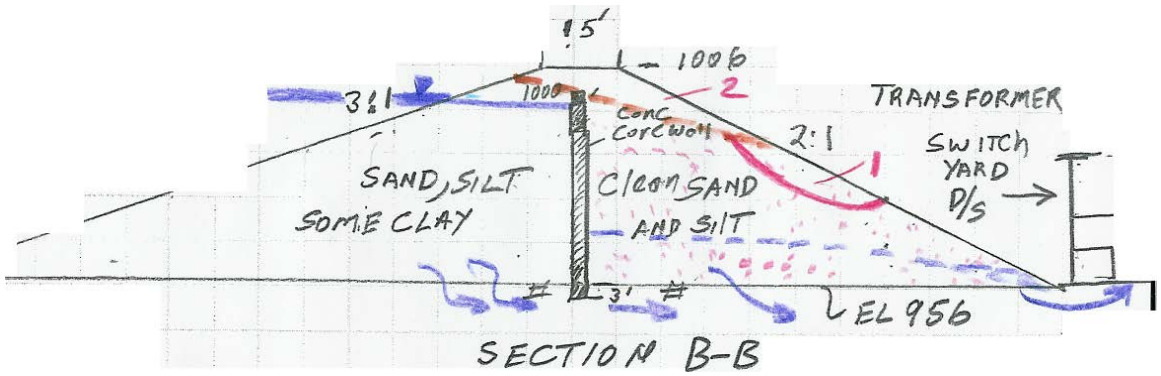
X-1. Possible Exercise Solutions

Section I-3, Potential Failure Mode Analysis

Evans Creek Dam

Potential Failure Mode 1 – Piping of sand and silt from embankment founded on rock

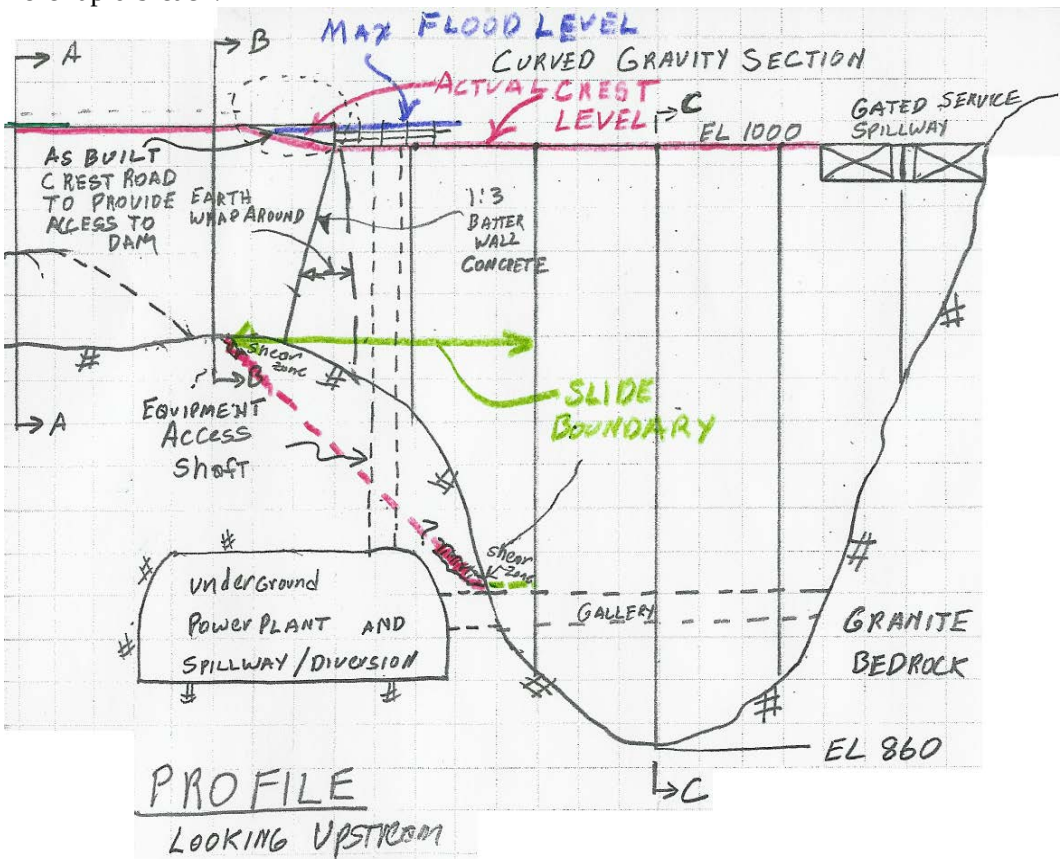
Seepage from beneath the embankment section gradually washes out the sand at the embankment contact, and causes periodic slumping and steepening of the downstream face, reducing the embankment cross section and allowing a slide to take place under a high water condition that leads to overtopping erosion, most likely in a location near the concrete corewall interface, and breach.



Adverse/ "More Likely" Factors	Favorable/ "Less Likely" Factors
Unprotected seepage exit	Seepage flow is monitored
Fines not captured by flume	No visual evidence of collapse noted to date
Seepage flow is significant	Water levels must be high to initiate internal erosion
	Movement of sand-size particles through joints in granite unlikely
	No visual evidence of fines movement noted to date
	Potential slip plane on downstream face likely shallow, would be noticed before embankment failure imminent

Potential Failure Mode 2 – Overtopping of the embankment dam due to major floods in excess of the gated spillway and powerplant capacity or a lesser flood with debris blockage

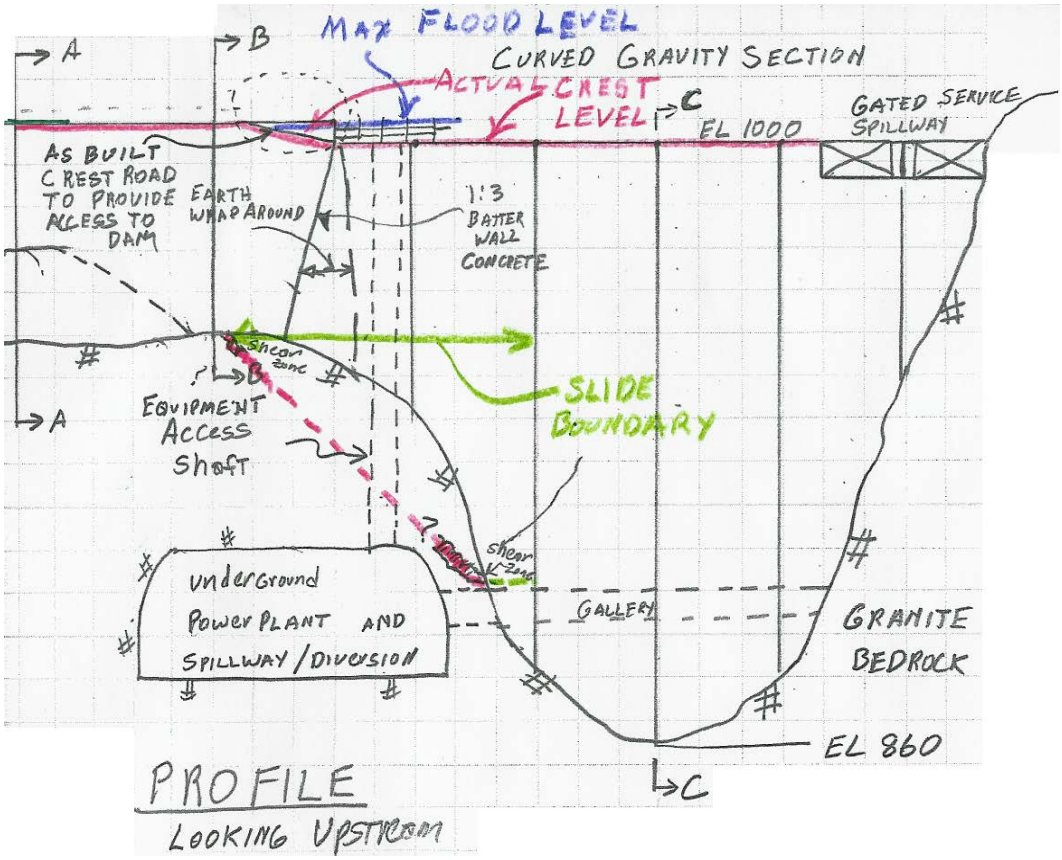
When floods in excess of the gated spillway capacity and powerplant discharge (or lesser with debris blockage of spillway) begin to overtop the concrete dam, they would also overtop the edge of the embankment section where the crest road was cut down to provide vehicle access to the spillway. The downstream shell would begin to erode, exposing the core wall. With loss of downstream support, the corewall would fail and/or the embankment would slump, and flow over the embankment would rapidly breach the dam. Flow over this section would cause loss of transmission capability and thus loss of the capability of plant to pass 5000 ft^3/s discharge leading to more overtopping and a more rapid breach.



Adverse/ "More Likely" Factors	Favorable/ "Less Likely" Factors
Overtopping can occur at relatively low flows	Crest Road is paved, reducing likelihood of initial downcutting
Debris is present that could block spillway	Concrete core wall could delay breach development
Sand / gravel embankment fill is erodible	Only a small zone is vulnerable, intervention may be possible
Switch-yard can fail wiping out generation and reducing outlet capacity	

Potential Failure Mode 3 - Wedge failure of concrete gravity dam foundation due to slide of upper right abutment

A potential foundation wedge exists under the right two blocks of the concrete gravity dam. The plunge of the intersection of the shear zone and a vertical joint is up towards the d/s and into the right abutment (or into the structure). Increased uplift and increased driving forces from a sustained high water condition or earthquake could initiate wedge sliding and rupture of the dam as it moves downstream with the wedge.



Adverse/ "More Likely" Factors	Favorable/ "Less Likely" Factors
Discontinuities defining block exist	Curved shape of dam may put additional thrust on shear and reduce potential for sliding
Water level observations indicate abutment pressures are likely high enough for significant uplift forces	The railing is in alignment, indicating no major movements have occurred
Crack occurred at shear on first filling indicating possible small movement	The shear strength of the vertical joint side plane could be high
No stability analyses of this condition have been performed to indicate relative stability of block	There is a drainage gllery and curtain, which historically have been very effective in mitigating this type of failure mode

Cobb Creek Levee

PFM 1 – Piping of sand and silt from foundation at Boils State Park

Seepage from existing boils continues during a flood. The seepage becomes dirty carrying sand and silt. Backward erosion continues, and the clayey levee acts as a roof. The backward erosion continues until a pipe is formed and breaks through to the river side of the levee forming a continuous pipe. Gross enlargement of the pipe continues until the roof collapses and the crest of the levee degrades and the levee is overtopped.

Adverse/ "More Likely" Factors	Favorable/ "Less Likely" Factors
Evidence of past seepage and piping initiation (Sand boils exist) Buried channels with fine sand and silt are likely below the levee The levee has only been loaded to 60 percent of its height Sand boils have occurred at other locations Vegetation obscures the toe of the levee	Sand boils have occurred in the past without failure Sand boils can be flood fought The locals are familiar with fighting sand boils most recently in 1995

PFM 2 – Operational Failure of the Highway 17 Closure

It has been 17 years since the post and panel structure was set in place. Flood water rises rapidly and the pieces cannot be found to set the closure structure. By the time this is realized there is no time for sand bagging because of the width of the opening and the rapid rise of the flood waters. The flood water begins flowing through the opening in the flood wall inundating eastern Ernieton.

OR

It has been 17 years since the post and panel structure was set in place. The city workers with experience in setting the floodwall in place have retired and moved out of the area or they have passed away. The post and panel closure structure across highway 17 is set in time, but the workers are unfamiliar with the pieces and important braces are not installed properly. The flood water rises to about four feet up on the closure structure when it suddenly collapses. A wave of flood water quickly inundates the eastern part of Ernieton.

Adverse/ "More Likely" Factors	Favorable/ "Less Likely" Factors
It has been 17 years since the structure was set Operational directions for setting the structure may have been lost The river is flashy with a small drainage basin which reduces the time to react The closure structure would likely need to be set during a rain storm The pieces of the structure are stored across town	Post and panel systems are relatively easy and quick to setup People may have time to evacuate from Ernieton if the closure structure is not installed People will work heroically to save their town The closure structure will likely show signs of distress prior to failure, allowing time for warning and possibly bracing the structure

PFM 3 – Collapse of the CMP Drainage Pipe

Collapse of the deteriorated CMP drainage pipe leads to an open erosional pipe through the embankment. The open erosional pipe enlarges and collapses, leading to degradation of the levee crest and overtopping.

Adverse/ "More Likely" Factors	Favorable/ "Less Likely" Factors
Heavy corrosion noted in the last inspection Previous sinkhole adjacent to the pipe, with unknown backfill The CMP is > 50 years old The drain pipe is in a rural area and the collapse may not be noticed prior to a flood The pipe has not been video inspected	The pipe is in a rural area which is far from town and will allow time for evacuation or other means to deal with the flooding The pipe is near the upstream end of the basin The levee is inspected prior to or early during every flood event, which could allow time for flood fighting The locals have flood fought before and are familiar with sand bagging The pipe is only 48 inches in diameter

PFM 4 – A large flood exceeding the levee capacity occurs

A large flood approaching a 1/1,000 annual exceedance probability occurs and overtops the levee by up to about 1.5 feet. The levee is too long to sand bag its entire length, and the floodwall cannot be sandbagged along its crest. Burtvile experiences flood depths of up to 2 feet along most streets, with occasional areas with depth of up to 3 feet near storm drain inlets. Ernieton experiences severe flooding with floodwaters up to 15 feet deep.

Adverse/ "More Likely" Factors	Favorable/ "Less Likely" Factors
The drainage system is small and a large stalled storm can dump sufficient rain in the drainage basin in less than 24 hours to cause overtopping of the levee The drainage basin is considered flashy The levee embankments are erodible	Large storm will be forecast which allows time for evacuation It takes a rare storm event (more remote than 1/500) to overtop the levee

Section I-4, Semi-Quantitative Risk Estimates

Evans Creek Dam

Potential Failure Mode 1 – Piping of sand and silt from embankment founded on rock

Failure Likelihood – Low to Moderate: There is no direct or indirect evidence (observed movement of fines, sink holes, etc.) to suggest this potential failure mode is likely. Therefore, the evidence is weighted toward unlikely. However, with the potentially unprotected seepage exit, relatively high seepage volumes, and inability to detect fines movement it cannot be ruled out and some weight must be given to the potential for it to develop.

Consequence Category – 2: It is expected that this potential failure mode will take some time to develop and there will likely be indications that it is in progress (e.g. slumping on the downstream face) such that there should be time to initiate warnings. Even though the embankment represents a small height of the dam, downstream property damage is likely, and loss of life is possible due to difficulties in warning and evacuating all exposed individuals.

Potential Failure Mode 2 – Overtopping of the embankment dam due to major floods in excess of the gated spillway and powerplant capacity or a lesser flood with debris blockage

Failure Likelihood – High: Up to 9,000 cfs can be passed prior to dam overtopping. It is presumed that this likely corresponds to a flood with an annual exceedance probability between 1/1,000 and 1/10,000. Once overtopping occurs to any significant extent, it is likely that the switchyard will be inundated and flows through the powerplant lost. At that point the overtopping depth and embankment erosion will increase rapidly. With debris blockage of the spillway, the likelihood would go up. However, the probability of debris blockage along with the continued release of 5,000 cfs through the powerplant prior to overtopping would likely not change the category.

Consequence Category – 2 to 3: Since this is a flood-related potential failure mode, it is likely that there will be some chance for flood-forecasting, warning, and evacuation. Campers may also leave the campground due to inclement weather. It is likely that this would reduce the potential for life loss although there still is the possibility that some life loss would occur (Level 2). If the forecast and warning is not robust, and if flooding at the campground is not sufficient to cause its evacuation, the life loss could be higher – up to Level 3. This would lead to the need to get more information about flood forecasting, warning, and evacuation.

Potential Failure Mode 3 - Wedge failure of concrete gravity dam foundation due to slide of upper right abutment

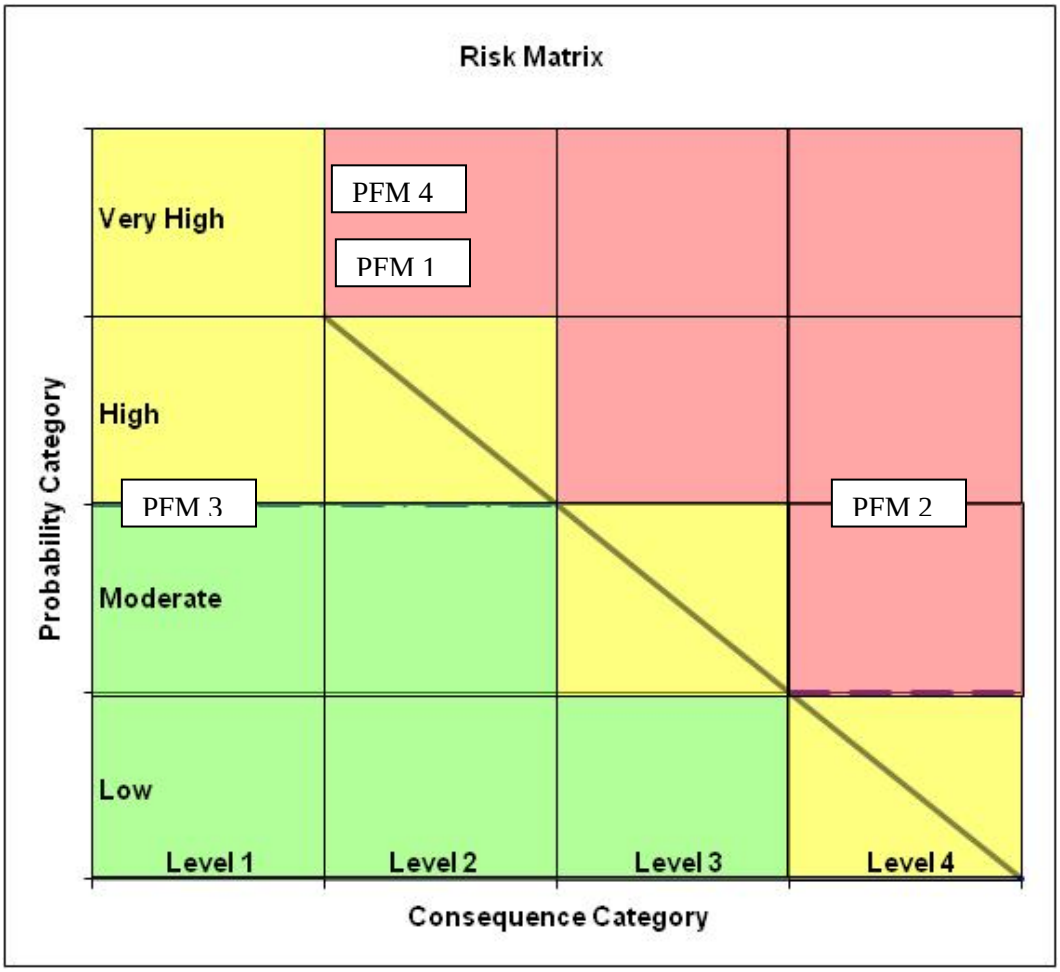
Failure Likelihood – Low to Moderate: There is no compelling evidence to suggest this might occur, but the flaw in terms of a well-defined potentially removable foundation block. This would normally place it in the Moderate category. However, the well drained abutment afforded by the drainage gallery and drain holes likely drives this lower.

Consequence Category – 3: This failure mode would be brittle and would likely occur quickly. It is unlikely there would be significant warning for anyone in the campground. This, along with the potential for large flows and life loss where permanent buildings are located, would put this in Category 3.

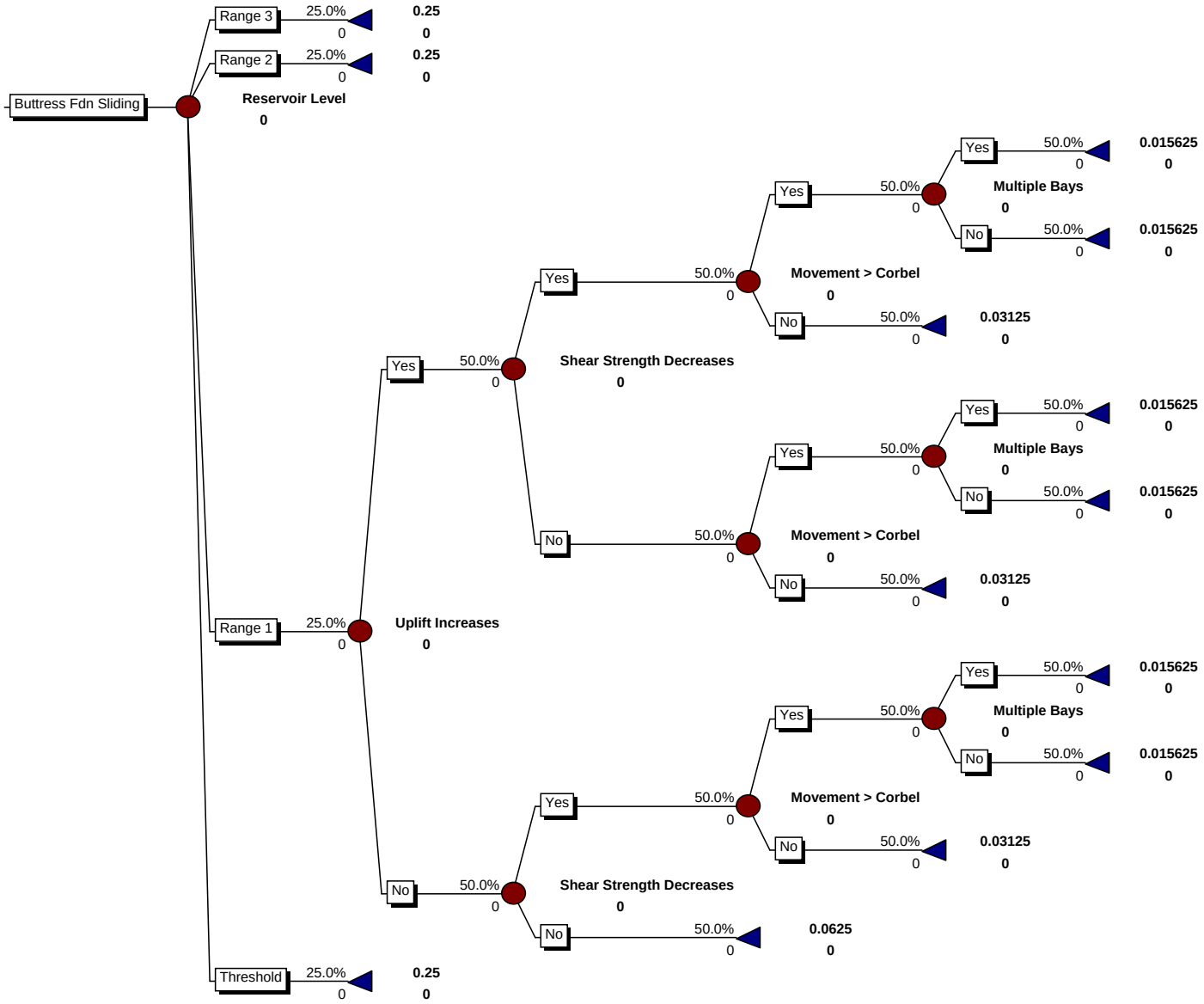
Cobb Creek Levee

For this exercise, a detailed analysis of each potential failure mode was not conducted, but rather a relative ranking was made using the risk matrix. The following chart shows the possible relative ranking of potential failure modes for the Cobb Creek Levee example. PFM 2 (highway closure) is the highest risk due the higher life loss potential (large flows directly into Ernieton especially if closure collapses), but PFM 4 (flood overtopping) and PFM 1 (foundation piping) most likely have higher failure likelihood due to overtopping flood frequency and previously observed boils, respectively (but with

fewer consequences). PFM 3 (CMP collapse) may be detected before it fully develops and occurs in a more remote area with lower flows and the potential for warning.



Section I-5, Event Trees



Section I-6, Subjective Probability

Item	Answer
Abraham Lincoln's age at death	56
Length of the Nile River (in miles)	4,187
Number of nations in NATO	28
Number of studio albums released by the Beatles	13
Diameter of the moon (in miles)	2,160
Weight of an empty Boeing 747 (in pounds)	390,000
Year in which Leonardo da Vinci was born	1452
Gestation period of an African Elephant (in days)	660
Air distance from London to Sydney (in miles)	10,562
Deepest known point in oceans (in feet)	36,198

Section I-7, Probabilistic Stability Analysis

Given a mean calculated factor of safety of 1.46 with a standard deviation of 0.26 and a standard normal distribution, what is the probability of a factor of safety of less than 1? What is the probability if the standard deviation is only 0.16?

Solution: How far away from the mean is a factor of safety of 1? The difference between the mean and the value representing poor performance, divided by the standard deviation is the number of standard deviation units from the mean, or the reliability index β. Then use the Z table to look up the portion of the distribution below the Z value, and subtract from 1 to get the portion of the distribution above the value (which is then the failure probability). Note that the Excel function NORMSDIST also gives the Z value.

$\beta = 1.46-1/0.26=1.77$

On the Z table look down the left hand column to 1.7, then across under the 0.7 to find the value of 0.9616. To find the percent of the distribution with a factor of safety of less than 1, subtract 0.9616 from 1, and multiply by 100. Answer = 3.8% probability of factor of safety (FS) < 1.0.

For a standard deviation of 0.16, the answer is 0.2% probability of FS<1.0.

Section I-8, Combining and Portraying Risks

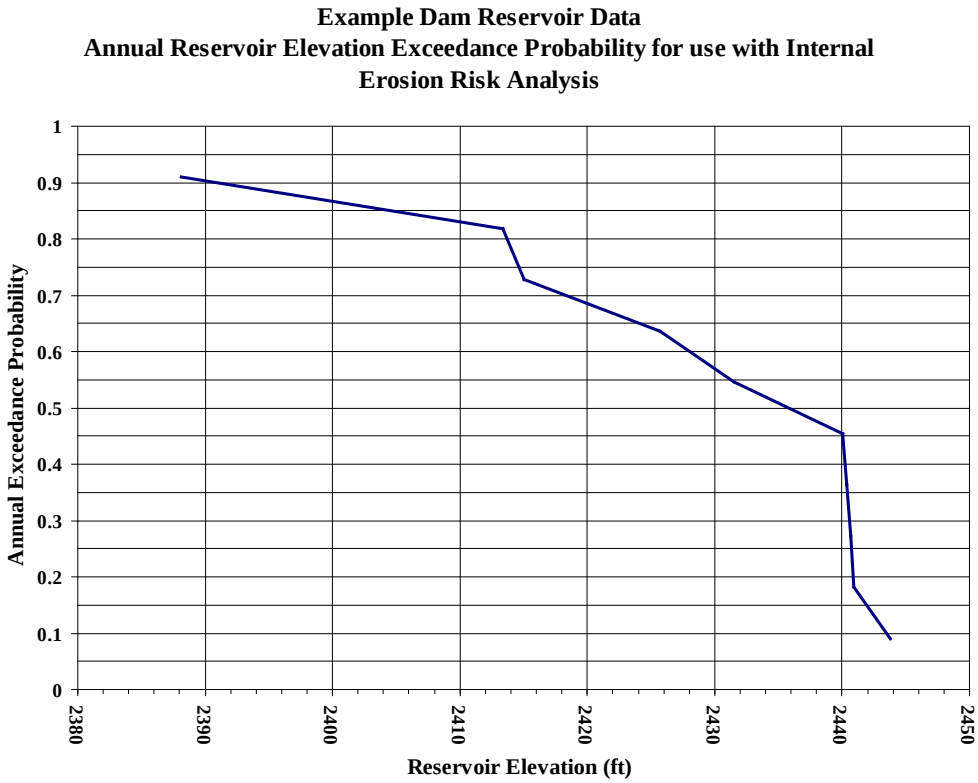
	p(f)	1 - p(f)	New P(f)
Sliding at base of gravity spillway section	0.23	0.77	0.16
Seepage erosion through embankment wing above core	0.14	0.86	0.10
Radial gate trunnion friction arm buckling	0.06	0.94	0.04
Spillway wall overtopping erosion and headcutting	0.31	0.69	0.22
Stilling basin failure and undermining erosion	0.17	0.83	0.12
Total p(f)	0.91		
Total p(no failure)	0.36		
System Failure Probability	0.64		
Ratio of System Failure Probability and Total p(f)	0.71		
% Error	41%		

Section II-1, Reservoir Exceedance Curve Example

The first step is to rank the data. In this case the elevations are ranked from smallest to largest. Next, an annual probability is assigned to each of the 10 events using the Weibull plotting position, exceed prob = i/(n+1), where i is from 10 to 1 and n = 10. See the table below.

Calendar Year	**Maximum Water Surface Elevation (ft)**	**i**	**exceedance probability**
2000	2388.10	10	0.909090909
2006	2413.35	9	0.818181818
2002	2415.00	8	0.727272727
2003	2425.75	7	0.636363636
1999	2431.52	6	0.545454545
2004	2440.04	5	0.454545455
2007	2440.38	4	0.363636364
2008	2440.71	3	0.272727273
2001	2440.94	2	0.181818182
2005	2443.79	1	0.090909091

A graph of the relationship is shown below.



Next, one can determine reservoir exceedance probability for the range of 2415-2440 by subtracting the two probabilities, as shown below.

Calendar Year	Maximum Water Surface Elevation (ft)	i	exceedance probability		
2002	2415.00	8	0.727272727	prob	2440
2004	2440.04	5	0.454545455	0.272727	2415

Section IV-1, Erosion of Rock and Soil

Headcut Erodibility Index Calculation for Rock

You are at a site where there is a granite formation located immediately downstream of your spillway. Due to the weathered condition of the rock, there is concern that erosion could occur during a high discharge. You refer to the construction documents and the team geologist and have obtained the following information:

- The material of concern is granite with a uniaxial compressive strength of 20,000 psi.

- The rock quality designation (RQD) is 50 percent.
- The material appears to be jointed in a four joint set.
- The joints are planar and smooth, with a tight joint separation, and the walls are slightly altered with sandy particles.
- The blocks appear to have a length to thickness ratio of 1:2 and the blocks dip downward into flow at 80 degrees.

Using this information, and the tables found in Chapter IV-1, what is the headcut erodibility index for the material?

Solution:

$$K_h = M_s K_b K_d J_s$$

- Convert 20,000 psi to MPa
  - (20,000 psi)(1 Mpa / 145 psi) = 137.9 MPa
- M_s = UCS (when UCS > 10 MPa)
  - Ms = 137.9 MPa
- $K_b = RQD / J_n$
  - $K_b = (50)/4.09$ (J_n obtained from table IV-1-1)
  - $K_b = 12.225$
- $K_d = J_r / J_a$
  - $K_d = 1.0 / 2.0$ (J_r and J_a obtained from tables IV-1-2 and IV-1-3)
  - $K_d = 0.50$
- $J_s = 0.6$ (J_s obtained from table IV-1-4)
- $K_h = (137.9)(12.225)(0.50)(0.6) = 506$

Section IV-2, Flood Overtopping

An event tree was set up to estimate the annual probability of failure. For the initial case (Figure X-1-1), the conditional failure probability estimates for each branch were made by averaging estimates made at the lower and upper end of the matrix of the load combinations (for some branches there were four combinations of starting reservoir water surface elevation and frequency return period). A second set of estimates were made (Figure X-1-2) where the estimates were skewed to the lower corner or end of the load combinations. The estimate reduced for this case but not by a large amount.

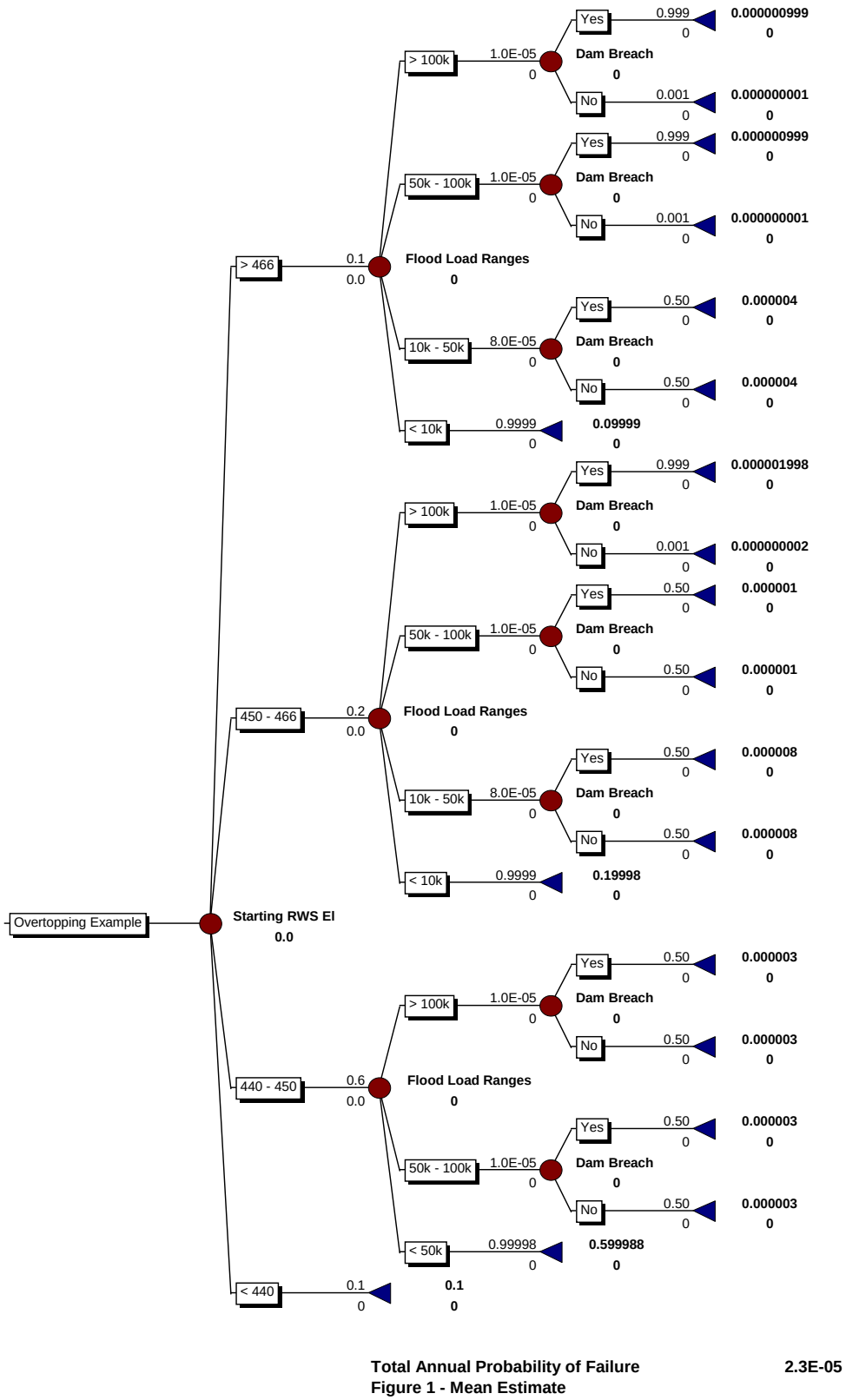

Overtopping Example
Starting RWS El
0.0
> 466
0.1
0.0
Flood Load Ranges
0
> 100k
1.0E-05
0
Dam Breach
0
Yes
0.999
0
0.000000999
0
No
0.001
0
0.000000001
0
50k - 100k
1.0E-05
0
Dam Breach
0
Yes
0.999
0
0.000000999
0
No
0.001
0
0.000000001
0
10k - 50k
8.0E-05
0
Dam Breach
0
Yes
0.50
0
0.000004
0
No
0.50
0
0.000004
0
< 10k
0.9999
0
0.09999
0
450 - 466
0.2
0.0
Flood Load Ranges
0
> 100k
1.0E-05
0
Dam Breach
0
Yes
0.999
0
0.000001998
0
No
0.001
0
0.000000002
0
50k - 100k
1.0E-05
0
Dam Breach
0
Yes
0.50
0
0.000001
0
No
0.50
0
0.000001
0
10k - 50k
8.0E-05
0
Dam Breach
0
Yes
0.50
0
0.000008
0
No
0.50
0
0.000008
0
< 10k
0.9999
0
0.19998
0
440 - 450
0.6
0.0
Flood Load Ranges
0
> 100k
1.0E-05
0
Dam Breach
0
Yes
0.50
0
0.000003
0
No
0.50
0
0.000003
0
50k - 100k
1.0E-05
0
Dam Breach
0
Yes
0.50
0
0.000003
0
No
0.50
0
0.000003
0
< 50k
0.99998
0
0.599988
0
< 440
0.1
0
0.1
0
Total Annual Probability of Failure
2.3E-05
Figure 1 - Mean Estimate

Figure X-1-1

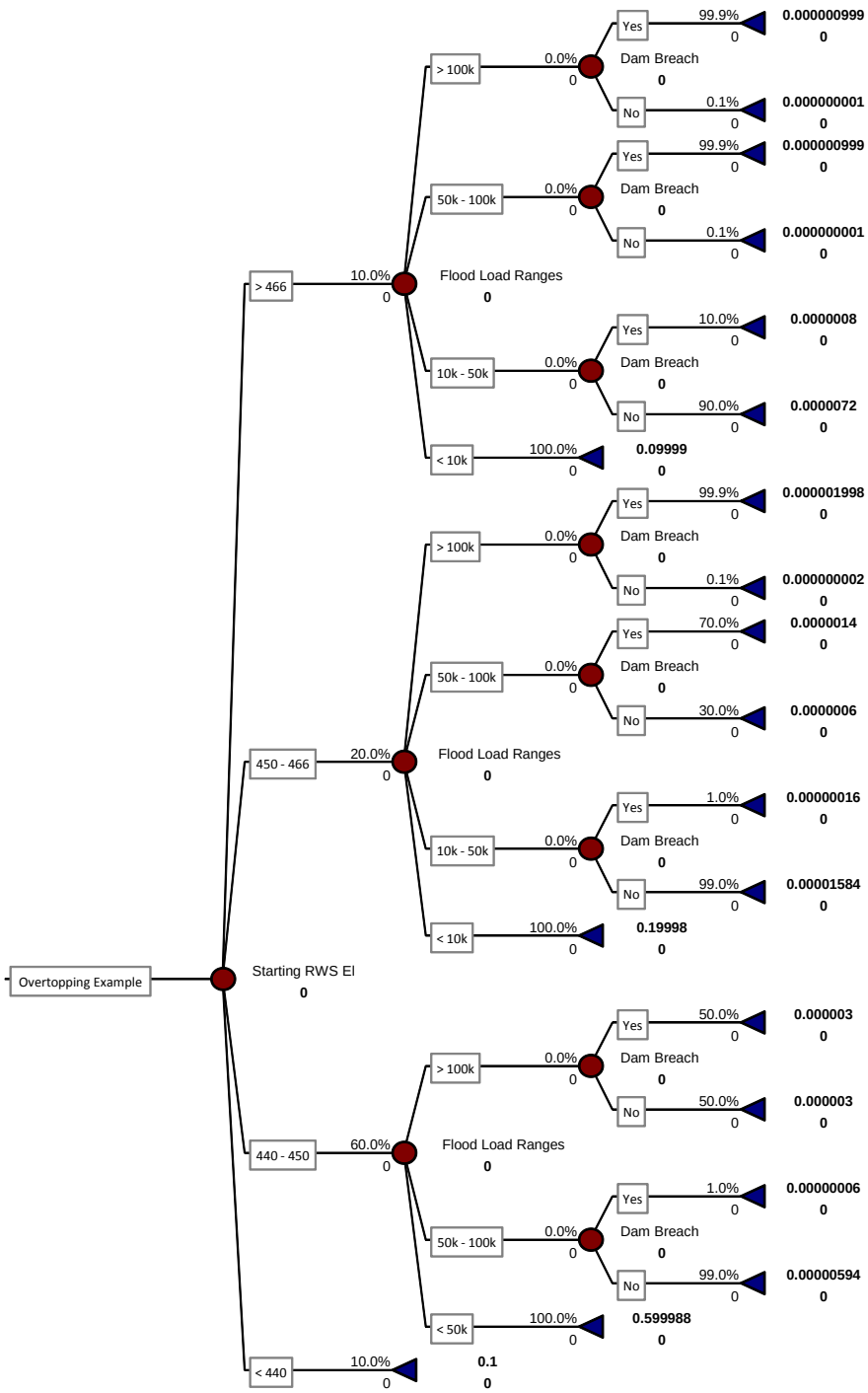

Overtopping Example
Starting RWS El
0
> 466
10.0%
0
Flood Load Ranges
0
> 100k
0.0%
0
Dam Breach
0
Yes
99.9%
0
0.000000999
0
No
0.1%
0
0.000000001
0
50k - 100k
0.0%
0
Dam Breach
0
Yes
99.9%
0
0.000000999
0
No
0.1%
0
0.000000001
0
10k - 50k
0.0%
0
Dam Breach
0
Yes
10.0%
0
0.0000008
0
No
90.0%
0
0.0000072
0
< 10k
100.0%
0
0.09999
0
450 - 466
20.0%
0
Flood Load Ranges
0
> 100k
0.0%
0
Dam Breach
0
Yes
99.9%
0
0.000001998
0
No
0.1%
0
0.000000002
0
50k - 100k
0.0%
0
Dam Breach
0
Yes
70.0%
0
0.0000014
0
No
30.0%
0
0.0000006
0
10k - 50k
0.0%
0
Dam Breach
0
Yes
1.0%
0
0.00000016
0
No
99.0%
0
0.00001584
0
< 10k
100.0%
0
0.19998
0
440 - 450
60.0%
0
Flood Load Ranges
0
> 100k
0.0%
0
Dam Breach
0
Yes
50.0%
0
0.000003
0
No
50.0%
0
0.000003
0
50k - 100k
0.0%
0
Dam Breach
0
Yes
1.0%
0
0.00000006
0
No
99.0%
0
0.00000594
0
< 50k
100.0%
0
0.599988
0
< 440
10.0%
0
0.1
0
Total Annual Probability of Failure
9.4E-06
Figure 2 - Weighted Average Estimate

Figure X-1-2

Levee Exercise

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

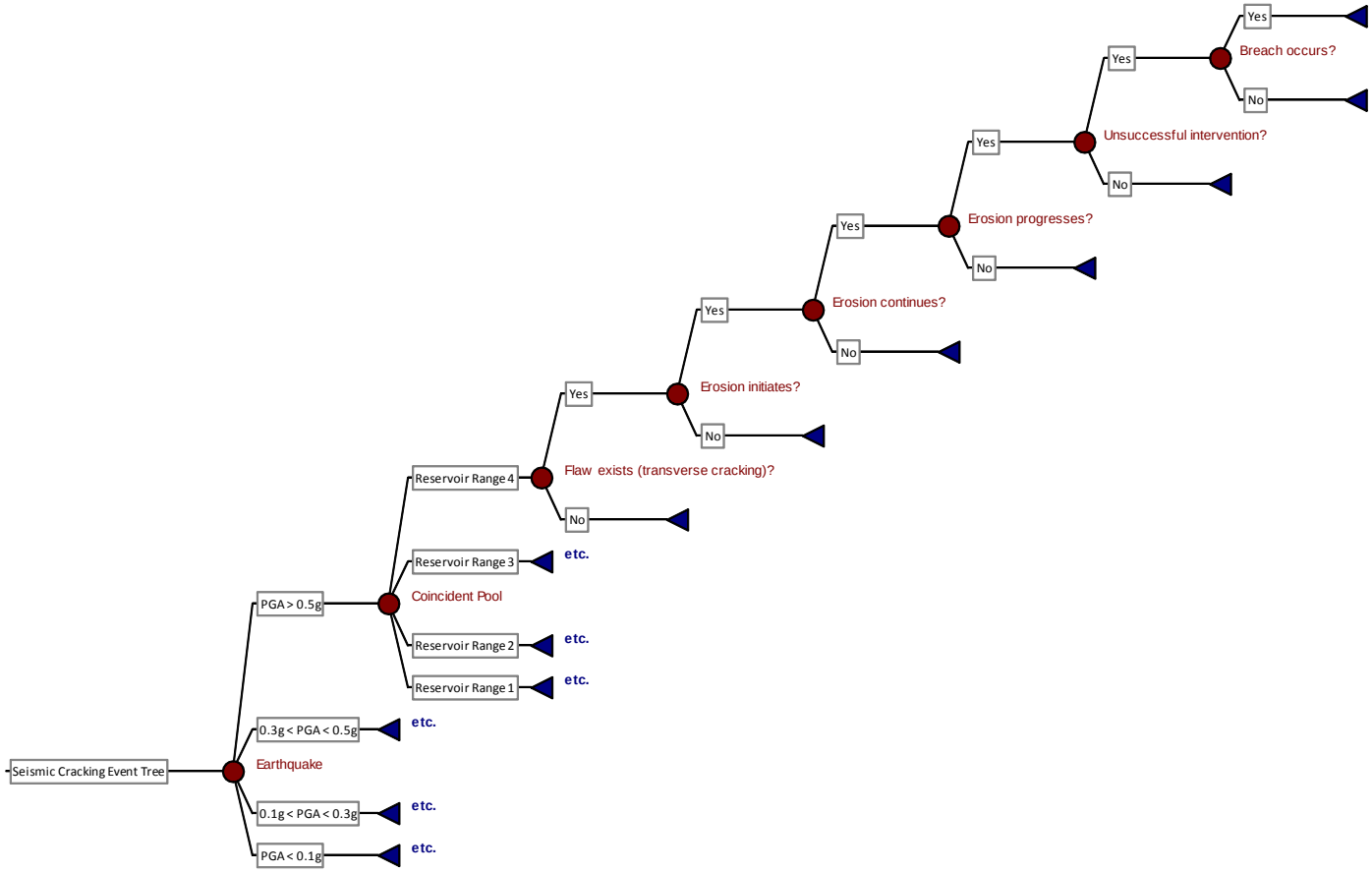
Riverine	Hurricane	WCA
Slope protection	Slope protection	Slope protection
Type of grass	Type of grass	Type of grass
Grass coverage	Grass coverage	Grass coverage
Depth of flow	Wave height	Depth of flow
Duration of flow	Wave frequency	Duration of flow
Low spots	Duration of storm	Wave height
Variation in vegetation	Slope of levee	Wave frequency
Bare spots	Slope of foreshore	Duration of storm
Gullies	Wind speed	Slope of levee
Water craft induced waves	Wind direction	Slope of foreshore
HPTRM use	Low spots	Wind speed
	Variation in vegetation	Wind direction
	Bare spots	Vegetation within the WCA
	Gullies	Low spots
	Water craft induced waves	Variation in vegetation
	HPTRM use	Bare spots
		Gullies
		Water craft induced waves
		HPTRM use

Section IV-4, Internal Erosion

Representative Base Soil Grading	Estimated Proportion of Filter Gradings Falling into Each Filter Erosion Category				
	No Erosion (NE)	Some Erosion (SE)	Excessive Erosion (EE)	Continuing Erosion (CE)	Sum
Fine base soil grading (represents 5% of grading curves on fine side)	20%	60%	20%	0%	100%
Average base soil grading (represents 90% of grading curves)	20%	70%	10%	0%	100%
Coarse base soil grading (represents 5% of grading curves on coarse side)	20%	70%	10%	0%	100%
Calculation of probabilities	(0.05*0.2) + (0.9*0.2) + (0.05*0.2) = 0.2	(0.05*0.6) + (0.9*70) + (0.05*0.7) = 0.695	(0.05*0.2) + (0.9*0.1) + (0.05*0.1) = 0.105	0[1]	1.0

[1]Note: Fell et al (2008) suggest that even though there are no filter gradations falling into the Continuing Erosion category, a small probability may be assigned to account for possibility of materials in the dam being coarser than indicated by the gradation curves, ranging from about 0.01 (when DF15 in the dam < 0.5 DF15 for the continuing erosion boundary) to 0.0001 (when DF15 in the dam < 0.1 DF15 for the continuing erosion boundary).

Section IV-6, Embankments Seismic

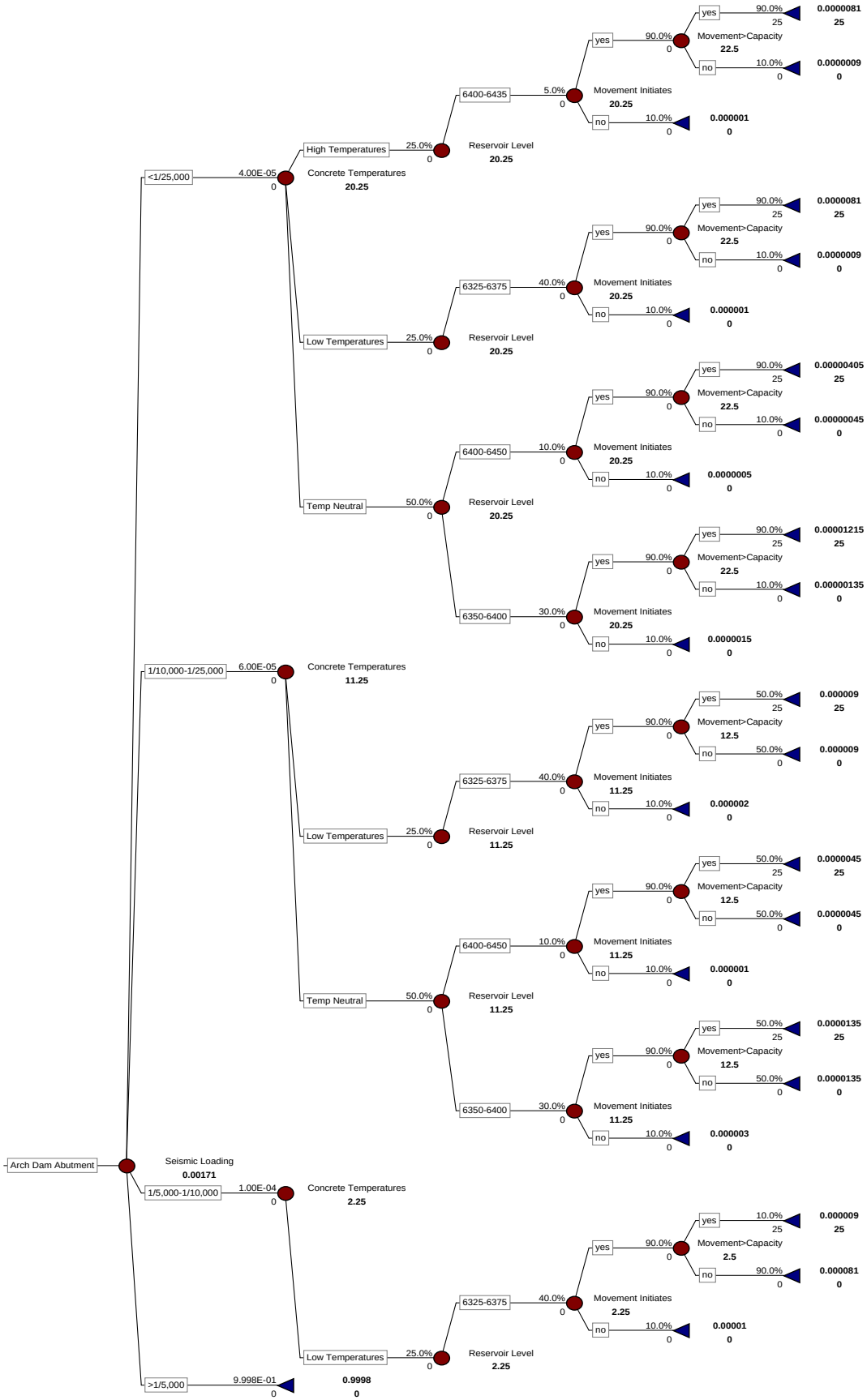


Section V-2, Concrete Gravity Dams

Weight =	64,400	lbf/ft	
Moment =	279,000	lbf-ft/ft	
Thickness =	16.2	ft	
Moment of Inertia =	354.29		B3^3/12
Water Depth @U/S face =	32.6	ft	
Water Pressure @ U/S Face =	2034.24	psf	B5*62.4
Negative stress tensile			
Stress @ Heel (no uplift - total stress)	-2403.29	psf	B1/B3-B2*(B3/2)/B4
Stress @ Heel (with uplift - effective stress)	-4437.53	psf	B8-B6
Converted to psi =	-30.82	psi	B9/144 tension
Concrete Compressive Strength =	3,500	psi	
Tensile Strength =	391.89	psi	1.7*B12^(2/3)
Reduce for large aggregate	352.70	psi	B13-0.1*B13
Reduce for vertical strength	282.16	psi	B14-0.2*B14
Reduce for lift joint	239.83	psi	0.85*B15

Section V-3, Concrete Arch Dams

The following event tree shows a possible way this could be developed. The important things are that the temperature conditions were added to the tree, and failure can occur during earthquake displacements.


Arch Dam Abutment
Seismic Loading
0.00171
<1/25,000
4.00E-05
0
Concrete Temperatures
20.25
High Temperatures
25.0%
0
Reservoir Level
20.25
6400-6435
5.0%
0
Movement Initiates
20.25
yes
90.0%
0
Movement>Capacity
22.5
yes
90.0%
25
0.0000081
25
no
10.0%
0
0.0000009
0
no
10.0%
0
0.000001
0
Low Temperatures
25.0%
0
Reservoir Level
20.25
6325-6375
40.0%
0
Movement Initiates
20.25
yes
90.0%
0
Movement>Capacity
22.5
yes
90.0%
25
0.0000081
25
no
10.0%
0
0.0000009
0
no
10.0%
0
0.000001
0
Temp Neutral
50.0%
0
Reservoir Level
20.25
6400-6450
10.0%
0
Movement Initiates
20.25
yes
90.0%
0
Movement>Capacity
22.5
yes
90.0%
25
0.00000405
25
no
10.0%
0
0.00000045
0
no
10.0%
0
0.0000005
0
6350-6400
30.0%
0
Movement Initiates
20.25
yes
90.0%
0
Movement>Capacity
22.5
yes
90.0%
25
0.00001215
25
no
10.0%
0
0.00000135
0
no
10.0%
0
0.0000015
0
1/10,000-1/25,000
6.00E-05
0
Concrete Temperatures
11.25
Low Temperatures
25.0%
0
Reservoir Level
11.25
6325-6375
40.0%
0
Movement Initiates
11.25
yes
90.0%
0
Movement>Capacity
12.5
yes
50.0%
25
0.000009
25
no
50.0%
0
0.000009
0
no
10.0%
0
0.000002
0
Temp Neutral
50.0%
0
Reservoir Level
11.25
6400-6450
10.0%
0
Movement Initiates
11.25
yes
90.0%
0
Movement>Capacity
12.5
yes
50.0%
25
0.0000045
25
no
50.0%
0
0.0000045
0
no
10.0%
0
0.000001
0
6350-6400
30.0%
0
Movement Initiates
11.25
yes
90.0%
0
Movement>Capacity
12.5
yes
50.0%
25
0.0000135
25
no
50.0%
0
0.0000135
0
no
10.0%
0
0.000003
0
1/5,000-1/10,000
1.00E-04
0
Concrete Temperatures
2.25
Low Temperatures
25.0%
0
Reservoir Level
2.25
6325-6375
40.0%
0
Movement Initiates
2.25
yes
90.0%
0
Movement>Capacity
2.5
yes
10.0%
25
0.000009
25
no
90.0%
0
0.000081
0
no
10.0%
0
0.00001
0
>1/5,000
9.998E-01
0
0.9998
0

Section V-4, Concrete Buttress Dams

Section	Demand [D] lb-in	Capacity [C] lb-in		D/C	P(rupture) Fig. 10-6 (B)
26-inch-thick	4.64E+05	1.90E+05	V	2.44	0.88
32-inch-thick	5.78E+05	2.40E+05	V	2.41	0.87

Section V-6, Spillway Piers Seismic

The magnification factor for the pier is determined from Figure V-6-3, using the period of the structure calculated below and 2 percent damping:

$$Ts = 0.000643 \times H^2/B = 0.000643 \times 1600/5 = 0.21$$

From Figure V-6-3, the magnification factor is 2.1. Figure X-1-3 depicts the inertial loading at the base of the pier for the 0.2g load case. The shear at the base of the pier is calculated below.

$$V = ½(150\ lb/ft^2 + 315\ lb/ft^2) \times 40 = 9300\ lb$$

The shear stress at the base of the pier is calculated below:

$$v = 9300/(60 \times 12) = 13\ lb/in^2$$

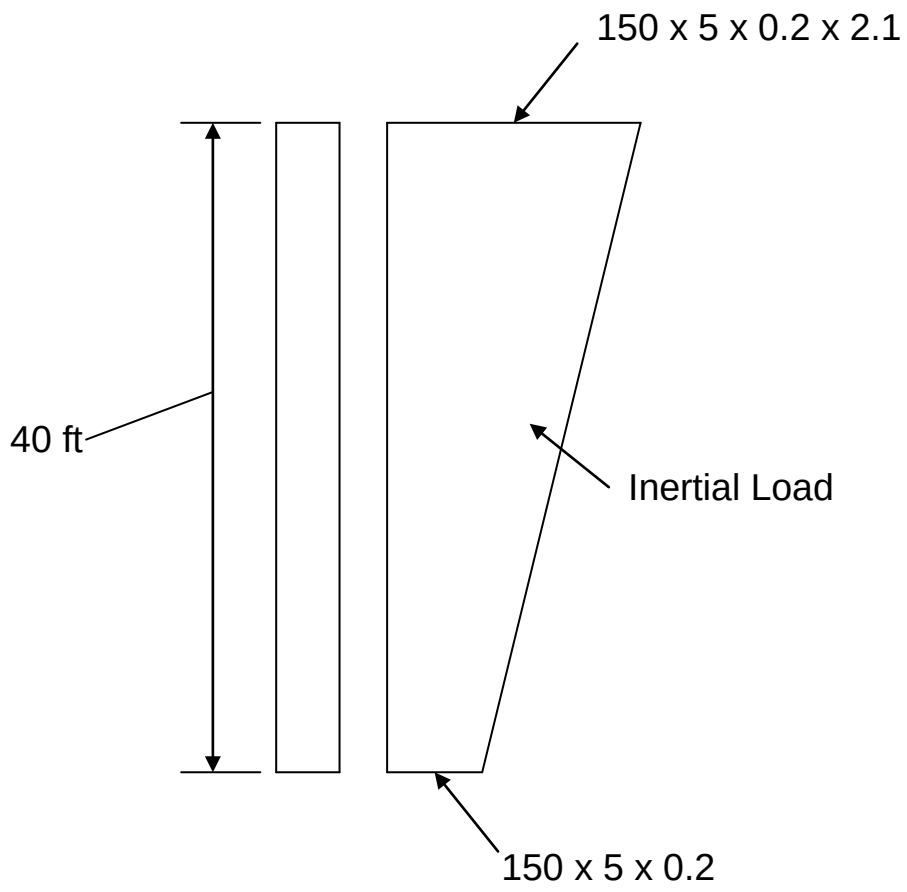


Figure X-1-3 – Loading on Pier

The following table summarizes the other load case results:

Shear Stresses at Base of Pier			
Recurrence Interval, yr	**Peak Horizontal Ground Acceleration**	**Total Shear Force at Base of Pier**	**Shear Stress at Base of Pier**
1000	0.2g	9300 lb	13 lb/in^2
5000	0.4g	18,600 lb	26 lb/in^2
10,000	0.5g	23,250 lb	33 lb/in^2
50,000	0.6g	27,900 lb	39 lb/in^2

Since the shear stresses are very low for all load cases and well below the stated shear capacity of 200 lb/in^2, the estimates for shear capacity being exceeded would all be very low (0.001).

Section V-7, Retaining Walls Seismic

The active earth pressure coefficients were obtained from Figure V-7-6 for a friction angle of 30°. The active earth pressure coefficients are shown in Table X-1-1. The total earth pressure (both static and dynamic) were calculated using the following equation:

$$P_{AE} = K_{AE}[1/2(\gamma_t(1 - k_v))H^2$$

The vertical ground acceleration was assumed to be 0. The shear at the base of the wall is calculated below.

$$V = K_{AE}[1/2(\gamma_t)]H^2 = K_{AE}[1/2(120)]40^2$$

The shear stress at the base of the wall is calculated below for the 1000-yr earthquake:

$$v = V/(24 \times 12) = 33{,}600/288 = 117 \text{ lb/in}^2$$

The following table summarizes the other load case results:

Table X-1-1 - Shear Stresses at Base of Pier				
Recurrence Interval, yr	**Peak Horizontal Ground Acceleration**	**K_{AE}**	**Total Earth Pressure Force**	**Shear Stress at Base of Pier**
1000	0.1g	0.35	33,600 lb	117
5000	0.2g	0.43	41,280 lb	143
10,000	0.3g	0.55	52,800 lb	184
50,000	0.4g	0.70	67,200 lb	233

Based on the calculated shear stresses above and the shear capacity of 200 lb/in^2, the following estimates that the shear capacity will be exceeded are as follows:

1000 yr – 0.001
5000 yr – 0.001
10,000 yr – 0.1
50,000 yr - 0.99

Section VI-1, Stagnation Pressures

The potential for initiating slab jacking can be evaluated by considering the jacking forces versus the resisting forces. Table X-1-2 provides an estimate of the potential jacking force, using Figure VI-1-4 (1/8-inch gap, 1/4-inch offset, sealed cavity representing an undrained condition). The total resisting force is the sum of the water depth, the weight of the concrete slab and the anchor bars. The flow depth varies for

each frequency flood. The concrete slab and anchor bar contributions are fixed and are as follows:

Concrete Slab – 1 foot x 150 lb/ft^3/62.4 = 2.4 feet of water
Anchor Bars – 40,000 lb capacity/100 ft^2/62.4 = 6.4 feet of water

Table X-1-2 – Slab Jacking Evaluation Due to Stagnation Pressures

Frequency Flood, yr	Flow Depth, Normal to Slope, ft	Flow Velocity, ft/s	Uplift Force, ft water	Total Resisting Force, ft water[1]	Net Uplift, ft water
1000	2.5	35	10	11.3	-1.3
10,000	4.6	42	15	13.4	1.6
100,000	8.1	47	19	16.9	2.1
1,000,000	10.5	50	22	19.3	2.7

[1] Uplift Force – Total Resisting Force

Table X-1-3 – Annual Probability of Chute Slab Jacking

Flood Load Range	Load Range Probability	Conditional Failure Probability	Annual Probability of Failure
> 1,000,000 yr	1E-06	0.9 to 0.99	9.4 E-07
100k – 1000k yr	9E-06	0.7 to 0.9	7.2E-05
10k – 100k yr	9E-05	0.5 to 0.9	6.3E-05
1k – 10k yr	9E-04	0.01 to 0.7	3.2E-04
< 1k yr	0.999	0	0
Total			5E-04

Conditional failure probability estimates were made considering the factors in Table X-1-4:

Table X-1-4 - Factors for Initiation of Slab Jacking

More Likely Factors	Less Likely Factors
Uplift forces exceed resisting forces in most cases	Chute Slab Reinforcement will hold slab together and prevent localized failure.
Offset extends across most of chute width	Uplift Pressures may dissipate away from joint.
Slab has separated from foundation allowing access for stagnation pressure flows	
Anchor bars may not develop full capacity	

Section VI-2, Spillway Chute Walls

The spillway chute is 20-foot wide with 10-foot high walls. The depth of flow for the frequency flood discharges can be determined from Q = VA, where Q is the discharge, V is the average flow velocity and A is the area of flow. Given Q and V, the area of the flow can be determined and then the depth of flow determined by dividing the flow area by the 20-foot chute width. The flow depths can then be compared to the 10 foot wall heights. Table X-1-5 shows the flow depth calculations. Table X-1-6 provides the annual probability of chute wall overtopping estimates and Table X-1-7 provides the factors considered in the estimates.

Table X-1-5 – Determination of Flow Depths

Frequency Flood, yr	Spillway Discharge, ft^3/s	Flow Velocity, ft/s	Flow Area, ft^2	Flow Depth, ft
1000	2000	40	50	2.5
10,000	7300	55	133	6.6
100,000	17,800	88	202	10.1
1,000,000	25,300	91	278	13.9

Table X-1-6 – Annual Probability of Chute Wall Overtopping

Flood Load Range	Load Range Probability	Conditional Failure Probability	Annual Probability of Damage
> 1,000,000 yr	1E-06	0.999	1.0 E-06
100k – 1000k yr	9E-06	0.5 to 0.99	6.7E-06
10k – 100k yr	9E-05	0.1 to 0.5	2.7E-05
1k – 10k yr	9E-04	0	0
< 1k yr	0.999	0	0
Total			**4E-05**

Note – Other nodes in the event tree will likely reduce the probability of the full failure developing.

Table X-1-7 - Factors for Chute Wall Overtopping	
More Likely Factors	**Less Likely Factors**
Walls are predicted to overtop for flows representing floods equal to and greater than the 100,000-year flood	Air bulking is not a factor.
	Cross-waves are not a factor.
	Large amounts of freeboard for 1000- and 10,000-year floods.

Section VI-3, Cavitation

A vertical ½-inch offset into the flow has been created in the spillway chute. From Table VI-1-3, for a slope of 0 and a rise of ½ -inch, the incipient cavitation value is about 1.7. Assume that cavitation damage occurs between one-fourth and one-sixth of this value, which indicates damage will initiate for flow cavitation indices between 0.4 and 0.3. Table X-1-8 summarizes the estimates for the initiation of cavitation damage. Table X-1-9 lists the major factors that were considered.

Table X-1-8 – Annual Probability of Initiation of Cavitation Damage			
Flood Load Range	Load Range Probability	Conditional Failure Probability	Annual Probability of Damage
> 1,000,000 yr	1E-06	0.999	9.9 E-07
100k – 1000k yr	9E-06	0.9 to 0.99	8.5E-06
10k – 100k yr	9E-05	0.1 to 0.9	4.5E-05
1k – 10k yr	9E-04	0.001 to 0.1	4.5E-05
< 1k yr	0.999	0	0
Total			**1E-04**

Note – Other nodes in the event tree will reduce the probability of the full failure developing.

Table X-1-9 - Factors for Initiation of Cavitation Damage	
More Likely Factors	**Less Likely Factors**
For larger floods, cavitation indices are at the point where initiation of damage is expected, based on model tests.	Minimal aeration may reduce potential for damage slightly.
Spillway flows for each frequency flood will maintained for an extended period.	
Flow is minimally aerated.	
Offset into the flow is continuous across most of chute.	

Section VII-1, Failure of Radial (Tainter) Gates under Normal Operational Conditions

- Load probability is 1.0 since reservoir reaches full pool every year.
- Gate operates probability is assumed to be 0.99, since gates are exercised annually when pool is full.
- Reduction Factor Due to Gate Arrangement/Condition – 0.1
- Reduction Factor Due to Inspections/Exercise – The gates are exercised annually and inspected every 3 years - 0.1
- Conditional Failure Probability Based on Interaction Ratio – From Table VII-1-7:
  - For as designed condition (0.99 probability of condition; critical IR = 0.6) – estimate of 0.001
  - For failed bushing condition (0.01 probability of condition; critical IR = 0.8) – estimate of 0.1
- The annual probability of failure is estimated to be 2E-05

Section VII-2, Drum Gates

Possible vulnerabilities

- Drain lines are old; if one fails, a gate could fill with water and inadvertently drop
- Gates are old and have not been painted/ maintained; failure of a weld or portion of the skin plate could result in filling and dropping of a gate
- Outlet valve can let out more water than can be let in; if outlet valve is accidentally opened, a gate would drop
- Remote operation means gates are operated without visual confirmation of proper position; if there is a glitch, it may not be detected
- Float chamber level is controlled by valve flow and does not account for seal leakage; if leakage is large, gate may slowly drop
- The gates are large; if one drops it could result in life-threatening flows at the campground

Section VII-3, Seismic Failure of Spilway Radial Tainter) Gates

Assume that the reduction factor for the gate arrangement/condition is 1.0 and the probability of unsuccessful intervention is 0.999.

Table X-1-10 – Seismic Failure of Spillway Radial Gate Exercise							
Reservoir Load Range	Reservoir Load Range Probability	Seismic Load Range	Seismic Load Range Probability	Conditional Failure Probability[1]	Mean Estimate for 1 Gate	Total Conditional Failure Probability[2]	Annual Probability of Failure
Reservoir El. > top of gates	0.1	>0.5g	0.0001	0.99	0.99	1.0	1.0E-5
		0.2 – 0.5g	0.0009	0.01 – 0.99	0.5	1.0	9.0E-5
Reservoir between top of gates and 20' up on gates	0.8	> 0.5g	0.0001	0.1 – 0.99	0.55	1.0	8.0E-5
		0.2 – 0.5g	0.0009	0.001 – 0.99	0.5	1.0	7.2E-4
Totals							9.0E-4

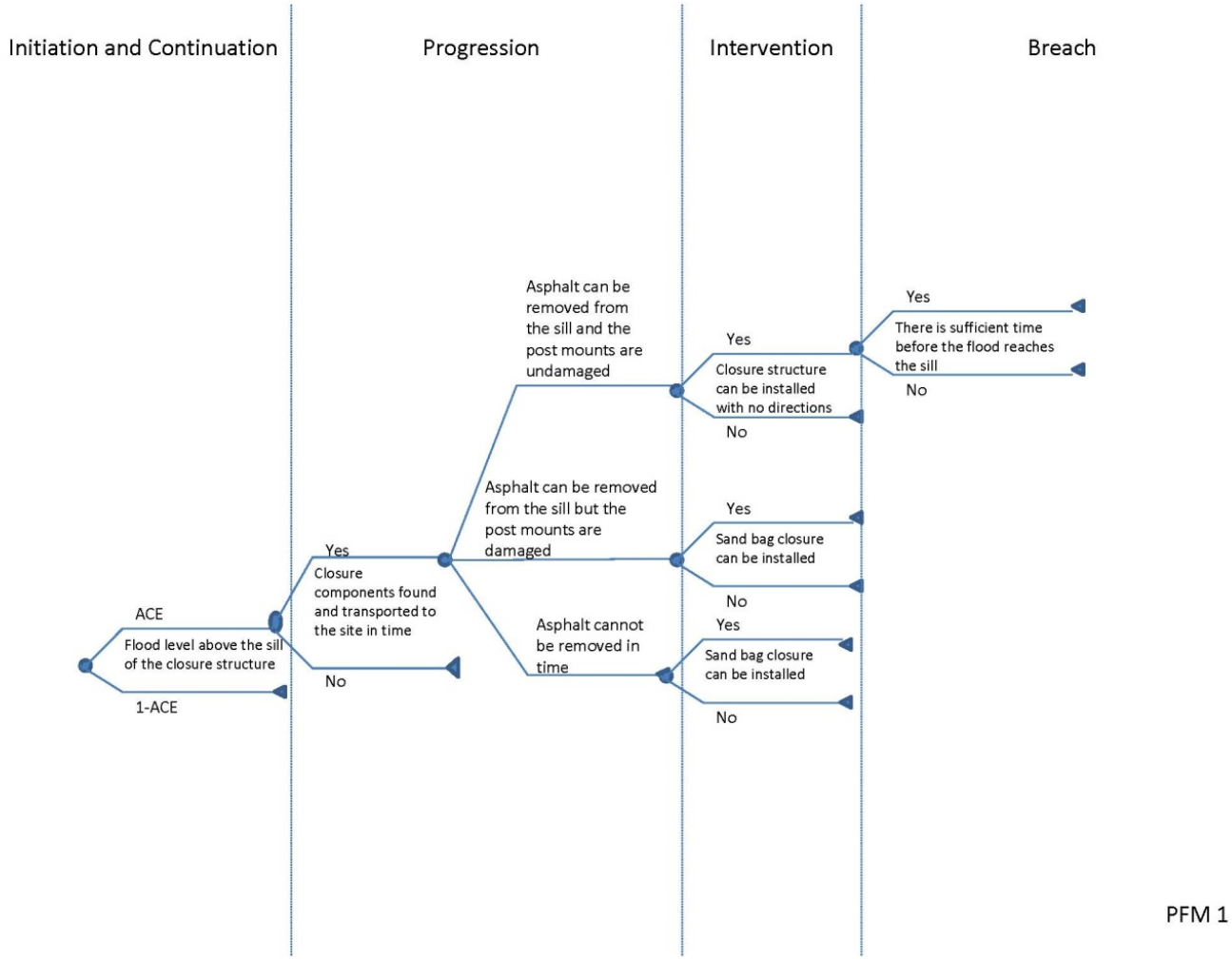
[1] From Table VII-3-2, using interaction ratio values
[2] From Table X-1-11

Table X-1-11 - Conversion of Single Gate Failure Probability to Total Gate Failure Probability				
Probability of Single Gate Failure	→	0.5	0.55	0.99
Number of Gates Failing	Equation for "x"gates failing	Probability of "x" gates failing	Probability of "x" gates failing	Probability of "x" gates failing
0	$1P^0(1-P)^2$	0.25	0.20	0.0001
1	$2P^1(1-P)^1$	0.50	0.50	0.0198
2	$1P^2(1-P)^0$	0.25	0.30	0.9801
Total		1.0 (∑ Failure Probability = 1.0)	1.0 (∑ Failure Probability = 1.0)	1.0 (∑ Failure Probability = 1.0)

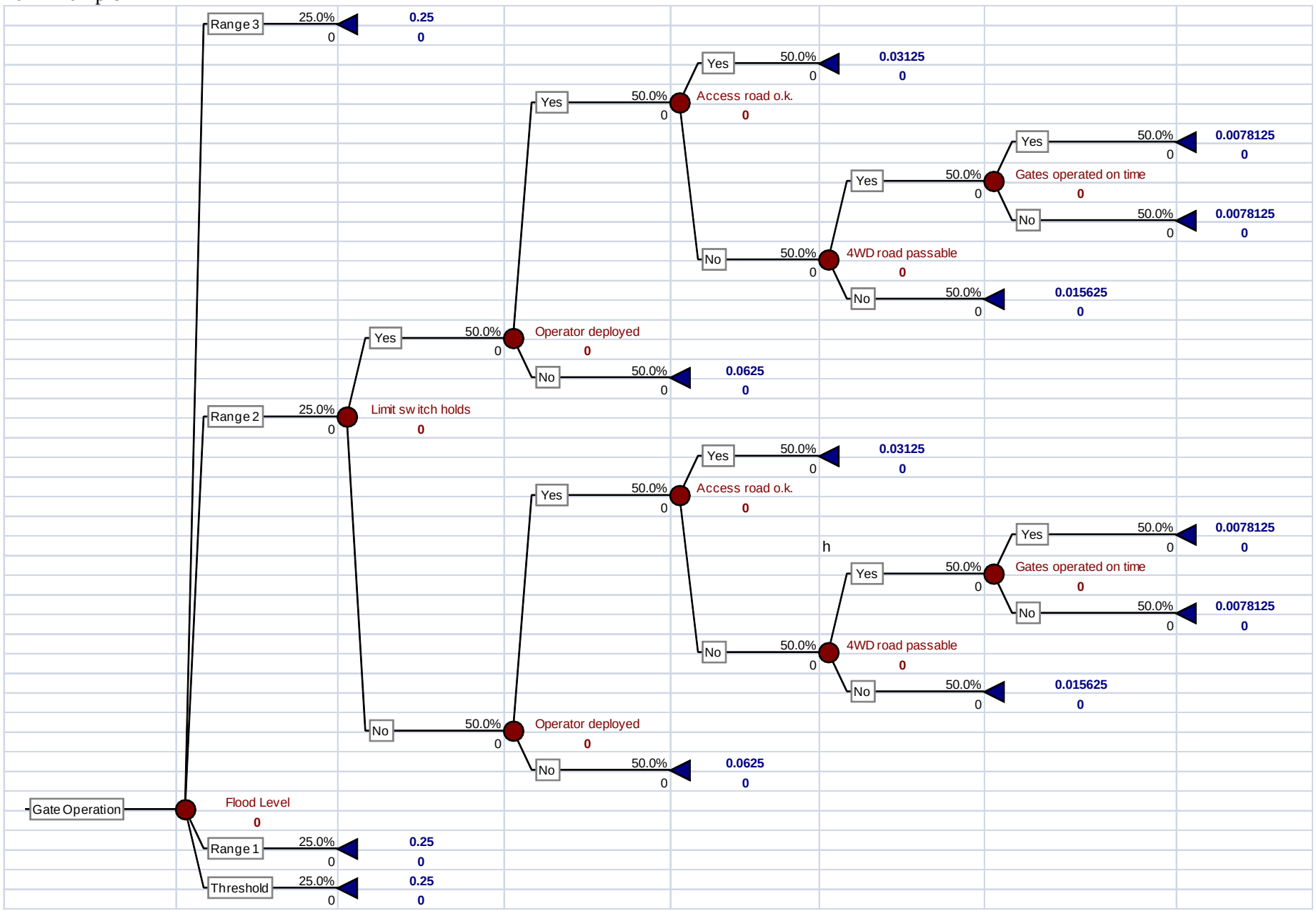
Table X-1-10 was used to set up the event tree for this failure mode. Conditional failure probability estimates were made using Table VII-3-2. The mean estimate for the range was used, which is conservative. Single gate failure probabilities from Table 1 were converted to a total failure probability as shown in Table X-1-11.

Section VIII-1, Operational Risks

Levee Example



Dam Example


Gate Operation
Flood Level
0
Range 3
25.0%
0
0.25
0
Range 2
25.0%
0
Limit switch holds
0
Yes
50.0%
0
Operator deployed
0
Yes
50.0%
0
Access road o.k.
0
Yes
50.0%
0
0.03125
0
No
50.0%
0
4WD road passable
0
Yes
50.0%
0
Gates operated on time
0
Yes
50.0%
0
0.0078125
0
No
50.0%
0
0.0078125
0
No
50.0%
0
0.015625
0
No
50.0%
0
0.0625
0
Range 1
25.0%
0
0.25
0
Threshold
25.0%
0
0.25
0
Yes
50.0%
0
Operator deployed
0
Yes
50.0%
0
Access road o.k.
0
Yes
50.0%
0
0.03125
0
No
50.0%
0
4WD road passable
0
Yes
50.0%
0
Gates operated on time
0
Yes
50.0%
0
0.0078125
0
No
50.0%
0
0.0078125
0
No
50.0%
0
0.015625
0
No
50.0%
0
0.0625
0
h

Section VIII-2, Landslides

Reservoir depth, D =	870	ft	
Slide volume, V =	350,000,000	cy	
Slide volume, V =	9,450,000,000	ft^3	
Distance from slide to dam, L =	870	ft approx	
SQRT(V/D^3)	3.788		
10^((L/D)/58)	1.040		
Wave Height =	443	ft approx	0.14*D1*D5/D6
Actual Height =	330	ft over dam	

Note: If roughly the mid-point of the slide is used as the distance from the dam to the slide, the predicted wave height at the dam is about 410 ft

Section VIII-3, Construction Risks

- Open Excavation
- Construction begins June 1
- Construction Duration is 6 months
- Baseline Annualized P(f) = 3.20E-06
- Using annualized p(f) from Table 27-3
- What is the increased risk during construction for this scenario?
- Answer should be in the form: Over the duration of construction, construction risks will increase by a factor of "x"

	Mar-Jun	July-Oct	Nov-Feb
Duration (Months)	1	4	1
Annualized p(f)	4.66E-05	2.02E-05	1.99E-05
Monthly p(f Over the duration of construction, construction risks will increase by a factor of)	3.88E-06	1.68E-06	1.66E-06
p(f) for Duration	3.88E-06	6.73E-06	1.66E-06
Sum	**1.23E-05**		
Annualized Baseline Risk	**3.20E-06**		
Baseline Risk (6 Months)	**1.60E-06**		
Multiplier	**7.7**		