CHAPTER D-3 FLOOD OVERTOPPING FAILURE OF DAMS AND LEVEES

Overtopping flow is a component event of many or even most potential failure modes resulting from floods. Dams and levees have been overtopped by a few inches to more than a foot without breaching, but other structures have failed quickly. Overtopping is a failure mode of concern since Costa (1985) reported that of all dam failures as of 1985, 34% were caused by overtopping, 30% due to foundation defects, 28% from piping and seepage, and 8% from other modes of failure. Costa (1985) also reports that for earth/embankment dams only, 35% have failed due to overtopping, 38% from piping and seepage, 21% from foundation defects; and 6% from other failure modes.

D-3.1 Key Concepts and Factors Affecting Risk

D-3.1.1 Type of Dam or Levee

Materials for dams and levees range from earthen and/or rockfill embankments to various types of concrete dams. Embankment dams typically cannot withstand any significant amount of overtopping, due to limited erosion resistance of the soil material used in their construction. The amount of erosion is dependent on the quality and type of vegetation cover, material in the embankment, depth, and duration of the overtopping flow.

Concrete dams are generally perceived to be more resistant to overtopping failure, due to the durability of the dam itself as well as the erosion resistance provided by a rock foundation. However, weak and fractured rock may be susceptible to significant erosion during overtopping flows, and if foundation support is lost due to overtopping erosion, the dam could be lost.

Levees typically are earthen embankments constructed from a variety of materials ranging from cohesive to cohesionless soils. The factors influencing the erosion are similar to those for earthen dam embankments.

For floodwalls the factors influencing the overtopping are similar to concrete dams but a much smaller scale in terms of head for wall stability, underseepage and energy of overtopping flows.

D-3.1.2 Types of Overtopping

Dams and levees can be overtopped with a continuous flow when the pool elevation or river elevation exceeds the low portion of the dam or levee. For these cases the computation of the depth and duration of flow can be relatively easy depending on the information available for the specific project.

For overtopping by waves, the water surface elevation approaches but does not exceed the low point in the elevation profile. Instead waves driven by wind produce waves that run-up and overtop the top of dam or levee. The wave action can form an "equivalent" discharge per liner foot of the structure and can lead to the erosion and potential failure of the structure. Waves are influenced by wind speed, wind direction, bathymetry, open water distance, and embankment slopes. (USACE, 2002)

D-3.1.3 Erosion Process

The erosion process is described in the chapter for Erosion of Rock and Soil but items specific to embankments will be included here. In general, the most erosive flow occurs on the downstream slope, where the velocity is highest and where the slope makes it easier to dislodge particles and move them away. On embankments that have been overtopped by floods, severe erosion has often been observed to begin where sheet flow on the slope meets an obstacle, such as a structure, a large tree, or the groin; a break in slope occurs; a change in material type, or vegetation is not uniform or soil is bare creating local turbulent flow. Based on the four phase erosion process in the chapter for Erosion of Rock and Soil, areas where vegetation has been removed or sparse, the erosion will proceed to attack the soil directly until a "headcut" or overfall is formatted. Erosion generally continues in the form of "headcutting," in an upstream progression of deep eroded channel(s) that can eventually reach the reservoir. For embankments made from cohesionless material a headcut may form or concentrated flow will erode a gully more uniformly.

In the case of an embankment dam, erosion of the soil comprising the embankment can ultimately lead to dam failure. For cohesive soils, the failure mechanism is typically headcut initiation and advance. A small headcut is typically formed on the downstream slope of the dam and then advances upstream until the crest of the dam is breached. For cohesionless soils, the failure process typically initiates as a result of tractive stresses from the flow removing material from the downstream face, but then progresses as headcut advance once a surface irregularity is formed. Predicting whether breach initiation and formation will occur can be a complicated procedure.

Pavement on the crest may be of some value in slowing uniform erosion of cohesionless materials once the gullies reach the crest, but should not be expected to affect initiation. Depending on the depth of the headcut, the headcut can actually undermine pavement leading to a mass wasting of the pavement material as cantilevered section collapse into the headcut.

In the case of a concrete dam, the erosion resistance of the foundation rock is typically the key to the likelihood of failure. The likelihood of rock erosion can be estimated using the methods described in the section on Erosion of Rock and Soil. If various weathering horizons or rock types exist in the abutment or foundation, the evaluation will need to be done for each. If significant depth of erosion is needed for undermining, it may be necessary to re-compute the erosion potential for various depths of erosion to obtain an indication of how deep the erosion is likely to go. If significant abutment erosion occurs, support for the dam may be compromised. It would be necessary to evaluate the potential for enough erosion to occur such that support for the dam would be lost for each pool loading.

If a parapet wall is provided on the embankment dam crest across the entire length of the dam, dam overtopping will initiate when the reservoir water surface exceeds the elevation of the top of the parapet wall. Parapet walls are typically designed to contain waves that might overtop the dam and may need to be evaluated for a sustained water load (considering instability of the wall and blowout at the toe of the wall for loads part way up on the wall). If a parapet wall overtops, the impinging jet from overtopping flows may erode the dam crest and undermine the parapet wall. If the parapet wall or a section of the wall fails, the depth of flows overtopping the dam crest will be significant and breach may occur quickly.

D-3.2 Dam Overtopping

D-3.2.1 Flood Frequency

Flood frequency is an important factor in the risk from overtopping and dam failure. The procedures for determining the frequency is in the chapter on Hydrologic Hazard. Items that may influence the frequency are spillway discharge capacity, debris blockage, and spillway and gate configuration. Determining the impacts from these factors will typically require multiple routings of inflowing hydrographs to determine their potential impacts on the pool elevation and ultimately the overtopping depth and duration.

D-3.2.2 Spillway Discharge Capacity

Spillway discharge capacity is usually determined based on the Inflow Design Flood (IDF) and determined in conjunction with routing of the inflow flood hydrograph through the reservoir based on the operations outlined in the Water Control Manual (USACE) or Standing Operating Procedures (Reclamation). When the reservoir has significant volume, the spillway capacity may be significantly less than the peak inflow discharge. When the reservoir has minimal storage volume, the spillway capacity may equal the peak inflow discharge. Variations on this occur when the dam is designed to pass the IDF using outlets works and/or hydropower units. In cases where the outlet works or hydropower units are critical to safely pass the IDF; these features need to be closely examined. For example, if overtopping would take out a switchyard or the power is not needed, the release capacity of the turbines would likely be lost at that point. If the outlet works were not designed to safely pass their contribution, their use may cause embankment or outlet damage and/or contribute to other failure modes.

For High Hazard Potential dams, when the dam safely passes the PMF and the PMF meets current guidance, overtopping is usually not an issue as explained in the chapter Hydrologic Hazard. If the PMF overtops the dam, the dam would be subject to erosion of the foundation and/or embankment. If the PMF approaches close to the top of the dam (typically three feet for embankment), the dam may be subject to erosion from overtopping from waves.

D-3.2.3 Spillway and Gate Configuration

The spillway configuration can affect the reliability and the ultimate discharge capacity of a spillway. Uncontrolled, overflow spillways are generally reliable with predictable discharges.

Gated spillways can have inherent reliability concerns, due to the potential for mechanical and power failures, and the potential for operations to differ from planned operations as a result of the inability of an operator to access the gate controls or an operator decision to delay opening the gates due to downstream flooding concerns. Fuseplug spillways may have some inherent uncertainty regarding when they will operate. For dams where the IDF has significantly increased and the spillway is gated, the new spillway flow may impact the gates and significantly reduce the capacity as the flow will switch from weir flow to orifice flow or impact access for gate operations.

D-3.2.4 Potential for Reservoir Debris to Block Spillway

If the full capacity of the spillway is not available, dam overtopping can occur under more frequent floods. Some watersheds produce large amounts of debris during rainstorms. Sturdy log booms may be able to capture the debris before it reaches the spillway, but if not, the debris may clog the spillway opening. As a rule of thumb, spillway bays with a clear distance less than 40 feet (less than 60 feet in the Pacific Northwest) are vulnerable to debris plugging. If a spillway is gated and the gates are being operated under orifice conditions or if the bottom of the raised gate is less than 5 feet above the flow surface the spillway openings will be further restricted, compounding the potential for debris blockage. References on debris potential in reservoirs are provided by the Federal Highway Administration (2005) and Wallerstein, Thorne and Abt (1997).

D-3.2.5 Depth and Duration of Overtopping

The depth and duration of overtopping and the erodibility of the embankment materials are the key parameters to determine the likelihood that dam failure will occur as a result of overtopping. The estimated probability of an embankment dam failure due to overtopping will be site specific and will also be a function of the zoning and details of the dam. Heavily armored downstream slopes and highly plastic embankment materials are more erosion resistant

Figure D-3-1 shows the progression of dam failure that initiates at the toe of the dam. Once erosion initiates at the toe of the dam (WinDam B, as discussed in Erosion of Rock and Soil, initiates the headcut near the crest to be conservative), a headcut forms at the toe and then advances upstream until the crest of the dam is breached (Wahl, 1998). Note that the breach does not initiate until the upstream crest begins to erode.

The likelihood of concrete dam failure for a given overtopping depth and duration is primarily a function of the erosion resistance of the abutment and foundation rock. The ability to accurately predict the allowable threshold for depth and duration of overtopping is still limited, but there are tools in the Erosion of Soil and Rock to assist with these estimates.

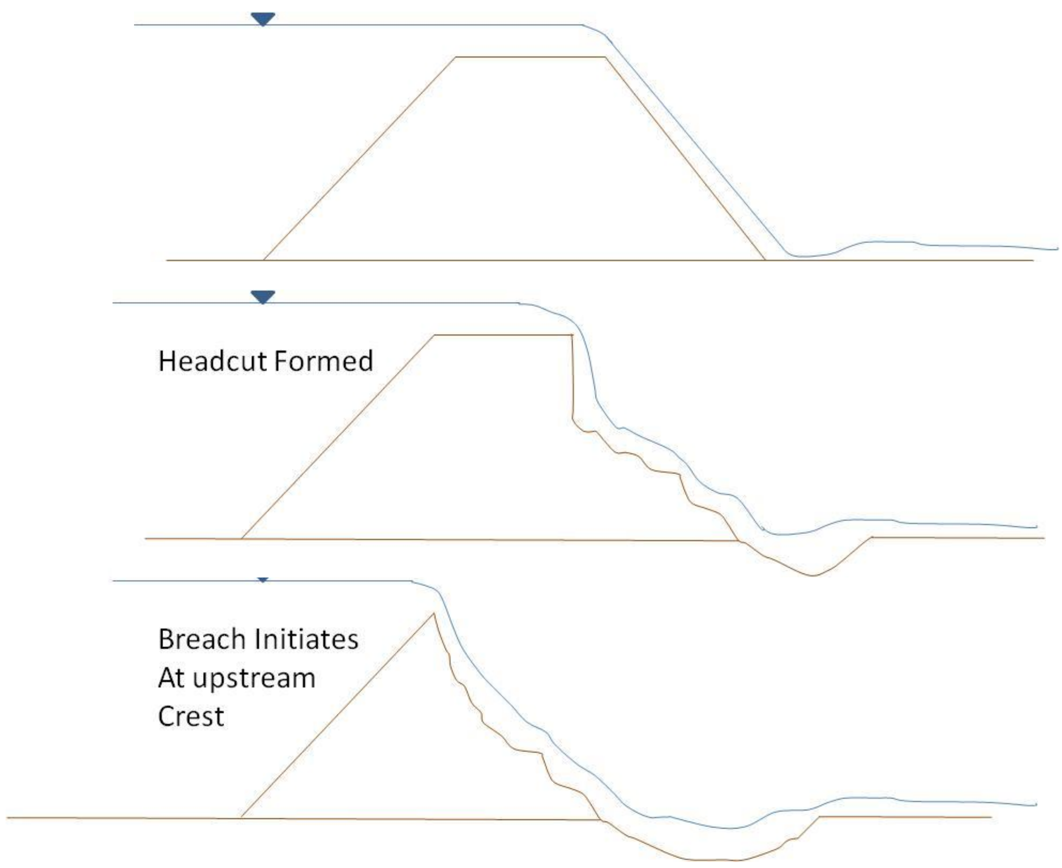


Figure D-3-1 Dam Overtopping Failure Progression for Embankment Dam

D-3.2.6 Top of Dam Profile

Some embankment dams were built with camber, meaning the portion of the dam near the maximum section was built higher than at the abutments, to allow the embankment to settle after construction without the crest dropping below the design crest elevation. However, in most cases, the embankment settlement has been less than the camber, so the embankment crest is still lower at the abutments. Embankment dam crests may also have low spots, due to localized settlement, which are areas where overtopping will initiate and flow concentrations may occur. Actual profile surveys of the embankment crest should be used when estimating the overtopping flow for embankments and where overtopping will initiate. These surveys should be used to determine the minimum elevation of the top of dam and this elevation should be used in lieu of the "design" top of dam elevation.

D-3.2.7 Wave Overtopping

When the water surface elevation is below the top of dam elevation, wave overtopping of the embankment may be a concern along coastal areas, larger lakes, etc. Typically a significant surface area would need to be present to allow winds to develop waves that would be directed towards the embankment and overtop it. For wave overtopping the wind and wave direction, embankment slope, and the local bathometry are critical components for determining how the wave runs up the levee leading to overtopping. While there is currently no rigorous method for evaluating overtopping failure due to wave action, it would require erosive embankment materials and a long duration of waves overtopping of the dam.

The Coastal Engineering Manual (USACE 2002) describes ways to calculate setup (increase in water surface from wind/water friction) and runup for various geometries and calculate an "equivalent" or "average" overtopping discharge per unit width. The estimate of the average overtopping discharge is strongly influenced by the distance between the still water level and the top of the embankment. Using estimated average overtopping discharges, (Table VI-5-6 CEM, 2002) the likelihood of erosion and failure can be estimated from existing allowable guidance. Note the data is for specific sea dikes and results will vary with variations in material and vegetation cover.

Subsequent studies have indicated that erosion from wave overtopping cannot always be described by the overtopping discharge. In fact larger volume, less frequent waves tend to cause more erosion than smaller volume, more frequent waves when both have the same calculated overtopping discharge (Van der Meer, 2010).

D-3.3 Levee Overtopping

Levees are subject to failure when overtopped similar to dams. Levees have potentially unique flow characteristics as the flow for overtopping may not always be perpendicular to the levee as it is with dams. Until this hydraulic difference is better understood the current best practice is to estimate the failure probability using similar techniques as that for dam overtopping and then make adjustments using judgment.

D-3.3.1 Channel Capacity and Water Surface Profiles

Channel conveyance is important to the levee to safely pass the flood event it was intended. Many levees were designed with freeboard to account for the uncertainty in estimating the discharge frequency curve and the water surface profile associated with the design discharge for the project. In risk analysis, the freeboard with be replaced with the likelihood of passing certain discharge events.

Changes in channel roughness, addition of structures such as bridges, outdated modeling practices, geomorphic changes, debris blockage, other encroachments, etc. may significantly alter the channel capacity since the levee was originally designed and constructed. Each of these changes may lead to overtopping of the structure from a discharge less than the design discharge. These changes need to be identified and incorporated into the current water surface model to determine the potential impacts to the levee capacity.

These modifications to the levee may be localized, or persistent throughout the reach, have the potential to change the slope of the water surface profile through the project. Changes in water surface profile may be a result of different modeling techniques, the type of model used, frequency of cross sections, etc. Such changes may lead to higher water surface in localized areas making the levee more prone to overtopping or lead to the levee overtopping in an area other than originally designed or thought to initially overtop.

D-3.3.2 Levee Top Elevation

The levee top elevation or levee profile is constructed with construction tolerances. If these tolerances were not followed, the levee profile may deviate from the original design profile. Over time the levee profile may also change from settlement of the levee foundation, levee embankment, and in some areas from subsidence. Vehicular traffic and grade maintenance may also lead to changes in the levee profile in localized areas or along the entire levee profile. These changes may lead to the levee overtopping in an area other than originally designed or thought to initially overtop.

D-3.3.3 Wave Overtopping

When the water surface elevation is below the levee top elevation, wave overtopping of levees may be a concern along coastal areas, lakes, wide inundation areas, etc. Typically a significant surface area would need to be present to allow winds to develop waves that would be directed towards the levee and overtop it. For wave overtopping typically the wind and wave direction and levee slope and the local bathometry is a critical component for determining how the wave runs up the levee leading to overtopping. Methods similar to those described earlier for dams can be used in estimating wave overtopping for levees.

D-3.4 Potential Failure Mode Evaluation

An initial conservative assumption should be made that breach is initiated with any overtopping of an embankment dam or levee. If this shows that risks are above agency risk guidelines, or if allowing a small depth of overtopping could change the conclusions for this failure mode, a more refined approach can be considered (see the section on Erosion of Rock and Soil). If refinement is needed a risk analysis team may consider developing fragility curves to relate the depth of overtopping to the probability of dam failure due to erosion and breach of the dam crest. If the team elects to do this, careful consideration should be given to the development of the fragility curve. For a given depth of overtopping, a range of failure probabilities and a best estimate should be developed. The following items should be considered in the development of the fragility curves:

- Depth of overtopping
- Duration of overtopping
- Potential concentration of overtopping flows at dam crest due to camber or low spots
- Potential concentration of overtopping flows on the dam face, along the groins or at the toe of the dam

- Erosional resistance of materials on the downstream face and in the downstream zones of the embankment
- Whether a parapet wall is provided and the potential for the wall to fail before or after it's overtopped
- WinDAM B or NWS-Breach may assist in determining erodibility. Empirical breach equations assume the embankment has breached, these programs will help determine a range of flow conditions that may or may not lead to full breach.

In the past, fragility curves have typically been developed through a combination of simplified analyses, judgment, and team consensus. The development of physically-based dam breach computer models such as WinDAM B (released in late 2011) makes it possible to develop fragility curves through a dedicated modeling effort. Even if such tools are used, it is still important to develop a range of failure probabilities and a best estimate, since the dam failure process is highly non-linear and sensitive to variable input parameters. Repeated WinDAM B simulations can be made using a range of input data representing uncertainty in hydrologic and geotechnical/erodibility parameters (Hanson et al. 2011). See chapter 15 for details about the modeling of potential dam breaches.

D-3.5 Event Tree

An example event tree described in this section is relatively simple, and is typical of what would be considered for an overtopping potential failure mode. Each branch consists of five events – the pool loading, vegetation or riprap removal, headcut initiation, headcut or erosion advancing, and finally breach.   In cases where debris plugging, gate failure, or wave overtopping is a concern, additional events can be added to account for the likelihood of these conditions developing. The erosion and breaching process can be subdivided into a sequence of necessary steps:

- Erosion of the surface of the downstream slope, which may consist of vegetation, riprap, or bare soil.
- Concentrated erosion on the downstream slope causing a deepening of the erosion channel until one or more headcuts are formed on the downstream slope (for

conservatism, physically-based dam breach models such as WinDAM B assume that a headcut is formed at the top of the slope / downstream edge of the dam crest; see chapter 15 Modeling Erosion of Rock and Soil for details).

- Advancement of headcuts upstream, usually accompanied by consolidation of multiple headcuts.
- When the most upstream headcut advances through the upstream edge of the dam crest, breach is initiated and the breach opening begins to enlarge. (After this point, intervention to save the dam is no longer possible).
- Headcuts continue to advance upstream, enlarging the breach and releasing reservoir storage.
- The breach widens as long as hydraulic stresses at the sides of the breach opening are sufficient to exceed the erosion threshold of the soil

For wave overwash, an additional node will be needed for the likelihood of waves forming sufficient height for erosion to initiate and of sufficient duration for the potential failure mode to lead to failure. When Reclamation evaluates this potential failure mode only one conditional event is typically considered beyond the loading (which considers starting reservoir water surface elevation and flood loadings for various return periods). A determination is made as to whether or not the dam will breach for the various loading combinations. An example event tree for this simplified approach is provided in Figure D-3-2.

In the case of a concrete dam, the erosion resistance of the foundation rock is typically the key to the likelihood of failure. The likelihood of rock erosion can be estimated using the methods described in the section on Erosion of Rock and Soil. If various weathering horizons or rock types exist in the abutment or foundation, the evaluation will need to be done for each. If significant depth of erosion is needed for undermining, it may be necessary to re-compute the erosion potential for various depths of erosion to obtain an indication of how deep the erosion is likely to go. If significant abutment erosion occurs, support for the dam may be compromised. An example event tree for overtopping of a concrete dam is shown in Figure D-3-3 at the end of this section. The event tree is shown for only one load range, but the complete tree should evaluate all load ranges.

D-3.5.1 Accounting for Uncertainty

The process of estimating risk for the overtopping potential failure mode should evaluate the uncertainties associated with the hydrologic loadings, the reservoir operations, gate or spillway operational failures, and the response of the dam. Additional flood routings should be considered that vary some of the key parameters to evaluate the sensitivity of the results to the assumptions. The results of these sensitivity routings may provide the basis for adjusting the risk estimates or for identifying more uncertainty with the risk estimates. Considerations specific to these uncertainties include, but are not limited to the following:

- Uncertainties Associated with the Flood Events
- Uncertainties Associated with Reservoir and/or Gate Operation
- Uncertainties Associated with Spillway Discharge

D-3.5.2 Uncertainties Associated with the Flood Events

Possible variations with frequency floods and/or the PMF exist. As an example, there were four 10,000-year frequency floods developed for corrective action studies which were based on historical events. The events differed in starting pool elevation (based on the time of year specific floods are likely to occur), the hydrograph for routing, etc. Only one of the four 10,000-year frequency floods resulted in overtopping of the dam. Such a result might lead to requesting additional frequency flood hydrographs to route for evaluating the likelihood of getting a 10,000-year hydrograph that overtops the dam (i.e., Monte Carlo simulations).

D-3.5.3 Uncertainties Associated with Reservoir and/or Gate Operation

The assumptions made regarding reservoir operations for flood routing studies should be evaluated for reasonableness. The Standing Operating Procedures (SOP; Reclamation) or Water Control Manual (USACE) for a given dam may require that the spillway gates be opened in direct response to increasing inflows, but if the gate openings dictated by this operation would exceed the safe channel capacity and flood homes and endanger downstream residents, there may be a reluctance to pursue an aggressive release schedule on the part of the dam operator. Building in a delay in making critical decisions on gate operations (such as the point where downstream populations are dramatically affected) into the flood routings is a way to test the sensitivity of the flood routing results to the flood operations.

Another potential issue with spillway gates is the potential for one or more of the gates to malfunction during a major flood. Gates can malfunction for a number of reasons including failure of the hoist mechanism, failure of the wire ropes or chains that lift the gates, binding of the gates due to pier deflections or expansions, power failure, or access limitations. This can be simulated by eliminating the discharge capacity of one of the gates during the flood routing to test the vulnerability of the operations to these types of situations.

Uncertainty regarding the erosion of embankment materials and foundation materials exposed to overtopping flows was mentioned previously in this chapter. The areas where erosion initiates, the potential for concentrated flows and the rates of erosion of soil and rock materials are all variables that need to be considered in the risk analysis.

D-3.5.4 Uncertainties Associated with Spillway Discharge

Spillway discharges assumed in flood routings are often based on idealized discharge curves. If the spillway discharge curve was not based on a site-specific hydraulic model study, and the approach conditions to the spillway are less than ideal, consideration should be given to the potential for reduced discharge. Another consideration is the potential for watershed debris to clog the spillway crest during a large flood and restrict spillway discharges. Sensitivity routings can be performed to evaluate these potential effects. For gated spillways, discharge conditions can vary from free flow to orifice flow depending on the gate opening and the reservoir water surface. Finally changes to the approach conditions may also impact flow conditions and flow capacity and should be considered. An example would be increasing the elevation of the approach channel into an ogee crest may change the weir coefficient for the spillway rating curve. These factors should be accounted for in the routings.

D-3.6 Case Histories

Case histories are summarized here for reference as to some types of overtopping failures. Considerable additional information would be required to apply these case histories to a risk assessment.

D-3.6.1 South Fork Dam (a.k.a. Johnstown Dam), Pennsylvania: 1889

The South Fork Dam, also known as Johnstown Dam, caused the famous "Johnstown Flood," one of the worst disasters in United States history. The dam was located in western Pennsylvania, about 70 miles east of Pittsburgh. The 72-foot high dam was an earthfill embankment, with the original construction completed in 1852. The dam failed in 1862, due to collapse of a stone culvert running underneath the dam. It was reconstructed from 1879 to 1881. Significant changes to the dam included the lowering of the dam crest by 2 feet and the construction of a bridge with wooden supports in the spillway inlet channel. Screens were attached to the spillway bridge supports to prevent fish from escaping the reservoir. The reconstructed dam failed on May 31, 1889, due to overtopping failure during a large flood. Over 2200 people were killed. Several factors contributed to the dam failure, including: 1) the lowering of the dam crest reduced surcharge capacity in the reservoir and correspondingly reduced the spillway capacity; 2) the bridge piers and the screens across the piers, in combination with debris that was caught on the screens reduced the spillway capacity; and 3) settlement of the dam resulted in lowering the dam crest at the maximum section by about 6 inches (Frank, 1988).

D-3.6.2 Secondary (saddle) Dam of Sella Zerbino: 1935

(www.molare.net) – The Secondary Dam of Sella Zerbino is one of two dams that were competed in 1925 to form a reservoir on the Orba River, in South Piedmont, Italy, near Liguria. The main dam is a 47-meter high gravity arch dam and the secondary dam was a 14-meter high concrete gravity dam. The secondary dam was added late in the design process to close off a low spot in the reservoir rim, when it was decided to increase the capacity of the reservoir. The secondary dam was designed and constructed quickly, without any geologic investigations. The foundation for the secondary dam consisted of highly faulted and fractured schistose rock. During initial filling, significant seepage was observed downstream of the dam. A large storm occurred in the drainage basin above the dams on August 13, 1935. It was reported that 363 mm of rain fell in the Orba basin in less than 8 hours, equating to about a 1000-year event. The inflow into the reservoir resulted in both dams being overtopped by about 2 meters. The Secondary Dam of Sella Zerbino failed as a result of the overtopping, resulting in over 100 fatalities.

D-3.6.3 Gibson Dam: 1964

Gibson Dam is a 199-foot-high concrete arch dam constructed by the Bureau of Reclamation on the Sun River on the east side of the Continental Divide in Montana. The dam was completed in 1929, and the spillway was modified in 1938. In June of 1964, a major flood developed in the area, producing 30-hour rainfall amounts from 8 to 16 inches. Overtopping of Gibson Dam began at 2:00 p.m. on June 8 and continued until 10:00 a.m. on June 9. High water marks indicated a maximum overtopping depth of 3.2 feet. The operators had left two of the spillway gates completely open, two partially open, and two completely closed. The access road was inundated by the overtopping flows, and personnel could not get to the spillway gate controls to operate them. However, even if all gates had been fully open, the dam would have overtopped. The dam survived the overtopping, with little damage to the limestone abutments (Anderson et al,1998).

In addition to the above case histories, the Operational Failures Section describes the Taum Sauk Dam failure case history, in which dam overtopping and failure resulted from operational failures.

D-3.7 Considerations for Routine Risk Assessments

Routine risk assessments consist of Comprehensive Facility Review (CFR) for Reclamation or Periodic Assessment (PA) for USACE. Typically, a screening-level study is performed for each CFR or PA, in which the ability to pass the PMF at a dam is evaluated. If the current PMF cannot be passed without overtopping the dam, the Senior Engineer is required to make further evaluations based on available information. Typically, a peak flow flood hazard curve is prepared by the hydrologists for the CFR or PA. The annual chance exceedance of the peak flow corresponding to maximum spillway capacity at the dam crest is determined. If the annual chance exceedance of this flow is less than 1/10,000 and when multiplied by the consequences is less than agency risk guidelines, then the risk objectives are met. In this case, the failure probability is assumed to be 1.0 as soon as overtopping initiates.

If the risk is close to agency risk guidelines or if there are conditions where the probability of failure becomes nearly certain at an elevation above or below the dam crest elevation, a more detailed evaluation may be warranted. An example (Table D-3-1, at the end of this section) summarizes how this evaluation is made. Spillway discharges are calculated for key reservoir water surface elevations, encompassing the range where an overtopping failure can occur, including elevations (with remaining freeboard) where wave overtopping failure from wind action is possible. A flood frequency curve is then used to estimate return periods of floods with peaks equal to the spillway discharges. This is based on the conservative assumption that equates maximum discharge to peak inflow, which discounts the effect of reservoir storage in helping to pass the flood. When loss of life estimates for dam overtopping failure (see the section on Consequences of Dam Failure) are multiplied by the annualized failure probability, annualized life loss estimates are obtained.

Conditional failure probabilities are likely judgmental during a CFR or PA. If the risks determined by comparing spillway discharge capacity to flood peaks are high and there is significant flood surcharge space in the reservoir, then additional methods to develop the pool frequency curve or discharge frequency are probably needed to fully evaluate the risks. Flood frequency hydrographs are usually developed that include floods in the critical range where an overtopping failure is possible. A flood routing study is then initiated with these floods, in which the maximum reservoir water surface and durations of overtopping (if applicable) are determined for each flood. Based on the routing results, flood return periods can be determined for various reservoir water surface elevations, usually based on peak inflow annual chance exceedances. This information can be used to estimate risks for the overtopping potential failure mode, similar the previous approach described, but this time based on flood routing information. Table D-3-2 (at the end of this section) summarizes how this evaluation is made, for a case where the starting reservoir elevation did not make much difference, and was therefore taken to be the top of active conservation for all routings.

D-3.8 Dam Exercise

Consider a 90 foot high embankment dam with the dam crest at elevation 480.5. The crest of the dam is surfaced with gravel and recent surveys indicate that the crest elevation is uniform, with no low spots or depressions along the crest. The downstream shell of the dam consists of a well graded mix of compacted sand and gravel. The critical floods for the dam are spring rain-on-snow events, which have long durations. The reservoir water surface typically varies between elevations 440 and 466 during flood season each year. At this time of year, historical reservoir water surface elevations indicate that the reservoir is above elevations 440, 450 and 466, 90 percent, 30 percent and 10 percent of the time, respectively. Frequency floods for the dam were developed and a flood routing study produced the results in Table D-3-3. Additional analysis has shown the embankment is highly likely to fail from 1 foot or more of overtopping given the duration of overtopping and type of embankment material. Estimate the expected value annual dam failure probability due to overtopping.

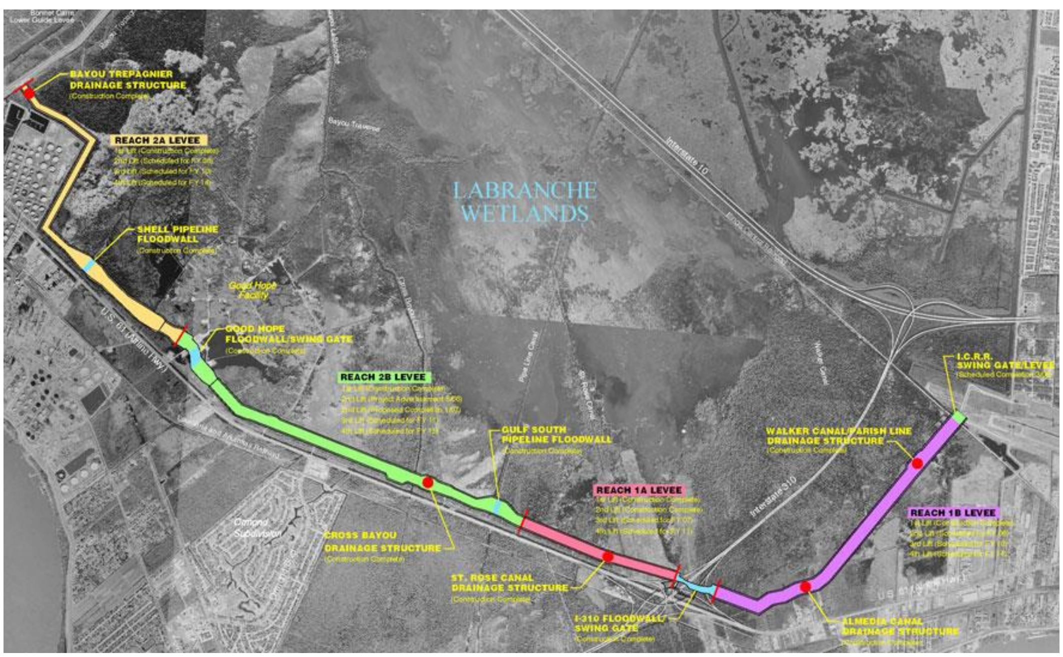
Table D-3-1 Flood Routing Results, Maximum Water Surface Elevation (feet)

Starting Reservoir Water Surface Elevation, feet	Flood Return Period, years			
	5000	10,000	50,000	100,000
466	468.2	475.1	480.9	484.0
450	467.4	473.4	480.0	482.3
440	466.0	471.2	475.6	479.7

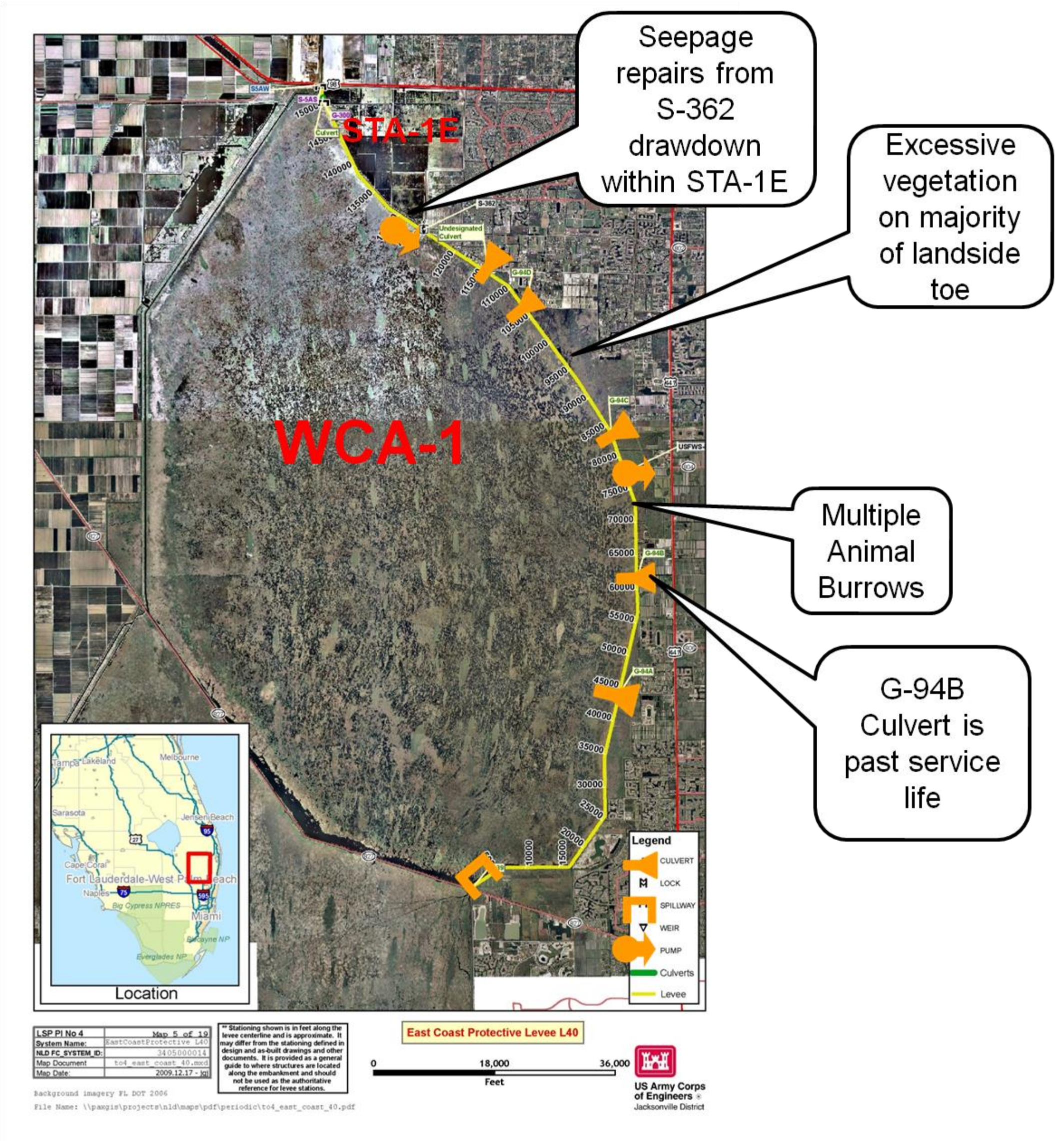
D-3.9 Levee Exercise

List factors that affect the overtopping erosion for a riverine levee, a hurricane (coastal) levee, a water conservation area levee (Florida):

Riverine levee crest near transition to floodwall , which is visible below bridge. The river under normal flow conditions is located about 300 feet to the right of the embankment.



Coastal Levee along Lake Pontchartrain



Annotated satellite image of water conservation area levee (yellow line shows levee alignment) is southern Florida.

D-3.10 References

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Table D-3-2 Summary of Risk Estimates for Dam Overtopping

Evaluation Based on Comparison of Spillway Discharge Capacity to Flood Frequency Peaks							
Reservoir Water Surface El Range, ft	Spillway Discharge Capacity, ft^3/s	Corresponding Frequency Flood, year	Probability of Flood Range	Freeboard (+) Overtopping (-) Depth, ft	Estimated Probability of Failure	Annual Probability of Failure	Annualized Loss of Life[1]
740 – 749	0 – 7400	100–10,000	.0099	9 to 2	0	0	0
749 – 750	7400 – 8670	10,000–50,000	.00008	2 to 1	0 to 0.1	4 E-06	4 E-04
750 – 751	8670 – 10,000	50,000–100,000	.00001	1 to 0	0.1 to 0.3	2 E-06	2 E-04
751 – 752	10,000 – 11,390	100,000-120,000	.00000167	0 to -1	0.3 to 0.999	1 E-06	1 E-04
> 752	11,390 – 12,848	> 120,000	.00000833	> -1	1	8 E-06	8 E-04
Totals						1.5 E-05	1.5 E-03

Table D-3-3 Summary of Risk Estimates for Dam Overtopping

Evaluation Based on Flood Routing Results of Frequency Floods							
Reservoir Water Surface El Range, ft	Corresponding Frequency Flood from Flood Routings	Spillway Discharge Capacity, ft^3/s	Probability of Flood Range	Freeboard (+) Overtopping (-) Depth, ft	Estimated Probability of Failure	Annual Probability of Failure	Annualized Loss of Life[1]
740 – 749	200-50,000	0 – 7400	.00498	9 to 2	0	0	0
749 – 750	50,000-300,000	7400 – 8670	.0000167	2 to 1	0 to 0.1	8 E-07	8 E-05
750 – 751	300,000-700,000	8670 – 10,000	.0000019	1 to 0	0.1 to 0.3	4 E-07	4 E-05
751 – 752	700,000-900,000	10,000 – 11,390	.00000032	0 to -1	0.3 to 0.999	2 E-07	2 E-05
752 – 753	> 900,000	11,390 – 12,848	.0000011	> -1	1	1 E-06	1 E-04
Totals						2.4 E-06	2.4 E-04

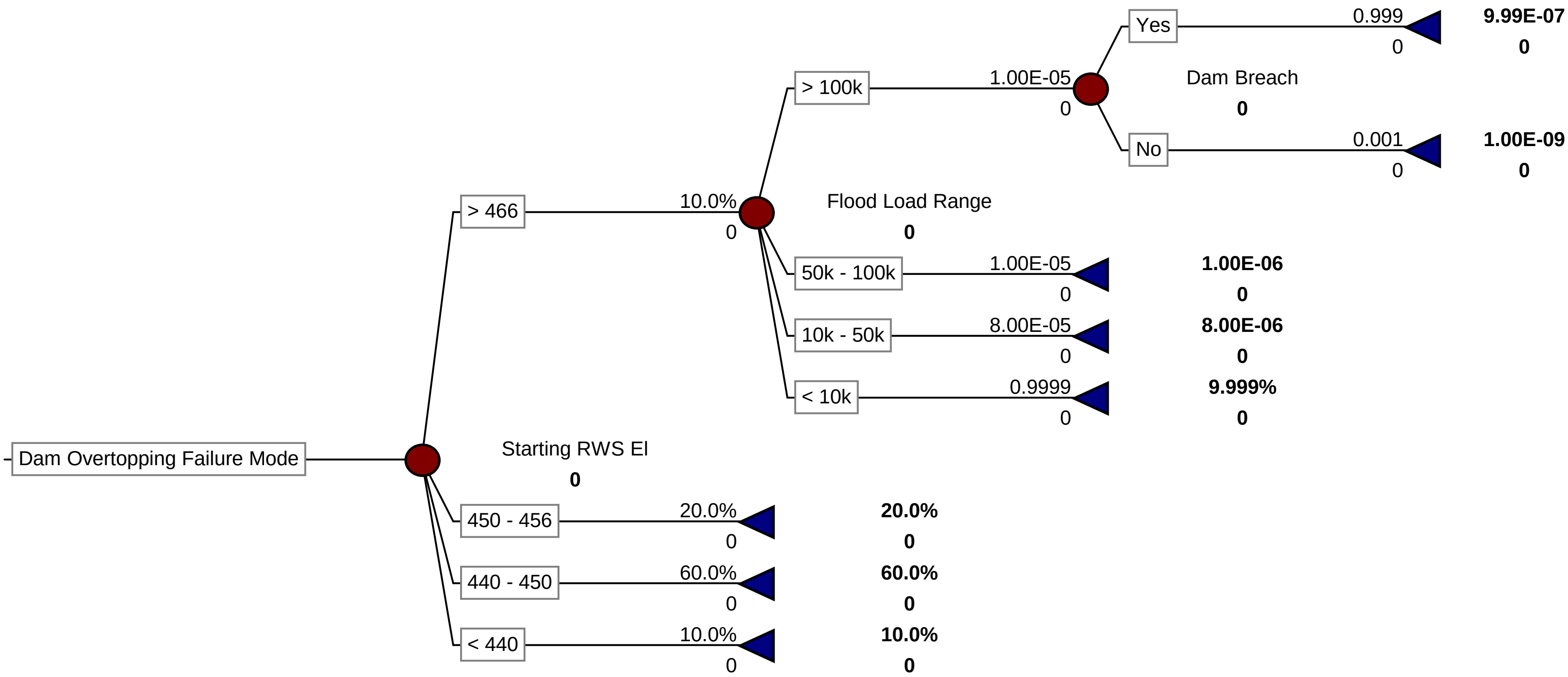

Figure D-3-2 Example Event Tree for Flood Overtopping Failure of an Embankment Dam

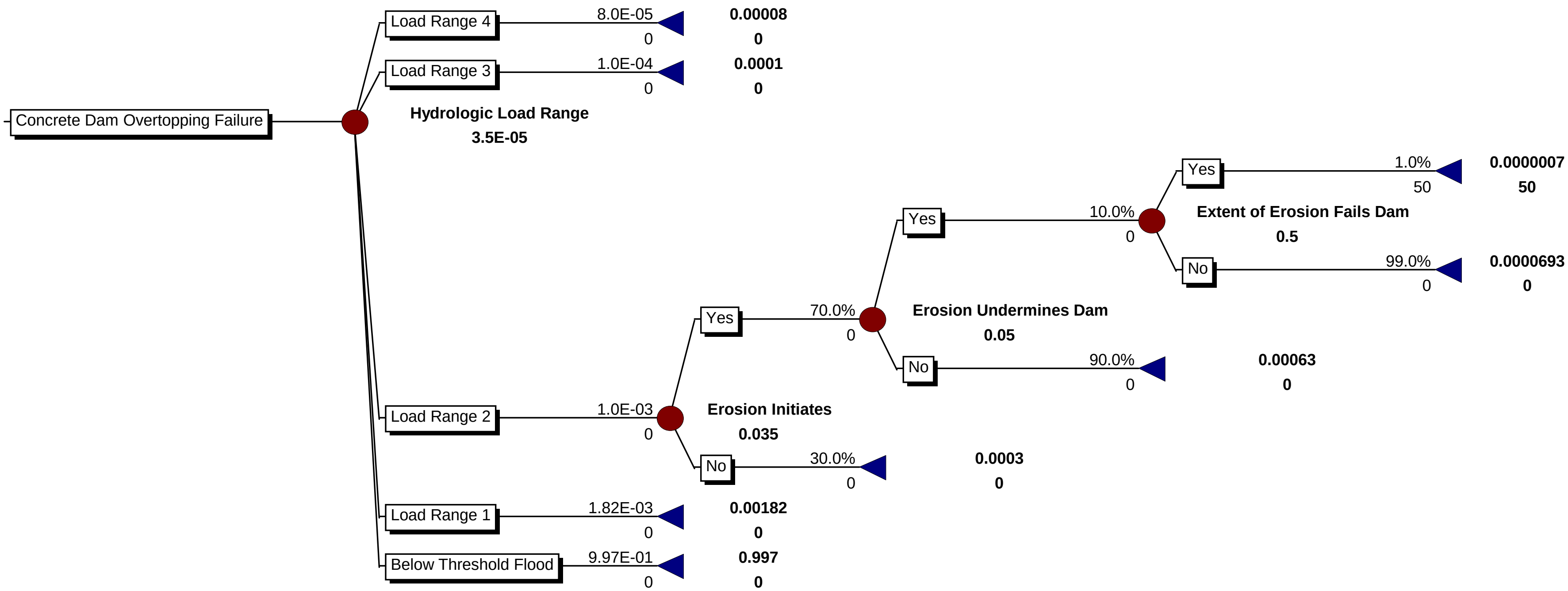

Figure D-3-3 Example Event Tree for Flood Overtopping Failure of a Concrete Dam