
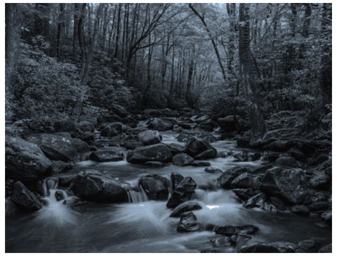










Detailed Inspection Training
Hydrology and Hydraulics







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December 2025

CDM Smith

Agenda

1. Review Available Information
2. Regulatory Overview
3. Hydrologic and Watershed Characteristics
4. Dam Hydraulics
5. Dam Breach Analysis
6. Incremental Consequence Analysis
7. Detailed Example of Calculating SDF using ICA
8. Conclusion

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Review Available Information

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Review Available Data

- Check with:
  - Dam Owners
  - SCDES Dam Safety Office
- Check for:
  - Dam Design Reports
  - Inspection Reports
  - H&H Studies
  - Breach Studies
  - Instrumentation on the Dam
  - Construction Reports
  - Emergency Action Plans
  - Gage records (WSEL and discharge)
  - O&M manual for gate/pool operation

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Regulatory Overview

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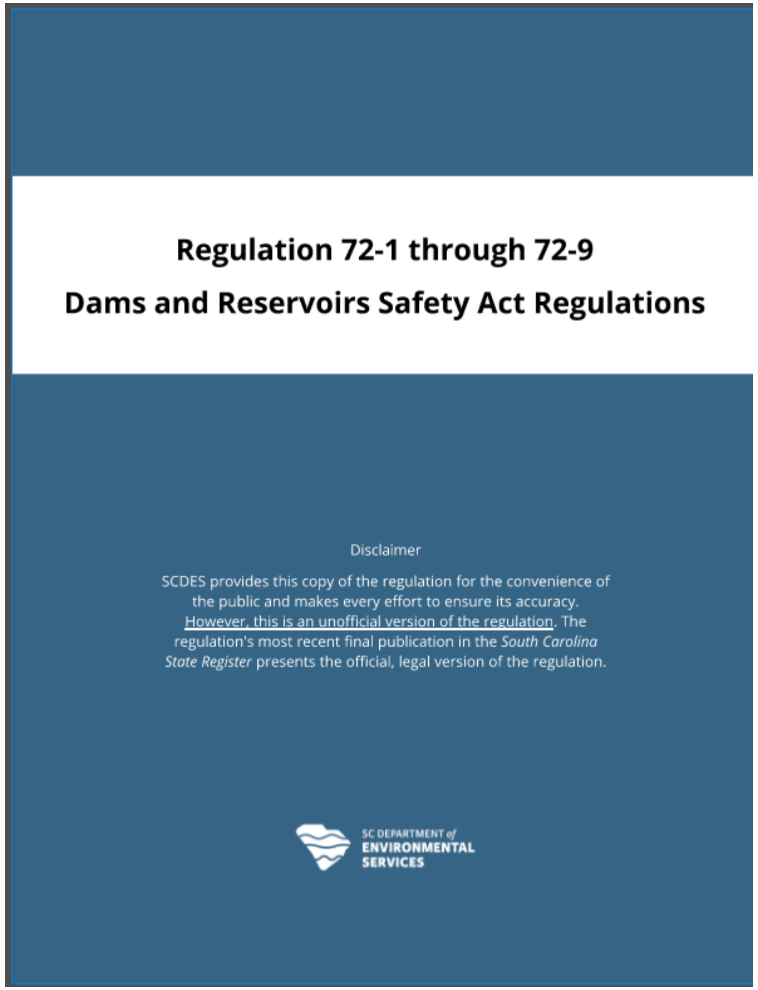
Regulatory Overview

- Dam Safety Act Requirements
  - Regulation 72-1.J
  - Regulation 72-4.D.1
  - Regulation 72.4.D.2


Regulation 72-1 through 72-9

Dams and Reservoirs Safety Act Regulations

Disclaimer

SCDES provides this copy of the regulation for the convenience of the public and makes every effort to ensure its accuracy. However, this is an unofficial version of the regulation. The regulation's most recent final publication in the *South Carolina State Register* presents the official, legal version of the regulation.

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Excerpt from Regulation 72-1.J

J. "Detailed inspection" means all studies, investigations and analyses necessary to evaluate conclusively the structural safety and hydraulic capacity of a dam or reservoir and appurtenant works. This inspection may include but is not limited to soil analysis, concrete or earth stability analysis, materials testing, foundation explorations, hydrologic analysis, including basin studies and flood potential. This inspection shall be performed by a qualified registered professional engineer.

Regulatory Overview

<u>**Approved Hydrology and Hydraulics methods and software:**</u>

“hydrologic analysis includes basin studies and flood potential.”

”hydraulic analysis evaluates capacity of a dam reservoir and appurtenant works.”

- US Army Corps of Engineers : HEC1, HEC HMS, MetVue, HEC2, HEC RAS
- US Dept of Agriculture: TR20, TR55, WinDam, WSP2
- U.S. Geological Survey (USGS) Software packages
- U.S. Bureau of Reclamation (USBR) Tools

Regulatory Overview

Hydrograph Routing:

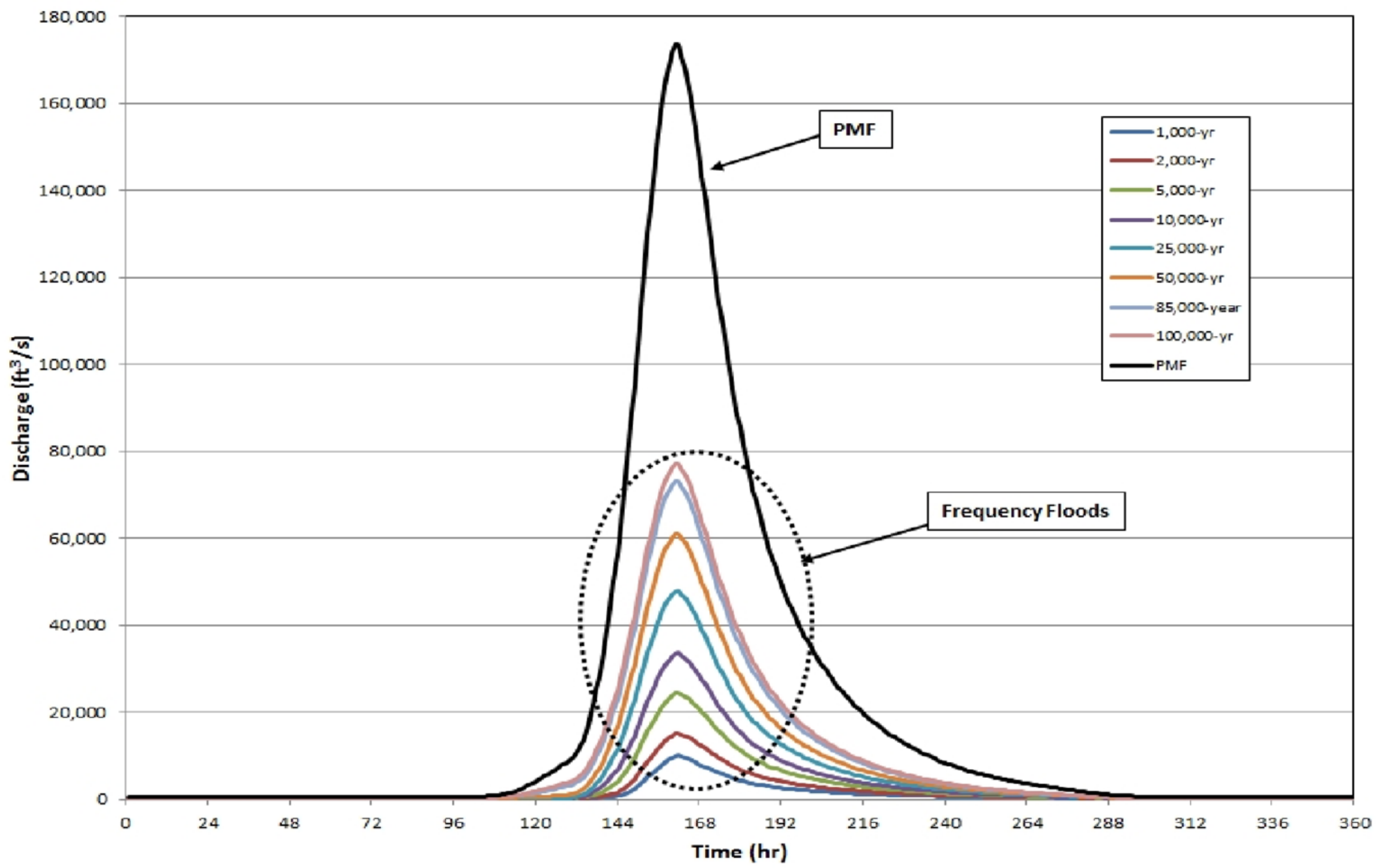

Figure 2.4.1-1. Example frequency flood hydrographs and relationship to the PMF

Regulatory Overview

Spillway Capacity Required:

TABLE I

SPILLWAY DESIGN FLOOD CRITERIA

Hazard	Size	Spillway Design Flood (SDF)*
High	Very Small	100-yr. to ½ PMF
	Small	½ PMF to PMF
	Intermediate	PMF
	Large	PMF
Significant	Small	100-yr. to ½ PMF
	Intermediate	½ PMF to PMF
	Large	PMF
Low	Small	50 to 100-yr. frequency
	Intermediate	100-yr. to ½ PMF
	Large	½ PMF to PMF

*Note: When appropriate, the spillway design flood may be reduced to the spillway discharge at which dam failure will not significantly increase the downstream hazard which exists just prior to dam failure.

Regulatory Overview

Spillway Capacity Required:

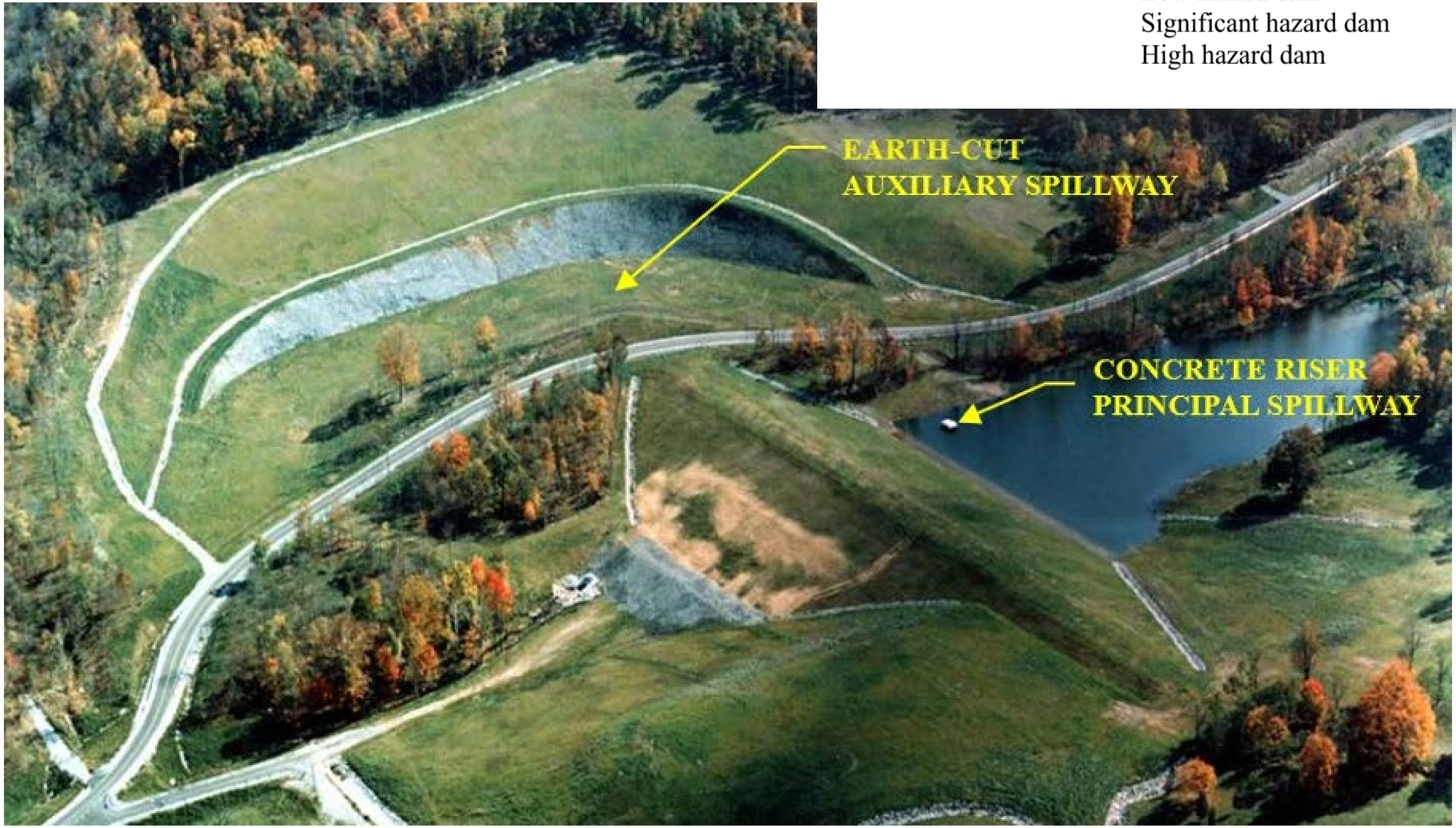

TABLE III
MINIMUM FREQUENCY OF USE
EARTH VEGETATED SPILLWAYS

Low hazard dam	1 year
Significant hazard dam	10 years
High hazard dam	25 years

Per Regulation 72-3.D.2.a. (10)

"… All earth vegetated spillways shall be designed for an appropriate minimum frequency of use, as shown in Table III. The spillway system shall have adequate capacity to remove from the reservoir within 10 days following passage of the design flood peak at least eighty percent of the water temporarily detained in the reservoir above the elevation of the primary spillway."

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Hydrologic and Watershed Characteristics

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Hydrologic and Watershed Characteristics

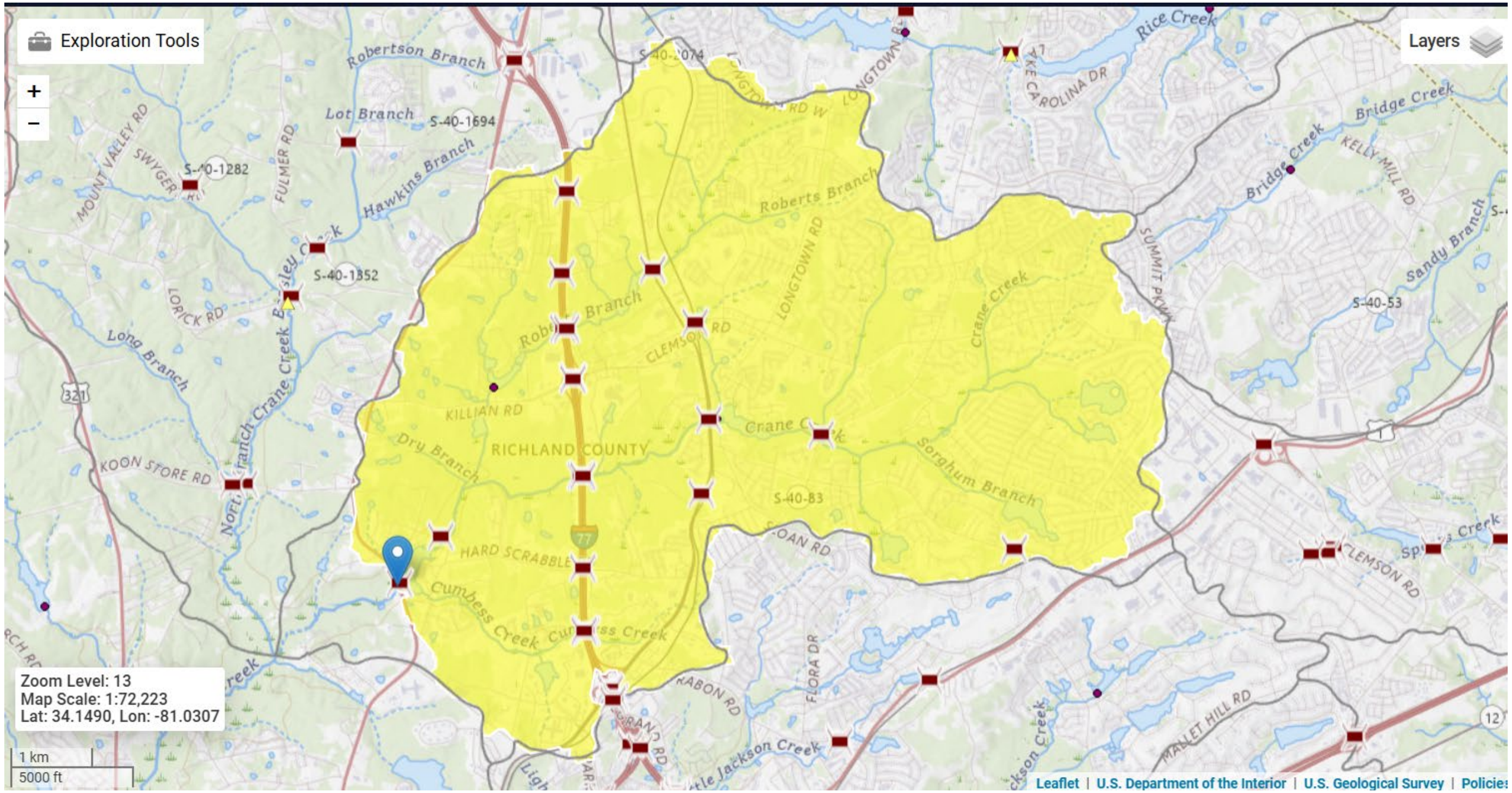
- Drainage Area:

  <u>Watershed Delineation:</u>

  - USGS Topographic Maps
  - USGS StreamStats

  <u>Runoff Coefficients:</u>

  - "LandUse" using national land coverage database (NLCD)
  - "Hydrologic Soil Group" using Soil Survey Geographic Database (SSURGO)



https://streamstats.usgs.gov/ss/

Hydrologic and Watershed Characteristics

- Meteorological Evaluation
  - Probable Maximum Precipitation (PMP)

  …"the greatest depth of precipitation for a given duration meteorologically possible for a design watershed or a given storm area at a particular location at a particular time of year."

  - Represents worst case maximum rainfall.
  - Probable Maximum Flood (PMF)
    - The largest reasonably plausible flood discharge that could occur at a specific location.
- Meteorological Evaluation Methods
    - HMR51- Hydrometeorological Report No. 51 (PMP charts)
    - HMR52 - Hydrometeorological Report No. 52 (Storm Maximization Program)
    - USACE HEC-HMS
    - USACE MetVue software

Hydrologic and Watershed Characteristics

- HMR 51- Hydrometeorological Report No. 51 (PMP charts)

  Hydrometeorological Report No. 51 (HMR 51) contains the PMP index maps for the Eastern U.S. (NWS 1978)

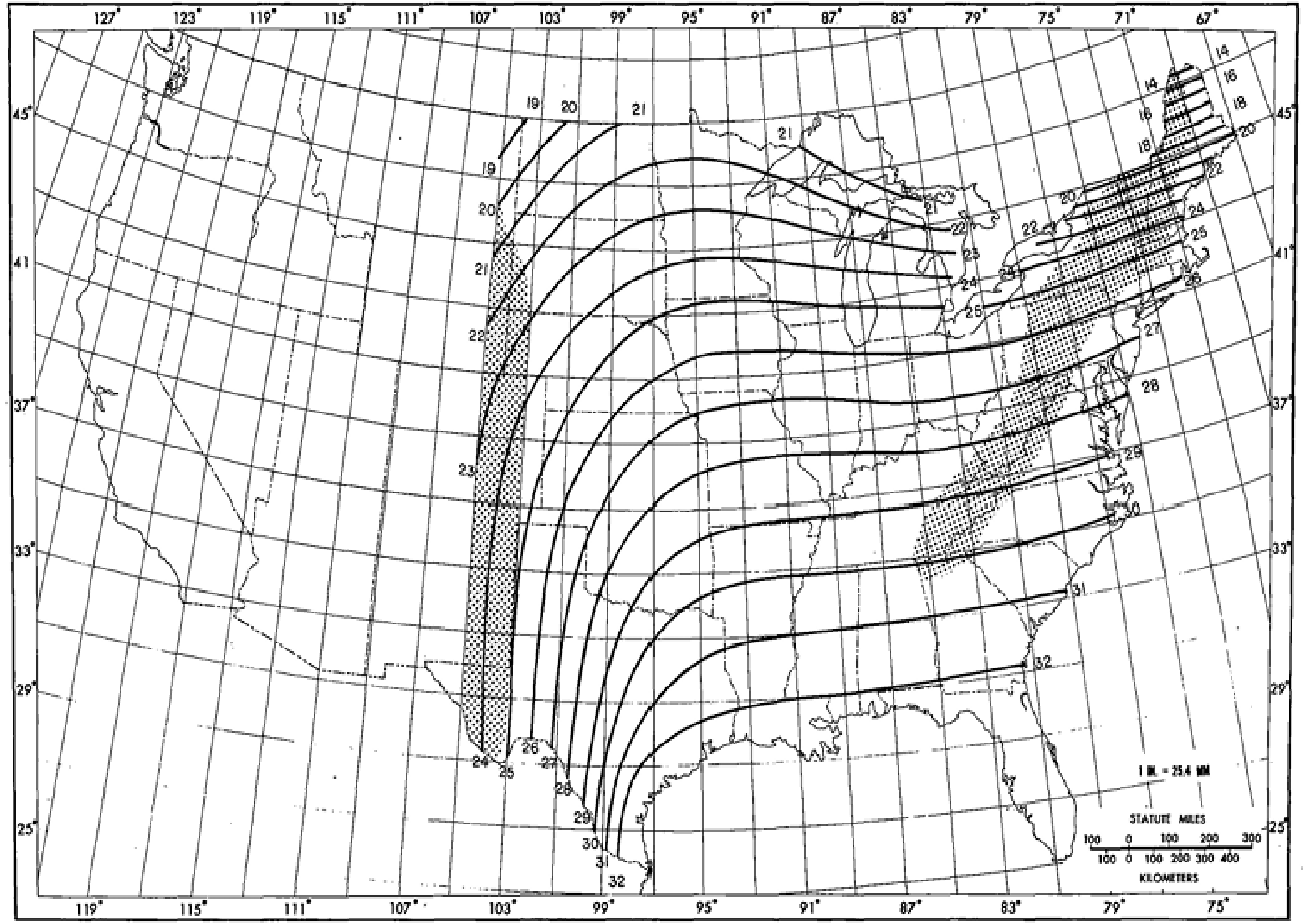


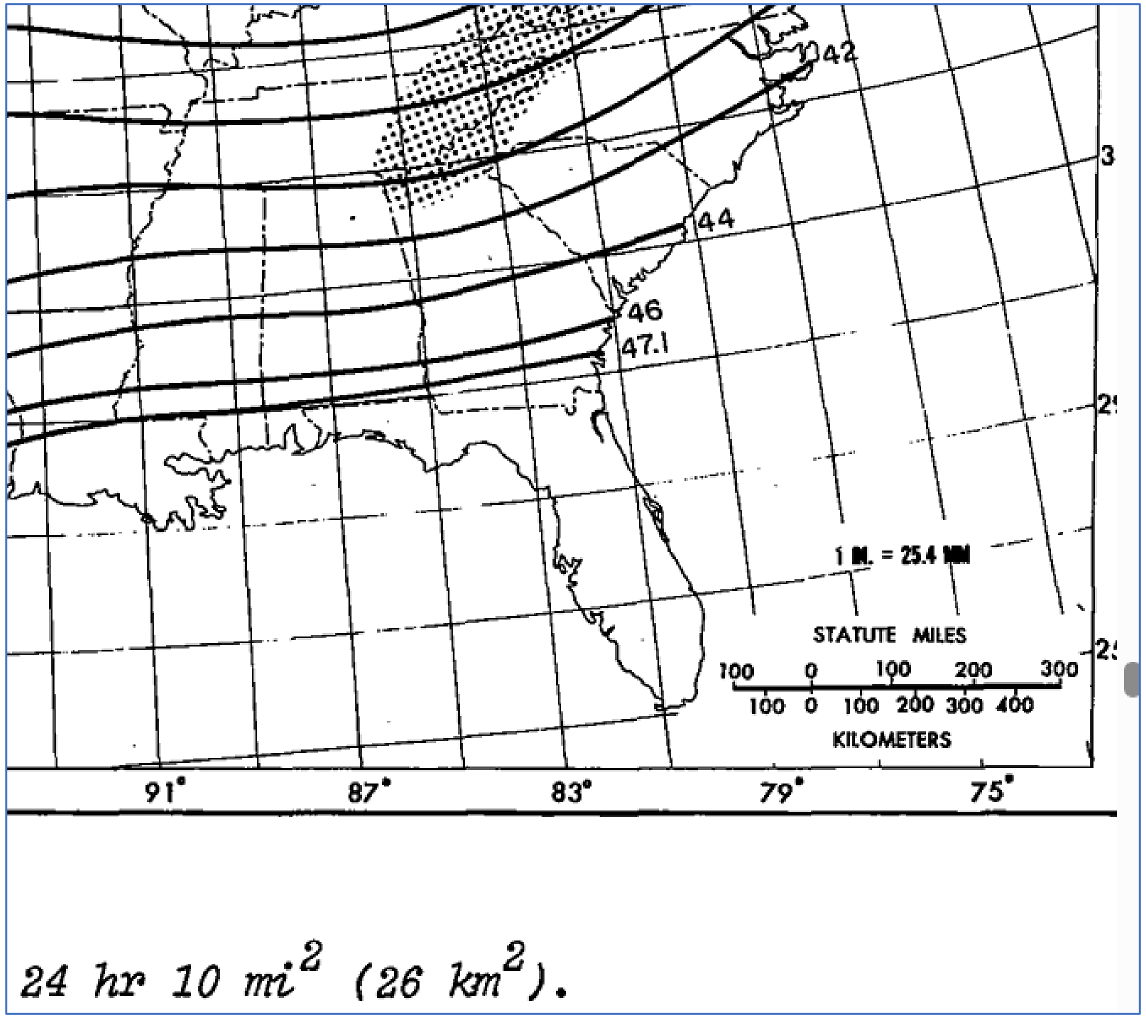
Figure 18.--All-season PMP (in.) for 6 hr 10 mi^2 (26 km^2).

https://www.weather.gov/owp/hdsc_pmp

Hydrologic and Watershed Characteristics

- HMR 51- Hydrometeorological Report No. 51 (PMP charts)

South Carolina 24 hr duration PMP ranges between 40" and 46" for 10 sq mi.



https://www.weather.gov/owp/hdsc_pmp

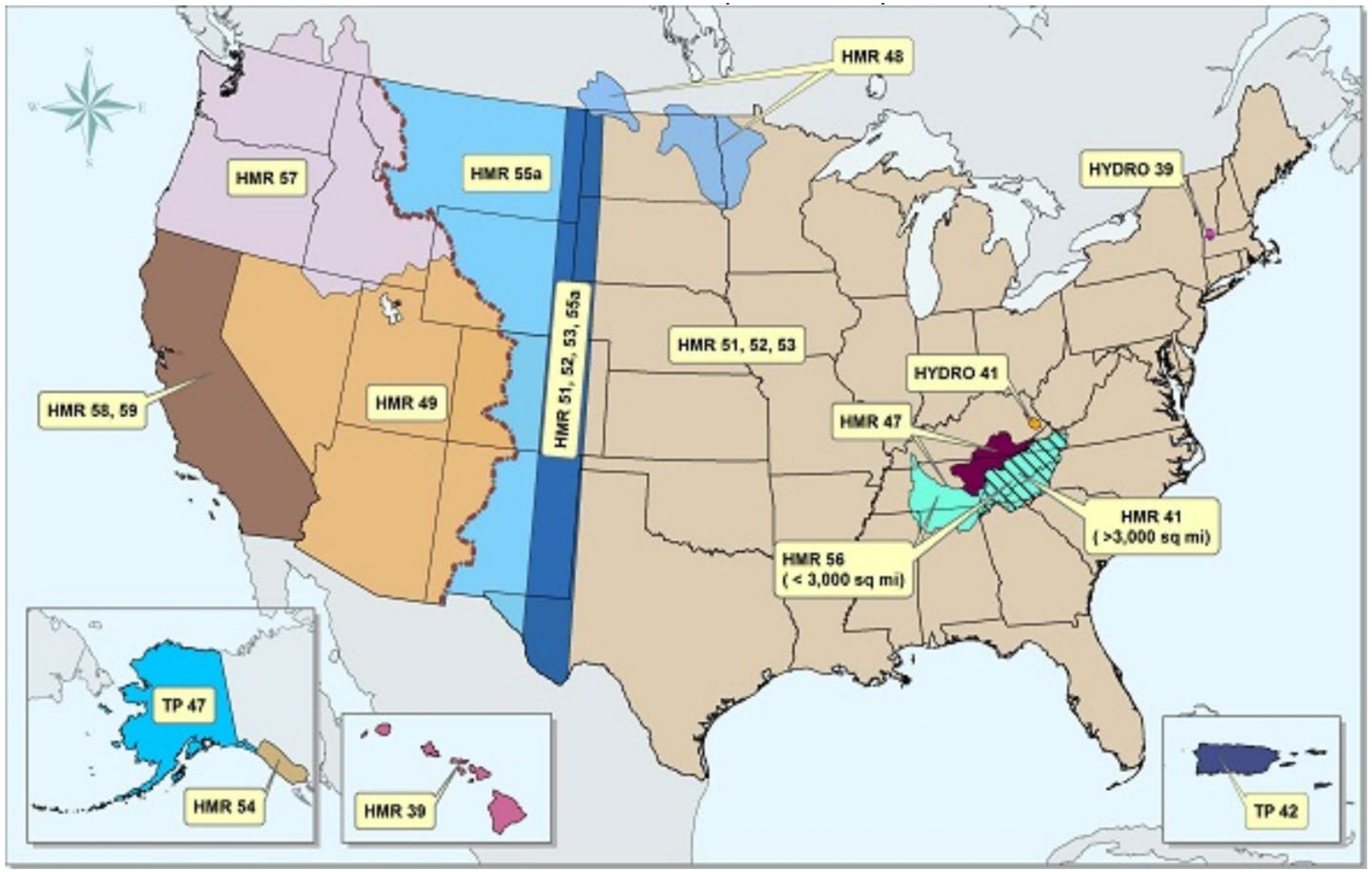
Hydrologic and Watershed Characteristics

HMR 52 - Hydrometeorological Report No. 52 (Storm Maximization Program)

The HMR 52 Storm method generates a probable maximum precipitation (PMP) hypothetical storm as detailed in Hydrometeorological Report No. 52 (HMR 52) (Hansen, Schreiner, and Miller, 1982).

Hydrometeorological Report No. 51 (HMR 51) contains the PMP index maps for the Eastern U.S., and HMR 52 contains information about the application of the PMP depths to a watershed.

HMR 51 and HMR 52 apply to those areas of the United States east of the 105th meridian, with some exclusions, as shown in the figure.

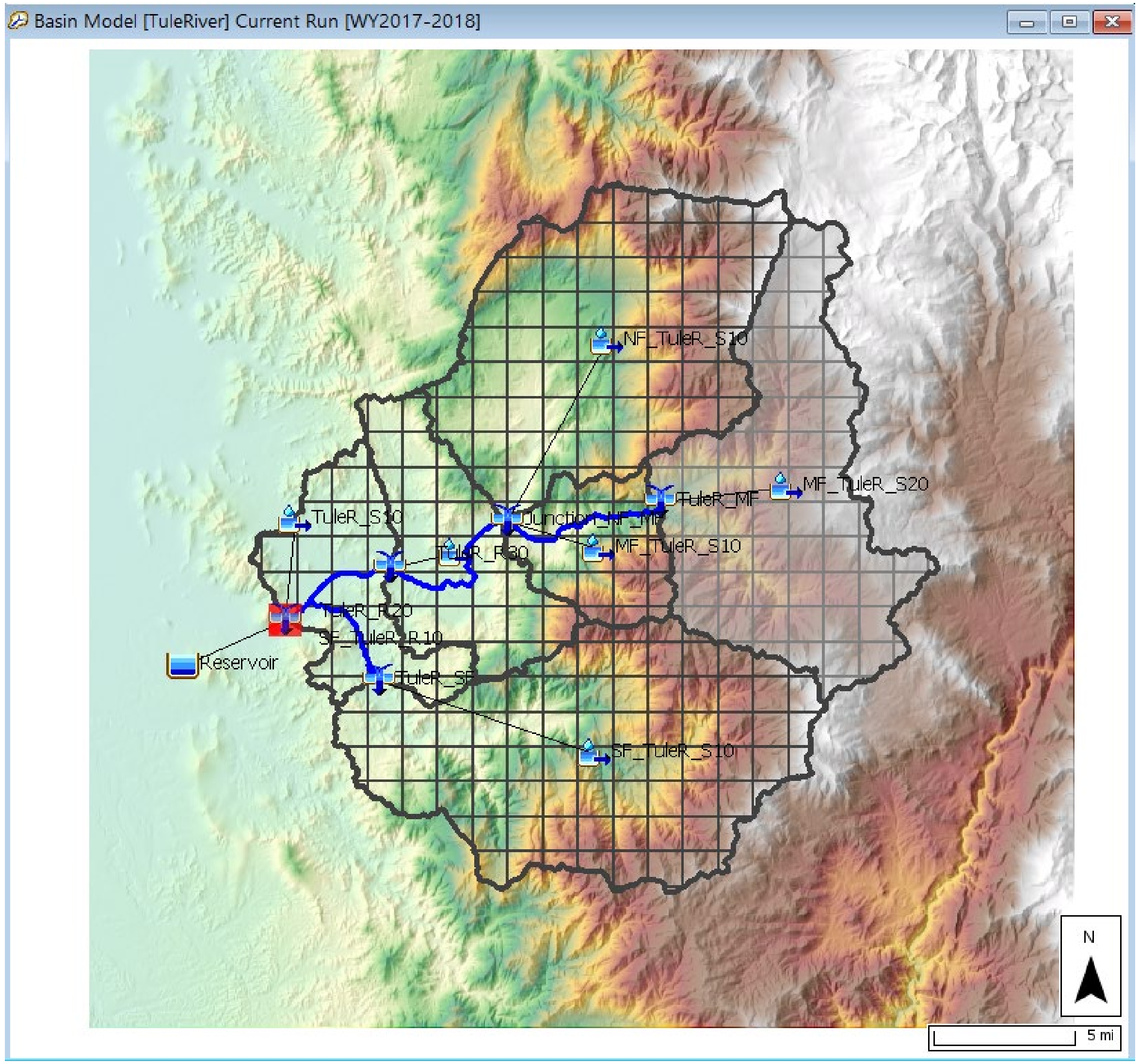


https://www.hec.usace.army.mil/confluence/hmsdocs/hmstrm/meteorology/precipitation/hmr-52-storm

Hydrologic and Watershed Characteristics

USACE HEC-HMS

The Hydrologic Modeling System (HEC-HMS) is designed to simulate the complete hydrologic processes of dendritic watershed systems. The software includes many traditional hydrologic analysis procedures such as event infiltration, unit hydrographs, and hydrologic routing.



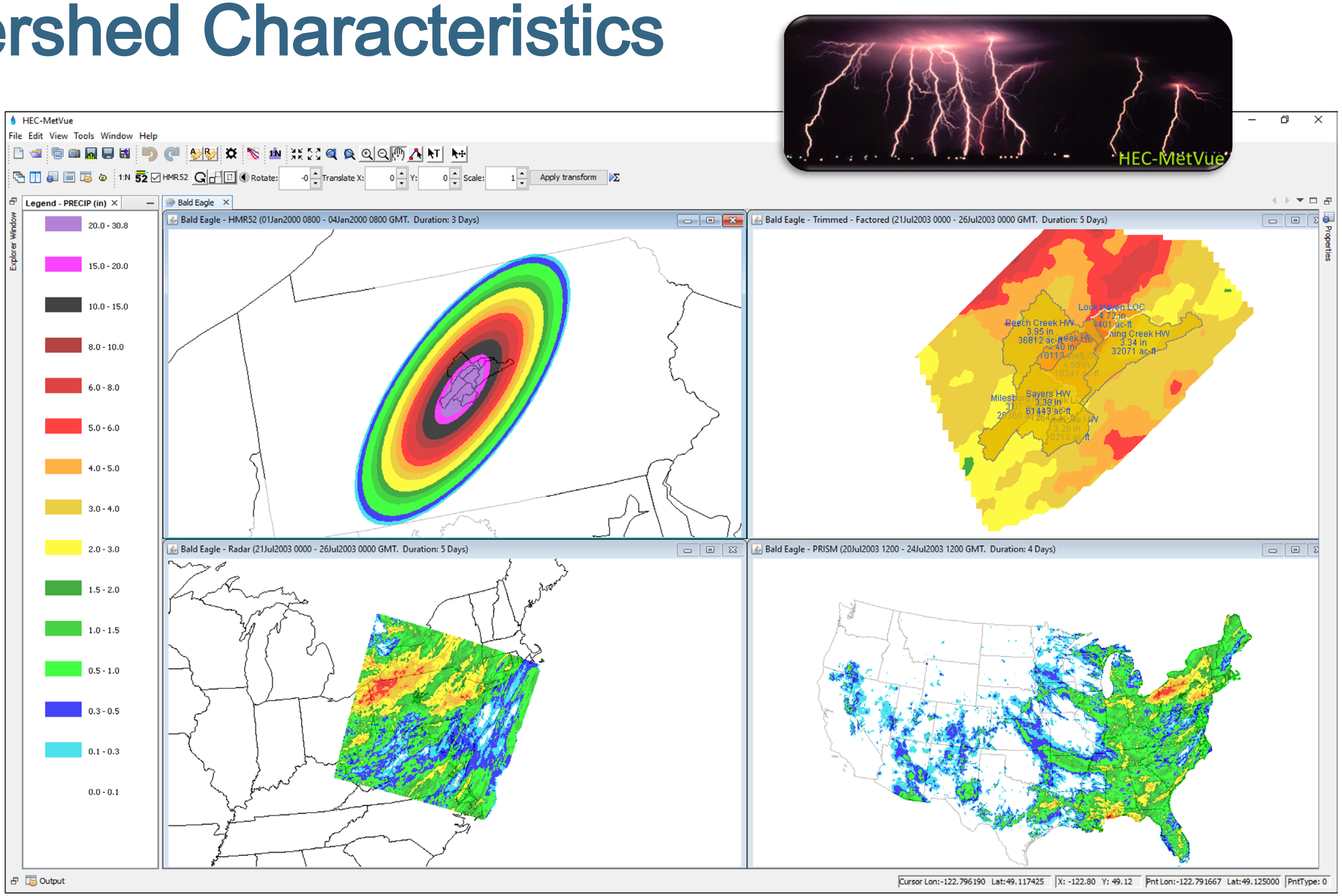
https://www.hec.usace.army.mil/software/hec-hms/

Hydrologic and Watershed Characteristics

- USACE MetVue software

Meteorological Visualization Utility Engine (HEC-MetVue)

HEC-MetVue facilitates viewing and manipulation of meteorological datasets as well as performing a variety of computations and analyses, including temporal and spatial aggregation of datasets and areal average computations.

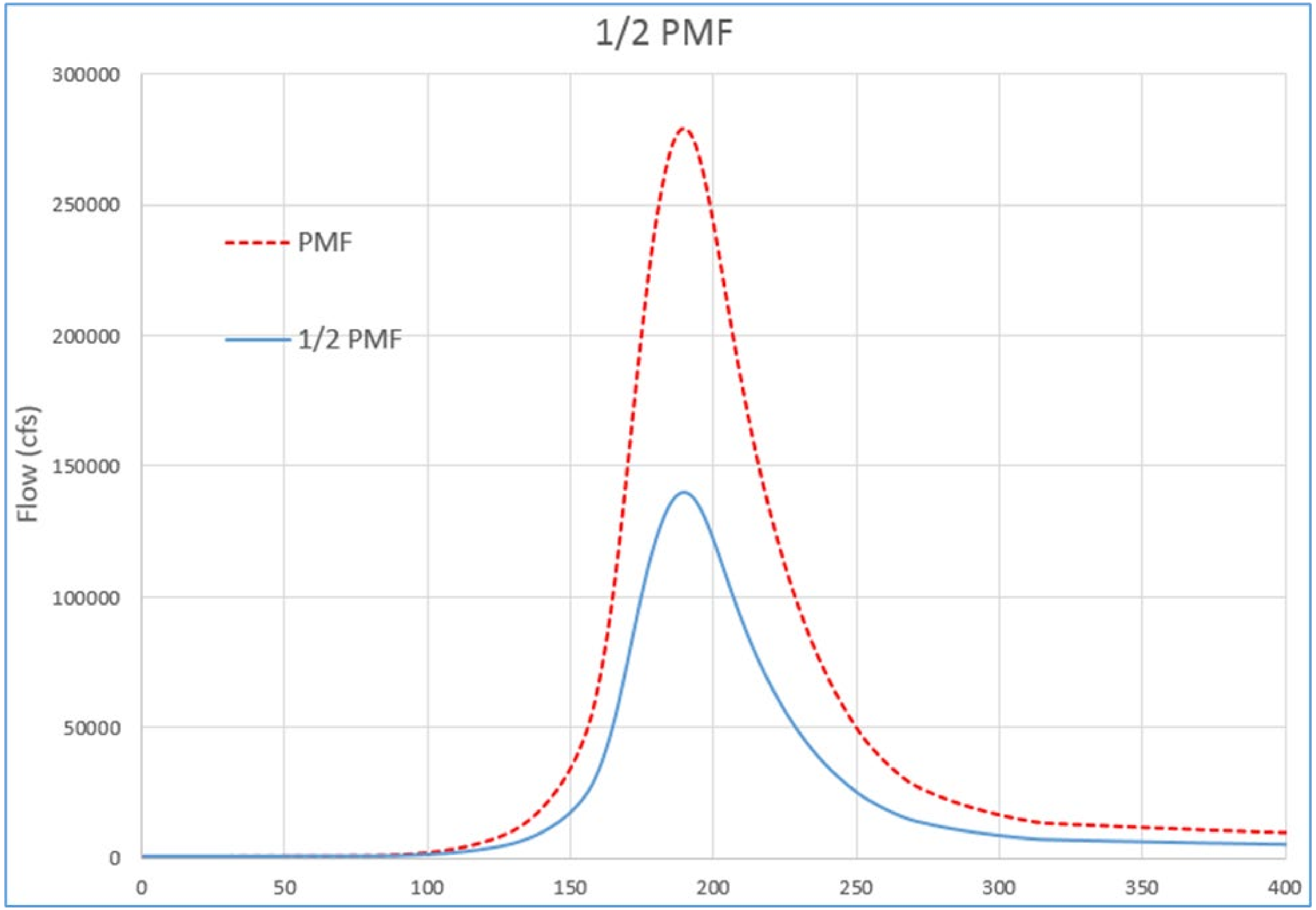


https://www.hec.usace.army.mil/software/hec-metvue/

Hydrologic and Watershed Characteristics

- Dam Evaluation
  - Spillway Design Flood (SDF)

  Sometime referred as Inflow Design Flood (IDF) which is the maximum flood hydrograph that a dam, its reservoir, and spillway system are designed to pass safely without risking the integrity of the dam.

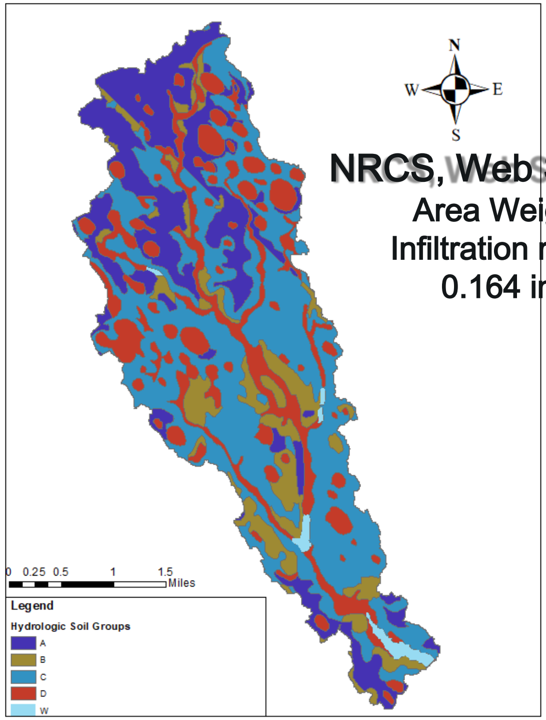


Hydrologic and Watershed Characteristics

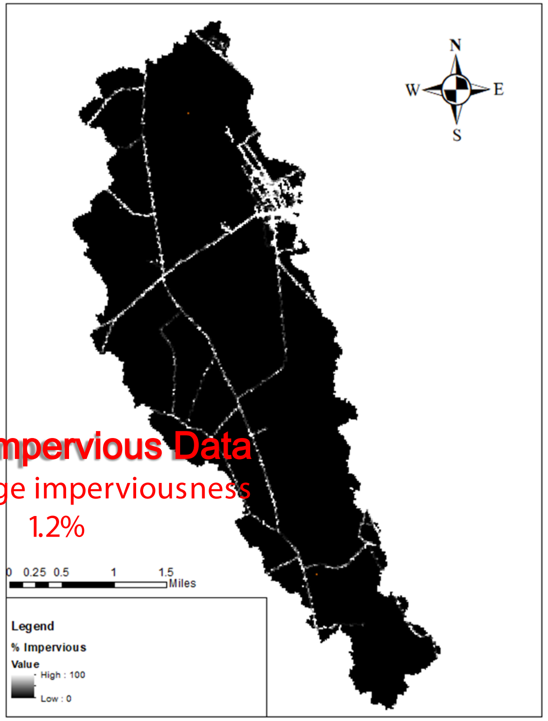
Case Study- McNair Millpond Dam

Basin Delineation and Watershed Parameters

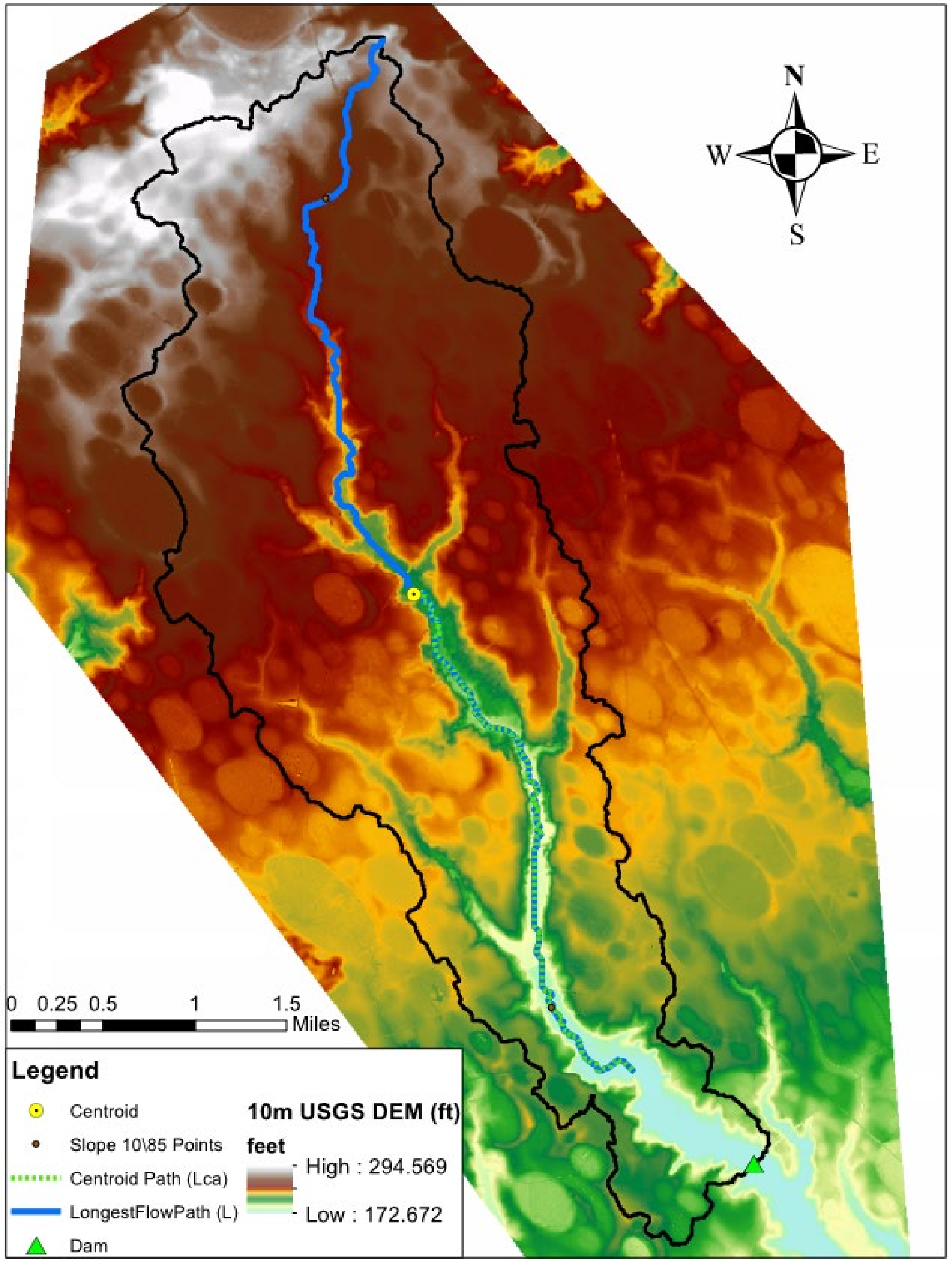
Parameter	Value
Watershed Area =	9.56 sq.mi.
Longest Flow Path (L) =	7.83 miles
Longest Path to Centroid (Lca) =	3.76 miles
Elevation at 10% of L =	187.88 ft
Elevation at 85% of L =	245.93 ft
Basin Slope using 10/85% along Flowpath (S) =	9.87 ft/mile
Lumped roughness coefficient (Kn)	0.07



NRCS, Web Soil Survey
Area Weighted
Infiltration rate of
0.164 in/hr

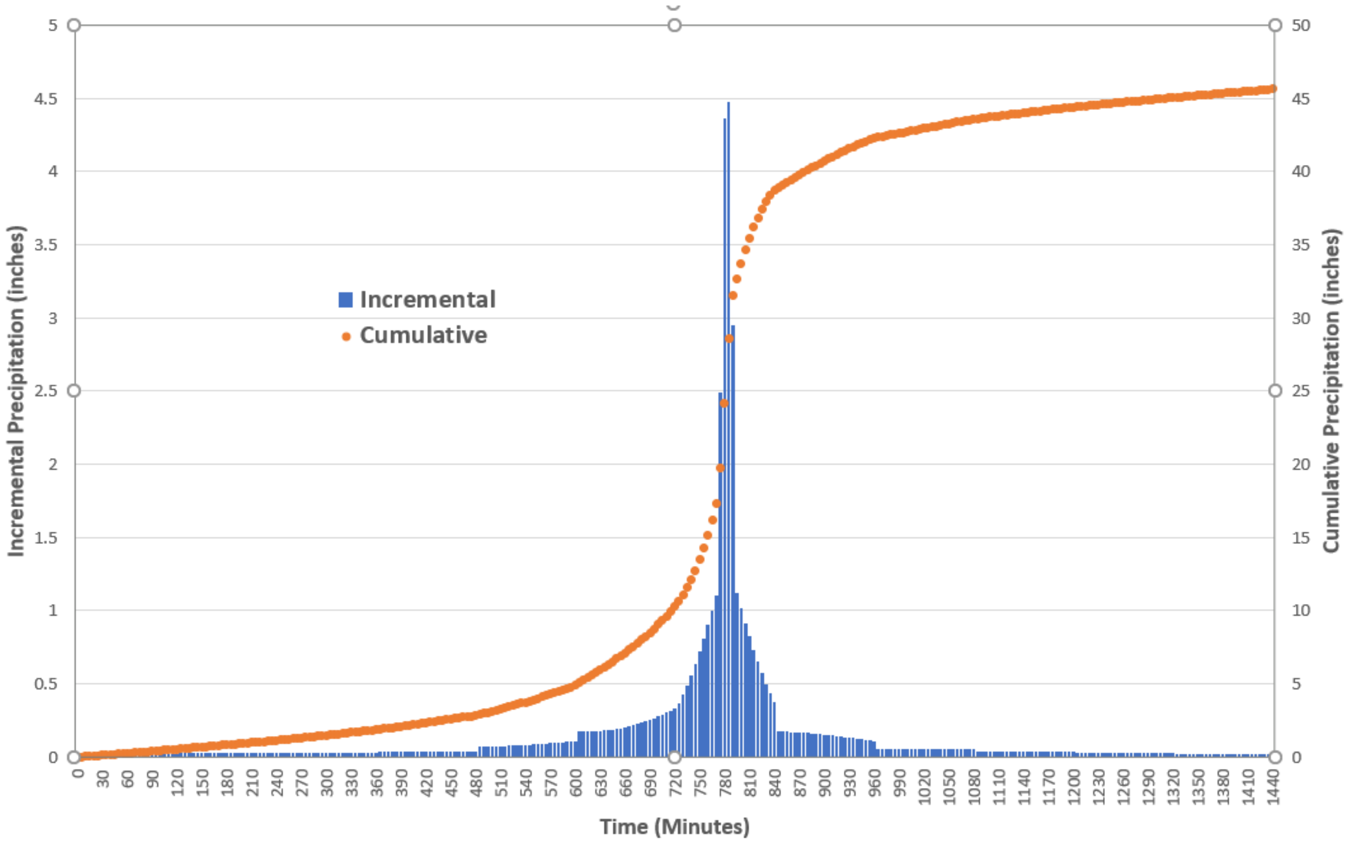


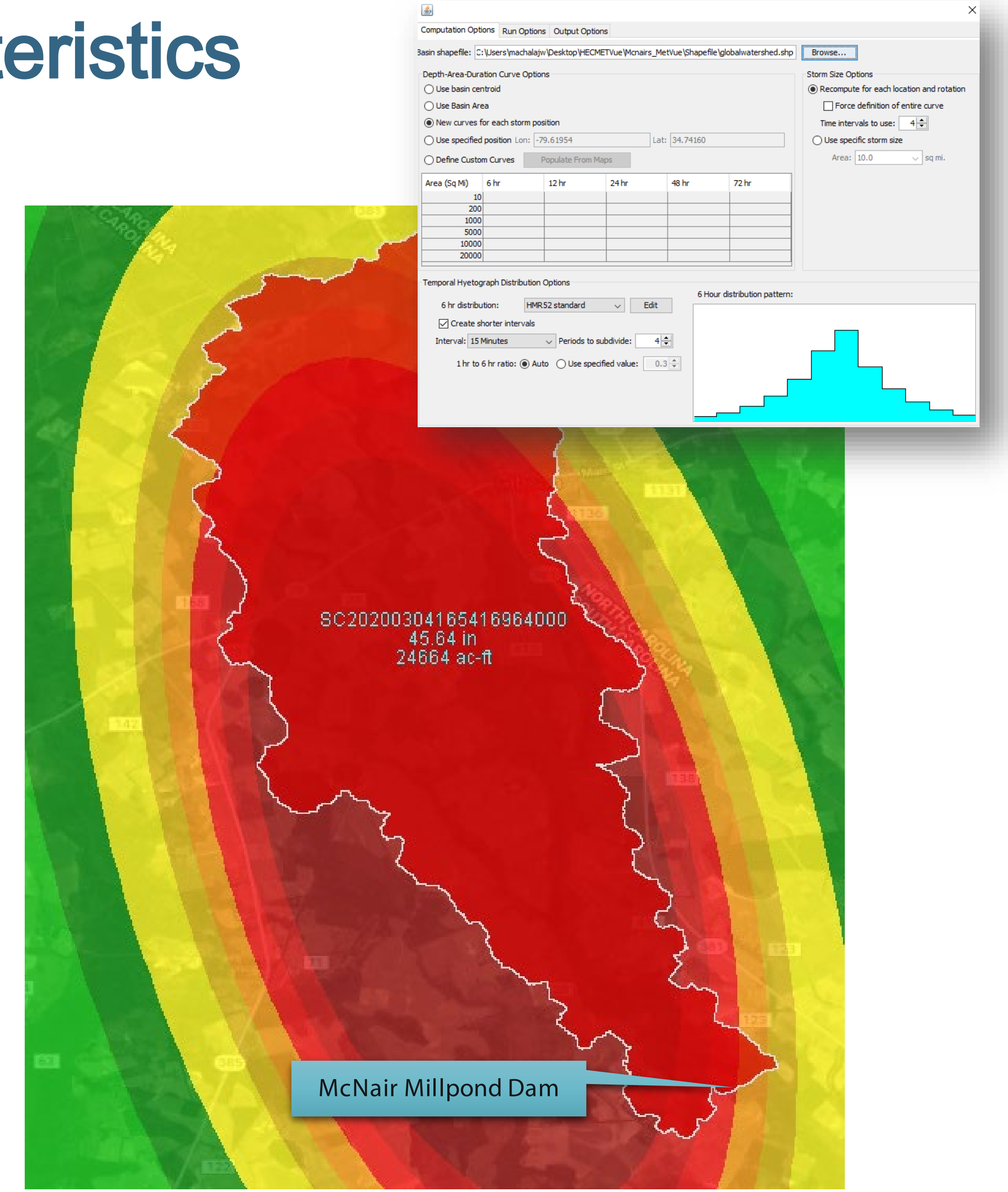
NLCD Impervious Data
An average imperviousness
1.2%



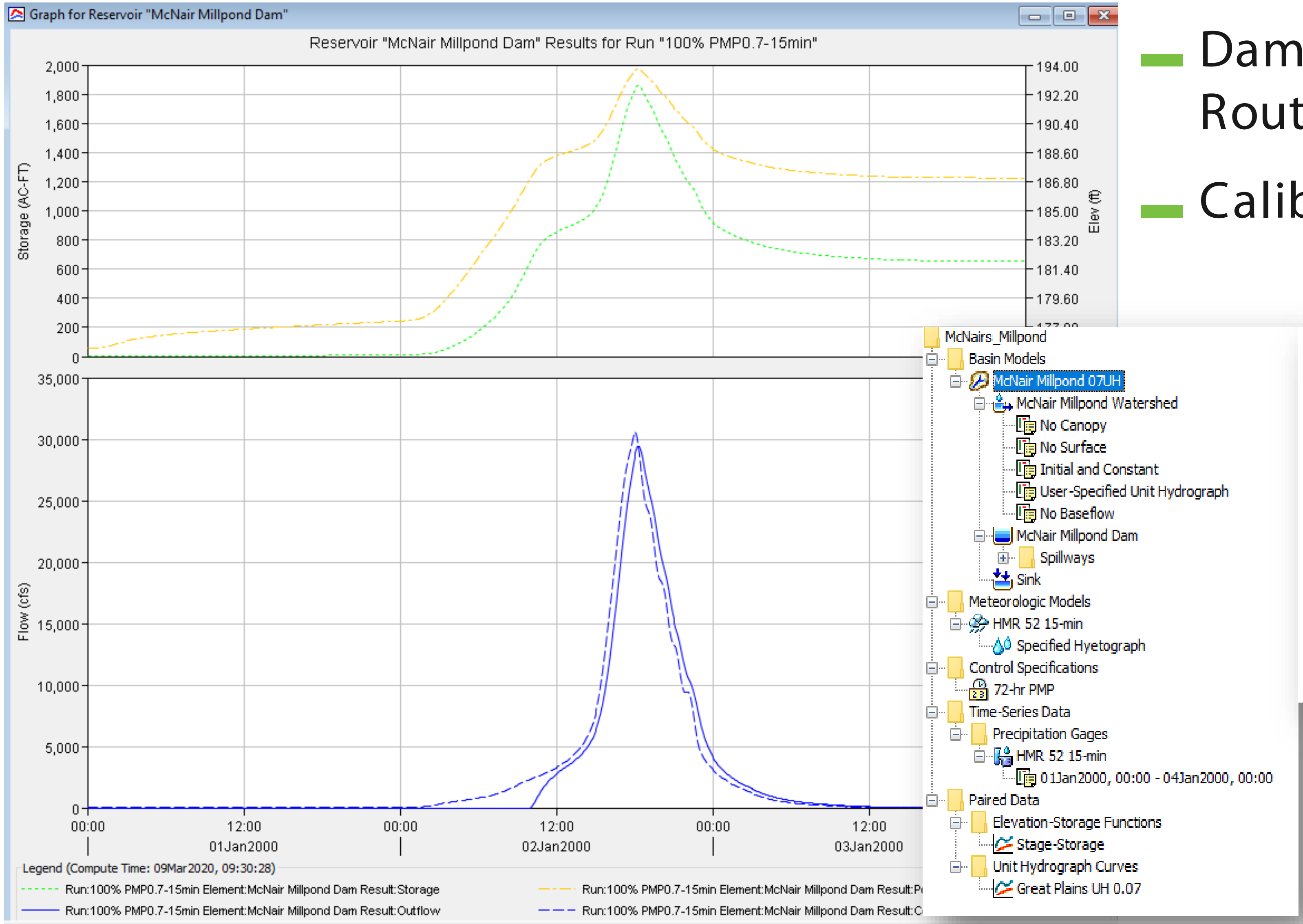
Hydrologic and Watershed Characteristics Case Study- McNair Millpond Dam

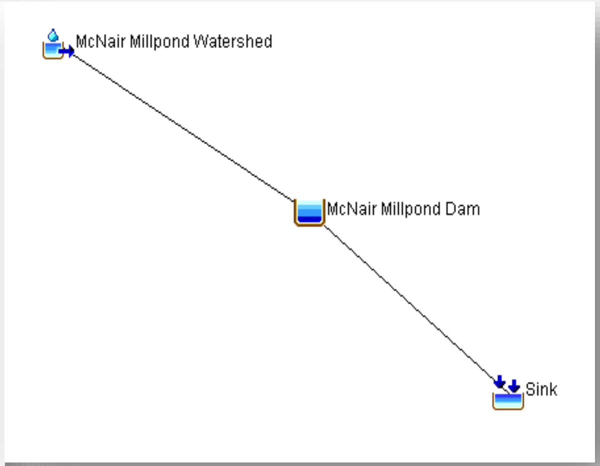
- HEC-MetVue: HMR 52 PMP





Hydrologic and Watershed Characteristics Case Study- McNair Millpond Dam





- Dam Watershed PMP Hydrological Routing Using HEC-HMS Program
- Calibration of Watershed Model

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Dam Hydraulics

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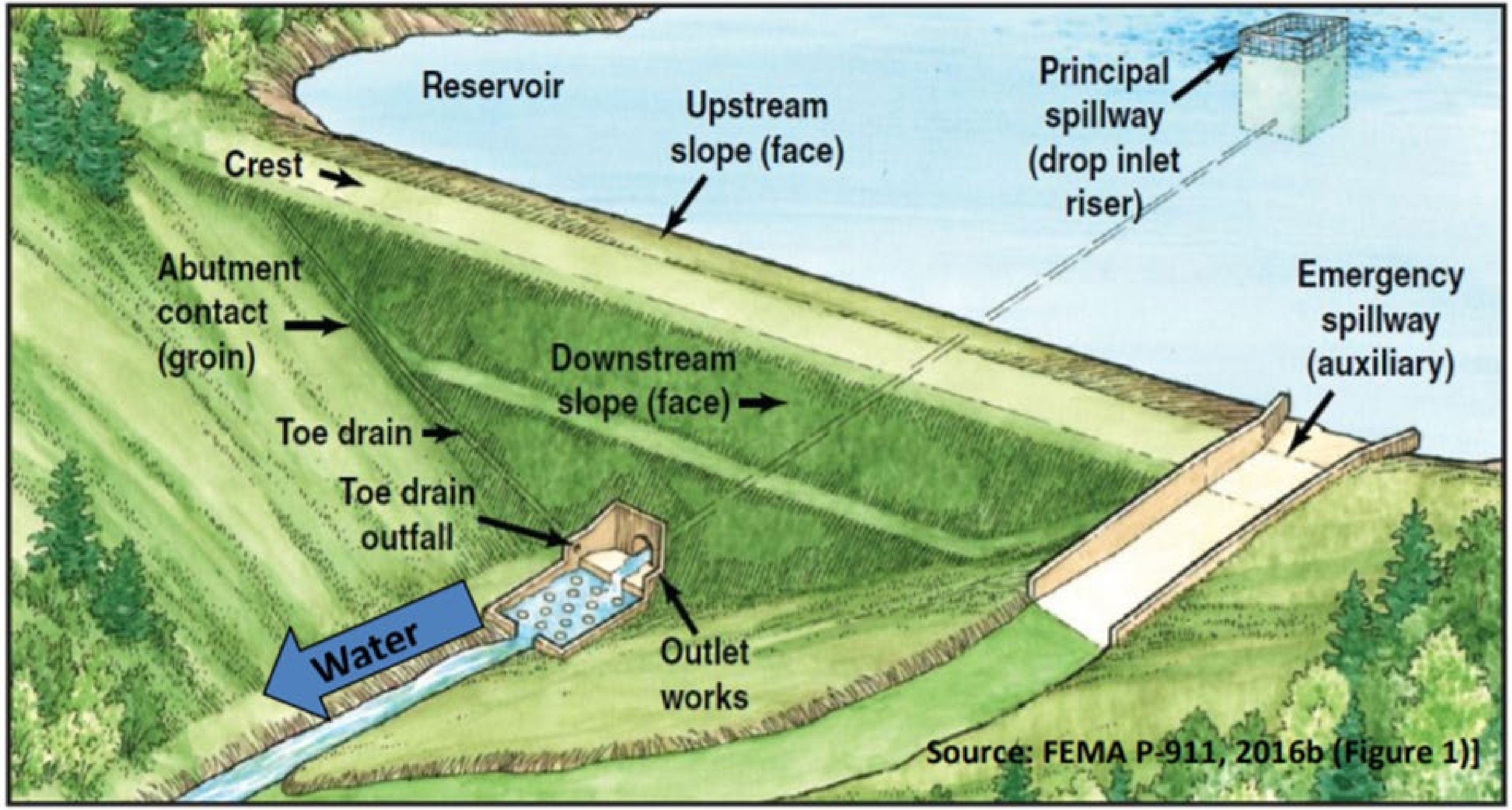
Dam Hydraulics

Dam Components:

Dam structure: The main body of the dam, Earth fill/Rock fill/Concrete (gravity or arch)/composite with core.

Spillway: Principal (drop inlet riser) / Auxiliary (open channel with inlet weir or earth cut open channel).

Reservoir: The stored water behind the dam.



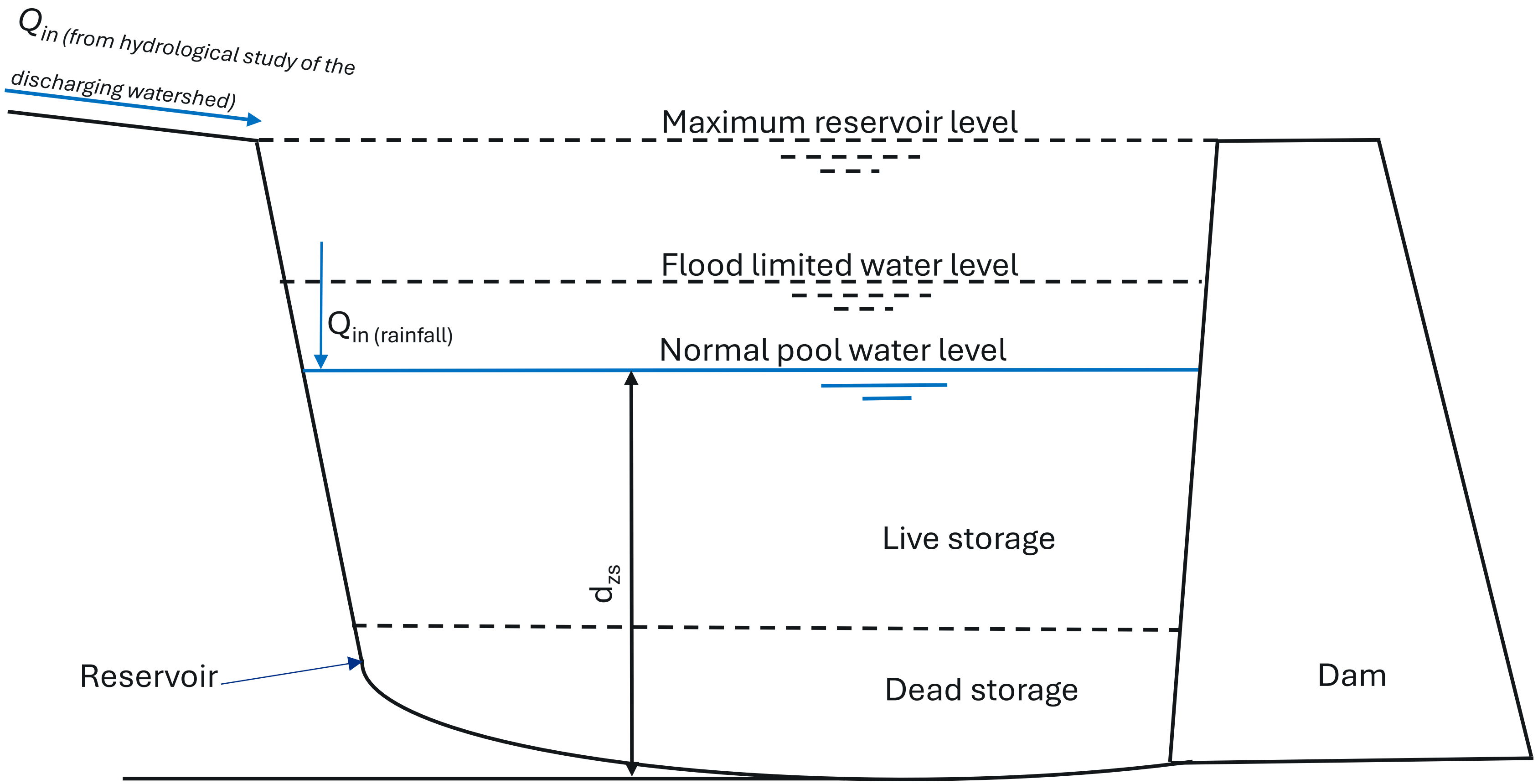
Dam Hydraulics

Mass Balance, Water Budget:

Mainly used to predict changes in Reservoir Water Surface Elevations due to incoming and outgoing flow to and from the reservoir

Incoming discharges to the reservoir

$$Q_{in\,(\text{watershed})} + Q_{in\,(\text{rainfall})}$$

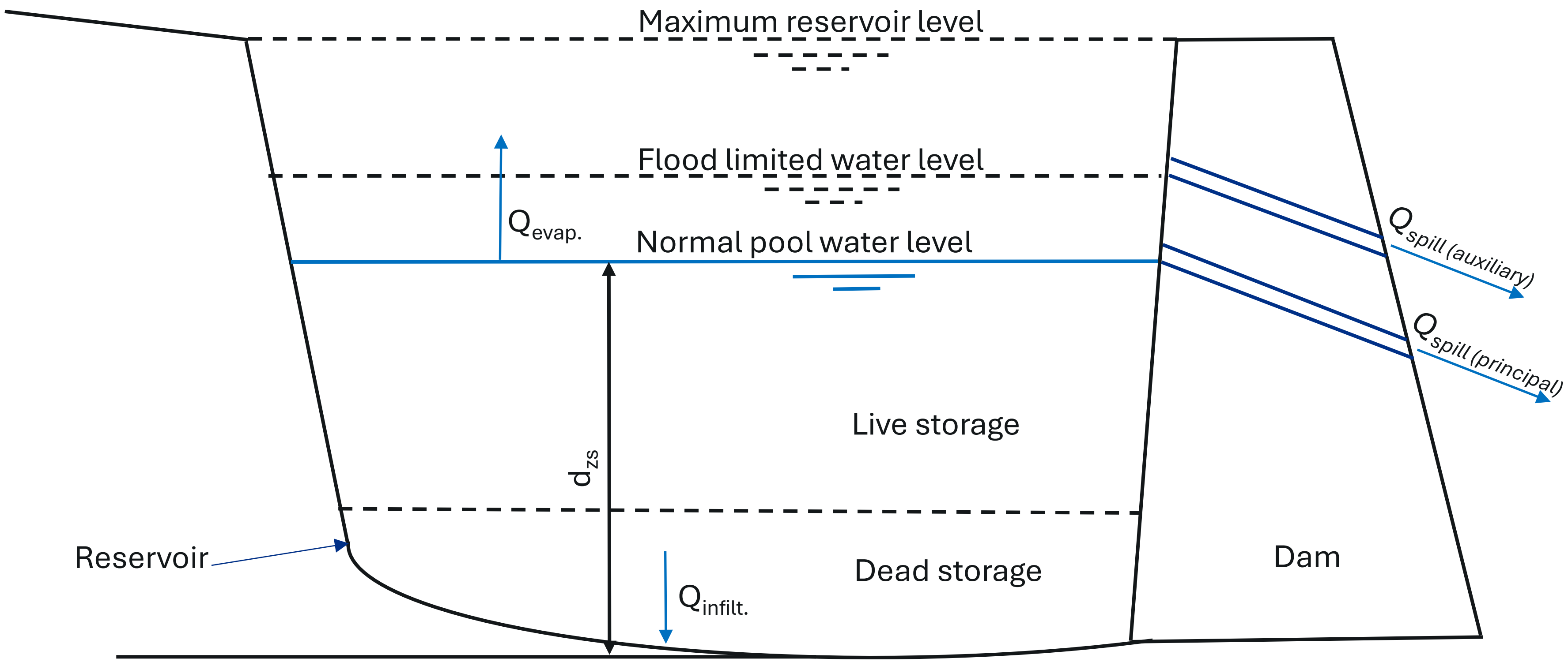


Dam Hydraulics

Mass Balance, Water Budget (continued):

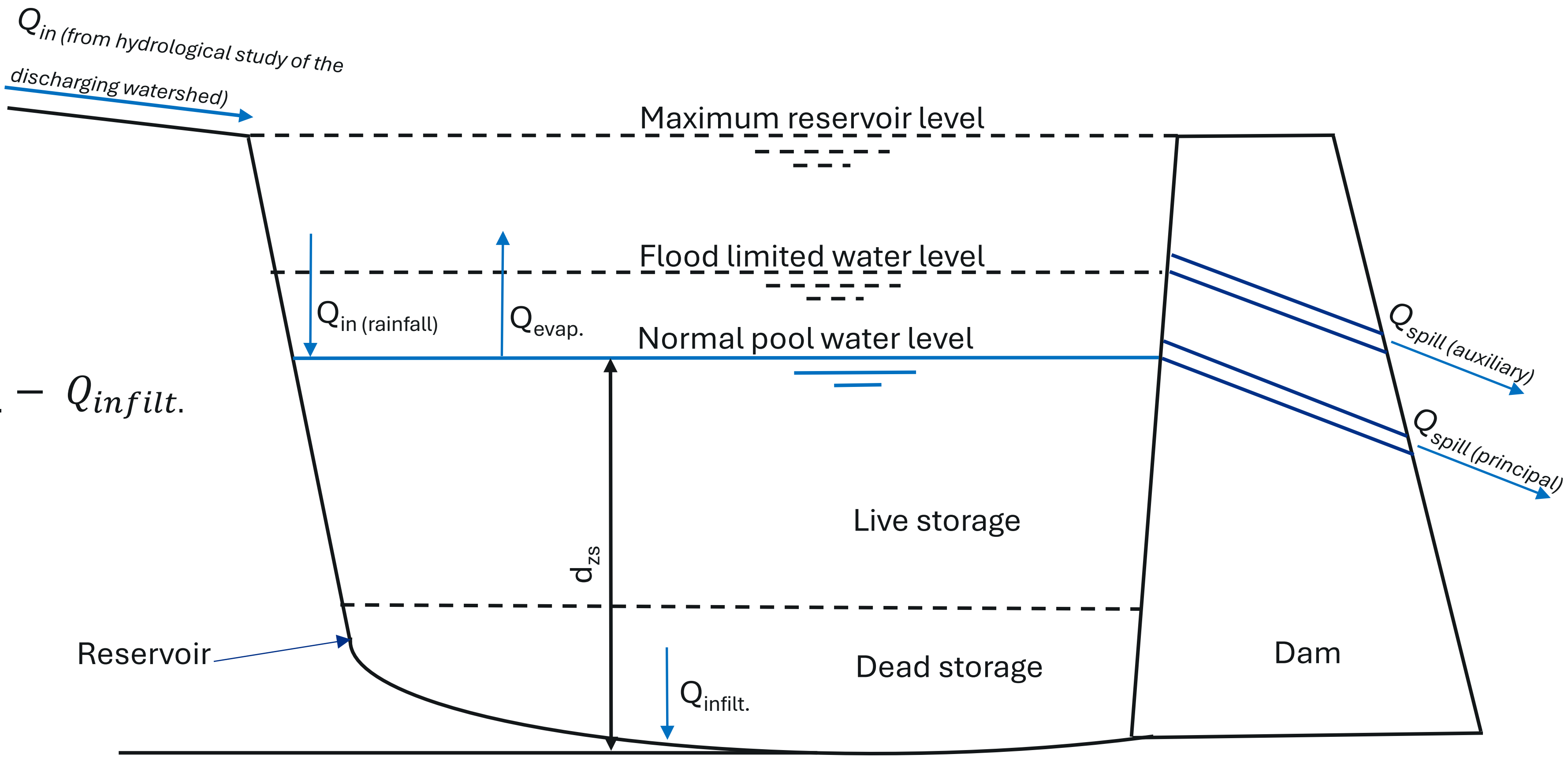
Outgoing discharges from the reservoir

$Q_{spill}\,\text{(principal)} + Q_{spill}\,\text{(auxillary)}$
$+ Q_{evap.} + Q_{infilt.}$



Dam Hydraulics

Mass Balance, Water Budget (continued):



Mass balance equation including all discharges

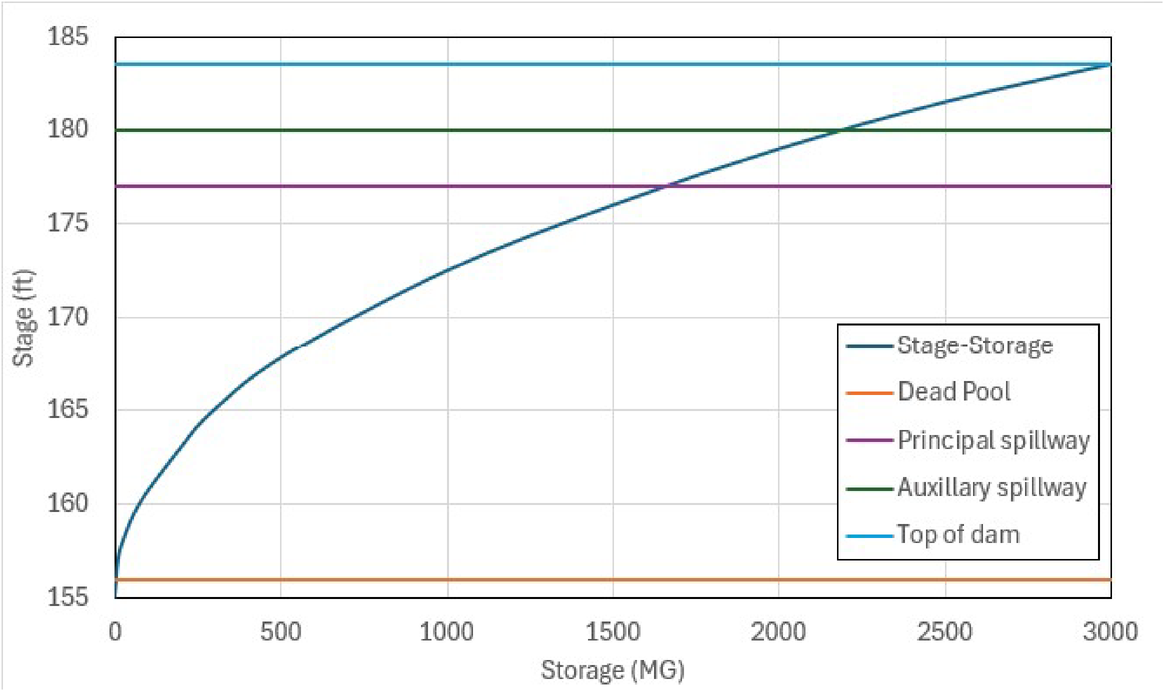
$$A_s \frac{dz_s}{dt} = Q_{in} - Q_{spill} - Q_{evap.} - Q_{infilt.}$$

Where A_s = surface area of reservoir; z_s = water level in the reservoir; t= time; Q_{in} = inflow discharge; Q_{spill} = flow through spillways; $Q_{evap.}$ = flow through evaporation; and $Q_{infilt.}$ = flow through infiltration.

Stage-Storage Relationship

- Critical for hydraulic reservoir routing
- Can use new bathymetric survey for basis
- Using historical information from prior inspection reports

EXAMPLE



Dam Hydraulics

Spillway Capacity:

For the principal spillway (drop inlet riser):

The capacity of the principal spillway for a given reservoir water surface elevation should be determined as **the lowest** of the following four controls:

1- Weir flow at the riser inlet (Weir Control).

2- Orifice flow at the riser inlet (High Orifice Control).

3- Orifice flow at the conduit entrance (Low Orifice Control).

4- Pipe flow through the conduit (Outlet Control).

The equations to solve for each control are as follows:

Dam Hydraulics

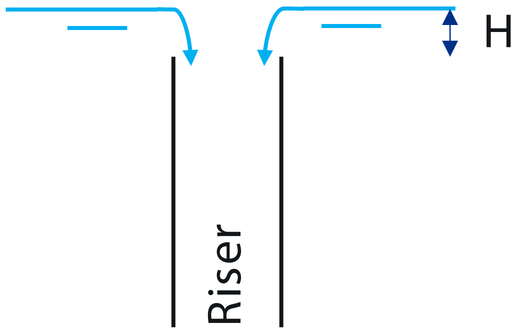
Spillway Capacity (continued):

For the principal spillway (drop inlet riser) (continued):

1- Weir flow at the riser inlet (Weir Control):

$$Q = CLH^{3/2}$$

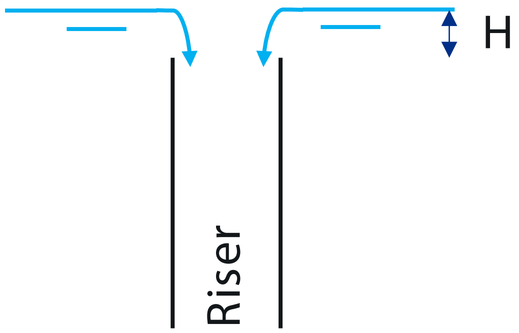
Where,
Q=discharge
C=discharge coefficient
L=weir length
H=head over spillway crest



2- Orifice flow at the riser inlet (High Orifice Control):

$$Q = Ca\sqrt{2gH}$$

Where,
Q=discharge
C=discharge coefficient
a=cross section area of riser
H=head over spillway crest



Dam Hydraulics

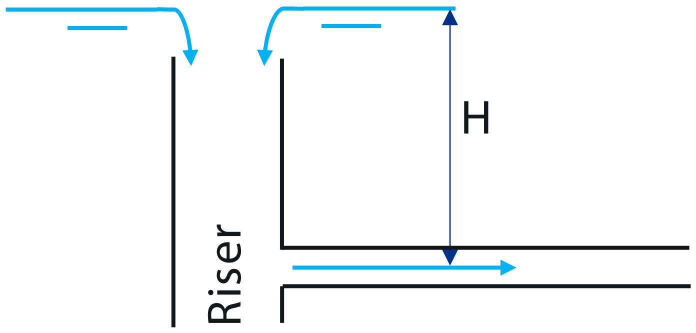
Spillway Capacity (continued):

For the principal spillway (drop inlet riser) (continued):

3- Orifice flow at the conduit entrance (Low Orifice Control):

$$Q = Ca\sqrt{2gH}$$

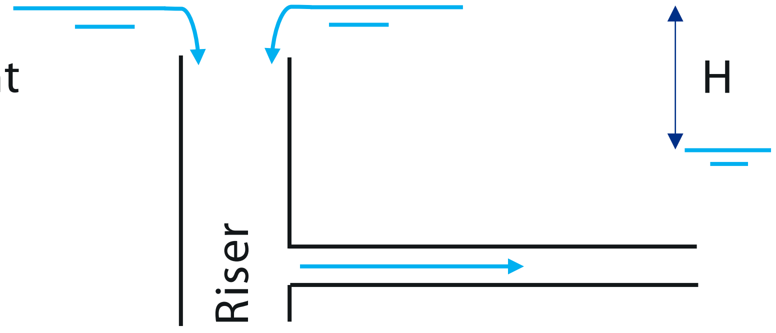
Where,
Q=discharge
C=discharge coefficient
a=cross section area of conduit
H=head over the conduit inlet centerline



4- Pipe flow through the conduit (Outlet Control):

$$Q = \frac{a\sqrt{2gH}}{\sqrt{K_{ent.} + K_{exit} + \frac{FL}{d}}}$$

Where,
Q=discharge
a=cross section area of conduit
H = head difference between reservoir and tailwater
$K_{ent.}$=entrance loss coefficient
K_{exit}=exit loss coefficient
F=conduit friction coefficient
L=conduit length
d=conduit diameter



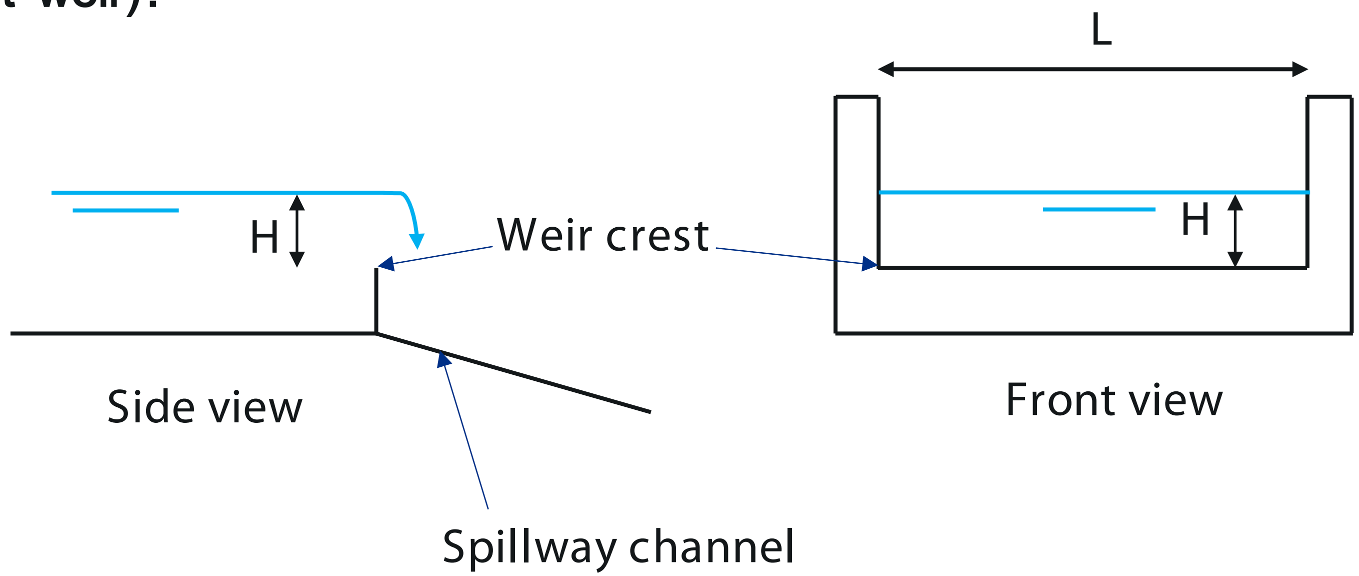
Dam Hydraulics

Spillway Capacity (continued):

For the auxiliary spillway (open channel with inlet weir):

The flow can be calculated using the Weir equation as follows:

$$Q = CLH^{3/2}$$



Where,
Q=discharge
C=discharge coefficient, which depends on the crest shape
L=weir length
H=head over spillway crest

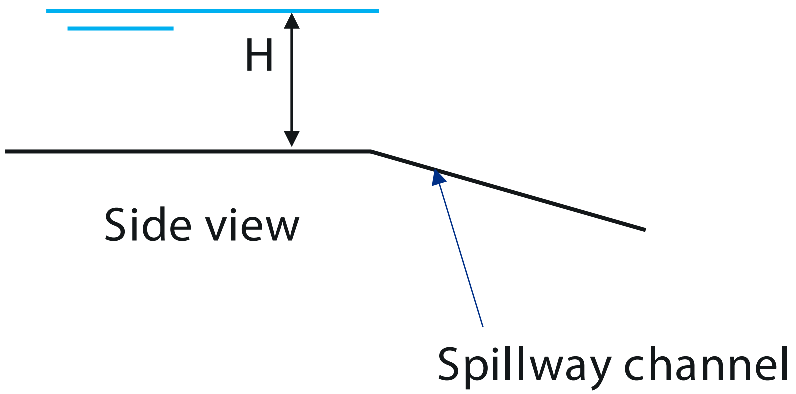
Dam Hydraulics

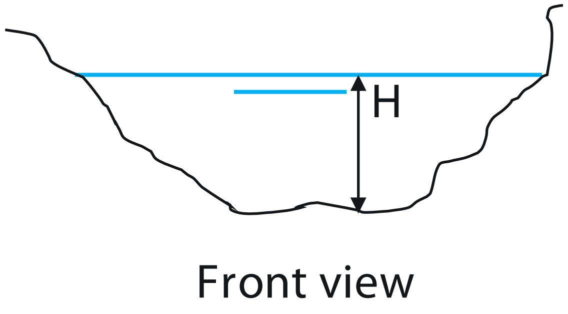
Spillway Capacity (continued):

For the auxiliary spillway (Earth cut open channel):

The flow can be calculated using Manning's equation as follows:

$$Q = \frac{c}{n} R^{2/3} S_o^{1/2} A$$

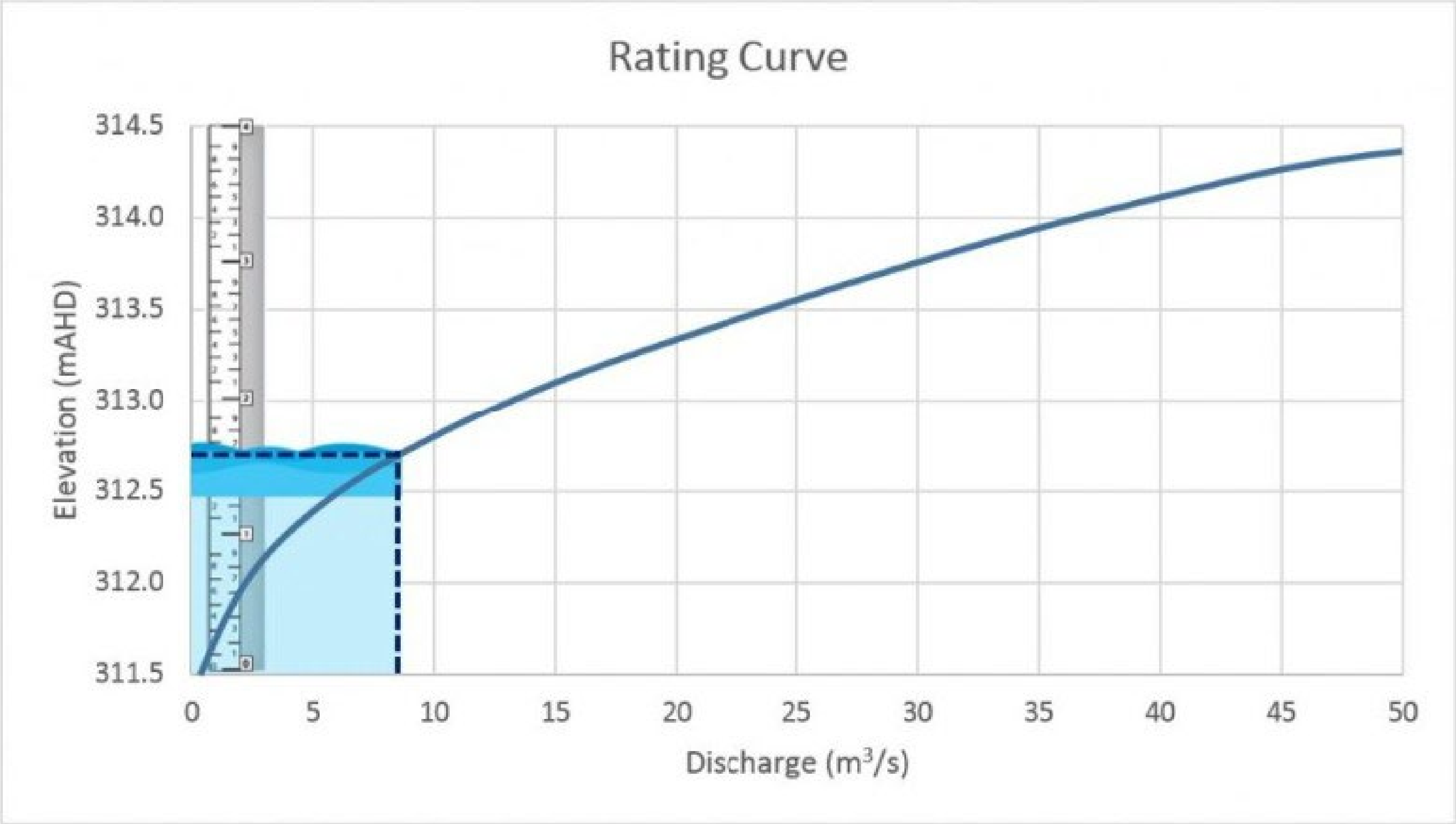




Where,
Q=discharge
c=constant, 1 for SI units and 1.49 US customary
R=hydraulic radius, equal A/P
A=cross-section area
P=Wetted perimeter
S_o=bed slope

Dam Hydraulics

Spillway Rating Curve:



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Dam Breach Analysis

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Dam Breach Analysis

Objective and methods:

To determine the hazard potential classification of a dam by estimating downstream flood inundation maps, Population at Risk (PAR) and prepare Emergency Action Plan (EAP)

This is accomplished by determining breach hydrograph, peak discharge and time to peak using software packages

What are needed to know?

Failure mode:
Overtopping 34%, Foundation defects 30%, Piping 28%
Costa (1985)

Estimating Breach parameters such as: Breach dimensions, breach location, and breach formation time

- Regression equations
  - Historical dam breach cases
- Physically Based breach models
  - Approximation of the hydrodynamic and morphodynamic processes

Driving hydrologic event, probable maximum flood (PMF) during rain event or Sunny day failure at maximum pool elevation

PMF can be calculated by running a hydrological model of a PMP on the contributing watershed to the dam

Dam Breach Analysis

Methods of estimating breach parameters

A- Regression equations:

1- Froehlich (1995a)

Data collected from 63 earthen, zoned earthen, earthen with a core wall (i.e. clay), and rockfill dams

The applicable range:

Dam height: 12 - 305 ft with 90% < 98.43 ft, and 76% < 49.21 ft

Volume of water at breach time: 11 - 535,000 acre-ft with 87% <20,268 acre-ft, and 76% <12,161

$$\boldsymbol{B_{ave}} = 0.1803\, K_0\, V_w^{0.32}\, h_b^{0.19}$$

$$\boldsymbol{t_f} = 0.00254\, V_w^{0.53}\, h_b^{-0.90}$$

Where, B_{ave} is the average breach width in m, K_0 is constant (1.4 for overtopping failures, 1.0 for piping), V_w is reservoir volume at time of failure in m^3, h_b is height of the final breach in m, and t_f is breach formation time in hrs. Froehlich states that the average side slopes should be: 1.4H:1V for Overtopping failures and 0.9H:1V Otherwise (i.e. piping/seepage).

Dam Breach Analysis

Methods of estimating breach parameters (continued)

A- Regression equations (continued):

<u>2- Froehlich (2008)</u>

Data collected from 74 earthen, zoned earthen, earthen with a core wall (i.e. clay), and rockfill dams

The applicable range:

Dam height: 10 - 305 ft with 93%<98.43 ft, and 81%<49.21 ft

Volume of water at breach time: 11.3 - 535,000 acre-ft with 86%<20,268 acre-ft, and 82%<12,161

$$\boldsymbol{B_{ave}} = 0.27\, K_0\, V_w^{0.32}\, h_b^{0.04}$$

$$\boldsymbol{t_f} = 63.2 \sqrt{\frac{V_w}{g\, h_b^2}}$$

Where, B_{ave} is the average breach width in m, K_0 is constant (1.3 for overtopping failures, 1.0 for piping), V_w is reservoir volume at time of failure in m^3, h_b is height of the final breach in m, g is gravitational acceleration (9.81 m/s^2), and t_f is breach formation time in hrs. Froehlich states that the average side slopes should be: 1H:1V Overtopping failures and 0.7H:1V Otherwise (i.e. piping/seepage).

Dam Breach Analysis

Methods of estimating breach parameters (continued)

A- Regression equations (continued):

3- MacDonald and Langridge –Monopolis (MLM) (1984)

Data collected from 42 earthen, earthen with a core wall (i.e. clay), and rockfill dams

The applicable range:

Dam height: 14 - 305 ft with 76% < 98.43 ft, and 57% < 49.21 ft

Volume of water at breach time: 3 - 535,000 acre-ft with 79% <20,268 acre-ft, and 69% <12,161

For earthfill dams:

$$\boldsymbol{V_{eroded}} = 0.0261\ (V_{out}\ x\ h_w)^{0.769}$$

$$\boldsymbol{t_f} = 0.0179\ (V_{eroded})^{0.364}$$

For earthfill with clay core or rockfill dams:

$$\boldsymbol{V_{eroded}} = 0.00348\ (V_{out}\ x\ h_w)^{0.852}$$

Where, V_{eroded} is the volume of material eroded from the dam embankment in m^3, V_{out} is the volume of water that passes through the breach i.e. storage volume at time of breach plus volume of inflow after breach begins, minus any spillway and gate flow after breach begins in m^3 , h_w is depth of water above the bottom of the breach in m, and t_f is breach formation time in hrs.

Dam Breach Analysis

Methods of estimating breach parameters (continued)

A- Regression equations (continued):

3- MacDonald and Langridge –Monopolis (MLM) (1984) (continued)

The base width of the breach can be computed from the dam geometry with the following equation (State of Washington, 1992):

$$\boldsymbol{W_b} = \frac{V_{eroded} - h_b^2\left(CZ_b + \frac{h_b Z_b Z_3}{3}\right)}{h_b\left(C + \frac{h_b Z_3}{2}\right)}$$

Where W_b is bottom width of the breach in m, and h_b is height from the top of the dam to bottom of breach in m, C is crest width of the top of dam in m, $Z_3 = Z_1 + Z_2$, Z_1 is average slope (Z_1: 1) of the upstream face of dam, Z_2 is average slope (Z_2 : 1) of the downstream face of dam, and Z_b is side slopes of the breach (Z_b : 1), 0.5 for the MacDonald method

Dam Breach Analysis

Methods of estimating breach parameters (continued)

A- Regression equations (continued):

<u>4- Von Thun and Gillette (1990)</u>

Data collected from 57 dams from both the Froehlich (1987) and the MLM (1984) papers

The applicable range:

Dam height: 12 - 305 ft with 89% < 98.43 ft, and 75% < 49.21 ft

Volume of water at breach time: 22 - 535,000 acre-ft with 89% <20,268 acre-ft, and 84% <12,161

The method proposes to use breach side slopes of 1.0H:1.0V, except for dams with cohesive soils, where side slopes should be on the order of 0.5H:1V to 0.33H:1V

$$B_{ave} = 2.5\, h_w + C_b$$

Where B_{ave} is average breach width in m, h_w is depth of water above the bottom of the breach in m, and C_b is coefficient, which is a function of reservoir size as follows

Reservoir size, acre-feet	C_b, feet
<1000	20
1000-5000	60
5000-10000	140
>10000	180

Dam Breach Analysis

Methods of estimating breach parameters (continued)

A- Regression equations (continued):

4- Von Thun and Gillette (1990) (continued)

There are two forms of equations according to the inputs as follows

$$\boldsymbol{t_f} = 0.02\, h_w + 0.25\ (erosion\ resistant)$$

$$\boldsymbol{t_f} = 0.015\, h_w (easily\ erodible)$$

Where t_f is the breach formation time in hrs, and h_w is depth of water above the bottom of the breach in m.

$$\boldsymbol{t_f} = \frac{B_{ave}}{4\, h_w}\ (erosion\ resistant)$$

$$\boldsymbol{t_f} = \frac{B_{ave}}{4\, h_w + 61.0}\ (easily\ erodible)$$

Where B_{ave} is the average breach width in m

Dam Breach Analysis

Methods of estimating breach parameters (continued)

A- Regression equations (continued):

<u>5- Xu and Zhang (2009)</u>

Data collected from 75 homogeneous earth fill, zoned-filled, dams with core walls, and concrete faced dams

The applicable range:

Dam height: 10 - 305 ft with 78%<98.43 ft, and 58%<49.21 ft

Volume of water at breach time: 11.3 - 535,000 acre-ft with 80%<20,268 acre-ft, and 67%<12,161

$$\frac{\boldsymbol{B_{ave}}}{\boldsymbol{h_b}} = 0.787 \left(\frac{h_b}{h_r}\right)^{0.133} \left(\frac{V_w^{1/3}}{h_w}\right)^{0.652} e^{B_3}$$

Where B_{ave} is average breach width in m, V_w is reservoir volume at time of failure in m^3, h_b is height of the final breach, h_d is height of dam, h_r is 15 m which is considered to be a reference height for distinguishing large dams from small dams, h_w height of water above the breach bottom elevation at time of breach in m, $B_3 = b_3+b_4+b_5$ coefficient that is a function of dam properties, b_3 is -0.041, 0.026, and -0.226 for dams with corewalls, concrete faced dams, and homogenous/zoned-fill dams, respectively, b_4 is 0.149 and -0.389 for overtopping and seepage/piping, respectively, and b_5 is 0.291,-0.14, and -0.391 for high, medium, and low dam erodibility, respectively.

Dam Breach Analysis

Methods of estimating breach parameters (continued)

A- Regression equations (continued):

5- Xu and Zhang (2009) (continued)

$$\frac{\boldsymbol{B_t}}{\boldsymbol{h_b}} = 1.062 \left(\frac{h_b}{h_r}\right)^{0.092} \left(\frac{V_w^{1/3}}{h_w}\right)^{0.508} e^{B_2}$$

Where B_t is Breach top width, B_2 is b_3+b_4+b_5 coefficient that is a function of dam properties, b_3 is 0.061, 0.088, and -0.089 for dams with corewalls, concrete faced dams, and homogenous/zoned-fill dams, respectively, b_4 is 0.299 and -0.239 for overtopping and seepage/piping, respectively, and b_5 is 0.411,-0.062, and -0.289 for high, medium, and low dam erodibility, respectively.

$$\boldsymbol{Z} = \frac{B_t - B_w}{h_b}$$

Where Z is side slope

$$\frac{\boldsymbol{T_f}}{\boldsymbol{T_r}} = 0.304 \left(\frac{h_d}{h_r}\right)^{0.707} \left(\frac{V_w^{1/3}}{h_w}\right)^{1.228} e^{B_5}$$

Where T_f is Breach Formation time (hrs), T_r is 1 hour (unit duration), V_w is Reservoir volume at time of failure (m^3), h_d is Height of the dam (m), h_r is 15 m, which is considered to be a reference height for distinguishing large from small dams, h_w is Height of the water above the breach bottom elevation at time of breach (m), B_5 is b_3+b_4+b_5 coefficient that is a function of dam properties, b_3 is -0.327, -0.674, and -0.189 for dams with corewalls, concrete faced dams, and homogenous/zoned-fill dams, respectively, b_4 is -0.579 and -0.611 for overtopping and seepage/piping, respectively, and b_5 is -1.205,-0.564, and 0.579 for high, medium, and low dam erodibility, respectively.

Dam Breach Analysis

Methods of estimating breach parameters (continued)

A- Regression equations (continued):

6- Tony L. Wahl (2004) (Uncertainty analysis)

Wahl (2004) quantified the uncertainty in prediction of the breach parameters (B_{avg} , t_f) of compiled historical database of 108 dam failures using number of regression equations and concluded that:

For the average breach width, **Froehlich (1995a)** and **Von Thun and Gillette (1990)** give the least mean prediction error (log cycles) of +0.01 and +0.09 and width of uncertainty band (log cycles) of ±0.39 and ±0.35, respectively.

For the failure time, **Froehlich (1995a)** and **MacDonald and Langridge-Monopolis (1984)** give the least mean prediction error (log cycles) of -0.22 and -0.21 and width of uncertainty band (log cycles) of ±0.64 and ±0.83, respectively.

Dam Breach Analysis

B- Physically based breach models:

Model	Breach cross section	Flow over the breach	Sediment erosion	Geomechanics	Failure mode	Base erosion	Embankment type
NWS BREACH (Fread 1988)	Rectangular and trapezoidal	Weir formula for overtopping and orifice for piping	It uses a noncohesive sediment transport model and does not consider headcut erosion (Meyer-Peter-Mueller modified by Smart)	Breach side slope stability, roof wedge failure during piping	Overtopping, piping	No	Homogeneous/ composite dams
HR BREACH (Mohamed et al. 2002; Morris et al. 2009b)	Effective shear stress dependent	Variable weir plus 1D steady nonuniform equation	It considers headcut and surface erosion modes (Various equations, noncohesive and cohesive soils)	Two- and one-sided lateral erosion; slope stability, core stability, multiple zones of variable erodibility	Overtopping, piping	No	Homogeneous/ composite dams
WinDAM/SIMBA (Temple et al. 2005, 2006; Hanson et al. 2010)	Rectangular and trapezoidal	Weir formula	Headcut advance, bottom and lateral erosion	Breach side slope erosion	Overtopping	No	Homogenous cohesive dams
DLBreach (Wu 2013)	Trapezoidal with variable slope	Weir formula for overtopping and orifice for piping	Nonequilibrium sediment transport, noncohesive and cohesive soils, headcut advance	Two- and one-sided lateral erosion; slope, headcut, clay core, and pipe roof wedge stabilities	Overtopping, piping	Yes	Homogenous/composite dams and levees

Dam Breach Analysis

Common software packages to conduct dam breach analysis:

A. HEGRAS 2D: developed by the U.S. Army Corps of Engineers' Hydrologic Engineering Center (HEC)

Methods of conducting Dam Breach Analysis

1- User Entered Data:

Input data

-Failure location: Centerline stationing of the breach in the dam.

-Failure mode: Overtopping / Pipping.

-Shape: Final breach dimensions.

-Trigger mechanism: pool elevation, pool elevation +duration.

-Discharge coefficient: weir coefficient for overtopping flow and orifice coefficient for piping flow.

-Critical breach development time:

Overtopping: Starts when the erosion reached to the upstream slope and ending when the breach is fully formed.

Piping: Starts when significant flow and material are coming out of the piping failure and ending when the breach is fully formed.

-Breach fraction versus time fraction: relation between fraction of breach completion and fraction of breach time.

Dam Breach Analysis

Common software packages to conduct dam breach analysis (continued):

A. HEGRAS 2D (continued):

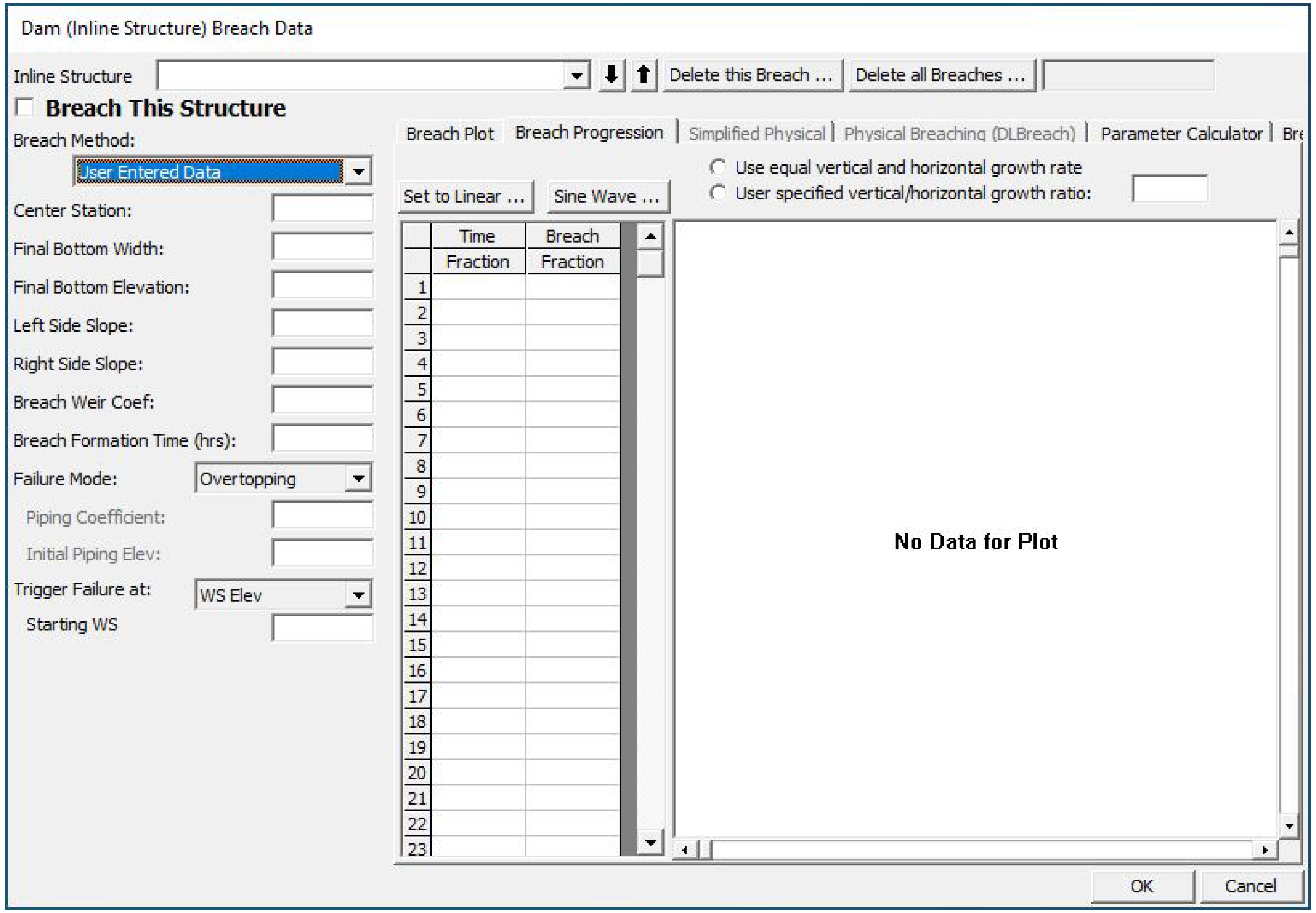
Methods of conducting Dam Breach Analysis (continued)

1- User Entered Data (continued):

Sources to gather input data:

Federal guidelines :
Based on comparative analysis with historical failures of dams of similar size, materials, and water volume.

Regression equations: equations developed from historical dam failures.



Dam Breach Analysis

Common software packages to conduct dam breach analysis (continued):

A. HEGRAS 2D (continued):

Methods of conducting Dam Breach Analysis (continued)

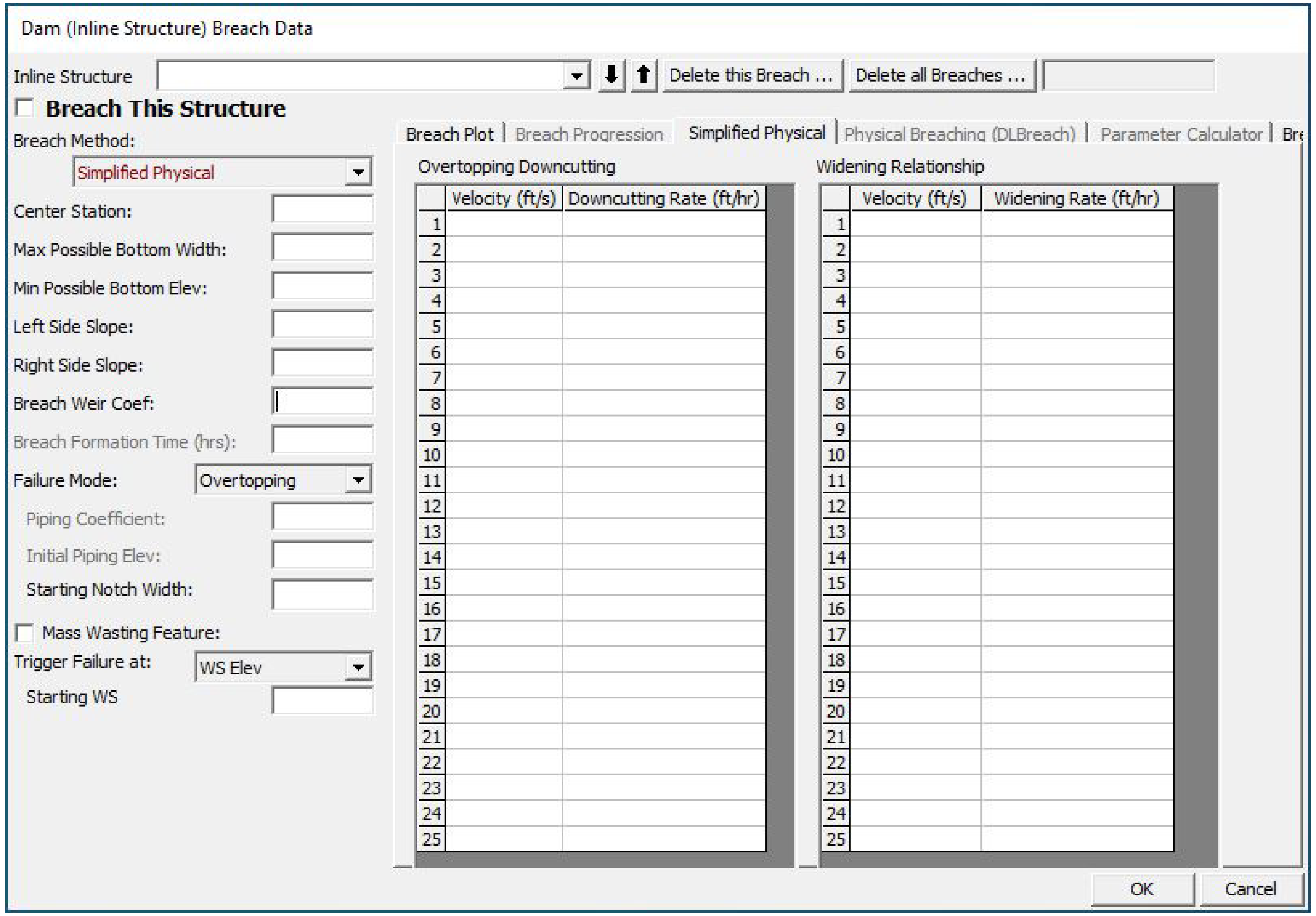
2- Simplified Physically Based:

Input data

-The same input data as User Entered data method except using the maximum expected breach dimensions rather than final breach dimensions.

-Starting notch width.

-Velocity versus downcutting rate and velocity versus widening rate.



Dam Breach Analysis

Common software packages to conduct dam breach analysis (continued):

A. HEGRAS 2D (continued):

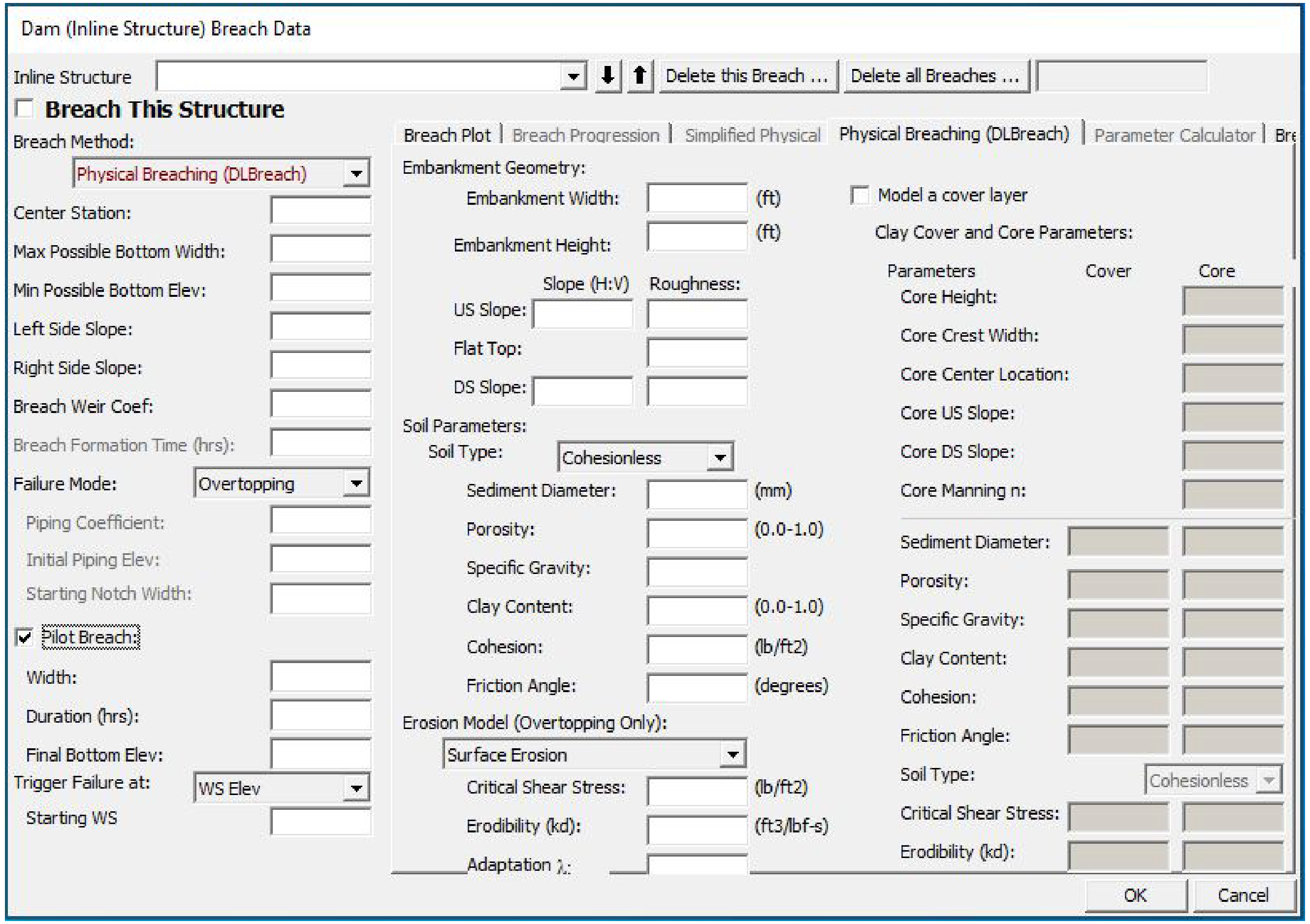
Methods of conducting Dam Breach Analysis (continued)

3- Physical Breaching (DLbreach):

Input data

-The same input data as simplified physically based method plus the capability of entering Pilot Breach.

-The user can choose the soil type, soil properties, erosion model (surface erosion, headcut erosion, and composite structure.



Dam Breach Analysis

Common software packages to conduct dam breach analysis (continued):

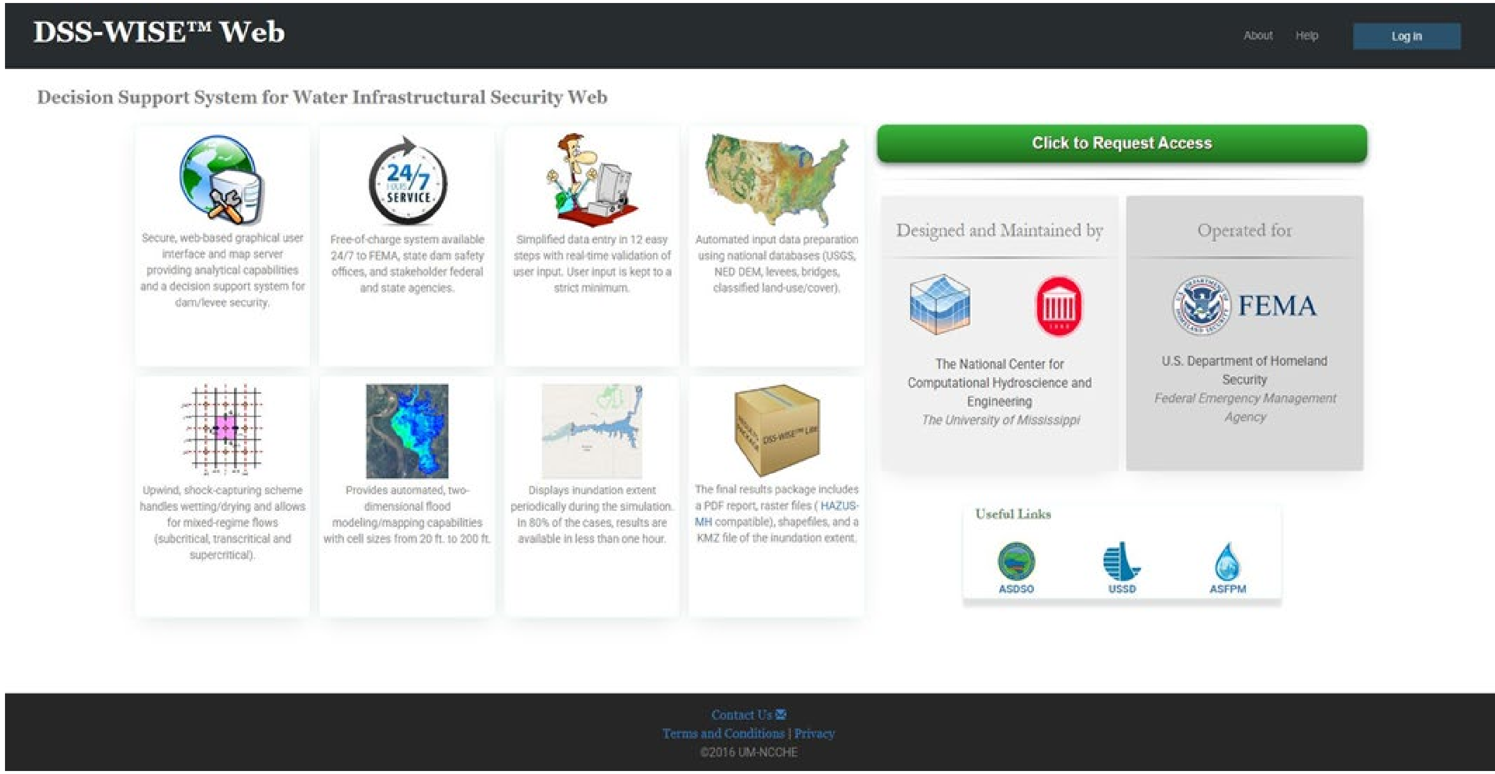
B. DSSWISE Lite:

Background

-Stands for Decision Support System for Water Infrastructure Security, web-based automated two-dimensional dam-break flood modeling and mapping.

-Developed and operated by the University of Mississippi's National Center for Computational Hydroscience and Engineering (NCCHE) with funding provided by the department of Homeland Security (DHS).

-This web tool is primarily used by registered users from FEMA, federal agencies, and state dam safety offices.



-SCDES has used this program to provide screening results for all regulated dams in SC for emergency planning and hazard potential classification.

-SC Dam Safety controls access to DSS-WISE Lite for simulations run in SC. SC Dam Safety will grant access to consulting engineers for a limited time upon request.

Dam Breach Analysis

Common software packages to conduct dam breach analysis (continued):

B. DSSWISE Lite (continued):

Main objective SCDES uses the DSSWISE Lite

Prepare flood inundation map due to individual dam failure due to sudden release of maximum reservoir level on sunny day condition.

Components of DSSWISE Lite

1- A map viewer, which provides access to several analytical capabilities.

2- A status and results page to list and manage the simulations, view and monitor simulation results, and download final results package.

3- A web page for group managers to manage their groups, accept and reject memberships, etc.

4- A web page with manuals, tutorials, webinars, frequently asked questions.

Key benefits of DSSWISE Lite

1- Does not require prior experience in numerical modeling.

2- Simulations can be executed very quickly with minimum user inputs.

3- Input files are automatically prepared using input data and national data sets.

4- Produces geospatial results files compatible with GIS software and HAZUS-MH.

5- Can be used for hazard classification, ranking and prioritizing, EAP preparation and emergency response planning.

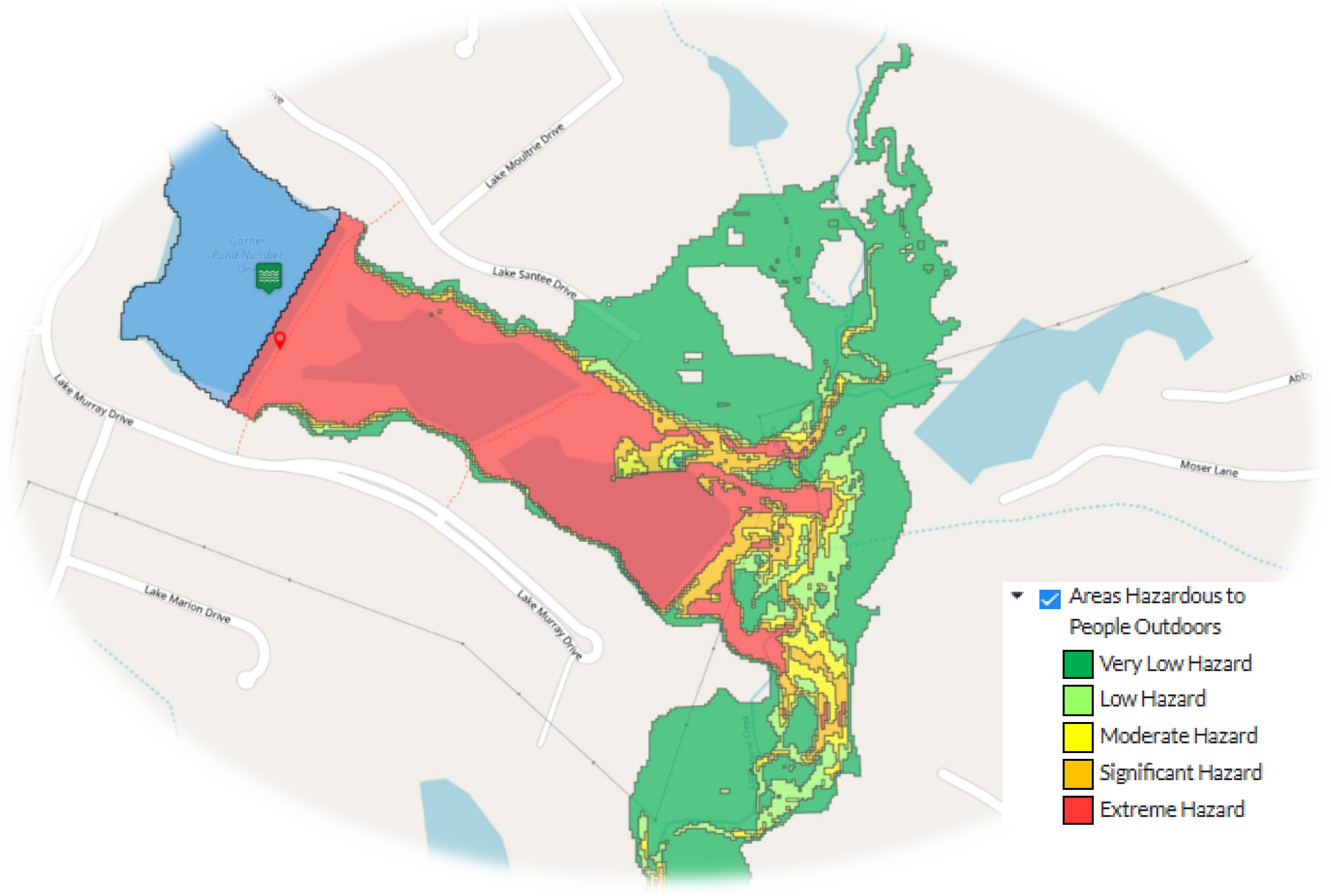
Dam Breach Analysis

Inundation Mapping and Population at Risk (PAR)

-Creating visual representation of areas that are likely to be flooded, showing the extent and depth of potential floodwaters for various scenarios.

-By overlying population data on inundation maps, it is possible to identify specific areas and communities most at risk.

Human Consequences Module (HCOM) can also be added to show PAR. The figure shows one of the simulations of dam breach case in South Carolina using DSS-WISE



Dam Breach Analysis

Emergency Action Plan (EAP)

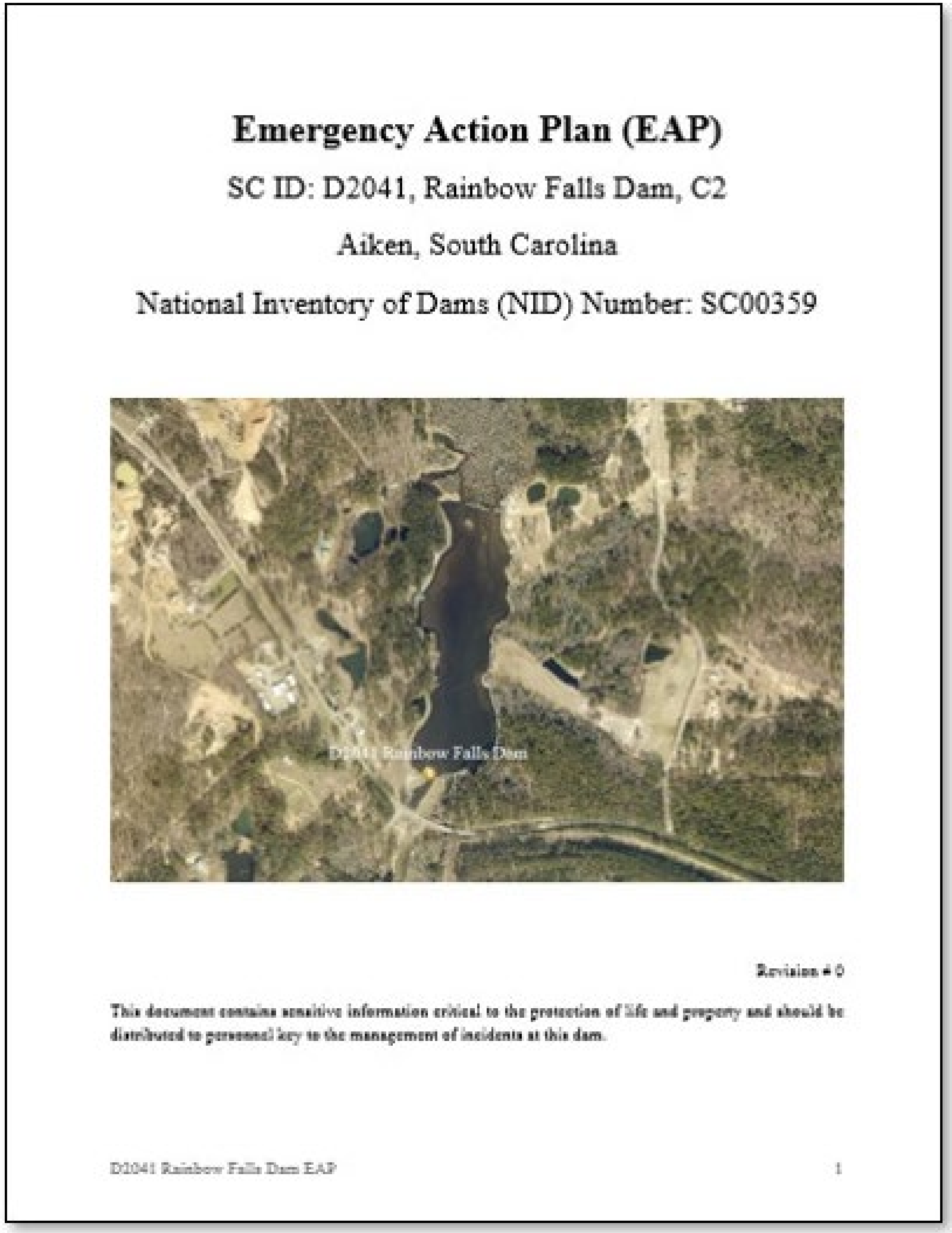
-Is an official document designed to help dam owners recognize unusual or emergency situations at their dam and outline a course of action in the event of such incidents.

-Its main objective is to minimize potential downstream impacts, including loss of life and property damage due to dam failure.

-Mandatory for high, significant hazard dams and encouraged for low hazard dams .

For more details on the process of preparing the EAP, please vis

https://des.sc.gov/programs/bureau-water/dams-reservoirs/emergency-action-plans-dams



Emergency Action Plan (EAP)

SC ID: D2041, Rainbow Falls Dam, C2

Aiken, South Carolina

National Inventory of Dams (NID) Number: SC00359

Revision # 0

This document contains sensitive information critical to the protection of life and property and should be distributed to personnel key to the management of incidents at this dam.

D2041 Rainbow Falls Dam EAP 1

SC DEPARTMENT of ENVIRONMENTAL SERVICES

Incremental Consequence Analysis (ICA)

CDM Smith

Incremental Consequence Analysis (ICA)

Objective

To determine the Spillway Design Flood (SDF). Initially the SDF can be estimated using a criteria presented by The South Carolina Dams and Reservoirs Safety Act Regulation 72-1 through 72-9 using the dam hazard potential and size classification as shown in the following table. The SDF can be reduced to spillway discharge at which dam failure will not significantly increase the downstream hazard which exists just prior to dam failure using ICA following the guidance document DSG-501 developed by The South Carolina Dam and Reservoirs Safety Program.

SPILLWAY DESIGN FLOOD CRITERIA

Hazard	Size	Spillway Design Flood (SDF)*
High	Very Small	100-yr. to ½ PMF
	Small	½ PMF to PMF
	Intermediate	PMF
	Large	PMF
Significant	Small	100-yr. to ½ PMF
	Intermediate	½ PMF to PMF
	Large	PMF
Low	Small	50 to 100-yr. frequency
	Intermediate	100-yr. to ½ PMF
	Large	½ PMF to PMF

*Note: When appropriate, the spillway design flood may be reduced to the spillway discharge at which dam failure will not significantly increase the downstream hazard which exists just prior to dam failure.

Hazard Potential Classification Table

Hazard Classification	Hazard Potential
High Hazard (Class I)	Dams located where failure will likely cause loss of life or serious damage to home(s), industrial and commercial facilities, important public utilities, main highway(s) or railroads.
Significant Hazard (Class II)	Dams located where failure will not likely cause loss of life but may damage home(s), industrial and commercial facilities, secondary highway(s) or railroad(s) or cause interruption of use or service of relatively important public utilities.
Low Hazard (Class III)	Dams located where failure may cause minimal property damage to others. Loss of life is not expected.

Size Classification Table

Category	Impoundment Storage (Acre Feet)		Height (Feet)
Very Small	<50	and	<25
Small	≥50 and <1000	or	≥25 and <40
Intermediate	≥1000 and <50,000	or	≥40 and <100
Large	≥50,000	or	≥100

Incremental Consequence Analysis (ICA)

Methods of Analysis (Dam Safety Guidance-DSG 500 & 501)

1- Prepare flood inundation maps for an initial flood level following the criteria in previous table with and without dam-break.

2- Compare flood depths and velocities between the case with and without dam-break at selected critical locations downstream of the dam.

3- Use the USRB ACER-11 charts to determine the flood risk and assess if there is a zone shift between the case with and without the dam-break.

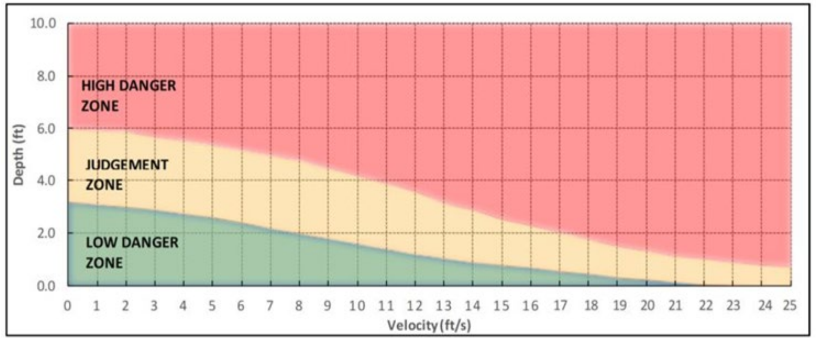
4- If the results of the case with and without dam-break has no zone shift or they are within 10% of the boundary curve level, the design flood level should be reduced and the steps 1 through 3 will be repeated.

5- Once the flood level that cause a zone shift is determined, the design flood level should be the next higher flood level which does not cause a zone shift.

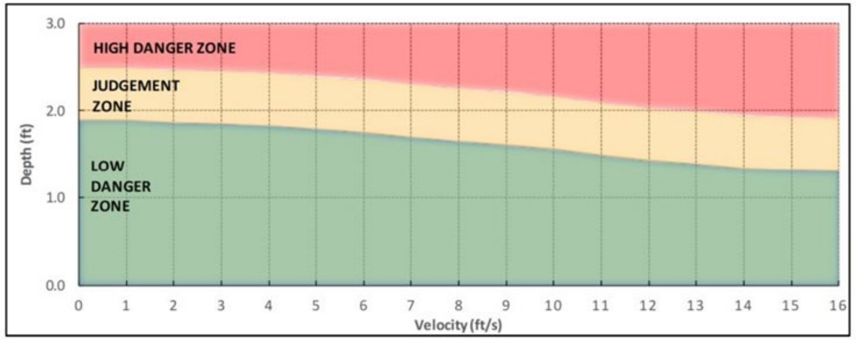
6- It should be noted that, the established design flood level should not be lower than the limits defined in FEMA 94, or no less than 500-yr flood for high hazard potential dams, and 100-yr flood for significant hazard potential dams.

Incremental Consequence Analysis (ICA)

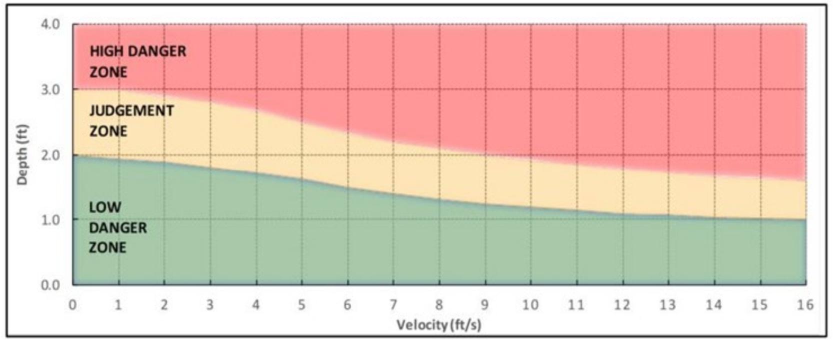
USBR ACER 11 charts



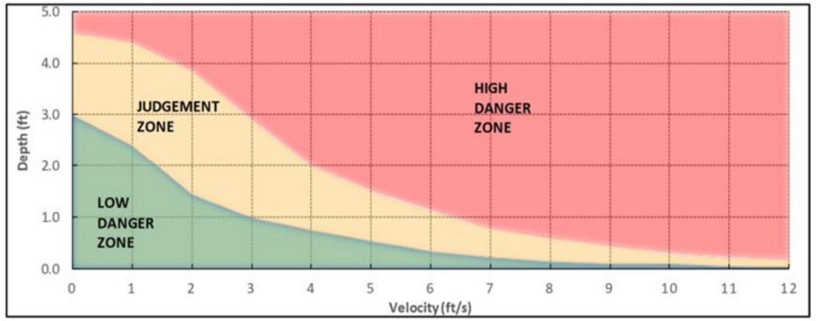
Depth-Velocity Flood Danger Level Relationship for Houses Build on Foundation



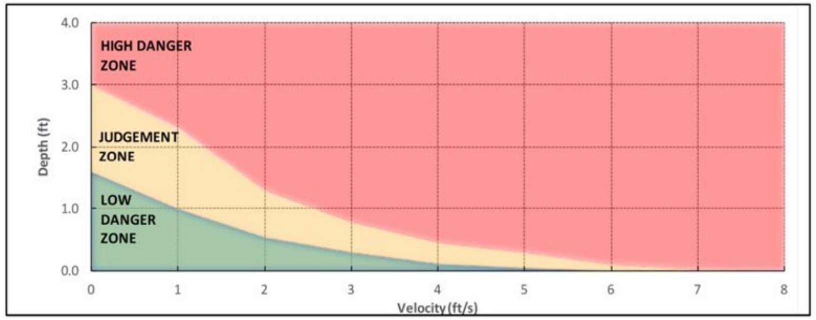
Depth-Velocity Flood Danger Level Relationship for Mobile Homes



Depth-Velocity Flood Danger Level Relationship for Passenger Vehicles (Roads)



Depth-Velocity Flood Danger Level Relationship for Adults



Depth-Velocity Flood Danger Level Relationship for children

Incremental Consequence Analysis (ICA)

Probable Maximum Flood (PMF)

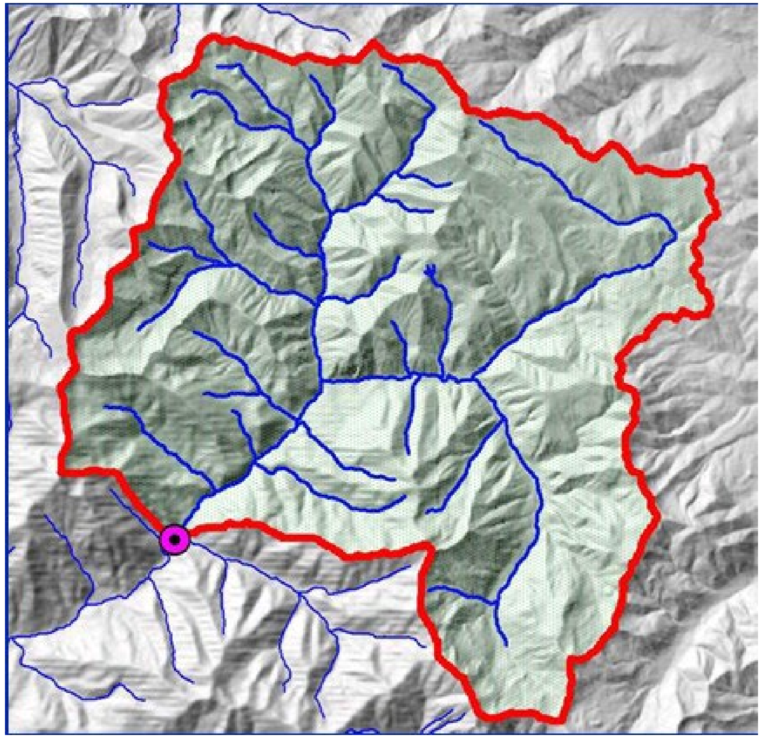
Objective

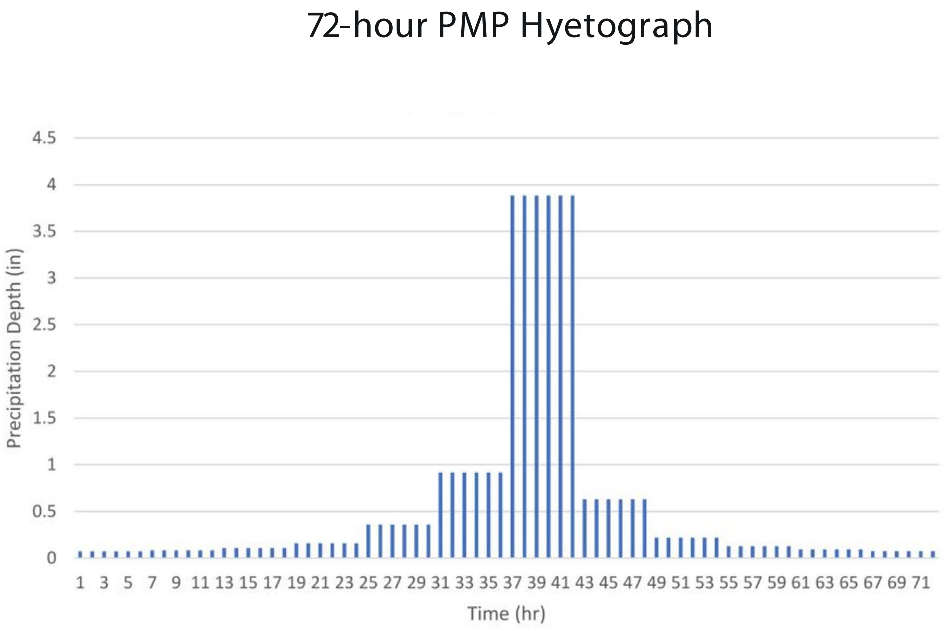
Determine probable maximum flood (PMF) inflow discharge to the dam using probable maximum precipitation (PMP).

Steps

1. Delineate the contributing watershed to the dam using ArcGIS.

2. Calculate PMP storm depths for 72 hrs. at 1hr. interval by using HMR52 tool within HEC-MetVue 3.0 using the watershed shapefile.





Incremental Consequence Analysis (ICA)

Example of determining the SDF

Intermediate size dam, initially classified as high hazard potential.

According to the spillway design criteria in (The South Carolina Dams and Reservoirs Safety Act Regulation 72-1 through 72-9), the SDF should be PMF.

The governing case for the breach conditions was justified as the road downstream of the dam (vehicle case).

Incremental consequence analysis is conducted for dam-break and no dam-break cases, results are listed in the following table and table.

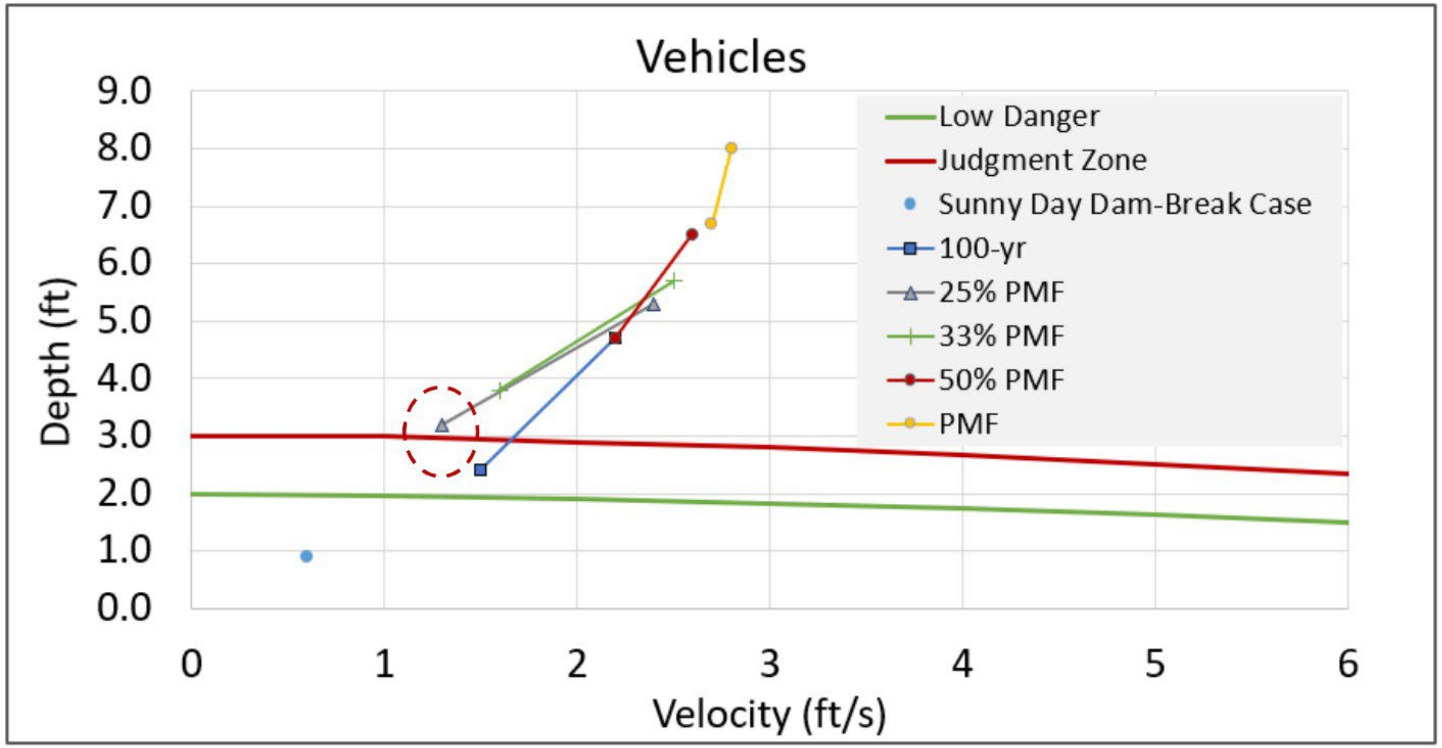
Storm Event	Overtopping Depth (ft)		Overtopping Velocity (ft/s)		Flood Danger Level USBR ACER	
	Flooding without Breach	Flooding with Breach	Flooding without Breach	Flooding with Breach	Flooding without Breach	Flooding with Breach
Sunny Day	--	0.9	--	0.6	--	Low
100-YR	2.4	4.7	1.5	2.2	Judgement	High
25%PMF	3.2	5.3	1.3	2.4	*High	High
33%PMF	3.8	5.7	1.6	2.5	High	High
50%PMF	4.7	6.5	2.2	2.6	High	High
PMF	6.7	8.0	2.7	2.8	High	High

*25% PMF value is very close to the boundary between the judgement and high danger zones

Incremental Consequence Analysis (ICA)

Example of determining the SDF (continued)

Initially, it seems that the 100-yr flood is the lowest flood that causes the zone shift from judgement zone to high hazard zone but after a closer look, the case of no dam-break 25% PMF lies close to the border between the judgement and the high hazard zone so to be conservative this case should belong to the judgement zone, so the SDF for this dam should be 33% PMF which is the following higher flood that does not cause a zone shift.



After incremental hazard assessment and based on DSG 500, the dam is re-classified as low hazard potential.

SC DEPARTMENT of ENVIRONMENTAL SERVICES

Detailed Example of Calculating SDF using ICA

CDM Smith

Detailed Example of Calculating SDF using ICA

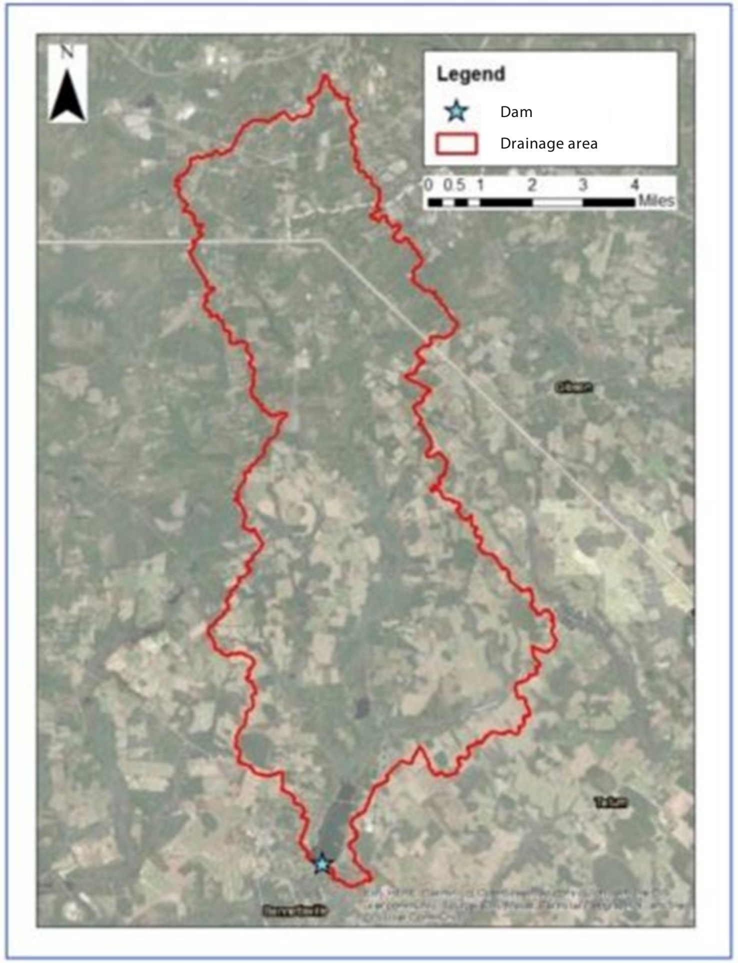
Probable Maximum Flood (PMF)

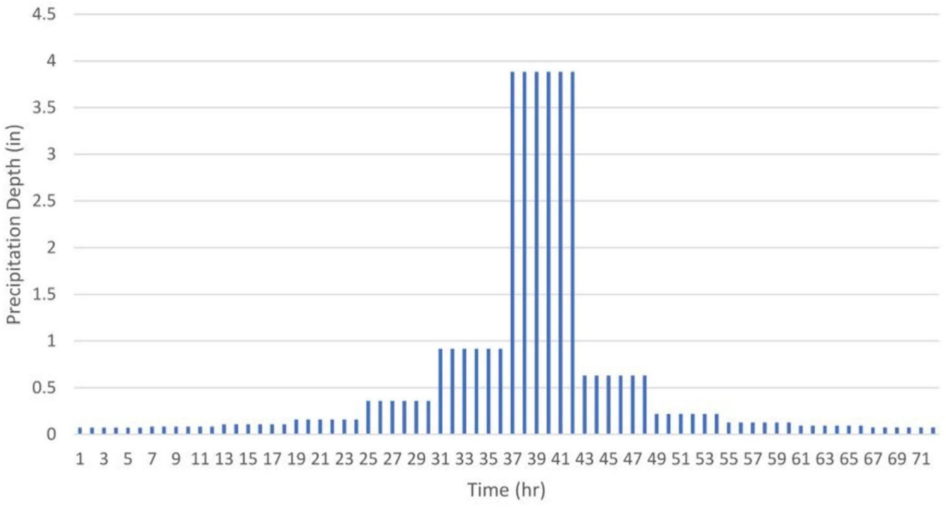
Objective: Determine probable maximum flood (PMF) inflow discharge to the dam using probable maximum precipitation (PMP).

Steps

1. Delineate the contributing watershed to the dam using ArcGIS.

2. Calculate PMP storm depths for 72 hrs. at 1hr. interval by using HMR52 tool within HEC-MetVue 3.0 using the watershed shapefile.

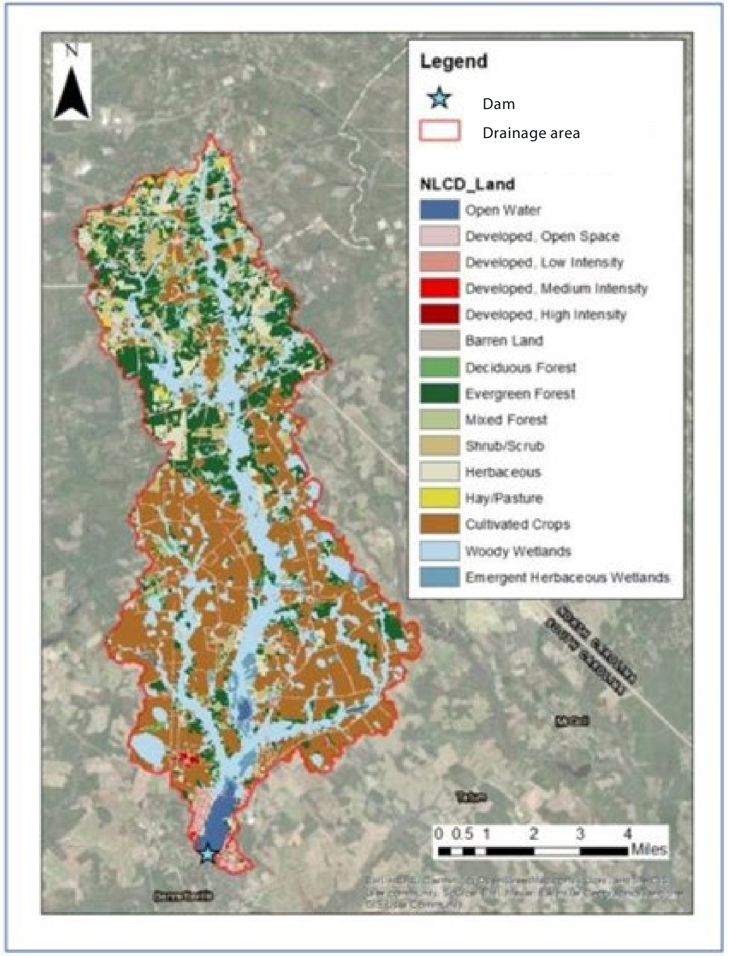




Detailed Example of Calculating SDF using ICA

Probable Maximum Flood (PMF) (continued)

3. Calculate the weighted curve number for the watershed using the national land cover database (NLCD) and soil data from the USDA Natural Resources Conservation Service. Web Soil Survey.



Land Use Grid Code	Land Cover Description	Hydrologic Soil Group				Reference (NEH Part 630 Chapter 9)
		A	B	C	D	
11	Open Water	99	99	99	99	
12	Perennial Ice/Snow	99	99	99	99	
21	Developed, Open Space	49	69	79	84	Table 9-5 Open space, Fair condition
22	Developed, Low Intensity	61	75	83	87	Table 9-5 Residential, ¼ acre
23	Developed, Medium Intensity	81	88	91	93	Table 9-5 Urban Districts, Industrial
24	Developed, High Intensity	89	92	94	95	Table 9-5 Urban Districts, Commercial
31	Barren Land	49	69	79	84	Table 9-1 Pasture..., Fair
41	Deciduous Forest	30	48	57	63	Table 9-2 Oak-aspen..., Fair
42	Evergreen Forest	35	58	73	80	Table 9-2 Pinyon-Juniper..., Fair
43	Mixed Forest	36	60	73	79	Table 9-1 Woods..., Fair
52	Shrub Scrub	35	56	70	77	Table 9-1 Brush..., Fair
71	Herbaceous	55	71	81	89	Table 9-2 Herbaceous, Fair
81	Hay Pasture	49	69	79	84	Table 9-1 Pasture..., Fair
82	Cultivated Crops	63	73	80	83	Table 9-1 Close-seeded..., C&T, Poor
90	Woody Wetlands	72	80	87	93	Table 9-2 Herbaceous, Poor
95	Emergent Herbaceous Wetlands	72	80	87	93	Table 9-2 Herbaceous, Poor

Detailed Example of Calculating SDF using ICA

Probable Maximum Flood (PMF) (continued)

4. Calculate the average basin roughness coefficient (K_n) by comparing the landcover type in the watershed with a similar coastal region from the historic data for the *Design of small Dams* (1987).

5. Calculate the lag time in hours using the basin parameters using the following equation

$$L_g = 26\, K_n \left(\frac{L\, L_{ca}}{S^{0.5}} \right)^{0.33}$$

Where

L_g = lag time, in hours
K_n = average basin roughness coefficient
L = distance of longest watercourse, in miles
L_{ca} = centroidal longest watercourse, in miles
L_g = longest watercourse slope, in feet per mile

Detailed Example of Calculating SDF using ICA

Probable Maximum Flood (PMF) (continued)

6. Calculate the elevation-storage curve and the elevation-discharge curve for the reservoir element to be utilized in the HEC-HMS model.

7. Calibrate the HEC-HMS model by adjusting the basin roughness coefficient K_n and the peaking coefficient of the Snyder Unit Hydrograph transform inputs before running the 72-hr. PMP storm. The calibration is conducted using NOAA Atlas 14 depth-duration precipitation for the 100 and 500-year storms. The resulted peak discharge was compared to the peak discharge calculated using the StreamStats.

Elevation (ft, NAVD88)	Storage (acre-ft)
130.0	0
131.0	53
132.0	125
133.0	197
134.0	269
135.0	342
136.0	414
137.0	487
138.0	560
139.0	633
140.0	706
141.0	780
141.9	943
142.0	961
143.0	1,406
144.0	1,931
144.8	2,423
145.0	2,546
146.0	3,220
147.0	3,949
148.0	4,736
149.0	5,627
150.0	12,000

Elevation (ft, NAVD88)	Discharge (cfs)
141.9	0
142.4	400
142.8	600
143.4	1,200
144.4	2,400
145.4	4,000
145.9	5,000
146.02	6,000
146.22	7,020
146.4	8,000
147.0	10,000
147.08	12,000
147.4	14,000
147.8	18,000
147.81	20,000
147.98	21,000
148.0	22,000
148.06	23,000
148.2	24,000
148.4	28,000
148.8	36,000
153.8	150,000

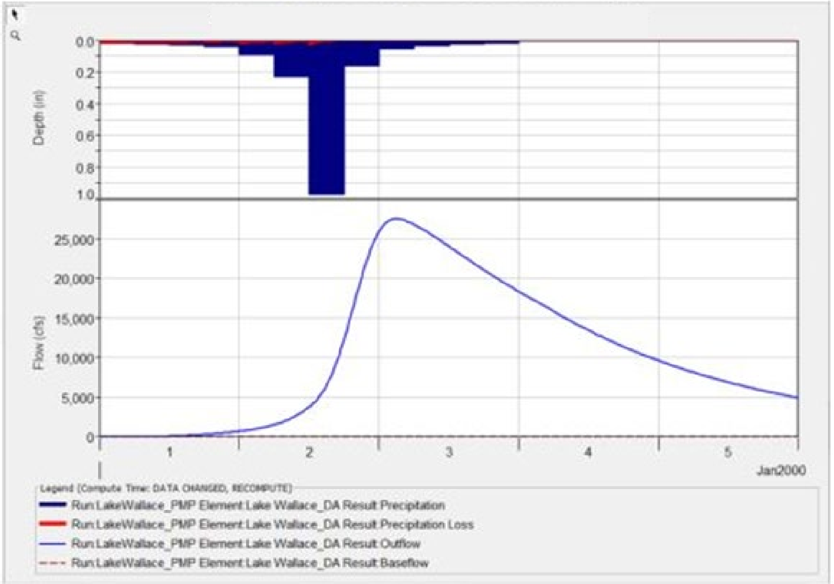
Detailed Example of Calculating SDF using ICA

Probable Maximum Flood (PMF) (continued)

8. The following figure shows the summary of the basin input parameters in the HEC-HMS.

Basin Parameter	Lake Wallace Basin
Basin Area (sq.mi.)	56.4
Longest Watercourse – L (mi.)	21.24
Centroidal Longest Flow Path – L_{ca} (mi.)	9.88
Overall Slope of Longest Watercourse (ft/mi)	12.0
Basin Factor (LL_{ca}/S^0.5)	60.6
Area Weighted Curve Number (ARC II)	69
Area Weighted Curve Number (ARC III)	83
Average basin roughness coefficient, K_n	0.104
Lag Time (hr)	10.5
Snyder Unit Hydrograph Peaking Coefficient	0.25

9. Run the HEC-HMS model using the PMP which produced the PMF as follows.



Detailed Example of Calculating SDF using ICA

Incremental Consequence Analysis (ICA)

Objective

Determine if the spillway design flood (SDF) can be reduced from the probable maximum flood (PMF)

Hydraulic Modeling for Incremental Consequence

Suppose a dam is classified as Intermediate, High Hazard (Class I) according to The South Carolina Dams and Reservoirs Safety Act Regulation 72-1 through 72-9. According to the regulation, the SDF for this dam is PMF, however this SDF can be reduced if the flood depth and velocity at the dam downstream area will not significantly increase if the dam is removed.

Approach

1- Dam failure during Sunny day with normal pool level. The flood danger is assessed according to ACER-11 at critical locations downstream of the dam such as residences, roads, and habitable buildings.

2- Dam failure during PMF with the maximum pool elevation, flood danger is assessed according to ACER-11 at the same locations as previous scenarios.

3- Passing of the PMF with the maximum pool elevation but without dam failure then compare the flood danger of this simulation with the previous simulation.

Detailed Example of Calculating SDF using ICA

Incremental Consequence Analysis (ICA) (continued)

4- If the flood danger at critical locations does not shift zones on ACER11charts when simulating the PMF with and without dam failure, the SDF can be reduced as fraction of PMF and rerun the hydraulic model with and without dam failure and reassessthe flood danger at the critical locations. This process is repeated until a zone shift is noticeable between the case with and without the dam failure. Once the lowest flood level that causes a shift is identified, the design flood level should be identified as the next higher analyzed flood level the does not cause a shift.

Details of the Hydraulic Model

The existing HEC-RAS hydraulic model provided by USACE is used as the base model, however the Storage/2D Connectors surrounding the dam and impoundment are adjusted using rating curves of the outlet works developed by separate hydraulic modeling of the spillway and culverts, rather than having the model complete the calculations. The lake dimensions are as follows

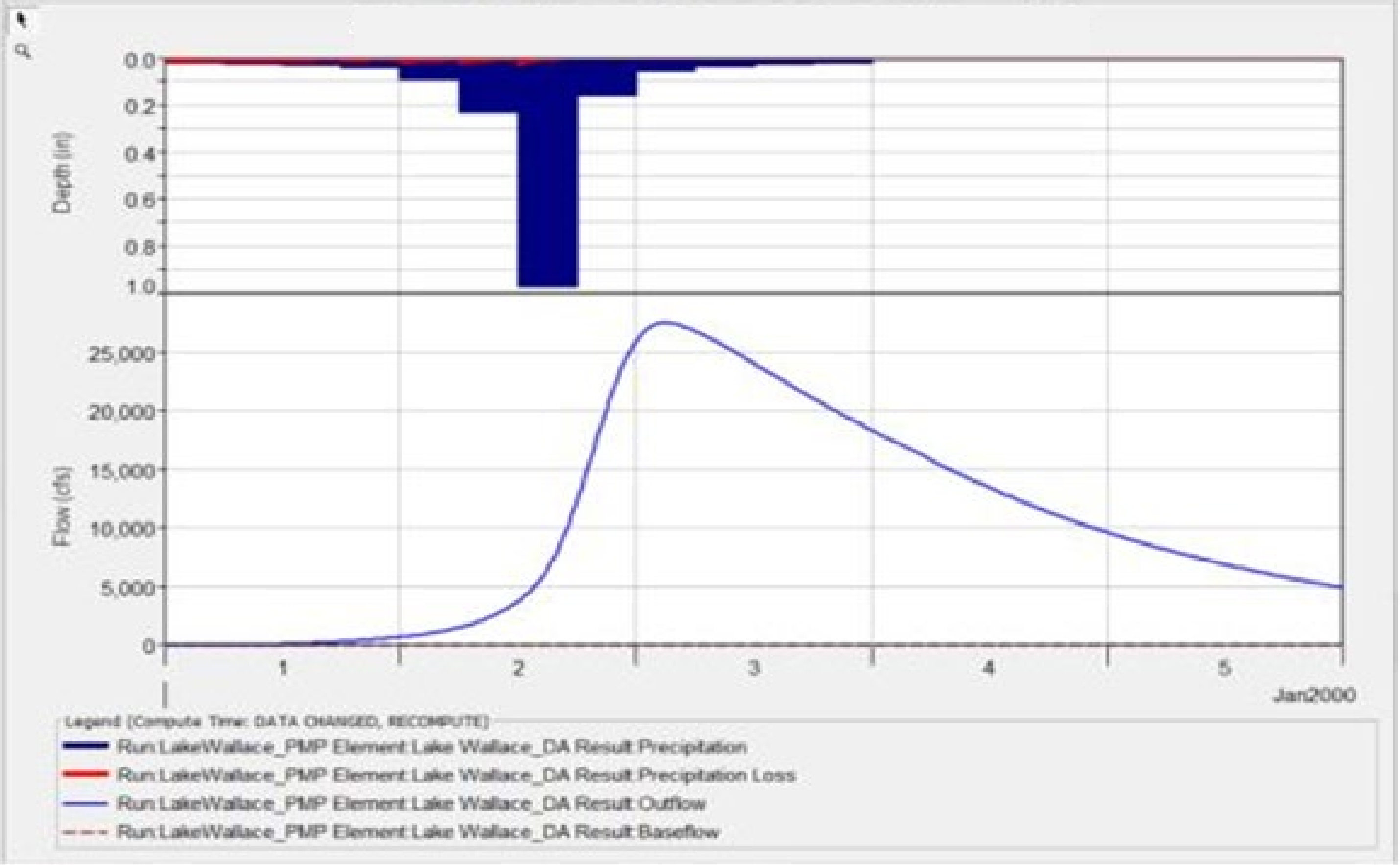
Contributing Drainage Area	56.4 sq. mi.
Dam Height (NID)	19 ft
Dam Crest El.	145.40 ft NAVD88
Primary Spillway Crest El.	141.25 ft NAVD88
Primary Outlet Dimensions	370 ft-long concrete spillway, in line with Five 10-ft H x 8-ft V box culverts

Detailed Example of Calculating SDF using ICA

Incremental Consequence Analysis (ICA) (continued)

Inflow Boundary Conditions

The Sunny Day peak flow was consistent with the USACE model using a constant baseflow of 80 cfs. The peak PMF inflow to the dam was calculated to be 27,595 cfs. The PMF inflow hydrograph is estimated using the HEC-HMS as follows



Detailed Example of Calculating SDF using ICA

Incremental Consequence Analysis (ICA) (continued)

Breach Parameters

The breach parameters for the Sunny day breach and PMF breach remained the same as the USACE model. The overtopping failure for the full PMF is triggered when water level is 18 inch above the dam crest. For some trial fractions of the PMF, the breach scenarios were set to trigger at a water surface elevation just below the maximum non-breach pool elevation in the West storage area as the breach criteria of 18-inches above the top of dam was not reached. The following table summarizes the breach parameters for the sunny day and PMF breach.

Parameter	Sunny Day Breach	PMF Breach
Final Bottom Width (ft)	93	98
Final Bottom El. (ft NAVD88)	129	129
Left/Right Side Slope	0.5	0.5
Breach Weir Coeff.	2.6	2.6
Breach Formation Time (hr)	0.5	0.5
Failure Mode	Piping	Overtopping
Piping Coeff.	0.5	-
Initial Piping El. (ft NAVD88)	134.2	-
Failure Trigger	1 hr into Simulation	WS El. 147.4 ft NAVD88

Detailed Example of Calculating SDF using ICA

Incremental Consequence Analysis (ICA) (continued)

Simulation results without a breach

Simulations of the dam, impoundment, and downstream channel under storm scenarios without breach form the basis for two important components of the dam hazard classification analysis:

- The simulated hydraulic conditions at the dam prior to the breach, which are the basis for the parameters used to define the size and progression of the breach.
- The inundation extents without a breach provide a basis with which to compare the dam failure inundation to evaluate the incremental impact of dam failure.

The following table summarizes the results at dam reservoir without breach

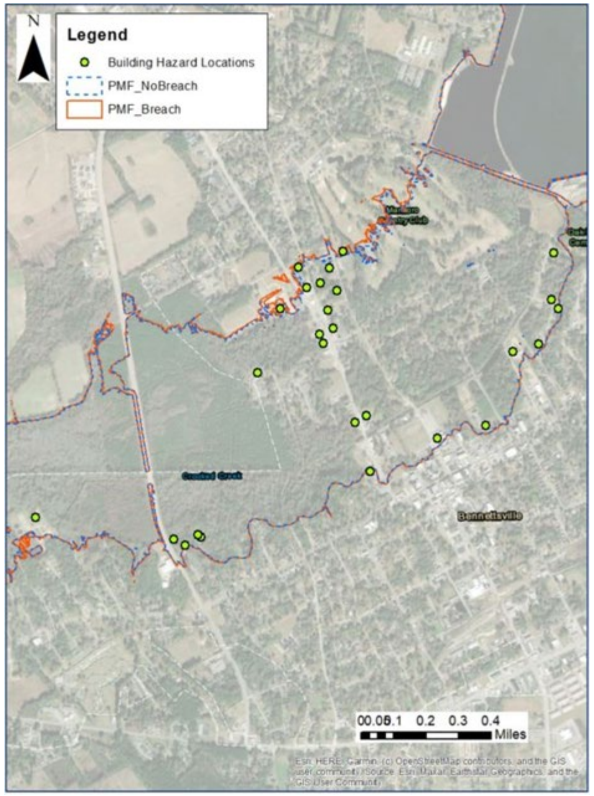
Storm Scenario	Simulated Impoundment Conditions			
	Peak Water Surface	Water Height West Dam Face	Storage	Approximate Surface Area
"Sunny Day"	El 141.4 ft NAVD88	15.4 ft	2,425 ac-ft	426 acres
Full PMF (100%)	El 147.6 ft NAVD88	21.6 ft	6,117 ac-ft	828 acres

Detailed Example of Calculating SDF using ICA

Incremental Consequence Analysis (ICA) (continued)

Incremental Structure Hazard

Twenty-seven locations of building structures are selected downstream of the dam that fall on the edge of the non-breach PMF maximum inundation extent so they can be analyzed for increased hazard during the breach event. The following figure and table depict these locations on the map as well as the ground elevation for them.



Location	Address	Elevation (ft NAVD88)	Location	Address	Elevation (ft NAVD88)
Location 1	131 Stornaway St	130.2	**Location 15**	533 Cheraw St	132.4
Location 2	255 Huckabee St	124.6	**Location 16**	404 Cheraw St	133.2
Location 3	706 Hamlet Hwy	134.4	**Location 17**	711 N Marlboro St	135.8
Location 4	224 Dudley St	127.9	**Location 18**	723 Grace Ct	126.8
Location 5	399 Cheraw St	127.5	**Location 19**	598 Hamlet Hwy	132.4
Location 6	121 James S. McLeod	132	**Location 20**	300 N Everett St	128
Location 7	531 Cheraw St	132	**Location 21**	701 Cheraw St	134
Location 8	400 E Market St	132.3	**Location 22**	516 McLeod St	128.9
Location 9	714 N Marlboro St	137.4	**Location 23**	222 Dudley St	129.2
Location 10	717 Hamlet Way	136.5	**Location 24**	534 Miles St	131.5
Location 11	709 Hamlet Hwy	134.9	**Location 25**	101 Tee St	132.9
Location 12	503 Miles St	134.3	**Location 26**	622 McLeod St	129.5
Location 13	205 E Market St	128	**Location 27**	106 Stornaway St	133.9
Location 14	703 N Marlboro St	134.3			

Detailed Example of Calculating SDF using ICA

Incremental Consequence Analysis (ICA) (continued)

Dam Failure Simulation Results

Sunny Day Failure

Evaluating the Sunny Day Failure hazard represents the lower bound to the ICA. The following figure shows the inundation map due to Sunny day failure. Four out of the twenty-seven locations are identified in the Judgement zone of the ACER-11 charts

The following table shows the peak depth and peak velocity and the danger classification according to ACER-11

Location	Sunny Day Breach Peak Depth (ft)	Sunny Day Breach Peak Velocity (feet per second)	Sunny Day Breach Occupant Hazard[1]
Location 13	4.64	0.8	JUDGEMENT
Location 20	5.03	1.17	JUDGEMENT
Location 22	3.91	0.95	JUDGEMENT
Location 26	3.63	0.69	JUDGEMENT

Detailed Example of Calculating SDF using ICA

Incremental Consequence Analysis (ICA) (continued)

Probable Maximum Flood (PMF)Failure

Under this scenario a full PMF inflow coincides with a dam breach event to evaluate the flood hazards at the dam downstream. The following figure shows the inundation map for this scenario under the no-breach and the breach scenarios. Flooding extent is almost the same between the two scenarios except for the area just downstream the dam.

Detailed Example of Calculating SDF using ICA

Incremental Consequence Analysis (ICA) (continued)

The following table shows a comparison between the ACER-11 results due to the PMF scenario with and without the dam breach. It is shown that three locations (12, 14, 24) out of the twenty-seven shifts the danger zone between the two scenarios.

The adjacent figure shows the buildings in which the danger classification has been shift for the case of no-breach and breach cases, also the figure shows the buildings in the judgement zone for the Sunny day failure.

Location	PMF without Breach			PMF with Breach		
	Peak Depth (ft)	Peak Velocity (feet per second)	Occupant Danger Zone[1]	Peak Depth (ft)	Peak Velocity (feet per second)	Occupant Danger Zone[1]
Location 12	2.99	0.64	LOW	3.49	0.79	JUDGEMENT
Location 14	2.66	0.64	LOW	3.12	0.78	JUDGEMENT
Location 24	5.87	0.52	JUDGEMENT	6.38	0.62	HIGH
Location 1	5.09	0.2	JUDGEMENT	5.41	0.29	JUDGEMENT
Location 2	3.97	1.99	JUDGEMENT	4.42	2.16	JUDGEMENT
Location 3	2.55	0.31	LOW	3.01	0.43	LOW
Location 4	7.41	0.53	HIGH	7.74	0.59	HIGH
Location 5	8.38	1.24	HIGH	8.78	1.4	HIGH
Location 6	4.6	1.19	JUDGEMENT	5.02	1.31	JUDGEMENT
Location 7	3.98	1.18	JUDGEMENT	4.39	1.36	JUDGEMENT
Location 8	4.71	0.35	JUDGEMENT	5.18	0.41	JUDGEMENT
Location 9	0	0	LOW	0.05	0.14	LOW
Location 10	0.43	0.03	LOW	0.89	0.21	LOW
Location 11	1.91	0.46	LOW	2.36	0.61	LOW
Location 13	8.86	0.96	HIGH	9.31	1.1	HIGH
Location 15	3.63	1.22	JUDGEMENT	4.06	1.42	JUDGEMENT
Location 16	3.35	1.42	JUDGEMENT	3.77	1.6	JUDGEMENT
Location 17	1.17	0.25	LOW	1.63	0.35	LOW
Location 18	8.7	0.95	HIGH	9.06	1.08	HIGH
Location 19	4.33	1.5	JUDGEMENT	4.76	1.72	JUDGEMENT
Location 20	9.41	1.1	HIGH	9.92	1.29	HIGH
Location 21	1.59	0.31	LOW	1.98	0.4	LOW
Location 22	8.27	0.95	HIGH	8.76	1.11	HIGH
Location 23	6.12	0.48	HIGH	6.44	0.54	HIGH
Location 25	3.94	0.9	JUDGEMENT	4.39	1.09	JUDGEMENT
Location 26	8	0.91	HIGH	8.52	1.06	HIGH
Location 27	1.39	0.17	LOW	1.71	0.21	LOW

Detailed Example of Calculating SDF using ICA

Incremental Consequence Analysis (ICA) (continued)

-According to DSG-501 Section 501.1, a more detailed review is required to confirm whether a point shifts hazard zones when its results fall close to a zone boundary.

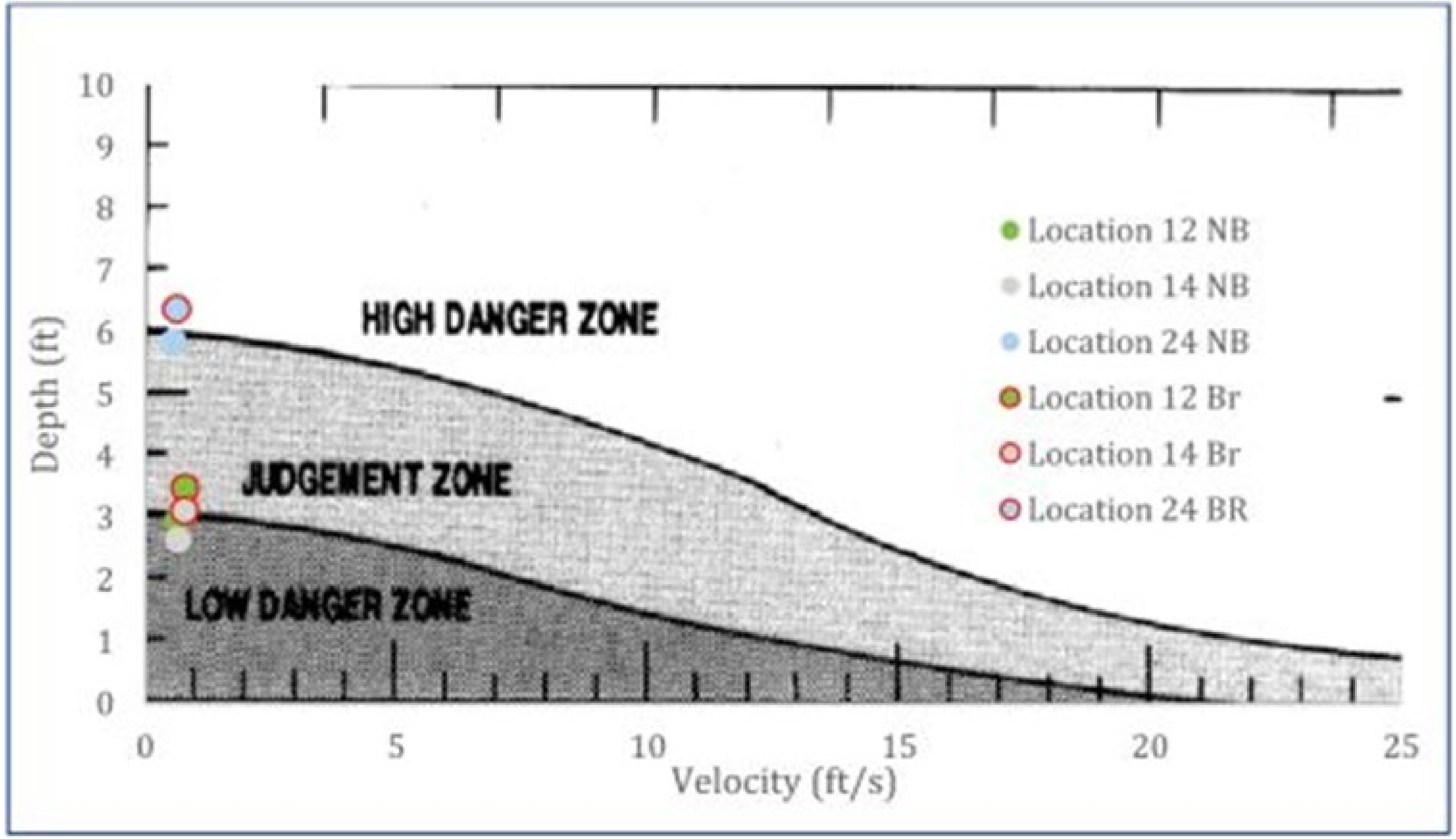
-Step 6 states that when a non-failure water level is within 10% of the boundary curves in Figure 501-1 (ACER-11), a conservative approach should be used. If both the failure and non-failure cases fall within 10% of a boundary, the result can be treated as “no shift.”

To verify whether any locations truly change hazard classifications, each point’s depth and velocity were compared to the ±10% bounds of the boundary curves. The table below summarizes the zone-shift check. Based on this detailed analysis, only Location 12 shows a confirmed shift—from Low to Judgment.

Location	Without Breach w/in 10%	Breach w/in 10%	Shift/No shift	Determination
12	Yes	No	Possible Shift	Confirmed Low to Judgement
14	No	Yes	Possible Shift	Breach should be LOW, so **No shift**
24	Yes	Yes	No Shift	N/A

Detailed Example of Calculating SDF using ICA

Incremental Consequence Analysis (ICA) (continued)



As a conclusion for the incremental consequences analysis, the SDF should be the PMF of inflow peak equal to 24,595 cfs since at least one hazard zone shift is detected when comparing the no-breach with the breach scenarios.

Conclusions

1- Dam-breach analysis is important for regulated dams in South Carolina. It is used to map downstream inundation, estimate Population at Risk (PAR), and support hazard classification and Emergency Action Plan (EAP) development. State regulations require an EAP for all high and significant hazard dams; for low-hazard dams, EAPs are not required but considered good practice.

2- Dam-breach parameters can be estimated using regression equations based on historical dam failure provided in the literature of similar limits to the studied dam.

3- Software packages, such as HEC-RAS, have multiple methods to conduct dam-breach analysis with the aid of the results of the regression equations.

4- The spillway system must be able to safely pass the Spillway Design Flood (SDF). An initial SDF is selected from the range in Table 1 of DSG-501 (SC dam safety regulations), based on the dam's size and expected hazard potential classification. This SDF may then be refined using incremental consequence analysis, comparing flood hazards with and without a dam failure. The SDF may be reduced, within the Table 1 range, to the smallest flood for which dam failure does not significantly increase downstream hazard or change the hazard zone at any critical location.