OKLAHOMA
Water Resources Board

Hydrologic and Hydraulic Guidelines for Dams in Oklahoma

OKLAHOMA WATER RESOURCES BOARD

DAM SAFETY PROGRAM

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1. Introduction

The National Dam Safety Review Board monitors the safety of dams in the United States, advises the Administrator of the Federal Emergency Management Agency (FEMA) on national dam safety policy, and consults with the Administrator of FEMA for the purpose of establishing and maintaining a coordinated National Dam Safety Program. The National Dam Safety Review Board and the dam safety community recognize that development of Emergency Action Plans (EAPs) for all high and significant hazard potential dams in the United States is critical to reducing the risks of loss of life and property damage from dam failures. However, only about one-half of state regulated high hazard potential dams within the United States currently have an EAP.

Development of EAPs for all high hazard-potential dams in Oklahoma is critical to reducing the risks of loss of life and property damage from dam failures. Oklahoma does not require an EAP for significant hazard-potential dams. Approximately 89% of high hazard-potential dams in Oklahoma have an EAP although many of them are old and need to be updated.

Detailed breach engineering analysis for inundation studies may provide a more precise representation of potential flooding for a given set of assumptions, but not necessarily a more accurate representation of actual flooding should dam failure occur. For small and intermediate size dams, simplified methods may provide useful inundation maps at a reduced cost. Inundation maps based on simplified engineering analysis are developed by employing simplified assumptions and methods. Simplified engineering analysis can either form the permanent basis of emergency and evacuation planning or be used for the interim, until more detailed mapping can be obtained.

These guidelines represent instructions, standards, and accepted procedures for the hydrologic and hydraulic analyses of existing and proposed dams in the State of Oklahoma. They also serve to clarify the expectations of the OWRB with respect to submitted analyses, and simplify the review procedure by standardizing processes and elements that will be acceptable to the reviewer. Though the guidelines are relatively specific, the engineer will always be able to submit alternate procedures that are either more conservative or are sufficiently explained and justified.

These guidelines assume that anyone using or referencing them is a licensed professional engineer or is working under the guidance of a professional engineer, has appropriate knowledge of the processes and methodologies referenced within, and has suitable capabilities to utilize the computer software that are of standard use in the engineering profession and are appropriate to the analysis effort.

2. Dam Classification

Dams subject to OWRB jurisdiction are described in Chapter 25 of the Board's rules (OWRB Rule 785:25-3-1). The size classification for dams located in Oklahoma is based on the following ranges (OWRB Rule 785:25-3-3):

1) **Size Classification of Dams.** The size classification shall be based on the following ranges:

   A) Small dams are those with maximum storage of less than 10,000 acre-feet and a dam height of less than 50 feet;
   B) Intermediate dams are those with maximum storage of between 10,000 and 50,000 acre-feet or a dam height of between 50 and 100 feet; and
   C) Large dams are those with maximum storage of greater than 50,000 acre-feet or a dam height of greater than 100 feet.

2) **Hazard-Potential Classification of Dams.** The hazard-potential classification of a dam is determined by the downstream risk in the event of a failure, without regard to the physical condition of the dam, as follows:

   A) **Low hazard-potential:** those where failure would result in no probable loss of human life and low economic losses.
   B) **Significant hazard-potential:** those where failure would result in no probable loss of human life but can cause significant economic loss or disruption of lifeline facilities.
   C) **High hazard-potential:** those where failure will probably cause loss of human life.

The magnitude of the design flood to be used for any dam under the jurisdiction of the OWRB is dependent on its size and hazard-potential classification, and is described as a percentage of the Probable Maximum Flood (PMF) which is derived from the Possible Maximum Precipitation (PMP) as described herein. The PMP depth values to be used can be found in Hydrometeorological Report No. 51 (NWS, 1978).

The PMF refers to the flood that may be expected from the most severe meteorological and hydrologic conditions that are reasonably possible in the region where the dam is located as listed in Hydrometeorological Report No. 51 (NWS, 1978). The table below shows the design flood as a percentage of the PMF for each category of dams along with their associated minimum design freeboard.

Table 1: Spillway Capacity Criteria – Title 785:25-3-6-(b)

Size	Hazard	Design Flood	Minimum Freeboard
Small	Low	25% PMF	0 Feet
	Significant	40% PMF	0 Feet
	High	50% PMF	1 Foot
Intermediate	Low	25% PMF	1 Foot
	Significant	50% PMF	1 Foot
	High	75%PMF	3 Feet
Large	Low	50% PMF	1 Foot
	Significant	75% PMF	1 Foot
	High	100% PMF	3 Feet

Dams constructed prior to June 13, 1973

Any dam constructed prior to June 13, 1973 and which is classified as intermediate size and high hazard potential according to 785:25-3-3 shall be required to pass a minimum design of 50% of the PMF. Any dam constructed prior to June 13, 1973 and which is classified as large size and high hazard potential according to 785:25-3-3 shall be required to pass a minimum design flood of 75% of the PMF.

Design Flood

Once the full PMF hydrograph is estimated, the design flood level is determined, based on the size and hazard classification as described above. For hydrologic models that produce a full reservoir inflow hydrograph, the design flood is determined by multiplying each ordinate of the PMF inflow hydrograph by the required baseflow percentage. No adjustments to the precipitation data should be made. This design flood inflow hydrograph is then routed through the reservoir appropriately to determine the peak design flood level.

For models that utilize hydraulic flood routing methods for the streamflow components and the reservoir, each runoff hydrograph that represents a boundary or lateral inflow hydrograph should be adjusted in a similar manner, multiplying each ordinate of the hydrograph by the specified percentage. These adjusted hydrographs would then be routed through the hydraulic model to determine the design flood level.

3. Precipitation

Watershed Delineation

Many of the dams in Oklahoma can be modeled appropriately with a single basin. However, many will need to be subdivided into multiple sub-basins. The size and delineation of the sub-basins is dependent on the rainfall-runoff method used and various hydrologic factors. Sub-division should also be considered if there are portions of the drainage basin that:

- Possess hydrologic characteristics obviously different from the average characteristics of the total basin
- May contribute to delays in flood passage, such as upstream lakes
- Are controlled by large constrictions that can act as hydraulic control structures by restricting cross-sectional area and attenuating water flow, as may occur at some bridges
- Have a total drainage area that is too large for averaging a single storm distribution
- Have stream gages or observed data that may used for calibration

Watersheds should be delineated and their characteristics determined in accordance with the standards of the following references:

- National Engineering Handbook, Part 630 Hydrology (NRCS, 1998)
- EM 110-2-1417 (COE, 1994)
- FERC PMF Determination

PMP Duration

The design storm should be of an appropriate duration to accurately represent the watershed. In most cases a 24-hour storm duration is appropriate.

Storm Location and Spatial Distribution of PMP

For drainage areas less than 10 square miles the total depth of the PMP, estimated as the point values delineated in HMR-51 and HMR-52, shall be applied uniformly over the entire drainage area. For drainage area greater than 10 square miles the total depth of the PMP should be spatially distributed over the drainage area using the single-centered concentric ellipse pattern and methodology specified in HMR-52 (NWS, 1982).

For single basins, the center for the storm should generally be at the centroid of the basin and a basin-average total depth of design storm precipitation calculated for the specified duration. For larger basins, when the watershed is divided into multiple sub-basins, the center of the PMP storm isohyets must be moved to multiple locations away from the geometric centroid of the overall drainage area to verify the critical design storm location and orientation that produces the maximum corresponding PMF level in the reservoir. This will generally be the same storm center that produces the maximum basin-average total PMP depth. However, in very large basins greater than about 10,000 square miles, this may not be the case depending on the size and orientation of various tributaries, so the full flood routing through the reservoir should be included in the iterative process.

4. Runoff Calculations

Antecedent Moisture Conditions

The PMP shall be superimposed upon watershed soils that are assumed to be saturated. This would equate to initial losses at the beginning of the design storm equal to zero or NRCS Antecedent Runoff Conditions III (ARC III), or some other equivalent and approved assumptions. In Oklahoma there is not a need to analyze snowmelt contributions to runoff or frozen ground conditions for infiltration for baseflow calculations.

Infiltration Losses – Excess Precipitation

Determination of excess rates of precipitation and infiltration losses can be determined by one of several precipitation los methods. The two most common methods are:

- Initial and Constant-Rate
- NRCS Curve Number

Other possible methods would include:

- Green and Ampt
- Holton Loss Rate
- Exponential Loss Rate

These methods can be utilized and are described in most hydrology textbooks and in appropriate computer model user manuals. For certain areas, it may be appropriate to assume a certain percentage of the basin or sub-basin has no infiltration at all. This would apply to paved areas, areas representing buildings or homes, as well as open water areas. These areas are typically designated by a percentage impervious area in the description of the basin characteristics. Large areas, such as the reservoir area itself, if it is a significant portion of the drainage area, can be modeled as a unique sub-basin with zero infiltration losses.

Transform

The PMP needs to be transformed into the PMF runoff hydrograph for each basin or sub-basin using an acceptable unit hydrograph method. The two most commonly used methods for hydrologic and hydraulic studies associated with dams are:

- Snyder Unit Hydrograph
- NRCS Dimensionless Unit Hydrograph

Other possible methods that are used include:

- Clark Unit Hydrograph
- Kinematic Wave

These methods can be utilized and are described in most hydrology textbooks and in appropriate computer model user manuals.

5. Storm Runoff Hydrograph

Hydrologic and Hydraulic Streamflow Routing Methods

In models where routing of the design storm runoff hydrographs through a stream channel is needed, either a hydrologic or hydraulic routing method may be used. The hydrologic methods are either empirical or semi-empirical. Approved hydrologic routing methods would include:

- Muskingum-Cunge Routing
- Modified Puls Routing
- Average-Lag Routing
- Muskingum Routing

Acceptable hydraulic modeling methods would include:

- Kinematic Wave Routing
- Dynamic Wave Routing
- Diffusion Wave Routing

Baseflow

Estimations of baseflow conditions upon which calculated runoff hydrographs are added are sometimes appropriate on larger rivers, particularly for frequency flood level events, such as the ten year or 100-year floods. However, in Oklahoma, they are rarely a significant component of the Design Storm peak flows. For any river that would have a dependable flow that could be counted as a baseflow, the drainage area is usually quite large and the resulting baseflow runoff will still dwarf the baseflow. For these reasons, baseflow is not required within baseflow calculations. They may be utilized, if deemed appropriate by the analyst, and would typically be of the order of magnitude of median flows or would be estimated as the receding limb of an antecedent event.

Hydraulic Input Parameters

All of the streamflow routing methods utilize various input parameters, generally measured or estimated from the physical characteristics of the channel. Methods will include a parameter that measures the length of the channel directly or reflects it in an estimate of travel time. For these, the full length of the channel should be utilized without reflecting assumed shortening due to overbank flooding. Even in very high flows, the predominant conveyance is usually within the channel itself. If the overbank flows are thought to be a dominant factor in the flood conveyance, then a routing method that takes differing floodplain flow characteristics into account, including the possible use of hydraulic models, should be used.

Most routing methods will also typically use some description of a typical cross section or multiple cross sections. These should simply be measured from available mapping and should represent average conditions. For hydraulic models, where the entire length of the channel is modeled with cross sections, reasonable mapping will be necessary, though rarely will surveyed cross sections be justified. For unsteady flow hydraulic models, such as the NWS models and HEC-RAS, it is important that the cross sections reasonably accurately reflect the floodplain storage that exists. In steady state models, there is a tendency to place cross sections at constrictions and other features that control the flow rate. However, in unsteady models, the full flood storage of the floodplain tends to control the downstream movement of the hydrograph. Therefore, cross section should reflect the constricted sections and the wider sections, as well as the storage that occurs in tributaries and adjacent draws. Off channel storage, or ineffective flow area, which is often ignored in steady state models, might have a significant impact in unsteady models.

Manning's Roughness Coefficients

Except for the purely empirical hydrologic equation, each of the streamflow routing models will use the most widely recognized flow relationship, the Manning's equation. Input parameters to the Manning's equation consist of descriptions of the topography through the use of cross sections, either detailed or a simplified version, and the roughness coefficient. Numerous sources exist that describe the use of the equation and provide means to estimate the Manning's roughness coefficient (n), both for channel flow and overbank flow. The primary criterion is the size of the roughness particle, usually vegetation or the exposed channel surface, relative to the overall flow area. For example, the same grass lined channel will have a smaller n value if the channel area is large than if it is small. For that reason, roughness values will decrease with increasing flow for a consistent channel. However, this trend is often countered by the fact that roughness in the form of vegetation tends to increase quickly when significant portions of the flow are in the overbank areas. Single n values for a channel regardless of flow apply only in simplified estimates or in a narrow range of flows. For design flood calculations, where there is likely to be a large component of flow area in the floodplain, some variation in n value will need to be taken into account. The simplest way to do this is to utilize three n values, one for the channel and one for each overbank.

Modeling Through Reservoirs

Hydrologic Routing:

For hydrologic routing, the peak baseflow reservoir elevation is determined by routing the estimated inflow hydrograph through the reservoir using one of the various hydrologic models.

The methods most typically used for routing a hydrograph through a reservoir in hydrologic models are:

- Level Pool Routing
- Modified Puls Method

The following considerations should apply:

- Reservoir losses should be assumed to equal zero
- For gated dams, the design flood inflow hydrograph should be routed through the reservoir and through the dam's hydraulic control structures using the planned flood operational rules for the spillway or in a manner that takes into consideration downstream flood risks
- The antecedent reservoir elevation in the reservoir should be the maximum normal operating pool (MNOP) level, which is the highest water surface elevation within the range of planned operating levels for the reservoir, above which floodwaters would be released. No antecedent storm is required. The following criteria apply to all upstream reservoirs in the drainage area:
    - For detention ponds that are dry or do not have significant permanent storage, the MNOP would be considered to be at the level of the primary outlet, above which water is always released
    - For recharge reservoirs that are normally dry but have no release capabilities, the MNOP would be an empty reservoir provided that it can be shown that the lake has historically or will typically be dry within a week of a major storm event. If not, the design engineer should show a statically based justification for an appropriate starting water level
    - For existing storage reservoirs that have not filled up to their MNOP within the last ten years, starting levels at both the MNOP and the maximum level of the lake within the last ten years should be used. If the dam can safely pass its appropriate baseflow at the lower historical level, but not at the MNOP, then modifications to enable the dam to pass the design flood will still be required. However, these modifications do not need to be initiated until such time as the reservoir reaches a water level starting at which it cannot safely pass the design flood. This "trigger starting elevation" at which the dam is overtopped by the design flood should be determined and reported along with the rest of the analysis
    - The reservoir shall not be assumed to be drawn down below the maximum normal operating level in advance of the design storm

<u>Hydraulic Routing</u>

For hydraulic routing of hydrographs through reservoirs, there is no distinction between streamflow and reservoir routing. The reservoir is simply modeled with cross sections as part of the stream with the spillway modeled either directly in the computer model or as an internal boundary based on the spillway rating curve. Though the modeling methodology is the same, certain issues should be considered.

- Through all but the shallowest portions of the reservoir, water levels are not sensitive to roughness coefficients.
- Cross sections and the intervening lengths should be adjusted to match the overall reservoir elevation – storage relationship.
- For gated dams, the design storm inflow hydrograph should be routed through the reservoir and through the dam's hydraulic control structures using the planned flood operational rules for the spillway or in a manner that takes into consideration downstream flood risks.
- The reservoir should not be assumed to be drawn down below the maximum normal operating level in advance of the design storm.

6. Hydraulic Design Criteria

Elevation – Area – Capacity (EAC)

The EAC should be determined from the best mapping available and is commonly performed by simply measuring the area within the reservoir at all available contours. This can often be done using GIS techniques, though it is important to properly account for islands within the overall area. In many areas of Oklahoma, U.S. Geological Survey (USGS) 1:24,000 mapping is available with 10 foot contour intervals. The areas at each contour interval from the bottom of the reservoir to the first contour above the top of the dam should be measured and plotted. The curve generated through these points should then be used to pull off areas at individual one foot increments and a summation of the average area for each one foot increment tabulated. This process will provide appropriate volumes of storage at each elevation needed for the flood routing process.

For existing reservoirs for which no mapping below the water surface is available, an EAC curve can be constructed starting with an assumed storage at the maximum normal operating level, which will be used as the starting water surface elevation for the flood routing. Incremental storage amounts below the starting water surface do not impact hydrologic flood routing procedures.

Spillway Rating Curves

The spillway rating curve needs to be determined for both existing and proposed reservoirs and for each component of the dam that will be utilized to pass flood flows during the design flood event. For many dams, this will be a combination of a principal spillway that is utilized in larger, rarer events. There are numerous sources available for descriptions of appropriate methodologies for determining rating curves. Widely used references include *Design of Small Dams,* by the Bureau of Reclamation and NRCS methodologies. These can be used for both existing and proposed spillways.

Principle Spillways

Rating curve development needs to be performed in a manner reflecting the unique characteristics of the individual spillway. Ungated spillways will generally be shaped as a weir, either ogee, sharp crested, or broad crested, or as a drop inlet, or morning glory spillway. The weirs would utilize the weir equation, $Q = CLH^{3/2}$ with C set by the shape and dimensions of the structure. In planning stages, C can be assumed to be constant, but in design level analyses, C would also vary with the height of water over the crest. Gated spillways would typically utilize the same procedure for determining total capacity, assuming the gates were opened sufficiently so as to not affect the flow. Discharges through partial gate openings would utilize orifice flow assumptions. Standard drawdown profiles would be needed, assuming unimpeded flows, in order to determine the size of the gate opening needed to switch back to weir control for the rating curve.

Rating curves for drop inlet, or morning glory spillways are generally calculated assuming two different types of hydraulic control. For low flows, the circumference of the inlet operates as a weir, with discharge estimated using the weir equation. For higher flows, the inlet will work as an orifice at the narrowest portion, or throat, of the vertical column. If the spillway conduit through the dam is designed for pressure flow, then the hydraulic control may rest with the sum of energy losses acting through the closed system as a whole. In these cases, entrance, bend, friction, and exit loss coefficients, along with other losses that may apply, are summed up and the rating curve determined for a closed, pressure flow system. For these types of spillways, calculations assuming each of the types of flow patterns would be made and the lowest discharge used as the controlling situation.

In the final rating curve developed for reservoir routing, points on the curve reflect the total discharge. This will typically require more points at the lower end of the curve where the rates change more rapidly. It will also require points at key break points such as the crest of the emergency spillway or where changes in the gate operating procedures occur.

For proposed dams and new spillways, the following concepts should be considered in order to determine the most appropriate principle spillway type:

- Drop inlets, or morning glory spillways, are generally used when there is plenty of available flood storage volume. The flow capacity of the spillway does not significantly increase once the reservoir reaches a level at which the spillway “plugs” or operates under pressure or orifice control. They are well suited for flood control purposes.

- In morning glory spillways, it is preferable that the conduit through the dam be designed to have open channel flow with depths not more than 75% of the height of the conduit. This will generally require a hydraulically steep slope carrying supercritical flow and a diameter that is greater than the diameter of the throat of the spillway riser. If this is not practical, then a conduit significantly smaller than the riser that forces pressure flow through the conduit quickly would be preferable. Both of these concepts attempt to minimize the large pressure fluctuations that typically occur with flow transitioning from open channel to pressure flow.

- Larger morning glory inlets will need anti-vortex devices to break up naturally occurring vortices in the entering flow.

- Gated spillways require considerable additional cost for the operating system, operating personnel, and maintenance. In addition, it is generally perceived that an owner takes on significant additional liability with a gated spillway.

Emergency Spillways

Emergency spillways are generally cut into an abutment and have little or no erosion protection from flows discharging through them. For this reason, they are an economical way to pass large quantities of flow for only the largest and rarest of floods. It is often accepted that erosion damage will occur should the emergency spillway operate, but that the effective cost of the repairs that will be needed on a very infrequent basis are much less than the upfront capital costs of the means to prevent the erosion. Most emergency spillways are built to prevent passage of flows for less than about the 50- or 100-year flood.

Rating curves for emergency spillways should generally be determined using a backwater analysis with a steady state water surface profile model, such as HEC-RAS. This process consists of performing several runs with varying discharges, relating each to a reservoir water surface elevation. Enough sections are needed such that the most upstream section has minimal approach velocity and any energy losses upstream from that point can be ignored. The rating curve is then found by assuming the energy level, not the water surface elevation, at the most upstream section equates to the reservoir level for the specified discharge. These values are then plotted up and the discharges for set elevations determined from the curve. A standard broad crested weir equation should be used only for rough planning efforts. For a broad crested weir equation to be accurate, the slope downstream from the crest would need to be steep enough to effect supercritical flow, which, if operated, would cause much more damage than would occur under subcritical flow.

For proposed dams and new emergency spillways, the following concepts should be considered in order to determine the most appropriate configuration and location:

- Emergency spillways should be located such that any flows that discharge through will not impact on or flow against the dam embankment.

- The channel should be configured such that critical flow will occur as far downstream as reasonably possible so as to maximize the length of any erosion path back to the reservoir. The crest can be centered, but the slope downstream from the crest should be set to effect subcritical flow. This will generally be a slope of about 0.25% or less depending on the cover. An alternative would be to address the potential erosion from supercritical flow, either with the provision of an erosion-resistant surface or a determination that the final configuration will not erode sufficiently to make a significant release of water from the reservoir.

- The crest of the spillway should be set above the 50- or 100-year flood level to minimize its frequency of operation. In general, the less frequent that an emergency spillway will operate, the more erosion that will tend to be acceptable.

- The roughness coefficients used in the analysis should reflect ultimate vegetative conditions of the emergency spillway, not newly constructed conditions.

7. <u>Dam Breach Analyses</u>

Dam breach analyses should be performed to determine a dam's hazard-potential classification, and for all high hazard dams to delineate the dam breach inundation areas for use in EAPs. This section describes the appropriate methodologies and assumptions to be used when performing a dam breach analysis.

Hydrologic Conditions

Full breach analyses shall be performed for the following hydrologic conditions at a minimum:

- Sunny-day breach – Reservoir at its maximum normal operating pool level

- Barely Overtopping breach – Inflow design flood set to the percentage of the PMF or design flood that equals the top of the dam. If the dam passes 100% of the design flood without overtopping, this scenario does not need to be run.

- Design flood breach – Inflow hydrograph equal to the full design flood.

For terminal storage reservoirs only the breach analysis under sunny-day conditions is required.

The Barely Overtopping and the design flood breach runs would be compared to runs for the same event assuming that the dam does not fail. <u>The breach inundation area should extend far enough downstream such that the difference between the peak flood stages of the breach and non-breach runs are less than 1 foot in height.</u>

If the design flood overtops the dam, the analysis will need to either assume flow over the top of the dam or not. The former adds complexity to the model as the length of the dam that is overtopped is reduced by the breach width, but it also provides a more exact and less conservative determination of breach discharge for existing conditions. For dams for which upgrades to the dam are being considered, no flow over the crest should be assumed, as if the dam were raised to contain the design flood. This would tend to be simpler and more conservative. The breach should be initiated at the peak reservoir level under each scenario.

Downstream Conditions

In the Barely Overtopping and design flood conditions, inflows from downstream tributaries should be estimated by extending the design flood ellipses under the HMR-52 methodology over the associated areas. The ellipses, their size, location, and orientation from the critical design flood should be used without adjustment. Downstream runoff hydrographs would be adjusted to the same degree that the Barely Overtopping flood is upstream of the reservoir. Runoff parameters from the dam's watershed should be assumed to apply or be extrapolated appropriately to the adjacent basins. No additional calibration downstream is warranted. If the stream on which the breach hydrograph is being routed opens into a much larger river, then multiple considerations need to be made. These would include:

- The stage hydrograph on the receiving stream would serve as a downstream boundary of the initial stream and the discharge hydrograph of the upstream river would serve as a lateral inflow hydrograph to the downstream river. This process is performed iteratively until agreement is reached. HEC-RAS and FLDWAV can model a dendritic system and would usually be appropriate.

- A large volume of water discharging into the receiving stream will tend to distribute flows in both the upstream and downstream directions. Depending on the initial flow on the receiving stream, this can be seen as a reduction in flow rate or even in negative flow traveling upstream. Therefore, sufficient cross sections on the receiving stream need to be included upstream of the junction to allow for this phenomenon, if appropriate.

- If the drainage area of the receiving stream is too large for it to be effectively covered by the ellipses representing the design flood, an assumption on initial flows needs to be made. It can be assumed that significant flows will be occurring coincidental to the design flood on the tributary, however, the timing and magnitude would be virtually unrelated and indeterminate. Therefore a constant flow hydrograph can be assumed. As an example, this could be equal to the 10, 50, or 100-year flood, depending on the relative sizes of the two streams. The large the ratio of the drainage areas, the small the assumed flow level should be on the receiving stream. A reasonable level that puts the receiving stream at or slightly above flood stage before the breach flows arrived will usually produce the critical incremental impact due to the breach. This should be considered in the decision.

Breach Parameters

Breach Location

The breach analysis should be performed on the component of the dam for which failure would create the worst impact downstream regardless of relative likelihood of failure. This component of the structure would then be analyzed for the hydrologic conditions listed above. Each major component of the dam should be reviewed to determine the maximum discharge. This review would not take into account the likelihood of failure of any component, but should look at the most likely configuration of a breach, should one occur. For each structure component, the breach section should be at the maximum portion of the structure that can contain the full bottom width of the breach. For example, if the breach width for an embankment is 200 feet wide, the location should be planned for the lowest 200 foot section of the dam above natural ground at the toe. If the channel downstream is only 50 feet wide, then this would not be in the original channel or at the maximum height of the dam. However, the 200 foot wide breach at a higher level should be compared to a 50 foot wide breach at the maximum section over the river channel. Whichever has the higher peak discharge should be used.

Methods for Estimating Embankment Breach Characteristics

Several methods have emerged that estimate breach parameters of earthen embankments. Two of the most common methods are those developed by MacDonald and Langridge-Monopolis (1984) and Froehlich (2008). A comparison study by Wahl (2004) indicated the prediction equations developed by Froehlich (1995) had consistently low uncertainty orders of magnitude when compared with other published methods using a historical database of dam failures and comparing results to the physically-based erosion model NWS-BREACH (Fread, 1991). Based on Wahl’s study, the Dam Safety Program suggests the use of Froehlich equations in determining breach parameters for earthen dams. The 2008 Froehlich equations have been changed slightly from the 1995 equations used in Wahl’s study.

Froehlich's equations are as follows:

Average Breach Width: $B_{avg} = 8.289K_0V_w^{0.32}H_b^{0.04}$

Where:

B_{avg} is the average breach width in feet

K_0 is the Failure Mode Factor

$K_0 = 1.0$ for piping

$K_0 = 1.3$ for overtopping

V_w is the volume of the reservoir in acre-feet above the bottom of the breach

H_b is the breach height in feet, which is the vertical distance from the dam crest to the breach invert

Breach Development Time: $T_f = 3.664(V_w/(gH_b^2))^{0.5}$

Where:

T_f is the breach development time in hours

g is the gravitational acceleration = 32.2 ft/sec^2

Froehlich recommends breach side slopes of 0.7:1 (horizontal:vertical) for piping and 1.0:1 for overtopping.

Breach Configuration (Structural Components)

Breach configurations in a structural component of the dam such as the spillway or gravity section will need to be determined on a case by case basis. The minimum breach width will generally match individual elements of the structure, such as one buttress in a slab and buttress dam or one monolith in a gravity non-overflow section. Multiple adjacent components will also need to be reviewed with varying failure times in order to determine the critical configuration.

Time of Failure (Structural Components)

Time of failure for a breach configuration in a structural component of the dam such as the spillway or gravity section will need to be determined on a case by case basis. When an individual element of the structure, such as one buttress in a slab and buttress dam or one monolith in a gravity non-overflow section, is assumed, the time should be instantaneous. Multiple adjacent components will also need to be reviewed with varying failure times in order to determine the critical configuration. The incremental failure time of adjacent structures will be based on the estimated time for the component to fail due to erosion of the foundation. In all cases, some justification shall be presented. However, the amount of failure time per adjacent component shall not exceed 30 minutes per component in an alluvial foundation and one hour in a rock foundation. The analyst should also take into consideration the likelihood that this failure mechanism of adjacent structural components will occur in both directions outward from the original component.

Breach Inundation Lengths

Breach floodwave models should be extended sufficiently downstream to analyze the area that is likely to be significantly impacted by a breach of the dam and to provide sufficient information for proper development and execution of an EAP. Though judgment is needed on the part of the engineer performing the analysis, the following guidelines will generally be suitable:

- Sunny day breach – The floodwave should be modeled for a length downstream, beyond which approximately 75% of the flow is within the channel and no structures are threatened.
- Barely Overtopping and baseflow breaches – these floodwaves should be modeled for a length downstream, beyond which there is an insignificant incremental increase in the peak flood level due to the breach over the non-breach condition. This is generally considered to be one foot in developed areas. A higher differential may be used in undeveloped areas.

Dams in Sequence

For design flood PMF Analyses, upstream reservoirs should generally be assumed to remain intact. Downstream dams should be assumed to breach if overtopped by either the breach or non-breach condition assumed to not breach if not overtopped. It is possible that the design flood for the upstream dam does not overtop the downstream dam, but does with a failure of the upstream dam. In this case, the downstream dam should be assumed to fail in the breach scenario and to not fail in the non-breach scenario. This would be considered part of the impact of the failure.

8. Simplified Dam Breach Inundation Mapping

For some small, high-hazard dams where there is a low population at risk, a more simplified approach in creating dam breach inundation maps may be more appropriate. Simplified methods for breach mapping generally have the benefit of being less labor intensive, and therefore less costly, while still providing an adequate evacuation map for use in EAPs. Simplified models may be created using either hydrologic or hydraulic models such as HEC-HMS or HEC-RAS.

The simplified approaches described here are appropriate when:

- The dam is less that 25ft in height
- The resulting map is being used for the purpose of including it in an EAP and not when being used to prove the hazard-potential of a dam

*Dam breach inundation maps created for the purpose of classifying or disputing a dam's hazard-potential level using a simplified approach will not be accepted.

Simplified Inundation Mapping Using HEC-HMS

HEC-HMS requires input data within specified model components. The first basin input component is a Basin Model, which is comprised of a watershed basin or sub-basins, reservoir, dam and spillway features, and downstream channel data for flood routing. The second component is a Meteorological Model that defines precipitation that falls within the watershed. Dam breach models that are not triggered by a storm event will still include a Meteorological Model because it is required by the HEC-HMS software but no precipitation data will be entered. The program also requires a Control Specifications component to specify run time length and time intervals. The final components of the program include Time-Series data, such as specified hydrographs and precipitation hydrographs, and Paired Data, which is used to define reservoir storage-area-elevation relationships, spillway discharge-storage-elevation data, or downstream channel cross sections for flood routing.

Downstream Channel

The breach flood wave will be routed along the downstream channel. In order to do this in HEC-HMS, channel data is input in the Basin Model as a routing component. There are several routing methods available in HEC-HMS, but the most common used for natural channels is the Muskingum-Cunge method. The reason for this is two-fold: first, the Muskingum-Cunge method estimates attenuation of the breach flood peak discharge by recalculating the routing parameter at every time step of the computation process; and second, the method uses an eight-point cross section to approximate a natural channel and overbank cross section. All other routing methods in HEC-HMS use a standard shape cross section. Each Muskingum-Cunge cross section represents a relatively homogeneous reach along the channel. Each of the eight points of the inserted cross section are identified by a station and an elevation. Manning's *n* friction coefficient values are assigned to the channel and overbanks. Length values represent the homogeneous reach length. Other routing methods can be used if the channel characteristics can be reasonably estimated by a standard shape.

Breach Characteristics and Starting Conditions

The outflow hydrograph from the dam breach will be determined by the size of the breach and the time it takes to fully develop the breach. HEC-HMS does not simulate the breach by physical erosion estimation computations. Instead it uses breach parameters input by the user. Breach parameters are calculated outside the program by published empirical relationships taken from historical earthen dam failures. Progression of the breach is a factor of the reservoir hydrostatic head on the dam and the volume of water impounded. Breach flow will be dependent on the starting reservoir level at the beginning of breach simulation. For the simplified approach, the reservoir level at the beginning of the computation is at the top of the dam crest.

Simplified Inundation Mapping Using HEC-RAS

Another simplified breach modeling approach is to use HEC-RAS under a steady state assumption, using the peak breach flow as estimated by Froehlich's equation, which is written as follows.

$$Q_p = 0.607(V_w^{0.295} \cdot H_w^{1.24})$$

Where:

Q_p is the estimated peak breach flow (m_3/s)

V_w is the volume of water behind the dam at the time of breach (m^3)

H_w is the height of the water behind the dam from the streambed (m)

In this case, the reservoir should be modeled by setting the upstream cross sections, which represent the reservoir, to the maximum water surface elevation of the reservoir. The flood extents are then estimated as the water surface profile obtained from a steady flow analysis using the peak breach flow. The downstream control will normally be set to the friction slope, but other boundary conditions may be appropriate. This should be decided by the user on a case-by-case basis.

It should be noted that only the equations for estimating the peak breach flow value are applicable when using HEC-RAS under the steady-state assumption. Since there is no time dependence included in the analysis, the time of breach formation and the flood wave travel time are not included.

9. Report and Supporting Documentation

Rainfall – Runoff Information

- Characteristics for the entire watershed and all sub-basins, as applicable to calculation methods
- Data used to develop parameters describing the watershed characteristics, including any available calibration data
- Design flood inflow and discharge hydrographs

- Reservoir routing data and parameters
- Discharge-frequency relationships
- Hydraulic roughness determinations
- Water surface profiles

Dam and Spillway Information

- Spillway Stage- Discharge relationships
- Maximum height and reservoir storage values
- Elevation – Area – Storage Relationship
- Key operational elevations for the dam and spillway
- Pertinent spillway dimensions

Breach Analysis Information

- Breach parameters (location, configuration, formation time)
- Model boundary conditions
- Profile of peak flood levels
- Profile of warning time versus distance downstream
- Delineation on the best available mapping base of the extent of inundation for the normal pool and design flood breach events for the project
- Identification of any potential loss of public services and of critical facilities
- Assessment of hazard potential classification

Hydrological and Hydraulic Models

- Electronic copies of the final simulation models used in the analysis

Breach Analysis Report

The dam breach inundation report must provide a description of the type of model used, any assumptions used in the model, description of the data used for the breach analysis and breach flow routing. The antecedent flow conditions on which the maps are based should be identified. Also included shall be information on the peak discharge at the dam site, the times of arrival of the flood wave (in hours or minutes), peak discharge rates, peak elevations, and duration of the breach flood flows at significant points along the flood path including bridges or major culverts, the first inhabited structures the flood wave would encounter, and at representative cross-sections along the inundation path. In addition, electronic copies of the model simulations shall also be submitted for review purposes. If the HEC-RAS and/or HEC-HMS models are used, it is not necessary to include the output files, only the files that are required to run the simulation.

A narrative description of the areas affected by the dam break should be included to clarify unusual conditions. It should describe the specific area threatened and include information on the size and depth of expected flooding relative to known landmarks and historical flood heights.

Also included is a list of the numbers and types of habitable structures located within the breach inundation area and an estimate of the population at risk should be included in report, as well as the Appendix of the EAP. This list should be referenced, where practical, to the inundation map.

The accuracy and limitation of the information supplied on the inundation maps and how best to use the maps should be described. Because local officials are likely to use the maps for evacuation purposes, a note should be included on the map to advise that because of the method, procedures, and assumptions used to develop the flooded areas, the limits of flooding shown and flood wave travel times are approximate and should be used only as a guideline for establishing evacuation zones. Actual areas inundated will depend on nature of the actual dam failure or flooding conditions and may differ from areas shown on the maps. The owner should review the inundation maps with the local jurisdictions and resolve any problems.

If inundation maps are to be shown on several pages, a map index should be included to orient the individual pages. Inundation maps and reports may be submitted electronically in pdf format.

Inundation maps should be reviewed every five years and updated to reflect changes in downstream areas. Include any other pertinent information as a result of coordination with the appropriate emergency management authorities. Emergency management agencies may request that inundation maps highlight evacuation routes and emergency shelters.

10. Dam Breach Inundation Map Requirements

In the breach analysis report the dam breach inundation maps shall show the areal extent of flooding which would be produced by the dam break flood for each scenario. However, inundation mapping to be used in the EAP should only include the outline of the ‘sunny-day’ breach for the simplified engineering analysis or the percent PMF breach for the detailed engineering analysis. Non-breach runs should not be shown on the EAP maps.

Location of representative cross-sections used in the routing modeling below the dam shall be taken at no more than 1,320 foot intervals and perpendicular to the direction of flow and contour lines. Each cross-section used in the model should be shown on the inundation map and properly labeled to identify the cross-section. Dam breach flood routing shall continue downstream to the point where depth of the dam breach flood water is one foot above the spillway design flood water elevation if the dam had not breached and is not infringing on the minimum freeboard requirement.

Should the dam breach inundation area encounter another dam down-stream, then the inundation boundary should be drawn to represent an increase in the water level of the lake or reservoir. Should this increased water level overtop the dam, the appropriate inundation lines should be drawn downstream of the dam to represent expected inundation to include the failure of the downstream dam.

Inundation maps shall be made on the best available mapping and presented in sequence of 11”x17” maps for ease of inclusion in the EAP. Scales should be used such that structures and major infrastructures such as major streets and roads, railroads, buildings, residences, and other well known features with labels indentifying significant features. Aerial photographs, if of reasonable clarity and scale, are preferred as background for inundation maps. Topographic contours at 10 foot intervals should be included.

The lines of the dam breach inundation boundaries should be drawn with a thickness to be easily recognizable so as not to be confused with other lines on the map, but not of such thickness to obscure houses or other important features. If the inundation area is shaded or cross-hatched they should be done in such as manner as not to obscure features within the inundation area.

Electronic copies of the inundation boundaries are also to be included with the report. These should be in the form of ESRI ArcGIS shapefiles. If the inundation boundaries are presented through AutoCAD, the .dxf or .dwg file extensions should be submitted all with the report.

Tables shall be included with the map showing time of arrive of the initial flood wave following initiation of the dam breach. In addition peak flood depths or elevations should be included. As an alternative the times and peak flood depths can be labeled directly on the map. Initial flood wave times and peak flood depths should be shown at occasional intervals and at critical structures.

Also, a qualifying statement should be included with the inundation map saying that the map is an estimation of the flood extents and actual dam breach flood conditions may differ from those shown. The FERC recommends the following statement to be included with the dam breach inundation map:

“…the methods, procedures and assumptions used to develop the flooded areas, the limits of flooding shown and flood wave travel times are approximate and should only be used as a guideline for establishing evacuation zones. Actual areas inundated will depend on actual failure of flood conditions and may differ from areas shown on the maps...” (FERC 2007)

11. Limitations

Through the course of development of these guidelines, every effort was made to make the recommendations as useable as possible over a broad range of conditions. Each dam is unique, and individual cases may require different modeling approaches than those recommended herein. Different modeling approaches may be appropriate on a case-by-case basis with proper justification provided. Due to unknowns within these analyses and assumptions that must be made, engineering judgment must be applied to the results of all dam breach analyses. In most cases a sensitivity analysis will be required to verify that the assumptions used provide conservative, yet realistic results.