
Migena Zagonjolli
Format: Paperback, 142 pages, Illustrations (chiefly col.)
Publisher: Taylor & Francis Ltd
ISBN-10: 0415455944

More information to be provided in a future version.

**BEED Model**

More information to be provided in a future version.

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**BRDAM Model**
See following publication:

**NRCS SITES Model**
See following website for more information and for downloading the software:

**Useful Links:**

- USBR web page for Dam Breach, Dam Failure, and Flooding Links
  http://www.usbr.gov/pmts/hydraulics_lab/twahl/breach/breach_links.html

- IMPACT Web site
  http://www.samui.co.uk/impact-project/impact_project_overview.htm

- Floodsite Project
  http://www.floodsite.net/

**Dam Safety Organizations**

- The Bureau of Reclamation's Dam Safety Program performs site evaluations to identify potential safety deficiencies on both Reclamation dams and dams belonging to other bureaus of the Department of the Interior.
  http://www.usbr.gov/ssle/dam_safety/

- Association of State Dam Safety Officials - ASDSO
  http://www.damsafety.org/

- U.S. Society on Dams (USA member of ICOLD) – USSD
Databases

- National Performance of Dams Program (Stanford University) is a national (U.S.) effort to create an information resource on the performance of dams. This site facilitates the retrieval, archival, and dissemination of information on the performance of dams in the United States, via the NPDP program.
  http://npdp.stanford.edu/index.html

- National Inventory of Dams
  http://crunch.tec.army.mil/nidpublic/webpages/nid.cfm

- Tony Wahl’s personal web page:
  http://www.usbr.gov/pmts/hydraulics_lab/twahl/index.cfm

References

This section will be completed in a future version.

Messages and Remarks from Sub-Group Members

In an email dated 10/1/2008, Sam Crampton wrote:
I would like to get involved in the discussion of modeling tools for dam break analysis. A few key points I would like to bring up are:
  • What tools are available?
  • 1-d v’s 2-d and when are each suitable
  • Gradually varied flow versus rapidly varied flow and when these models are suitable
  • Parameters for modeling breaches (time of formation, breach progression, sunny day v’s storm in progress)
My background is primarily modeling of dams using HEC-RAS unsteady and levee breaches using ISIS (Wallingford Software)

In an email dated 10/1/2008, Jill Butler wrote:
I'm very interested in your work and would like to be included in the discussion about modeling. Currently I am using a 2-D model for dam breach analysis, but I agree with you that some of the larger and/or more complicated situations need a better numerical model.

In an email dated 10/1/2008, Salam Murtada wrote:
I am a member of the Dam Safety workgroup and would like to participate in addressing modeling tools with regards to simulating dam systems. Please let me know if this is possible. I work with the Floodplain Unit of Minnesota as a Floodplain Hydrologist. I previously worked for the Ecosystem Enhancement Program, North Carolina and assisted in addressing FEMA requirement with regards to stream restoration projects.

In an email dated 10/1/2008, **Marc C.F. McIntosh** wrote:
I am interested in the modeling aspects of dam safety, please let me know how I can help

In an email dated 10/7/2008, **Saleem Ashraf** wrote:
I would like to join your sub-group, which will be working on the issues related to modeling tools for Dam Break Analysis within the Dam Safety working group. Please let me know your plans for this sub-group during October.

In an email dated 10/7/2008, **Kurt M. Baumann** wrote:
I participated in the Dam Working Group teleconference on September 30 and am interested in participating in the Modeling tools for dam break analysis subcommittee that you are chairing. Please let me know how I can be of assistance.

**Issues for Discussion**

- Preparation of tables comparing the modeling capabilities and options offered by different dam-break flood routing models and breach models, their advantages and disadvantages, etc.
- Benchmark tests for testing at least some of the models by the sub-group members.
- How to compare different models (1D vs 1D, and 1D vs 2D)?
- Guidelines, or at least some rules of thumb, regarding the selection of the most appropriate model for a given application
- Federal (FEMA?) guidelines for the use of unsteady models in dam break studies
- Tools need to analyze model results from the point of view of floodplain management and mapping
- Current scientific and technological gaps and needs with regard to “Modeling Tools for Dam Break Analysis”