US Army Corps
of Engineers®

Papillion Creek and Tributaries Lakes, Nebraska

General Reevaluation Report

Appendix L – Life Safety Analysis

June 2021

Omaha District
Northwestern Division

THIS PAGE INTENTIONALLY LEFT BLANK

Table of Contents

List of Tables

List of Figures

1 INTRODUCTION

This technical appendix is a summation of the life safety analysis completed for the optimized design of new levee and floodwall sections on the Little Papillion Creek and of Papillion Creek Dam Sites 10 and 19. According to Planning Bulletin (PB) 2019-04, issued 20 June 2019, "Studies that include existing and proposed levee systems and dams must take special care in evaluating the risk imposed by the infrastructure on the population downstream or in the leveed area." The goal of evaluating the life safety risk during the planning stage is to formulate, recommend, and implement cost effective plans to reduce the risk posed by the infrastructure to achieve all four Tolerable Risk Guidelines (TRGs). It also allows for advanced planning, including cost estimating, for preconstruction engineering and design (PED) activities should the project enter the implantation phase. In order to evaluate TRG 1 an abbreviated Semi-Quantitative Risk Assessment (SQRA) was conducted which was based on the most current project design, hydrology, hydraulic and geotechnical information provided in Appendix A, B, and C of the General Reevaluation Report. Additional hydrologic loading and consequence data the SQRA team used to assess the dams is provided in this appendix.

All elevation data in this report references the North American Vertical Datum of 1988 (NAVD88) unless otherwise specified.

The horizontal coordinate system used for civil modeling was NAD 1983 (2011) State Plane Nebraska FIPS 2600 (US Feet).

1.1 TOLERABLE RISK GUIDELINES

One of the main goals of planning studies that include proposed levee systems and dams is to achieve all four Tolerable Risk Guidelines (TRG) (USACE 2019a). Where TRGs are not currently met, measures and alternatives can be formulated to reduce risk and achieve all four TRGs. USACE considers risk to life safety related to the TRGs from two perspectives, societal life risk and individual life risk.

1. Societal life risk is the risk of widespread or large-scale catastrophes from the inundation of a leveed or dammed area that would result in a negative societal response. In general, society is more averse to risk if multiple fatalities were to occur from a single event. In contrast, society tends to be less averse to risks that result from many events resulting in only one or two fatalities, even if the total losses from the small events is larger than that from the single large event.

2. Individual life risk is represented by the probability of life loss for the identifiable person or group by location that is most at risk of loss of life due to a levee or dam breach. Individual life risk is influenced by location, exposure, and vulnerability within a leveed or dammed area.

The four TRGs are provided below, with some discussion of how the local project sponsor, the Papio-Missouri River NRD, addresses the TRGs:

- **TRG 1 – Understanding the Risk**. The first tolerable risk guideline involves considering whether society is willing to live with the risk associated with the levee system or dam to secure the benefits of living and working in the leveed or dammed area. In other words, it answers a basic question: are the risks commensurate with the benefits? The life risk matrix is used to evaluate compliance with TRG 1. Risks that plot above the societal life risk line are considered unacceptable except in exceptional circumstances. The alternatives considered in this study plot below the societal life risk line.

- **TRG 2 – Building Risk Awareness**. The second tolerable risk guideline involves determining that there is a continuation of recognition and communication of the levee and/or dam risk, because the risk associated with levee systems and/or dams are not broadly acceptable and cannot be ignored. The rationale for addressing TRG 2 will be determined qualitatively and may be met through USACE levee and dam safety program activities and/or the levee or dam sponsor's activities, which includes risk communication. If the non-federal sponsor is very active in building risk awareness, then the levee system or dam project addresses TRG2. The sponsor coordinates with the Corps, FEMA, the National Weather Service and the US Geological Survey to spread awareness of available mapping and electronic flood warning systems.

- **TRG 3 – Fulfilling Daily Responsibilities**. The third tolerable risk guideline involves determining that the risks associated with the levee system and/or dam are being properly monitored and managed by those responsible for managing the risk. The rationale for addressing TRG 3 will be determined qualitatively and may be met through USACE levee and dam safety program activities and/or levee or dam sponsor activities. TRG 3 can be met through demonstrated monitoring and risk management activities. This would include an active operation and maintenance program, visual monitoring (documented regular inspections), updated and tested emergency plan, instrumentation program, and interim risk reduction measures plan. The non-federal sponsor addresses TRG3 through their day-to-day operations. The Papio-Missouri River NRD continues to be one of the most active and responsive non-federal levee and dam sponsors in the Omaha District portfolio. They have continuously maintained good status in the Rehabilitation and Inspection Program. When concerns are noted in periodic inspections that might push the rating to unacceptable, the sponsor is very quick to respond to them.

- **TRG 4 – Actions to Reduce Risk**. The fourth guideline is determining if there are cost effective, socially acceptable, or environmentally acceptable ways to reduce risks from an individual or societal risk perspective. If it is determined that there are no cost effective or acceptable ways to further reduce risks, USACE may consider this an exceptional circumstance and therefore might consider the levee and/or dam risk to be tolerable even if the life safety risk exceeds the associated tolerability guideline under TRG 1. The non-federal sponsor addresses TRG4 to the extent practicable. The sponsor actively commissions and participates in making their levees NFIP compliant and building flood control dam sites. The non-federal sponsor also routinely updates their long-range implementation plan (LRIP) which facilitates the execution of a long range flood control plan as funding becomes available. They have funded levee and dam evaluation studies, assessments of economic impacts of levee and dam failure, and studies of the vulnerability of the Papillion Creek System to climate change. The sponsor was instrumental in initiating this feasibility study to help reduce flood risk further in the system.

2 PAPILLION CREEK FUTURE WITHOUT-PROJECT CONSEQUENCE ANALYSIS

A baseline assessment of the consequences associated with flooding in a future without-project condition at various magnitude events was conducted to better understand life safety risk within the watershed without the proposed dams, levee, and floodwall improvements in place. While the results of this analysis may not be directly comparable to those in the subsequent sections covering the findings of dam and levee/floodwall breach modeling, they are provided to better illustrate the existing risk in the watershed as required in ER 1105-2-101.

The tables below provide the results of the future without-project condition modeling using HEC's LifeSim (HEC-LifeSim). The 2 percent, 1 percent, 0.5 percent, and 0.2 percent AEP events were modeled based on the same methodology and assumptions regarding Hazard Identification, Hazard Communication Delay, Warning Issuance Delay, Warning Diffusion, and Protective Action Initiative described in Section 4.5.4 and 5.5.4 of this report. Uncertainty around the timing of the event (day or night) was also included in the modeling. An annual life loss estimate was then calculated based on the single-event life loss estimates and their probabilities of occurring. Note that the study area for this effort is the same used for dam and levee/floodwall breach modeling in the sections below and omits portions of the Big Papillion Creek and West Papillion Creek watersheds included in the damage analysis modeling. There may be additional life safety risk in these areas not captured in the HEC-LifeSim modeling, however it is thought that the additional risk is relatively low as it includes less developed portions of the Papillion Creek watershed.

Population at risk (PAR) is defined as the number of people downstream of a dam that would be subject to inundation risk. PAR and life loss estimates were generated for daytime and nighttime inundation scenarios.

Table 1. Future Without-Project Estimated Population at Risk by AEP Event

AEP Event	Population at Risk	
	Day	Night
2%	2,456	665
1%	6,642	1,563
0.5%	13,878	3,589
0.2%	18,627	5,068

The table below provides summary results of the modeling for future without-project condition. Information provided includes range of depths, number of inundated structures, population at risk and median life loss estimates.

Table 2. Future Without-Project Estimated Downstream Information by AEP Event and Time of Day (Minimal Warning)

AEP Event	Depth Ranges (feet)	Number of Structures	Population at Risk		Median Life Loss	
			Day	Night	Day	Night
2%	0-2	272	2,456	665	0	0
1%	0-3	630	6,642	1,563	1	0
0.5%	0-3	1,100	13,878	3,589	20	1
0.2%	1-4	1,600	18,627	5,068	37	2

Based on the single-event median life loss estimates presented in the table above, an annual life loss estimate of .01 to .18 was calculated assuming a minimal warning scenario.

Life loss results are presented below with five number statistics in order to understand the potential range of life loss. The estimated life loss statistics for the two generic warning issuance scenarios (described in more detail in the sections below) are summarized in the following tables.

Table 3. Estimated Life Loss for 2% AEP Event

Statistic	Life Loss Estimates (2% AEP)			
	Minimal Warning Scenario		Ample Warning Scenario	
	Day	Night	Day	Night
95th Percentile	10	1	8	2
75th Percentile	2	0	1	0
Median	0	0	0	0
25th Percentile	0	0	0	0
5th Percentile	0	0	0	0

Table 4. Estimated Life Loss for 1% AEP Event

Statistic	Life Loss Estimates (1% AEP)			
	Minimal Warning Scenario		Ample Warning Scenario	
	Day	Night	Day	Night
95th Percentile	11	1	8	2
75th Percentile	3	0	2	0
Median	1	0	1	0
25th Percentile	0	0	0	0
5th Percentile	0	0	0	0

Table 5. Estimated Life Loss for 0.5% AEP Event

Statistic	Life Loss Estimates (0.5% AEP)			
	Minimal Warning Scenario		Ample Warning Scenario	
	Day	Night	Day	Night
95th Percentile	31	3	27	7
75th Percentile	26	1	19	3
Median	20	1	12	1
25th Percentile	11	0	6	0
5th Percentile	2	0	0	0

Table 6. Estimated Life Loss for 0.2% AEP Event

Statistic	Life Loss Estimates (0.2% AEP)			
	Minimal Warning Scenario		Ample Warning Scenario	
	Day	Night	Day	Night
95th Percentile	60	5	51	14
75th Percentile	48	3	36	6
Median	37	2	24	3
25th Percentile	23	1	11	1
5th Percentile	4	0	2	0

3 LITTLE PAPILLION CREEK LEVEE AND FLOODWALL SECTIONS

3.1 BACKGROUND AND PURPOSE

The purpose of this life safety analysis for the new floodwall and levee sections on the Little Papillion Creek is to determine the life loss associated with the proposed levee and floodwall designs vs the existing conditions. In order to do this, features and assumptions used in the Levee Screening Tool (LST) were used to quantify life loss for the selected alternative. The existing condition life loss was calculated using the LST depth fatality curve.

The primary inputs used to calculate loss of life in the LST are the delineated leveed area and the annual probability of inundation due to overtopping and breach prior to overtopping. The LST provides a simplified method to calculate annualized life loss given a population at risk and frequency at which the levee will be loaded and overtopped. Estimation of specific breach parameters, location of the breach, and the breach hydraulics are not considered in the LST analysis.

3.1.1 STUDY AREA

The life safety analysis for the tentatively selected plan incorporates 3 damage reaches (LP5, LP6, and LP7) and the upper end of reach LP8 which mainly consists of the Baxter arena and surrounding parking lots (Figure 1). Additional analysis and optimization after the TSP and Draft Report resulted in changes to the reach delineations and numbering resulting the reaches LP5R,

LP6R & LP6L; LP7R & LP7L; LP8R & LP8L; LP9R & part of LP9L being the area evaluated for life safety performance of the proposed levee (Figure 2). Even though the damage reaches were updated, the land use and population at risk from the TSP analysis still capture essentially the same proposed leveed areas, and therefore no update was made to the life safety damage curves derived from the TSP analysis. Aside from a few more residential homes being left out of the final life-loss analysis in the upstream levee alignment (which would only be exposed to shallow overland flooding), the majority of the leveed area that is not captured by the original damage curves consist of parking lots for the Baxter Arena. It is not expected that including these areas in the levee life safety analysis would increase life loss substantially.

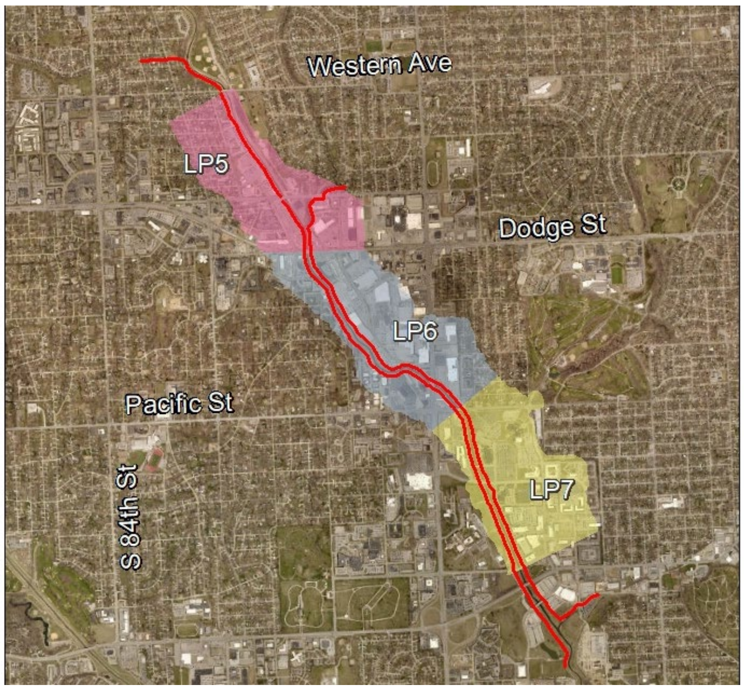


Figure 1. Tentatively Selected Plan Levee Alignment and Life Safety Analysis Reaches

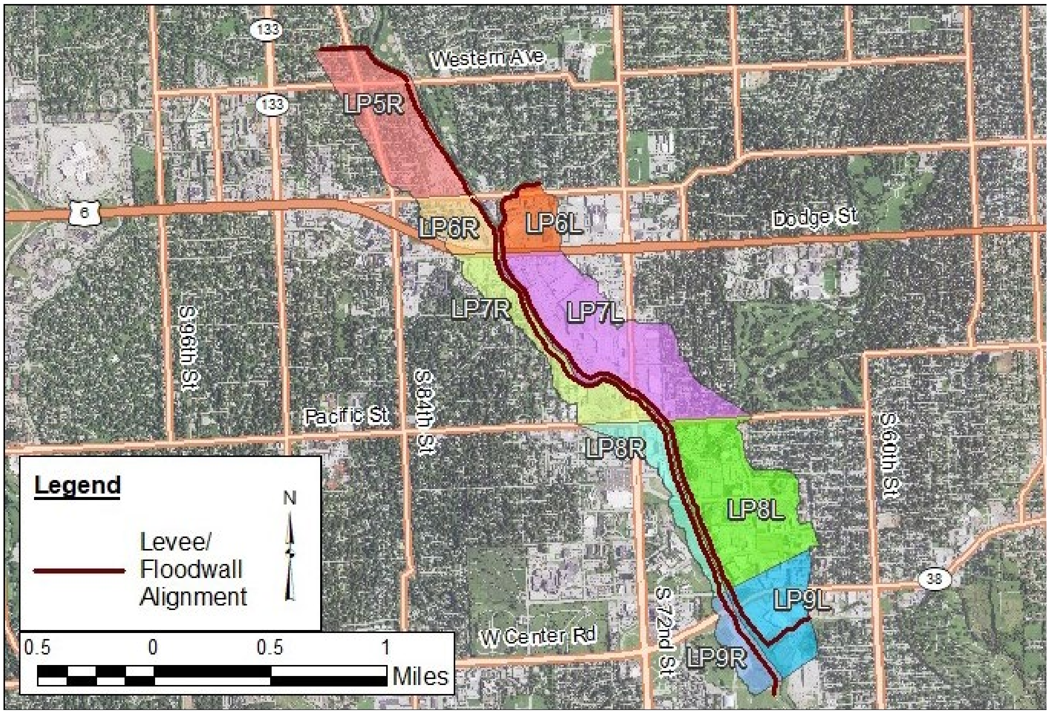


Figure 2. Final Recommended Plan Life Safety Analysis Reaches

3.1.2 POPULATION

The LST consequence estimation methodology estimates population in the leveed area using exposure curves calculated from National Structure Inventory (NSI) data. Exposure curves for population at risk (PAR) were developed by the Kansas City District for the damage reaches LP5, LP6 and LP7.

3.1.3 LEVEE SCREENING TOOL

The Levee Screening Tool (LST) includes an approximate assessment of life loss risk for all levees in the nationwide USACE portfolio. Levee screening methods are intended to be uniform throughout the nation's portfolio of levees, so that fair comparisons can be made. Levee screenings were completed in 2015-2016 for levee segments downstream of the proposed project. The same sponsor, performance mode ratings, evacuation effectiveness and emergency response details are assumed to apply to the proposed levees.

3.1.4 INCREMENTAL RISK

Flooding in a levee system can occur from four generalized mechanisms, as shown in Figure 3:

1. Breach prior to overtopping
2. Overtopping with breach
3. Component malfunction or improper operation
4. Overtopping without breach

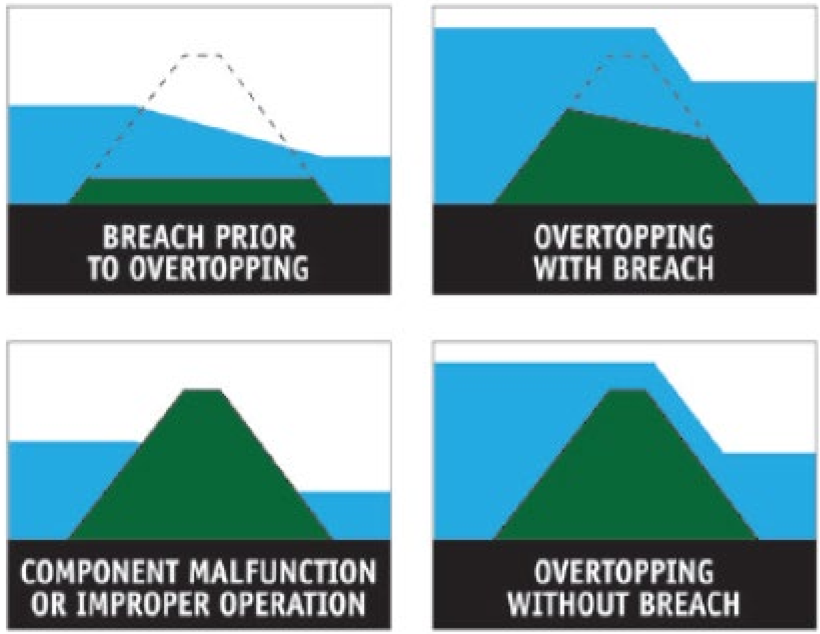


Figure 3. General Mechanisms of Flooding in a Levee System

Incremental risk for an existing levee system is defined in ECB 2019-15 as the risk of inundation posed by a levee system for the following three inundation scenarios: breach prior to overtopping, overtopping with breach, and component malfunction/mis-operation. In other words, the incremental risk is the risk associated with non-performance of the levee. It is the risk to the floodplain occupants that can be attributed to the presence of the levee. Total flood risk includes both incremental risk (scenarios 1-3) and the non-breach risk (scenario 4: overtopping without breach).

3.2 APPROACH, METHODOLOGY AND LIMITATIONS

The methodology used to compute a fatality rate in the LST is based on the research of S. N. Jonkman of Delft University. Jonkman's research states that loss of life caused by levee failure can be divided into two zones determined by their flood characteristics: the breach zone and the non-breach zone. The breach zone is located where the combination of flow velocities and depths are relatively high, and fatalities are heavily influenced by the destruction of structures in the zone. For this screening level approach, resources are not available to estimate properly the influence of the breach zone on the overall fatality rate. Therefore, fatalities in the breach zone are not accounted for in the assessment. The fatality rate for the non-breach zone is based on the maximum depth. The limitations of the LST are listed below:

- All life loss is based on depth for with-project condition based on levee system max (elevation at which there is a 2ft overland potential) and levee system minimum (induces bathtub effect in leveed area).
- PAR is reduced to a threatened population by evacuation effectiveness, flood warning time, and community awareness. Flood warning times are short for breach prior to

overtopping and for overtopping due to the flashiness of events. Arrival times of flood waters due to breach prior to overtopping or breach due to overtopping are not accounted for.

- There are no levees within the study area in the existing condition, however, there are several levees downstream. The proposed levees would be maintained by the same sponsor and within the same community. All performance modes are assumed to be rated similarly to the downstream systems, as well as the evacuation effectiveness factors.
- The LST does not calculate with project residual risk (does not calculate the capacity exceedance risk). Capacity exceedance was not accounted for in this analysis.
- The LST does not calculate the existing conditions total risk. The existing conditions total risk was estimated using the LST depth fatality curve.

3.3 LIFE LOSS COMPUTATIONS

The LST uses Evacuation Effectiveness factors (evacuation planning, flood warning, community awareness, and flood warning time) to estimate the number of fatalities for the overtopping and breach prior to overtopping flood scenarios. The LST also assumes during an overtopping flood event, a community would generally have significantly more warning time than it would if the levee breached at a lower stage, especially if the breach occurred below flood stage. For the purpose of the Papio GRR study life safety analysis, all evacuation effectiveness factors will be considered the same for existing conditions, the proposed alternative and for breach prior to overtopping and overtopping.

The LST consequence estimation methodology estimates population in the leveed area using exposure curves calculated from National Structure Inventory (NSI) data. Exposure curves for the PAR were developed for the damage reaches being used to assess economic benefit of the proposed Papio GRR alternatives. These damage reaches include LP5, LP6 and LP7. The below factors taken from levees in the vicinity of the proposed project are combined to reduce the PAR down to a threatened population (0.95*0.98*0.95*0.85 = 0.75):

- Evacuation Planning (0.95)
    - Douglas County, NE has a general evacuation plan in the Local Emergency Operations Plan, dated December 15, 2015. This plan is less than 10 years old but does not outline specific routes for evacuations.

- Community Awareness (0.98)
    - The community is well aware of the potential for Big Papillion Creek flooding and the protection provided due to the 2010 and 2014 floods and recent Missouri River flooding.

- Flood Warning Effectiveness (0.95)
    - Flood warning information along the Big Papillion Creek is generated automatically from a flood warning system and routed through the National

Warning System (NAWAS) to designated points. However, the notice to evacuate would be spread by local emergency responders.

- Warning Time OT and Breach Prior to OT (0.85)
    - Floods in the Papillion Basin are generally of short duration, with overbank flows lasting 12 hours, peaking in less than 4-6hrs, leaving little time for evacuation.

The calculation of the fatality rate is a function of the depth described in the equation below (from the LST technical manual):

$$FR = \Phi_N\left(\frac{\ln(h) - 5.2}{2}\right)$$

Where:
FR = the fatality rate.
h = water depth (meters).
Φ_N = cumulative normal distribution

An example of the fatality rate calculations can be seen below in Table 7. The calculation results in incremental life loss totals for the LP5 damage reach (plotted on the x-axis, see f-N chart in Section 3.6). Because total risk (or residual risk) is not directly calculated by the tool, an attempt was made to quantify the existing condition total risk with the same fatality rate equation seen above. An example of the existing conditions total risk calculation can be seen in

Table 8. The existing conditions provide on average a 50yr return interval level of protection, or 2 percent annual exceedance probability (AEP). Events that exceed a 50yr return interval will produce depths and life loss in the overbanks. The proposed levee raise would provide a 0.2% AEP level of protection.

Table 7. Incremental Life Loss Totals for LP5

Elevation (ft)	Est Population (Day)	Est Population (Night)	Average Threatened Population	Average Threatened Pop HL	Fatality Rate	Life Loss
1035	0	0	0	0	0.22%	0.000737206
1035.5	6	0	0.335093625	0.22545	0.22%	0.000737206
1036	6	0	0.335093625	0.22545	0.22%	0.001501716
1036.5	11	1	0.682598125	0.45925	0.22%	0.002484657
1037	19	1	1.129389625	0.75985	0.22%	0.002484657
1037.5	19	1	1.129389625	0.75985	0.22%	0.002484657
1038	19	1	1.129389625	0.75985	0.22%	0.002634829
1038.5	19	2	1.197649438	0.805775	0.22%	0.008491521
1039	63	5	3.859782125	2.59685	0.22%	0.008491521
1039.5	63	5	3.859782125	2.59685	0.22%	0.015945492
1040	120	8	7.247951	4.8764	0.22%	0.015945492
1040.5	120	8	7.247951	4.8764	0.22%	0.015945492
1041	120	8	7.247951	4.8764	0.22%	0.015945492
1041.5	120	8	7.247951	4.8764	0.22%	0.015945492
1042	120	8	7.247951	4.8764	0.22%	0.159291098
1042.5	986	254	72.40504475	48.7139	0.22%	0.159291098
1043	986	254	72.40504475	48.7139	0.22%	0.183741763
1043.5	1174	263	83.51898331	56.191325	0.22%	0.184438013
1044	1176	266	83.83546063	56.40425	0.22%	0.19206946
1044.5	1210	289	87.30430019	58.738075	0.22%	0.19206946
1045	1210	289	87.30430019	58.738075	0.22%	0.197311814
1045.5	1227	310	89.68718819	60.341275	0.22%	0.202909119
1046	1264	317	92.23141756	62.053025	0.22%	0.212069585
1046.5	1297	351	96.39526613	64.85445	0.22%	0.213516694
1047	1299	359	97.0530425	65.297	0.22%	0.246731918
1047.5	1441	464	112.1508719	75.454775	0.22%	0.269885647
1048	1616	475	122.6752939	82.535575	0.22%	0.318172638
1048.5	1921	547	144.6239264	97.30255	0.22%	0.321217026
1049	1925	564	146.0077389	98.233575	0.22%	0.333763179
1049.5	1966	614	151.710536	102.0704	0.22%	0.334787076
1050	1967	620	152.1759438	102.383525	0.22%	0.343401465
1050.5	1987	661	156.0915749	105.01795	0.22%	0.346937323
1051	1995	678	157.6987832	106.099275	0.22%	0.349503892
1051.5	2000	691	158.8654054	106.884175	0.22%	0.350227446
1052	2001	695	159.1942936	107.10545	0.22%	0.355688231
1052.5	2010	724	161.6764686	108.77545	0.22%	0.356288917
1053	2010	728	161.9495079	108.95915	0.22%	0.361449359
1053.5	2019	755	164.2951633	110.5373	0.22%	0.366309458
1054	2028	780	166.504299	112.0236	0.22%	0.371224164
1054.5	2035	807	168.7382565	113.5266	0.22%	0.371224164
1055	2035	807	168.7382565	113.5266	0.22%	0.371947718
1055.5	2036	811	169.0671447	113.747875	0.22%	0.37369517
1056	2038	821	169.8614407	114.282275	0.22%	0.373995513

Table 8. Annualized Life Loss calculated for LP5 for existing conditions

Elev. (ft)	Est Pop. (Day)	Est Pop (Night)	Average Threatened Population	Fatality Rate for Each Annual Probability of Inundation								Cumulative Annualized Life Loss
				2yr	5yr	10yr	25yr	50yr	100yr	200yr	500yr	
1011.5	0	0	0.0	3.51%	4.07%	4.56%	5.21%	5.78%	6.31%	6.99%	7.60%	0.00
1012	0	0	0.0	3.39%	3.95%	4.44%	5.09%	5.66%	6.19%	6.87%	7.48%	0.07
1012.5	6	12	1.2	3.27%	3.83%	4.32%	4.97%	5.54%	6.07%	6.75%	7.36%	0.07
1013	6	12	1.2	3.15%	3.71%	4.20%	4.85%	5.42%	5.95%	6.63%	7.24%	0.07
1013.5	6	12	1.2	3.03%	3.59%	4.08%	4.73%	5.30%	5.83%	6.51%	7.12%	0.08
1014	6	13	1.2	2.91%	3.47%	3.96%	4.61%	5.18%	5.71%	6.39%	7.00%	0.08
1014.5	6	13	1.2	2.79%	3.35%	3.84%	4.49%	5.06%	5.59%	6.27%	6.88%	0.08
1015	6	13	1.2	2.67%	3.23%	3.72%	4.37%	4.94%	5.47%	6.14%	6.76%	0.13
1015.5	11	24	2.3	2.55%	3.11%	3.60%	4.25%	4.82%	5.35%	6.02%	6.64%	0.14
1016	12	26	2.4	2.43%	2.99%	3.48%	4.13%	4.70%	5.23%	5.90%	6.52%	0.14
1016.5	12	26	2.4	2.31%	2.87%	3.36%	4.01%	4.58%	5.11%	5.78%	6.40%	0.14
1017	12	26	2.4	2.19%	2.75%	3.24%	3.89%	4.46%	4.99%	5.66%	6.28%	0.14
1017.5	12	26	2.4	2.07%	2.63%	3.12%	3.77%	4.34%	4.87%	5.54%	6.16%	0.22
1018	22	44	4.2	1.95%	2.51%	3.00%	3.65%	4.22%	4.75%	5.42%	6.04%	0.22
1018.5	22	44	4.2	1.83%	2.39%	2.88%	3.53%	4.10%	4.63%	5.30%	5.92%	0.22
1019	22	44	4.2	1.71%	2.27%	2.76%	3.41%	3.98%	4.51%	5.18%	5.80%	0.22
1019.5	22	44	4.2	1.59%	2.15%	2.64%	3.29%	3.86%	4.39%	5.06%	5.68%	0.22
1020	22	44	4.2	1.47%	2.03%	2.52%	3.17%	3.74%	4.27%	4.94%	5.56%	0.22
1020.5	22	44	4.2	1.35%	1.91%	2.40%	3.05%	3.62%	4.15%	4.82%	5.44%	0.30
1021	34	71	6.7	1.23%	1.79%	2.28%	2.93%	3.50%	4.03%	4.70%	5.32%	0.30
1021.5	34	72	6.8	1.11%	1.67%	2.16%	2.81%	3.38%	3.91%	4.58%	5.20%	0.30
1022	34	72	6.8	0.99%	1.55%	2.04%	2.69%	3.26%	3.79%	4.46%	5.08%	0.31
1022.5	35	73	6.9	0.87%	1.43%	1.92%	2.57%	3.14%	3.67%	4.34%	4.96%	0.31
1023	35	73	6.9	0.75%	1.31%	1.80%	2.44%	3.02%	3.55%	4.22%	4.84%	0.33
1023.5	40	81	7.8	0.63%	1.19%	1.68%	2.32%	2.90%	3.43%	4.10%	4.72%	0.33
1024	40	81	7.8	0.51%	1.07%	1.56%	2.20%	2.78%	3.31%	3.98%	4.60%	0.33
1024.5	40	81	7.8	0.39%	0.95%	1.44%	2.08%	2.66%	3.19%	3.86%	4.48%	0.33
1025	40	81	7.8	0.27%	0.83%	1.32%	1.96%	2.54%	3.07%	3.74%	4.36%	0.33
1025.5	40	81	7.8	0.22%	0.71%	1.20%	1.84%	2.42%	2.95%	3.62%	4.24%	0.33
1026	40	81	7.8	0.22%	0.59%	1.08%	1.72%	2.30%	2.83%	3.50%	4.12%	0.33
1026.5	40	81	7.8	0.22%	0.47%	0.96%	1.60%	2.18%	2.71%	3.38%	4.00%	0.33
1027	40	81	7.8		0.34%	0.84%	1.48%	2.06%	2.59%	3.26%	3.88%	0.33
1027.5	40	81	7.8		0.22%	0.72%	1.36%	1.94%	2.47%	3.14%	3.76%	0.33
1028	40	81	7.8		0.22%	0.60%	1.24%	1.82%	2.35%	3.02%	3.64%	0.33
1028.5	40	83	7.9		0.22%	0.48%	1.12%	1.70%	2.23%	2.90%	3.52%	0.35
1029	51	109	10.3		0.22%	0.36%	1.00%	1.58%	2.11%	2.78%	3.40%	0.35
1029.5	54	113	10.7		0.22%	0.24%	0.88%	1.46%	1.99%	2.66%	3.28%	0.37
1030	66	137	13.0		0.22%	0.22%	0.76%	1.34%	1.87%	2.54%	3.16%	0.37
1030.5	69	144	13.7			0.22%	0.64%	1.22%	1.75%	2.42%	3.04%	0.41
1031	103	204	19.7			0.22%	0.52%	1.10%	1.63%	2.30%	2.92%	0.51
1031.5	427	235	39.9			0.22%	0.40%	0.98%	1.51%	2.18%	2.80%	0.53
1032	460	309	46.8			0.22%	0.28%	0.86%	1.39%	2.06%	2.68%	0.54
1032.5	476	342	49.9				0.16%	0.74%	1.27%	1.94%	2.56%	0.55
1033	479	350	50.6				0.22%	0.62%	1.15%	1.82%	2.44%	0.55
1033.5	485	361	51.7				0.22%	0.50%	1.03%	1.70%	2.32%	0.55
1034	493	376	53.2				0.22%	0.38%	0.91%	1.58%	2.20%	0.59
1034.5	687	482	71.3				0.22%	0.26%	0.79%	1.46%	2.08%	0.59
1035	691	489	72.0					0.22%	0.67%	1.34%	1.96%	0.59
1035.5	722	556	78.3					0.22%	0.55%	1.22%	1.84%	0.59
1036	740	594	81.9					0.22%	0.43%	1.10%	1.72%	0.59
1036.5	750	612	83.7					0.22%	0.31%	0.98%	1.60%	0.59
1037	756	622	84.7					0.22%	0.19%	0.86%	1.48%	0.60
1037.5	1003	662	101.2						0.22%	0.74%	1.36%	0.60
1038	1016	699	104.5						0.22%	0.62%	1.24%	0.60
1038.5	1033	748	108.8						0.22%	0.50%	1.12%	0.60
1039	1047	789	112.3						0.22%	0.38%	1.00%	0.60
1039.5	1216	847	125.7							0.26%	0.88%	0.61
1040	3312	980	251.9							0.22%	0.76%	0.61
1040.5	3909	1032	288.8							0.22%	0.64%	0.61
1041	3910	1037	289.2							0.22%	0.52%	0.61
1041.5	3911	1044	289.7							0.22%	0.40%	0.61
1042	3911	1048	290.0							0.22%	0.28%	0.61
1042.5	3911	1048	290.0								0.22%	0.61
1043	3911	1050	290.1								0.22%	0.61
1043.5	4094	1058	300.9								0.22%	0.61
1044	4094	1058	300.9								0.22%	0.61
1044.5	4094	1058	300.9								0.22%	0.61

3.4 PROBABILITY OF FAILURE

The conditional probabilities of failure for breach prior to overtopping were calculated by the LST. Because the non-federal sponsor already maintains several levee systems in the vicinity of the proposed project, the LST ratings for the existing downstream levees were used for the proposed levees. The conditional probabilities of failure for seepage, stability, erosion, and closures were calculated by the LST assuming low-likelihood was used for every rating in the below systems, so the same rating of low-likelihood is expected for all performance modes on the proposed levee system. The below excerpt from the LST Technical Reference Chapter 12 was used to calculate the annual probability of inundation (API) for each levee proposed levee segment:

$$API(prior\,OT) = \sum_i \left[1/2 \cdot (ACE_{TOE} - ACE_{OT}) \cdot \mathrm{P}_{\mathrm{TOP}_i} + ACE_{OT} \cdot P_{TOP_i} \right] \quad (1)$$

$$API(OT) = ACE_{OT} \cdot (\mathrm{P}_{\mathrm{OT}} - \sum_i P_{TOP_i}) \quad (2)$$

where ACE_{TOE} is the annual chance exceedance for a flood level at the toe of the levee, ACE_{OT} is the annual chance exceedance for a flood level at the onset of overtopping (i.e., top of levee), P_{TOPi} is the probability of breach for an individual performance mode conditional on flood loading at the top of the levee, and P_{OT} is the probability of inundation conditional on an overtopping flood loading.

A simplifying assumption is made in the current levee screening methodology by assigning a value of 1.0 to the conditional probability of inundation for levee overtopping (P_{OT}) in Equation 2.

The annual probabilities of inundation for breach prior to overtopping and overtopping were plotted on the f-N chart in Section 3.6, and can be seen below in Figure 4.

Breach Prior to Overtopping		
Seepage	API(priorOT) =	1.85E-06
Stability	API(priorOT) =	5.38E-07
Erosion	API(priorOT) =	1.35E-06
Closure	API(priorOT) =	2.73E-07
	∑API(priorOT) =	4.01E-06
Overtopping		
Overtopping	API(OT) =	2.00E-03
Variables		
Flood level at top of levee	ACE_{OT} =	2.00E-03
Flood level at toe of levee	ACE_{TOE} =	2.00E-02
Gate Closure Toe	$ACE_{TOE\ of\ GATE}$ =	1.00E-02
Probability of Breach Seepage	$P_{TOP\ seepage}$ =	1.68E-04
Probability of Breach Stability	$P_{TOP\ stability}$ =	4.89E-05
Probability of Breach Erosion	$P_{TOP\ erosion}$ =	1.23E-04
Probability of Breach Closure	$P_{TOP\ closure}$ =	4.55E-05
Conditional Probability of Indundation for Overtopping	P_{OT} =	1.00E+00

Figure 4. Annual Probabilities of Inundation calculated by the LST

3.5 LIFE LOSS CONSEQUENCE RESULTS

The average annual life loss reduction for the proposed levees in damage reaches LP5, LP6 and LP7 are listed below in Table 9.

Table 9. Annualized Life loss expected for existing conditions and levee alternative

Damage Reach	Annualized Risk			
	Existing Conditions Total Risk	With Project Total Risk (Incremental Risk)*	Risk Reduction	% Reduction in Risk
LP5	5.14E-02	7.71E-04	5.06E-02	98.5%
LP6	6.09E-01	5.74E-04	6.09E-01	99.91%
LP7	1.51E+00	2.50E-04	1.51E+00	99.98%
LST 2.0 Left Bank Only**	2.78E-01	3.38E-02	2.44E-01	87.84%

*Reference Section 3.1.4 which states that incremental risk is assumed to be equivalent to total risk.

**The LST 2.0 method accounts for breach and non-breach (overtopping without breach) risk

3.6 LIFE RISK MATRIX (F-N CHART)

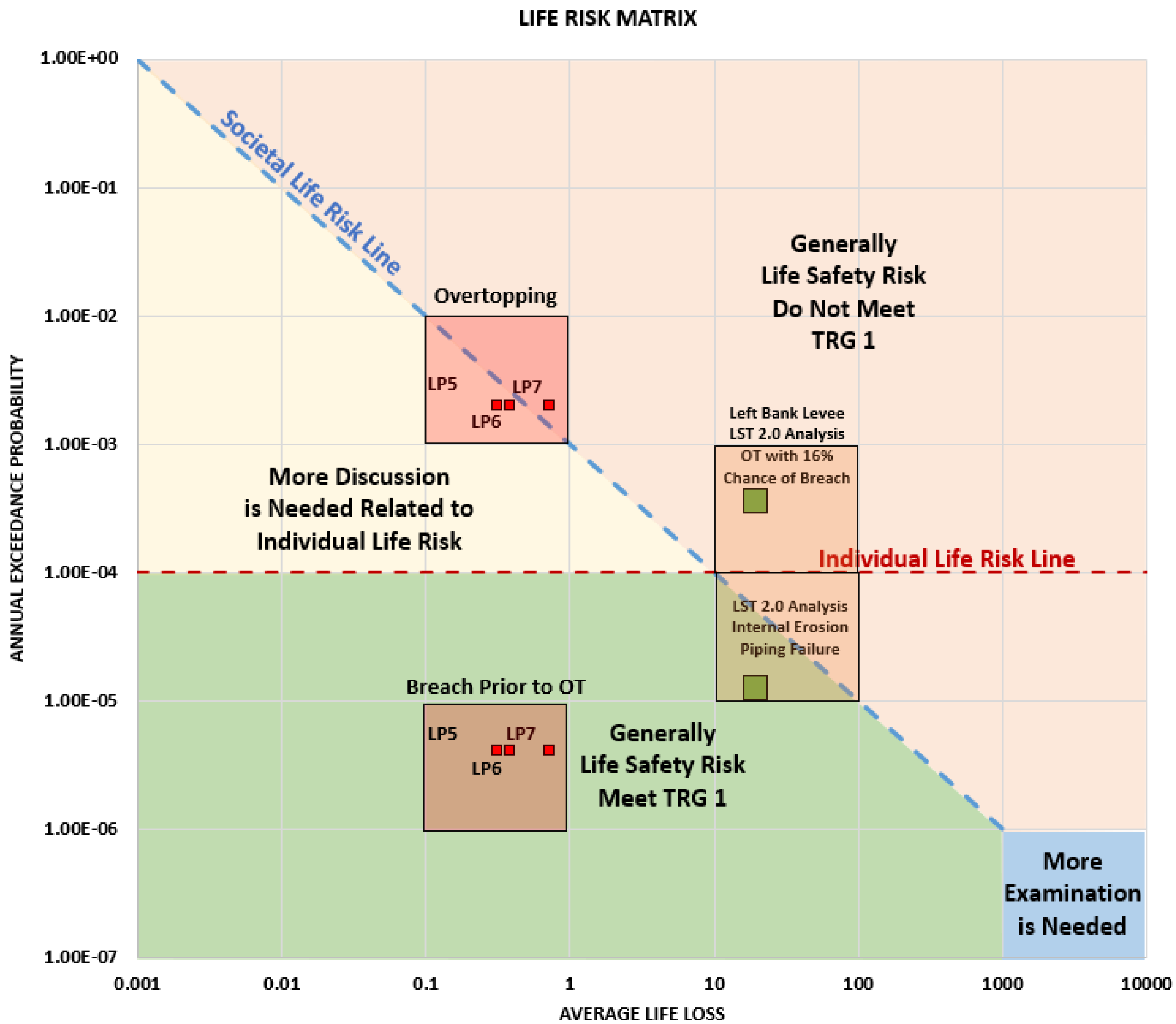


Figure 5. f-N chart for overtopping and breach prior to overtopping risk for the proposed project

The chart in Figure 5 shows that the risk associated with the currently proposed projects straddles the societal life risk line and also shows an additional analysis done with LST 2.0 (plotted as green squares). The LST 2.0 analysis was a semi-quantitative and brief analysis using LifeSim and levee breach scenarios that would help confirm the validity of the LST method described in this report. The LST 2.0 analysis shows that the actual risk may range across several orders of magnitude Figure 6 shows the same proposed projects if evacuation effectiveness factors and overtopping resiliency is also added into the system. A resilient component or system is capable of absorbing energy during loading without experiencing permanent deformation, extensive damage, cumulative degradation, or catastrophic failure. The proposed probability of failure during overtopping ($P_{OT} = 1$) in Section 3.4 is 100%. However, resilient components may not be necessary to justify further reducing the probability of overtopping on this system for several reasons:

1. The system would only overtop for a very short duration due to the flashiness of the watershed.
2. The downstream levees have been reported as overtopped with no damage.
3. The sponsor has a very robust maintenance program and does an excellent job maintaining sod cover
4. Bike paths are often placed over the top of the levees.
5. The levees and foundation will consist of cohesive material.

For the above reasons, the probability of failure during overtopping the levee is $P_{OT} = 0.25$, or a 25% chance of levee failure during overtopping. This equates to a 2000-yr return interval failing the levee or return interval between 500-yr and 2000-yr with a ¼ chance of failing the levee. This factor for overtopping failure was added to Figure 6. It should be noted that this chance of failure during overtopping assumes life loss would not occur until the levee was breached, as the flashy flows would not overtop the levee for a long enough duration to create significant ponding depths that would impact the PAR and/or the population would have enough time to effectively evacuate during an overtopping without breach event. The LST 2.0 points were also added to Figure 6 and already account for a reduce probability of failure during overtopping. The LST 2.0 method utilized LifeSim to estimate life loss and used the information provided in Section 2.3 of the Life Safety Appendix to determine evacuation effectiveness in the leveed area.

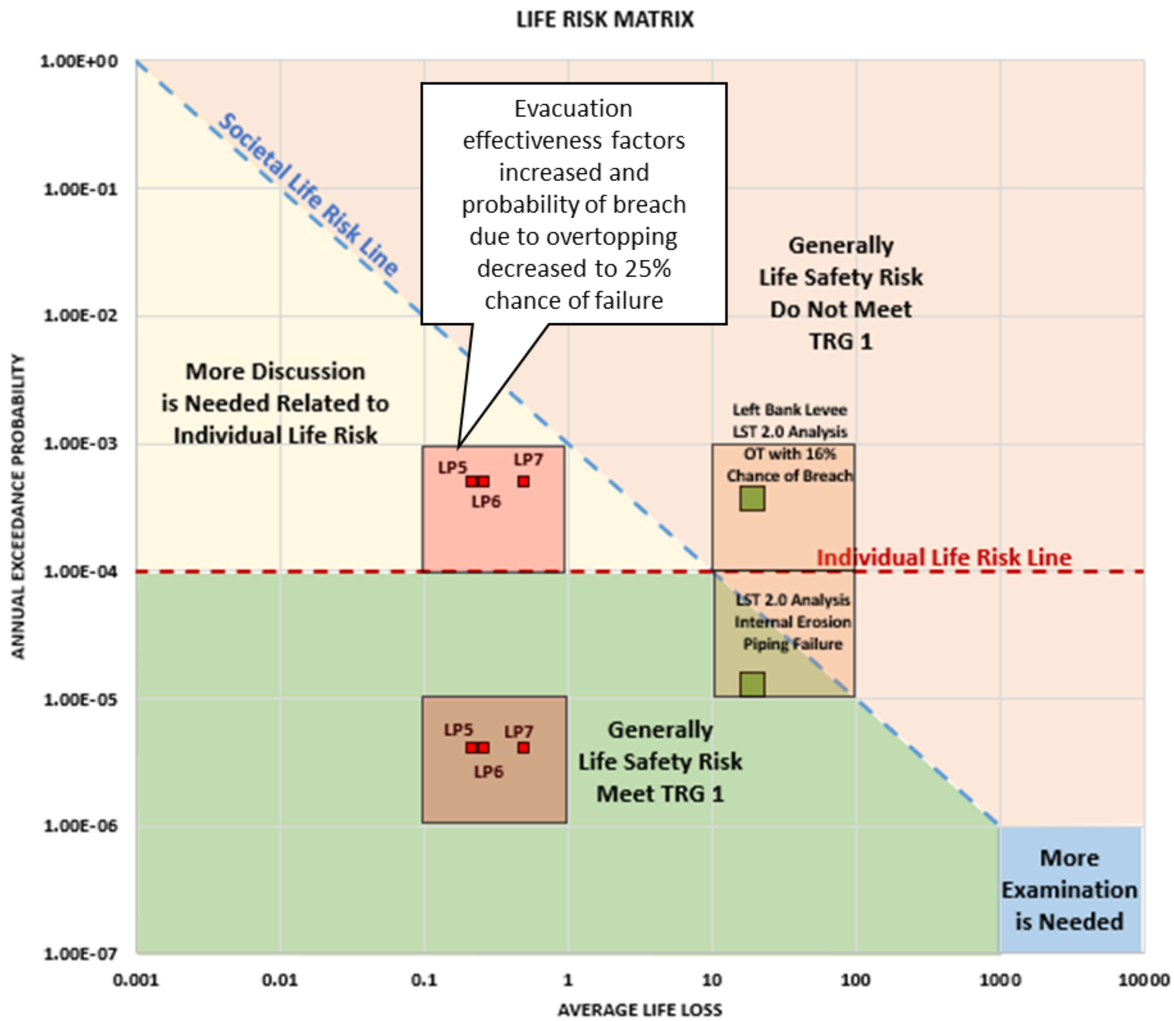


Figure 6. f-N chart for overtopping and breach prior to overtopping risk for the proposed project with potential increased overtopping resiliency and evacuation effectiveness.

3.7 CONCLUSION

If evacuation effectiveness is increased, the average life loss, during an overtopping event, of the proposed levees do not plot above the societal risk line using the LST method. However, if using the LST 2.0 method, the average life loss will likely remain about the societal and individual risk lines regardless of how effective the evacuations are. The only structural option to further reduce incremental risk would be to raise the levee higher, which is not practical and cost prohibitive due to real estate and closure structure size/height constraints. Figure 6 shows the resulting life safety risk matrix if the probability of breach due to overtopping could be reduced resulting in LP7 plotting below the societal risk guideline. If the sponsor improves their evacuation effectiveness (update evacuation plans and flood warning effectiveness), the project will plot even further below, or in the case of the LST 2.0 plotted points; closer to, the societal risk guideline by reducing the average life loss. Therefore, the Recommended Plan (selected based upon NED benefits) appear to adequately address TRG 1 and 4 to the extent practicable.

4 PAPILLION CREEK DAM SITE 10

4.1 FINDINGS AND RECOMMENDATIONS

The abbreviated SQRA performed by the Omaha District did not identify any potential failure modes that would prevent Papillion Creek Dam Site 10 (DS10) from meeting the tolerable risk and essential USACE guidelines. This abbreviated SQRA used the optimized DS10 design to be included in the final recommended plan of the GRR which incorporates current USACE design criteria as discussed in Appendix C – Geotechnical Analysis. See Section 4.2 Background for additional details of the DS10 design to be included in the final recommended plan. It should be noted that significant design changes, such as modifying DS10 to a wet dam with a permanent pool or changes to the spillway, outlet works, or dam embankment that affect the hydrologic loading will require an updated or new SQRA to ensure the modifications do not significantly increase the risks of the project.

One of the risk-driving potential failure modes (PFMs) identified in the abbreviated SQRA was erosion of the unlined, earth-cut emergency spillway (PFM 15) due to high flow velocities (up to 12.2 ft/sec) and no control sill or cutoff structure to prevent headcutting. However, a significant length (1,285 feet) would have to erode within a relatively short duration (8 hours with more than one foot of flow depth) to breach the spillway crest, and the resulting breach discharge has minimal incremental inundation and loss of life consequences compared to design spillway flows (non-breach).

The other risk driver PFM carried forward during the abbreviated SQRA was backwards erosion piping (BEP) of a non-plastic layer in the glacial till foundation at the outlet works channel excavation (PFM 14). There is the potential for a fine grained, cohesionless sand stratum in the left abutment to be exposed during channel excavations for the outlet works. However, the "critical" max high pool (MHP) reservoir loading event is infrequent (AEP 1/5225.050,000) and the pool duration (41 hours) and global gradients through the foundation (0.08 ft/ft at MHP) are considered low compared to what is considered necessary to initiate and progress BEP to breach.

4.1.1 SOCIETAL INCREMENTAL LIFE SAFETY RISK

Twenty-five (25) potential failure modes (PFM) were identified prior to the abbreviated SQRA for consideration. Twenty-three (23) were not developed in detail as they were not considered to be "risk-drivers" for the project. Non risk-driver PFMs are discussed in Section 4.6.2. The following risk-driver PFMs were evaluated by the PA team:

- PFM 14: Backwards Erosion Piping (BEP) of a non-plastic layer in the glacial till foundation at the outlet works channel excavation
- PFM 15: Spillway Erosion

A risk matrix has been established to portray the incremental life safety risk (due to failure or breach) associated with the identified risk-driving PFMs, with annual probability of failure (APF) on the vertical axis and the associated incremental life loss on the horizontal axis, using cell divisions corresponding to order of magnitude ranges of APF and incremental life loss. The matrix is similar to the f-N diagram used to portray incremental life safety risk estimated from quantitative risk analysis. The societal incremental life safety risk matrix is shown in Figure 7.

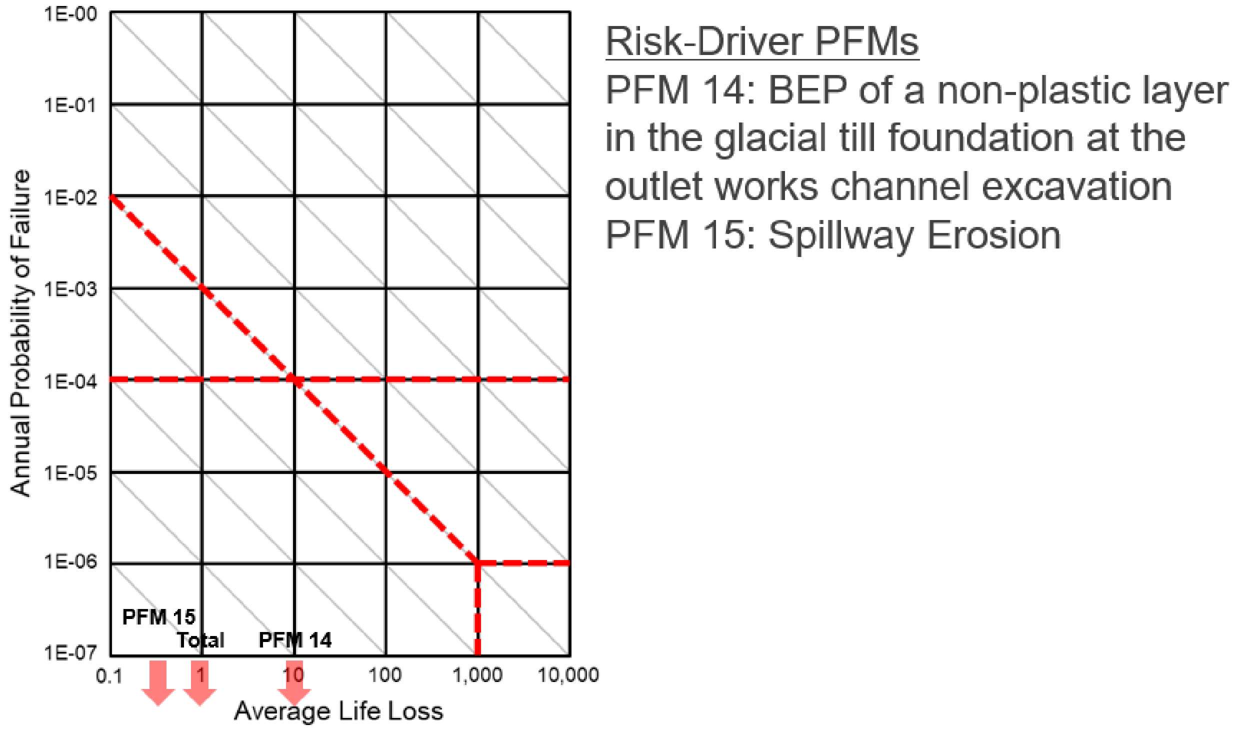


Figure 7. DS10 Societal Incremental Life Safety Risk Matrix.

An approximate numerical estimate of APF and average incremental life loss were obtained for each PFM using the centroid (geometric mean) of the box (order-of-magnitude estimate). The total APF was calculated by summing the APFs for all of the primary risk-driver PFMs assuming they are mutually exclusive. The total average annual incremental life loss (AALL) was calculated by summing the product of the APF and average incremental life loss for all of the primary risk-driver PFMs. The weighted average incremental life loss was then calculated by dividing by the total AALL by the total APF.

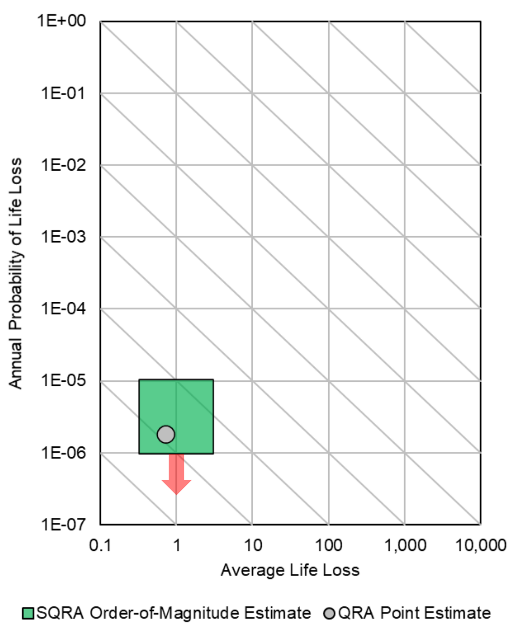
The estimated total APF is between 1E-09 and 1E-08 failures per year, and the estimated weighted average incremental life loss is between 0.3 and 3 lives per failure. Therefore, the best estimate of the average annual incremental life loss is 3E-09 lives per year. The total risk of the project is below the individual and societal life risk lines and therefore meets TRG 1. Additional details about the risk-driving PFMs and associated incremental life loss are provided in Section 4.5.5.

4.1.2 NON-BREACH LIFE SAFETY RISK

Non-breach risk occurs when the flood capacity of the dam is exceeded. At this point, the dam transitions from managing the flood to passing the flood. For dams, the transition occurs when the spillway activates at the top of active storage (TAS) elevation. This elevation corresponds to the annual probability of non-breach inundation but may not result in life loss. The top of active storage at DS10 is at the proposed spillway crest elevation of 1191.6 feet NAVD88 and has an estimated ACE of 1/5,000.

The annual chance exceedance (ACE) when life loss begins to occur ($ACE_{N>0}$) was not determined as part of the abbreviated SQRA due to time and funding restraints. The consequence modeling indicates that non-breach life loss is not expected for loadings up to the modeled MHP (ACE 1/550,000) elevation of 1204.4 feet NAVD88 or a total peak outflow of 14,033 cfs from the dam. Even though portions of Thomas Creek overflow at the estimated non-damaging discharge of 5,020 cfs, it is assumed that when the pool nears the spillway crest, a warning will be communicated to the downstream areas to allow the population at risk (3,127 during the day and 1,139 at night) to mobilize and evacuate before the discharge during the MHP event peak reaches the impacted population. Therefore, there is no estimated life loss at the MHP non-breach event and the annual probability of inundation with non-breach life loss was estimated to be less than 1.82E-6 floods per year.

The results are plotted on a separate non-breach life safety risk matrix, similar to the societal incremental life safety risk matrix previously described. However, the vertical axis is labeled "annual probability of life loss," and no tolerable risk limit lines are shown since they are not applicable. The non-breach life safety risk matrix assuming life loss for pool loading conditions above the MHP non-breach event is shown in Figure 8.



Flooding Threshold
Top of Active Storage: 1/5,000 AEP
Non-damaging outflow: 5,020 cfs

Life Loss Threshold
No loss of life for non breach up to the modeled MHP elevation

Top of Dam
AEP = 1/1,900,000 (>100% PMF)

PMF
Freeboard: 3 feet
Outflow: 14,033 cfs

Downstream FRM Structures
Little Papillion Creek Channel Improvements are exceeded from 11,000 to 20,000 cfs

Figure 8. DS10 Non-Breach Life Safety Risk Matrix

The lowest dam crest elevation has an estimated 1/1,900,000 AEP. At this elevation, the dam is capable of storing 100 percent of the estimated Probable Maximum Flood (PMF) inflow hydrograph. The estimated PMF event results in 3 feet of freeboard, not considering wind/wave effects. The TAS elevation has an estimated 1/5,000 AEP. The primary consequence center is Omaha, NE which is located three river miles downstream of the dam. The total peak outflow from the dam for the PMF event is 14,033 cfs, while the estimated downstream non-damaging discharge for Thomas Creek is 5,020 cfs. The inundation area adjacent to Thomas Creek is sparsely populated with a mix of residential and industrial buildings. Residential, commercial, and industrial use increases as Thomas Creek joins the Little Papillion Creek approximately 5 miles downstream of DS10. Channel improvements completed in 1964 on the Little Papillion Creek increased its capacity between 11,000 and 20,000 cfs. The SQRA team has moderate confidence that there would be ample warning time to mobilize and evacuate the population at risk, especially due to the sparse population directly downstream of DS10 on Thomas Creek. However, due to the limited team discussion and consequence information for the abbreviated SQRA, there is low confidence in the non-breach life loss results.

4.1.3 MAJOR FINDINGS AND UNDERSTANDINGS FROM THE RISK ASSESSMENT

The following major findings and understandings were developed by the abbreviated SQRA team:

- The total risk of DS10 is below individual and societal life risk lines and therefore meets TRG 1.
- DS10 is located approximately 3 miles northwest of Papillion Creek Dam Site 11 (DS11). Therefore, it is assumed that the geologic site conditions at DS10 and DS11 are similar, and DS11 has performed adequately since construction.
- The uncontrolled, unlined, earth-cut emergency spillways at the existing Papillion Creek Dams have never been tested, so there is considerable uncertainty in the duration of flow required to breach the spillway.
- A total of 5 feet of embankment and valley alluvium foundation settlement occurred at DS11. Two of the five feet of settlement occurred post-construction. Similar settlement is expected at DS10; therefore, the current design includes two feet of overbuild.
- Dry dams have less associated risk due to shorter hydraulic event durations and no permanent pool to achieve a steady-state seepage conditions in the dam embankment. Modifying the outlet works to hold a permanent pool to make DS10 a wet dam will require a 408 and a new/modified SQRA to assess the increased risk of a permanent pool.
- The team considered whether a control sill or cutoff structure is necessary to reduce the risk of a headcut advancing through the crest of the spillway during spillway flow events; however, the team determined that the low probability and life loss of the failure mode made the cost not necessary.

4.1.4 RECOMMENDATIONS

The following recommendations address deficiencies identified by the abbreviated SQRA team in the preliminary design for DS10 in the GRR final recommended plan. The goal is to incorporate the recommendations in the preconstruction engineering and design if the project enters the implementation phase to reduce the risk of the project to the downstream population.

- Perform additional site characterization and lab testing of the sand stratums in the Kansan glacial drift foundation material at the proposed outlet works location.
- Require blanketing or filtering of sand seams discovered in the excavation for the intake and outlet channels for the outlet works.

- Armor the intake and outfall to ensure erosion does not uncover a sand seam in the glacial drift foundation.
- Fill the drainage ditch downstream of the spillway that has the potential to concentrate flows and initiate headcutting.
- Any proposed recreation, utility, or other features submitted through the 408 process within the spillway will be thoroughly reviewed prior to approval. The inclusion of such features is likely to increase the erosion potential of the spillway due to increased turbulence and localized velocities caused by knickpoints.
- Prioritize routine maintenance of the trash rack on the intake of the outlet works in the O&M Manual to ensure the design capacity of the outlet works is maintained to prevent increased frequency and duration of spillway flow.
- Construct upstream impervious blankets at the abutments to reduce potential seepage through the loess.
- Further develop consequence modeling and create mapping products to define the population at risk and effectively communicate the risks from breach and non-breach releases if DS10 is constructed.

4.2 BACKGROUND

4.2.1 LOCATION

Papillion Creek Dam Site 10 (DS10) is proposed to be constructed approximately 2.5 miles east and 0.5 miles north of Bennington, Nebraska. The dam will be located on Thomas Creek, a tributary of Little Papillion Creek. The dam and reservoir site is primarily in Douglas County, but about one-half of the drainage area is in Washington County. The contributing drainage area to the site is approximately 4.3 square miles. Dam Site 10 is one of twenty-one (21) dams and reservoirs authorized by the Flood Control Act of 1968 (Public Law 90-483) in accordance with the recommendations of the Chief of Engineers in House Document No 349, 90th Congress, 2nd session, and the Energy and Water Development Appropriation Act of 1981. The dams were initially authorized for flood control, recreation, fish and wildlife enhancement, and water quality. Due to significant changes in policy following project authorization, all proposed dams were reevaluated and at the time only four dams (Dam Sites 11, 16, 18 and 20) were determined to be either economically feasible and/or met the benefit/cost requirements set forth in Section 9 of Public Law 89-72.

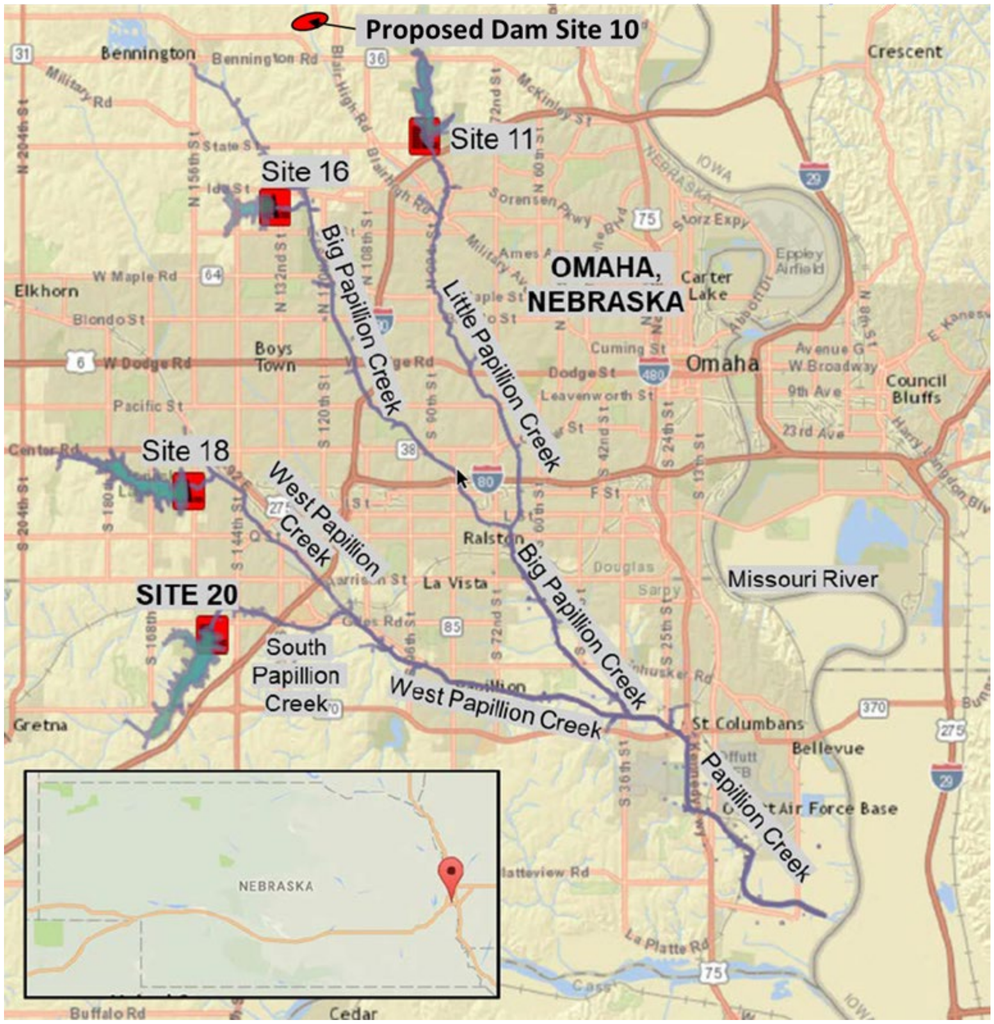


Figure 9. Map of Existing Papillion Creek Dam Sites in Relation to Dam Site 10

4.2.2 GENERAL PROJECT DESCRIPTION

Proposed Papillion Creek Dam Site 10 will be a dry dam project consisting of a rolled earth filled dam embankment; an 8-ft (Span) x 7-ft (Rise) box culvert placed near the bottom of the pool; and an uncontrolled, earth-cut spillway. See Figure 8 for a map of DS10 in relation to the existing federal Papillion Creek Dam Sites, Figure 9 for a plan view of the project features, Table 10 for storage allocations for DS10, and Table 11 for pertinent project data.

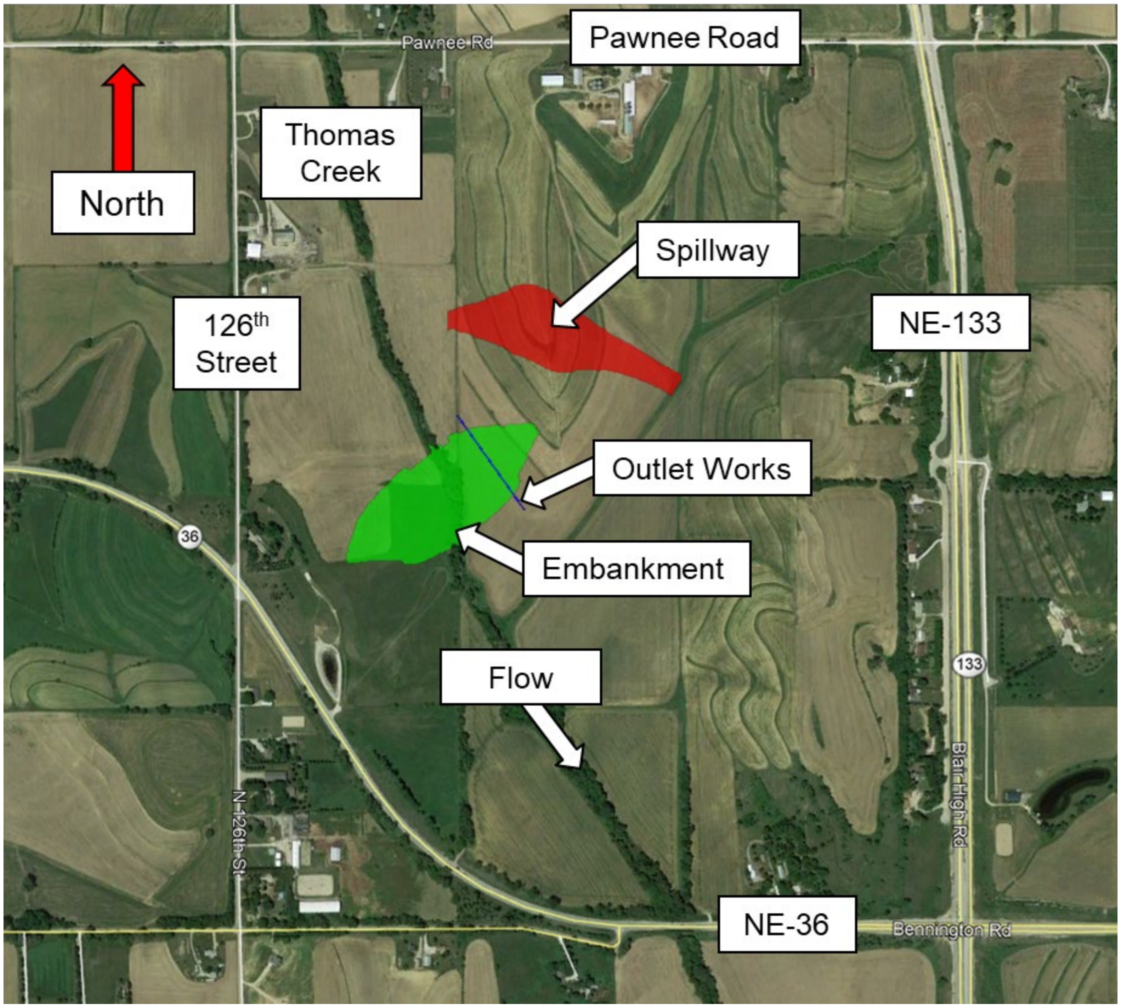


Figure 10. DS10 Plan View of Project Features

4.2.2.1 General Project Pertinent Data

Table 10. Papillion Creek Dam Site 10 Storage Allocations

Type of Storage	Elevation (ft NAVD88)	Storage Volume (acre-feet)
Top of Dam	1,207.4	5,726
Reasonable High (RH) PMF	1,205.6	5,172
Most Reasonable (MR) PMF	1,204.4	4,821
Spillway Crest	1,191.6	1,992
Top of Flood Control Pool	1,185.0	1,097
Top of Multipurpose Pool [1]	-	-
Outlet Invert Elevation	1,154.0	-
Minimum Pool	1,151.0	0

[1]Dam Site 10 is a dry dam and therefore does not have a permanent pool.

Table 11. Papillion Creek Dam Site 10 Pertinent Data

Embankment	
Design Crest Elevation	1,207.4 Feet NAVD88 (excluding overbuild)
Design Freeboard	3 Feet
Crest Length	1,400 Feet
Crest Width	25 Feet
Height above Flood Plain	44.6 Feet
Height above Streambed	56.0 Feet
Type of Fill	Homogeneous Rolled Earth
Estimated Volume of Fill	408,000 CY
Slope Protection	Grass Cover
Wave Protection	None
Seepage Control	Internal Pervious Fill Drain
Emergency Spillway	
Type and Location	Ungated, Grass-Lined Earthcut Channel in Left Abutment
Design Discharge Capacity (at Most Reasonable PMF)	14,050 cfs (at Elevation 1,204.4 Feet NAVD88)
Design Crest Elevation	1,191.6 Feet NAVD88
Bottom Width	100 Feet
Length	1,285 Feet at Centerline
Side Slopes	1V on 3H
Excavation	374,700 CY
Outlet Works	
Inlet Type	Low-level with Trashrack (Uncontrolled)
Design Invert Elevation at Inlet	1,154.0 Feet NAVD88
Design Invert Elevation at Outlet	1,150.0 Feet NAVD88
Conduit Length	700 Feet
Conduit Dimensions	8 Foot Span by 7 Foot Rise Box Culvert
Conduit Type	Reinforced Concrete
Design Discharge Capacity	1,860 cfs (at Most Reasonable PMF)

4.2.2.2 Embankment

The embankment will be a homogeneous rolled earth filled structure constructed of primarily lean clay (CL) impervious fill from spillway and upstream pool area borrow excavations. The 1,400 feet long, 25-foot-wide crest is designed at elevation 1,207.4 feet NAVD88 and will include two foot of overbuild to account for post-construction settlement in the valley founded on alluvium. At its maximum section, the embankment is about 44.6 feet above the valley floor and 56 feet above the streambed. A typical embankment section is shown on Figure 10.

The upstream slope of the embankment consists of a 1V on 5H slope from the crest to the ground surface. Since DS10 is a dry dam without a permanent pool, it was determined that riprap protection on the upstream face was not necessary to protect the embankment from wave action. The downstream slope of the embankment consists of a 1V on 6H slope from the crest to the ground surface. All embankment slopes are protected by grass cover.

A 6-foot-deep by 10-foot wide-inspection trench with 1V on 2H side slopes will be excavated along nearly the entire length of the embankment. The center of the trench is located along the embankment centerline and will extends up the abutments to the spillway crest elevation of 1,191.6 feet NAVD88. The purpose of the inspection trench is to break the continuity of the surface soil structure, particularly sand seams, by replacing it with compacted impervious fill. It will also be used to identify unforeseen soft areas near the ground surface that will require procedural changes during construction. The excavated fill will be recompacted into the inspection trench if it meets fill requirements.

Seepage through the embankment will be controlled by an internal pervious fill drain designed to prevent saturation of the downstream slope under all "normal" seepage conditions. For unusual seepage conditions such as embankment cracking, the drain is intended to reduce seepage pressures while the pool is drawn down. The drain will be composed of imported, free-draining (pervious) fill. The top of the drain will be constructed to the most reasonable PMF elevation of 1204.4 feet NAVD88 and begin at the centerline of the dam. The drain consists of a 6-foot-thick continuous 1V on 1H upstream inclined pervious fill chimney and a continuous 3-foot-thick horizontal pervious fill blanket that extends over the flood plain for an approximate distance of 220 feet. In addition, the base of the former/original channel will be lined with 3 feet of pervious fill to provide a controlled outlet for seepage.

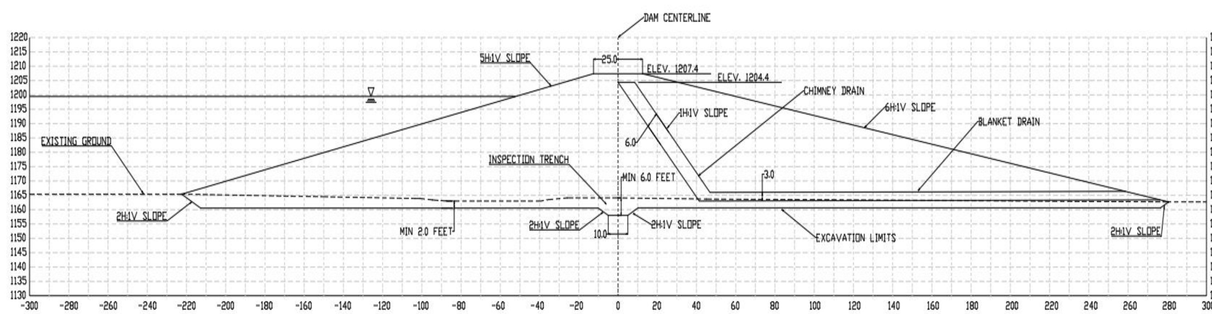


Figure 11. DS10 Typical Embankment Cross Section

4.2.2.3 Outlet Works

The DS10 outlet works will consist of a reinforced concrete box culvert with an 8 foot span and a 7 foot rise. The outlet works was designed to be self-cleaning and large enough for equipment to remove potential sedimentation within the span of the conduit. The intake will be at elevation 1154.0 feet NAVD88 and will include a trash rack to prevent large debris from potentially clogging or damaging the box culvert. The reinforced concrete box culvert will span 700 feet and will be founded entirely on stiff to very stiff glacial drift in the left abutment. Seepage along the outlet works under "normal" seepage conditions or due to cracks or flaws adjacent to the outlet works will be collected by a 10-foot-long and 3-foot-wide pervious backfill drain near the outfall of the box culvert. The stilling basin will be protected with riprap revetment. Grading and design of the intake, stilling basin, and the channel excavation to connect the existing streambed to the outlet works was not fully developed for the General Reevaluation Report (GRR).

4.2.2.4 Spillway

The centerline of the earth-cut, grass lined spillway will be located about 600 feet north of the left abutment of the dam embankment. It will be approximately 1,285 feet long from the start of the crest to the end of the spillway at its centerline and have a minimum 200-foot-long and 100-foot-wide crest at design elevation 1191.6 feet NAVD88. Due to the low estimated probability of spillway failure and associated life loss discussed in Section 4.5.5 Abbreviated Semi-Quantitative Risk Assessment, the spillway crest design does not currently include additional erosion protection such as a concrete control sill or cutoff wall. Most of the channel base is founded in more erosive loess material and will therefore be excavated to a minimum depth of 5 feet and backfilled with impervious fill consisting of highly plastic clays with liquid limits in excess of 40 percent to limit the potential for erosion. Figure 11 shows a typical section of the spillway with the over-excavation of exposed loess.

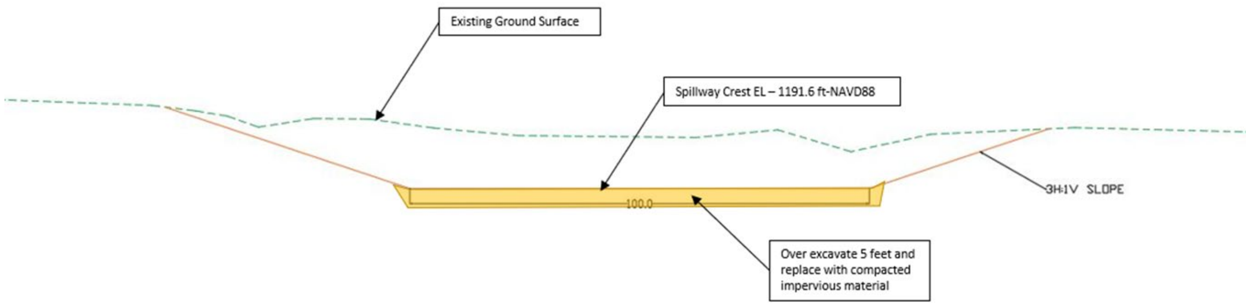


Figure 12. DS10 Typical Spillway Cross-Section

4.2.3 SITE CHARACTERIZATION

4.2.3.1 Geologic Setting

Papillion Creek Dam Site 10 lies within the Dissected Till Plains section of the Central Lowland Physiographic Province. The major topographic feature of the area is the dissected loess-mantled upland, characterized by gently rolling to rolling hills with well-developed drainages. Surface geology of this tributary valley, with the exception of recent alluvium in the valley, is Pleistocene in age and is entirely of eolian (wind-blown) origin. These eolian deposits are represented by the Peorian Loess and the underlying Loveland Loess. For the purpose of this report, they are treated as one unit and are designated the Peorian-Loveland Loess (Undifferentiated). The older Pleistocene deposits of the Kansan and Nebraskan glacial stages underlie the entire basin but are not exposed at the surface. In general, most of the glacial material encountered is Kansan in age; however, this does not preclude the possibility that some Nebraskan age deposits may be encountered at the lower elevations. As the Nebraskan age deposits are very similar to those of the Kansan, with no particular differences in the foundation properties from an engineering viewpoint, all glacial deposits will be considered Kansan for simplicity. Cretaceous sandstones or Pennsylvanian limestones and shales form the bedrock surface underlying most of the Papillion Creek drainage basin; however, they occur at a depth that was not a factor in design or encountered during construction of the project. See Figure 12 for the geologic profile of Papillion Creek Dam Site 10.

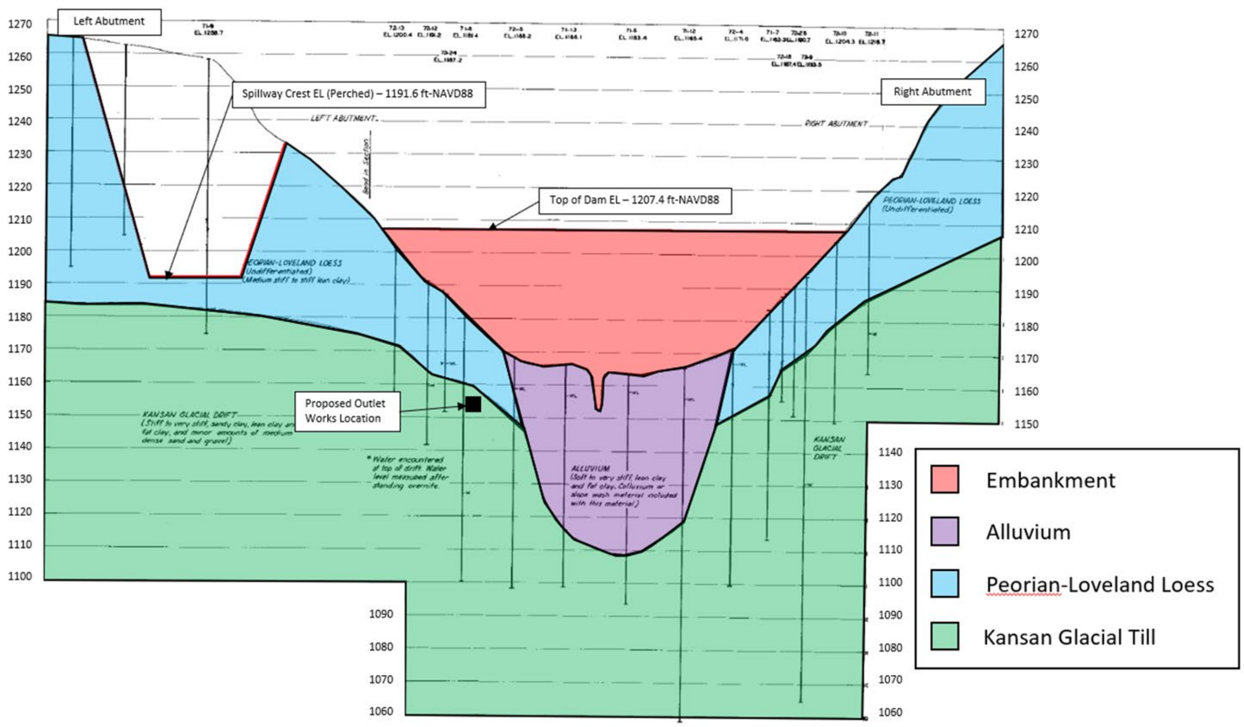


Figure 13. DS10 Geologic Profile

4.2.3.2 Foundation Investigation

A total of 45 borings were completed during the 1975 design completed by USACE and documented in the following report:

- Specific Design Memorandum No. MPC-33, Site 10, Papillion Creek and Tributaries, Nebraska, USACE dated May 1975

These borings ranged from 15 to 129 feet in depth, penetrating three types of material: alluvium, loess, and glacial drift. Disturbed jar and moisture samples were taken in each boring at 5 foot intervals or at change of material, whichever occurred first. Standard penetration blow counts, using the rope and drum method, were made in representative borings within the flood plain and outlet works area. Undisturbed Shelby tube samples were taken at prescribed intervals in representative holes in the alluvium and loess.

Laboratory tests were performed on disturbed and undisturbed samples in accordance with procedures in the Laboratory Soils Testing Manual EM 1110-2-1906 dated November 1970. This testing consisted of classification by mechanical analyses and Atterberg limits of jar and undisturbed samples. Moisture was determined on materials from moisture tins. Undisturbed testing consisted of unconsolidated-undrained "Q" tests, consolidated-undrained "R" tests, direct shear and consolidation tests. Remolded testing consisted of compaction, classification, and "Q", "R" and direct shear tests on material to be used for embankment fill. Loess testing consisted of collapse consolidation, density, and pin-hole dispersion tests. Foundation properties are summarized in Table 12. Detailed boring records and laboratory test results for DS10 can be found in the 1975 design report.

Table 12. Summary of Foundation Properties

Unit	USCS	Strength Description	Dry Density (pcf)	Liquid Limit	Plasticity Index
Alluvium	Mostly lean clay (CL)	Soft to Stiff	72-100	30-69	-
Loess	Mostly lean clay (CL)	Medium Stiff to Stiff	88-102	30-52	11-34
Glacial Drift Till	Sandy clay (CL-CH)	Very Stiff	-	-	-

4.2.3.3 Ground Water

Ground water was encountered in both abutments and in the valley during the foundation investigation for the 1975 design report. Most of the borings indicating the presence of groundwater were left open for a sufficient time to allow the water surface, if present, to reach static level; however, static water levels were not established in many of the borings due to the presence of drilling fluid in the holes. The water levels in the valley borings were fairly uniform in depth and ranged from elevation 1054.0 to 1058.0 feet. The ground water level in the left abutment was established in two borings (72-22 and 72-24) at elevation 1164 and 1166 feet, respectively. None of the borings in the location of the spillway encountered a water level above the design spillway channel elevation of 1191.6 feet NAVD88. It can be concluded from the foundation investigation for the 1975 design report that proper dewatering will be required to ensure dry working conditions in the outlet works excavation; however, it is not expected to present construction problems.

4.2.4 DESIGN ANALYSES

Stability, seepage, settlement, and loess collapse analyses were performed at DS10 for Specific Design Memorandum No. MPC-33. Below is a list of the key points summarizing the analyses:

- The stratigraphy and soil properties are similar to other Papillion Creek Dam Sites which have performed well since their construction.
- The foundation materials are suitable to support the construction of a dam embankment and appurtenant structures
- The material from the spillway and upstream pool area borrow excavations is suitable to use as fill for the dam embankment.
- The factor of safety determined from each of the stability analysis based on material properties from the foundation investigation meets or exceeds minimum factor of safety requirements.
- A maximum centerline foundation settlement of 4.5 feet is expected. Approximately half to two-thirds of the settlement at the existing Papillion Creek Dams occurred during construction, so approximately 2 feet of overbuild is expected to be necessary to account for post-construction settlement.
- The dam embankment will require a chimney drain and horizontal blanket drain to mitigate for seepage through the embankment.
- The Loess foundation soils at DS10 have dry densities and moisture contents above the threshold to exhibit any potential for consolidation or collapse upon wetting.

4.3 SIMPLIFIED HYDROLOGIC LOADING

4.3.1 PURPOSE

This section documents the simplified hydrologic loading development for incorporation into risk analysis for the potential Dam Site 10 (DS-10) of the Papillion Creek basin. DS-10 is a dry dam with no permanent pool. Due to time, funding, and data limitation associated with an ungaged new site, precipitation frequency hydrologic modeling was focused upon. The Risk Management Center Reservoir Frequency Analysis (RMC-RFA) model was not used and the stage frequency curve was developed from information from existing dams, peak flows determined from extreme precipitation, and engineering judgment. The loading curve should be further developed for advanced risk analysis to incorporate approximated period of record inflow and Monte Carlo simulation.

4.3.2 PROJECT SITE AND BACKGROUND

Figure 13 shows the location of DS-10 and Figure 14 shows the After ADM Dry Dam design. This was the design adopted after the Agency Decision Milestone (ADM). It is called the After ADM Dry Dam design in the Papillion Creek GRR study. The drainage area to the dam is 4.3 square miles. Refer to the Hydrology Appendix for more information if needed. Outflows from DS-10 contribute to Little Papillion Creek.

4.3.3 HYDROLOGIC MODEL

Figure 15 shows the HMS version 4.4 beta model (HEC, 2020). DS-10 was modeled with the 8 ft (Span) x 7 ft (Rise) box culvert outlet fully open for all frequency events. Unit hydrograph (UH) peaking was varied depending on the size of the event modeled. Events smaller than the 1/500 AEP had no UH peaking, the 1/1000 AEP had 25-percent UH peaking, and the 1/10,000 AEP had 50-percent UH peaking. Refer to the Hydrology Appendix for information on the Clark UH parameters used as well as the rating and storage curves for the dam. The hydrology model used the frequency storm meteorologic model to develop hyetographs. Basin model assumptions were consistent with those used in the most reasonable Probable Maximum Flood (PMF) modeling documented in the Hydrology Appendix.

4.3.4 PRECIPITATION

Peak flows used to inform the stage-frequency curve were developed from three sources: the Applied Weather Associates (AWA) report used in the FYRA study (FYRA, 2018), NOAA Atlas 14, and the RMC Best Fit model. Refer to the Hydrology Appendix and FYRA analysis (FYRA, 2018) for information on the AWA precipitation. Table 13 shows the NOAA Atlas 14 precipitation depths as well as the 1/10,000 AEP precipitation estimated with the RMC Best Fit model through consultation with subject matter experts from RMC. The depths of other durations were approximated using average ratios from the NOAA Atlas 14 data of the duration of interest to the 24-hour value.

Table 13. NOAA Atlas 14 precipitation & Best Fit 1/10,000 AEP estimate

DS10: NOAA Atlas 14 Median Depths (Inches)											
Return Interval (YRS)	**1**	**2**	**5**	**10**	**25**	**50**	**100**	**200**	**500**	**1,000**	**10,000**
AEP	0.999	0.5	0.2	0.1	0.04	0.02	0.01	0.005	0.002	0.001	0.0001
5-Minute	0.36	0.42	0.53	0.63	0.76	0.86	0.97	1.08	1.23	1.34	2.46
15-Minute	0.64	0.76	0.95	1.12	1.35	1.54	1.73	1.93	2.19	2.40	4.39
60-Minute	1.23	1.48	1.90	2.26	2.78	3.20	3.63	4.09	4.71	5.20	9.00
2-Hour	1.51	1.83	2.38	2.85	3.54	4.11	4.69	5.31	6.17	6.85	11.47
3-Hour	1.67	2.02	2.64	3.19	4.01	4.68	5.39	6.15	7.21	8.06	13.04
6-Hour	1.95	2.34	3.03	3.66	4.62	5.43	6.29	7.23	8.57	9.65	15.21
12-Hour	2.26	2.64	3.32	3.96	4.93	5.75	6.65	7.62	9.01	10.10	16.42
24-Hour	2.59	2.95	3.62	4.25	5.23	6.07	7.00	8.01	9.48	10.70	17.68

4.3.5 STAGE FREQUENCY CURVE

Figure 16 shows the adopted graphical stage frequency curve. This curve was informed by the peak inflows and stages produced by the precipitation discussed previously, through reference to the existing Papillion Creek Dams stage frequencies, and engineering judgement.

As mentioned before, the stage frequency curve presented has significant uncertainty, especially for infrequent events. A full risk analysis would include modeling with RMC-RFA which uses a Monte Carlo simulation with hundreds of realizations of observed inflow hydrographs scaled to randomly sampled peak flow frequencies and routed into the reservoir with different starting conditions. This modeling would provide a much more robust solution and confidence bounds around the stage frequency curve.

The stage-frequency curve for DS-19 beyond the 1/10,000 event was estimated by maintaining the same slope as that between the 1/1,000 and 1/10,000 annual exceedance probability (AEP) events and extrapolating out to include the top of dam.

Based on this estimate, the top of dam has an AEP of 1/1,900,000 and spillway flows have an AEP of 1/5,000 when the outlet is operational. If the outlet becomes blocked (no outlet), the top of dam would have an AEP of 1/800,000 and spillway flows would have an AEP of 1/2,000.

4.3.6 COMPARISON TO EXISTING PAPILLION CREEK DAMS

Table 14 shows the current stage-frequency curves from the most recent Periodic Assessments (PA) of the existing Papillion Creek dams along with that of the proposed DS-10 (USACE 2015, 2017, 2018a, 2018b DRAFT). Note that the existing Papillion Creek dam's stage-frequency curves were estimated using graphical fits and sometimes the outdated MCRAM Monte Carlo software. Only one PA, Dam Site 20-Wehrspann, leveraged today's best practice RMC-RFA software, and the report for that project is a draft.

It is estimated that DS-10 would have spillway flows and overtopping probabilities within the range of the existing Papillion Creek dams.

Table 14. Simplified Hydrologic Loading Curve Spillway and Overtopping Frequencies

Project	Number	Spillway Freq (AEP)	Top of Dam Freq (AEP)
Cunningham	Papio 11	1/70,000	1/10,000,000
Standing Bear	Papio 16	1/20,000	1/75,000
Zorinsky	Papio 18	1/3,300	1/1,000,000
Wehrspann	Papio 20	1/80,000	1/500,000
Proposed (DS-10)	Papio 10	1/5,000 (with outlet) 1/2,000 (no outlet)	1/1,900,000 (with outlet) 1/800,000 (no outlet)

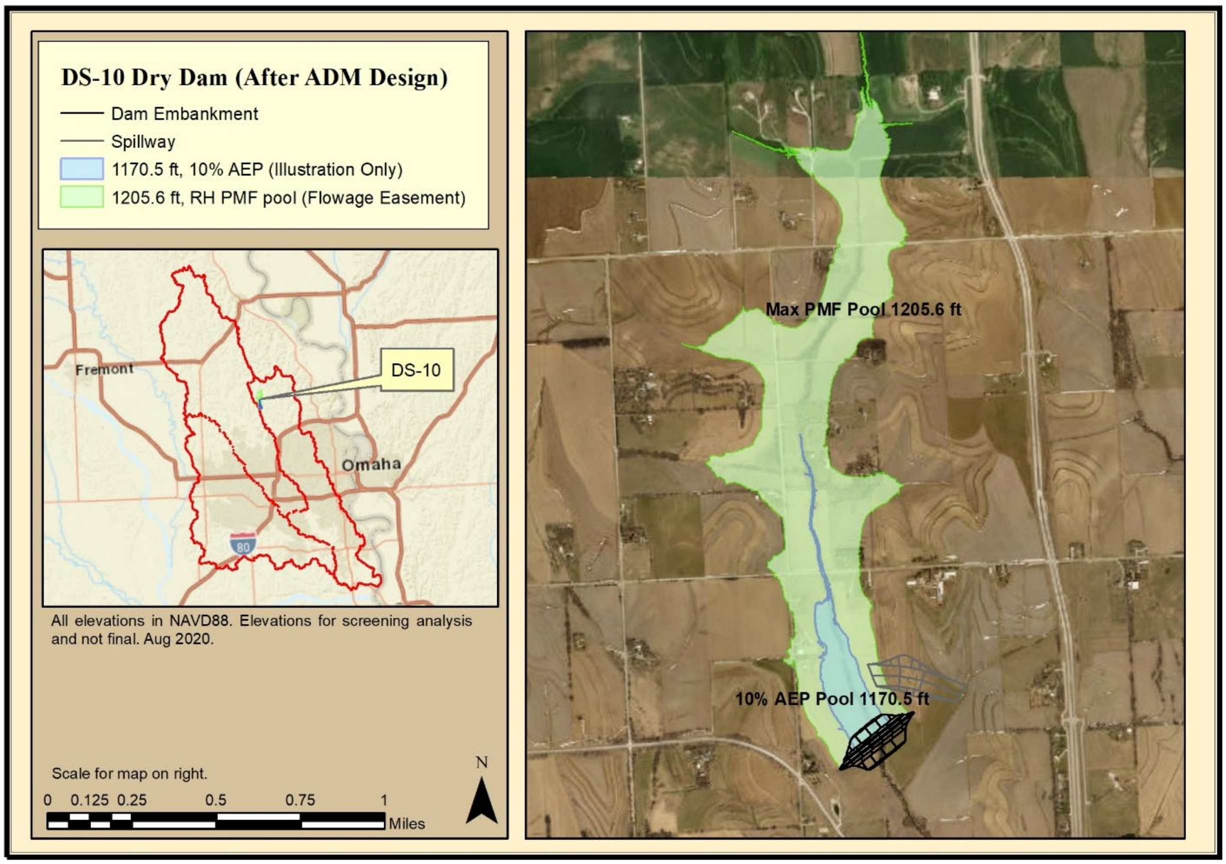


Figure 14. DS-10 Location. After ADM Dry Dam design

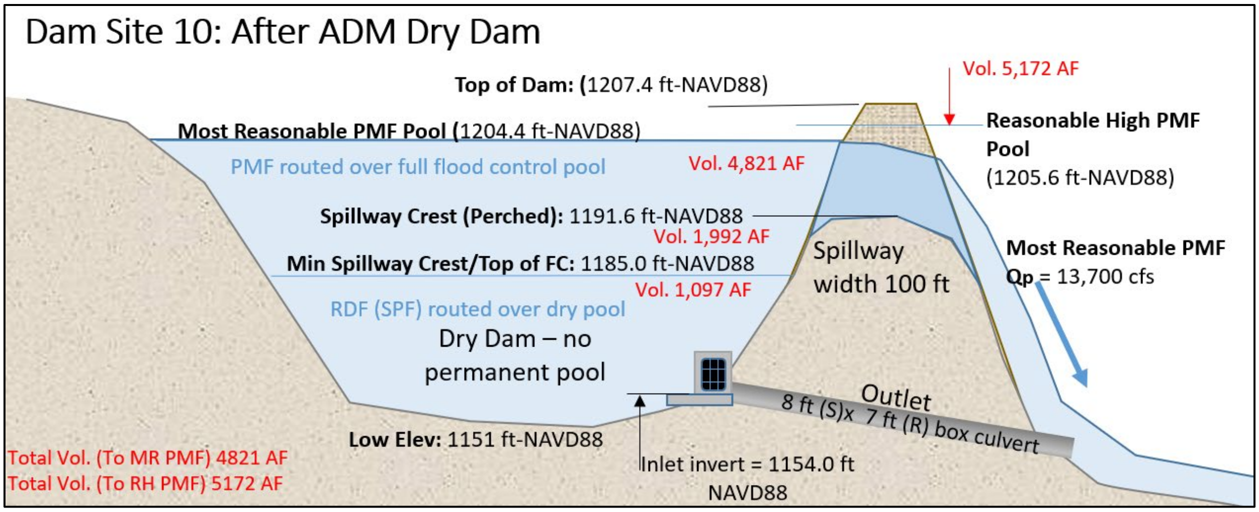


Figure 15. Dry Dam design

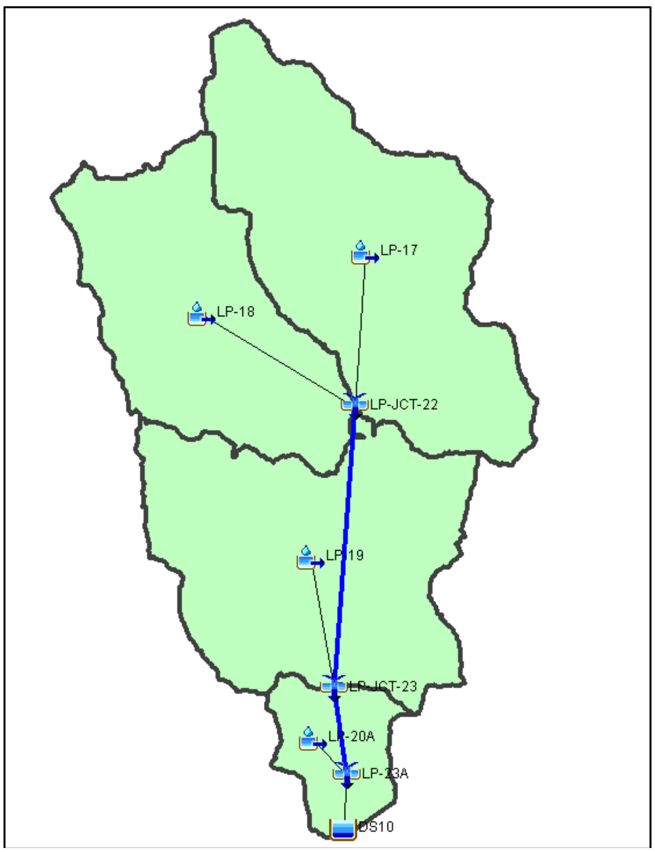


Figure 16. HEC-HMS model

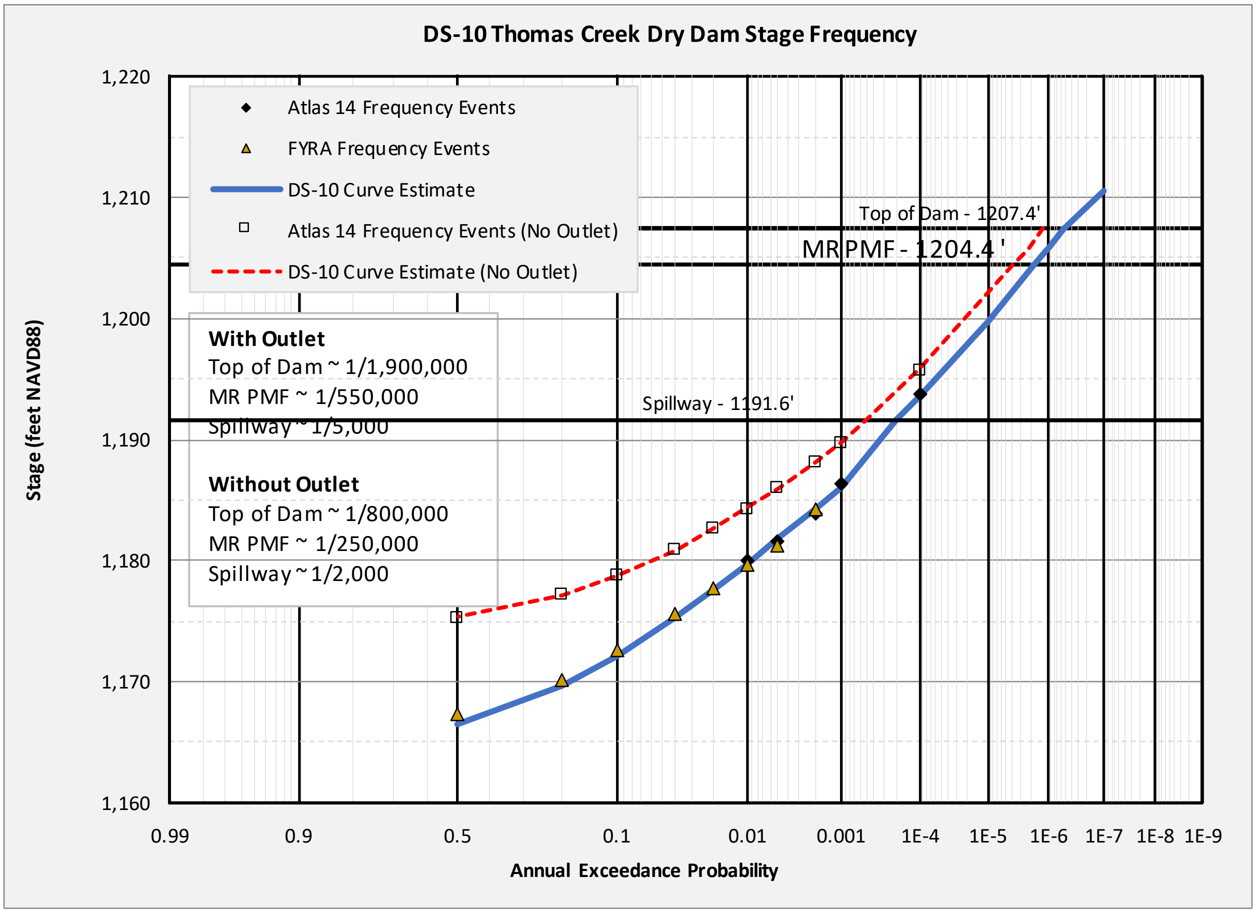


Figure 17. DS-10 adopted stage-frequency

4.4 SEISMIC HAZARDS

Previous seismic evaluations completed for existing Papillion Creek Dam Sites 11, 16, 18, and 20 determined that the central plains area where the Papillion Creek Dam Sites reside is considered tectonically stable with only occasional, minor earthquake activity. Due to the low seismicity in the area of DS10, all seismic related potential failure modes were excluded from consideration.

4.5 CONSEQUENCES

4.5.1 BACKGROUND

USACE has established a national standard of modeling procedures to support the estimation of consequences for breach and non-breach flood inundation scenarios over a range of loading conditions. Inundation models extend from the dam downstream to a point of no significant consequences. USACE developed baseline consequence estimates for breach and non-breach inundation scenarios, uncertainty statistics for life loss estimates, and inundation mapping products. The difference between breach and non-breach consequences for a particular loading condition is the incremental consequences (i.e., those directly attributable to the dam breach for that loading condition).

4.5.2 INUNDATION SCENARIOS

For DS10's abbreviated SQRA, several breach and non-breach scenarios were performed covering a range of pool elevations: max high pool (MH), top of active storage pool (TAS), and normal high pool (NH).

The maximum high pool elevation corresponds to the most reasonable probable maximum flood (PMF) pool elevation. For typical flood risk management dams with uncontrolled spillways, the top of active storage pool elevation corresponds to the emergency spillway crest elevation. Figure 17 provides a dam cross-section for DS10. The normal high pool elevation, also referred to as the 10% exceedance duration pool elevation, corresponds to the pool elevation that is exceeded approximately 10% of the time (36 to 37 days per year, on average) under normal operating conditions. This scenario represents a relatively high, though normal, pool condition that can be expected to occur every year.

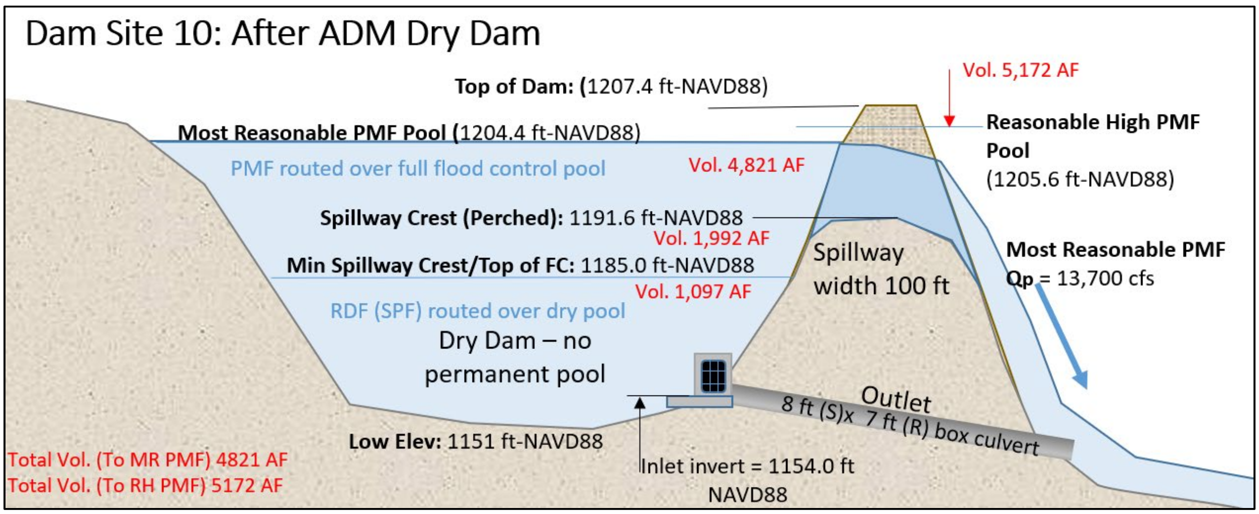


Figure 18. Dam Site 10 Cross-Section

Per the MMC guidance, project hydrology includes both inflows into the proposed reservoir as well as inflows to the downstream tributaries and is held constant between breach and non-breach scenarios. The max high pool scenario routes the most reasonable PMF hydrograph, from a starting pool elevation equal to that of the top of active storage pool, through the proposed dam. Meanwhile, the 10% Annual Exceedance Probability (AEP) event is routed along the downstream tributaries. The top of active storage pool scenario scales the most reasonable PMF hydrograph and routes that through the proposed dam, from a starting pool elevation equal to the 10% exceedance pool elevation. The scaling factor used must result in the pool reaching the top of active storage elevation. The 50% AEP event is routed along the downstream tributaries. The normal high pool scenario routes a constant inflow to maintain the 10% exceedance duration pool elevation through the proposed dam site and the 50% AEP event along the downstream tributaries. The values used for the 50% and 10% AEP events consisted of subbasin runoff for full build out conditions. Minimum flows equal to 10% of the 50% AEP were used at each boundary condition downstream of the dam sites to enhance model stability. Minimum flows were also used for dam inflows to hold starting pool elevations constant until the peak could be routed through.

Each embankment failure mode was assumed to be initiated by piping. The max high pool failure is initiated when the max high pool elevation is reached. Likewise, the top of active storage pool failure is initiated when the top of active storage pool elevation is reached. However, as the normal high pool failure scenario requires a constant pool elevation equal to the 10% exceedance duration pool elevation, failure is initiated 24 hours after the start of the model simulation.

Details for each failure and non-failure scenario are summarized in the Table 15 below.

Table 15. Details for DS10 Embankment Failure and Non-failure Scenarios

Model Scenario	Inflow Requirements	Downstream Flow Requirements	Starting Pool Elevation (ft)	Minimum Inflow (cfs)	Breach Trigger
MH-F	Most Reasonable PMF	10% AEP Future Build-out	1191.6	1642.36	1204.4’
MH-NF	Most Reasonable PMF	10% AEP Future Build-out	1191.6	1642.36	-
TAS-F	0.47*Most Reasonable PMF	50% AEP Future Build-out	1166.5	934.06	1191.6’
TAS-NF	0.47*Most Reasonable PMF	50% AEP Future Build-out	1166.5	934.06	-
NH-F	934.06 cfs	50% AEP Future Build-out	1166.5	934.06	24 hours after model initiation
NH-F	934.06 cfs	50% AEP Future Build-out	1166.5	934.06	-

Additionally, a breach of the spillway at the max pool elevation was evaluated. The breach trigger was assumed to happen at the maximum pool elevation. Details of this scenario is provided in Table 16.

Table 16. Details for DS10 Spillway Failure Scenario

Model Scenario	Inflow Requirements	Downstream Flow Requirements	Starting Pool Elevation (ft)	Minimum Inflow (cfs)	Breach Trigger
Spillway MH-F	Most Reasonable PMF	10% AEP Future Build-out	1191.6	1642.36	1204.4’

For additional information regarding the determination of the most reasonable PMF, pool elevations, and the downstream tributary flows, refer to Section 4.3 Simplified Hydrologic Loading.

4.5.3 BREACH ASSUMPTIONS

Breach parameters were calculated using four regression equations: MacDonald and Langridge-Monopolis, Froelich (1995a), Froelich (2008), and Von Thun and Gillette. These four equation sets have typically been used for earth dams. The embankment breaches were assumed to be initiated by piping that followed a sine wave progression. The Max High Pool breach plans were run with each set of regression equations and the resulting max inundations were compared. The outflow hydrographs from the DS10 breach do not converge until the confluence of the West Papillion and Big Papillion Creeks, although differences downstream of the Little Papillion and Big Papillion confluence appear minor. The first habitable structures are approximately 1.5 miles downstream of DS10. Figure 18 shows the difference in the hydrographs produced by the regression equations at this location.

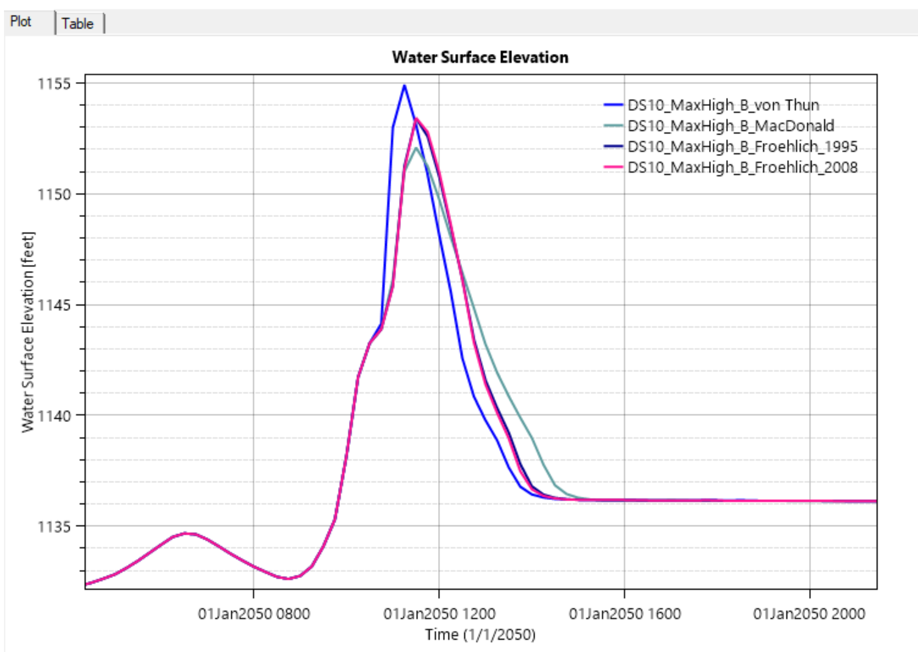


Figure 19. Outflow hydrographs from DS10 at closest habitable structures

Conservatively high (resulting from Von Thun and Gillette) and low (resulting from MacDonald and Langridge-Monopolis) values were not used. The remaining results were very similar in each case and Froelich (2008) was chosen to be the most appropriate for this application. Table 17 provides the required embankment information and resulting breach parameters for each scenario for DS10.

Table 17. Breach Parameters for DS10

Parameter	Max High Pool	Top of Active Storage	Normal High Pool
Top of Dam Elevation (ft)	1207.7		
Dam Crest Width (ft)	25		
Average u/s Slope of Dam Face (H:V)	5:1		
Average d/s Slope of Dam Face (H:V)	6:1		
u/s slope protection	Topsoil and Grass		
d/s slope protection	Topsoil and Grass		
Breach Bottom Elevation (ft)*	1163.63	1163.63	1157.94
Pool Elevation at Failure (ft)	1204.4	1191.6	1166.5
Pool Volume at Failure (acre-ft)	4821	1992	32.1
Failure Mode	Piping	Piping	Piping
Breach Bottom Width (ft)**	114	78	-5
Resulting Side Slopes (H:V)	0.7	0.7	0.7
Breach Development Time (hrs)	1.02	0.66	0.07

* Breach bottom elevation was taken to the lowest elevation terrain would allow
** negative values were input as zero

To adequately model a spillway breach, several assumptions were made. To determine an appropriate assumption for breach bottom elevation, past studies on the current Papillion Creek dams were reviewed. It is assumed that studies on the current dam sites are applicable to DS10 because of the similar design criteria, terrain, and expected soil conditions. Table 18 compares the details of each spillway.

Table 18. Comparison of Papillion Creek Dam Sites Spillway Criteria

	Papio Dam Site 11	Papio Dam Site 16	Papio Dam Site 18	Papio Dam Site 20	Papio Dam Site 10
Spillway Crest Length (ft)	200	282	200	232	100
Spillway Side Slopes (H:V)				3:1	3:1
Total Spillway Length (ft)	741	1,287	1,333	746	1,285
Total Elevation Drop (ft)	30.9	28.5	39.2	41	9.6
Long. Slope away from Crest (ft/ft)	.0020	0.0025	0.0020	0.0020	.0075
Peak Discharge (cfs)	18,700	9,500	30,000	17,500	13,300
Flow Duration (hrs)	30	27.8	31.8	40	12.5
Avg Spillway Velocity (ft/s)	6.95	8.10	10.17	7.14	7
Depth over Spillway (ft)	3.8	4.5	7.2	4.0	9

In 2007, a spillway erosion study was done on Dam Site 20 (DS20). The purpose of the study was to use the latest computer modeling techniques to determine erosion impacts to the DS20 emergency spillway. Based on the study model and the best available data at that time, the spillway head cut erosion is not generally considered severe enough to breach the spillway crest.

However, the model did indicate that a breach is possible using a distribution sampling analysis and conservative soil variables. Figure 19 shows the resulting spillway cross-section under three distribution conditions. The maximum condition is denoted by the yellow line, the minimum condition is denoted by the green line, and the mean condition is denoted by hatching. Due to the longer spillway length and shorter spillway duration, to assume a full breach, as is shown in the maximum condition, would be overly conservative. Therefore, the mean condition, with a most upstream head cut depth of 6 feet, was assumed for the DS10 spillway breach depth.

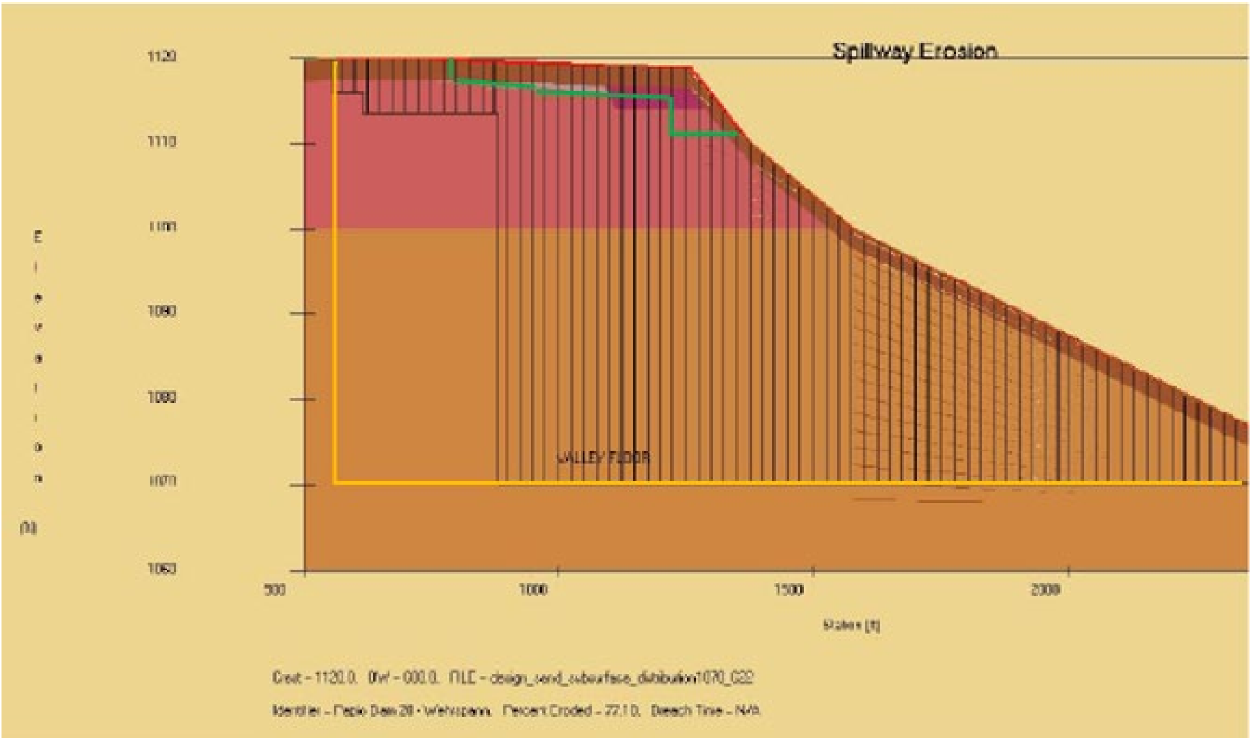


Figure 20. Distribution Conditions for the expected erosion on DS20 assuming conservative soil variables

The next assumption is that there will be mass wasting of the spillway downstream of the crest until finally there only remains a sliver of spillway, similar to that of a dam embankment, making the regression equation calculator in HEC-RAS applicable for this scenario. For consistency, Froelich (2008) was again used to calculate breach parameters. The failure mode for each is overtopping triggered at the time the max high pool elevation was reached. This is a conservative assumption because erosion of the spillway would begin at the downstream edge and require some time to work its way back to the pool. This would likely take more time than is necessary to reach the peak pool elevation, resulting in quicker arrival times. Table 19 provides the required embankment information and resulting breach parameters for the DS10 spillway failure.

Table 19. DS10 Spillway Breach Parameters

Parameter	DS10 Spillway
Spillway Crest Elevation (ft)	1191.6
Spillway Crest Width (ft)	25
Average u/s Spillway Channel Slope (H:V)	5:1
Average d/s Spillway Channel Slope (H:V)	6:1
u/s spillway channel protection	Topsoil and Grass
d/s spillway channel protection	Topsoil and Grass
Spillway Breach Bottom Elevation (ft)*	1185.6
Pool Elevation at Failure (ft)	1204.4
Pool Volume at Failure (acre-ft)	4821
Failure Mode	Spillway Erosion
Breach Bottom Width (ft)	168
Resulting Side Slopes (H:V)	1
Breach Development Time (hrs)	7.47

4.5.4 LIFE LOSS

Population at risk (PAR) is defined as the number of people downstream of a dam that would be subject to inundation risk. PAR and life loss estimates were generated using HEC's LifeSim (HEC-LifeSim) software for breach and non-breach inundation scenarios.

Table 20. DS10 Estimated Population at Risk

Reservoir Level	Population at Risk					
	Breach		Non-Breach		Incremental	
	Day	Night	Day	Night	Day	Night
MHP	13,893	4,289	3,127	1,139	10,766	3,150
MHP – Spillway Breach	11,359	3,172	3,127	1,139	8,232	2,033
TAS	1,020	484	4	8	1,016	476
NHP	4	8	4	8	0	0

4.5.4.1 Assumptions and Methodology

The life loss methodology in *HEC-LifeSim* is based on the *LifeSim* methodology. To determine the percentage of population at risk (PAR) within a structure that is warned and mobilized over time, several parameters are used within *HEC-LifeSim* to estimate the probable values of warning and mobilization percentages at each time step. These include when warnings will be issued (hazard identification and delays), how long they will take to become effective (warning diffusion), and the rate at which PAR will mobilize in response (mobilization). Figure 20 represents an example dam breach warning and mobilization timeline.

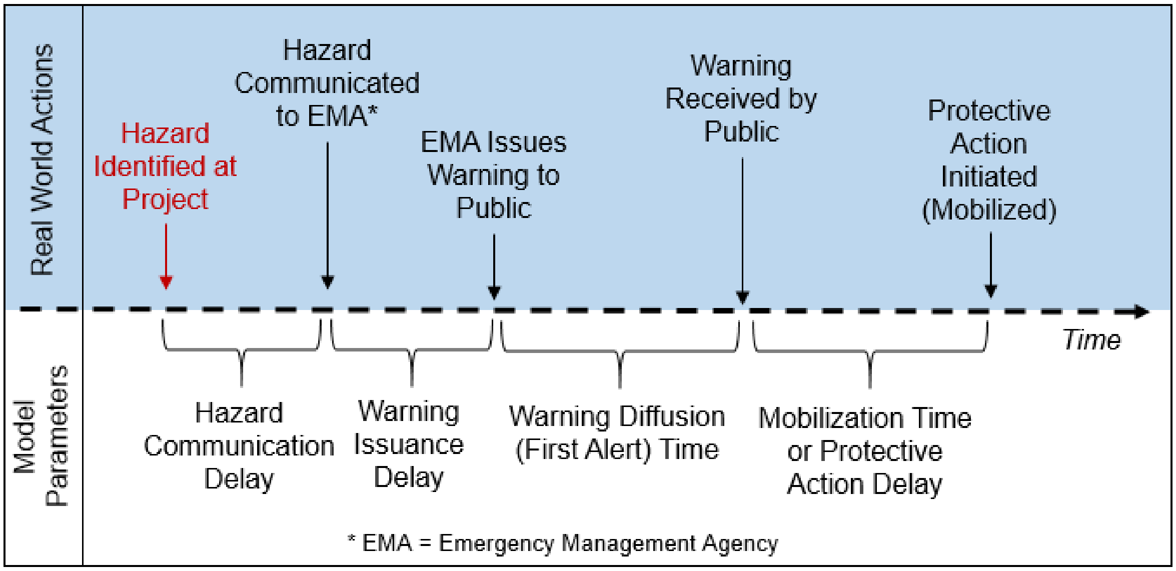


Figure 21. Dam Breach Warning and Mobilization Timeline

- The Hazard Identification time is the time at which a hazard is identified (dam breach or major flooding) relative to when it actually occurs (the actual breach initiation time). For example, a hazard identification one hour prior to breach initiation would be "-1 hour," meaning that the hazard was initially identified one hour before it actually occurred. The MMC uses two different warning scenarios with different ranges of hazard identification time: minimal warning and ample warning.
  - Minimal Warning scenarios have the hazard identification relative time set as a uniform distribution between 2 hours prior to breach initiation and at time of breach initiation (-2 to 0 hours).
  - Ample Warning scenarios have the hazard identification relative time set as a uniform distribution between 6 hours prior to breach initiation and 2 hours prior to breach initiation (-6 to -2 hours).
  - For both scenarios, in-pool areas and non-breach double-warning areas are set at least 72 hours prior to the simulation start.

- The Hazard Communication Delay is the time that it would take from when the hazard is identified to when the emergency planning zone (EPZ) representatives would be notified. For example, if a breach occurs when no one is observing the project then the emergency managers could be notified 1 hour after the hazard is identified. The hazard communication delay is set as a uniform uncertainty distribution between 0.01 hours and 0.5 hours.

- The Warning Issuance Delay is the time it takes from when the emergency managers receive the notification of the imminent hazard to when they issue the first evacuation order to the public. The warning issuance delay is set at the preset configuration of "Preparedness Unknown," which utilizes a Lindell uncertainty distribution. The delay is randomly sampled from 0 to 6 hours, but it is positively skewed such that results from 0 to 1.5 hours are more likely.

- The Warning Diffusion or First Alert parameter defines the warning diffusion curve for daytime and nighttime. The diffusion curve represents the percentage of the population which will receive a first alert warning over time during daytime hours from when the warning was issued. The first alert curves are set at the preset configuration of "Unknown" which samples from a uniform uncertainty distribution where the upper bound curve warns 100 percent of the PAR after 1.5 hours and the lower bound curve warns 100 percent of the PAR after 6 hours.

- The Protective Action Initiation (PAI) parameter defines the mobilization curve. The PAI curve represents the percentage of the population which will take protective action over time from when the first alert is received. For areas downstream of the dam the PAI/mobilization curve for is set at the preset configuration of "Preparedness: Unknown / Perception: Unknown" which samples a uniform uncertainty distribution with maximum mobilization rates between 83 and 100 percent after 72 hours. For in-pool areas, the curve is set at the preset "Preparedness: Unknown / Perception: Likely to Impact" which samples a uniform uncertainty distribution with maximum mobilization rates between 94 and 100 percent after 72 hours.

4.5.4.2 Life Loss Uncertainty

Life loss was estimated using uncertainty sampling methods on the parameters in *HEC-LifeSim*. These parameters include the warning issuance, the warning delay and diffusion curves, and the mobilization rate curve. For this reason, life loss results are presented below with five number statistics in order to understand the potential range of life loss. In order to provide a generic suite of warning scenarios that could be used during the risk assessment for the risk-driver potential failure modes, minimal and ample warning scenarios (as described above in the HEC-LifeSim parameters section) are used. It should be noted that there is also a standard delay parameter added onto the warning issuance time based on case histories. The ample warning scenario is generally more appropriate for internal erosion PFMs where failure progression is observed and discovery occurs before breach initiation (i.e., dams that are watched) and overtopping. The minimal warning scenario is generally more appropriate for seismic PFMs where failure can be instantaneous or where failure progression is not observed (i.e., dams that are not watched). The estimated life loss statistics for the two warning issuance scenarios are summarized in the following tables. While the breach and non-breach statistics represent the outcome from the simulations, the incremental life loss "statistics" were obtained by subtracting the breach and non-breach statistics.

4.5.4.3 Life Loss Results

Table 21 provides a summary of consequence information with minimal warning from DS10 modeling. Information provided includes number of inundated structures, population at risk and median life loss estimates. As shown in Table 21, no median life loss estimates are greater than 0 up to the MHP loading scenario.

Table 21. DS10 Estimated Downstream Information by Reservoir Level and Breach Scenario (Minimal Warning)

Reservoir Level	Number of Structures	Population at Risk		Median Life Loss	
		Day	Night	Day	Night
MHP	1,430	13,893	4,289	14	12
MHP – Spillway Breach	998	11,359	3,172	0	0
TAS	217	1,020	484	0	0
NHP	2	4	8	0	0

The tables below display the results for modeling DS10. Results are provided MHP, TAS, and NHP pool heights. An additional table is provided for each dam displaying modeling results for MHP with a spillway failure.

Table 22. DS10 Estimated Life Loss for MHP Breach

Statistic	Life Loss for Minimal Warning Scenario					
	Breach		Non-Breach		Incremental	
	Day	Night	Day	Night	Day	Night
95th Percentile	28	22	0	0	28	22
75th Percentile	19	16	0	0	19	16
Median	14	12	0	0	14	12
25th Percentile	9	9	0	0	9	9
5th Percentile	4	4	0	0	4	4

Statistic	Life Loss for Ample Warning Scenario					
	Breach		Non-Breach		Incremental	
	Day	Night	Day	Night	Day	Night
95th Percentile	24	21	0	0	24	21
75th Percentile	16	14	0	0	16	14
Median	11	10	0	0	11	10
25th Percentile	7	6	0	0	7	6
5th Percentile	2	2	0	0	2	2

Table 23. DS10 Estimated Life Loss for TAS Breach

Statistic	Life Loss for Minimal Warning Scenario

	Breach		Non-Breach		Incremental	
	Day	Night	Day	Night	Day	Night
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0

Statistic	Life Loss for Ample Warning Scenario					
	Breach		Non-Breach		Incremental	
	Day	Night	Day	Night	Day	Night
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0

Table 24. DS10 Estimated Life Loss for NHP Breach

Statistic	Life Loss for Minimal Warning Scenario					
	Breach		Non-Breach		Incremental	
	Day	Night	Day	Night	Day	Night
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0

Statistic	Life Loss for Ample Warning Scenario					
	Breach		Non-Breach		Incremental	
	Day	Night	Day	Night	Day	Night
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0

The table below displays the results of a spillway failure at the MHP level.

Table 25. DS10 Estimated Life Loss for MHP - Spillway Failure

Statistic	Life Loss for Minimal Warning Scenario					
	Breach		Non-Breach		Incremental	
	Day	Night	Day	Night	Day	Night
95th Percentile	2	1	0	0	2	1
75th Percentile	1	0	0	0	1	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0

Statistic	Life Loss for Ample Warning Scenario					
	Breach		Non-Breach		Incremental	
	Day	Night	Day	Night	Day	Night
95th Percentile	2	1	0	0	2	1
75th Percentile	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0

4.5.5 ADDITIONAL CONSIDERATIONS

ER 1110-2-1451 requires lands downstream of spillways to be acquired if spillway discharge could create or significantly increase a hazardous condition. The ER further defines non-hazardous conditions to be those areas with:

1. Maximum flood depths of 2 feet in both urban and rural areas
2. Flood depths that are essentially non-damaging to urban property
3. Flood durations of a maximum of 3 hours in urban areas and 24 hours in agricultural areas
4. Velocities that do not exceed 4 ft/s
5. Minimal debris and erosion potential
6. Flood frequency less than 1%

To evaluate the creation of and increase to hazardous conditions downstream of the DS10 spillway, the 2D hydraulic model was run with both with- and without-project conditions using PMF project hydrology upstream of the dam location and the 10% AEP event downstream of the dam to determine the increase to flood depths and velocities. Once areas of significant increase to and creation of hazard conditions were identified, further analysis was conducted to determine areas where flood depths were greater than 2ft or velocities were greater than 4 ft/s. 11.5 acres were identified for potential acquisition due to expected hazard conditions downstream of the DS10 spillway, see Figure 21 below.

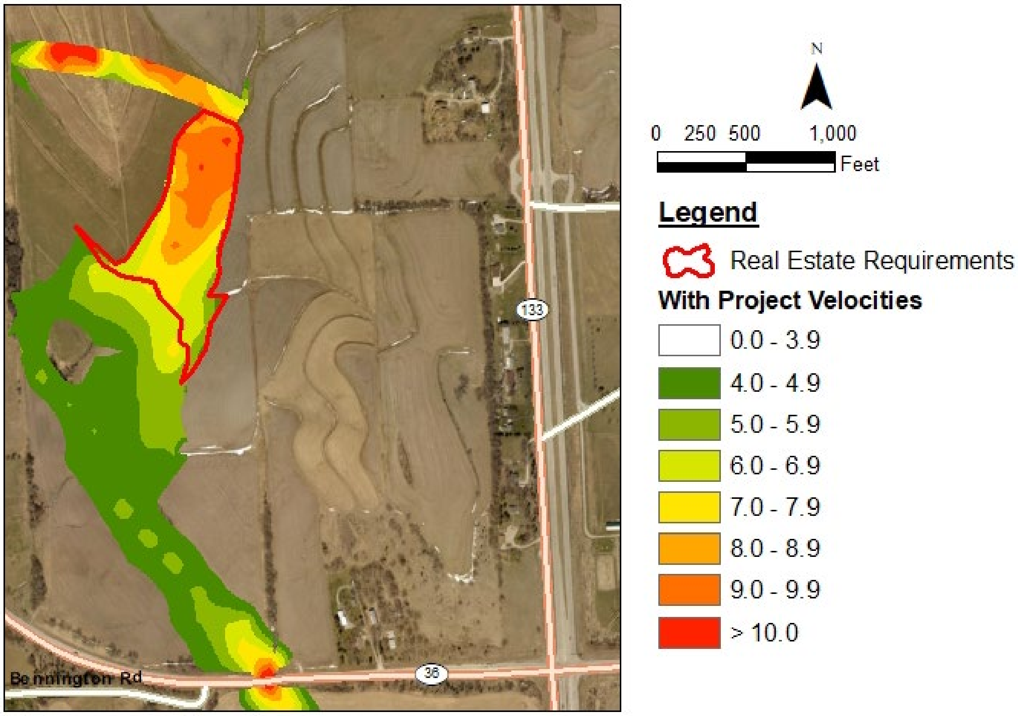


Figure 22. Real Estate Requirements downstream of DS10 Spillway

4.6 ABBREVIATED SEMI-QUANTITATIVE RISK ASSESSMENT

4.6.1 POTENTIAL FAILURE MODE ANALYSIS

A failure mode is a unique set of conditions and/or sequence of events that could result in failure, where failure is "characterized by the sudden, rapid, and uncontrolled release of impounded water" (FEMA 2003). A Potential Failure Mode Analysis (PFMA) is the process of identifying and fully describing potential failure modes. Due to an aggressive schedule to complete the General Reevaluation Report (GRR) for the Papillion Creek Basin, the SQRA for DS10 and DS19 was abbreviated to complete in one day with the team. To save time, brain-storming to identify potential failure modes (PFMs) was completed by a smaller team of engineers prior to meeting based off the available preliminary design for DS10 and review of design, performance, and failure mode analyses conducted for existing Papillion Creek Dam Sites 11, 16, 18 and 20. The goal of the abbreviated SQRA team was then to further evaluate and develop the potential failure modes, based on the team's understanding of the project vulnerabilities resulting from the review of the preliminary design. The abbreviated SQRA team is summarized in Table 26.

Table 26. Abbreviated SQRA Team

Name	Office	Discipline
Kyle Heddens	Omaha District	Geotechnical Engineer (Facilitator)
Bob Worden	Omaha District	Geotechnical Engineer (Co-Facilitator)
Ross Cullin	Omaha District	Geotechnical Engineer (DSPM)
Steve Butler	Omaha District	Geotechnical Engineer
Jamie Bond	Walla Walla District	Geotechnical Engineer
Laila Berre	Northwest Division	Geotechnical Engineer (Division DSPM)
Brad Bird	Northwest Division	Hydraulics Engineer
Roger Kay	Omaha District	Hydraulics Engineer
Laura Knapp Leiferman	Omaha District	Hydraulics Engineer
Ben Lorenzen	Omaha District	Hydraulics Engineer
Joshua Melliger	Omaha District	Hydrology Engineer
Jennifer Christensen	Omaha District	Hydrology Engineer
Rachel Shrader	Omaha District	Planning Project Manager
Greg Johnson	Omaha District	Planning Project Manager

From the list of potential failure modes developed prior to the abbreviated SQRA (Table 27), the team identified the failure modes judged to be risk drivers. Failure modes that were determined to be non-risk drivers were excluded from further consideration. An abbreviated justification for the exclusion of the non-risk drivers is provided in Section 4.6.2. For the risk driver failure modes, the pertinent background and performance data was discussed. Then, a complete failure description was prepared from initiation to breach. The discussion was then expanded to listing factors, data, or conditions that suggest the failure mode is more likely or less likely to occur and establishing the appropriate level of consequences. Lastly, any recommendations for risk-reduction actions to be incorporated into the preliminary design of DS10 to achieve all four Tolerable Risk Guidelines (TRGs) were discussed.

Table 27. DS10 Potential Failure Modes

PFM	Description
PFM 01	Seismic liquefaction of foundation causes crest settlement and overtopping
PFM 02	Seismic slope deformation (liquefaction/cyclic softening) of embankment and overtopping
PFM 03	Seismic slope stability failure
PFM 04	Seismic induced transverse cracking
PFM 05	Static slope stability failure (US/DS)
PFM 06	Overwash erosion
PFM 07	Overtopping
PFM 08	Concentrated Leak Erosion (CLE) through transverse crack in embankment at closure contact
PFM 09	CLE through transverse crack in embankment above chimney drain
PFM 10	CLE along the conduit
PFM 11	Erosion of embankment material into the conduit joints
PFM 12	CLE along conduit, driven by pressurized conduit flow and water exiting the joints

PFM	Description
PFM 13	CLE through conduit joints
PFM 14	Backwards Erosion Piping (BEP) of non-plastic layer in glacial till foundation at outlet works channel excavation
PFM 15	Spillway Erosion
PFM 16	BEP of alluvial foundation
PFM 17	Overtopping of the dam due to clogged/damaged outlet works
PFM 18	CLE due to filter incompatibility of drains or conduit filter
PFM 19	Seismic failure of intake structure causing uncontrolled release
PFM 20	Seismic failure of conduit, causing CLE of embankment soils into joints
PFM 21	CLE at closure section due to poor compaction at interface
PFM 22	CLE erosion at old stream channel due to poor compaction
PFM 23	CLE at embankment/loess abutment interface due to collapse of loess from wetting
PFM 24	CLE along a poorly compacted layer at the embankment / alluvial foundation interface
PFM 25	Clogged internal drains

4.6.2 EXCLUDED FAILURE MODES

The following sections summarize the potential failure modes that were excluded from further consideration because they were deemed non-credible or credible but non-risk drivers.

4.6.2.1 Seismic Failure Modes

The following excluded potential failure modes are seismic failure modes:

- PFM 01: Seismic liquefaction of foundation causes crest settlement and overtopping;
- PFM 02: Seismic slope deformation (liquefaction/cyclic softening) of embankment and overtopping;
- PFM 03: Seismic slope stability failure;
- PFM 04: Seismic induced transverse cracking;
- PFM 19: Seismic failure of intake structure causing uncontrolled release
- PFM 20: Seismic failure of conduit, causing CLE of embankment soils into joints

There is low seismicity in the area and all seismic related potential failure modes were excluded from consideration based on coincident probability and earthquake analysis from the existing Papillion Creek Dams.

4.6.2.2 Slope Stability Failure Modes

Stability analyses based on material properties from foundation investigations at DS10 were completed in Specific Design Memorandum No. MPC-33. The calculated factor of safety determined from each of the stability analysis meets or exceeds minimum factor of safety requirements. Therefore, PFM 05: Static slope stability failure (US/DS) was excluded from consideration.

4.6.2.3 Overtopping Failure Modes

The following excluded potential failure modes are related to overtopping:

- PFM 06: Overwash erosion;
- PFM 07: Overtopping;
- PFM 17: Overtopping of the dam due to clogged/damaged outlet works

The annual exceedance probability (AEP) of a hydrologic event that would raise the pool elevation to the design crest elevation is 1/1,900,000 with an operational outlet works or 1/800,000 without no assumed outflow from the outlet works. DS10 is approximately 280 feet wide from the landside toe to the crest centerline and is designed with a 25 foot wide crest, so a hydrologic event significantly less probable than the top of dam event (AEP 1/1,900,000 w/ outlet works, AEP 1/800,000 w/o outlet works) would be necessary to have the depth and duration of overtopping required to initiate and progress erosion of the downstream embankment slope to breach. Due to the improbable hydrologic loading condition, potential failure modes relating to overtopping were excluded from consideration.

4.6.2.4 CLE/BEP through the Embankment

The following excluded potential failure modes are related to concentrated leak erosion (CLE) through the embankment:

- PFM 08: CLE through transverse crack in embankment at closure contact;
- PFM 09: CLE through transverse crack in embankment above chimney drain
- PFM 18: CLE due to filter incompatibility of drains or conduit filter
- PFM 21: CLE at closure section due to poor compaction at interface
- PFM 25: Clogged internal drains

The potential failure modes related to CLE through the embankment will require a hydrologic event that would raise the pool elevation above the top of active storage (TAS) and spillway crest loading elevation of 1191.6 feet NAVD88 (AEP 1/5,000) resulting in a tailwater elevation of 1160.5 feet NAVD88 and a head differential of 31.1 feet. The duration of such an event is 38 hours from the starting elevation of 1166.5 feet NAVD88 to the peak pool elevation of 1191.6 feet NAVD88, and back down to the box culvert intake at elevation 1154 feet NAVD88. For reference, the MHP or Most Reasonable (MR) PMF elevation of 1204.4 ft NAVD88 would require a hydrologic event with an AEP of 1/550,000. The MHP scenario would result in a tailwater of 1166.3 feet NAVD88 and a head differential of 38.1 feet. The duration of the MHP event is 41 hours from the starting elevation of 1191.6 feet NAVD88 to the peak pool elevation of 1204.4 feet NAVD, and back down to the box culvert intake at elevation 1154 feet NAVD88. Since DS10 is a dry dam with no permanent pool and short duration events, the embankment is very unlikely to develop steady-state seepage conditions before the pool recedes during high water events.

The outlet works structure, consisting of a box culvert with a 8 foot span and a 7 foot rise, can pass a hydrologic event with an AEP of 1/50 with a maximum rise in the pool of 1166 feet NAVD88, which is only approximately one foot higher than the existing floodplain elevation. This should allow construction to occur without the need for a closure section. Additionally, the size of the project allows for construction to be completed in one season. These factors reduce or eliminate the potential for a crack or poorly compacted layer at the closure section or due to poor construction practices caused by winter shutdowns.

If there is a flaw in the embankment, such as a crack or poorly compacted layer, the internal chimney and blanket drain are designed to reduce seepage pressures and retain eroded material to stop the progression of CLE. The material for the internal chimney and blanket drain will be properly sized to meet filter criteria for permeability, particle retention, and flow. Additionally, the embankment has a 25-foot-wide crest, 1V on 5H upstream slopes, and 1V on 6H downstream slopes, and is therefore wide. The typical embankment section is 500 feet wide at its base at elevation 1165 feet NAVD88, resulting in a global gradient of 0.05 (1191.6 TAS EL – 1165 flaw EL / 500 feet progression) at TAS (AEP 1/5,000) and 0.08 (1204.4 MHP EL – 1165 flaw EL / 500 feet progression) at MHP (AEP 1/550,000) which are insufficient to initiate and progress erosion.

For all the above reasons, it was determined that all potential failure modes related to CLE through the embankment would be excluded from consideration.

4.6.2.5 CLE at the Outlet Works

The following potential failure modes are related to concentrated leak erosion (CLE) at the outlet works:

- PFM 10: CLE along the conduit;
- PFM 11: Erosion of embankment material into the conduit joints;
- PFM 12: CLE along conduit, driven by pressurized conduit flow and water exiting the joints
- PFM 13: CLE through conduit joints

The potential failure modes related to CLE at the outlet works will require a hydrologic event that would raise the pool elevation above the top of active storage (TAS) and spillway crest loading elevation of 1191.6 feet NAVD88 (AEP 1/5,000) resulting in a tailwater elevation of 1160.5 feet NAVD88 and a head differential of 31.1 feet. The duration of such an event is 38 hours from the starting elevation of 1166.5 feet NAVD88 to the base of the box culvert at elevation 1154 feet NAVD88. For reference, the MHP or MR PMF elevation of 1204.4 ft NAVD88 would require a hydrologic event with an AEP of 1/550,000. The MHP scenario would result in a tailwater of 1166.3 feet NAVD88 and a head differential of 38.1 feet. The duration of the MHP event is 41 hours from the starting elevation of 1191.6 feet NAVD88 to the peak pool elevation of 1204.4 feet NAVD, and back down to the box culvert intake at elevation 1154 feet NAVD88.

The reinforced concrete box culvert will span 700 feet and will be founded entirely on stiff to very stiff glacial drift in the left abutment. Outlet works structures founded on similar glacial drift material at the existing Papillion Creek Dams have only experienced approximately 0.2 feet of settlement. Seepage along the outlet works under "normal" seepage conditions or due to cracks or flaws adjacent to the outlet works will be collected by a 10-foot-long and 3-foot-wide pervious backfill drain near the outfall of the box culvert. The drain material will be properly sized to meet filter criteria for permeability, particle retention, and flow. The stilling basin will be protected with riprap revetment.

CLE at the outlet works will have to progress the 700-foot length of the box culvert resulting in a global gradient of 0.04 (1191.6 TAS EL – 1160.5 tailwater EL / 700 feet progression) at TAS (AEP 1/5,000) and 0.05 (1204.4 MHP EL – 1166.3 tailwater EL / 700 feet progression) at MHP (AEP 1/550,000) which are insufficient to initiate and progress erosion. Additionally, since DS10 is a dry dam with no permanent pool and short duration events, the impervious backfill around the outlet works is very unlikely to experience these high of gradients before the pool recedes during high water events.

For all the above reasons, it was determined that all potential failure modes related to CLE through the embankment would be excluded from consideration.

4.6.2.6 CLE/BEP through the Foundation

The following potential failure modes are related to concentrated leak erosion (CLE) or backward erosion piping (BEP) through the foundation:

- PFM 16: BEP of alluvial foundation;
- PFM 22: CLE erosion at old stream channel due to poor compaction
- PFM 23: CLE at embankment/loess abutment interface due to collapse of loess from wetting
- PFM 24: CLE along a poorly compacted layer at the embankment / alluvial foundation interface

The potential failure modes related to CLE or BEP through the foundation will require a hydrologic event that would raise the pool elevation above the top of active storage (TAS) and spillway crest loading elevation of 1191.6 feet NAVD88 (AEP 1/5,000) resulting in a tailwater elevation of 1160.5 feet NAVD88 and a head differential of 31.1 feet. The duration of such an event is 38 hours from the starting elevation of 1166.5 feet NAVD88 to the base of the box culvert at elevation 1154 feet NAVD88. For reference, the MHP or MR PMF elevation of 1204.4 ft NAVD88 would require a hydrologic event with an AEP of 1/550,000. The MHP scenario would result in a tailwater of 1166.3 feet NAVD88 and a head differential of 38.1 feet. The duration of the MHP event is 41 hours from the starting elevation of 1191.6 feet NAVD88 to the peak pool elevation of 1204.4 feet NAVD, and back down to the box culvert intake at elevation 1154 feet NAVD88.

Boring and testing data performed at DS10 for Specific Design Memorandum No. MPC-33 found that the loess foundation soils at DS10 have dry densities between 88 pcf and 102 pcf and moisture contents consistently above the plastic limit. Since the dry densities and moisture contents are above the thresholds to exhibit any potential for consolidation or collapse upon wetting, Specific Design Memorandum No. MPC-33 for DS10 did not recommend foundation treatment such as prewetting or excavation of the loess.

Current construction practices will be used to construct DS10. All highly organic and objectionable foundation materials, such as rubbish, vegetation, roots, and muck will be removed from the foundation and old streambed channel. Foundation preparation, such as clearing, grubbing, scarifying, and recompacting the foundation surface, will be completed to ensure a good contact with the placed embankment fill. Finally, a 6-foot-deep by 10-foot wide-inspection trench with 1V on 2H side slopes will be excavated to the spillway crest elevation of 1,191.6 feet NAVD88 to break the continuity of the surface soil structure by replacing it with compacted impervious fill and to identify unforeseen soft areas near the ground surface that will require procedural changes during construction.

If there is a flaw at the contact between the embankment and abutment foundation material, such as a crack or poorly compacted layer, the blanket drain is designed to reduce seepage pressures and retain eroded material to stop the progression of CLE. Additionally, the original streambed channel will be lined with 3 feet of pervious fill to provide a controlled outlet for seepage.

The length of the pipe (BEP) or crack/flaw (CLE) will vary based off the location of the potential failure mode (PFM). The width of the typical embankment section from the upstream toe to the downstream toe, 500 feet, was assumed for the purpose of the abbreviated SQRA, and results in a global gradient of 0.05 (1191.6 TAS EL – 1165 flaw EL / 500 feet progression) at TAS (AEP 1/5,000) and 0.08 (1204.4 MHP EL – 1165 flaw EL / 500 feet progression) at MHP (AEP 1/550,000). If the pipe or crack/flaw is in the abutments, the length of erosion progression could be less than 500 feet, but the head will decrease due to the higher elevation of the flaw, so gradients will be similar. The calculated global gradients are insufficient to initiate and progress CLE; however, they could initiate and potentially progress BEP in a uniform, fine sand. Sand or gravel layers were not encountered in the borings for DS10 but have been observed in the glacial till foundations at the existing Papillion Creek Dams. Therefore, PFM 14: Backwards Erosion Piping (BEP) of non-plastic layer in glacial till foundation at outlet works channel excavation was carried forward as a risk-driving failure mode and PFM 16: BEP of alluvial foundation was excluded from consideration with the rest of the potential failure modes related to CLE through the foundation for all the above reasons.

4.6.3 RISK ASSESSMENT

A risk assessment was performed for the following potential failure modes judged to be risk drivers:

- PFM 14: Backwards Erosion Piping (BEP) of a non-plastic layer in the glacial till foundation at the outlet works channel excavation
- PFM 15: Spillway Erosion

The incremental risk (due to failure or breach) includes a consideration of both likelihood of failure and the incremental consequences. The likelihood of failure is a function of both the likelihood of the loading condition that could lead to the failure and the likelihood of failure given the loading condition. During the risk assessment, order-of-magnitude estimates were made for both likelihood of failure and incremental consequences (based on estimated consequences and the team's judgment) for each risk-driver potential failures mode. The evaluation of each risk-driver potential failure mode was documented as well as the team's confidence in the order-of-magnitude estimates. Confidence describes the potential impacts to the risk characterization and the decision to take action to reduce risk or reduce uncertainty.

Table 28. Confidence Categories

Confidence Level	Description
Low	The team is not confident in the risk characterization, and it is entirely possible that additional information would change the decision.
Moderate	The team is relatively confident in the risk characterization, but key additional information might possibly change the decision.
High	The team is confident in the risk characterization, and it is unlikely that additional information would change the decision.

4.6.4 RISK-DRIVER POTENTIAL FAILURE MODE DISCUSSION

4.6.4.1 PFM 14: Backwards Erosion Piping (BEP) of a non-plastic layer in the glacial till foundation at the outlet works channel excavation

4.6.4.1.1 Description

The DS10 reservoir is near the max high pool (MHP) storage elevation of 1204.4 feet NAVD88 (AEP 1/550,000) with a tailwater elevation of 1166.3 feet NAVD88. A continuous layer of poorly graded, low C_u sand in the Kansan glacial drift is exposed during the channel excavation to connect the outlet works with the existing streambed channel. Exit gradients are enough to initiate backward erosion piping of the foundation sand, which exits unfiltered and undetected into the stilling basin. The loess and/or glacial till foundation materials overlying the sand seam hold a roof for pipe progression. The pipe progresses unimpeded to the upstream side of the dam, where overlying foundation materials collapse into the pipe and form a stope. The upstream foundation and embankment materials are continuously eroded and do not limit the progression. Intervention fails to stop the pipe from enlarging and the pipe collapses, leading to lowering of the crest and an uncontrolled loss of the pool. See Figure 22 for a plan view of the approximate failure path of BEP through the foundation at the outlet works channel excavation.

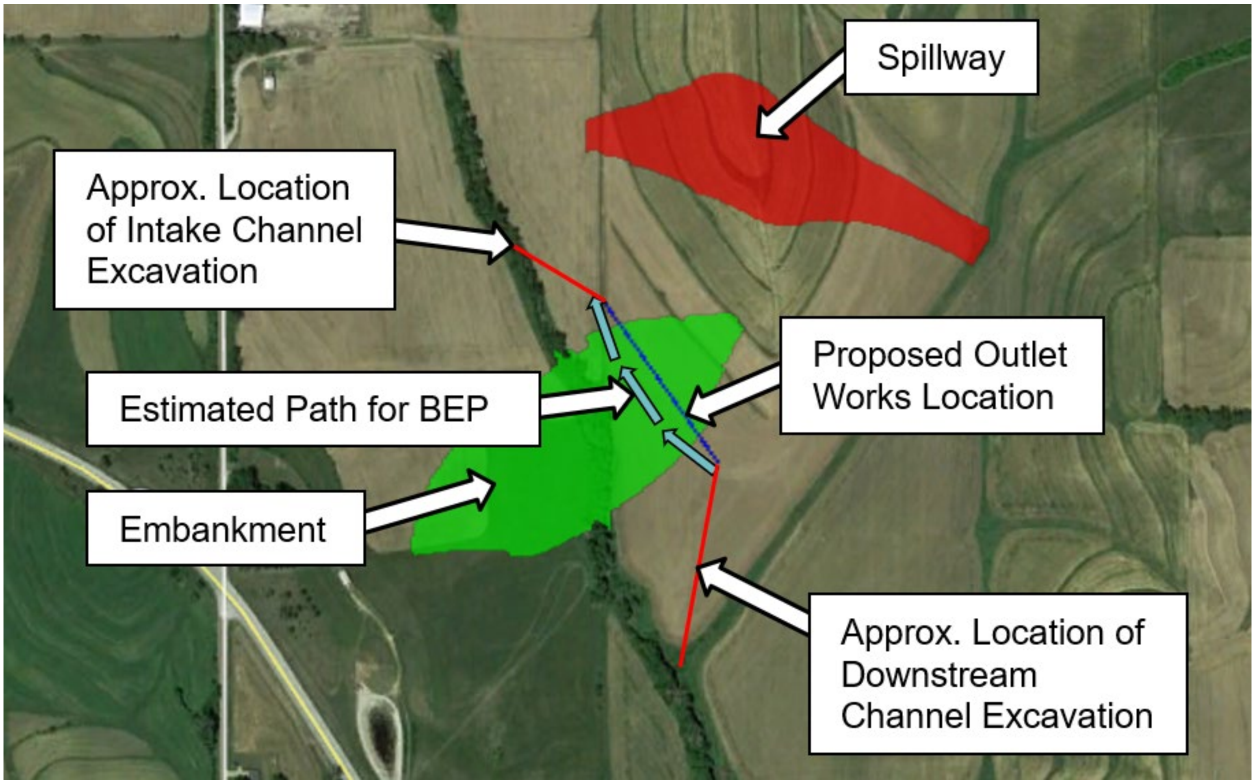


Figure 23. Plan View of the Approximate Failure Path of BEP

4.6.4.1.2 Background

A high gradient and long duration of loading are critical to progress BEP 700 feet from the stilling basin or downstream channel excavation to the upstream channel excavation near the intake. The MHP loading (AEP 1/550,000), equivalent to the MR PMF, was determined to be the critical loading condition because it results in the greatest head differential of 38.1 feet assuming a reservoir elevation of 1204.4 feet NAVD88 and tailwater elevation of 1166.3 feet NAVD88 across the 700-foot-long seepage path. The duration of the MHP event is 41 hours from the starting elevation of 1191.6 feet NAVD88 to the peak pool elevation of 1204.4 feet NAVD, and back down to the box culvert intake at elevation 1154 feet NAVD88.

The head differential (~31.1 feet) and event duration (38 hours from the starting elevation of 1166.5 feet NAVD88) of the TAS pool loading (AEP 1/5,000) is not significantly less than at the MHP loading (AEP 1/550,000), and the TAS pool loading is two orders more frequent than the MHP loading. However, there is no estimated life loss from an embankment breach at the TAS, so the SQRA team selected MHP as the critical pool loading.

The path of BEP through the Kansan glacial drift foundation was discussed by the PA team. Initiation was most likely to occur in the stilling basin due to the deep excavation into the Kansan glacial drift formation at a depth of 1150 feet NAVD88. It was then assumed that a continuous sand seam exists in the glacial drift foundation adjacent to the box culvert which is exposed by the excavation of the diversion channel from the existing streambed to the intake of the outlet works at invert elevation 1154 feet NAVD88. This seepage path is 700 feet in length and results in a global gradient of 0.05 ft/ft.

The Kansan glacial drift is an underlying, erosional surface consisting of till with associated sand and sand-gravel seams or lenses. The till is primarily a sandy clay with variable percentages of limestone and quartzite pebbles interspersed throughout. Most of the till is stiff to very stiff with minor amounts described as medium stiff and hard. Subsurface explorations at DS10 noted very few seams, lenses, or layers of sand and gravel, and they were no thicker than 3 feet.

Overlying the Kansan glacial drift at the left abutment is about 20 to 30 feet of Peorian-Loveland Loess. Loess consists primarily of medium stiff to stiff lean clay. The deposits have liquid limit values between 30 and 52 and Plasticity Index values between 11 and 34. According to Fell et al. (2008), plastic clays with a fines content greater than 50 percent have more than a 90 percent probability of holding a roof. Therefore, the 20 to 30 feet of loess deposits overlying the Kansan glacial drift are expected to hold a roof.

There are seven (7) borings in the left abutment foundation near the proposed alignment for the outlet works that extend in depth to at least elevation 1150 feet NAVD88 including 71-8, 71-13, 72-3, 72-5, 72-8, 72-21, and 72-22. See Figure 23 for the boring location plan from Specific Design Memorandum No. MPC-33 for DS10. Only boring 72-5 encountered a more pervious, clayey sand (SC) material from elevation 1143 to 1146 feet NAVD88, suggesting the sand deposits may not be continuous. Nonetheless, the boring information in the left abutment is limited, so there is still the potential of a localized, continuous pervious seam within the Kansan glacial drift formation from upstream to downstream. Gradations were not obtained of any of the sand and gravel seams, so their susceptibility to BEP is unknown.

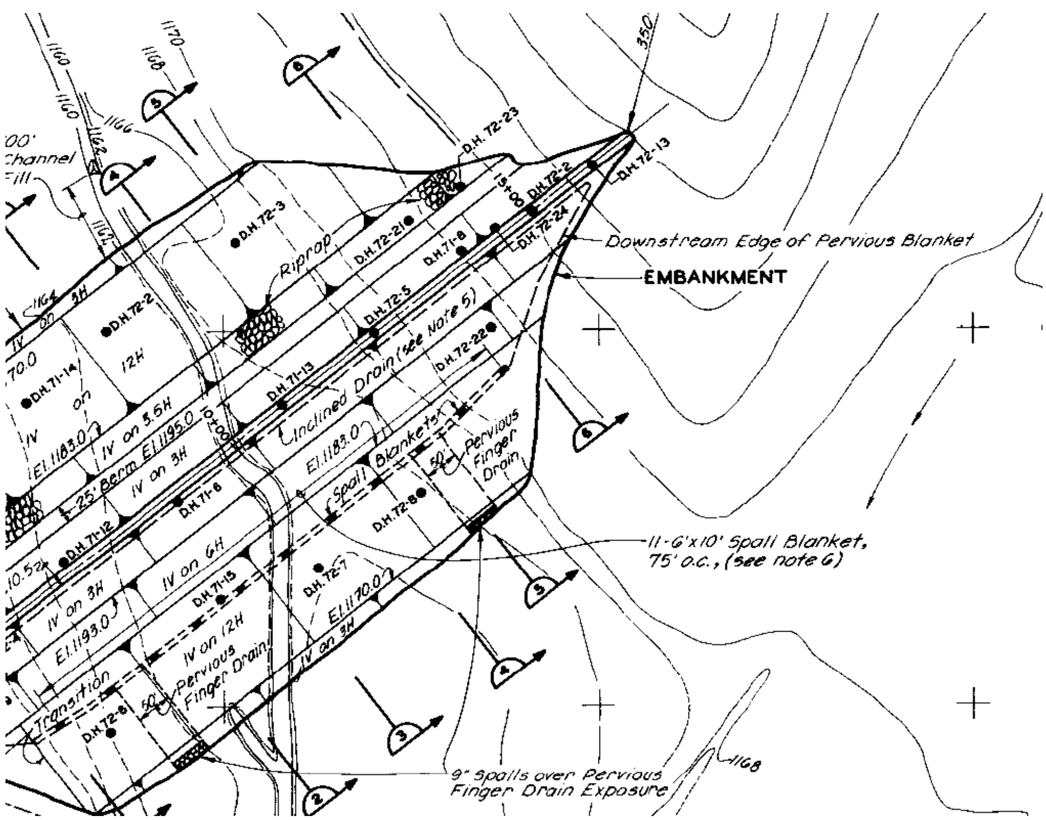


Figure 24. 1975 Boring Location Plan

4.6.4.1.3 Intervention

Although detection of BEP through a low C_u sand seam in the Kansan glacial drift may be possible, intervention is very unlikely. The Papio-Missouri River NRD will be the local sponsor for the project. Although they are responsible and reliable sponsors who fulfill their maintenance duties, closely monitor rain events, and perform surveillance during high pool events. The Papio-Missouri River NRD is the owner or local sponsor of over 100 miles of levee system and several dams within the Papillion Creek and confluence of the Platte and Missouri Rivers. A wide-spread flood event over the Papillion Creek Basin could affect a large percentage of these projects, making it difficult for the Papio-Missouri River NRD to closely inspect each of their projects.

If the local sponsor is able to perform surveillance during high pool events, internal erosion would likely not be detected since it would be under several feet of turbulent tailwater during the MHP loading. Depressions near the stilling basin or in the left abutment slope may indicate partial collapse of a progressing pipe. However, intervention such as construction of a filter or increasing tailwater to reduce the gradient is very unlikely due to the exit location in the outlet works plunge pool. Additionally, the pool cannot be drawn down faster if BEP is observed because the outlet works and spillway are not gated. It is therefore unlikely to stop the progression of BEP.

4.6.4.1.4 Likelihood of Failure

Table 29. Summary of Likelihood Factors for PFM 14.

More Likely Factors	Less Likely Factors
• The overlying Peorian-Loveland Loess foundation material can hold a roof. • BEP in the stilling basin may not be visible and would be under several feet of turbulent tailwater during MHP loading. • The pool cannot be operated to drain faster if BEP was observed in the stilling basin. • One of the borings near the outlet works alignment (72-5) encountered a pervious seam near the elevation of the outlet works excavation. • Channel excavation to connect the existing streambed to the outlet works exposes the Kansan glacial drift foundation material.	• **"Critical" loading at MHP is infrequent (AEP 1/550,000).** • **The global gradient (0.05 ft/ft at MHP) is unlikely to progress BEP.** • **The duration of the MHP event is approximately 41 hours, which makes it unlikely for steady-state gradients to develop and for BEP to progress to failure.** • Only one of the seven borings analyzed for the MHP encountered pervious seams in the Kansan glacial drift.

Note: Key factors that drive the likelihood of failure are shown in **bold**.

Annual Probability of Failure: Between 1E-10 and 1E-09 (with and without intervention)

Rationale: A fine grained, cohesionless sand stratum in the left abutment is potentially exposed by channel excavations to connect the existing streambed to the outlet works. Critical loading (MHP, AEP 1/550,000) causes the pool to rise to a peak elevation of 1204.4 feet NAVD88. The overlying loess and glacial till foundation material is likely to hold a roof. However, the global gradient through the foundation (0.08 ft/ft at MHP) is unlikely to initiate or progress BEP, and the 41 hour duration of the MHP loading from the starting elevation of 1191.6 feet NAVD88 to the peak pool elevation of 1204.4 feet NAVD, and back down to the box culvert intake at elevation 1154 feet NAVD88 is likely insufficient to progress BEP the entire 700 foot seepage path. Due to the significance of the less likely factors and the infrequent loading at MHP (AEP 1/550,000), the PA team determined (with low confidence) that the likelihood of failure is between 1E-10 and 1E-09.

Confidence: Low

Rationale: The major source of uncertainty is the limited information associated with a project in the planning phase including limited boring and testing information, no performance information, and large uncertainty in the hydrologic loading. The continuity and properties of the glacial drift sands are not well characterized in the Specific Design Memorandum No. MPC-33 for DS10. Additional site characterization and lab testing of the sand stratums in the Kansan glacial drift foundation material would give the team greater confidence in the likelihood of a continuous, erodible sand stratum existing from upstream to downstream across the dam. For these reasons, the team has low confidence in the assigned failure likelihood.

4.6.4.1.5 Incremental Life Loss

Average Incremental Life Loss: Between 3 and 30

Rationale: The modeled incremental life loss for a MHP failure ranged from 10 to 14 depending on warning time and exposure. There is no modeled non-breach life loss associated with the MHP loading, so all life loss will be incremental. Although early detection may not be possible due to the turbidity of the tailwater, the team felt that ample warning time would occur prior to breach due to the daily and/or 24-hour surveillance that would occur above the TAS loading once the spillway begins to flow. Visible/detectable distress such as depressions near the stilling basin during the formation of the pipe would likely be observed by the surveillance team, giving the downstream population at risk (13,893 during the day and 4,289 at night) more than two hours of warning prior to breach. Additionally, the duration and flow of the pipe in the glacial till foundation before enough material is eroded to collapse and breach the embankment will likely lower the peak reservoir elevation below the MHP elevation of 1204.4 feet.

Peak outflow for the modeled MHP breach scenario (elevation 1204.4 feet NAVD88) is 61,860 cfs with inundation depths ranging from 6.5 to 13 feet and velocities ranging from 5.5 to 9 feet per second immediately (0 to 3 miles) downstream of the dam. From 3 to 7 miles downstream of the dam, MHP breach modeled inundation depths range from 4 to 12 feet and velocities varied from 2 to 6 feet per second. The primary consequence center is Omaha, NE which is located three river miles downstream of the dam and is largely developed, consisting of a mix of residential and commercial/industrial structures along the creek bank. The inundation area directly downstream of DS10 and adjacent to Thomas Creek is sparsely populated with a mix of residential and industrial buildings.

The inundation depths and velocities are significant and likely to cause loss of life to any of the population at risk (~13,893 during the day and 4,289 at night) remaining in the inundation area. Therefore, adequate warning time is critical to mobilize and evacuate the downstream population at risk. Once the spillway begins flowing at the TAS loading condition, the team was confident that the downstream population would be warned and daily and/or 24-hour surveillance would be required at the dam site. Since visible/detectable distress caused by the formation of the pipe is likely to be observed during surveillance, the downstream population at risk will be warned again to evacuate the inundation area prior to breach. Finally, the duration and flow of the pipe in the glacial till foundation before enough material is eroded to collapse and breach the embankment will likely lower the peak reservoir elevation below the MHP elevation of 1204.4 feet NAVD88, reducing the peak discharge, inundation depths, and velocities. Therefore, the best estimate incremental life loss for this failure mode is between 3 and 30.

Confidence: Low

Rationale: Limited consequence data was available in the planning stage of this project to assess the effects of piping of the left abutment glacial till and eventual collapse of the overlying till, loess and embankment and its effect on the life loss estimate, or to develop and study the consequence mapping products to increase the team's confidence in the life loss estimates.

4.6.4.1.6 Recommendations

- Additional site characterization and lab testing of the sand stratums in the Kansan glacial drift foundation material at the proposed outlet works location.
- Require blanketing/filtering sand seams discovered in the intake and outlet channels excavated for the outlet works.
- Armoring of the intake and outfall to ensure erosion does not uncover a sand seam.

4.6.4.2 PFM 15: Spillway Erosion

4.6.4.2.1 Description

A significant inflow event occurs approaching the peak MHP elevation of 1204.4 feet NAVD88 (AEP 1/550,000) and the pool rises above the design spillway crest elevation of 1191.6 feet NAVD88 (AEP 1/5,000), initiating spillway flow. Velocities along the spillway exceed the allowable shear stress velocities for the in-situ grasses and the vegetation is stripped, initiating headcutting. The spillway flow duration enables headcut progression through the mild channel slope (0.0075 ft/ft) and 200-foot-wide spillway control section. Intervention is unsuccessful due to an inability to access the spillway during MHP flows, and defensive measures (cut off wall or concrete sill) do not exist between the headcut and reservoir. The headcut advances and a connection with the upstream pool is established. Down cutting and mass wasting occurs, allowing breach enlargement and an uncontrolled release of the reservoir.

4.6.4.2.2 Background

The centerline of the proposed earth-cut, grass lined spillway is located about 600 feet north of the left abutment of the dam embankment and founded almost entirely on Peorian-Loveland Loess. The Peorian-Loveland Loess is primarily a lean clay with liquid limit values between 30 and 52. Most of the loess is medium stiff to stiff and can be classified as moderately erodible according to Briaud (2008). Due to the more erodible nature of the loess foundation material, the base of the spillway will be excavated to a minimum depth of 5 feet and backfilled with impervious fill consisting of highly plastic clays with liquid limits in excess of 40 percent to limit the potential for erosion. There is no concrete sill or cutoff structure in the spillway. The spillway has a minimum 200-foot-long and 100-foot-wide earthen crest at design elevation 1191.6 feet NAVD88, and a 0.0075 ft/ft slope downstream of the crest. The spillway channel is 100 feet wide with an approximate length of 1,285 feet from the beginning of the crest to the downstream end of the mild slope at its centerline. The spillway side slopes will be cut at 1V on 3H slopes. A drainage ditch exists at the downstream end of the spillway.

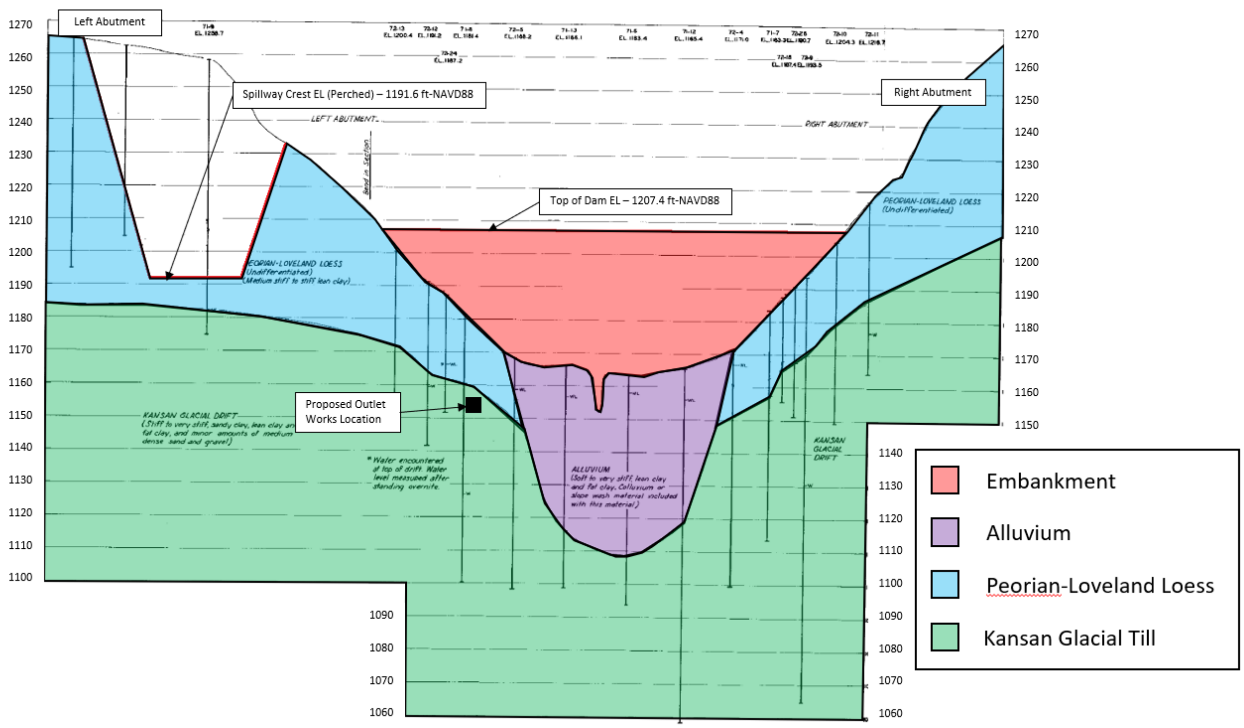


Figure 25. DS10 Geologic Section of the Spillway and Dam Embankment

The loading condition used to evaluate this potential failure mode was the MHP event (AEP 1/550,000), equivalent to the MR PMF. During the MHP event, the duration of flow (greater than 1 foot in depth) is estimated at 8 hours. The team also evaluated whether the performance of the outlet works significantly impacted the probability or duration of the MHP event. Plugging of the outlet works would increase the MHP loading frequency to AEP 1/250,000 and prolong the duration of spillway flow more than one foot in depth to 27 hours. However, the outlet works was designed to be self-cleaning and large enough for equipment to remove potential sedimentation within the span of the conduit, and the Papio-Missouri River NRD continues to be one of the most active and responsive non-federal levee and dam sponsors in the Omaha District portfolio at maintaining their projects. Therefore, the team determined that the coincident plugging of the box culvert would increase the probability of spillway failure by no more than a half order of magnitude.

A HEC-RAS 2D model was created for the DS10 spillway to evaluate the effects of flow concentration during the MHP event. Figure 25 shows the maximum velocity plot produced by HEC-RAS Mapper. Blue indicates lower velocity and dark red indicates higher velocity. Through the length of the spillway, an average velocity of 7 ft/s is observed, with maximum velocities of 12.2 ft/s directly downstream of the spillway crest.

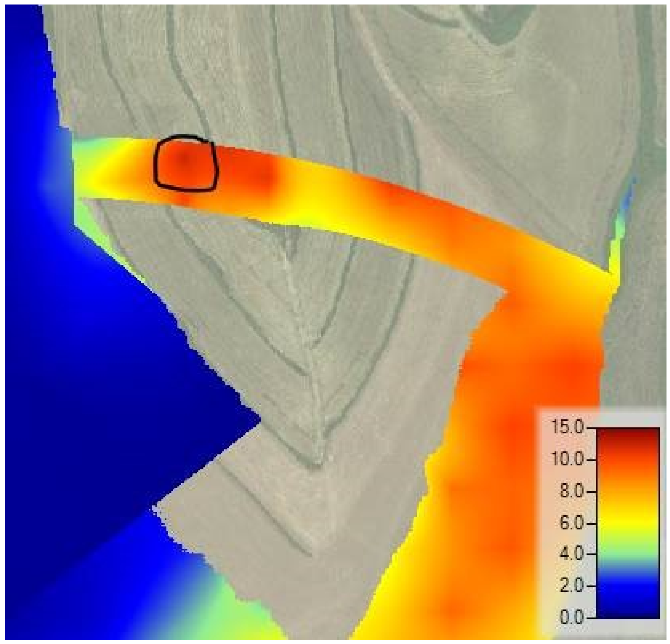


Figure 26. Maximum Velocity Plot Produced by 2D HEC-RAS Spillway Model.

Typical spillway vegetation at the existing Papillion Creek Dams consists of a grass mixture of perennial ryegrass, Primar Slender wheatgrass, Barton Western wheatgrass, and Pathfinder switchgrass. The specified grass mixture is similar to the vegetation types in red font color in Table 30, which include buffalo grass, Kentucky bluegrass, smooth brome, and blue grama. The team determined the velocity below which serious erosion would not occur in erosion resistant soils with Kentucky bluegrass cover and a 0 to 5% slope was 7 ft/sec based on SCS TP-61 and Chow 1959 in Table 30 which shows erosion characteristics of average, uniform stands of vegetation types. The maximum velocity computed in the HEC-RAS 2D model of 12.2 ft/sec is therefore enough to initiate and progress spillway erosion.

Table 30. Permissible Velocities Based on Vegetation Types (Published in TP-61)

Cover	Slope range[2] (Percent)	Permissible velocity (ft/sec)	
		Erosion resistant soils	Easily eroded soils
Bermuda grass	0-5	8	6
	5-10	7	5
	Over 10	6	4

Buffalograss Kentucky bluegrass Smooth brome Blue grama	0-5 5-10 Over 10	7 6 **5**	5 4 3
Grass mixture	0-5 5-10	5 4	4 3
Lespedeza sericea Weeping lovegrass Yellow bluestem Kudzu Alfalfa Crabgrass	[3]0-5	3.5	2.5
Common lespedeza[4] Sudangrass	[5]0-5	3.5	2.5

[1]Use velocities exceeding 5 ft/sec only where good cover and proper maintenance can be obtained.
[2]Do not use on slopes steeper than 10 percent, except for side slopes in a combination channel.
[3]Do not use on slopes steeper than 5 percent, except for side slopes in a combination channel.
[4]Annuals-used on mild slopes or as temporary protection until permanent covers are established.
[5]Use on slopes steeper than 5 percent is not recommended.

4.6.4.2.3 Intervention

Access to the dam embankment and spillway was not fully developed for the General Reevaluation Report (GRR) and will require additional real estate from private landowners. For the purposes of the abbreviated SQRA, it was assumed that the government would be able to purchase real estate for access roads unaffected by tailwater conditions during the MHP for heavy construction equipment access. Access roads for maintenance and emergency situations are critical to address TRG3 and reduce the risk of the project (TRG4).

The Papio-Missouri River NRD is the owner or local sponsor of over 100 miles of levee system and several dams within the Papillion Creek and confluence of the Platte and Missouri Rivers. A wide-spread flood event over the Papillion Creek Basin could affect a large percentage of these projects, making it difficult for the Papio-Missouri River NRD to closely inspect each of their projects to increase the likelihood of ample warning. Equipment for automated pool readings to alert the local sponsor if the pool nears the spillway crest and communication with USACE to provide additional engineering assistance will be critical to intervention. Intervention may be possible at lower discharges but would not be feasible during the MHP event due to the velocity (up to 12.2 ft/sec) and depth (average of 9 feet) of spillway flows.

4.6.4.2.4 Likelihood of Failure

Table 31. Summary of Likelihood Factors for PFM 15

More Likely Factors	Less Likely Factors
• **Maximum flow velocity of 12.2 ft/s is high enough to strip the vegetal cover and initiate and progress headcutting in the spillway.** • An existing drainage ditch at the end of the spillway will concentrate flow.	• **MR PMF loading is infrequent (AEP 1/550,000).** • The moderately erodible loess foundation will be over-excavated and replaced with 5 feet of high plasticity clay with low erodibility.

More Likely Factors	Less Likely Factors
• **There is no concrete control sill or cutoff structure to prevent headcutting.** • Plugging of the outlet works would increase the MHP loading frequency to AEP 1/250,000 and prolong the duration of spillway flow over one foot in depth to 27 hours. However, the team determined that plugging of the box culvert would be unlikely.	• **The duration of spillway flow over one foot in depth is relatively short (8 hours total during the MHP event).** • **Centerline length of spillway is 1,285 feet, which would require an erosion rate of approximately 161 feet/hour to progress to failure.**

Note: Key factors are shown in **bold**.

Annual Probability of Failure: Between 1E-09 and 1E-08 (with and without intervention)

Rationale: Flow velocities during the MR PMF event (AEP 1/550,000) as high as 12.2 ft/s are sufficient to strip vegetation and initiate and progress headcutting of the 5 foot layer of erosion resistant, high plasticity clay and underlying moderately erodible loess. The total duration of flow of more than one foot in depth is approximately 8 hours. There is no upstream control sill or cutoff structure to prevent a full breach. However, the short flow duration, 1,285 foot failure path length, and proactive removal of the more erodible exposed loess material during construction makes it unlikely that headcutting and/or down-cutting would be sufficient to progress the entire spillway length to breach the crest. Due to the significance of the less likely factors, the PA team determined (with moderate confidence) that the likelihood of failure is between 1E-08 and 1E-07.

Confidence: Low

Rationale: The major source of uncertainty is the limited information associated with a project in the planning phase including limited site characterization and testing information of the spillway foundation materials, no performance information, and large uncertainty in the hydrologic loading. Additionally, there is considerable uncertainty in the duration of flow required to breach the spillway; the processes of erosion are unpredictable and can vary significantly depending on material properties, flow concentration, and spatial variation of velocities.

4.6.4.2.5 Incremental Life Loss

Incremental Life Loss: Between 0.1 and 1

Rationale: The modeled incremental life loss for a MHP spillway failure was 0 for all warning times and exposure. The spillway breach model discussed in Section 4.5 Consequences calculated an incremental discharge of 20 cfs, which correlates to an incremental population at risk of X during the day and X at night. Additionally, the team was confident that daily and/or 24-hour surveillance would be required at the dam site and the downstream population would be warned once the spillway begins flowing at the TAS loading condition. With constant surveillance at the dam site during the MHP loading condition, it is very likely that the downstream population will be warned a second time when there is visible erosion of the spillway to ensure that the PAR is mobilized and evacuated from the inundation area. Therefore, the best estimate incremental life loss for this failure mode is between 0.1 and 1.

Confidence: Low

Rationale: Limited consequence data was available in the planning stage of this project and several assumptions were made in the model for MHP spillway failure as discussed in Section 4.5 Consequences.

4.6.4.2.6 Recommendations

- Fill the drainage ditch downstream of the spillway that has the potential to concentrate flows and initiate headcutting.
- The team considered whether a control sill or cutoff structure is necessary to reduce the risk of a headcut advancing through the crest of the spillway; however, the low probability of the failure mode made the cost not necessary.
- Prioritize routine maintenance of the trash rack on the intake of the outlet works in the O&M Manual to ensure the design capacity of the outlet works is maintained to prevent increased frequency and duration of spillway flow.
- Any proposed recreation, utility, or other features submitted through the 408 process within the spillway will be thoroughly reviewed prior to approval. The inclusion of such features is likely to increase the erosion potential of the spillway due to increased turbulence and localized velocities caused by knickpoints.

5 PAPILLION CREEK DAM SITE 19

5.1 FINDINGS AND RECOMMENDATIONS

The abbreviated SQRA performed by the Omaha District did not identify any potential failure modes that would prevent Papillion Creek Dam Site 19 (DS19) from meeting the tolerable risk and essential USACE guidelines. This abbreviated SQRA used the optimized DS19 design to be included in the final recommended plan of the GRR which incorporates current USACE design criteria as discussed in Appendix C – Geotechnical Analysis. See Section 5.2 Background for additional details of the DS19 design to be included in the final recommended plan. It should be noted that significant design changes to the spillway, outlet works, or dam embankment that affect the hydrologic loading will require an updated or new SQRA to ensure the modifications do not significantly increase the risks of the project.

One of the risk drivers carried forward during the abbreviated SQRA included erosion of the unlined, earth-cut emergency spillway (PFM 15) due to high flow velocities (up to 15 ft/sec) and no control sill or cutoff structure to prevent headcutting. However, a significant length (1,072 feet) would have to erode within a relatively short duration (5 hours with more than one foot of flow depth) to breach the spillway crest, and the resulting breach discharge has minimal incremental inundation and loss of life consequences compared to design spillway flows (non-breach).

The other risk driver PFM carried forward during the abbreviated SQRA included Backwards Erosion Piping (BEP) of the Red Cloud Formation through the left abutment (PFM 14). Excavation for the outlet works stilling basin has the potential to remove enough of the Kansan glacial till to cause blowout of the confining layer or exposure of a foundation layer composed of poorly graded sands or silty sands susceptible to BEP and continuous from upstream to downstream. However, it is unlikely that there is an upstream exposure of the Red Cloud Formation, limiting the flow required for progression of BEP, and the pool duration (2 days and 14 hours above the NHP) and global gradients through the foundation (0.09 ft/ft at TAS) are too low compared to what is considered necessary to initiate and progress BEP to breach.

5.1.1 SOCIETAL INCREMENTAL LIFE SAFETY RISK

Twenty-five (25) potential failure modes (PFM) were identified prior to the abbreviated SQRA for consideration. Twenty-three (23) were not developed in detail as they were not considered to be “risk-drivers” for the project. Non risk-driver PFMs are discussed in Section 5.6.2. The following risk-driver PFMs were evaluated by the PA team:

- PFM 14: BEP of the Red Cloud Formation through the left abutment
- PFM 15: Spillway Erosion

A risk matrix has been established to portray the incremental life safety risk (due to failure or breach) associated with the identified risk-driving PFMs, with annual probability of failure (APF) on the vertical axis and the associated incremental life loss on the horizontal axis, using cell divisions corresponding to order of magnitude ranges of APF and incremental life loss. The matrix is similar to the f-N diagram used to portray incremental life safety risk estimated from quantitative risk analysis. The societal incremental life safety risk matrix is shown in Figure 26.

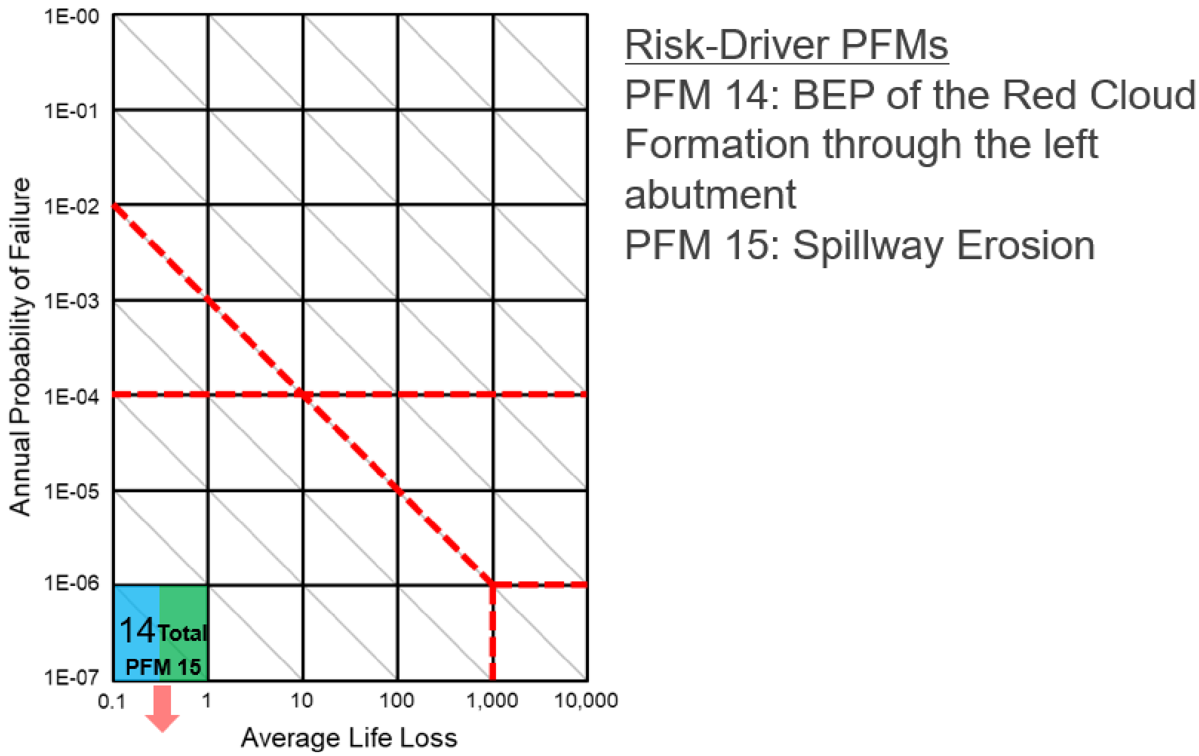


Figure 27. DS19 Societal Incremental Life Safety Risk Matrix

An approximate numerical estimate of APF and average incremental life loss were obtained for each PFM using the centroid (geometric mean) of the box (order-of-magnitude estimate). The total APF was calculated by summing the APFs for all of the primary risk-driver PFMs assuming they are mutually exclusive. The total average annual incremental life loss (AALL) was calculated by summing the product of the APF and average incremental life loss for all of the primary risk-driver PFMs. The weighted average incremental life loss was then calculated by dividing by the total AALL by the total APF.

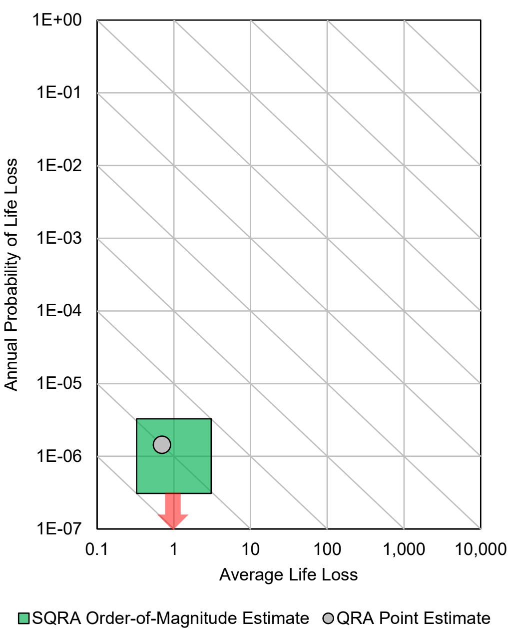
The estimated total APF is between 1E-07 and 1E-06 failures per year, and the estimated weighted average incremental life loss is between 0.1 and 1 lives per failure. Therefore, the best estimate of the average annual incremental life loss is 1E-07 lives per year. The total risk of the project is below the individual and societal life risk lines and therefore meets TRG 1. Additional details about the risk-driving PFMs and associated incremental life loss are provided in Section 5.5.5.

5.1.2 NON-BREACH LIFE SAFETY RISK

Non-breach risk occurs when the flood capacity of the dam is exceeded. At this point, the dam transitions from managing the flood to passing the flood. For dams, the transition occurs when the spillway activates at the top of active storage (TAS) elevation. This elevation corresponds to the annual probability of non-breach inundation but may not result in life loss. The top of active storage at DS19 is at the proposed spillway crest elevation of 1177.5 feet NAVD88 and has an estimated ACE of 1/1,600.

The annual chance exceedance (ACE) when life loss begins to occur ($ACE_{N>0}$) was not determined as part of the abbreviated SQRA due to time and funding restraints. The consequence modeling indicates that non-breach life loss is not expected for loadings up to the modeled MHP (ACE 1/700,000) elevation of 1184.7 feet NAVD88 or a total peak outflow of 25,580 cfs from the dam. Even though portions of South Papillion Creek overflow at the estimated non-damaging discharge of 12,890 cfs, it is assumed that when the pool nears the spillway crest, a warning will be communicated to the downstream areas to allow the population at risk (3,254 during the day and 1,707 at night) to mobilize and evacuate before the discharge during the MHP event peak reaches the impacted population. Additionally, the West Papillion, Big Papillion, and Papillion Creek Levee Systems in Omaha, NE, extending from approximately 8 river miles downstream of the dam to the mouth of Papillion Creek at the Missouri River, are not overtopped by the total peak outflow from the dam for the MHP non-breach event. Therefore, there is no estimated life loss at the MHP non-breach event and the annual probability of inundation with non-breach life loss was estimated to be less than 1.43E-6 floods per year.

The results are plotted on a separate non-breach life safety risk matrix, similar to the societal incremental life safety risk matrix previously described. However, the vertical axis is labeled "annual probability of life loss," and no tolerable risk limit lines are shown since they are not applicable. The non-breach life safety risk matrix assuming life loss for pool loading conditions above the MHP non-breach event is shown in Figure 8.



Flooding Threshold
Top of Active Storage: 1/1,600 AEP
Non-damaging outflow: 12,890 cfs

Life Loss Threshold
No loss of life for non breach up to the modeled MHP elevation

Top of Dam
AEP = 1/3,500,000 (>100% PMF)

PMF
Freeboard: 3 feet
Outflow: 25,580 cfs

Downstream FRM Structures
No dams/levees are overtopped.

Figure 1.2 Non-Breach Life Safety Risk Matrix

The lowest dam crest elevation has an estimated 1/3,500,000 AEP. At this elevation, the dam is capable of storing 100 percent of the estimated Probable Maximum Flood (PMF) inflow hydrograph. The estimated PMF event results in 3 feet of freeboard, not considering wind/wave effects. The TAS elevation has an estimated 1/1,600 AEP. The primary consequence center is Omaha, NE which is located approximately two river miles downstream of the dam and is largely developed, consisting of a mix of residential and commercial/industrial structures along the creek bank. The inundation area directly downstream of DS19 and adjacent to South Papillion Creek is sparsely populated with only a few residential and industrial structures. There are no downstream dams. The West Papillion, Big Papillion, and Papillion Creek Levee Systems in Omaha, NE, extend from approximately 8 river miles downstream of the dam to the mouth of Papillion Creek at the Missouri River, and are not overtopped by the MHP non-breach loading event. However, the population adjacent to the South and West Papillion Creeks within 8 river miles downstream of the dam are not protected by flood risk management (FRM) structures, such as levees, and the estimated downstream non-damaging discharge is 12,890 cfs. The total peak outflow from the dam for the PMF event is 25,580 cfs, The SQRA team has moderate confidence that there would be ample warning time to mobilize and evacuate the population at risk, especially due to the more sparse population directly downstream of DS19 on South Papillion Creek. However, due to the limited team discussion and consequence information for the abbreviated SQRA, there is low confidence in the non-breach life loss results.

5.1.3 MAJOR FINDINGS AND UNDERSTANDINGS FROM THE RISK ASSESSMENT

The following major findings and understandings were developed by the abbreviated SQRA team:

- The total risk of DS19 is below individual and societal life risk lines and therefore meets TRG 1.
- DS19 is located approximately 3.5 miles west of Papillion Creek Dam Site 20 (DS20). Therefore, it is assumed that the geologic site conditions at DS19 and DS20 are similar, and DS20 has performed adequately since construction.
- The red-cloud formation at DS19 is more extensive and continuous than at DS20 and needs to be further investigated/characterized during preconstruction engineering and design to reduce the risk of BEP through this formation.
- The uncontrolled, unlined, earth-cut emergency spillways at the existing Papillion Creek Dams have never been tested, so there is considerable uncertainty in the duration of flow required to breach the spillway.
- The DS19 spillway will flow more frequently than the other Papillion Creek Dams in order to prevent upstream inundation of Highway 6, potentially increasing the non-breach risk of the dam.
- The team considered whether a control sill or cutoff structure is necessary to reduce the risk of a headcut advancing through the crest of the spillway during spillway flow events; however, the team determined that the low probability and life loss of the failure mode made the cost not necessary.

5.1.4 RECOMMENDATIONS

The following recommendations address deficiencies identified by the abbreviated SQRA team in the preliminary design for DS19 in the GRR final recommended plan. The goal is to incorporate the recommendations in the preconstruction engineering and design if the project enters the implementation phase to reduce the risk of the project to the downstream population.

- Additional boring information, site characterization and lab testing of the Red Cloud Formation, especially at the location of the stilling basin, is required to determine its susceptibility to BEP and to develop critical gradients during PED for blowout and initiation and progression of BEP. The following information will determine if relief wells are required to reduce the foundation pressures at the stilling basin and downstream channel.
- Require blanketing or filtering of sand seams discovered in the excavation for the intake and outlet channels for the outlet works.
- Armor the stilling basin to ensure erosion does not uncover the Red Cloud Formation. Design the armoring as a staged filter to retain Red Cloud Formation material.
- Install additional piezometers at the stilling basin and a line of piezometers across the embankment at the left abutment and tipped in the Red Cloud Formation to monitor gradients.
- Prioritize routine maintenance of the trash rack on the intake of the outlet works in the O&M Manual to ensure the design capacity of the outlet works is maintained to prevent increased frequency and duration of spillway flow.
- Construct upstream impervious blankets at the abutments to reduce potential seepage through the loess.
- Any proposed recreation, utility, or other features submitted through the 408 process within the spillway will be thoroughly reviewed prior to approval. The inclusion of such features is likely to increase the erosion potential of the spillway due to increased turbulence and localized velocities caused by knickpoints.

5.2 BACKGROUND

5.2.1 LOCATION

Papillion Creek Dam Site 19 (DS19) is proposed to be constructed on the South Papillion Creek immediately upstream from 192nd street and ¼ miles south of Giles Road in Sarpy County NE. The contributing drainage area to the site is approximately 4.3 square miles. Dam Site 19 is one of twenty-one (21) dams and reservoirs authorized by the Flood Control Act of 1968 (Public Law 90-483) in accordance with the recommendations of the Chief of Engineers in House Document No 349, 90th Congress, 2nd session, and the Energy and Water Development Appropriation Act of 1981. The dams were initially authorized for flood control, recreation, fish and wildlife enhancement, and water quality. Due to significant changes in policy following project authorization, all proposed dams were reevaluated and at the time only four dams (Dam Sites 11, 16, 18 and 20) were determined to be either economically feasible and/or met the benefit/cost requirements set forth in Section 9 of Public Law 89-72.

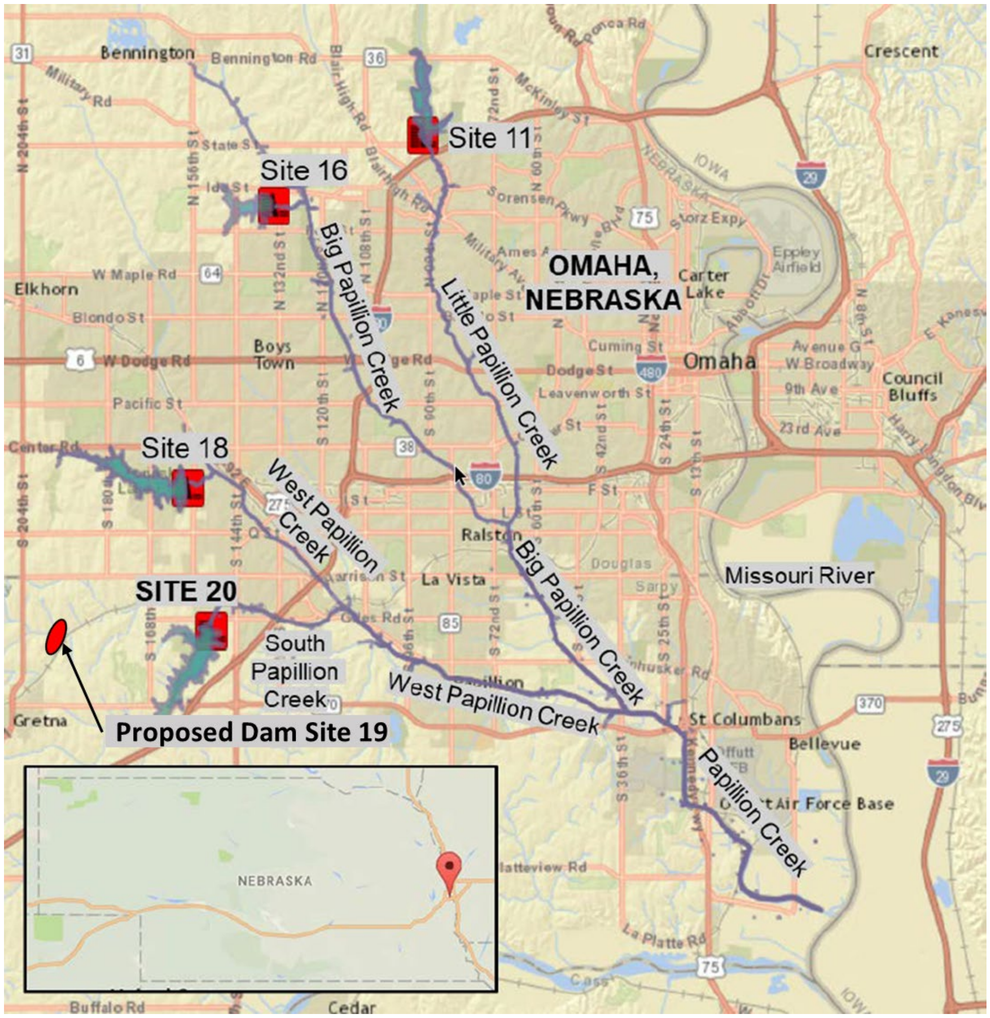


Figure 28. Map of Existing Papillion Creek Dam Sites in Relation to Dam Site 19

5.2.2 GENERAL PROJECT DESCRIPTION

Proposed Papillion Creek Dam Site 19 will be a wet dam project consisting of a rolled earth filled dam embankment; an outlet works consisting of a low-level intake conduit, an intake tower with openings at the multi-purpose pool elevation of 1164 feet NAVD88, and a 72" diameter RCP culvert; and an uncontrolled, earth-cut spillway. See Figure 27 for a map of DS19 in relation to the existing federal Papillion Creek Dam Sites, Figure 28 for a plan view of the project features, Table 32 for storage allocations for DS19, and Table 33 for pertinent project data.

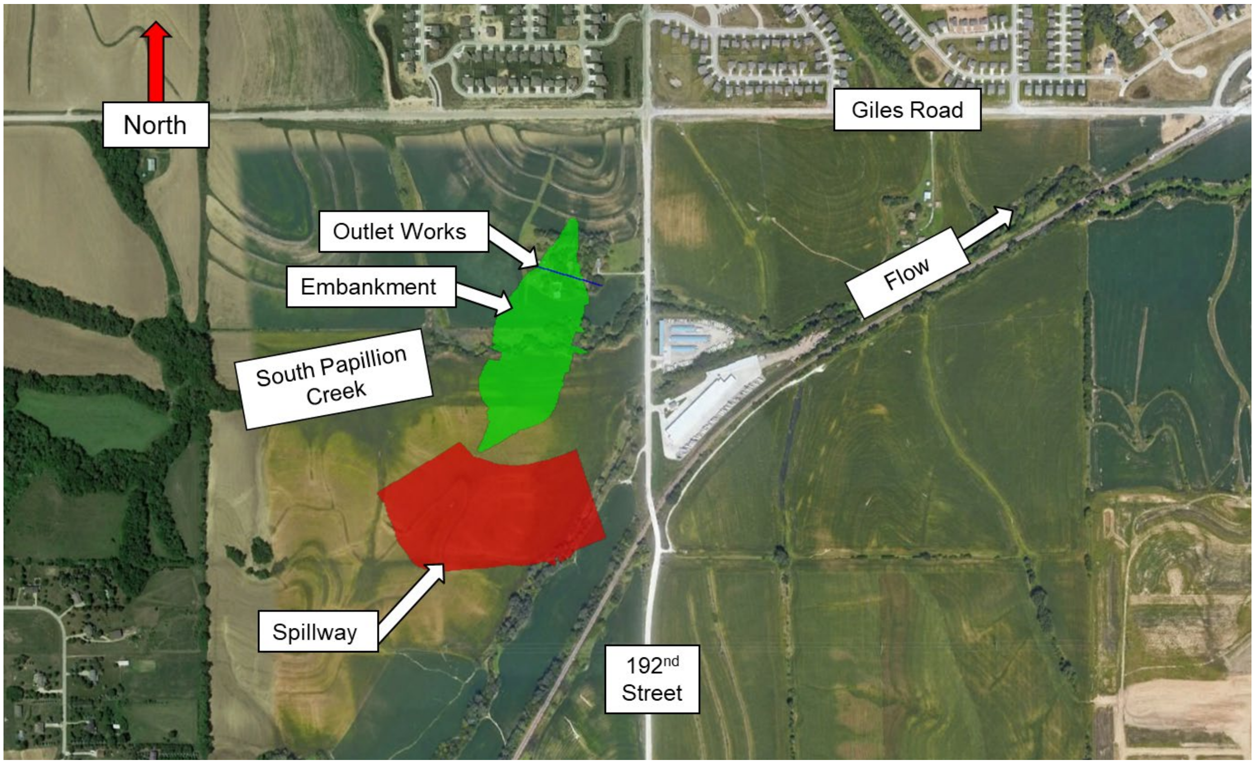


Figure 29. Plan View of Project Features

5.2.2.1 General Project Pertinent Data

Table 32. Papillion Creek Dam Site 19 Storage Allocations

Type of Storage	Elevation (ft NAVD88)	Storage Volume (acre-feet)
Top of Dam	1,187.7	4,529
Reasonable High (RH) PMF	1,185.4	3,917
Most Reasonable (MR) PMF	1,184.7	3,742
Top of Flood Control Pool/Spillway Crest	1,177.5	2,228
Top of Multipurpose Pool	1,164.0	571

Table 33. Papillion Creek Dam Site 19 Pertinent Data

Embankment	
Design Crest Elevation	1,187.7 Feet NAVD88 (excluding overbuild)
Design Freeboard	3 Feet
Crest Length	1,512 Feet
Crest Width	25 Feet
Height above Flood Plain	40.4 Feet
Height above Streambed	60.7 Feet
Type of Fill	Homogeneous Rolled Earth
Estimated Volume of Fill	322,250 CY

Slope Protection	Grass Cover
Wave Protection	Upstream Riprap Protection
Seepage Control	Internal Pervious Fill Drain
Emergency Spillway	
Type and Location	Ungated, Grass-Lined Earthcut Channel in Right Abutment
Design Discharge Capacity (at Most Reasonable PMF)	25,580 cfs (at Elevation 1,184.7 Feet NAVD88)
Design Crest Elevation	1,177.5 Feet NAVD88
Bottom Width	550 Feet
Length	1,072 Feet at Centerline
Side Slopes	1V on 3H
Excavation	329,000 CY
Outlet Works	
Low Level Intake	
Design Invert Elevation at Inlet	1,146.0 Feet NAVD88
Design Invert Elevation at Outlet	1,141.8 Feet NAVD88
Conduit Diameter and Type	30 Inch RCP
Low-Level Conduit Length	80 feet
Intake Structure	
Type	Reinforced concrete tower
Multipurpose Upper Level, Ungated Intakes	
Design Invert Elevation	1,164.0 Feet NAVD88
Low Level, Gated Intake	
Type	Manually Operated Steel Sluice Gate
Design Invert Elevation	1,141.8 Feet NAVD88
Size of Inlet	72 Inch Diameter
No. - Size of Low-Level Gates	1 - 30 Inch (W) by 30 Inch (H)
Conduit	
Diameter and Type	72 Inch RCP
Length	400 Feet
Design Invert Elevation at Inlet	1,139.5 Feet NAVD88
Design Invert Elevation at Outlet	1,130.0 Feet NAVD88
Design Discharge Capacity	1,066 cfs (at Most Reasonable PMF)

5.2.2.2 Embankment

The embankment will be a homogeneous rolled earth filled structure constructed of primarily lean clay (CL) impervious fill from spillway and upstream pool area borrow excavations. The 1,512 feet long, 25-foot-wide crest is designed at elevation 1,187.7 feet NAVD88 and, as determined by the preliminary settlement analysis completed by HDR, will include two foot of overbuild to account for post-construction settlement in the valley founded on alluvium. At its maximum section, the embankment is about 40.4 feet above the valley floor and 60.7 feet above the streambed. A typical embankment section is shown on Figure 29.

The upstream slope of the embankment consists of a 1V on 6H slope from the ground surface to elevation 1,159.5 feet NAVD88, a 1V on 3H slope from elevation 1,159.5 feet to 1,169 feet, a 1V on 20H bench for upstream maintenance access from elevation 1,169 feet to 1,170 feet, and a 1V on 3.5H slope from elevation 1,170 feet to the design crest elevation of 1,187.7 feet. Since DS19 is a wet dam with a permanent pool, it was determined that riprap protection on the upstream face was necessary to protect the embankment from wind-wave action. The riprap protection will extend from the upstream toe of the dam to elevation 1,169 feet, 5 feet above the multipurpose pool elevation to account for wind-wave action and for maintenance purposes. The typical embankment section shown on Figure 29 has not yet been updated to extend the riprap revetment to elevation 1,169 feet. The downstream slope of the embankment consists of a 1V on 3.5H slope from the crest to elevation 1,164 feet and a 1V on 6H slope from 1,164 feet to the ground surface. All embankment slopes outside of the riprap protection are protected by grass cover.

A 6-foot-deep by 10-foot wide-inspection trench with 1V on 2H side slopes will be excavated along nearly the entire length of the embankment. The center of the trench is located along the embankment centerline and will extends up the abutments to the spillway crest elevation of 1,177.5 feet NAVD88. The purpose of the inspection trench is to break the continuity of the surface soil structure by replacing it with compacted impervious fill. It will also be used to identify unforeseen soft areas near the ground surface that will require procedural changes during construction. The excavated fill will be recompacted into the inspection trench if it meets fill requirements.

Seepage through the embankment will be controlled by an internal pervious fill drain designed to prevent saturation of the downstream slope under all “normal” seepage conditions. For unusual seepage conditions such as embankment cracking, the drain is intended to reduce seepage pressures while the pool is drawn down. The drain will be composed of imported, free-draining (pervious) fill. The top of the drain will be constructed to the most reasonable PMF elevation of 1,184.7 feet NAVD88 and begin at the centerline of the dam. The drain consists of a 6-foot-thick continuous 1V on 1H upstream inclined pervious fill chimney and a continuous 3-foot-thick horizontal pervious fill blanket that extends over the flood plain for an approximate distance of 155 feet. In addition, the base of the former/original channel will be lined with 3 feet of pervious fill to provide a controlled outlet for seepage.

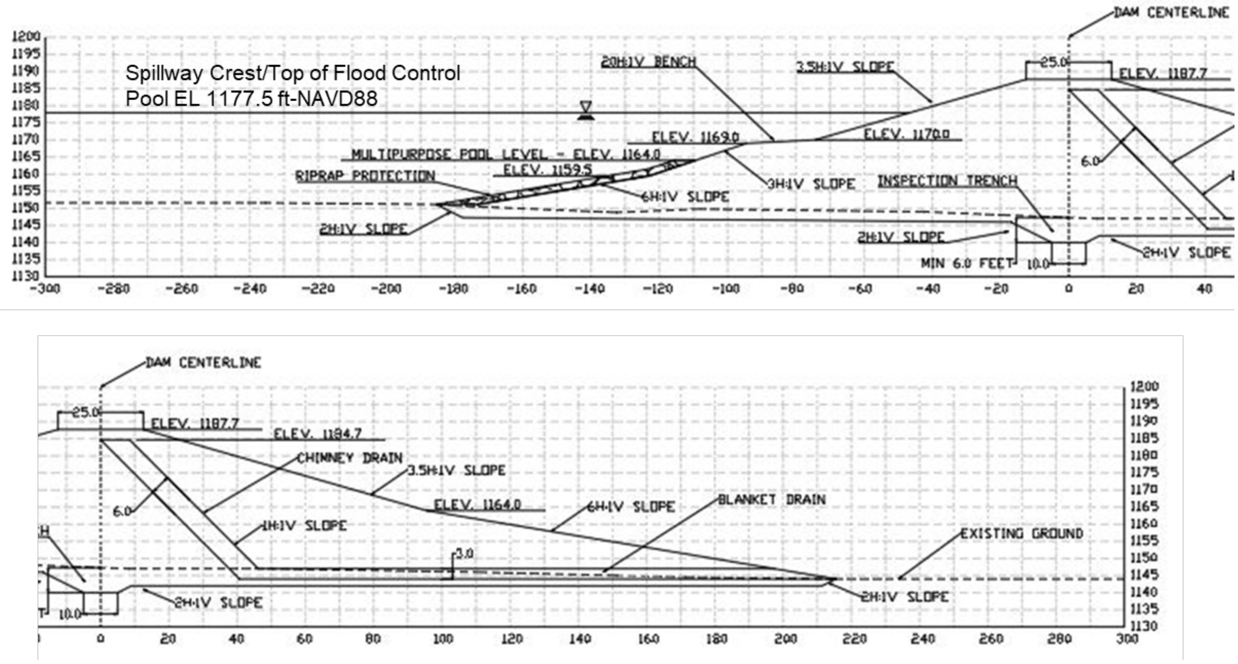


Figure 30. DS19 Typical Embankment Cross Section

5.2.2.3 Outlet Works

Grading and design of the intake tower, low-level intake, and the channel excavation to connect the existing streambed to the outlet works was not fully developed for the General Reevaluation Report (GRR). Therefore, it was assumed for the abbreviated SQRA that the outlet works design for DS19 would be very similar to the 2018 preliminary design completed by HDR except for replacing the 48” diameter RCP outlet in HDR’s design with a 72” diameter RCP to avoid upstream impacts to Highway 6.

5.2.2.3.1 Foundation

The intake tower, 72” diameter outlet conduit, and stilling basin will be founded entirely on glacial drift in the left abutment, represented primarily by very stiff, pebbly, sandy clay. The intake, conduit, and stilling basin will be founded approximately 20 to 25 feet below ground surface with a cut and cover operation. Negligible settlement (~ 0.2 feet) of the intakes, conduits, and stilling basins founded on glacial drift material at existing Papillion Creek Dam Sites 11, 16, 18, and 20 has been recorded, so negligible settlement of the outlet works founded on similar foundation material is expected at DS19.

5.2.2.3.2 Low-Level Intake

The low-level inlet design is based on the 2018 preliminary design completed by HDR and consists of an 80 foot long, 30 inch diameter reinforced concrete pipe carrying low-level inflows from the intake (design elevation 1,146 feet NAVD88) to the intake structure (design elevation 1,141.8 feet NAVD88). Grading will be required to divert flow from the existing South Papillion Creek channel to the intake for embankment construction. Low-level flows will be controlled with a 30-inch by 30-inch manually operated steel sluice gate with adjustable wedges to reduce leakage. The gate allows for lowering of the reservoir for inspections, maintenance, shore-line repairs, and fish population control. The gate may also be used to release water for downstream needs.

5.2.2.3.3 Intake Structure

The reinforced concrete intake structure will have uncontrolled inlets with trash racks and screens at the multipurpose pool elevation of 1,164 feet NAVD88.

5.2.2.3.4 Conduit

The reinforced concrete pipe (RCP) outlet conduit from the intake structure to the stilling basin will be 400-foot-long, 72-inch in diameter, and founded on a concrete cradle. The conduit joints will be steel-ring-type with neoprene O-ring gaskets. As an added precaution, the conduit will be cambered slightly under the embankment to keep the joints in compression. Seepage along the outlet works under "normal" seepage conditions or due to cracks or flaws adjacent to the outlet works will be collected by a 10-foot-long and 3-foot-wide pervious backfill drain near the outfall of the conduit.

5.2.2.3.5 Stilling Basin and Outlet Channel

For the abbreviated SQRA and cost estimating purposes, an impact stilling basin structure at the conduit outfall and a basin protected with riprap revetment similar to HDR's 2018 design were assumed.

5.2.2.4 Spillway

The centerline of the earth-cut, grass lined spillway will be located about 350 feet southwest of the right abutment of the dam embankment. It will be approximately 1,072 feet long from the start of the crest to the end of the spillway at its centerline and have a minimum 200-foot-long and 550-foot-wide crest at design elevation 1177.5 feet NAVD88. Due to the low estimated probability of spillway failure and associated life loss discussed in Section 5.6.4.2 Abbreviated Semi-Quantitative Risk Assessment, the spillway crest design does not currently include additional erosion protection such as a concrete control sill or cutoff wall. Approximately half of the channel base is founded in more erosive loess material and will therefore be excavated to a minimum depth of 5 feet and backfilled with impervious fill consisting of highly plastic clays with liquid limits in excess of 40 percent to limit the potential for erosion. Figure 30 shows a typical section of the spillway with the over-excavation of exposed loess.

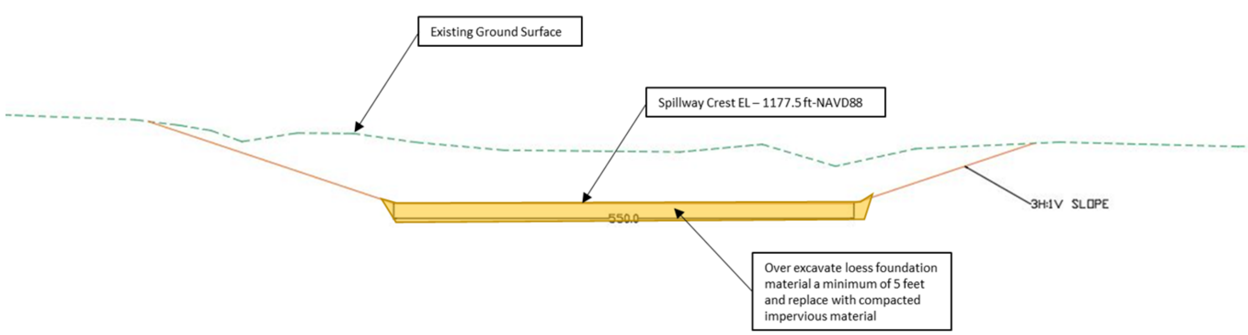


Figure 31. DS19 Typical Spillway Cross-Section

5.2.3 SITE CHARACTERIZATION

5.2.3.1 Geologic Setting

Papillion Creek Dam Site 19 lies within the Dissected Till Plains section of the Central Lowland Physiographic Province. The major topographic feature of the area is the dissected loess-mantled upland, characterized by gently rolling to rolling hills with well-developed drainages. Surface geology of this tributary valley, with the exception of recent alluvium in the valley, is Pleistocene in age and is entirely of eolian (wind-blown) origin. These eolian deposits are represented by the Peorian Loess and the underlying Loveland Loess. For the purpose of this report, they are treated as one unit and are designated the Peorian-Loveland Loess (Undifferentiated). The older Pleistocene deposits of the Kansan and Nebraskan glacial stages underlie the entire basin but are not exposed at the surface. In general, most of the glacial material encountered is Kansan in age; however, this does not preclude the possibility that some Nebraskan age deposits may be encountered at the lower elevations. As the Nebraskan age deposits are very similar to those of the Kansan, with no particular differences in the foundation properties from an engineering viewpoint, all glacial deposits will be considered Kansan for simplicity. Cretaceous sandstones or Pennsylvanian limestones and shales form the bedrock surface underlying most of the Papillion Creek drainage basin; however, they occur at a depth that was not a factor in design or encountered during construction of the project.

The surficial geology of the Papillion Creek Watershed consists of eolian (wind-blown) deposits primarily of Peoria and Loveland loess. The loess formed in dune shaped hills situated between the Elkhorn and Missouri Rivers. The Peoria loess typically consists of silty lean clays that are stiff when dry but become softer with increasing moisture content. The Peoria loess can exhibit low unit weight and moisture content and may be susceptible to collapse upon wetting or loading. The Peoria loess can also be relatively pervious depending on its silt and sand content. The underlying Loveland loess typically consists of lean clays and generally exhibits higher unit weights and shear strengths than the Peoria and is less susceptible to collapse upon wetting or loading. For the purpose of this report, they are treated as one unit and are designated the Peorian-Loveland Loess (Undifferentiated).

The loess overlies glacial deposits of the Kansan till. The till consists of lean to fat clay mixed with occasional sand, gravel, and cobbles. The glacial deposits are generally deep but can be at or near the surface at lower elevations on steep slopes. At DS19, a glacio-fluvial sand-gravel deposit called the Red Cloud Formation was encountered. The Red Cloud Formation lies near the interface of the Kansan till and the Nebraskan till, and consists of loose to very dense, moist to wet, poorly graded sand (SP), silty sand (SM), and poorly graded sand with silt (SPM-SM). It is thought that the Red Cloud sand and gravel was deposited as outwash from streams flowing from southwestward-advancing glaciers.

Nebraskan till was logged below the Red Cloud Formation in most borings where the Red Cloud Formation was encountered. In general, most of the glacial material encountered is Kansan in age; however, this does not preclude the possibility that some Nebraskan age deposits may be encountered at the lower elevations. As the Nebraskan age deposits are very similar to those of the Kansan, with no particular differences in the foundation properties from an engineering viewpoint, all glacial deposits will be considered Kansan for simplicity.

Alluvial and colluvial deposits are present within the floodplain and drainageways. These soils were formed by erosion of the adjoining loess-mantled hills. The upper several feet of alluvium are usually stiffer due to the effects of desiccation (drying). Alluvial deposits are generally present along creeks and in major drainage ways and are formed by deposition in flowing water. Colluvial soils are usually located at the base of steep slopes and in upland draws and are formed by local creep and sloughing.

Cretaceous formations consisting of Dakota sandstone and other materials were typically formed on top of Pennsylvanian limestone and shale. These formations are understood to comprise the bedrock unit below the glacial and alluvial deposits. The depth to this bedrock is normally greater than 100 feet below grade and is rarely encountered in construction within the Papillion Creek Watershed.

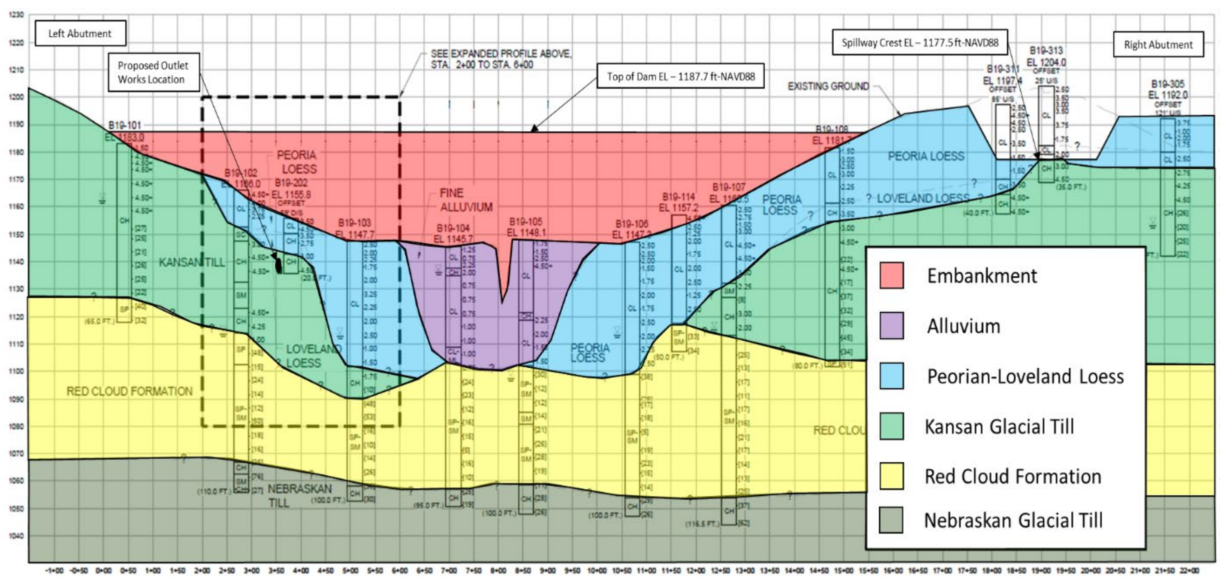


Figure 32. DS19 Geologic Profile

5.2.3.2 Foundation Investigation

A total of 36 borings were completed during the 2018 design completed by HDR Engineering and documented in the following report:

- Engineering Preliminary Design Report, Dam Site 19 and Associated Improvements, West Papillion Creek Subwatershed, HDR Engineering, Inc., dated April 2018.

These borings ranged from 10 to 110 feet in depth, penetrating four types of material: alluvium, loess, red cloud formation, and glacial drift. Disturbed soil samples from the borings were obtained using push and drive sampling, and undisturbed samples were obtained with thin-walled tube samples, 3-inch outside diameter, hydraulically pushed in general accordance with ASTM D 1587, "Standard Practice for Thin Walled Tube Sampling of Soils for Geotechnical Purposes." at prescribed intervals.

Split-barrel samples, designated "S" samples, were obtained while performing Standard Penetration Tests (SPT) with a thick walled sampler, 1.5-inch inside diameter, driven in general accordance with ASTM D 1586, "Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils." The N-value, reported in blows per foot, represents the number of blows required to drive the sampler over the last 12 inches of the 18-inch sample interval.

Laboratory tests were performed on disturbed and undisturbed samples in accordance with current ASTM or state-of-the-practice test procedures. The foundation soils were tested to determine moisture content, dry density, plasticity, grain size distribution, shear strength (UU triaxial and unconfined compression tests) and consolidation properties. Soils that will be excavated for potential use as borrow material for the Main Dam were tested to determine moisture content, dry density, plasticity, grain-size, and moisture-density relationship. Foundation properties are summarized in Table 34. Detailed boring records and laboratory test results for DS19 can be found in Appendix A, "Main Dam and Water Quality Basins" of the Engineering Preliminary Design Report for Dam Site 19 prepared by HDR Engineering.

Table 34. Summary of Foundation Properties

Unit	USCS	Strength Description	Dry Density (pcf)	Liquid Limit	Plasticity Index
Alluvium	Lean clay (CL) and fat clay (CH)	Soft to Stiff	85-104	24-50	5-30
Loess	Mostly lean clay (CL)	Very Soft to Very Stiff	-	-	-
Glacial Drift Till	Lean clay (CL) and fat clay (CH)	Stiff to Very Stiff	-	-	-
Red Cloud Formation	Poorly graded sand (SP) and silty sand (SM)	Soft to Very Stiff	-	-	-

5.2.3.3 Ground Water

Ground water within the alluvial valley ranged from elevation 1,098.5 to 1,136.5 feet NAVD88. At the abutments, the groundwater table ranged from elevation 1,114.5 to 1155.5 feet NAVD88. Fluctuations in the level of the groundwater may have occurred due to seasonal variations in local and regional precipitation and other factors not evident at the time of measurement. None of the borings in the location of the spillway encountered a water level above the design spillway channel elevation of 1,175.5 feet NAVD88. However, the groundwater elevations in a few of the borings in the left abutment were above the outlet works excavation where proper dewatering may be required to ensure dry working conditions.

5.2.4 DESIGN ANALYSES

Stability, seepage, settlement, and loess collapse analyses were performed at DS19 for the Engineering Preliminary Design Report prepared by HDR Engineering. The detailed analyses can be read in Appendix A, "Main Dam and Water Quality Basins" of the report. Below is a list of the key points summarizing the analyses:

- The stratigraphy and soil properties are similar to other Papillion Creek Dam Sites which have performed well since their construction.
- The foundation materials are suitable to support the construction of a dam embankment and appurtenant structures
- The material from the spillway and upstream pool area borrow excavations is suitable to use as fill for the dam embankment.
- The factor of safety determined from each of the stability analysis based on material properties from the foundation investigation meets or exceeds minimum factor of safety requirements.
- A maximum centerline foundation settlement of 3.3 feet is expected, so approximately one foot of overbuild is expected to be necessary to account for post-construction settlement. Adjustments to the amount of overbuild may be made based on the results of the instrumentation monitoring during construction.
- The chimney drain and horizontal blanket drain is considered adequate for controlling seepage and a foundation toe drain or relief wells will not be required to mitigate for seepage through and underneath the embankment.
- The Loess foundation soils at DS19 have dry densities and moisture contents above the threshold to exhibit any potential for collapse upon wetting.

5.3 SIMPLIFIED HYDROLOGIC LOADING

5.3.1 PURPOSE

This section documents the simplified hydrologic loading development for incorporation into risk analysis for the potential Dam Site 19 (DS-19) of the Papillion Creek basin. Due to time, funding, and data limitation associated with an ungaged new site, precipitation frequency hydrologic modeling was focused upon. The Risk Management Center Reservoir Frequency Analysis (RMC-RFA) model was not used and the stage frequency curve was developed from information from existing dams, peak flows determined from extreme precipitation, and engineering judgment. The loading curve should be further developed for advanced risk analysis to incorporate approximated period of record inflow and Monte Carlo simulation.

5.3.2 PROJECT SITE AND BACKGROUND

Figure 32 shows the location of DS-19 and Figure 33 shows the After ADM Design (Balanced). This was the design adopted as the National Economic Development (NED) plan. It is called the After ADM Design (Balanced) in the Papillion Creek GRR study. The drainage area to the dam is 4.3 square miles. Refer to the Hydrology Appendix for more information if needed. Outflows from DS-19 contribute to the South Papillion Creek.

5.3.3 HYDROLOGIC MODEL

Figure 34 shows the HMS version 4.4 beta model (HEC, 2020). DS-19 was modeled with the 72-inch diameter outlet fully open for all frequency events. Unit hydrograph (UH) peaking was varied depending on the size of the event modeled. Events smaller than the 1/500 AEP had no UH peaking, the 1/1000 AEP had 25-percent UH peaking, and the 1/10,000 AEP had 50-percent UH peaking. Refer to the Hydrology Appendix for information on the Clark UH parameters used as well as the rating and storage curves for the dam. The hydrology model used the frequency storm meteorologic model to develop hyetographs. Basin model assumptions were consistent with those used in the most reasonable Probable Maximum Flood (PMF) modeling documented in the Hydrology Appendix.

5.3.4 PRECIPITATION

Peak flows used to inform the stage-frequency curve were developed from three sources: the Applied Weather Associates (AWA) report used in the FYRA study (FYRA, 2018), NOAA Atlas 14, and the RMC Best Fit model. Refer to the Hydrology Appendix and FYRA analysis (FYRA, 2018) for information on the AWA precipitation. Table 35 shows the NOAA Atlas 14 precipitation depths as well as the 1/10,000 AEP precipitation estimated with the RMC Best Fit model through consultation with subject matter experts from the Risk Management Center (RMC); the depths of other durations were approximated using average ratios from the NOAA Atlas 14 data of the duration of interest to the 24-hr value.

Table 35. NOAA Atlas 14 precipitation & Best Fit 1/10,000 AEP estimate

DS19: NOAA Atlas 14 Median Depths (Inches)											
Return Interval (YRS)	**1**	**2**	**5**	**10**	**25**	**50**	**100**	**200**	**500**	**1,000**	**10,000**
AEP	0.999	0.5	0.2	0.1	0.04	0.02	0.01	0.005	0.002	0.001	0.0001
5-Minute	0.37	0.44	0.55	0.65	0.78	0.89	1.00	1.11	1.26	1.37	2.45
15-Minute	0.67	0.79	0.99	1.16	1.40	1.59	1.78	1.98	2.24	2.45	4.37
60-Minute	1.24	1.48	1.89	2.25	2.75	3.16	3.58	4.02	4.62	5.10	8.61
2-Hour	1.52	1.83	2.35	2.81	3.47	4.01	4.57	5.16	5.98	6.62	10.88
3-Hour	1.69	2.03	2.62	3.15	3.92	4.55	5.22	5.94	6.93	7.73	12.32
6-Hour	1.97	2.35	3.02	3.63	4.54	5.31	6.12	7.00	8.24	9.24	14.42
12-Hour	2.28	2.66	3.35	3.99	4.95	5.76	6.63	7.58	8.94	10.00	15.88
24-Hour	2.62	2.99	3.68	4.32	5.29	6.12	7.02	8.01	9.41	10.60	17.17

5.3.5 STAGE FREQUENCY CURVE

Figure 35 shows the adopted graphical stage frequency curve. This curve was informed by the peak inflows and stages produced by the precipitation discussed previously, through reference to the existing Papillion Creek Dams stage frequencies, and engineering judgement.

As mentioned before, the stage frequency curve presented has significant uncertainty, especially for infrequent events. A full risk analysis would include modeling with RMC-RFA which uses a Monte Carlo simulation with hundreds of realizations of observed inflow hydrographs scaled to randomly sampled peak flow frequencies and routed into the reservoir with different starting conditions. This modeling would provide a much more robust solution and confidence bounds around the stage frequency curve.

The stage-frequency curve for DS-19 beyond the 1/10,000 event was estimated by maintaining the same slope as that between the 1/1,000 and 1/10,000 annual exceedance probability (AEP) events and extrapolating out to the top of dam. Based on this estimate, the top of dam has an AEP of 1/3,5000,000 and spillway flows have an AEP of 1/1,600 when the outlet is fully open. If the outlet becomes blocked (no outlet), the top of dam would have an AEP of 1/250,000 and spillway flows would have an AEP of 1/500.

5.3.6 COMPARISON TO EXISTING PAPILLION CREEK DAMS

Table 36 shows the current stage-frequency curves from the most recent Periodic Assessments (PA) of the existing Papillion Creek dams along with that of the proposed DS-19 (USACE 2015, 2017, 2018a, 2018b DRAFT). Note that the existing Papillion Creek dam's stage-frequency curves were estimated using graphical fits and sometimes the outdated MCRAM Monte Carlo software. Only one PA, Dam Site 20-Wehrspann Lake, leveraged best practice RMC-RFA software, and the report for that project is a draft.

Compared with the existing Papillion Creek dams, the proposed DS-19 would experience more frequent spillway flows. This is influenced by the spillway crest being set to an elevation less than a Reservoir Design Flood less than the Standard Project Flood, which was used to set spillway crests of the existing Papillion Creek Dams.

The probability of DS-19 overtopping falls within the range of overtopping probabilities determined for the other dams.

Table 36. Spillway and Overtopping Frequencies

Project	Dam Site Number	Spillway Freq (AEP)	Top of Dam Freq (AEP)
Cunningham	Papio No. 11	1/70,000	1/10,000,000
Standing Bear	Papio No. 16	1/20,000	1/75,000
Zorinsky	Papio No. 18	1/3,300	1/100,0000
Wehrspann	Papio No. 20	1/80,000	1/500,000
	Papio No. 19 (w/ outlet)	1/1,600	1/3,500,000
Proposed (DS-19)	Papio No. 19 (w/o outlet)	1/500	1/3,000,000

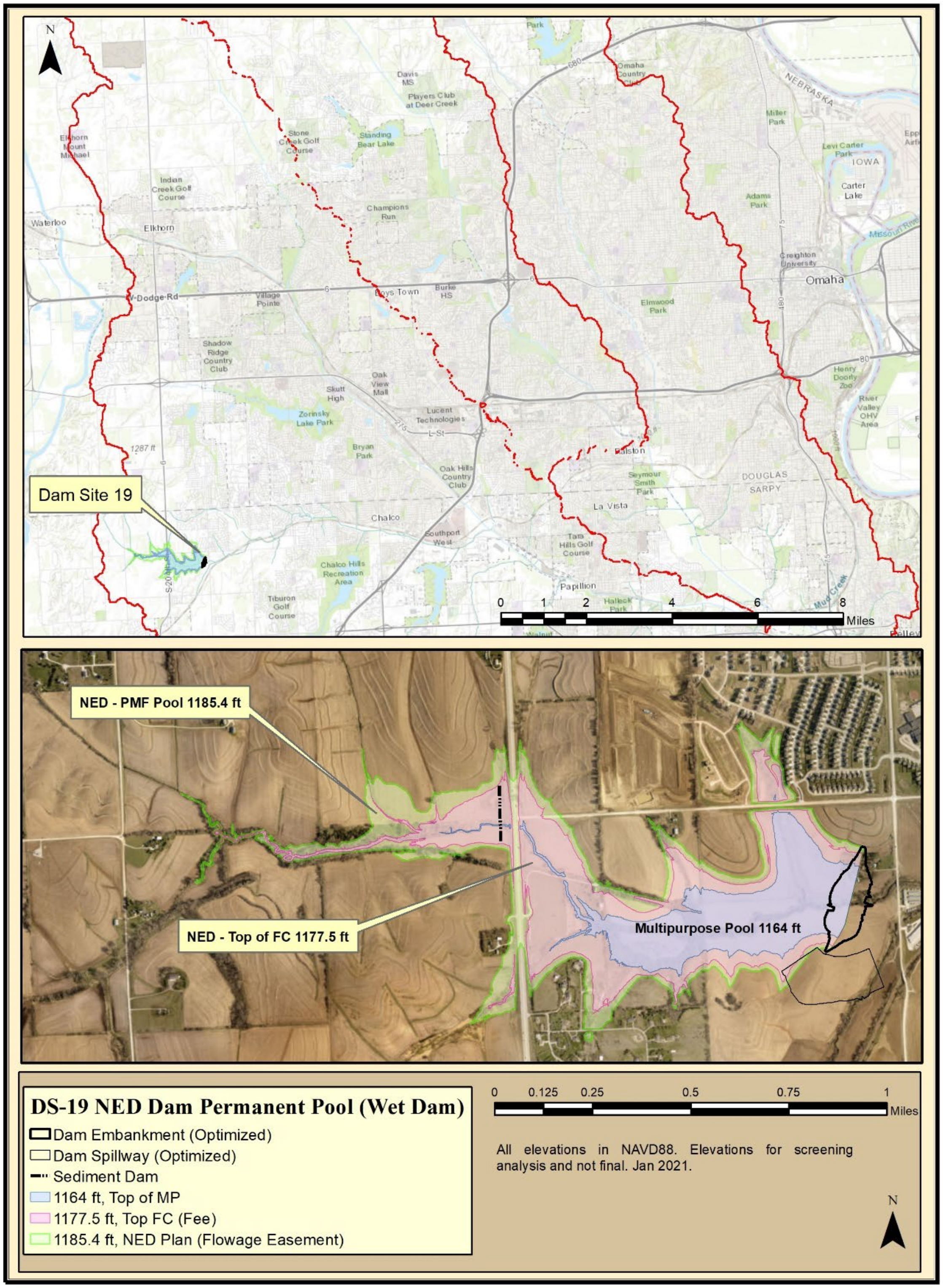


Figure 33. DS-19 Location. After ADM Wet Dam (Balanced) Alternative Shown

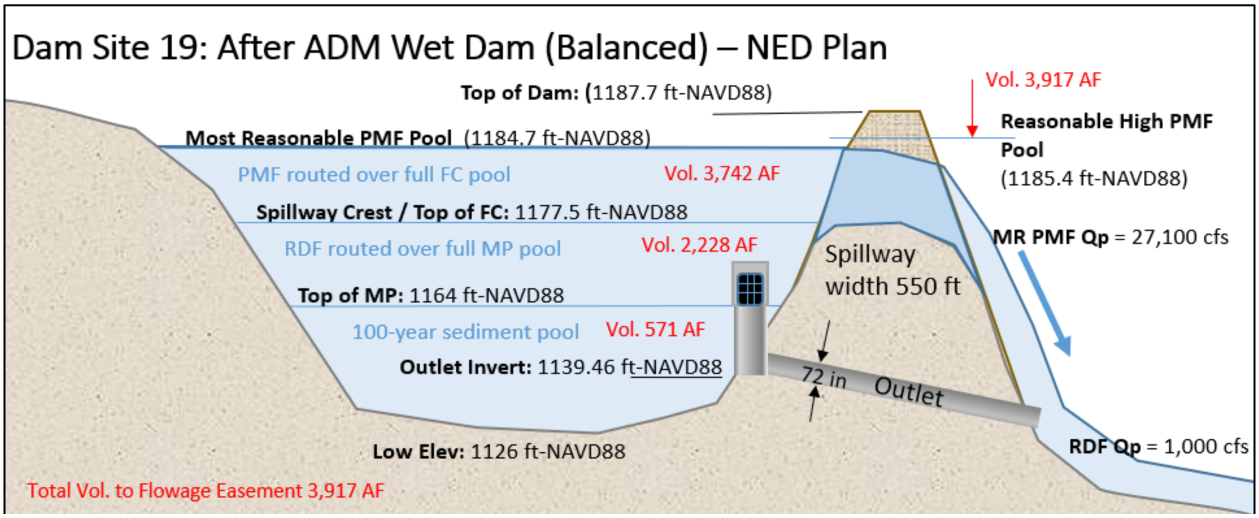


Figure 34. Wet Dam After ADM (Balanced) Design. NED design. Best balanced of cut and fill

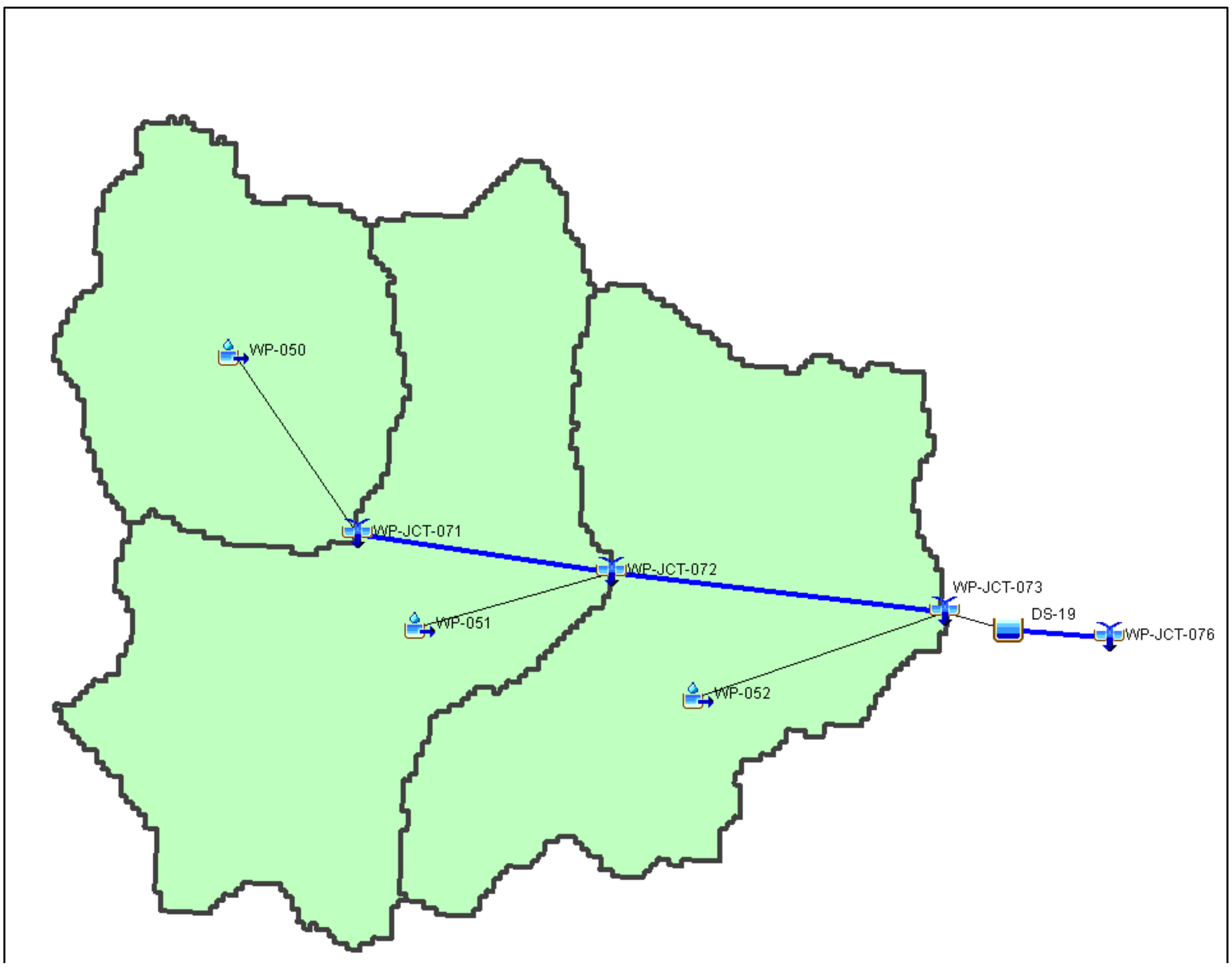


Figure 35. HEC-HMS model

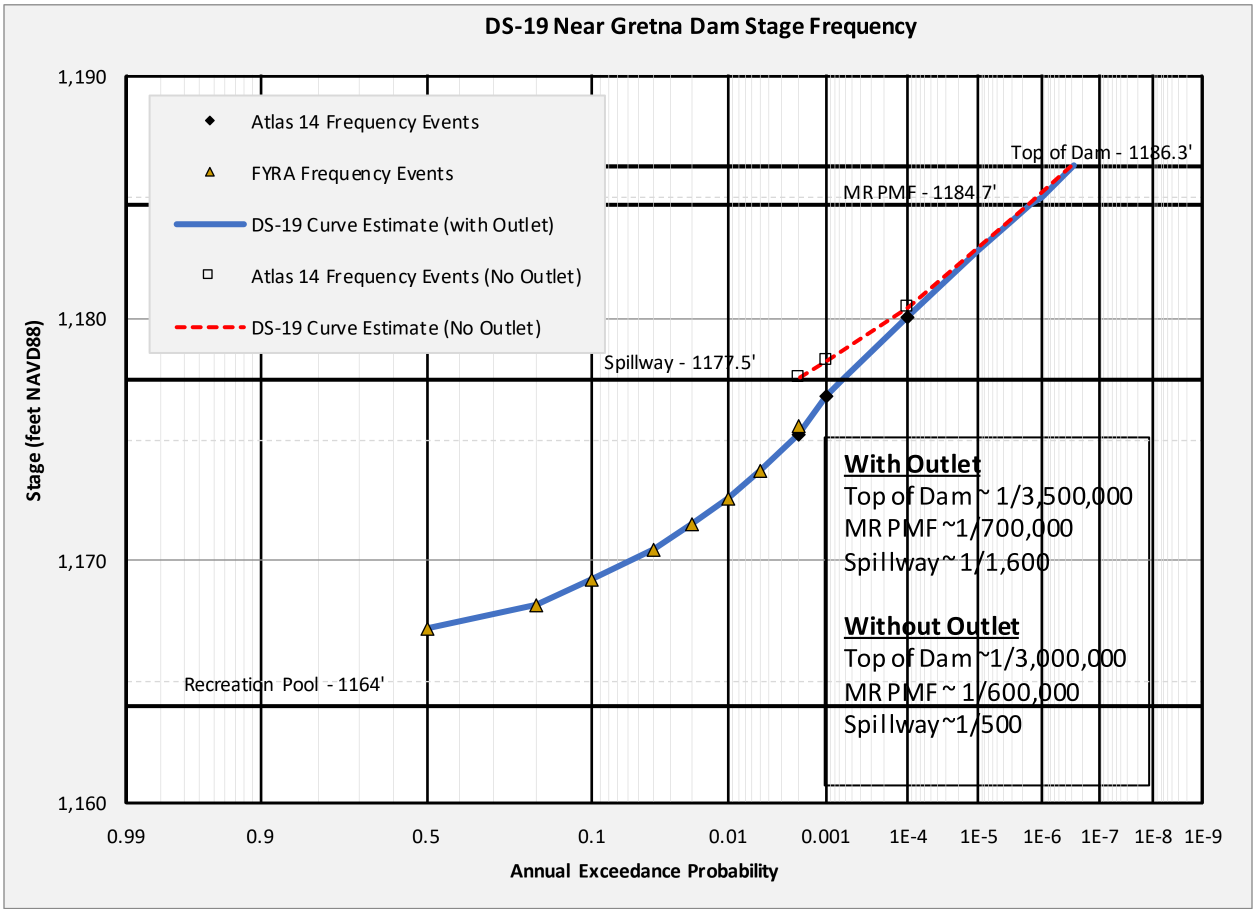


Figure 36. DS-19 adopted stage-frequency

5.4 SEISMIC HAZARDS

Previous seismic evaluations completed for existing Papillion Creek Dam Sites 11, 16, 18, and 20 determined that the central plains area where the Papillion Creek Dam Sites reside is considered tectonically stable with only occasional, minor earthquake activity. Due to the low seismicity in the area of DS19, all seismic related potential failure modes were excluded from consideration.

5.5 CONSEQUENCES

5.5.1 BACKGROUND

USACE has established a national standard of modeling procedures to support the estimation of consequences for breach and non-breach flood inundation scenarios over a range of loading conditions. Inundation models extend from the dam downstream to a point of no significant consequences. USACE developed baseline consequence estimates for breach and non-breach inundation scenarios, uncertainty statistics for life loss estimates, and inundation mapping products. The difference between breach and non-breach consequences for a particular loading condition is the incremental consequences (i.e., those directly attributable to the dam breach for that loading condition).

5.5.2 INUNDATION SCENARIOS

For DS19's abbreviated SQRA, several breach and non-breach scenarios were performed covering a range of pool elevations: max high pool (MH), top of active storage pool (TAS), and normal high pool (NH).

The maximum high pool elevation corresponds to the most reasonable probable maximum flood (PMF) pool elevation. For typical flood risk management dams with uncontrolled spillways, the top of active storage pool elevation corresponds to the emergency spillway crest elevation. Figure 36 provides a dam cross-section for DS19. The normal high pool elevation, also referred to as the 10% exceedance duration pool elevation, corresponds to the pool elevation that is exceeded approximately 10% of the time (36 to 37 days per year, on average) under normal operating conditions. This scenario represents a relatively high, though normal, pool condition that can be expected to occur every year.

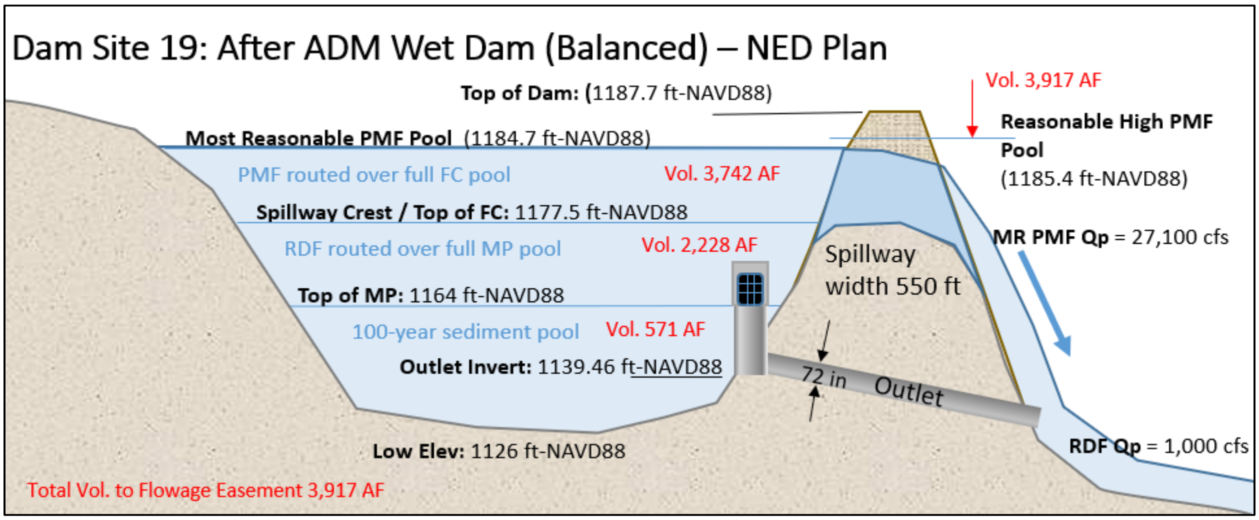


Figure 37. Dam Site 19 Cross-Section

Per the MMC guidance, project hydrology includes both inflows into the proposed reservoir as well as inflows to the downstream tributaries and is held constant between breach and non-breach scenarios. The max high pool scenario routes the most reasonable PMF hydrograph, from a starting pool elevation equal to that of the top of active storage pool, through the proposed dam. Meanwhile, the 10% Annual Exceedance Probability (AEP) event is routed along the downstream tributaries. The top of active storage pool scenario scales the most reasonable PMF hydrograph and routes that through the proposed dam, from a starting pool elevation equal to the 10% exceedance pool elevation. The scaling factor used must result in the pool reaching the top of active storage elevation. The 50% AEP event is routed along the downstream tributaries. The normal high pool scenario routes a constant inflow to maintain the 10% exceedance duration pool elevation through the proposed dam site and the 50% AEP event along the downstream tributaries. The values used for the 50% and 10% AEP events consisted of subbasin runoff for full build out conditions. Minimum flows equal to 10% of the 50% AEP were used at each boundary condition downstream of the dam sites to enhance model stability. Minimum flows were also used for dam inflows to hold starting pool elevations constant until the peak could be routed through.

Each embankment failure mode was assumed to be initiated by piping. The max high pool failure is initiated when the max high pool elevation is reached. Likewise, the top of active storage pool failure is initiated when the top of active storage pool elevation is reached. However, as the normal high pool failure scenario requires a constant pool elevation equal to the 10% exceedance duration pool elevation, failure is initiated 24 hours after the start of the model simulation.

Details for each failure and non-failure scenario are summarized in the Table 37.

Table 37. Details for DS19 Embankment Failure and Non-failure Scenarios

Model Scenario	Inflow Requirements	Downstream Flow Requirements	Starting Pool Elevation (ft)	Minimum Inflow (cfs)	Breach Trigger
MH-F	Most Reasonable PMF	10% AEP Future Build-out	1177.5	984.2	1184.7’
MH-NF	Most Reasonable PMF	10% AEP Future Build-out	1177.5	984.2	-
TAS-F	0.31*Most Reasonable PMF	50% AEP Future Build-out	1167.2	273.72	1177.5’
TAS-NF	0.31*Most Reasonable PMF	50% AEP Future Build-out	1167.2	273.72	-
NH-F	273.72 cfs	50% AEP Future Build-out	1167.2	273.72	24 hours after model initiation
NH-F	273.72 cfs	50% AEP Future Build-out	1167.2	273.72	-

Additionally, a breach of the spillway at the max pool elevation was evaluated. The breach trigger was assumed to happen at the maximum pool elevation. Details of this scenario is provided in Table 38.

Table 38. Details for DS19 Spillway Failure Scenario

Model Scenario	Inflow Requirements	Downstream Flow Requirements	Starting Pool Elevation (ft)	Minimum Inflow (cfs)	Breach Trigger
Spillway MH-F	Most Reasonable PMF	10% AEP Future Build-out	1177.5	984.2	1184.7’

For additional information regarding the determination of the most reasonable PMF, pool elevations, and the downstream tributary flows, refer to Section 5.3 Simplified Hydrologic Loading.

5.5.3 BREACH ASSUMPTIONS

Breach parameters were calculated using four regression equations: MacDonald and Langridge-Monopolis, Froelich (1995a), Froelich (2008), and Von Thun and Gillette. These four equation sets have typically been used for earth dams. The embankment breaches were assumed to be initiated by piping that followed a sine wave progression. The Max High Pool breach plans were run with each set of regression equations and the resulting max inundations were compared. the outflow hydrographs from the DS19 breach do not converge until the confluence of the West Papillion and Big Papillion Creeks, although downstream of the West Papillion and South Papillion confluence these differences appear minor. There are storage unites only 0.3 miles downstream of the DS19 embankment and the first habitable structures are approximately 2.0 miles downstream. Figure 37 shows the difference in the hydrographs produced by the regression equations at the latter location.

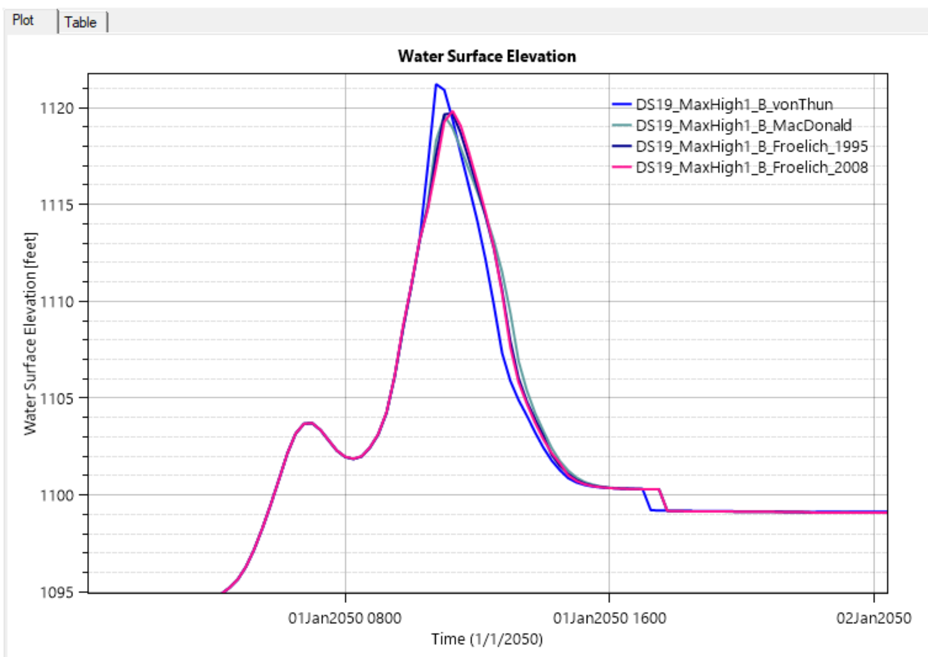


Figure 38. Outflow hydrographs from DS19 at closest habitable structures

Conservatively high (resulting from Von Thun and Gillette) and low (resulting from MacDonald and Langridge-Monopolis) values were not used. The remaining results were very similar in each case and Froelich (2008) was chosen to be the most appropriate for this application. Table 39 provides the required embankment information and resulting breach parameters for each scenario for DS19.

Table 39. Breach Parameters for DS19

Parameter	Max High Pool	Top of Active Storage	Normal High Pool
Top of Dam Elevation (ft)	1187.7		
Dam Crest Width (ft)	25		
Average u/s Slope of Dam Face (H:V)	4.5:1		
Average d/s Slope of Dam Face (H:V)	4.75:1		
u/s slope protection	Riprap to Multipurpose Pool, Topsoil and Grass		
d/s slope protection	Topsoil and Grass		
Breach Bottom Elevation (ft)*	1147.88	1147.88	1147.81
Pool Elevation at Failure (ft)	1184.7	1177.5	1167.2
Pool Volume at Failure (acre-ft)	3742	2228	845
Failure Mode	Piping	Piping	Piping
Breach Bottom Width (ft)	105	85	55
Resulting Side Slopes (H:V)	0.7	0.7	0.7
Breach Development Time (hrs)	0.99	0.77	0.47

* Breach bottom elevation was taken to the lowest elevation terrain would allow

To adequately model a spillway breach, several assumptions were made. To determine an appropriate assumption for breach bottom elevation, past studies on the current Papillion Creek dams were reviewed. It is assumed that studies on the current dam sites are applicable to DS19 because of the similar design criteria, terrain, and expected soil conditions. Table 40 compares the details of each spillway.

Table 40. Comparison of Papillion Creek Dam Sites Spillway Criteria

	Papio Dam Site 11	Papio Dam Site 16	Papio Dam Site 18	Papio Dam Site 20	Papio Dam Site 19
Spillway Crest Length (ft)	200	282	200	232	550
Spillway Side Slopes (H:V)				3:1	3:1
Total Spillway Length (ft)	741	1,287	1,333	746	1,072
Total Elevation Drop (ft)	30.9	28.5	39.2	41	31
Long. Slope away from Crest (ft/ft)	.0020	0.0025	0.0020	0.0020	0.0289
Peak Discharge (cfs)	18,700	9,500	30,000	17,500	24,000
Flow Duration (hrs)	30	27.8	31.8	40	12
Avg Spillway Velocity (ft/s)	6.95	8.10	10.17	7.14	10
Depth over Spillway (ft)	3.8	4.5	7.2	4.0	9

In 2007, a spillway erosion study was done on Dam Site 20 (DS20). The purpose of the study was to use the latest computer modeling techniques to determine erosion impacts to the DS20 emergency spillway. Based on the study model and the best available data at that time, the spillway head cut erosion is not generally considered severe enough to breach the spillway crest. However, the model did indicate that a breach is possible using a distribution sampling analysis and conservative soil variables. Figure 38 shows the resulting spillway cross-section under three distribution conditions. The maximum condition is denoted by the yellow line, the minimum condition is denoted by the green line, and the mean condition is denoted by hatching. Due to the longer spillway length and shorter spillway duration, to assume a full breach, as is shown in the maximum condition, would be overly conservative. Therefore, the mean condition, with a most upstream head cut depth of 6 feet, was assumed for the DS19 spillway breach depth.

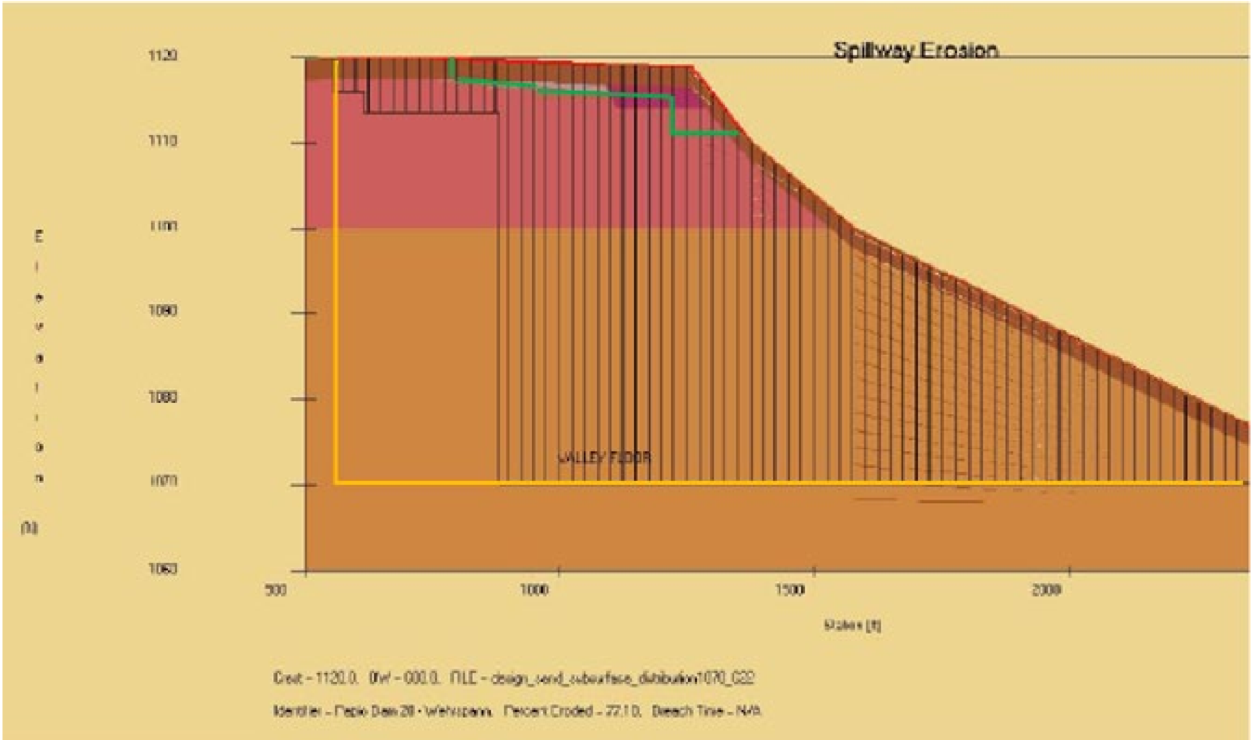


Figure 39. Distribution Conditions for the expected erosion on DS20 assuming conservative soil variables

The next assumption is that there will be mass wasting of the spillway downstream of the crest until finally there only remains a sliver of spillway, similar to that of a dam embankment, making the regression equation calculator in HEC-RAS applicable for this scenario. For consistency, Froelich (2008) was again used to calculate breach parameters. The failure mode for each is overtopping triggered at the time the max high pool elevation was reached. This is a conservative assumption because erosion of the spillway would begin at the downstream edge and require some time to work its way back to the pool. This would likely take more time than is necessary to reach the peak pool elevation, resulting in quicker arrival times. Table 41 provides the required embankment information and resulting breach parameters for the DS19 spillway failure.

Table 41. DS19 Spillway Breach Parameters

Parameter	DS19 Spillway
Spillway Crest Elevation (ft)	1177.5
Spillway Crest Width (ft)	25
Average u/s Spillway Channel Slope (H:V)	4.5:1
Average d/s Spillway Channel Slope (H:V)	4.75:1
u/s spillway channel protection	Topsoil and Grass
d/s spillway channel protection	Topsoil and Grass
Spillway Breach Bottom Elevation (ft)*	1171.5
Pool Elevation at Failure (ft)	1184.7
Pool Volume at Failure (acre-ft)	3742
Failure Mode	Spillway Erosion
Breach Bottom Width (ft)	149
Resulting Side Slopes (H:V)	1
Breach Development Time (hrs)	6.29

5.5.4 LIFE LOSS

Population at risk (PAR) is defined as the number of people downstream of a dam that would be subject to inundation risk. PAR and life loss estimates were generated using HEC's LifeSim (HEC-LifeSim) software for breach and non-breach inundation scenarios.

Table 42. DS19 Estimated Population at Risk

Reservoir Level (ft-NAVD88)	Population at Risk					
	Breach		Non-Breach		Incremental	
	Day	Night	Day	Night	Day	Night
MHP	6,684	3,045	3,254	1,707	3,430	1,338
MHP-Spillway Breach	3,303	1,713	3,254	1,707	49	6
TAS	647	582	1	3	646	579
NHP	2	7	1	3	1	4

5.5.4.1 Assumptions and Methodology

The life loss methodology in *HEC-LifeSim* is based on the *LifeSim* methodology. To determine the percentage of population at risk (PAR) within a structure that is warned and mobilized over time, several parameters are used within *HEC-LifeSim* to estimate the probable values of warning and mobilization percentages at each time step. These include when warnings will be issued (hazard identification and delays), how long they will take to become effective (warning diffusion), and the rate at which PAR will mobilize in response (mobilization). Figure 39 represents an example dam breach warning and mobilization timeline.

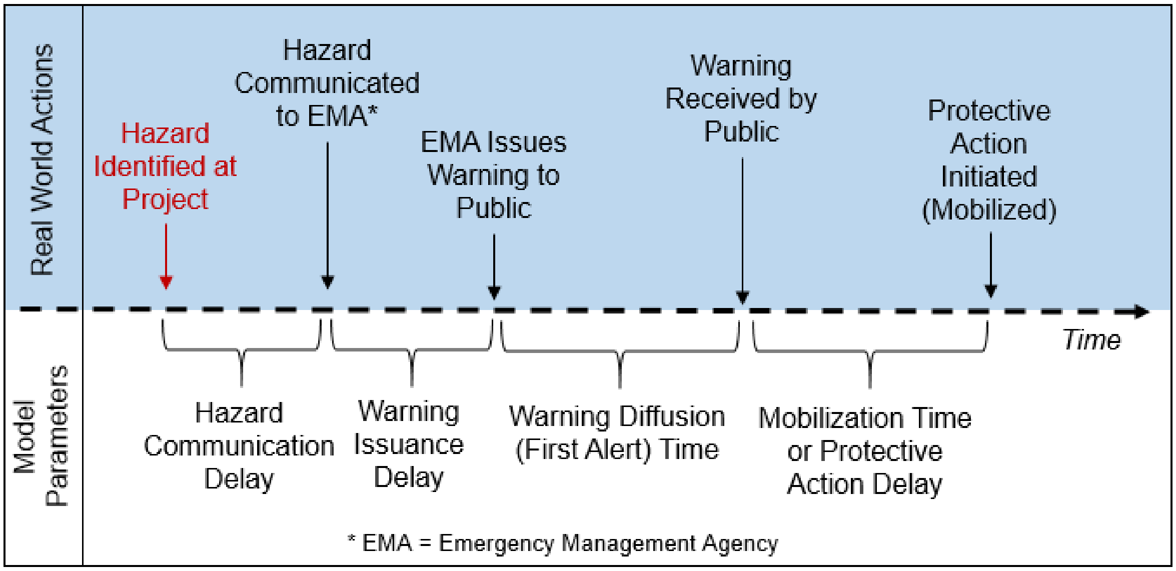


Figure 40. Dam Breach Warning and Mobilization Timeline

- The Hazard Identification time is the time at which a hazard is identified (dam breach or major flooding) relative to when it actually occurs (the actual breach initiation time). For example, a hazard identification one hour prior to breach initiation would be "-1 hour," meaning that the hazard was initially identified one hour before it actually occurred. The MMC uses two different warning scenarios with different ranges of hazard identification time: minimal warning and ample warning.
    - Minimal Warning scenarios have the hazard identification relative time set as a uniform distribution between 2 hours prior to breach initiation and at time of breach initiation (-2 to 0 hours).
    - Ample Warning scenarios have the hazard identification relative time set as a uniform distribution between 6 hours prior to breach initiation and 2 hours prior to breach initiation (-6 to -2 hours).
    - For both scenarios, in-pool areas and non-breach double-warning areas are set at least 72 hours prior to the simulation start.

- The Hazard Communication Delay is the time that it would take from when the hazard is identified to when the emergency planning zone (EPZ) representatives would be notified. For example, if a breach occurs when no one is observing the project then the emergency managers could be notified 1 hour after the hazard is identified. The hazard communication delay is set as a uniform uncertainty distribution between 0.01 hours and 0.5 hours.

- The Warning Issuance Delay is the time it takes from when the emergency managers receive the notification of the imminent hazard to when they issue the first evacuation order to the public. The warning issuance delay is set at the preset configuration of "Preparedness Unknown," which utilizes a Lindell uncertainty distribution. The delay is randomly sampled from 0 to 6 hours, but it is positively skewed such that results from 0 to 1.5 hours are more likely.

- The Warning Diffusion or First Alert parameter defines the warning diffusion curve for daytime and nighttime. The diffusion curve represents the percentage of the population which will receive a first alert warning over time during daytime hours from when the warning was issued. The first alert curves are set at the preset configuration of "Unknown" which samples from a uniform uncertainty distribution where the upper bound curve warns 100 percent of the PAR after 1.5 hours and the lower bound curve warns 100 percent of the PAR after 6 hours.

- The Protective Action Initiation (PAI) parameter defines the mobilization curve. The PAI curve represents the percentage of the population which will take protective action over time from when the first alert is received. For areas downstream of the dam the PAI/mobilization curve for is set at the preset configuration of "Preparedness: Unknown / Perception: Unknown" which samples a uniform uncertainty distribution with maximum mobilization rates between 83 and 100 percent after 72 hours. For in-pool areas, the curve is set at the preset "Preparedness: Unknown / Perception: Likely to Impact" which samples a uniform uncertainty distribution with maximum mobilization rates between 94 and 100 percent after 72 hours.

5.5.4.2 Life Loss Uncertainty

Life loss was estimated using uncertainty sampling methods on the parameters in *HEC-LifeSim*. These parameters include the warning issuance, the warning delay and diffusion curves, and the mobilization rate curve. For this reason, life loss results are presented below with five number statistics in order to understand the potential range of life loss. In order to provide a generic suite of warning scenarios that could be used during the risk assessment for the risk-driver potential failure modes, minimal and ample warning scenarios (as described above in the HEC-LifeSim parameters section) are used. It should be noted that there is also a standard delay parameter added onto the warning issuance time based on case histories. The ample warning scenario is generally more appropriate for internal erosion PFMs where failure progression is observed and discovery occurs before breach initiation (i.e., dams that are watched) and overtopping. The minimal warning scenario is generally more appropriate for seismic PFMs where failure can be instantaneous or where failure progression is not observed (i.e., dams that are not watched). The estimated life loss statistics for the two warning issuance scenarios are summarized in the following tables. While the breach and non-breach statistics represent the outcome from the simulations, the incremental life loss "statistics" were obtained by subtracting the breach and non-breach statistics.

5.5.4.3 Life Loss Results

Table 43 provides a summary of consequence information with minimal warning from DS19 modeling. Information provided includes number of inundated structures, population at risk and median life loss estimates. As shown in Table 43, no median life loss estimates are greater than 0, regardless of the reservoir level or breach scenario.

Table 43. DS19 Estimated Downstream Information by Reservoir Level and Breach Scenario (Minimal Warning)

Reservoir Level	Number of Structures	Population at Risk		Median Life Loss	
		Day	Night	Day	Night
MHP	771	6,684	3,045	0	0
MHP – Spillway Breach	456	3,303	1,713	0	0
TAS	163	647	582	0	0
NHP	5	2	7	0	0

The tables below display the results for modeling DS19. Results are provided MHP, TAS, and NHP pool heights. An additional table is provided for each dam displaying modeling results for MHP with a spillway failure.

Table 44. DS19 Estimated Life Loss for MHP Breach

Statistic	Life Loss for Minimal Warning Scenario					
	Breach		Non-Breach		Incremental	
	Day	Night	Day	Night	Day	Night
95th Percentile	2	1	0	0	2	1
75th Percentile	1	0	0	0	1	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0

Statistic	Life Loss for Ample Warning Scenario					
	Breach		Non-Breach		Incremental	
	Day	Night	Day	Night	Day	Night
95th Percentile	2	1	0	0	2	1
75th Percentile	1	0	0	0	1	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0

Table 45. DS19 Estimated Life Loss for TAS Breach

Statistic	Life Loss for Minimal Warning Scenario					
	Breach		Non-Breach		Incremental	
	Day	Night	Day	Night	Day	Night
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0

Statistic	Life Loss for Ample Warning Scenario					
	Breach		Non-Breach		Incremental	
	Day	Night	Day	Night	Day	Night
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0

Table 46. DS19 Estimated Life Loss for NHP Breach

Statistic	Life Loss for Minimal Warning Scenario					
	Breach		Non-Breach		Incremental	
	Day	Night	Day	Night	Day	Night
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0

Statistic	Life Loss for Ample Warning Scenario					
	Breach		Non-Breach		Incremental	
	Day	Night	Day	Night	Day	Night
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0

The table below displays the results of a spillway failure at the MHP level.

Table 47. DS19 Estimated Life Loss for MHP - Spillway Failure

Statistic	Life Loss for Minimal Warning Scenario					
	Breach		Non-Breach		Incremental	
	Day	Night	Day	Night	Day	Night
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0

Statistic	Life Loss for Ample Warning Scenario					
	Breach		Non-Breach		Incremental	
	Day	Night	Day	Night	Day	Night
95th Percentile	0	0	0	0	0	0
75th Percentile	0	0	0	0	0	0
Median	0	0	0	0	0	0
25th Percentile	0	0	0	0	0	0
5th Percentile	0	0	0	0	0	0

5.5.5 ADDITIONAL CONSIDERATIONS

ER 1110-2-1451 requires lands downstream of spillways to be acquired if spillway discharge could create or significantly increase a hazardous condition. The ER further defines non-hazardous conditions to be those areas with:

1. Maximum flood depths of 2 feet in both urban and rural areas
2. Flood depths that are essentially non-damaging to urban property
3. Flood durations of a maximum of 3 hours in urban areas and 24 hours in agricultural areas
4. Velocities that do not exceed 4 ft/s
5. Minimal debris and erosion potential
6. Flood frequency less than 1%

To evaluate the creation of and increase to hazardous conditions downstream of the DS10 spillway, the 2D hydraulic model was run with both with- and without-project conditions using PMF project hydrology upstream of the dam location and the 10% AEP event downstream of the dam to determine the increase to flood depths and velocities. Once areas of significant increase to and creation of hazard conditions were identified, further analysis was conducted to determine areas where flood depths were greater than 2ft or velocities were greater than 4 ft/s. 8.2 acres were identified for potential acquisition due to expected hazard conditions downstream of the DS19 spillway, see Figure 40 below.

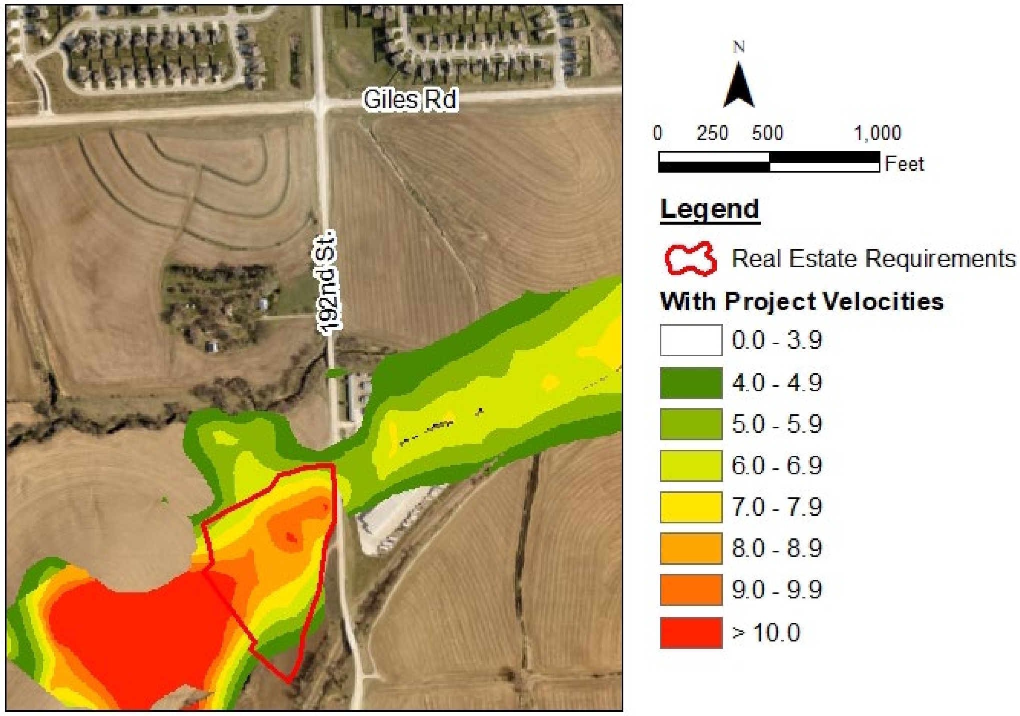


Figure 41. Real Estate Requirements downstream of DS19 Spillway

5.6 ABBREVIATED SEMI-QUANTITATIVE RISK ASSESSMENT

5.6.1 POTENTIAL FAILURE MODE ANALYSIS

A failure mode is a unique set of conditions and/or sequence of events that could result in failure, where failure is "characterized by the sudden, rapid, and uncontrolled release of impounded water" (FEMA 2003). A Potential Failure Mode Analysis (PFMA) is the process of identifying and fully describing potential failure modes. Due to an aggressive schedule to complete the General Reevaluation Report (GRR) for the Papillion Creek Basin, the SQRA for DS10 and DS19 was abbreviated to complete in one day with the team. To save time, brain-storming to identify potential failure modes (PFMs) was completed by a smaller team of engineers prior to meeting based off the available preliminary design for DS19 and review of design, performance, and failure mode analyses conducted for existing Papillion Creek Dam Sites 11, 16, 18 and 20. The goal of the abbreviated SQRA team was then to further develop the potential failure modes, based on the team's understanding of the project vulnerabilities resulting from the review of the preliminary design. The abbreviated SQRA team is summarized in Table 48.

Table 48. Abbreviated SQRA Team

Name	Office	Discipline
Kyle Heddens	Omaha District	Geotechnical Engineer (Facilitator)
Bob Worden	Omaha District	Geotechnical Engineer (Co-Facilitator)
Ross Cullin	Omaha District	Geotechnical Engineer (DSPM)
Steve Butler	Omaha District	Geotechnical Engineer
Jamie Bond	Walla Walla District	Geotechnical Engineer
Laila Berre	Northwest Division	Geotechnical Engineer (Division DSPM)
Brad Bird	Northwest Division	Hydraulics Engineer
Roger Kay	Omaha District	Hydraulics Engineer
Laura Knapp Leiferman	Omaha District	Hydraulics Engineer
Ben Lorenzen	Omaha District	Hydraulics Engineer
Joshua Melliger	Omaha District	Hydrology Engineer
Jennifer Christensen	Omaha District	Hydrology Engineer
Rachel Shrader	Omaha District	Planning Project Manager
Greg Johnson	Omaha District	Planning Project Manager

From the list of potential failure modes developed prior to the abbreviated SQRA (Table 49), the team identified the failure modes judged to be risk drivers. Failure modes that were determined to be non-risk drivers were excluded from further consideration. An abbreviated justification for the exclusion of the non-risk drivers is provided in Section 5.6.2. For the risk driver failure modes, the pertinent background and performance data was discussed. Then, a complete failure description was prepared from initiation to breach. The discussion was then expanded to listing factors, data, or conditions that suggest the failure mode is more likely or less likely to occur and establishing the appropriate level of consequences. Lastly, any recommendations for risk-reduction actions to be incorporated into the preliminary design of DS19 to achieve all four Tolerable Risk Guidelines (TRGs) were discussed.

Table 49. DS19 Potential Failure Modes

PFM	Description
PFM 01	Seismic liquefaction of foundation causes crest settlement and overtopping
PFM 02	Seismic slope deformation (liquefaction/cyclic softening) of embankment and overtopping
PFM 03	Seismic slope stability failure
PFM 04	Seismic induced transverse cracking
PFM 05	Static slope stability failure (US/DS)
PFM 06	Overwash erosion
PFM 07	Overtopping
PFM 08	Concentrated Leak Erosion (CLE) through transverse crack in embankment at closure contact
PFM 09	CLE through transverse crack in embankment above chimney drain
PFM 10	CLE along the conduit
PFM 11	Erosion of embankment material into the conduit joints
PFM 12	CLE along conduit, driven by pressurized conduit flow and water exiting the joints
PFM 13	CLE through conduit joints
PFM 14	Backwards Erosion Piping (BEP) of non-plastic Red Cloud Formation through the left abutment
PFM 15	Spillway erosion
PFM 16	BEP of alluvial foundation
PFM 17	Overtopping of the dam due to clogged/damaged outlet works
PFM 18	CLE due to filter incompatibility of drains or conduit filter
PFM 19	Seismic failure of intake structure causing uncontrolled release
PFM 20	Seismic failure of conduit, causing CLE of embankment soils into joints
PFM 21	CLE at closure section due to poor compaction at interface
PFM 22	CLE erosion at old stream channel due to poor compaction
PFM 23	CLE at embankment/loess abutment interface due to collapse of loess from wetting
PFM 24	CLE along a poorly compacted layer at the embankment / alluvial foundation interface
PFM 25	Clogged internal drains

5.6.2 EXCLUDED FAILURE MODES

The following sections summarize the potential failure modes that were excluded from further consideration because they were deemed non-credible or credible but non-risk drivers.

5.6.2.1 Seismic Failure Modes

The following excluded potential failure modes are seismic failure modes:

- PFM 01: Seismic liquefaction of foundation causes crest settlement and overtopping;
- PFM 02: Seismic slope deformation (liquefaction/cyclic softening) of embankment and overtopping;
- PFM 03: Seismic slope stability failure;
- PFM 04: Seismic induced transverse cracking;
- PFM 19: Seismic failure of intake structure causing uncontrolled release

- PFM 20: Seismic failure of conduit, causing CLE of embankment soils into joints

There is low seismicity in the area and all seismic related potential failure modes were excluded from consideration based on coincident probability and earthquake analysis from the existing Papillion Creek Dams.

5.6.2.2 Slope Stability Failure Modes

Stability analyses based on material properties from foundation investigations at DS19 were completed in Appendix A of Engineering Preliminary Design Report, Dam Site 19 and Associated Improvements, West Papillion Creek Subwatershed, HDR Engineering, Inc., dated April 2018. The calculated factor of safety determined from each of the stability analysis meets or exceeds minimum factor of safety requirements. Therefore, PFM 05: Static slope stability failure (US/DS) was excluded from consideration.

5.6.2.3 Overtopping Failure Modes

The following excluded potential failure modes are related to overtopping:

- PFM 06: Overwash erosion;
- PFM 07: Overtopping;
- PFM 17: Overtopping of the dam due to clogged/damaged outlet works

The annual exceedance probability (AEP) of a hydrologic event that would raise the pool elevation to the design crest elevation is 1/3,500,000 with an operational outlet works or 1/2,500 ,000 without no assumed outflow from the outlet works. DS19 is approximately 210 feet wide from the landside toe to the crest centerline and is designed with a 25 foot wide crest, so a hydrologic event significantly less probable than the top of dam event (AEP 1/3,500,000 w/ outlet works, AEP 1/3,000,000 w/o outlet works) would be necessary to have the depth and duration of overtopping required to initiate and progress erosion of the landside embankment slope to breach. Due to the improbable hydrologic loading condition, potential failure modes relating to overtopping were excluded from consideration.

5.6.2.4 CLE/BEP through the Embankment

The following excluded potential failure modes are related to concentrated leak erosion (CLE) through the embankment:

- PFM 08: CLE through transverse crack in embankment at closure contact;
- PFM 09: CLE through transverse crack in embankment above chimney drain
- PFM 18: CLE due to filter incompatibility of drains or conduit filter
- PFM 21: CLE at closure section due to poor compaction at interface
- PFM 25: Clogged internal drains

The potential failure modes related to CLE through the embankment will require a hydrologic event that would raise the pool elevation above the top of active storage (TAS) and spillway crest loading elevation of 1177.5 feet NAVD88 (AEP 1/1,600) resulting in a tailwater elevation of 1134.6 feet NAVD88 and a head differential of 42.9 feet. The duration of such an event is 2 days and 14 hours above the normal high pool (NHP) elevation of 1167.2 feet NAVD88. For reference, the MHP or Most Reasonable (MR) PMF elevation of 1184.7 ft NAVD88 would require a hydrologic event with an AEP of 1/700,000. The MHP scenario would result in a tailwater of 1145.5 feet NAVD88 and a head differential of 39.2 feet. The duration of the MHP event is 2 days and 19 hours above the NHP elevation 1167.2 feet NAVD88. During these short duration, high water events, the embankment is unlikely to develop steady-state seepage conditions before the pool recedes.

The low-level intake consists of a 30-inch diameter RCP, which is insufficient to pass the 50-year or 100-yr exceedance frequency flood. Therefore, the embankment will have to be constructed to a minimum elevation to meet the minimum storage requirements prior to closure. Current construction practices will be used to construct DS19, so the existing embankment will be benched and the foundation interface will be cleared, grubbed, scarified, and recompacted prior to placing the closure section, to ensure a good contact between the closure section and previously placed embankment fill. Additionally, the size of the project allows for construction to be completed in one season. These factors reduce or eliminate the potential for a crack or poorly compacted layer at the closure section or due to poor construction practices caused by winter shutdowns.

If there is a flaw in the embankment, such as a crack or poorly compacted layer, the internal chimney and blanket drain are designed to reduce seepage pressures and retain eroded material to stop the progression of CLE. The material for the internal chimney and blanket drain will be properly sized to meet filter criteria for permeability, particle retention, and flow. Additionally, the embankment is wide (~390 feet at the floodplain elevation of 1147.3 feet NAVD88), resulting in a global gradient of 0.08 (1177.5 TAS EL – 1147.3 flaw EL / 390 feet progression) at TAS (AEP 1/1,600) and 0.10 (1184.7 MHP EL – 1147.3 flaw EL / 390 feet progression) at MHP (AEP 1/700,000) which are insufficient to initiate and progress CLE.

For all the above reasons, it was determined that all potential failure modes related to CLE through the embankment would be excluded from consideration.

5.6.2.5 CLE at the Outlet Works

The following potential failure modes are related to concentrated leak erosion (CLE) at the outlet works:

- PFM 10: CLE along the conduit;
- PFM 11: Erosion of embankment material into the conduit joints;
- PFM 12: CLE along conduit, driven by pressurized conduit flow and water exiting the joints
- PFM 13: CLE through conduit joints

The potential failure modes related to CLE at the outlet works will require a hydrologic event that would raise the pool elevation above the top of active storage (TAS) and spillway crest loading elevation of 1177.5 feet NAVD88 (AEP 1/1,600) resulting in a tailwater elevation of 1134.6 feet NAVD88 and a head differential of 42.9 feet. The duration of such an event is 2 days and 14 hours above the normal high pool (NHP) elevation of 1167.2 feet NAVD88. For reference, the MHP or MR PMF elevation of 1184.7 ft NAVD88 would require a hydrologic event with an AEP of 1/700,000. The MHP scenario would result in a tailwater of 1145.5 feet NAVD88 and a head differential of 39.2 feet. The duration of the MHP event is 2 days and 19 hours above the NHP elevation 1167.2 feet NAVD88.

The intake tower, 72” diameter RCP outlet conduit, and stilling basin will be founded entirely on glacial drift in the left abutment, represented primarily by very stiff, pebbly, sandy clay. Outlet works structures founded on similar glacial drift material at the existing Papillion Creek Dams have only experienced approximately 0.2 feet of settlement. Seepage along the outlet works under “normal” seepage conditions or due to cracks or flaws adjacent to the outlet works will be collected by a 10-foot-long and 3-foot-wide pervious backfill drain near the outfall of the 72” diameter RCP. The drain material will be properly sized to meet filter criteria for permeability, particle retention, and flow. The stilling basin will be protected with riprap revetment.

CLE at the outlet works will have to progress the 400-foot length of the 72” diameter RCP outlet conduit, resulting in a global gradient of 0.11 (1177.5 TAS EL – 1134.6 tailwater EL / 400 feet progression) at TAS (AEP 1/1,600) and 0.10 (1184.7 MHP EL – 1145.5 tailwater EL / 400 feet progression) at MHP (AEP 1/700,000). Additionally, due to the short duration of the high-water events, the impervious backfill around the outlet works is very unlikely to experience these high of gradients before the pool recedes during high water events.

For all the above reasons, it was determined that all potential failure modes related to CLE through the embankment would be excluded from consideration.

5.6.2.6 CLE/BEP through the Foundation

The following potential failure modes are related to concentrated leak erosion (CLE) or backward erosion piping (BEP) through the foundation:

- PFM 16: BEP of alluvial foundation;
- PFM 22: CLE erosion at old stream channel due to poor compaction
- PFM 23: CLE at embankment/loess abutment interface due to collapse of loess from wetting
- PFM 24: CLE along a poorly compacted layer at the embankment / alluvial foundation interface

The potential failure modes related to CLE or BEP through the foundation will require a hydrologic event that would raise the pool elevation above the top of active storage (TAS) and spillway crest loading elevation of 1177.5 feet NAVD88 (AEP 1/1,600) resulting in a tailwater elevation of 1134.6 feet NAVD88 and a head differential of 42.9 feet. The duration of such an event is 2 days and 14 hours above the normal high pool (NHP) elevation of 1167.2 feet NAVD88. For reference, the MHP or MR PMF elevation of 1184.7 ft NAVD88 would require a hydrologic event with an AEP of 1/700,000. The MHP scenario would result in a tailwater of 1145.5 feet NAVD88 and a head differential of 39.2 feet. The duration of the MHP event is 2 days and 19 hours above the NHP elevation 1167.2 feet NAVD88.

Boring and testing data performed at DS19 and documented in Appendix A of Engineering Preliminary Design Report, Dam Site 19 found that the loess foundation soils at DS19 have dry densities and moisture contents above the threshold to exhibit any potential for collapse upon wetting. If there is a flaw at the contact between the embankment and abutment foundation material, such as a crack or poorly compacted layer, the blanket drain is designed to reduce seepage pressures and retain eroded material to stop the progression of CLE. Additionally, the original streambed channel will be lined with 3 feet of pervious fill to provide a controlled outlet for seepage.

Current construction practices will be used to construct DS19. All highly organic and objectionable foundation materials, such as rubbish, vegetation, roots, and muck will be removed from the foundation and old streambed channel. Foundation preparation, such as clearing, grubbing, scarifying, and recompacting the foundation surface, will be completed to ensure a good contact with the placed embankment fill. Finally, a 6-foot-deep by 10-foot wide-inspection trench with 1V on 2H side slopes will be excavated to the spillway crest elevation of 1,191.6 feet NAVD88 to break the continuity of the surface soil structure by replacing it with compacted impervious fill and to identify unforeseen soft areas near the ground surface that will require procedural changes during construction.

The length of the pipe (BEP) or crack/flaw (CLE) will vary based off the location of the potential failure mode (PFM). The width of the typical embankment section from the upstream toe to the downstream toe, 390 feet, was assumed for the purpose of the abbreviated SQRA, and results in a global gradient of 0.08 (1177.5 TAS EL – 1147.3 flaw EL / 390 feet progression) at TAS (AEP 1/1,600) and 0.10 (1184.7 MHP EL – 1147.3 flaw EL / 390 feet progression) at MHP (AEP 1/700,000). If the pipe or crack/flaw is in the abutments, the length of erosion progression could be less than 500 feet, but the head will decrease due to the higher elevation of the flaw, so gradients will be similar. The calculated global gradients are insufficient to initiate and progress CLE; however, they could initiate and potentially progress BEP in a uniform, fine sand. At DS19, borings that were completed for HDR Engineering's Engineering Preliminary Design Report of Dam Site 19 encountered glacio-fluvial sand-gravel deposits called the Red Cloud Formation near the interface of the Kansan till and the Nebraskan till, but did not encounter any pervious layers within the alluvial foundation. Therefore, PFM 14: Backwards Erosion Piping (BEP) of non-plastic Red Cloud Formation through the left abutment was carried forward as a risk-driving failure mode and PFM 16: BEP of alluvial foundation was excluded from consideration with the rest of the potential failure modes related to CLE through the foundation for all the above reasons.

5.6.3 RISK ASSESSMENT

A risk assessment was performed for the following potential failure modes judged to be risk drivers:

- PFM 14: Backwards Erosion Piping (BEP) of non-plastic Red Cloud Formation through the left abutment
- PFM 15: Spillway Erosion

The incremental risk (due to failure or breach) includes a consideration of both likelihood of failure and the incremental consequences. The likelihood of failure is a function of both the likelihood of the loading condition that could lead to the failure and the likelihood of failure given the loading condition. During the risk assessment, order-of-magnitude estimates were made for both likelihood of failure and incremental consequences (based on estimated consequences and the team's judgment) for each risk-driver potential failures mode. The evaluation of each risk-driver potential failure mode was documented as well as the team's confidence in the order-of-magnitude estimates. Confidence describes the potential impacts to the risk characterization and the decision to take action to reduce risk or reduce uncertainty.

Table 50. Confidence Categories

Confidence Level	Description
Low	The team is not confident in the risk characterization, and it is entirely possible that additional information would change the decision.
Moderate	The team is relatively confident in the risk characterization, but key additional information might possibly change the decision.
High	The team is confident in the risk characterization, and it is unlikely that additional information would change the decision.

5.6.4 RISK-DRIVER POTENTIAL FAILURE MODE DISCUSSION

5.6.4.1 PFM 14: Backwards Erosion Piping (BEP) of the Red Cloud Formation through the left abutment

5.6.4.1.1 Description

The DS19 reservoir is near the top of active storage (TAS) elevation of 1177.5 feet NAVD88 (AEP 1/1,600) with a tailwater elevation of 1134.6 feet NAVD88. A continuous layer of poorly graded, low C_u sand in the Red Cloud Formation underlies the dam between elevations 1055 and 1125 feet NAVD88. Exit gradients are sufficient to initiate backward erosion piping of the foundation sand, which exits unfiltered and undetected into the stilling basin. The loess and/or glacial till foundation materials overlying the sand seam hold a roof for pipe progression. The pipe progresses unimpeded to the upstream side of the dam, where overlying foundation materials collapse into the pipe and form a stope. The upstream foundation and embankment materials are continuously eroded and do not limit the progression. Intervention fails to stop the pipe from enlarging and the pipe collapses, leading to lowering of the crest and an uncontrolled loss of the pool. See Figure 41 for a plan view of the approximate failure path of BEP through the Red Cloud Formation at the left abutment.

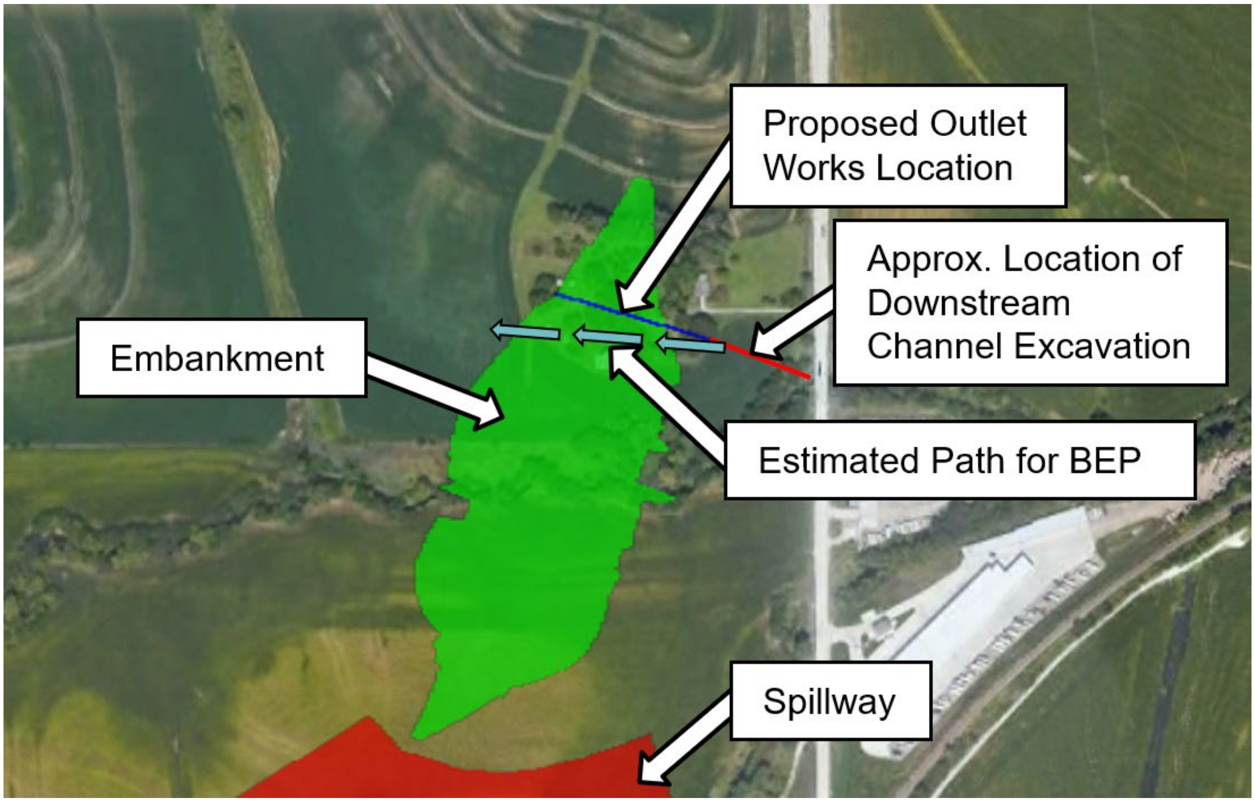


Figure 42. Plan View of the Approximate Failure Path of BEP

5.6.4.1.2 Background

A high gradient and long duration of loading are critical to progress BEP 400 feet from the stilling basin or downstream channel excavation to the upstream toe of the dam near the intake structure. The TAS pool loading (AEP 1/1,600) was determined to be the critical loading condition because it results in the greatest head differential of 42.9 feet assuming a reservoir elevation of 1177.5 feet NAVD88 and tailwater elevation of 1134.6 feet NAVD88 across the 400-foot-long seepage path. The duration of the TAS event is 2 days and 14 hours above the normal high pool (NHP) elevation of 1167.2 feet NAVD88.

The path of BEP through the Red Cloud Formation was discussed by the PA team. Initiation was most likely to occur in the stilling basin due to the deep excavation into the Kansan glacial drift formation at a depth of 1025 feet NAVD88. However, determining the most likely location of the stope to connect the reservoir through left abutment piping in the Red Cloud Formation was critical in determining the length of the seepage path. Although permeability tests were not performed on the loess, Peoria Loess can be relatively pervious depending on its silt and sand content, so the PA team determined that the stope would be more likely to occur at the upstream toe of the embankment in the Peoria Loess foundation material (see Figure 42). This seepage path is approximately 500 feet in length and results in a global gradient of 0.09 ft/ft.

The Red Cloud Formation lies near the interface of the Kansan till and the Nebraskan till and consists of loose to very dense, moist to wet, poorly graded sand (SP), silty sand (SM), and poorly graded sand with silt (SPM-SM). Each of the deeper borings that were completed for

HDR Engineering's Engineering Preliminary Design Report of Dam Site 19 encountered glacio-fluvial sand-gravel deposits varying from 30 feet to 60 feet in thickness and in elevation from 1055 to 1125 feet NAVD88. It is thought that the Red Cloud sand and gravel was deposited as outwash from streams flowing from southwestward-advancing glaciers. Gradations of the Red Cloud Formation were not obtained, so the susceptibility of the pervious material to BEP is unknown.

Overlying the Red Cloud Formation is Kansan glacial till and/or 10 to 50 feet of Peorian-Loveland Loess at the left abutment. The Kansan glacial till is a lean to fat clay mixed with occasional sand, gravel, and cobbles that is composed of plastic fines making it capable of holding a roof. Loess consists primarily of very soft to very stiff silty lean clay. Gradations or Atterberg limits of the Peorian-Loveland Loess were not obtained for the Engineering Preliminary Design Report, so there is uncertainty whether the Peorian-Loveland Loess would hold a roof. Testing information of the Peorian-Loveland Loess at the existing Papillion Creek Dam Sites would suggest that the loess is composed of plastic fines to make it capable of holding a roof.

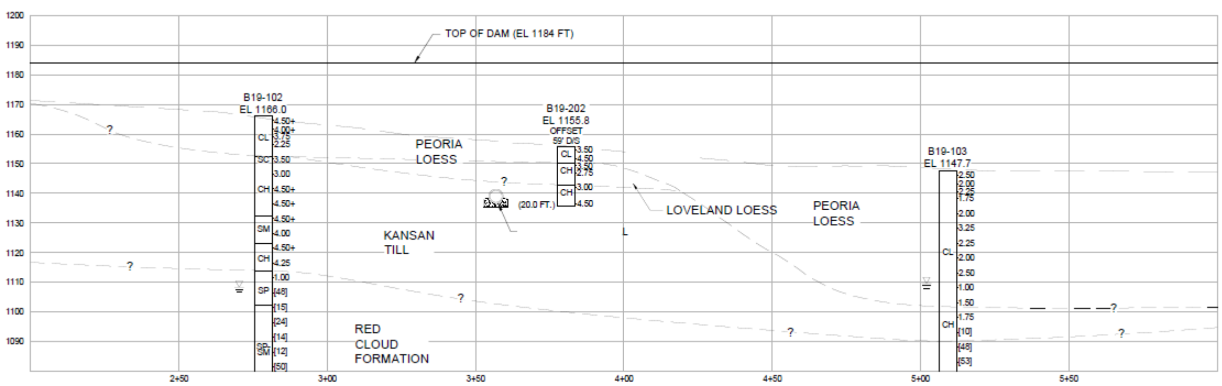


Figure 43. Geologic Profile of the Outlet Works along the Dam Centerline

Grading and design of the stilling basin and downstream channel excavation to connect the existing streambed to the outlet works was not fully developed for the General Reevaluation Report (GRR). Therefore, it was assumed for the abbreviated SQRA that the outlet works design for DS19 would be very similar to the 2018 preliminary design completed by HDR (see Figure 43) except for replacing the 48" diameter RCP outlet in HDR's design with a 72" diameter RCP to avoid upstream impacts to Highway 6. The stilling basin excavation from the preliminary design is at an approximate depth of 1125 feet NAVD88.

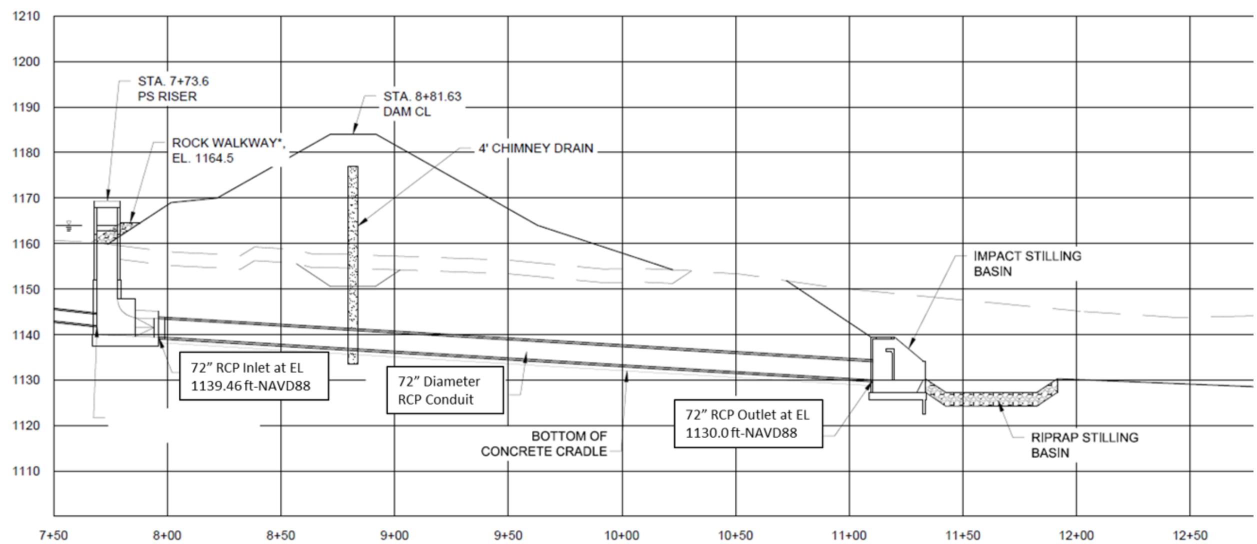


Figure 44. Outlet Works Design from HDR Engineering's 2018 Engineering Preliminary Design Report of Dam Site 19

A positive cutoff through the foundation, relief wells or other means of underseepage control have not been considered necessary during design to control seepage through the foundation due to the depth (40-60 feet) of the high permeability Red Cloud Formation. However, there is the potential for the Red Cloud Formation to be exposed by stilling basin excavation or for the groundwater pressure to exceed the weight of the overlying cohesive glacial till foundation material and cause heave to initiate BEP. The groundwater levels in the borings for the Engineering Preliminary Design Report varied from the top of the Red Cloud Formation to artesian pressures as much as 20 feet above the top of the Red Cloud Formation.

5.6.4.1.3 Intervention

Although detection of BEP may be possible, intervention is very unlikely. The Papio-Missouri River NRD will be the local sponsor for the project. Although they are responsible and reliable sponsors who fulfill their maintenance duties, closely monitor rain events, and perform surveillance during high pool events. The Papio-Missouri River NRD is the owner or local sponsor of over 100 miles of levee system and several dams within the Papillion Creek and confluence of the Platte and Missouri Rivers. A wide-spread flood event over the Papillion Creek Basin could affect a large percentage of these projects, making it difficult for the Papio-Missouri River NRD to closely inspect each of their projects.

If the local sponsor is able to perform surveillance during high pool events, internal erosion would likely no be detected since it would be under several feet of turbulent tailwater during the MHP loading. Depressions near the stilling basin or in the left abutment slope may indicate partial collapse of a progressing pipe. However, intervention such as construction of a filter or increasing tailwater to reduce the gradient is very unlikely due to the exit location in the outlet works plunge pool. It is therefore unlikely to stop the progression of BEP.

5.6.4.1.4 Likelihood of Failure

Table 51. Summary of Likelihood Factors for PFM 14

More Likely Factors	Less Likely Factors
• **The Red Cloud Formation is composed of poorly graded sands or silty sands susceptible to BEP and likely continuous from upstream to downstream.** • The overlying Kansan glacial till and Peorian-Loveland Loess foundation material can likely hold a roof. • Excavation for the stilling basin could potentially expose the Red Cloud Formation or at minimum decrease the thickness of the confining layer above the Red Cloud Formation, decreasing the factor of safety of blowout. • There is no positive cutoff through the foundation. Underseepage is controlled by the impervious alluvial, loess, and glacial till foundation materials. • BEP in the stilling basin may not be visible and would be under several feet of turbulent tailwater during MHP loading. • Artesian pressures as much as 20 feet above the top of the Red Cloud Formation were observed in the DS19 borings • Papillion Creek Dam Site 20 has similar foundation conditions with observed boils in the stilling basin during normal high pools.	• **Unlikely that there is an upstream exposure of the Red Cloud Formation, limiting the flow required for progression of BEP.** • **The global gradient (0.09 ft/ft at TAS) is likely less than the critical gradient required for progression of BEP.** • **The duration of the TAS event is approximately 2 days and 14 hours above the NHP, which makes it unlikely for steady-state gradients to develop and for BEP to progress to failure.**

Note: Key factors that drive the likelihood of failure are shown in **bold**.

Annual Probability of Failure: Between 1E-07 and 1E-06 (with and without intervention)

Rationale: A fine grained, cohesionless sand Red Cloud Formation underlies the dam between elevations 1055 to 1125 feet NAVD88. Excavation for the stilling basin to a depth of 1125 feet NAVD88 removes enough of the Kansan glacial till to cause blowout of the confining layer or exposure of the Red Cloud Formation. Critical loading (TAS, AEP 1/1,600) causes the pool to rise to a peak elevation of 1177.5 feet NAVD88. The overlying loess and glacial till foundation material is likely to hold a roof. However, the global gradient through the foundation (0.09 ft/ft at TAS) is likely less than the critical gradient required for progression of BEP, and the 2 day and 14 hour duration of the TAS loading above the NHP elevation of 1167.2 feet NAVD88 is likely insufficient to develop steady-state gradients to progress BEP the entire 500 foot seepage path. Due to the significance of the less likely factors including the thick, upstream impervious blanket above the Red Cloud Formation limiting the flow required for progression of BEP, the

PA team determined (with low confidence) that the likelihood of failure is between 1E-07 and 1E-06.

Confidence: Low

Rationale: The major source of uncertainty is the limited information associated with a project in the planning phase including limited testing information, no performance information, and large uncertainty in the hydrologic loading. The properties of the Red Cloud Formation are not well characterized in the Engineering Preliminary Design Report of Dam Site 19. Additional site characterization and lab testing of the Red Cloud Formation material would give the team greater confidence the susceptibility of the pervious material to BEP and development of critical gradients for blowout and initiation and progression of BEP. For these reasons, the team has low confidence in the assigned failure likelihood.

5.6.4.1.5 Incremental Life Loss

Average Incremental Life Loss: Between 0.1 and 1

Rationale: The modeled incremental life loss for a TAS failure was 0 for all warning times and exposure. Early detection may not be possible due to the turbidity of the tailwater, and the Papio-Missouri River NRD may not be able to dedicate daily and/or 24 hour surveillance until close to the TAS loading elevation of 1177.5 feet NAVD88. If there is surveillance at the site, visible/detectable distress such as depressions near the stilling basin during the formation of the pipe may be observed by the surveillance team, potentially giving the downstream population at risk (647 during the day and 582 at night) more than two hours of warning prior to breach. However, since surveillance during the TAS loading elevation is expected to be limited, the minimal warning time scenario was assumed.

Peak outflow for the modeled TAS breach scenario (elevation 1177.5 feet NAVD88) is 30,300 cfs with inundation depths ranging from 3 to 8 feet and velocities ranging from 3.5 to 7 feet per second immediately (0 to 3 miles) downstream of the dam. From 3 to 7 miles downstream of the dam, TAS breach modeled inundation depths range from 1 to 6.5 feet and velocities varied from 1 to 5 feet per second. However, the duration and flow of the pipe in the Red Cloud Formation before enough material is eroded to collapse and breach the embankment will likely lower the peak reservoir elevation below the TAS elevation. The primary consequence center is Omaha, NE which is located approximately two river miles downstream of the dam and is largely developed, consisting of a mix of residential and commercial/industrial structures along the creek bank. The inundation area directly downstream of DS19 and adjacent to South Papillion Creek is sparsely populated with only a few residential and industrial buildings.

The inundation depths and velocities are significant enough to cause loss of life, especially to those immediately (0 to 3 miles) downstream of the dam and minimal warning time is assumed due to expected limited surveillance up to the TAS loading elevation. However, the duration and flow of the pipe in the Red Cloud Formation before enough material is eroded to collapse and breach the embankment will likely lower the peak reservoir elevation below the TAS elevation of 1177.5 feet NAVD88, reducing the peak discharge, inundation depths, and velocities, and the inundation area immediately downstream of DS19 is sparsely populated with only a few residential and industrial buildings that could be quickly evacuated. Therefore, the best estimate incremental life loss for this failure mode is between 0.1 and 1.

Confidence: Low

Rationale: Limited consequence data was available in the planning stage of this project to assess the effects of piping of the left abutment Red Cloud Formation and eventual collapse of the overlying till, loess and embankment and its effect on the life loss estimate, or to develop and study the consequence mapping products to increase the team's confidence in the life loss estimates.

5.6.4.1.6 Recommendations

- Additional boring information, site characterization and soils lab testing of the Red Cloud Formation, especially at the location of the stilling basin, to determine its susceptibility to BEP and to develop critical gradients for blowout and initiation and progression of BEP.
- Require blanketing/filtering sand seams discovered in the stilling basin and downstream channel excavation.
- Armor the stilling basin to ensure erosion does not uncover the Red Cloud Formation. Design the armoring as a staged filter to retain Red Cloud Formation material.
- Installation of relief wells to reduce the foundation pressures at the stilling basin and downstream channel may be necessary based off the additional geotechnical information during PED.
- Install additional piezometers at the stilling basin and a line of piezometers across the embankment at the left abutment and tipped in the Red Cloud Formation to monitor gradients.

5.6.4.2 PFM 15: Spillway Erosion

5.6.4.2.1 Description

A significant inflow event occurs approaching the peak MHP elevation of 1184.7 feet NAVD88 (AEP 1/700,000) and the pool rises above the design spillway crest elevation of 1177.5 feet NAVD88 (AEP 1/1,600), initiating spillway flow. Velocities along the spillway exceed the allowable shear stress velocities for the in-situ grasses and the vegetation is stripped, initiating headcutting. The spillway flow duration enables headcut progression through the channel slope (0.029 ft/ft) and 200-foot-wide spillway control section. Intervention is unsuccessful due to an inability to access the spillway during MHP flows, and defensive measures (cut off wall or concrete sill) do not exist between the headcut and reservoir. The headcut advances and a connection with the upstream pool is established. Down cutting and mass wasting occurs, allowing breach enlargement and an uncontrolled release of the reservoir.

5.6.4.2.2 Background

The centerline of the proposed earth-cut, grass lined spillway is located about 350 feet southwest of the right abutment of the dam embankment and founded on Peorian-Loveland Loess and Kansan glacial till (see Figure 44). The Peorian-Loveland Loess is primarily consists of silty lean clays that are stiff when dry but become softer with increasing moisture content and vary in strength from very soft to very stiff; however, most of the loess is medium stiff to stiff and can be classified as moderately erodible according to Briaud (2008). Due to the more erodible nature of the loess foundation material, the base of the spillway founded on loess will be excavated to a minimum depth of 5 feet and backfilled with impervious fill consisting of highly plastic clays with liquid limits in excess of 40 percent to limit the potential for erosion. The Kansan glacial till consists of lean to fat clay mixed with occasional sand, gravel, and cobbles, and is considered stiff to very stiff. The current spillway design does not require the removal of the Kansan glacial till since it is a competent foundation material. There is no concrete sill or cutoff structure in the spillway. The spillway has a minimum 200-foot-long and 550-foot-wide earthen crest at design elevation 1177.5 feet NAVD88, and a 0.029 ft/ft slope downstream of the crest. The spillway channel is 550 feet wide with an approximate length of 1,072 feet from the beginning of the crest to the downstream end of the channel slope at its centerline. The spillway banks will be cut at 1V on 3H slopes. An existing stream channel at the end of the spillway has the potential to concentrate flow.

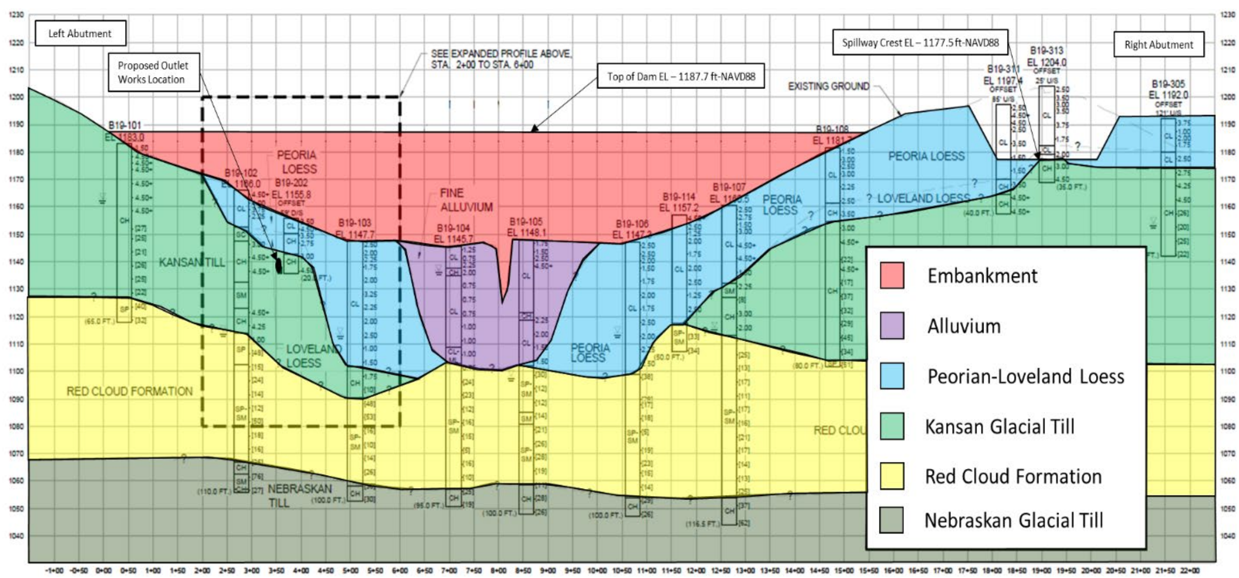


Figure 45. DS19 Geologic Section of the Spillway and Dam Embankment

The loading condition used to evaluate this potential failure mode was the MHP event (AEP 1/700,000), equivalent to the MR PMF. During the MHP event, the duration of flow (greater than one foot in depth) is estimated at 5 hours. The team also evaluated whether the performance of the outlet works significantly impacted the probability or duration of the MHP event. Damage or plugging of the outlet works resulting in inoperability would increase the MHP loading frequency to AEP 1/600,000 and prolong the duration of spillway flow to 12 hours. However, the outlet works is designed to be self-cleaning, and the Papio-Missouri River NRD continues to be one of the most active and responsive non-federal levee and dam sponsors in the Omaha District portfolio at maintaining their projects. Therefore, the team determined that the coincident inoperability of the 72" diameter RCP outlet conduit would increase the probability of spillway failure by no more than a half order of magnitude.

A HEC-RAS 2D model was created for the DS19 spillway to evaluate the effects of flow concentration during the MHP event. Figure 45 shows the maximum velocity plot produced by

HEC-RAS Mapper. Blue indicates lower velocity and dark red indicates higher velocity. Through the length of the spillway, an average velocity of 10 ft/s is observed, with maximum velocities of 15 ft/s directly downstream of the spillway crest.

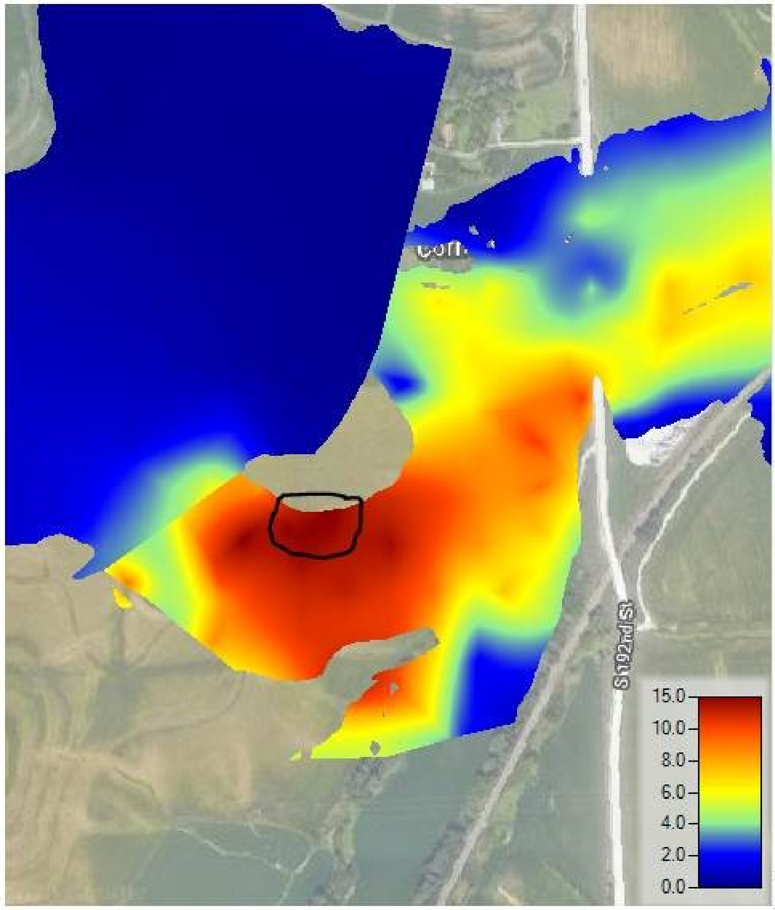


Figure 46. Maximum Velocity Plot Produced by 2D HEC-RAS Spillway Model.

Typical spillway vegetation at the existing Papillion Creek Dams consists of a grass mixture of perennial ryegrass, Primar Slender wheatgrass, Barton Western wheatgrass, and Pathfinder switchgrass. The specified grass mixture is similar to the vegetation types in red font color in Table 52, which include buffalo grass, Kentucky bluegrass, smooth brome, and blue grama. The team determined the velocity below which serious erosion would not occur in erosion resistant soils with Kentucky bluegrass cover and a 0 to 5% slope was 7 ft/sec based on SCS TP-61 and Chow 1959 in Table 52 which shows erosion characteristics of average, uniform stands of vegetation types. The maximum velocity computed in the HEC-RAS 2D model of 15 ft/sec is therefore enough to initiate and progress spillway erosion.

Table 52. Permissible Velocities Based on Vegetation Types (Published in TP-61)

Cover	Slope range[2] (Percent)	Permissible velocity (ft/sec)	
		Erosion resistant soils	Easily eroded soils
Bermuda grass	0-5	8	6
	5-10	7	5
	Over 10	6	4

Buffalograss Kentucky bluegrass Smooth brome Blue grama	0-5 5-10 Over 10	7 6 5	5 4 3
Grass mixture	0-5 5-10	5 4	4 3
Lespedeza sericea Weeping lovegrass Yellow bluestem Kudzu Alfalfa Crabgrass	[3]0-5	3.5	2.5
Common lespedeza[4] Sudangrass	[5]0-5	3.5	2.5

[1]Use velocities exceeding 5 ft/sec only where good cover and proper maintenance can be obtained.
[2]Do not use on slopes steeper than 10 percent, except for side slopes in a combination channel.
[3]Do not use on slopes steeper than 5 percent, except for side slopes in a combination channel.
[4]Annuals-used on mild slopes or as temporary protection until permanent covers are established.
[5]Use on slopes steeper than 5 percent is not recommended.

5.6.4.2.3 Intervention

Access to the dam embankment and spillway was not fully developed for the General Reevaluation Report (GRR) and will require additional real estate from private landowners. For the purposes of the abbreviated SQRA, it was assumed that the government would be able to purchase real estate for access roads unaffected by tailwater conditions during the MHP for heavy construction equipment access. Access roads for maintenance and emergency situations are critical to address TRG3 and reduce the risk of the project (TRG4).

The Papio-Missouri River NRD is the owner or local sponsor of over 100 miles of levee system and several dams within the Papillion Creek and confluence of the Platte and Missouri Rivers. A wide-spread flood event over the Papillion Creek Basin could affect a large percentage of these projects, making it difficult for the Papio-Missouri River NRD to closely inspect each of their projects to increase the likelihood of ample warning. Equipment for automated pool readings to alert the local sponsor if the pool nears the spillway crest and communication with USACE to provide additional engineering assistance will be critical to intervention. Intervention may be possible at lower discharges but would not be feasible during the MR PMF event due to the velocity (up to 15 ft/sec) of spillway flows and width (550 feet) of the spillway.

5.6.4.2.4 Likelihood of Failure

Table 53. DS19 Summary of Likelihood Factors for PFM 15

More Likely Factors	Less Likely Factors
• **Maximum flow velocity of 15 ft/s is high enough to strip the vegetal cover and initiate and progress headcutting in the spillway.** • An existing stream channel at the end of the spillway will concentrate flow.	• **MR PMF loading is infrequent (AEP 1/700,000).** • The moderately erodible loess foundation will be over-excavated and replaced with 5 feet of high plasticity clay with low erodibility.

More Likely Factors	Less Likely Factors
• **There is no concrete control sill or cutoff structure to prevent headcutting.** • Plugging of the outlet works would increase the MHP loading frequency to AEP 1/600,000 and prolong the duration of spillway flow above one foot in depth to 12 hours. The team determined that plugging of the 72" RCP would increase the probability of spillway failure by no more than a half order of magnitude.	• **The duration of spillway flow above one foot in depth is relatively short (5 hours total during the MHP event).** • **Centerline length of spillway is 1,072 feet, which would require an erosion rate of approximately 214 feet/hour to progress to failure.**

Note: Key factors are shown in **bold**.

Annual Probability of Failure: Between 3E-09 and 3E-08 (with and without intervention)

Rationale: Flow velocities during the MR PMF event (AEP 1/700,000) as high as 15 ft/s are sufficient to strip vegetation and initiate and progress headcutting of the Kansan glacial till, the 5 foot layer of erosion resistant, high plasticity clay, and the underlying moderately erodible loess. The total duration of flow more than one foot in depth is approximately 5 hours. There is no upstream control sill or cutoff structure to prevent a full breach. However, the short flow duration, 1,072 foot failure path length, and proactive removal of the more erodible exposed loess material during construction makes it unlikely that headcutting and/or down-cutting would be sufficient to progress the entire spillway length to breach the crest. Due to the significance of the less likely factors, the PA team determined (with moderate confidence) that the likelihood of failure is between 3E-08 and 3E-07.

Confidence: Low

Rationale: The major source of uncertainty is the limited information associated with a project in the planning phase including limited site characterization and testing information of the spillway foundation materials, no performance information, and large uncertainty in the hydrologic loading. Additionally, there is considerable uncertainty in the duration of flow required to breach the spillway; the processes of erosion are unpredictable and can vary significantly depending on material properties, flow concentration, and spatial variation of velocities.

5.6.4.2.5 Incremental Life Loss

Incremental Life Loss: Between 0.1 and 1

Rationale: The modeled incremental life loss for a MHP spillway failure was 0 for all warning times and exposure. The spillway breach model discussed in Section 5.5 Consequences calculated an incremental discharge of 60 cfs which correlates to a small incremental population at risk of 49 during the day and 6 at night. Additionally, the team was confident that daily and/or 24-hour surveillance would be required at the dam site and the downstream population would be warned once the spillway begins flowing at the TAS loading condition. With constant surveillance at the dam site during the MHP loading condition, it is very likely that the downstream population will be warned a second time when there is visible erosion of the spillway to ensure that the PAR is mobilized and evacuated from the inundation area. Therefore, the best estimate incremental life loss for this failure mode is between 0.1 and 1.

Confidence: Low

Rationale: Limited consequence data was available in the planning stage of this project and several assumptions were made in the model for MHP spillway failure as discussed in Section 5.5 Consequences.

5.6.4.2.6 Recommendations

- The team considered whether a control sill or cutoff structure is necessary to reduce the risk of a headcut advancing through the crest of the spillway; however, the low probability of the failure mode made the cost not necessary.
- Prioritize routine maintenance of the trash rack on the intake of the outlet works in the O&M Manual to ensure the design capacity of the outlet works is maintained to prevent increased frequency and duration of spillway flow.
- Any proposed recreation, utility, or other features submitted through the 408 process within the spillway will be thoroughly reviewed prior to approval. The inclusion of such features is likely to increase the erosion potential of the spillway due to increased turbulence and localized velocities caused by knickpoints.

6 REFERENCES

USACE. 1975. Specific Design Memorandum No. MPC-33, Site 10, Papillion Creek and Tributaries, Nebraska. U.S. Army Corps of Engineers. Northwestern Division Omaha District. May 1975

HDR. 2018. Engineering Preliminary Design Report, Dam Site 19 and Associated Improvements, West Papillion Creek Subwatershed. Prepared for the Papio-Missouri Natural Resources District. April 2018.

FYRA. 2018. Papillion Creek Watershed Hydrologic Analysis. Final Submittal. Prepared for the Papio-Missouri Natural Resources District. October 2018.

HEC. 2020. HEC-HMS version 4.4beta. February 2020.

NOAA. 2020. NOAA Atlas 14 Precipitation Depths. National Oceanic Atmospheric Administration. Accessed October 2020 at: https://hdsc.nws.noaa.gov/hdsc/pfds/.

USACE. 2016. Papillion Creek Dam Site 16-Standing Bear Lake (NE01065) Right Bank Tributary to Big Papillion Creek, Nebraska Periodic Inspection No. 13 Periodic Assessment No. 01). U.S. Army Corps of Engineers. Northwestern Division Omaha District.

USACE. 2017. Papillion Creek Dam Site 11-Cunningham Lake (NE01518-AS1) Knight Creek, Nebraska Periodic Inspection No. 13 Periodic Assessment No. 01). U.S. Army Corps of Engineers. Northwestern Division Omaha District.

USACE. 2018a. Papillion Creek Dam Site 18-Zorinsky Lake (NE02185) Boxelder Creek, Nebraska Periodic Inspection No. 11 Periodic Assessment No. 01). U.S. Army Corps of Engineers. Northwestern Division Omaha District.

USACE. 2018b DRAFT. Papillion Creek Dam Site 20-Wehrspann Lake (NE01882) Unnamed Tributary to South Papillion Creek, Nebraska Periodic Inspection No. 13 Periodic Assessment No. 01). U.S. Army Corps of Engineers. Northwestern Division Omaha District.

USACE. Hydrologic Engineering Center (2018). HEC-RAS 5.0.7 [Computer Software]. Davis, CA

USACE. Hydrologic Engineering Center. 2D Modeling User's Manual, Version 5.0. February 2016

USACE. Modeling, Mapping, and Consequences Production Center. Technical Manual for Dams, Modeling, Mapping, and Consequences. March 2020

USACE. Hydrologic Engineering Center. Using HEC-RAS for Dam Break Studies, Training Document No. 39. August 2014

USACE. Omaha District, Hydrologic Engineering Branch. Spillway Erosion Study Papillion Creek Dam Site 20. October 2007