“TD-39: Using HEC-RAS for Dam Break Studies”
Hydrologic Engineering Center, Davis California

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1D and 2D Unsteady Flow Routing
Pre-defined Breach Development
Inundation Mapping
Velocity Mapping
Arrival Times
Limitations

- 1D and 2D Unsteady Flow Modeling (not 3D or CFD)
- Breach is prescribed, not computed
- Simple Trapezoidal Breach Representation
- Numerical Stability
Scope of a HEC-RAS Dam Breach Model

- Hydrology
- HEC-RAS
  - Storage Area, 1D, or 2D
- Prescribed Breach Parameters
- HEC-RAS 1D or 2D
Types of Dam Breach Studies

- **Sunny Day Dam Failure**
  - Breach triggered by anything other than a hydrologic event
  - No hydrology necessary to run a simulation
  - Assumptions for failure trigger

- **Hydrologic Dam Failure**
  - Must develop an inflow hydrograph to the reservoir
  - Multiple flood peaks?
  - Typically much higher peak flows
HEC-RAS can route flows through the reservoir before, during, and after the dam breach event.

- Level Pool Routing – Storage Area
- 1D Dynamic – Cross Sections
- 2D Dynamic – 2D Mesh
**Level Pool Routing:** The discretized form of the Continuity Equation and an analytical or empirical relationship between stage or storage in the reservoir and discharge at the outlet of the reservoir.

\[
\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0
\]

\(A = \text{Flow Area}\)
\(Q = \text{Discharge}\)

\[
Q_{out} = Q_{in} - \frac{\Delta V}{\Delta t}
\]

\(V = \text{Reservoir Volume, Storage}\)

\[V = f(Q_{out})\]
Level Pool Routing in HEC-RAS

\[ Q_{out} = Q_{in} - \frac{\Delta V}{\Delta t} \]
• 1D Dynamic Routing: The full Dynamic Wave form of the St. Venant partial differential equation of Conservation of Momentum coupled with the Continuity Equation in the streamwise direction.

\[
\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = Sources - Sinks
\]

\[
\frac{1}{A} \frac{\partial Q}{\partial t} + \frac{1}{A} \frac{\partial}{\partial x} \left( \frac{Q^2}{A} \right) + g \frac{\partial y}{\partial x} - g(S_o - S_f) = 0
\]
Dynamic Drawdown – 1D

- 1D Dynamic Routing in HEC-RAS
  Discretized forms of the St. Venant Equations are used to solve for Stage and Flow at each cross section simultaneously for each timestep.
Dynamic Drawdown – 2D

- 2D Dynamic Routing: The full Dynamic Wave or Diffusive Wave form of the St. Venant partial differential equation of Conservation of Momentum coupled with the Continuity Equation in two dimensions (the x-y plane).

\[
\frac{\partial H}{\partial t} + \frac{\partial (hu)}{\partial x} + \frac{\partial (hv)}{\partial y} = \text{Sources} - \text{Sinks}
\]

\[
\frac{\partial u}{\partial t} + u \frac{\partial u}{\partial x} + v \frac{\partial u}{\partial y} = -g \frac{\partial H}{\partial x} + v_t \left( \frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2} \right) - c_f u + f v
\]

\[
\frac{\partial v}{\partial t} + u \frac{\partial v}{\partial x} + v \frac{\partial v}{\partial y} = -g \frac{\partial H}{\partial y} + v_t \left( \frac{\partial^2 v}{\partial x^2} + \frac{\partial^2 v}{\partial y^2} \right) - c_f v - f u
\]
2D Dynamic Routing in HEC-RAS
Discretized forms of the St. Venant Equations are used to solve for Stage and Flow at each cell in a 2D mesh.

\[ Q = CLH^{3/2} \]
Level Pool or Dynamic?
The 4 independent variables were combined into a Compaction Factor, which measures the “Compactness” of a reservoir, and a Translation Factor, which measures the rate at which water can replenish the drawdown effect.

\[ F_c = \frac{H}{L} \quad F_t = \frac{ct}{L} \]

Where:  
\[ F_c = \text{Reservoir Compaction Factor} \]
\[ F_t = \text{Reservoir Translation Factor} \]
\[ H = \text{Breach Height} \]
\[ t = \text{Failure Time} \]
\[ c = \text{Wave Celerity} = (gD)^{0.5} \]
\[ L = \text{Reservoir Length} \]
The Drawdown Number, $D_n$, is a gage that can be used to determine when Level Pool Reservoir Routing for Dam Breach events is an Acceptable Routing Method.

$$D_n = F_c \times F_t$$

- High values of $D_n$ correspond with low QDiffs
  - Level Pool is a good representation of reservoir drawdown.

- Low values of $D_n$ correspond with high QDiffs
  - Level Pool is NOT a good representation of reservoir drawdown.
Level Pool or Dynamic?

Dynamic vs. Level Pool Reservoir Routing

Q_{diff} (Dynamic vs. Level Pool), %

\( D_n \)

5%
A reservoir is 60 ft high and about 1800 ft long.

1. Compute the Failure Time using Von Thun and Gillette’s equation: \( t = 0.015h_w \) (after converting to metric), \( t = 0.27 \) hours = 972 seconds.

2. Wave Celerity, \( c = (gD)^{0.5} = 31 \) ft/s (\( D = H/2 \))

3. Compaction Factor, \( F_c = H/L = 0.033 \)

4. Translation Factor, \( F_t = ct/L = 16.74 \)

5. \( D_n = 0.55 > 0.41 \), Level Pool is an appropriate Dam Breach Reservoir Drawdown Method (to within 5%).
Breach Development

Overtopping

Piping
## Breach Development

<table>
<thead>
<tr>
<th>Dam Type</th>
<th>Average Breach Width</th>
<th>Average Component of Breach Side Slope (H) (H:V)</th>
<th>Failure Time, $t_f$ (hours)</th>
<th>Agency</th>
</tr>
</thead>
<tbody>
<tr>
<td>Earthen/Rockfill</td>
<td>(0.5 to 3.0) x HD</td>
<td>0 to 1.0</td>
<td>0.5 to 4.0</td>
<td>USACE 1980</td>
</tr>
<tr>
<td></td>
<td>(1.0 to 5.0) x HD</td>
<td>0 to 1.0</td>
<td>0.1 to 1.0</td>
<td>FERC NWS</td>
</tr>
<tr>
<td></td>
<td>(2.0 to 5.0) x HD</td>
<td>0 to 1.0 (slightly larger)</td>
<td>0.1 to 1.0</td>
<td>NWS USACE 2007</td>
</tr>
<tr>
<td></td>
<td>(0.5 to 5.0) x HD*</td>
<td>0 to 1.0</td>
<td>0.1 to 4.0*</td>
<td></td>
</tr>
<tr>
<td>Concrete Gravity</td>
<td>Multiple Monoliths</td>
<td>Vertical</td>
<td>0.1 to 0.5</td>
<td>USACE 1980</td>
</tr>
<tr>
<td></td>
<td>Usually ≤ 0.5 L</td>
<td>Vertical</td>
<td>0.1 to 0.3</td>
<td>FERC USACE 2007</td>
</tr>
<tr>
<td></td>
<td>Usually ≤ 0.5 L</td>
<td>Vertical</td>
<td>0.1 to 0.2</td>
<td>NWS USACE 2007</td>
</tr>
<tr>
<td></td>
<td>Multiple Monoliths</td>
<td>Vertical</td>
<td>0.1 to 0.5</td>
<td></td>
</tr>
<tr>
<td>Concrete Arch</td>
<td>Entire Dam</td>
<td>Valley wall slope</td>
<td>≤ 0.1</td>
<td>USACE 1980</td>
</tr>
<tr>
<td></td>
<td>Entire Dam</td>
<td>0 to valley walls</td>
<td>≤ 0.1</td>
<td>FERC NWS</td>
</tr>
<tr>
<td></td>
<td>(0.8 x L) to L</td>
<td>0 to valley walls</td>
<td>≤ 0.1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(0.8 x L) to L</td>
<td>0 to valley walls</td>
<td>≤ 0.1</td>
<td>USACE 2007</td>
</tr>
<tr>
<td>Slag/Refuse</td>
<td>(0.8 x L) to L</td>
<td>1.0 to 2.0</td>
<td>0.1 to 0.3</td>
<td>FERC NWS</td>
</tr>
<tr>
<td></td>
<td>(0.8 x L) to L</td>
<td>1.0 to 2.0</td>
<td>0.1 to 1.0</td>
<td></td>
</tr>
</tbody>
</table>

*Note: Dams that have very large volumes of water, and have long dam crest lengths, will continue to erode for long durations (i.e., as long as a significant amount of water is flowing through the breach), and may therefore have longer breach widths and times than what is shown in Table 3. HD = height of the dam; L = length of the dam crest; FERC - Federal Energy Regulatory Commission; NWS - National Weather Service*
### Table 2 - Breach Parameter relations based on dam-failure case studies.
For explanations of symbols see the Notation section at the end of this report.

<table>
<thead>
<tr>
<th>Reference</th>
<th>Number of Case Studies</th>
<th>Relations Proposed (S.I. units, meters, m^3/s, hours)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Johnson and Illes (1976)</td>
<td>20</td>
<td>$0.5h_d \leq B \leq 5h_d$ for earthfill dams</td>
</tr>
<tr>
<td>Singh and Snorrason (1982, 1984)</td>
<td></td>
<td>$2h_d \leq B \leq h_d$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$0.15$ meters $\leq d_{avg} \leq 0.61$ meters</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$0.25$ hours $\leq t_f \leq 1.0$ hours</td>
</tr>
<tr>
<td>MacDonald and Langridge-Monopolis</td>
<td>42</td>
<td><strong>Earthfill dams:</strong> $V_{out} = 0.0261(V_{out} * h_d)^{0.769}$ [best-fit]</td>
</tr>
<tr>
<td>(1984)</td>
<td></td>
<td>$t_f = 0.0179(V_{out})^{0.564}$ [upper envelope]</td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>Non-earthfill dams:</strong> $V_{out} = 0.00348(V_{out} * h_d)^{0.852}$ [best-fit]</td>
</tr>
<tr>
<td>FERC (1987)</td>
<td></td>
<td>$B$ is normally 2-4 times $h_d$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$B$ can range from 1-5 times $h_d$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$Z = 0.25$ to 1.0 [engineered, compacted dams]</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$Z = 1$ to 2 [non-engineered, slag or refuse dams]</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$t_f = 0.1$-1 hours [engineered, compacted earth dams]</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$t_f = 0.1$-0.5 hours [non-engineered, poorly compacted]</td>
</tr>
<tr>
<td>Froehlich (1987)</td>
<td>43</td>
<td>$B = 0.47K_o(S)^{0.25}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$K_o = 1.4$ overtopping; 1.0 otherwise</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$Z = 0.75K_e(h_w)^{1.57}(W)^{0.73}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$K_e = 0.6$ with corewall; 1.0 without a corewall</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$t_f = 79(S)^{0.47}$</td>
</tr>
<tr>
<td>Reclamation (1988)</td>
<td></td>
<td>$B = (3)h_w$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$t_f = (0.011)B$</td>
</tr>
<tr>
<td>Singh and Scarlato (1988)</td>
<td>52</td>
<td>Breach geometry and time of failure tendencies $B_{top}/B_{bottom}$ averages 1.29</td>
</tr>
<tr>
<td>Von Thun and Gillette (1990)</td>
<td>57</td>
<td>$B$, $Z$, $t_f$ guidance (see discussion)</td>
</tr>
<tr>
<td>Dewey and Gillette (1993)</td>
<td>57</td>
<td>Breach initiation model; $B$, $Z$, $t_f$ guidance</td>
</tr>
<tr>
<td>Froehlich (1995b)</td>
<td>63</td>
<td>$B = 0.1803K_oV_{sw}^{0.32}h_{w}^{0.19}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$t_f = 0.00254V_{sw}^{0.55}h_{w}^{0.90}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$K_o = 1.4$ for overtopping; 1.0 otherwise</td>
</tr>
</tbody>
</table>
Breach Development

Dam (Inline Structure) Breach Data

- **Inline Structure**: Bald Eagle Cr. Lock Haven 81454
- **Center Station**: 5250
- **Final Bottom Width**: 446
- **Final Bottom Elevation**: 585
- **Left Side Slope**: 0.9
- **Right Side Slope**: 0.9
- **Breach Weir Coef**: 2.6
- **Full Formation Time (hrs)**: 3.2
- **Failure Mode**: Piping
- **Piping Coefficient**: 0.5
- **Initial Piping Elev**: 620
- **Trigger Failure at**: WS Elev
- **Starting WS**: 676.8

Breach Plot | Breach Progression | Breach Repair (optional)

Legend:
- Ground
- Bank Sta
- Final Breach
Cross sections should first be placed at representative locations to describe the changes in geometry.

Additional cross sections should be added at locations where changes occur in discharge, slope, velocity, and roughness.

Cross sections must also be added at levees, bridges, culverts, and other structures.

Finally, there must be adequate cross sections to satisfy the numerical solution.

Samuels (1989) and Fread (2003) have equations for maximum spacing.

Other references (USGS, USACE)
Cross Section Spacing
Samuels’ Equation for Cross Section Spacing

\[ \Delta x \leq \frac{0.15D}{S_0} \]

Where: $\Delta x$ is the cross section spacing distance; 
$D$ is the bankfull depth; and 
$S_0$ is the bed slope

Fread’s Equation for Cross Section Spacing

\[ \Delta x \leq \frac{c T_r}{20} \]

Where: $\Delta x$ is the cross section spacing distance; 
c is the wave celerity; and 
$T_r$ is minimum rise time of the Dam Breach hydrograph (use Breach Development Time for rough approximation)
At problem areas try “halving” the spacing and continue until the profile looks more reasonable.

Stay Reasonable. i.e. Spacing less than 10 m probably suggests a reach that is too steep or has some other problem.
Courant Condition

\[ C = \frac{V_w \Delta T}{\Delta X} \leq 1 \]

\[ \Delta T \leq \frac{\Delta x}{V_w} \]
Minimum Flow

- For Dams on very small or ephemeral streams, may need to specify a minimum flow to keep the channel from going dry during the simulation prior to the dam failure.
- Be sure the minimum flow is negligible compared to the dam break floodwave peak flow.
- Should be less than 10% of the peak.
When the Froude Number approaches 1 during a simulation, the inertial terms of the St. Venant equation tend to propagate errors towards instability.

Turn on “Mixed Flow” in the Unsteady Flow Simulation Window.
Manning’s n values can have a big influence on the stability of dam break models.

- Common to underestimate n values particularly in steep reaches.
- Can cause water surface elevations to dip too low.
- Can cause supercritical flow.
- Can cause velocities to be too high.
- Jarrett’s equation is a good check for steep streams.

\[ n = 0.39S^{0.38}R^{-0.16} \]

where \( S \) = energy slope (0.002 < \( S \) < 0.04)
\( R \) = Hydraulic Radius (0.5 < \( R \) < 7.0)
Manning’s n Values

- May need to increase n values just downstream of the dam (particularly for replication studies).
  - Highly turbulent during the breach
  - Lots of sediment/debris mixed in with the water
  - Very subjective!
  - HEC suggests doubling the n value just downstream of a breached dam until a point downstream where sediment and debris have fallen out.
Thank You!

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WEST Consultants

“The RAS Solution”
www.rasmodel.com