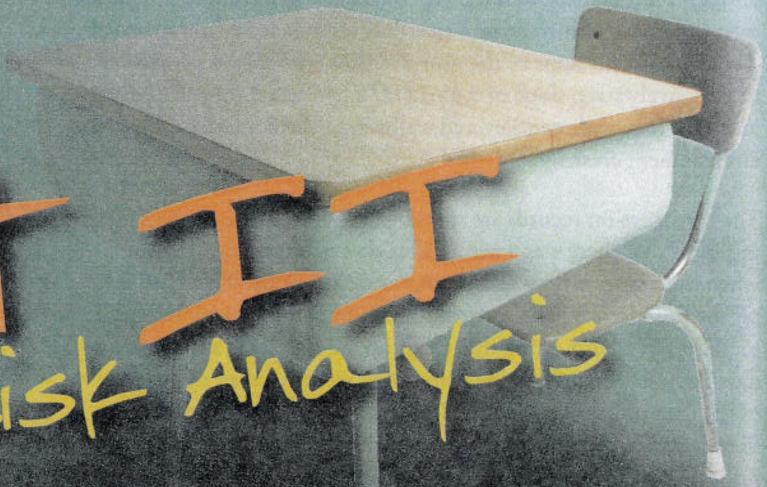
SHEDDING SOME LIGHT ON THIS THING CALLED RISK ASSESSMENT



Part II
Example Risk Analysis

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INTRODUCTION

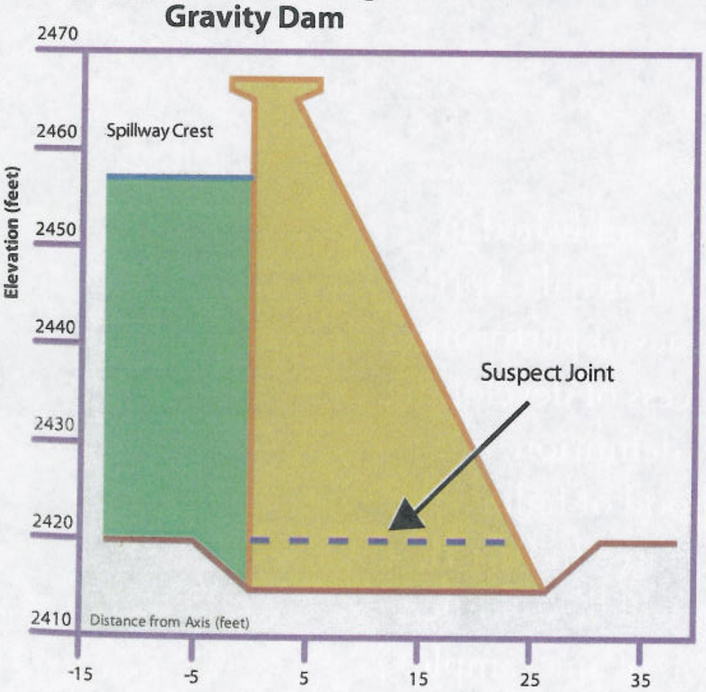
Part I of this article was published in Volume 8, Issue 1 of the Journal (April, 2010). It covered the basics of performing risk analyses and the process of developing risk estimates. Risk, by definition, must consider both the likelihood of failure and the consequences of failure. In Part II, the risk analysis process will be solidified through the presentation of an example.

EXAMPLE DESCRIPTION

Let's take a look at flood-related risks for a moderate-sized dam. Consider, for example, an embankment dam with an uncontrolled ogee spillway section. The embankment and spillway are connected by a non-overflow concrete gravity section. The dam was completed in the 1930's. The embankment is about 65 feet high and consists of loosely compacted silty sands with some gravel and a concrete core wall which extends from the foundation contact under the centerline of the dam to within about a foot of the dam crest. The corewall ties into the non-overflow gravity section and the massive granite foundation rock. The upstream and downstream embankment slopes are 2:1 (H/V), with a 10-foot-wide unpaved crest at elevation 2467. The non-overflow gravity section has a maximum structural height of about 52 feet and is 240 feet long. Vertical keyed contraction joints were provided within the non-overflow section at 50-foot spacings. The downstream slope of the non-overflow section was constructed at a relatively steep slope of 0.45:1 (H/V), with a 3-foot-wide crest and an 8-foot-wide cantilevered walkway at the same elevation as the embankment crest.

The construction records are sparse, but a letter in the owner's files indicates that there was a winter shut down during construction when the non-overflow section was at elevation 2420, about 5 feet off the foundation. The construction joint at this elevation leaks in localized areas, indicating that clean-up the following spring may have been less than desirable. No drains were installed in this section of the dam or its foundation. Construction photos indicate the foundation excavation surface was extremely rough and well cleaned prior to concrete placement. No adverse jointing was observed within the foundation rock mass exposures. Figure 1 shows a cross section through the non-overflow gravity dam. Placement of the spillway section did not begin until the non-overflow section was completed, and the overflow section does not suffer from the same vulnerabilities as the non-overflow section. The crest of the overflow section is 10 feet lower than the crest of the non-overflow section.

FIGURE 1 **Section through Non-overflow Gravity Dam**



Historically, a maximum of about 5 feet of water has passed over the spillway (in 1984) with no signs of distress. More than 2 feet of water has passed over the spillway crest several times. Routing of the PMF indicates both the embankment and gravity section would be overtopped for 14 hours by up to 1.7 feet.

POTENTIAL FAILURE MODE IDENTIFICATION AND DESCRIPTION

An obvious potential failure mode involves overtopping of the embankment. As detailed in Part I of this article series, a potential failure mode description should include three parts (initiator, step-by-step progression, and breach characteristics). This potential failure mode description might look something like the following:

Hydrologic Potential Failure Mode No. 1: *As the result of a large flood, the reservoir rises to and exceeds the crest of the dam* ***(the initiator)*** *causing (1) erosion of the embankment on the downstream side of the concrete corewall, (2) collapse of the concrete corewall due to loss of support, and (3) subsequent collapse of the upstream embankment* ***(the step-by-step progression).*** *It is likely the embankment would be eroded down to the granite foundation (through about 13 feet of alluvium) at about elevation 2402, although a stub of the corewall could remain for a few feet above that elevation. Erosion of the embankment will occur quickly, but it may take some time for the corewall to fail* ***(the breach characteristics).***

Another potential failure mode relates to the potentially poorly bonded lift joint in the non-overflow gravity section. Hand calculations indicate the entire construction joint at elevation 2420 is in compression when the reservoir is at the spillway crest, but the upstream face at elevation 2420 goes into tension when the reservoir rises to about elevation 2461.8, coincidentally about the level of the maximum spillway release. Two holes were cored through each of the two non-overflow monoliths nearest the spillway to intersect the construction joint. The core indicated generally good concrete, except that two of the cores were not bonded at the construction joint, one in each of the monoliths. The core was small diameter and not tested. A clear vulnerability has been identified here if the potential for cracking along the joint is considered during flood loading. A description of the potential failure mode might be as follows:

Hydrologic Potential Failure Mode No. 2: *As a result of a large flood, the reservoir rises to unprecedented levels* ***(the initiator)*** *causing (1) the tensile strength of the concrete to be exceeded on the upstream face at the winter shut-down lift joint (elevation 2420) of the non-overflow section, (2) a crack to propagate across the section or a sufficient distance to significantly weaken the structure, (3) uplift to increase within the crack, and (4) sliding to initiate and rapidly continue until uncontrolled release of the reservoir occurs* ***(the step-by-step progression).*** *If the breach occurs adjacent to the spillway, the reservoir would be lost down to the joint at about elevation 2420; if the breach occurs adjacent to the embankment, it is likely the embankment would be eroded down to about elevation 2402. Breach inundation studies suggest that the depth of downstream flooding will be about the same in either case* ***(the breach).***

EVENT TREES

Embankment Overtopping (Hydrologic Potential Failure Mode No. 1)

As a result of the team discussions, it was decided to only include two branches in the event tree for Hydrologic Potential Failure Mode No. 1; one with the maximum reservoir water surface above the corewall but prior to overtopping (elevation 2466 to elevation 2467), and one for overtopping conditions (greater than elevation 2467). The reasons for this are described further below in the section on Subjective Probability. The event tree for this potential failure mode is shown on Figure 2. The event trees and calculations described in this article were created using the PrecisionTree and @RISK software developed by Palisade Corporation. Similar software is available from other vendors (for example, Crystal Ball by Decisioneering, Inc.).

Lift Joint Sliding (Hydrologic Potential Failure Mode No. 2)

Given the description for Hydrologic Potential Failure Mode No. 2, the event tree shown on Figure 3 was developed. In this case, three flood load ranges were selected above the threshold to capture changes in structural response. The threshold, below which the contribution to risk is considered to be negligible, was taken as reservoir elevation 2462, where good performance was experienced in 1984, and the approximate level where calculations indicate the heel would go into tension. Three load ranges were established between this elevation and the elevation of the dam crest (2467) to provide good resolution in the area of potential cracking. Load ranges above the crest of the non-overflow section were not included for this potential failure mode since overtopping of the non-overflow section would also result in overtopping of the embankment, and erosion of the embankment was considered to control the risk under these conditions (i.e. the probability of failure due to overtopping of the embankment was considered to dominate the risk once overtopping commences). The team initially included three nodes in the event tree after the "tensile strength exceeded" node (significant cracking, increase in uplift, and sliding instability). Upon further discussion, it was agreed that by using a probabilistic stability

FIGURE 2 Event Tree for Overtopping Potential Failure Mode

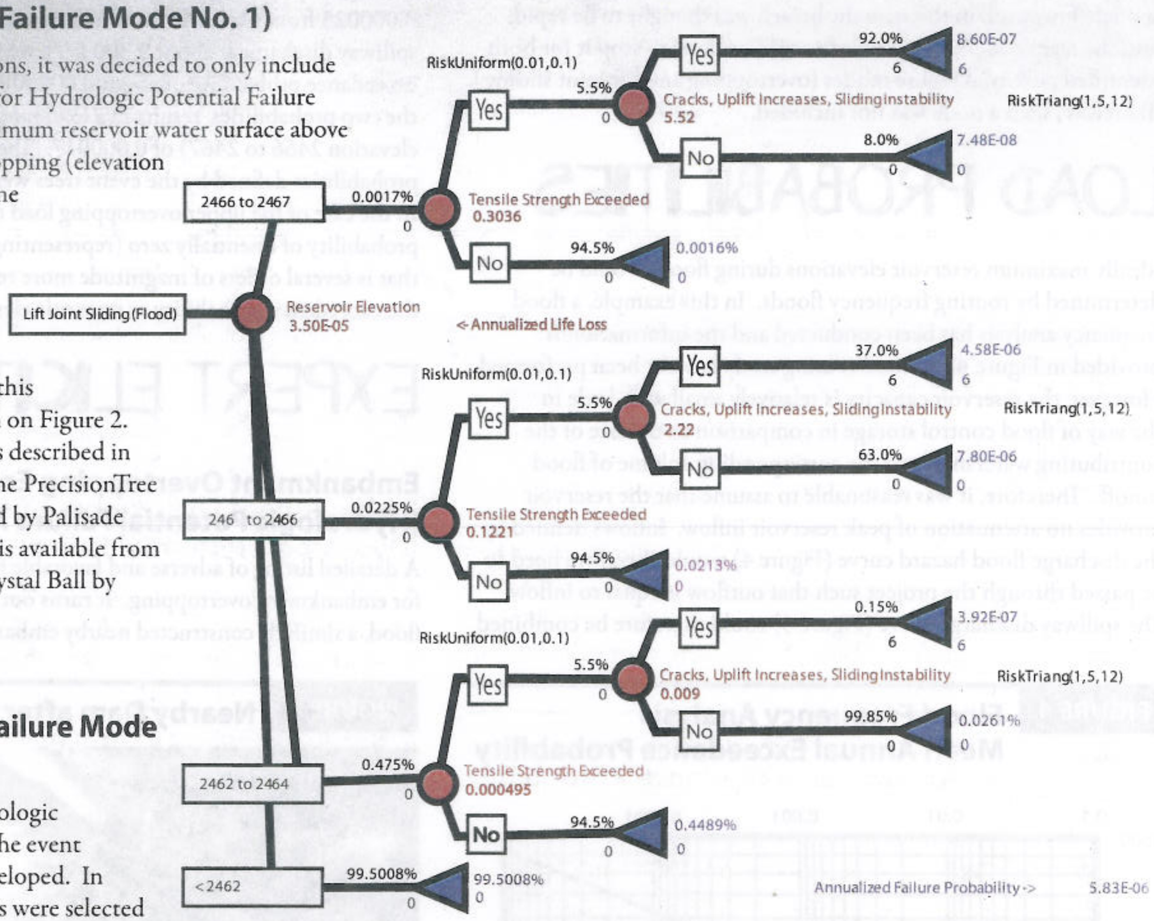
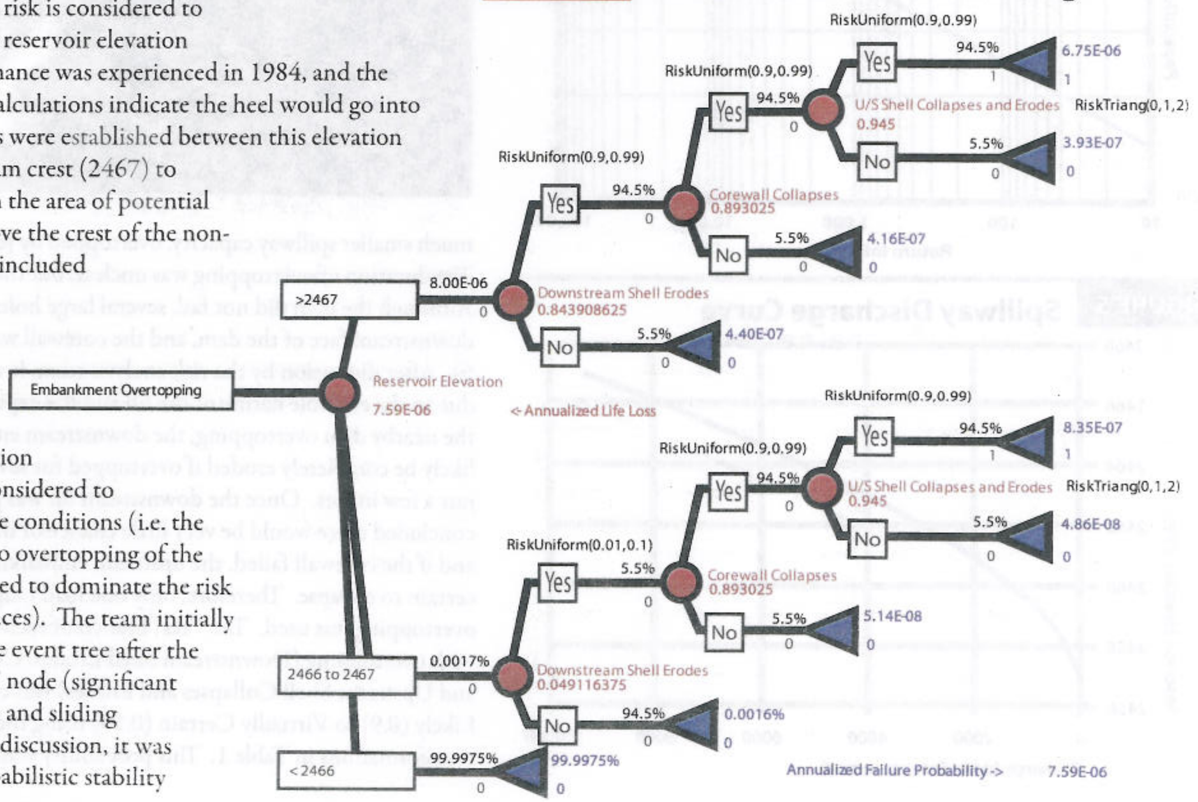


FIGURE 3 Event Tree for Lift Joint Sliding



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analysis (described in more detail below) that these three nodes could be evaluated in one step.

In some cases the chances of detecting an impending breach and intervening are included as an event tree node in the risk estimation model. However, in this case the breach was thought to be rapid, and the team concluded that little could be done to stop it for both identified potential failure modes (overtopping and lift joint sliding). Therefore, such a node was not included.

LOAD PROBABILITIES

Ideally maximum reservoir elevations during floods would be determined by routing frequency floods. In this example, a flood frequency analysis has been conducted and the information is provided in Figure 4. A flood routing study has not been performed. However, the reservoir capacity is relatively small with little in the way of flood control storage in comparison to the size of the contributing watershed and the corresponding volume of flood runoff. Therefore, it was reasonable to assume that the reservoir provides no attenuation of peak reservoir inflow. Inflows defined by the discharge flood hazard curve (Figure 4) would therefore need to be passed through the project such that outflow is equal to inflow. The spillway discharge curve (Figure 5) could therefore be combined with the flood hazard curve (Figure 4) to estimate the flood load range probabilities corresponding to the reservoir water surface elevations in the risk estimation model. For example, at reservoir elevation 2466, the spillway discharge is about 7,800 ft³/s, which corresponds to an annual flood exceedance probability of about 0.000025 from Figure 4. Similarly, at reservoir elevation 2467, the spillway discharge is about 9,300 ft³/s, which corresponds to a flood exceedance probability of about 0.000008 from Figure 4. Subtracting the two probabilities, results in a load range probability (for reservoir elevation 2466 to 2467) of 0.000017. The remaining load range probabilities defined by the event trees were similarly calculated. In the case of the upper overtopping load range, an exceedance probability of essentially zero (representing an upper bound flood that is several orders of magnitude more remote) is subtracted from the exceedance probability at reservoir elevation 2467.

FIGURE 4 Flood Frequency Analysis Mean Annual Exceedance Probability

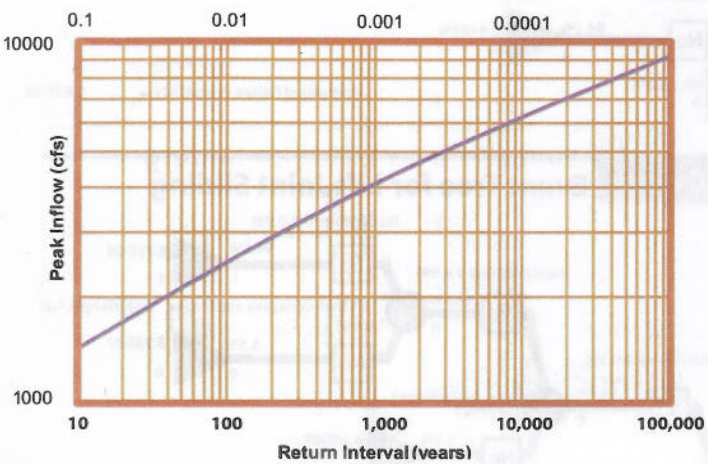
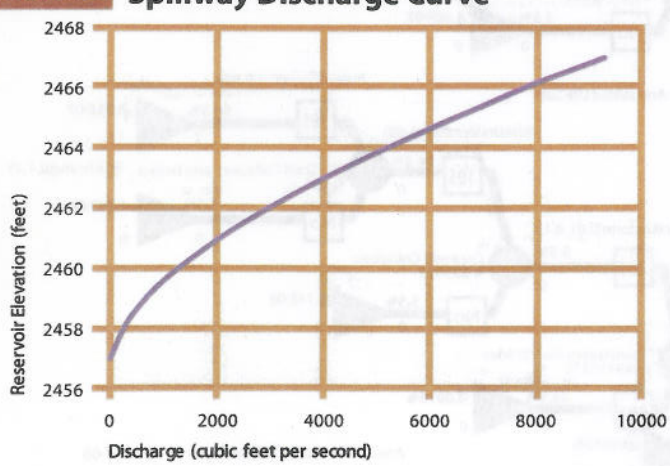


FIGURE 5 Spillway Discharge Curve



EXPERT ELICITATION

Embankment Overtopping Erosion (Hydrologic Potential Failure Mode No. 1)

A detailed listing of adverse and favorable factors was not developed for embankment overtopping. It turns out that during the 1984 flood, a similarly constructed nearby embankment dam, with a much smaller spillway capacity, overtopped by just over an inch. The duration of overtopping was unclear, but thought to be short. Although the dam did not fail, several large holes were eroded in the downstream face of the dam, and the corewall was exposed (see Figure 6). After discussion by the risk analysis team, it was concluded that due to the erodible nature of the fill and the experience afforded by the nearby dam overtopping, the downstream embankment would likely be completely eroded if overtopped for several hours by even just a few inches. Once the downstream fill was gone, the team concluded there would be very little chance of the core wall standing, and if the corewall failed, the upstream embankment was almost certain to collapse. Therefore, only one load range corresponding to overtopping was used. The "Yes" branch of each event node associated with overtopping (Downstream Shell Erodes, Corewall Collapses, and Upstream Shell Collapses and Erodes) was considered to be Very Likely (0.9) to Virtually Certain (0.99) using the verbal descriptor transformations in Table 1. This probability was defined using a

FIGURE 6 Nearby Dam after Overtopping

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Table 1. Verbal Descriptors and Approximate Probabilities (adapted from Vick, 2002)

Verbal Descriptor	Suggested Probability	Approximate Probability Range from Reagan et al
Virtually Impossible, due to known physical conditions or processes that can be described and specified with almost complete confidence	0.01	0-0.05
Very Unlikely, although the possibility cannot be ruled out	0.1	0.02-0.15
Equally Likely, with no reason to believe that one outcome is more or less likely than the other (when given two outcomes)	0.5	0.45-0.55
Very Likely, but not completely certain	0.9	0.75-0.9
Virtually Certain, due to known physical processes and conditions that can be described and specified with almost complete confidence	0.99	0.9-0.995

uniform distribution, meaning it is equally likely anywhere within the range.

The team concluded that there would be a negligible chance of embankment failure with the reservoir below the top of the concrete core wall, since the wall is reinforced and tied into rock. For reservoir elevations above the corewall but prior to overtopping (Reservoir elevation 2466 to 2467), only a small likelihood of embankment failure was considered appropriate based on the low head, gradient, and duration of loading, but the team decided to carry this load range through the tree. Using the verbal descriptors, erosion of the downstream shell for this load range was judged to be in the range 0.01 to 0.1. Since each node in an event tree assumes the previous branch is "true", the remaining conditional probabilities all must assume the downstream shell has been eroded. Thus, the same probabilities were assigned to these nodes as in the overtopping load range, for reasons discussed above.

Lift Joint Sliding (Hydrologic Potential Failure Mode No. 2)

In assessing risks, it is important to include all reasonable resisting mechanisms. Therefore, tensile strength is not ignored, and the node after the loading in the lift joint sliding event tree considers whether the tensile strength is exceeded on the upstream face at the location of the potential lift joint sliding plane. Since there is significant uncertainty surrounding both the tensile strength of the concrete and the stress at the upstream face in this location, expert elicitation was used to estimate the likelihood of exceeding the tensile strength for each load range.

The following list of adverse and favorable factors was developed. Most apply to all load ranges; however, the calculated tensile stress at the subject location would change with load range.

Adverse (more likely) Factors:

- The coring indicates a portion of the suspect lift joint is unbonded; therefore a portion of the suspect lift joint may be unbonded near the upstream face, in which case that portion of the joint would have no tensile strength.
- Gravity method calculations indicate a tensile effective stress (25 lb/in² at the heel of the dam when the reservoir level reaches the crest of the dam, and 20 lb/in² with the reservoir at elevation 2466).
- The tensile stress calculations were performed assuming plane sections remain plane, resulting in a trapezoidal stress

distribution. The actual stress distribution will be nonlinear, with higher tensile stresses at the upstream face than indicated by the analysis results. (Note: The subject cold joint was thought to be the weak link; therefore, the team did not consider the possibility of a crack opening at another location, which would change the distribution of tensile stresses at the location of the lift joint in question.)

- Apparently less than rigorous joint cleanup following the winter shutdown could have resulted in low strength.

Favorable (less likely) Factors

- Offsetting the first bullet above, a portion of the suspect lift joint is bonded, meaning that a portion of the joint possesses tensile strength at the heel.
- The cored concrete was in good condition, suggesting a compressive strength of at least 3,000 lb/in². This implies the parent concrete would have a corresponding estimated tensile strength of about 350 lb/in² (e.g. $f_t = 1.7(f_c')^{2/3}$, Raphael, 1984). If the bonded portions of the lift surface have even half this strength over more than 30 percent of the area or so, the lifts are likely stronger than the induced tensile stress.
- The un-bonded cores were in different monoliths; it does not appear that an un-bonded lift extends across an entire monolith. This is supported by the fact that lift joint leakage occurs only locally across the joint.

Although the team recognized the shortcomings of the analysis method for estimating tensile stresses at the upstream face, it estimated the chance of exceeding the concrete tensile strength as being small in all cases, and opted to assign a likelihood of 0.01 to 0.1 for all load ranges. The primary reason for this is that coring showed a significant portion of the joint in each monolith to be bonded and the bond strength is likely to be considerably higher than the calculated tensile stresses for all load ranges, even considering a potential nonlinear stress distribution and only partial joint bonding. In addition, the change in the calculated tensile stress was not significant between load ranges.

The estimated probability of the third node of the lift joint sliding event tree (Figure 3) was evaluated as follows. Given that the tensile stress was exceeded on the upstream face at the lift joint (the Yes branch of node 2), the team decided to use the analysis method described in *Design of Small Dams* (Bureau of Reclamation, 1987) to examine the potential for crack propagation and sliding instability. In the uncracked condition, uplift pressures acting along the lift surface were assumed to vary linearly from full reservoir head at the upstream face to tailwater at the downstream face. In this case, tailwater pressure head is zero since the estimated elevation of the tailwater during the PMF was below the base of the non-overflow section. If a crack were to form, uplift was assumed to be at full reservoir pressure at the tip of the crack varying from that point linearly to tailwater. If the analysis indicated that the crack would propagate completely through the structure, a linear distribution of uplift pressure was again assumed from reservoir pressure at the upstream face to tailwater at the downstream face. A probabilistic analysis, described below, was used to estimate the potential for ultimate sliding failure resulting from crack propagation and an increase in uplift pressure along the crack.

PROBABILISTIC STABILITY ANALYSIS

With the development of new computer tools, probabilistic analyses are relatively easy to perform (Scott, 2007). The standard deterministic equations for calculating the factor of safety (FS) are programmed into a spreadsheet, but instead of defining the input parameters as single constant values, they are defined as distributions. Then, instead of calculating a single value for the output factor of safety, a distribution of factor of safety is generated from numerous iterations using the so-called Monte-Carlo approach, whereby each of the input distributions is sampled in a manner consistent with its shape each time a factor of safety is calculated. This output distribution is used to determine the probability of the factor of

Table 2 – Input Distributions for Probabilistic Stability Analysis

Property	Distribution	Minimum	Peak	Maximum
Concrete Unit Weight, γ (lb/ft³)	Uniform	145	N/A	152
Tan φ'	Triangular	0.84	1.0	1.19
Cohesion, C' (lb/in²)	Triangular	50	100	150
Percent of Lift Surface Bonded	Triangular	33	50	67

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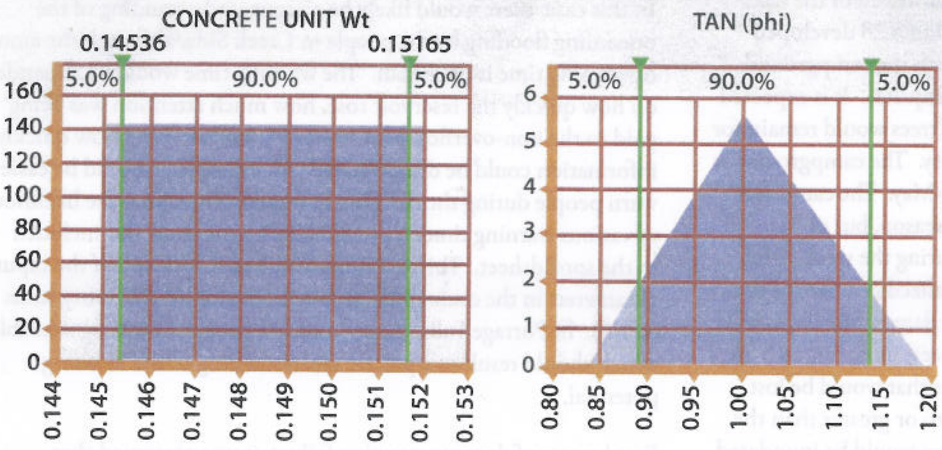
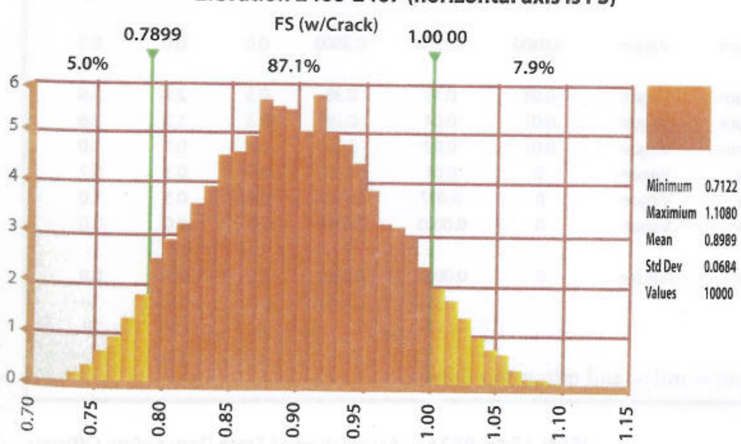
FIGURE 7 **Example Input Parameter Probability Distributions (input parameter on horizontal axis, concrete unit weight in units of kips/ft3)**



safety being less than a critical value. In most cases, this can readily be determined from the percentage of simulation points that are less than the critical factor of safety. For the purposes of this article, the probability of a factor of safety less than 1.0 was calculated. However, other values can be used. For example, if a dam is particularly susceptible to deformation damage, a larger value of safety factor may appropriately define the state at which "unsatisfactory performance" occurs.

The equations from *Design of Small Dams* were programmed into a spreadsheet for the probabilistic stability analysis. Due to the limited amount of data available for the dam concrete, distributions for material properties were assigned based on the experience of the team members, as shown in Table 2. Specifically, test data results from the Electric Power Research Institute were reviewed (EPRI, 1992) and strength values were appropriately reduced based on the understanding of joint clean-up and concrete placement. A uniform *(equally likely anywhere between a minimum and maximum value)* or a triangular probability distribution (*bounded by a lower and an upper value, with the most likely value given as the peak of the distribution*) was used to represent the input values. Examples of the resulting input distributions are shown on Figure 7. Estimates of the minimum and maximum lift joint bond percentage were made by assuming two additional holes were drilled, and both cores were either unbonded (minimum percentage) or bonded (maximum percentage), respectively. The analysis assumes the strength parameters selected from the distributions apply to the entire potential sliding surface, and therefore these distributions must be representative of the entire surface and not localized areas.

FIGURE 8 **FS output Distribution Lift Joint Sliding for Reservoir Elevation 2466-2467 (horizontal axis is FS)**



A Monte-Carlo analysis with 10,000 iterations was performed using the analysis spreadsheet and input distributions. The load ranges and probability of each load range is already accounted for in the first node of the event tree. The probability of exceeding the concrete tensile strength for each load range is estimated in the second node. The conditional failure probability, given that the concrete tensile strength is exceeded, is needed for the third node at each load range. Therefore, a simulation was run for each load range. The probability of being at a given reservoir elevation within each load range varies and is less likely toward the upper end of the load range. For example, in the upper load range, reservoir elevation 2466 occurs three times more frequently than reservoir elevation 2467. A general distribution was set up to account for this, and the reservoir elevation was selected from this general distribution for each iteration in the spreadsheet Monte-Carlo calculations. A similar procedure was used for the other load ranges. The resulting output distribution of factor of safety (FS) is shown on Figure 8 for the upper load range (reservoir elevation 2466 to 2467). The Monte-Carlo results indicate about 92 percent of the calculated factors of safety are less than 1.0. Similar analyses indicate a probability of FS<1.0 of 0.37 for the middle load range (reservoir elevation 2464 to 2466). None of the calculated factors of safety in the Monte-Carlo simulation were less than 1.0 for the lower load range (reservoir elevation 2462 to 2464). Therefore a different method, based on a term called the "reliability index", was used to estimate the probability of safety factor less than 1.0. This method is not described in detail here. Those interested in further explanation of the method are referred to Scott (2007). This analysis resulted in an estimated probability of FS<1.0 of about 0.0015 for the lower load range. Since the structure is relatively long compared to its height, three-dimensional influences would be limited and the probability of FS<1.0 was taken as the conditional probability of failure for all load ranges.

The Monte-Carlo simulation program displays what are called "rank sensitivity coefficients" for each input distribution. The larger the absolute value of the rank sensitivity coefficient, the more sensitive the results are to that particular input distribution (see Scott, 2007). The results were most sensitive to the friction angle input distribution for the higher load range where crack propagation was likely, and most sensitive to the cohesion input distribution in the lower load range where cracking was limited. Typically, parametric studies would be performed, varying the distributions to which the results were most sensitive. However, since relatively broad and conservative ranges were selected for the input distributions, no additional sensitivity studies were run in this case.

CONSEQUENCES OF DAM FAILURE

A campground is located about four miles downstream of the dam within the river canyon. This campground includes 28 developed campsites. A dam failure flood wave would reach the campground in about 15 minutes and would inundate all campsites. It is expected that the water would rise relatively rapidly, but trees would remain for refuge and campers could climb the hill to safety. The campground is essentially empty from mid-October until mid-May. The campsites are typically full on weekends during camping season, but on average, only about one-fourth are occupied during the week. This information could be used to arrive at an annualized estimate of the number of campers that would be exposed to a dam failure flood wave and to estimate potential loss of life consequences; however, such an estimate would over predict the number of lives that would be lost for the following reason: During a flood equal to or greater than the maximum historical flood, most of the campsites would be inundated with a foot or two of water from spillway releases. The dam operators are required to evacuate the campground before releasing this much water. In addition, releases from the spillway would increase at a relatively slow rate, providing campers an opportunity to leave the campground prior to a dam failure event if they did not receive a warning. With this in mind, the team judged that camping activities would be curtailed, and that all campers would leave during a large flood.

At the mouth of the canyon another 10 miles downstream is the town of Creek Side, with a permanent population of about 200 people. Hydraulic inundation analysis indicates that about a third of the town's structures and residents are located adjacent to the river where inundation flows would likely wash houses off their foundations; the remainder of the structures and residents are up a little higher on terraces adjacent to the river where the buildings would be flooded, but would likely remain in place. A dam failure flood wave would reach this town in about 1 hour and 15 minutes. The town of Portage Falls, with a population of about 25,000 is more than 50 miles downstream. The flood wave travel time is about 8 hours, and due to flood wave attenuation and widening of the river valley, only about 10 percent of the town near the river would be flooded by shallow and slowly rising water.

The spreadsheet shown in Table 3 was set up to evaluate the dam failure consequences for sliding of the non-overflow section prior to dam overtopping using the fatality rates proposed by Graham (1999). In this case, there would likely be a vague understanding of the oncoming flooding by the people in Creek Side, although the amount of warning time is uncertain. The warning time would be dependent on how quickly the reservoir rose, how much attention was being paid to the non-overflow area, reservoir forecasts, and how quickly information could be disseminated (for example, it would be easier to warn people during the day than at night). Therefore, the likelihood of various warning times was estimated by the team and included in the spreadsheet. This technique can be used for any of the input parameters in the evaluation (as long as the total probability sums to 1.0). In Portage Falls, there should be hours of warning available which should result in a precise understanding of the flooding potential.

For the case of dam overtopping failure, it was presumed that warnings and evacuations would commence as soon as flood forecasts and reservoir projections indicated the dam would overtop. Therefore, the population at risk would receive more than an hour of warning and would have a clear understanding of what was about to descend upon them. This would change the fatality rates accordingly, resulting in an estimated loss of life between 0 and 2 with a best estimate of 1 for an overtopping failure.

RISK ESTIMATES

The event tree branch estimates and consequences described above were entered into the event trees shown on Figures 2 and 3. Individual branch probabilities are determined by multiplying the probabilities from left to right through the tree; the total annual failure probability for each potential failure mode is determined by summing the individual branch probabilities for branches that end in failure.

Since there is a load range common to the two potential failure modes described above (reservoir elevation 2466 to 2467), a common

Table 3 – Consequence Estimates for Lift Joint Sliding

Reach	PAR	Distance (mi)	Travel Time (min)	Warning Time (min)	Warning Time Probability	Severity	Understanding	Fatality Rates low	Fatality Rates medium	Fatality Rates high	Life Loss low	Life Loss medium	Life Loss high
Campground	0	4	15	0	1	Medium	Vague	0.0300	0.1500	0.3500	0.0	0.0	0.0
Creek Side	66	14	75	0	0.25	Medium	Vague	0.03	0.15	0.35	0.5	2.5	5.8
	66	14	75	15 to 60	0.5	Medium	Vague	0.01	0.04	0.08	0.3	1.3	2.6
	66	14	75	>60	0.25	Medium	Vague	0.01	0.02	0.06	0.1	0.3	1.0
	134	14	75	0	0.25	Low	Vague	0	0.01	0.02	0.0	0.3	0.7
	134	14	75	15 to 60	0.5	Low	Vague	0	0.007	0.015	0.0	0.5	1.0
	134	14	75	>60	0.25	Low	Vague	0	0.0003	0.0006	0.0	0.0	0.0
Portage Falls	2500	>50	480	168	1	Low	Precise	0	0.0002	0.0003	0.0	0.5	0.8
											1	5	12
Total	2700												

Note: PAR = Population at Risk, mi = miles, and min = minutes

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