VI-2. Overtopping of Walls and Stilling Basin Failure

Key Concepts and Factors Affecting Risk

Description of Potential Failure Mode

Spillway structures often rely on a concrete chute to safely convey spillway releases from the crest structure to an energy dissipation structure near the river channel. The spillway chute forms a rectangular or sometimes trapezoidal open channel. Spillway chute walls were typically sized for the flow depths that would occur during the design spillway discharge, plus some freeboard to accommodate variations in flow depths due to air bulking (air entrainment) and cross-waves. If the spillway chute is subjected to discharges larger than the design discharge or air bulking or cross waves were not incorporated properly into the design, flow depths in the chute will increase and the walls may overtop. Overtopping flows will likely initiate erosion in the wall backfill as in Figure VI-2-1 which has the potential to progress to the point of undermining the spillway chute slab and failing the invert of the spillway. Once this occurs, headcutting can initiate and progress upstream, ultimately leading to a breach of the reservoir. Overtopping of stilling basin walls can also lead to erosion of backfill behind stilling basin walls and progression of the failure to the point of reservoir breach, but the tailwater surrounding the basin will typically dissipate energy from overtopping flows and lessen the potential for erosion. This section also addresses ball-milling and stilling basin sweepout failure modes, which initiate from different mechanisms but can also lead to chute undermining, headcutting and ultimately breach of the reservoir.

Figure VI-2-1.Wall Overtopping at El Guapo Dam, Venezuela

Spillway Design Discharge

The discharge that the spillway was designed for will determine the flow capacity of the spillway chute and stilling basin. If current flood loadings indicate that the spillway design discharge will be exceeded for some flood events, then the flow depths in the spillway chute and stilling basin will increase and wall overtopping becomes more likely for those floods. Whether the walls actually overtop during a given flood will then be influenced by the freeboard provided in the original design and factors that may not have been accounted for in the original design, including air bulking, cross waves, debris and/or debris jams, or variations in boundary roughness. If the current flood loadings indicate that the spillway design capacity will not be exceeded, and if a review of the design documentation indicates that the design methods were adequate and there are no other concerns, overtopping of the chute and stilling basin walls will generally not be a concern.

Spillway Discharges (Depths and Durations)

Water surface profiles in the spillway can be calculated for discharges that are obtained from the routings of frequency floods. A range of discharges which correspond to given frequency floods should be evaluated to provide flow depths and velocities at selected stations in the chute and can be completed with either models (ex. ZPROFILE) or boundary layer theory calculations. A simplified approach that neglects friction losses can be used to calculate an upper bound for the velocity at any point in the chute. This can be determined by the equation $V_T = [2g(H+h)]^{1/2}$, where H is the reservoir head above the crest, and h is the change in elevation from the crest to a point in the chute (Reclamation, 1987).

For hydraulic jump stilling basins, the conjugate depth of the hydraulic jump (the flow depth at the downstream end of a hydraulic jump) can be calculated and compared to the stilling basin wall heights. Flood routings will provide information on the duration of certain discharge levels. If durations of spillway flows are limited, failure of the spillway chute and stilling basin, due to wall overtopping, may initiate but may not have time to fully develop into a breach of the reservoir.

Convergence and Divergence of Chute Walls

The best hydraulic performance of a spillway chute is obtained when the confining sidewalls are parallel to the flow direction and the distribution of flow across the channel is relatively uniform. In order to optimize a spillway design, however, it may have been desirable to make the chute narrower or wider than either the crest structure or the terminal structure. Sidewall convergence must be made gradual to avoid cross waves, wave runup on the walls and uneven distribution of flow within the chute. In a similar manner, the divergence of spillway chute walls should be limited or else the flow will not spread to uniformly fill the chute. Experiments have shown that an angular variation of the flow boundaries should be limited by the following equation (Reclamation, 1987):

$$\tan\alpha = \frac{1}{3F}$$

where the Froude number, $F = v/\sqrt{gd}$,

α is the angular variation of the sidewall with respect to the channel centerline and v and d are the velocity and depth at the start of the transition.

If the convergence or divergence of the chute walls exceeds the published guidelines, cross waves will need to be considered as part of a wall overtopping evaluation.

Superelevation

Curved spillway chutes result in a rise in water surface on the outside wall of the chute and a depression of the surface along the inside wall due centrifugal force caused by flow around a curve. This phenomenon is called superelevation. The amount of superelevation in chutes subjected to subcritical flow is generally small. For chutes subjected to supercritical flow, the rise in water surface has been found experimentally to be about twice that for subcritical flow. Another potential issue with supercritical flow is that standing waves can be generated, if simple curves are used to form the chute. For curved spillway chutes subjected to supercritical flow, the use of spiral transitions with circular curves and invert banking will reduce the effects of wave heights. The following equation for the transverse slope of the water surface in a curved channel was obtained by balancing outward centrifugal and gravitational forces [Woodward and Posey, 1941]:

$$\Delta y = CV^2W/gr$$, where

C = coefficient (0.5 or 1.0 depending on conditions

V = mean channel velocity

W = channel width at elevation of centerline of water surface

g = acceleration of gravity

r = radius of channel centerline curvature

The above equation allows for the computation of the total rise in water surface due to both superelevation plus surface disturbances. A more detailed discussion on the effects of superelevation in curved spillway chutes and guidance on coefficients is provided in EM 1110-2-1601, ch2 (USACE, 1994).

Air Bulking in Flow

Air bulking occurs where the turbulent water boundary reaches the water surface and air is introduced into the flow (entrained air) as a result of this turbulence. Bulking will generally increase the depth of flow but is currently not accounted for in ZPROFILE. To adjust the flow depths for air bulking, the following equation can be applied:

$$d_b/d = 1/1\text{-}C\text{, where}$$

d = flow depth (non-bulked)

d_b = bulked flow depth

C = mean air concentration, obtained from Figure VI-2-2, where X^* = the distance from the point of inception to the location of interest and Y_i = the depth of flow at the point on inception

The point of inception is the location where the boundary layer reaches the flow surface and the point where air entrainment in the flow initiates illustrated in Figure VI-2-3. It has been found that the concept of a bulked flow depth does not fully capture the water surface that must be contained in a spillway chute (Falvey, 1980). Due to the turbulence at the surface of aerated flow, the surface at any one location in the chute fluctuates and has been found to vary with the peaks and valleys of waves that exist in the flow. Air is entrapped in the surface waves of self-aerated flow and this contributes to the fluctuating flow depths that occur in spillway chutes. The following equation provides a method of estimating the bulked flow depth to the top of the waves on the flow surface and considers both entrained air and entrapped air (Falvey 2007):

$$d_b/d = 1/1\text{-}(C_e + C_E)\text{, where}$$

d = flow depth (without aeration)

d_b = bulked flow depth to the top of the waves

C_e = mean entrained air concentration, obtained from Figure VI-2-2

C_E = mean entrapped air concentration, found to be relatively constant at 0.23 (Wilhelms and Gulliver, 2005)

When evaluating spillway chutes for wall overtopping, adjustments to the estimated non-aerated flow depths should be considered, due to both entrained air and entrapped air. The bulked flow depth resulting from entrained air will be a more constant value, while the bulked flow depth resulting from the entrapped air component will be a fluctuating value based on wave action. If the slope of a spillway chute is flat (less than about 10 percent) the boundary layer will not be able to reach the flow surface and entrain air into the flow. For flatter slopes adjustments for air bulking are not required.

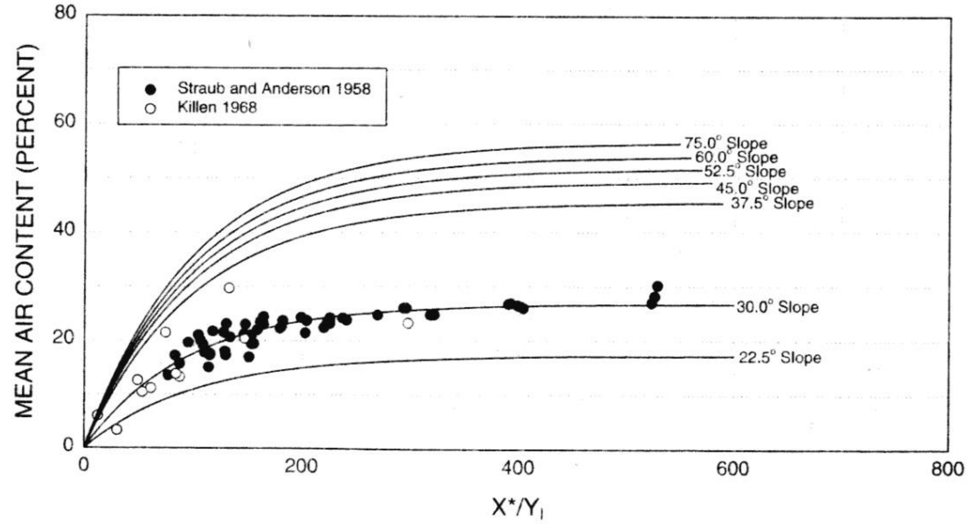

Figure VI-2-2. Mean Concentration of Entrained Air (percent by volume)

(from Wilhelms and Gulliver, 2005)

20 AIR-WATER FLOW IN HYDRAULIC STRUCTURES

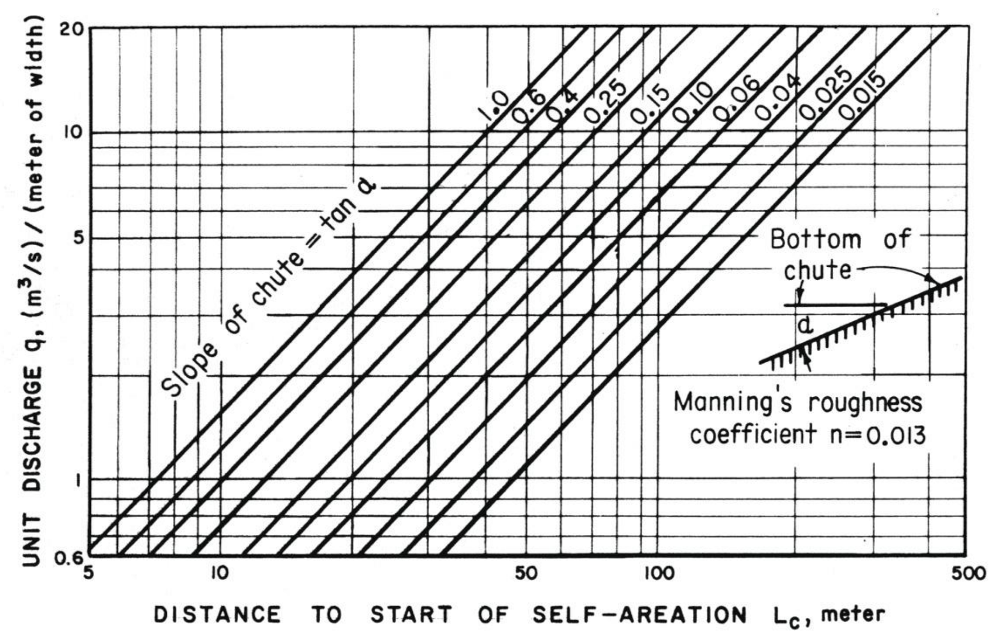

FIGURE 8.—*Location of inception of air entrainment.*

Figure VI-2-3. Determination of Point of Inception (from Falvey, 1980)

It has also been found that the depth of flow decreases and velocity increases compared to that calculated with the above considerations as the air concentration increases above a value of 25 percent. There is an apparent reduction in the coefficient of friction for highly aerated flow. Adjustments to the flow depth can be made to account for the reduction related to air concentration as shown in Figure VI-2-4.

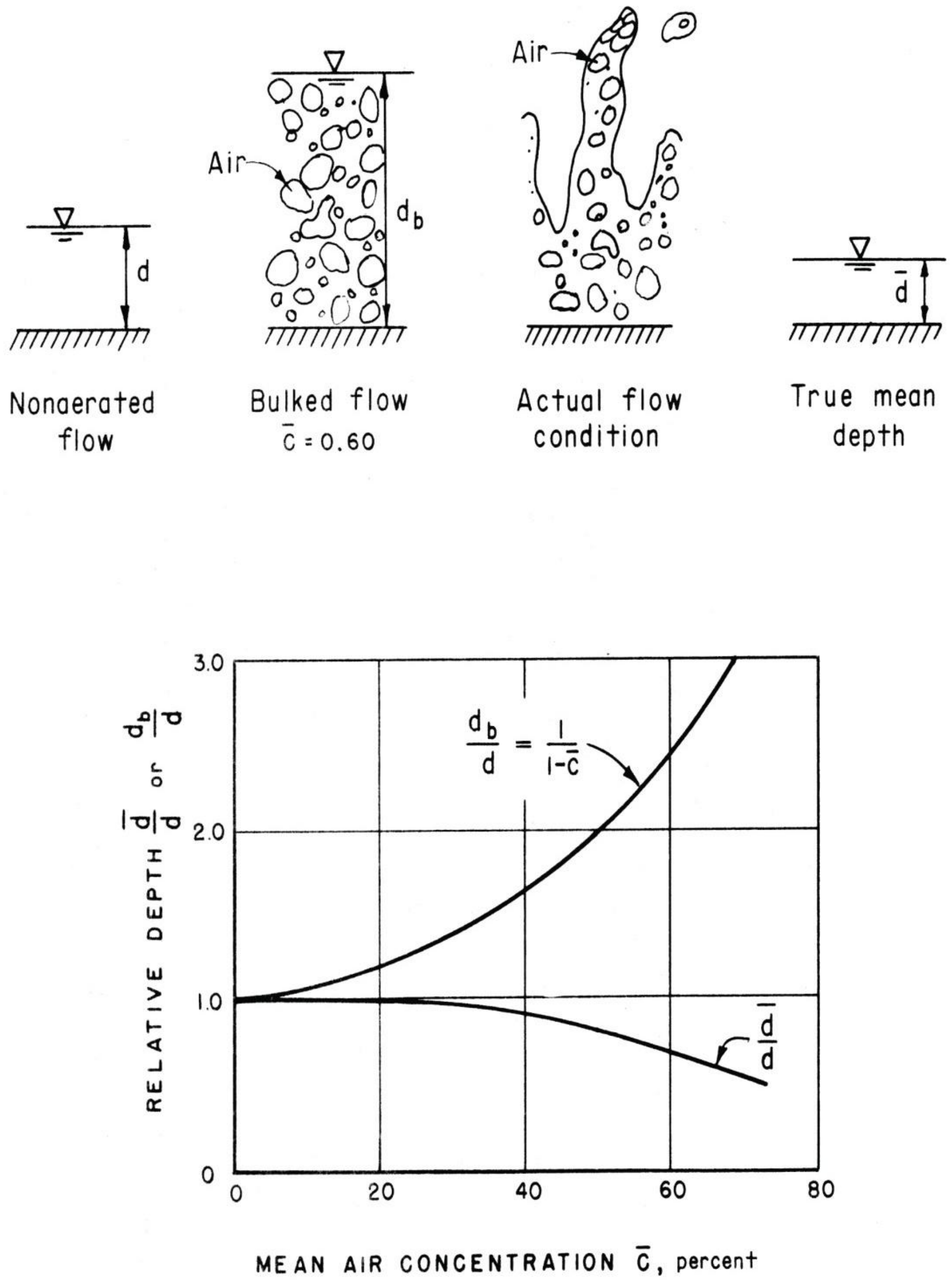

Figure VI-2-4. Air Concentration in Flow (Flavey, 1980)

Cross Waves in Spillway Chutes

Cross waves can form in a spillway chute from a variety of sources – from non-symmetrical entrance conditions into the spillway control structure, from chute walls that converge too rapidly, from piers that are introduced into the flow and then terminate or from curved chute walls. Cross waves will be superimposed on the flow depths that would occur under normal conditions and could lead to wall overtopping. For trapezoidal channels, cross waves can lead to run-up and wall overtopping sooner than for rectangular sections.

Erodibility of Foundation Materials

Soil foundations are generally more erodible than rock foundations. If erosion of the foundation materials initiates and progresses, this could lead to undermining of the chute slab foundation and collapse of the chute slab. In order to allow headcutting to progress upstream, the chute slab must fail at some location, and must fail progressively along with the headcutting. Otherwise the flow initiating erosion will enter the eroded area at a fixed location (where wall overtopping occurs) and the flow will not have the energy to sustain the erosion in the upstream direction. If the chute slab does not fail, erosion can still initiate and progress laterally outside of the chute. Headcutting and progression of erosion will be a function of the erodibilty of the foundation materials (see the section on Erosion of Rock and Soil). If the foundation consists of competent rock, upstream progression of erosion may be limited.

Spillway Configuration

Uncontrolled spillways cannot be regulated and provide little or no opportunity to reduce discharges and control flows should problems develop during flood releases. Gated spillways may allow the opportunity reduce flows (assuming that there is adequate reservoir storage to allow this to happen without risking an overtopping failure of the dam) and slow down or arrest failure of the entire spillway if this potential failure mode is in progress. Closing spillway gates may also buy some time to perform temporary repairs and armor the potentially erodible materials.

Ball Milling

Ball milling is a mechanism, where material trapped in a hydraulic jump stilling basin is circulated within the flow and abrades and erodes the stilling basin concrete due to a repetitive grinding process. Abrasion is the mechanism in which the stilling basin concrete is ground down, with aggregate and concrete fragments also being loosened in the process. Erosion is the process in which abraded material and loosened concrete materials are removed from the concrete surface by flows that pass through the stilling basin. Material that becomes trapped in the stilling basin is typically sand, gravel, cobble or boulder size material that is pulled into the stilling basin from the downstream channel. Ball milling can progressively fail the stilling basin floor lining and result in complete failure of the lining and undermining of the spillway stilling basin. Once this occurs, headcutting can initiate and progress upstream, ultimately leading to a breach of the reservoir.

While the material that becomes trapped in the stilling basin is typically pulled in from the downstream channel, material can also be introduced into the spillway structure by being thrown into the basin by the public, by being introduced into the crest structure at the spillway crest structure and deposited into the stilling basin, or can consist of rock or soil material from the cutslopes above the spillway chute or stilling basin can ravel and fall into the structure. Figure VI-2-5 portrays the flow patterns that pull material into the stilling basin and Figure VI-2-6 shows the creation of a riprap lined pre-formed scour hole located downstream of the stilling basin to prevent ball milling.

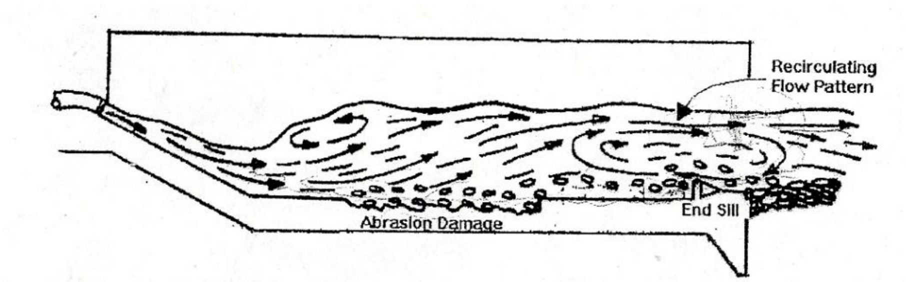

Figure VI-2-5. Recirculating Flows in Hydraulic Jump Stilling Basin

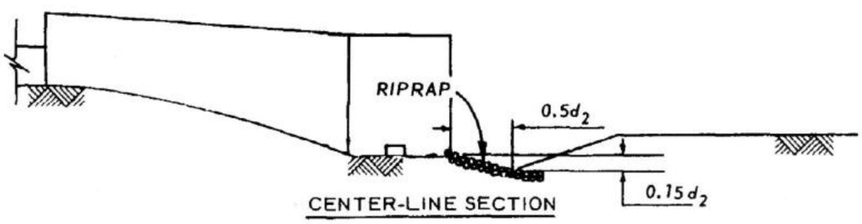

Figure VI-2-6. Section view of stilling basin showing downstream pre-formed scour hole (plate C-43, EM 1110-2-1602)

While ball milling damage is a common occurrence for hydraulic jump stilling basins, removal of stilling basin concrete is a very slow process. This is reflected in Table VI-2-1. It is unlikely that this mechanism would progress to complete failure (breach of the reservoir) unless ball milling damage progressed undetected over a long period of time (where accumulated spillway operation extended for months or even years) and the stilling basin foundation and spillway chute foundation were highly erodible. The time since the last inspection of the stilling basin is critical in determining the potential for a ball milling PFM.

Table VI-2-1. Ball Milling Case Histories

Dam	Agency	Concrete Compressive Strength, lb/in^2	Depth of Erosion, in	Duration of Spillway Flows, days	Abrasion/ Erosion Rate in/day
Libby	USACE	5000	24	720	**1 inch / 30 days**
Dworshak	USACE	n/a	3	53	**1 inch / 18 days**
Bull Shoals	USACE	3600	18	224	**1 inch / 12 days**
Pomona	USACE	5000 - 5600	2	960	**1 inch / 480 days**
Chief Joseph	USACE	n/a	12	420	**1 inch / 35 days**
Table Rock	USACE	n/a	3	45	**1 inch / 15 days**
Oologah	USACE	4000 - 5000	17	1100	**1 inch / 65 days**
Folsom	Reclamation	n/a	30	122	**1 inch / 4 days**

Stilling Basin Sweepout

Stilling basin sweepout can occur in hydraulic jump stilling basins *when the tailwater is insufficient to allow a hydraulic jump to develop or to be maintained.* If sweepout occurs, failure can initiate and progress in several ways. One mechanism is that the stilling basin sweepout leads to high-velocity flows exiting the downstream end of the stilling basin causing erosion in the downstream river channel, headcutting and a progressive failure up through the spillway chute or erosion of the toe of an embankment dam if erosion progresses laterally.

A second mechanism occurs with high tailwater surrounding the stilling basin but insufficient to force the hydraulic jump to occur within the structure. With the jump occurring downstream of the stilling basin, very shallow high-velocity flow conditions with minimal water weight are occurring within the stilling basin. This can lead to flotation of the stilling basin due to uplift pressures, failure of the stilling basin, erosion of the stilling basin foundation and headcutting upstream or erosion of the toe of an embankment dam if erosion progresses laterally. Factors to be considered include the weight of the structure relative to the volume of tailwater displaced by the structure and the presence or absence of anchorage to the underlying foundation. If either of these mechanisms have been observed, it is likely that the stilling basin is not adequately designed for the conceived capacity and modifications might be needed.

This failure mode can be evaluated by comparing the conjugate depths for various spillway flows to the predicted tailwater for the same flow. If the water surface elevation for the required conjugate depth (conjugate depth plus invert elevation) is greater than the tailwater elevation, sweepout is possible and progression of the failure mode will be dependent on the erodibility of the downstream channel and spillway stilling basin foundation materials or the proximity of the downstream toe of an embankment dam to the location where erosion occurs. Addition discussion on the potential for sweepout can be found in Peterka (1978).

Flood Studies/Flood Routing Analyses/Water Surface Profiles

A flood frequency study, along with the development of frequency hydrographs, is required to fully evaluate this potential failure mode. Flood hydrographs should include a range of floods from the point where spillway releases become significant up to the Probable Maximum Flood (PMF). A flood routing study is then conducted in which the frequency floods are routed and spillway discharges and durations determined for each flood event. While current guidance dictates peaking of the hypothetical hydrograph for PMF events, longer duration (flatter) hydrographs with lower peak flows may also be important to investigate due to prolonged durations. If the starting reservoir water surface elevation is likely to vary (based on historical reservoir elevations) and the results are sensitive to the reservoir elevation that exists when the flood occurs, the routings should be performed with a number of different starting reservoir water surface elevations.

Water surface profiles are then generated, using spillway discharge information from the frequency flood routings. For a given discharge and starting water depth at the spillway crest, flow depths and velocities can be determined at key stations along the spillway chute through the use of models such as ZPROFILE. Combining this information with wall heights along the spillway chute can be used to estimate probabilities for the development of this potential failure mode

For Type I, II and III stilling basins, the depth of flow at the end of the basin or the conjugate depth can be calculated from the following equation:

$$\frac{d_2}{d_1} = \frac{\sqrt{1+8F^2}-1}{2}\text{, where}$$

d_2 = the conjugate depth or depth at the downstream end of the hydraulic jump

d_1 = the depth of flow entering the stilling basin

F = Froude number: $v_1/(g \text{ x } d_1)^{1/2}$, where v_1 and d_1 are the velocity and depth of the flow entering the stilling basin and g is the acceleration of gravity.

Overtopping of stilling basin walls will typically be less of a concern than chute walls. The main reason for this is that tailwater adjacent to the stilling basin walls will dissipate the energy from overtopping flows and minimize the erosion of the backfill and foundation. However, waves that overtops the stilling basin walls can lead to erosion of the backfill and loss of support in the stilling basin walls. If a basin is in a narrow channel, it's possible that the tailwater will be pushed downstream of the basin, and the benefits describe above will not exist.

Spillway discharges generally pass through critical depth at the spillway crest structure and enter the spillway chute at supercritical flow. The flow in the channel may be uniform or it may be accelerated or decelerated, depending on the slopes and the dimensions of the channel and on the total drop to the river channel. Flow at any point along the spillway chute will depend on the specific energy, $d + h_v$, available at that point, where d is the flow depth and h_v is the velocity head. The velocities and depths of open channel flow in a chute conform to the principle of the conservation of energy, expressed by Bernoulli's theorem, which states that "the absolute energy of flow at any cross section is equal to the absolute energy at a downstream section plus intervening losses of energy." This relationship can be expressed by the following equation:

$$\Delta Z + d_1 + h_{v1} = d_2 + h_{v2} + \Delta h_L, \text{ where:}$$

ΔZ = the difference in chute floor elevation between points 1 and 2

d_1 = the flow depth at point 1

h_{v1} = the velocity head at point 1

d_2 = the flow depth at point 1

h_{v2} = the velocity head at point 1

Δh_L = the losses in the chute between points 1 and 2, including friction, turbulence, impact and transition losses

ZPROFILE computes the water surface profile using the Standard Step method for gradually varied flow. In this method, the distance between stations is known and the correct depth at each station is determined in the computations. The computation is carried forward in a series of steps, beginning with a known depth of flow (such as critical depth) at the first station. The depth of flow is used in the computations to obtain area, velocity, velocity head and hydraulic radius. Friction slope is calculated by the theoretically derived Chezy equation which is based on Reynolds number and boundary roughness rather than the empirical subcritical flow loss equations of Mannings, Scobeys or Hazen-Williams. The loss in head due to friction is then computed by multiplying the friction slope by the length of the reach. ZPROFILE accounts for the boundary layer thickness, correction of the invert slope for steeper spillway chutes (converts flow depths normal to the chute invert to flow depths in the vertical direction for use with the energy equation), and air entrainment into the flow.

Event Tree

Figure VI-2-7 is an example of an event tree for this potential failure mode, for the case where lateral erosion outside the chute is not a major concern. The event tree consists of a number of events that lead from initiation, through progression, to full development of a spillway crest structure breach. The first two nodes represent the combined load probability (which defines the range of reservoir elevations and spillway flows), while the remaining nodes detail the conditional probability of failure given the loading.

For the Ball Milling and Stilling Basin Sweepout variations of this failure mode progression to the point of stilling basin failure would then progress to node 5 and the subsequent nodes described above.

The first node represents the starting reservoir surface elevation (range of elevations to include flood pool, normal pool, top of active storage, etc. See chapter on Reservoir Level Exceedance Curves) and the second node represents the range of flood loadings. Since the flood load range probability is typically dominated by more frequent, low load events, the failure probability should also be weighted toward the lower end of the range (up to PMF, with 3 to 6 intermediate frequency floods or scaled hydrographs represented. See chapter on Hydrologic Hazard Analysis). For the overtopping of walls, consider the lowest flood range as the threshold flood that begins to overtop the wall as discharge at a level which failure due to chute wall overtopping is judged to be remote. Refer also to the section on Event Trees for other event tree considerations. With the tools currently available, the estimates for many nodes on the event tree must by necessity by subjective (see section on Subjective Probability and Expert Elicitation).

The remaining nodes in the event tree represent the conditional probability of failure given the load probability. For overtopping of the spillway chute walls, consideration should be given to the likelihood of air bulked and non-bulked water surface profiles, and friction modifications. For overtopping of stilling basin walls, a comparison of the hydraulic jump conjugate depth to the tailwater should be performed.

Once overtopping has occurred, erosion must initiate in the wall backfill, which is typically erodible with sustained overtopping unless it is protected with riprap or material with lower erosion potential. Erosion of the backfill could then lead to undermining of the chute walls and slabs. The undermining node should consider the foundation materials (see chapter on Erosion of Rock and Soil), and whether the structural details of the walls/slabs and anchors would limit the progression of the undermining. If anchor bars are provided for the concrete slab, additional erosion resistance will help reinforce the rock mass.

After the chute slabs and walls have begun to fail, headcut erosion can progress through erosion of the foundation and progressive collapse and eventually lead to breach. Key factors to consider with this node are: duration of overtopping flows and erosion rate of the foundation; lateral erosion could occur without chute wall failure, if so, replace this node and add nodes for progressive slumping and erosion of embankment; both headcutting and lateral erosion are possible and both may need to be evaluated; erosion rate of foundation material is critical, as is the location most likely to overtop and fail first; and deep cutoffs and or rock foundation may prolong or stall out the headcutting process.

In case of spillway failure, there are several methods to intervene during the process, including: if gated, can close to divert or reduce flows or for rapid emergency repairs; full closure of gates only if adequate reservoir storage for event; and use of emergency spillway, outlet or creation of a temporary spillway in a benign saddle or other area may all increase the likelihood of successful intervention.

Assuming headcutting or lateral erosion, the breach could progress to the reservoir depending on duration of flood event. Spillways adjacent to embankment dams may carry the added threat of erosion to the embankment leading to a breach whether or not the chute wall fails.

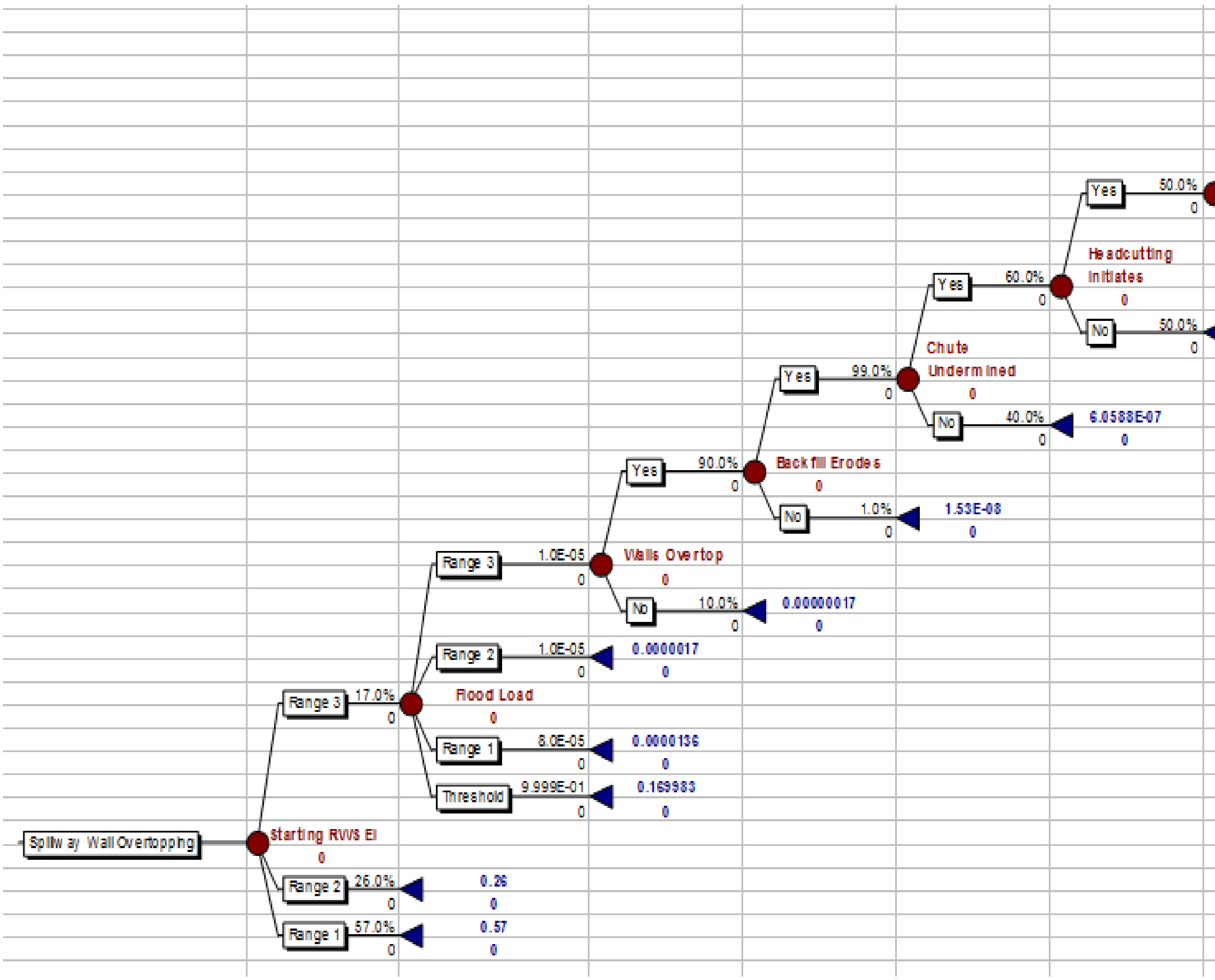

Figure VI-2-7. Example Event Tree

Accounting for Uncertainty

The method of accounting for uncertainty in the flood loading is described in the sections on Hydrologic Hazard Analysis and Event Trees. Typically, the reservoir elevation exceedance probabilities are taken directly from the historical reservoir operations data, directly, which do not account for uncertainty. Uncertainty in the failure probability and consequences are accounted for by entering the estimates as distributions (as described above) rather than single point values. A “Monte-Carlo” simulation is then run to display the uncertainty in the estimates, as described in the section on Combining and Portraying Risks.

There may be some uncertainty regarding spillway discharges for a given frequency flood, because of unpredictability in how the spillway will actually operate during a flood event. Spillway discharge capacity may be limited due to debris plugging or malfunctioning of spillway gates during a flood event, which would reduce the spillway discharge for a given frequency flood. It is not recommended that concerns over reduced spillway discharge capacity be considered for this potential failure mode, since in most cases the probability of these reductions are low and they are difficult to quantify and would reduce the likelihood of this potential failure mode developing.

There may be uncertainty regarding the possible effects of air bulking and the formation of cross waves in the spillway flow. Where conditions are unknown and the assumptions are critical, risk estimates can be made for two extreme possibilities and the results evaluated. The difference in the two estimates may provide justification to investigate these effects further. Relevant Case Histories

El Guapo Dam Spillway: December 1999

El Guapo Dam was located on the Rio Guapo, 3 miles south of the city of El Guapo, in the state of Miranda, Venezuela. The reservoir formed by the dam provided potable water for the local area, flood control, and irrigation water. The reservoir volume was 33,000 acre-feet. The dam was constructed from 1975 to 1980. The original spillway at El Guapo Dam consisted of an uncontrolled ogee crest, located on the left abutment of the dam, a concrete chute and a concrete hydraulic jump stilling basin. The spillway had a width of 40 feet, a length of 925 feet and a design discharge capacity of 3600 ft^3/s. Initial hydrologic studies were based on a similar basin but not the Rio Guapo basin. During construction of the spillway, the chute walls were overtopped, which triggered a new flood study. A tunnel spillway was constructed through the dam's left abutment, 820 feet from original spillway.

On December 14, 1999, the reservoir was 3 feet above the normal pool and 17 feet below the dam crest. The radial gate on the tunnel spillway was fully open, both spillways were operating normally. Early on the morning of December 15th the reservoir rose quickly and was 2.5 feet below the dam crest. Early the next morning the reservoir was 8 inches below the dam crest, the spillway chute walls just below the spillway crest began to overtop, and erosion of the adjacent fill initiated. By 4:30 a.m. on December 16th, cities below the dam were evacuated. At 9:00 a.m. the dam was inspected by helicopter and the reservoir level had subsided (2.5 feet below crest); people believed that flood had crested and the crisis was over. At 4:00 p.m. on December 16th, the reservoir rose again quickly; the bridge over spillway collapsed; erosion of spillway backfill accelerated and the reinforced concrete chute, basin and crest structure failed; but the concrete lined approach channel remained intact and controlled flows through the spillway. At 5:00 p.m. the approach channel failed and the reservoir was breached through the spillway area. El Guapo Dam never overtopped. Overtopping of the spillway chute walls initiated erosion of backfill behind chute walls and undermining and failure of spillway chute. Headcutting progressed upstream and lead to reservoir breach. The spillway foundation consisted of decomposed rock, which was erodible (Villar, 2002). Figures VI-2-8 through VI-2-13 provide a sequence of photographs from initiation through completion of the spillway failure.

Figure VI-2-8. Sweepout of spillway stilling basin

Figure VI-2-9. Overtopping along entire length of chute

Figure VI-2-10. Overtopping of upstream chute walls

Figure VI-2-11. Headcutting progressed to reservoir

Figure VI-2-12. Breach formation nearing completion

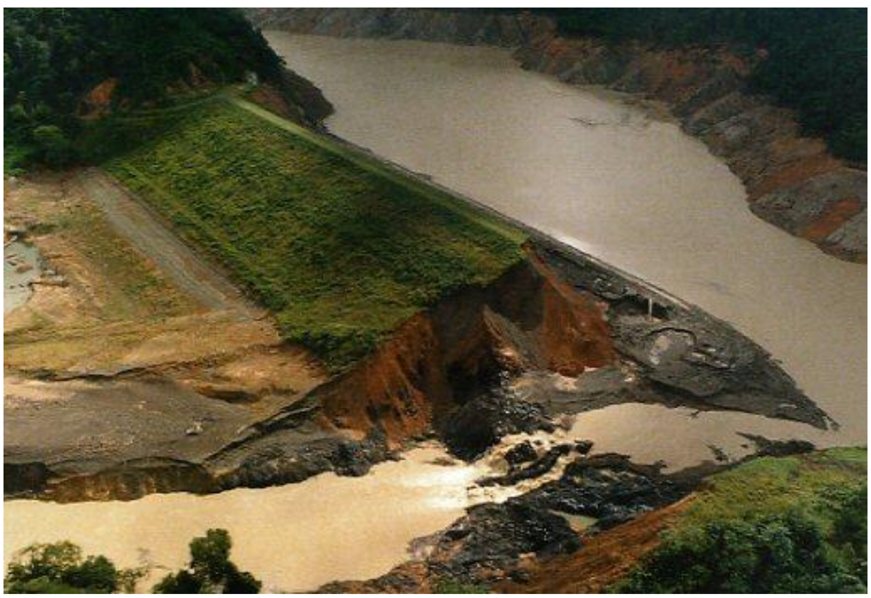

Figure VI-2-13. Aftermath of reservoir breach

Considerations for Risk Analysis

The complete analysis as described in this section is likely to be too time consuming to be performed for a quick risk assessment. Therefore, simplifications must be made. Fewer load ranges are typically evaluated with @Risk and only the expected value estimates are entered into the event tree. If the results of water surface profiles are available, they can be used to define the depth and velocity of flow at stations along the spillway chute. If water surface profiles are not available possible ranges of flow depths and velocities can be estimated.

The minimum depth of flow can be determined from the simplified velocity equation presented in the section on Spillway Discharges, using the relationship that $Q = VA$, where Q is the spillway discharge, V = the flow velocity and A equals the cross-sectional area of flow (width of chute x depth of flow). It should be noted that the above procedure will define the minimum depths that can be expected at a point along a spillway chute. Losses will reduce the available head at a given location, which will reduce velocities and increase flow depths.

Exercise

Consider a spillway with a concrete lined chute. The rectangular chute is 20-feet wide. The chute walls are 10-feet high. Estimate the annual probability that the chute walls will be overtopped at Station 10+00, using the information provided in Table VI-2-2. Neglect the effects associated with air bulking of the flow and assume that there is no potential for cross waves in the chute.

Table VI-2-2. Spillway Discharge and Flow Velocities in Spillway Chute, Station 10+00

Frequency Flood, ACE	Spillway Discharge, ft^3/s*	Flow Velocity, ft/s
0.001	2000	40
0.0001	7300	55
0.00001	17,800	88
0.000001	25,300	91

* Spillway discharges did not change appreciably with a variable starting water surface elevation.

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