Appendix K

Life Safety Risk Assessment

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1. LIFE SAFETY RISK ASSESSMENT

The United States Army Corps of Engineers (USACE) recognizes that risks to human life are a fundamental component of all flood risk management studies and must receive explicit consideration in the planning process. Current USACE guidance (PCB 2019-4, ECB 2019-03, ECB 2019-15, and the January 2021 Policy Directive – Comprehensive Documentation of Benefits in Decision Documents) on risk assessments in planning studies specifies how studies should be performed on new or existing dams and levees. This risk assessment's purpose is to make sure that the feasibility level designs follow the four Tolerable Risk Guidelines:

a. TRG 1 – Understanding the Risk

b. TRG 2 – Building Risk Awareness

c. TRG 3 – Fulfilling Daily Responsibilities

d. TRG 4 – Actions to Reduce Risk

While all these guidelines are important, TRGs 1 and 4 are critical to Planning studies. The risk assessment below is the first step to Understanding the Risk (TRG 1) of the proposed features and makes recommendations on changes that could Reduce the Risk (TRG 4).

An additional benefit of the risk assessment is the identification of areas of concern in the proposed design that may require extra attention during design or changes to design to ensure minimal risk to the public.

For this study, the life safety risk consideration was accomplished by performing an abbreviated Life Safety Consequence Assessment and a feasibility screening level Potential Failure Mode Analysis.

As part of this life safety analysis the primary alternative being evaluated is Detention Basin 4 (DB4). The main General Revaluation Report (GRR) covers the other alternatives and provides the context for this specific aspect of the study. The consequence evaluation deals with both the with and without project condition for the DB4 alternative as well as delves into the breach and non-breach scenarios for the with project condition.

2. PROJECT SUMMARY - CURRENT LEVEL OF DESIGN

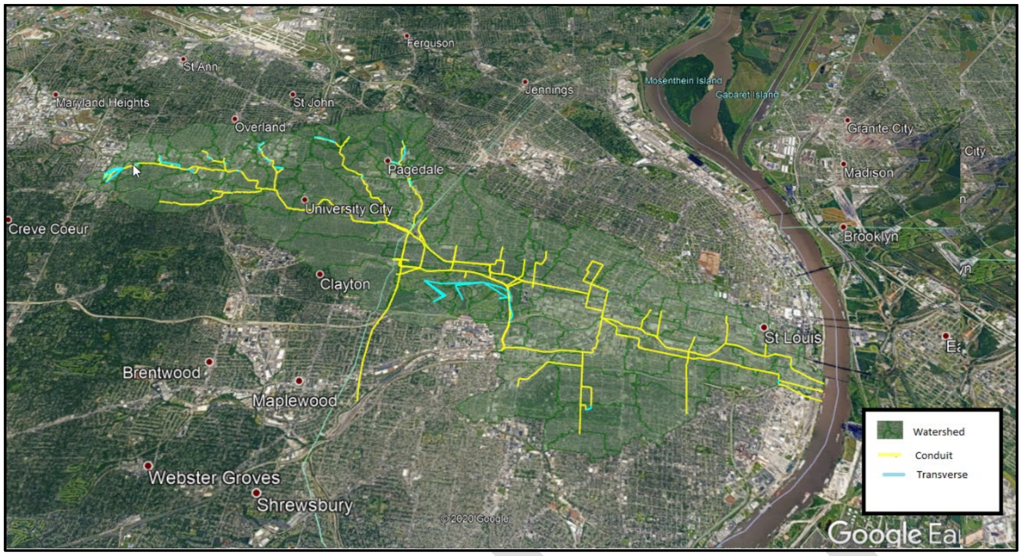
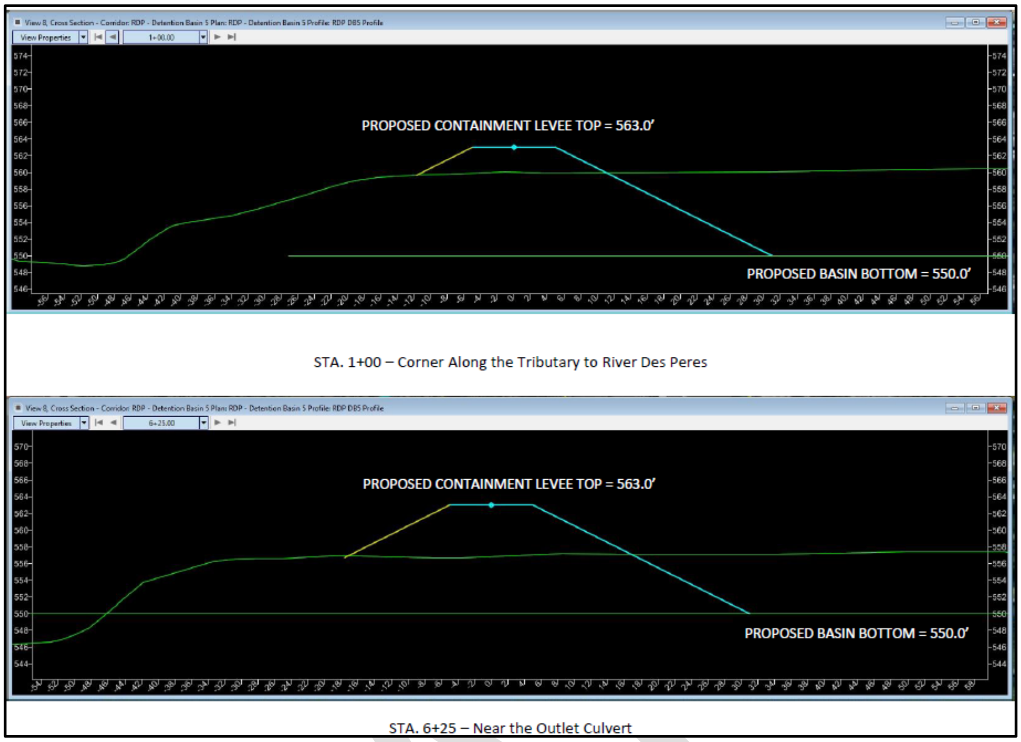


Figure 1: PCSWMM Geometry for the River Des Peres Watershed.

The main body of the GRR covers in greater detail how USACE and the Non-Federal sponsor have arrived at evaluating the DB4 alternative and provides greater context for the alternative formulation and final arrays that have been developed.

As of the date of this assessment, the level of design for the Detention Basin is substantially less than what is typically encountered in a 35% design submittal. Typically, for a GRR or feasibility study, there are preliminary levels of site-specific subsurface information as well as preliminary design and analysis (i.e., stability, seepage, foundation design considerations) to deterministically evaluate the structure's reliability. Traditionally, a project soils report (i.e., geotechnical report) is in draft form at the 35% level of design subject to further refinement in coordination with civil, structural, and other engineers. For the River Des Peres Detention Basin 4 (DB4) the soils report is basically non-existent, except for the regional level understanding of the geology that is included in the current draft of the GRR. Additionally, there is some general understanding of the near surface soils based on US Department of Agriculture – Natural Resource Conservation Survey (USDA – NRCS) soils survey maps. Figure 1 shows the PCSWMM geometry for the River Des Peres watershed (see Appendix A for more information). Design is currently at the conceptual level at approximately 5 - 10% level of completion with plan view CADD alignments and crude typical cross-sections sufficient to estimate ROM quantities. Figure 2 shows an example of actual sections of the design that exemplify the infancy of the design.



STA. 1+00 – Corner Along the Tributary to River Des Peres

STA. 6+25 – Near the Outlet Culvert

Figure 2: Example Design Cross Section from DB4.

3. CONSEQUENCES

For the selected and locally preferred plan that has the construction of DB4 as the structural option, the worst-case breach condition was examined using HEC-RAS version 5.0.7. The worst-case condition hydraulically would be if a breach formed at the outlet pipe during the peak stage in the detention basin during the higher magnitude frequency events. Though this would breach before the highest head difference between the detention basin interior and the river, this would yield impacts when the river was already out of bank.

In regard to the breach characteristics, degradation progression would have normally been used, but because of the short duration of flooding on the River Des Peres the containment embankment would not adequately degrade. For this analysis, a complete blow-out breach with a width of 75 feet at the outlet pipe was simulated yielding the worst possibility of failure. Table 1 tabulates the results of the breach analysis as compared to the with and without project conditions. (The breach hydraulic analysis is discussed in greater detail in Appendix A). Figure 2 shows cross-section 20402.2. Table 2 shows average flood depths and velocities on structures.

Table 1. Water Surface Elevations Downstream of DB4

AEP	Water Surface Elevation NAVD 88 (feet) XS 20402.2			Proposed DB4 Difference	
	Existing	Proposed DB4 Alternative	Proposed DB4 Breached	Difference with Existing	Difference with Breach
0.2%	562.05	560.1	560.58	1.95	0.48
0.5%	560.53	557.05	558.01	3.48	0.96
1%	558.95	556.05	556.05	2.9	0
2%	557.03	555.36	555.36	1.67	0
4%	556.72	555.11	555.11	1.61	0
10%	555.66	554.02	554.02	1.64	0

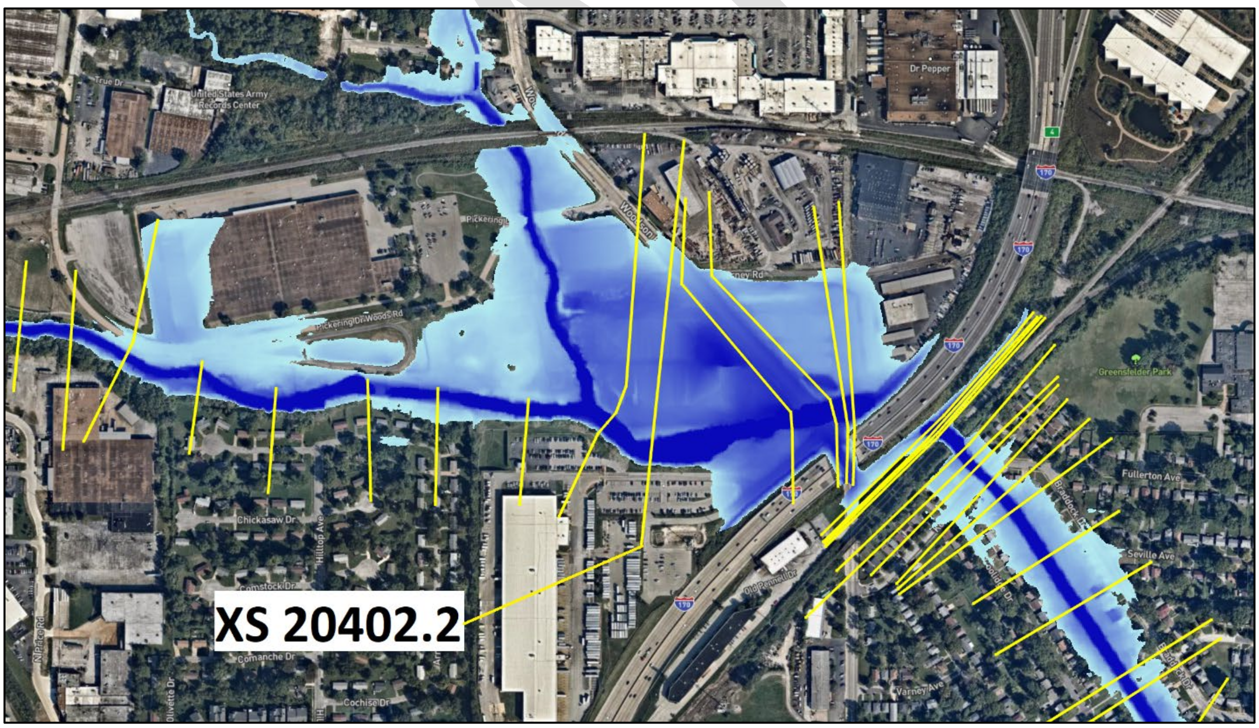


Figure 3. Cross-Section 20402.2

Table 2. Depths and Velocities on Structures, .2% AEP Event

Scenario	Average Depth (ft)	Average Velocity (ft/s)
Existing Condtion	1.94	0.54
Non-Breach	1.72	0.50
Breach	1.75	0.51

3.1. Population at Risk

Population at risk (PAR) is defined as the number of people within an affected area that would be subject to inundation during a flood hazard event. Estimates of PAR were generated using the National Structure Inventory 2.0 (NSI 2.0) for breach and non-breach inundation scenarios, as well as the existing without-project condition. The NSI 2.0 population data was developed in 2018 for both day and night population. The US Census Bureau estimates there has been no change in population in St. Louis County. Structure placement was verified by aerial imagery and windshield survey. The mix of residential and non-residential structures is illustrated in Figure 4. The estimated PAR by event is summarized in Table 3.

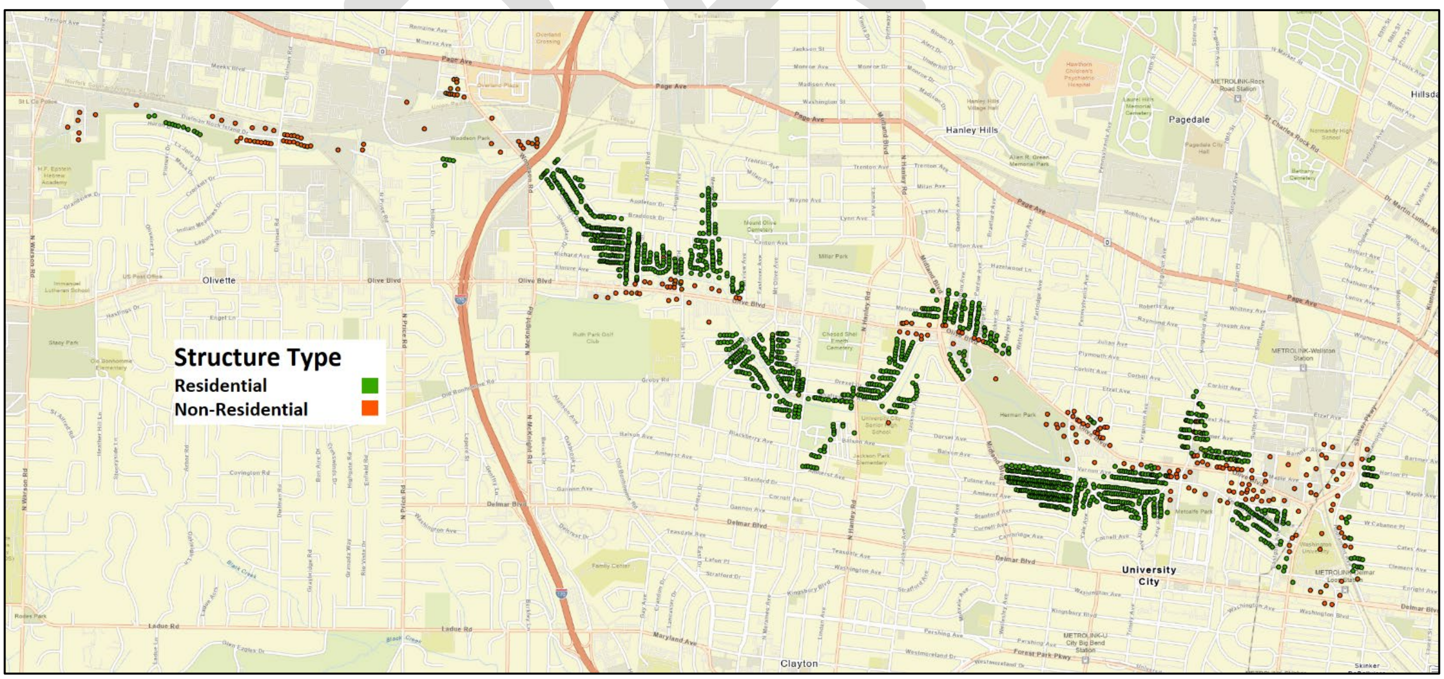


Figure 4. Structure Type

Table 3. Population and Structures at Risk.

Population and Structures at Risk				
Scenario	AEP Event	Structures Inundated	PAR Day	PAR Night
Without-Project	.2% AEP Event	980	3200	3100
	.5% AEP Event	850	2900	2700
	1% AEP Event	720	2500	2300
Breach	.2% AEP Event	950	3200	3000
	.5% AEP Event	800	2800	2600
	1% AEP Event	650	2400	2200
Non-Breach	.2% AEP Event	930	3200	2900
	.5% AEP Event	780	2800	2500
	1% AEP Event	640	2300	2100

3.2. Life Loss Model Parameters

The consequence modeling was conducted with the Loss of Life Simulation software, LifeSim 2.0.1. To determine the percentage of population at risk (PAR) within a structure that is warned and mobilized over time, several parameters are used to estimate the probable values of warning and mobilization percentages as time passes. These include when warnings will be issued (hazard identification and communication delay), how long they will take to become effective (warning issuance and warning diffusion), and the rate at which PAR will mobilize in response (mobilization). Figure 5 shows the warning and response timeline utilized in LifeSim.

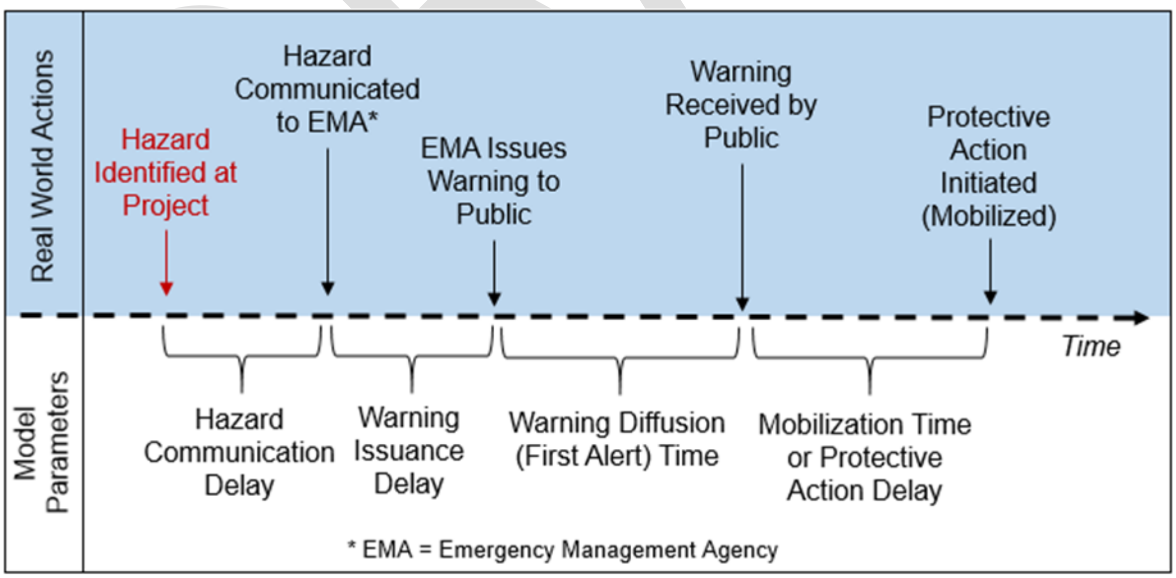


Figure 5. Warning and Response Timeline.

Relative Hazard Identification

The Hazard Identification time is the time at which a hazard is identified (dam breach or major flooding) relative to when it occurs (actual breach time). The standard operating procedures from the USACE Mapping, Modeling and Consequence (MMC) production center uses two different warning scenarios with different distributions 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 the event and at the time of the event (-2 to 0 hours). Ample warning scenarios have the hazard identification relative time set as a uniform distribution between 6 hours prior to the event and 2 hours prior to the event (-6 to -2 hours). The Relative Hazard Identification Times used for this study are summarized in Table 4.

Table 4. Relative Hazard Identification Times.

Warning Scenario	Distribution Type	Minimum (hours)	Maximum (hours)
Minimal	Uniform	-2	0
Ample	Uniform	-6	-2

Hazard Communication Delay

The Hazard Communication Delay is the time that it would take from when the hazard is identified to when the emergency managers are notified. For example, if a breach occurs when no one is observing the project then the emergency managers would be notified after the hazard is identified. This delay may be short if emergency managers are aware of an issue with the dam and there is cell service, or this may be longer if there is difficulty in reaching emergency managers downstream. The hazard communication delay is set as a uniform distribution between 0.01 hours and 0.5 hours.

Warning Issuance Delay

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.

For this model, Preparedness Unknown was selected from among LifeSim's preset distributions for Warning Issuance Delay. In the absence of a Mileti & Sorenson interview, the distribution with the greatest uncertainty was chosen. This means the time it takes the emergency managers to issue the first evacuation order is most likely within 30 minutes of receiving the notification of an imminent hazard from the official monitoring storm activity. The Preparedness Unknown curve is shown in Figure 6.

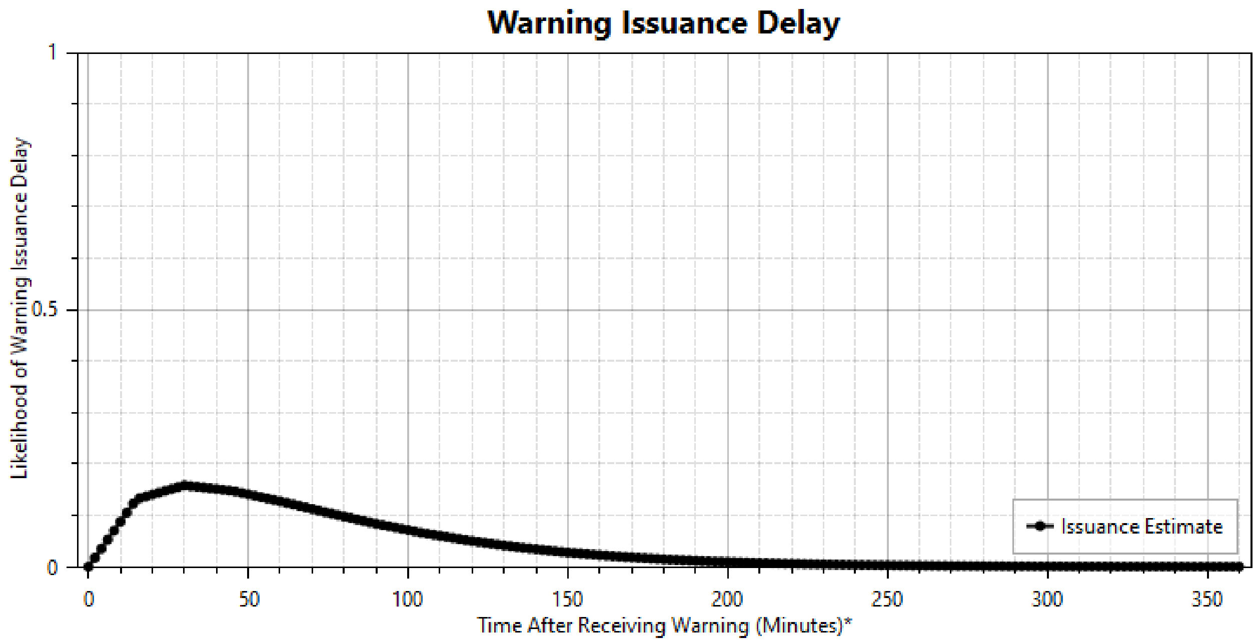


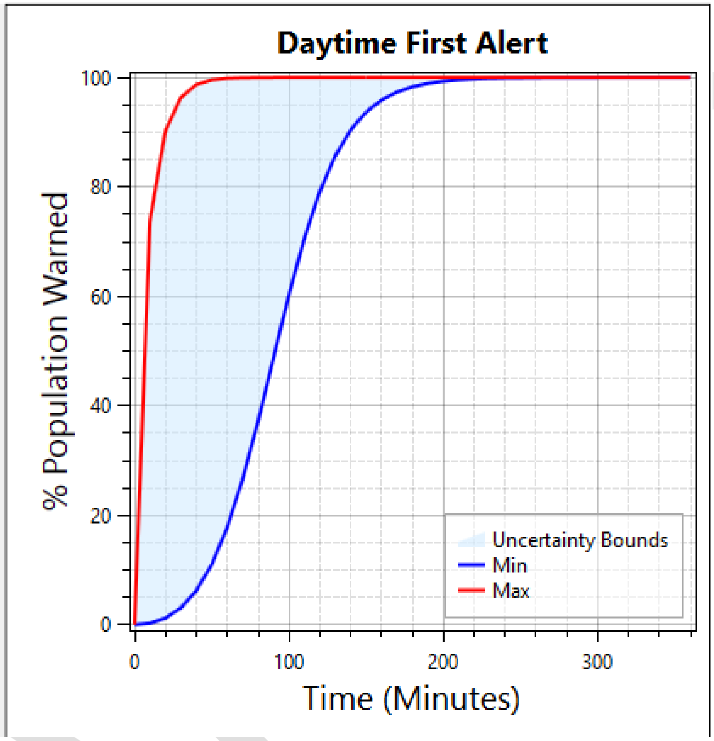
Figure 6. Warning Issuance Delay.

Warning Diffusion (First Alert)

Warning diffusion is the time between a first alert or warning issuance and the time that PAR receive that warning. It is primarily dependent on what type of warning systems and procedures are in place and the ability of the population to receive the warning via those systems. The warning diffusion curve represents the efficiency of a warning after it is issued.

The Warning Diffusion curves in Figure 7, set to LifeSim's Unknown/Unknown preset due to the absence of a Mileti & Sorenson Interview, provide a distribution for warning dissemination at 2am and 2pm. The daytime diffusion curve represents the percentage of the population which will receive a first alert warning over time during daytime hours after the warning is issued. The nighttime diffusion curve represents the percentage of the population which will receive a first alert warning over time during nighttime hours after the warning is issued.

Time (Minutes)	Minimum % Warned	Maximum % Warned
0	0	0
10	0.28	73.75
20	1.21	90.199997
30	3.06	96.32
40	6.19	98.68
50	10.96	99.540001
60	17.700001	99.839996
70	26.49	99.940002
80	37.060001	99.980003
90	48.639999	99.989998
100	60.130001	100
110	70.489998	100
120	79.040001	100
130	85.589996	100
140	90.330002	100
150	93.629997	100



Time (Minutes)	Minimum % Warned	Maximum % Warned
0	0	0
10	0.18	39.889999
20	0.73	72.139999
30	1.73	83.360001
40	3.24	89.400002
50	5.33	93.360001
60	8.07	95.910004
70	11.51	97.510002
80	15.67	98.489998
90	20.52	99.089996
100	26.030001	99.449997
110	32.07	99.669998
120	38.490002	99.800003
130	45.119999	99.879997
140	51.740002	99.93
150	58.16	99.959999

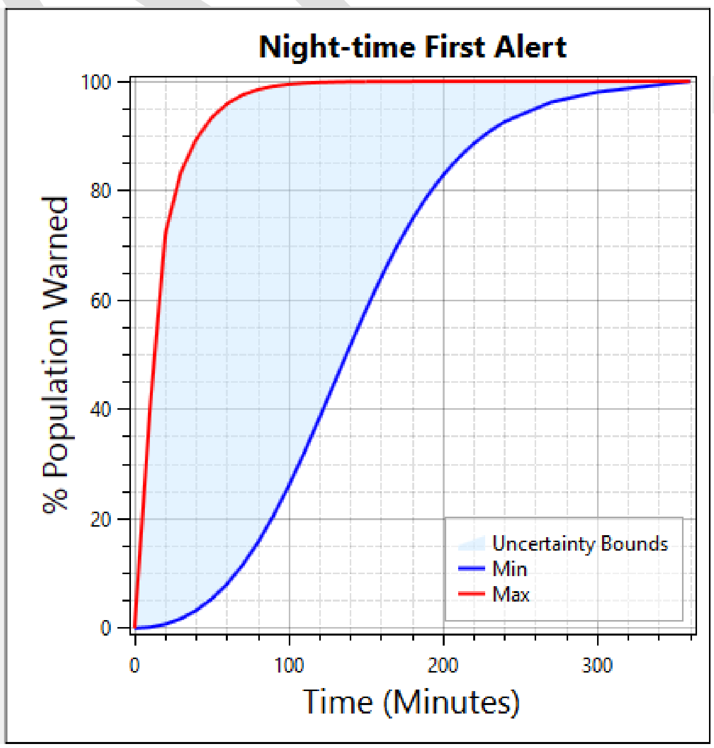


Figure 7. Warning Diffusion Curves for Daytime and Night-Time First Alert.

Protective Action Initiation

Protective Action Initiation (PAI) is the rate at which PAR take action after receiving an evacuation order (warning) (Figure 8). Unlike the warning diffusion curves, the PAI curves include a perception element as well. The perception element describes the relative awareness of the PAR. Preparedness: Unknown/Perception: Unknown was again selected for this study area.

Time (Minutes)	Minimum % Mobilized	Maximum % Mobilized
0	0	0
10	0.07	14.79
20	0.14	47.27
30	0.31	76.309998
40	0.54	92.269997
50	0.85	98.169998
60	1.21	99.68
70	1.64	99.949997
80	2.12	99.949997
90	2.66	99.949997
100	3.26	99.949997
110	3.9	99.949997
120	4.59	99.949997
180	9.48	99.949997
240	14.99	99.949997
300	20.32	99.949997
360	24.870001	99.949997

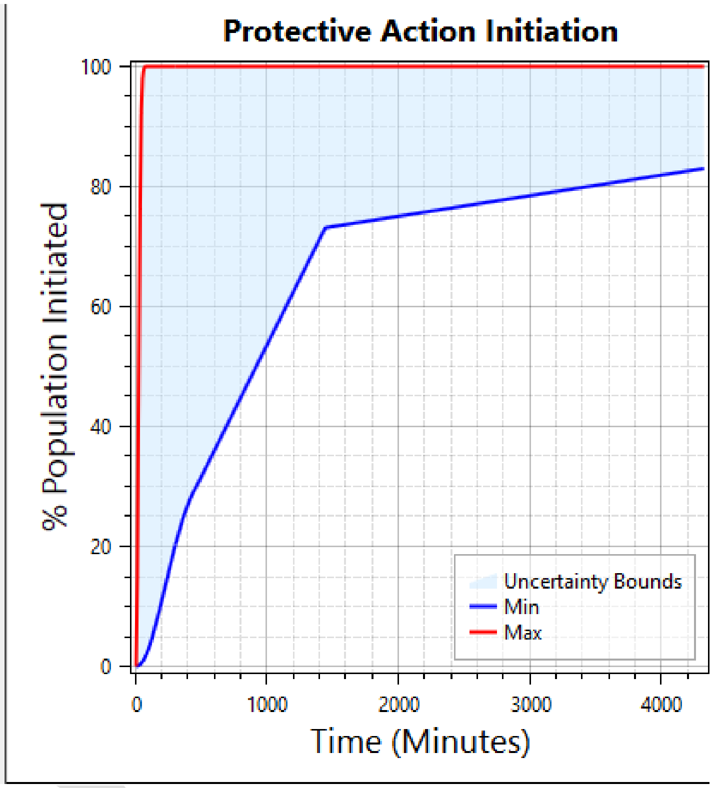


Figure 8. Protective Action (PAI) Curve.

3.3. Life Loss

Existing Without-Project Condition

Estimates of life loss were generated using LifeSim for without-project, breach, and non-breach inundation scenarios. LifeSim utilizes Monte Carlo uncertainty analysis, and 1,000 Monte Carlo iterations were run for each scenario. The estimated life loss totals in the existing without-project condition by warning time are summarized below in Table 5 and Table 6. The life loss statistics for each run are shown in the Figure 9 and Figure 11 box-and-whisker plots. The ranges only reflect the uncertainty parameters for life loss as modeled in the LifeSim scenarios and do not include uncertainties for the breach parameters or other hydraulic/hydrologic factors. A life loss heat map is shown in Figure 10.

Minimal Warning

Table 5. Without Project Life Loss with Minimal Warning.

Minimal Warning - Without Project Life Loss					
Scenario Name	Structures Inundated	Population at Risk		Median Total Life Loss	
		Day	Night	Day	Night
.2% AEP Event	980	3200	3100	7	8
.5% AEP Event	850	2900	2700	4	5
1% AEP Event	720	2500	2300	2	2

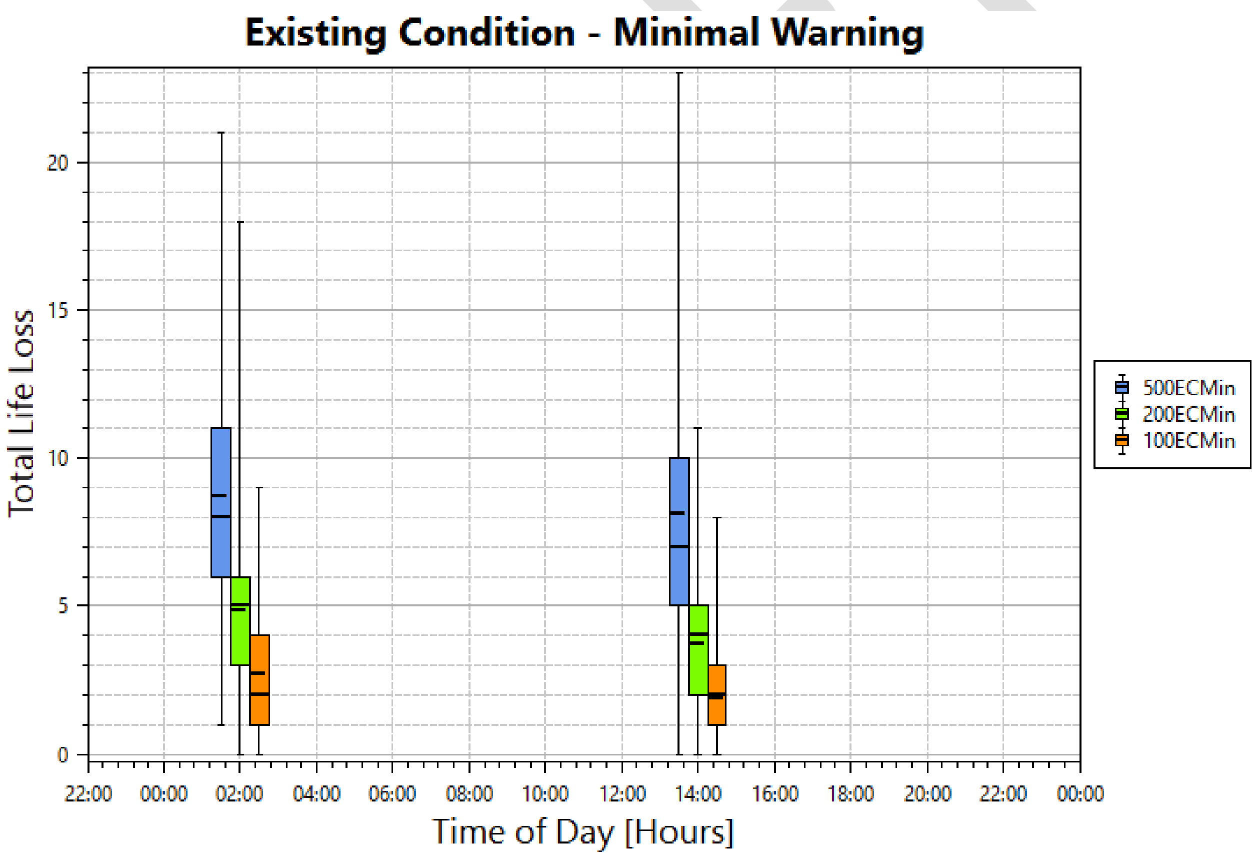


Figure 9. Life Loss with Minimal Warning in the Existing Condition.

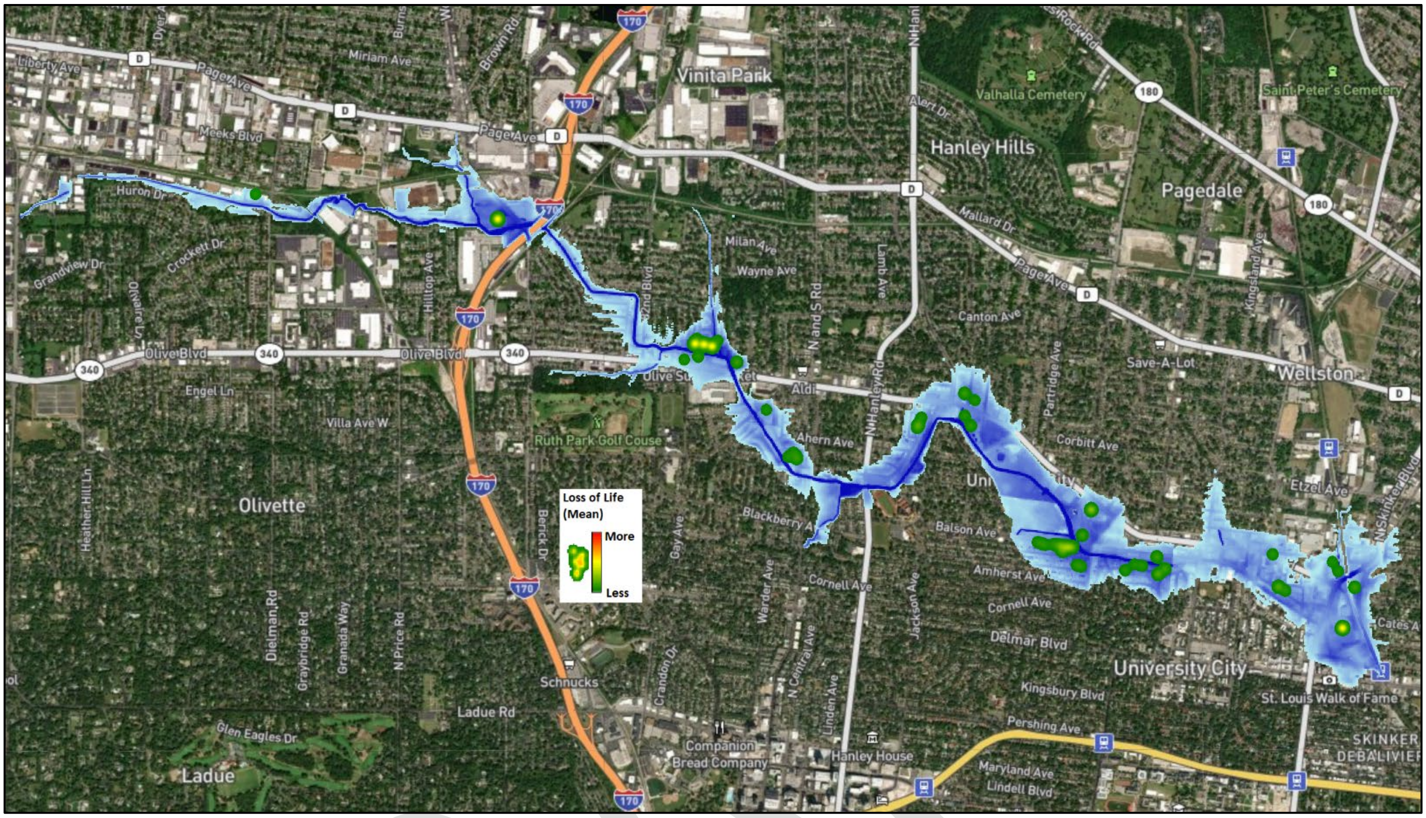


Figure 10. Heat Map - Life Loss with Minimal Warning in the Existing Condition, .2% AEP Event.

Ample Warning

Table 6. Without Project Life Loss with Ample Warning.

Ample Warning - Without Project Life Loss					
Scenario Name	Structures Inundated	Population at Risk		Median Total Life Loss	
		Day	Night	Day	Night
.2% AEP Event	980	3200	3200	5	6
.5% AEP Event	850	2900	2800	2	3
1% AEP Event	720	2500	2400	1	2

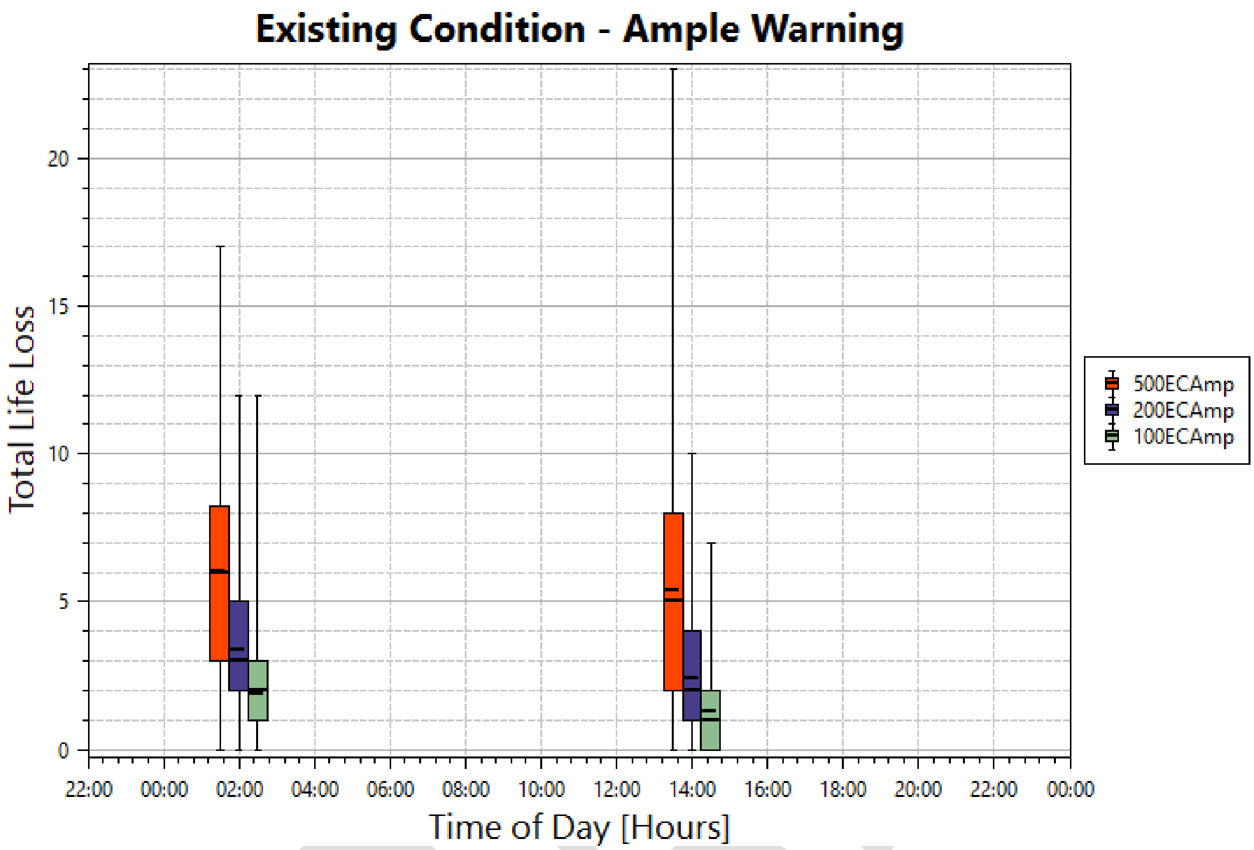


Figure 11. Life Loss with Ample Warning in the Existing Condition.

With-Project Condition – Non-Breach Scenario

The estimated life loss totals in the non-breach scenario by warning time are summarized in Table 7, Table 8, Figure 12, and Figure 14. A life loss heat map is in Figure 13.

Minimal Warning

Table 7. Life Loss and PAR with Minimal Warning, With Project, Non-Breach.

Minimal Warning - Non-Breach Life Loss					
Scenario Name	Structures Inundated	Population at Risk		Median Total Life Loss	
		Day	Night	Day	Night
.2% AEP Event	930	3200	2900	5	7
.5% AEP Event	780	2800	2500	2	3
1% AEP Event	640	2300	2100	1	2

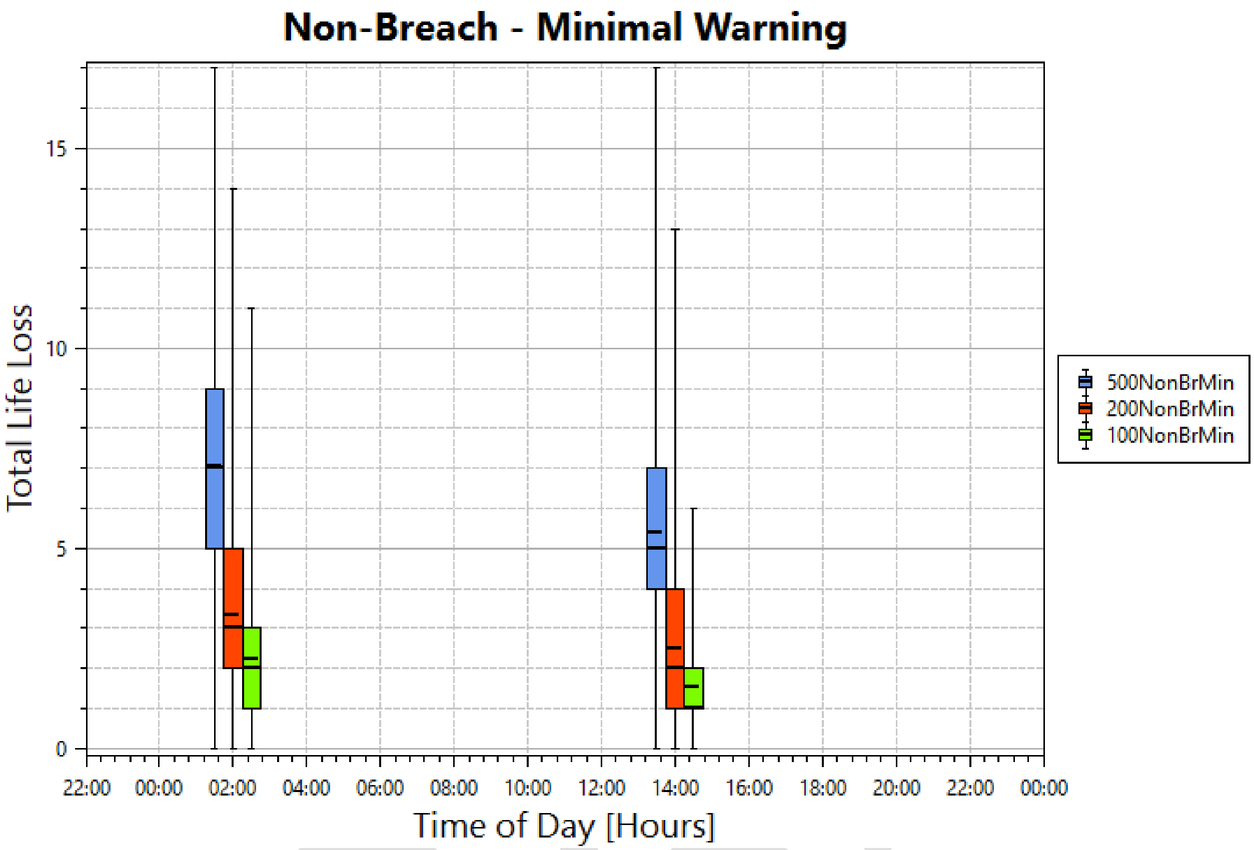


Figure 12. Life Loss with Minimal Warning, With Project, Non-Breach.

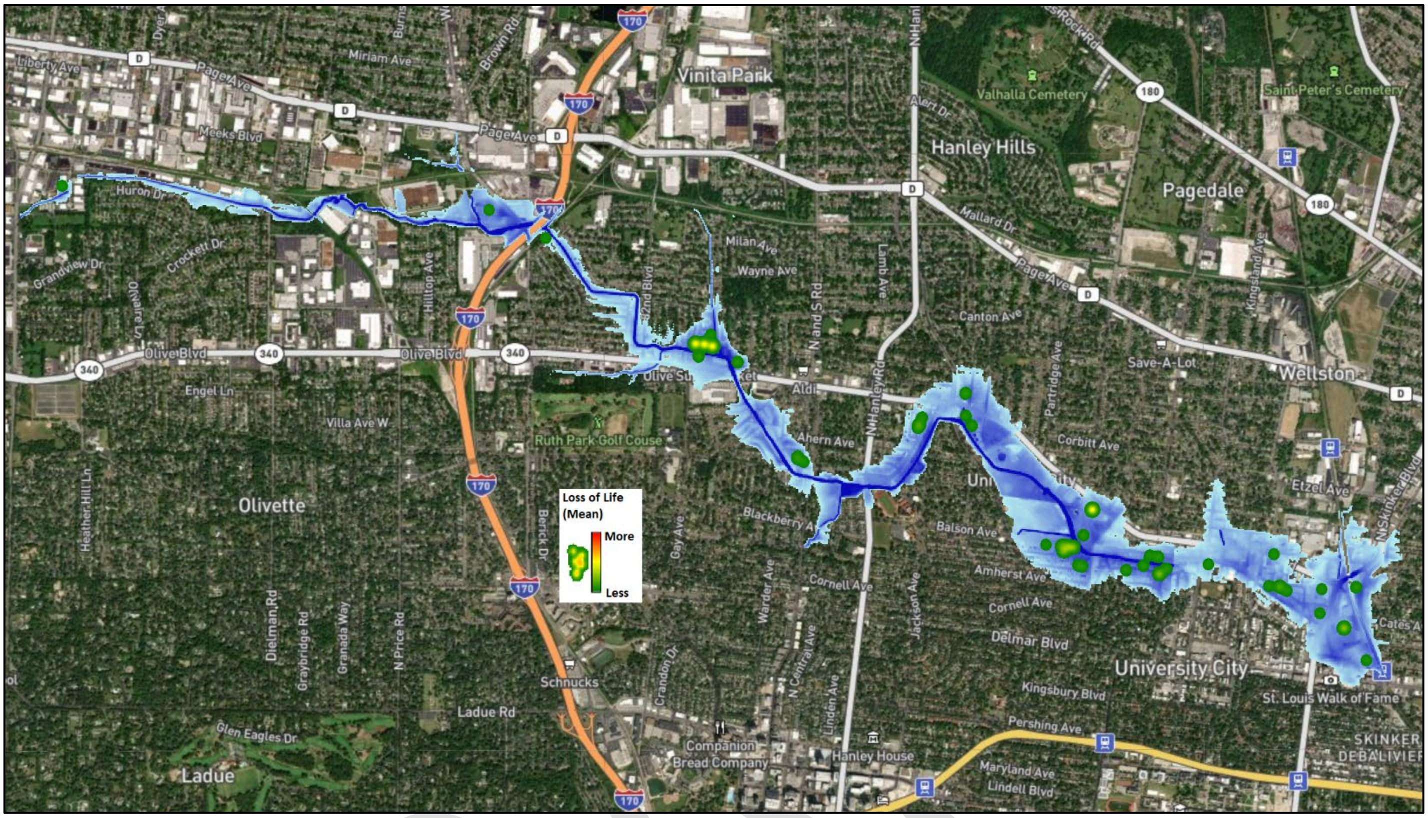


Figure 13. Heat Map - Life Loss with Minimal Warning in the Non-Breach Scenario, .2% AEP Event

Ample Warning

Table 8. Life Loss and PAR with Ample Warning, With Project Non-Breach.

Ample Warning - Non-Breach Life Loss					
Scenario Name	Structures Inundated	Population at Risk		Median Total Life Loss	
		Day	Night	Day	Night
.2% AEP Event	930	3200	3000	3	5
.5% AEP Event	780	2800	2600	1	2
1% AEP Event	640	2300	2200	1	1

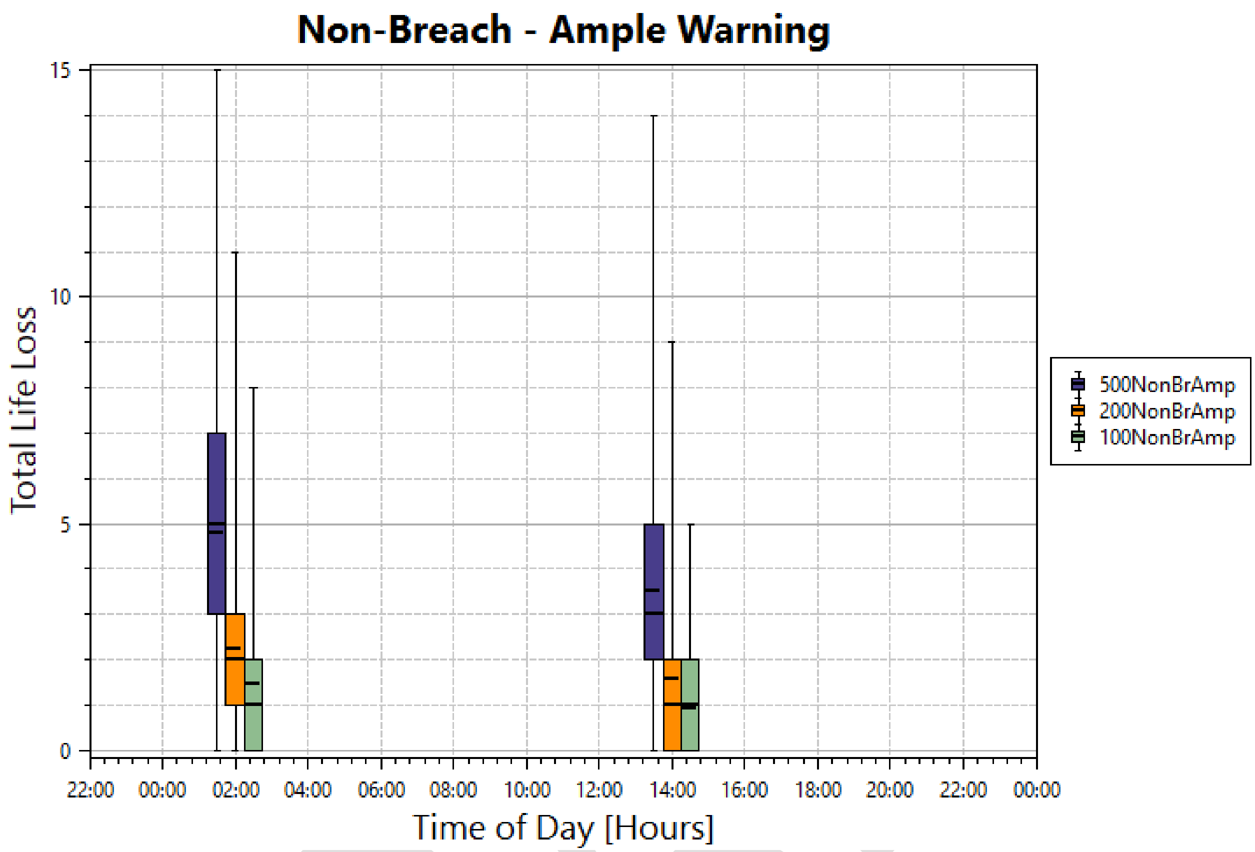


Figure 14. Life Loss with Ample Warning, With Project, Non-Breach.

With-Project Condition – Breach Scenario

The estimated life loss totals in the breach scenario by warning time are summarized in Table 9, Table 10, Figure 15, and Figure 17. Figure 16 shows the life loss heat map.

Minimal Warning

Table 9. Life Loss and PAR with Minimal Warning, With Project, Breach Scenario.

Minimal Warning - Breach Life Loss					
Scenario Name	Structures Inundated	Population at Risk		Median Total Life Loss	
		Day	Night	Day	Night
.2% AEP Event	950	3200	3000	6	7
.5% AEP Event	800	2800	2600	2	3
1% AEP Event	650	2400	2200	1	2

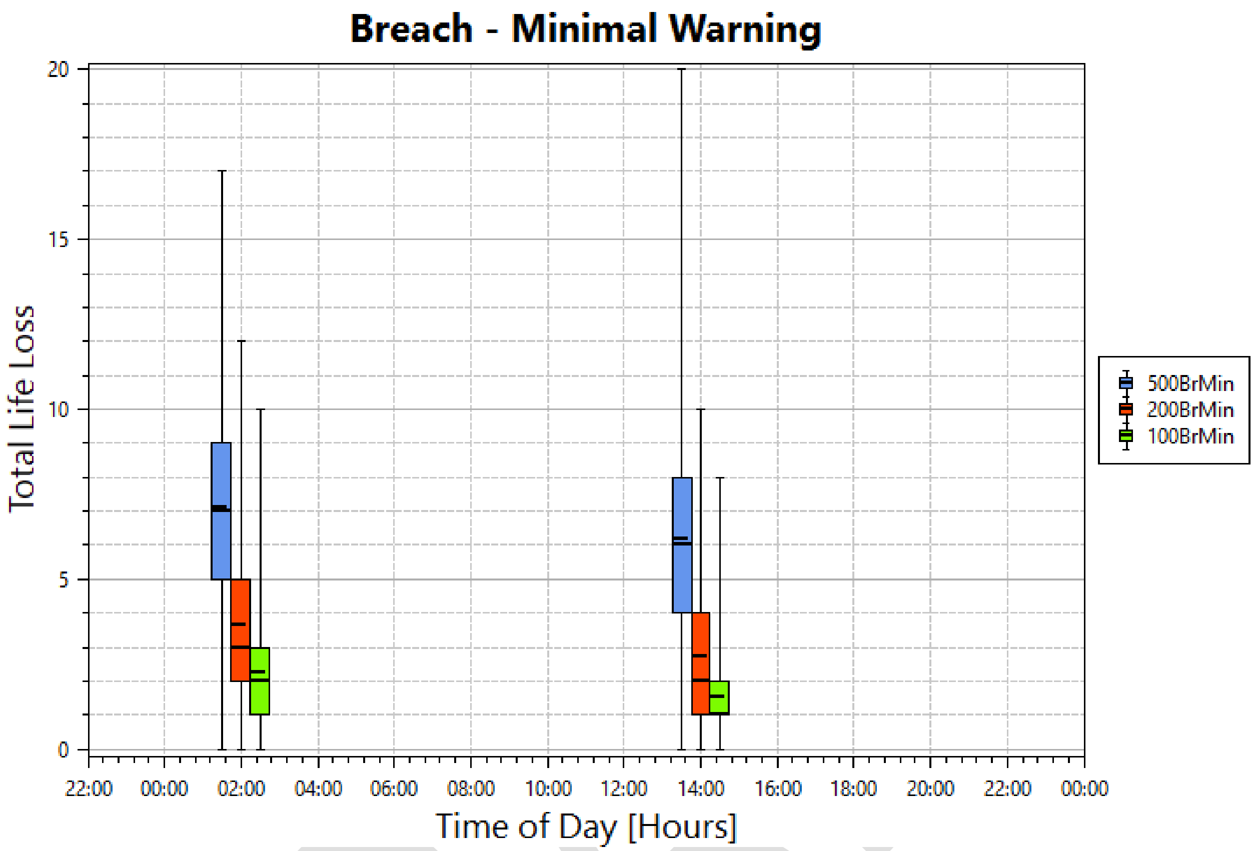


Figure 15. Life Loss with Minimal Warning, With Project, Breach Scenario.

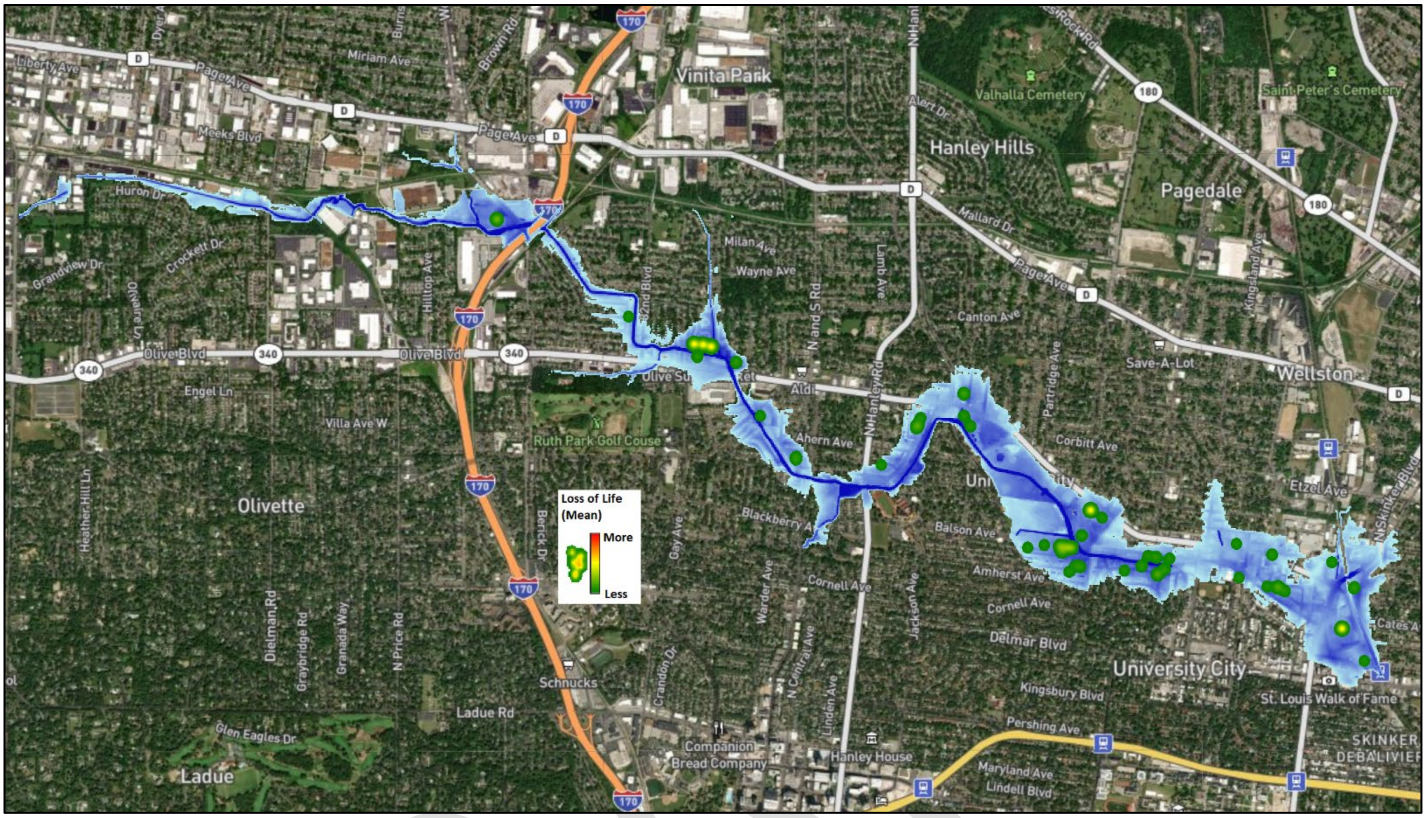


Figure 16. Heat MapLife Loss with Minimal Warning in the Breach Scenario, .2% AEP Event.

Ample Warning

Table 10. Life Loss and PAR with Ample Warning, With Project, Breach Scenario.

Ample Warning - Breach Life Loss					
Scenario Name	Structures Inundated	Population at Risk		Median Total Life Loss	
		Day	Night	Day	Night
.2% AEP Event	950	3200	3100	4	5
.5% AEP Event	800	2800	2700	1	2
1% AEP Event	650	2300	2200	1	1

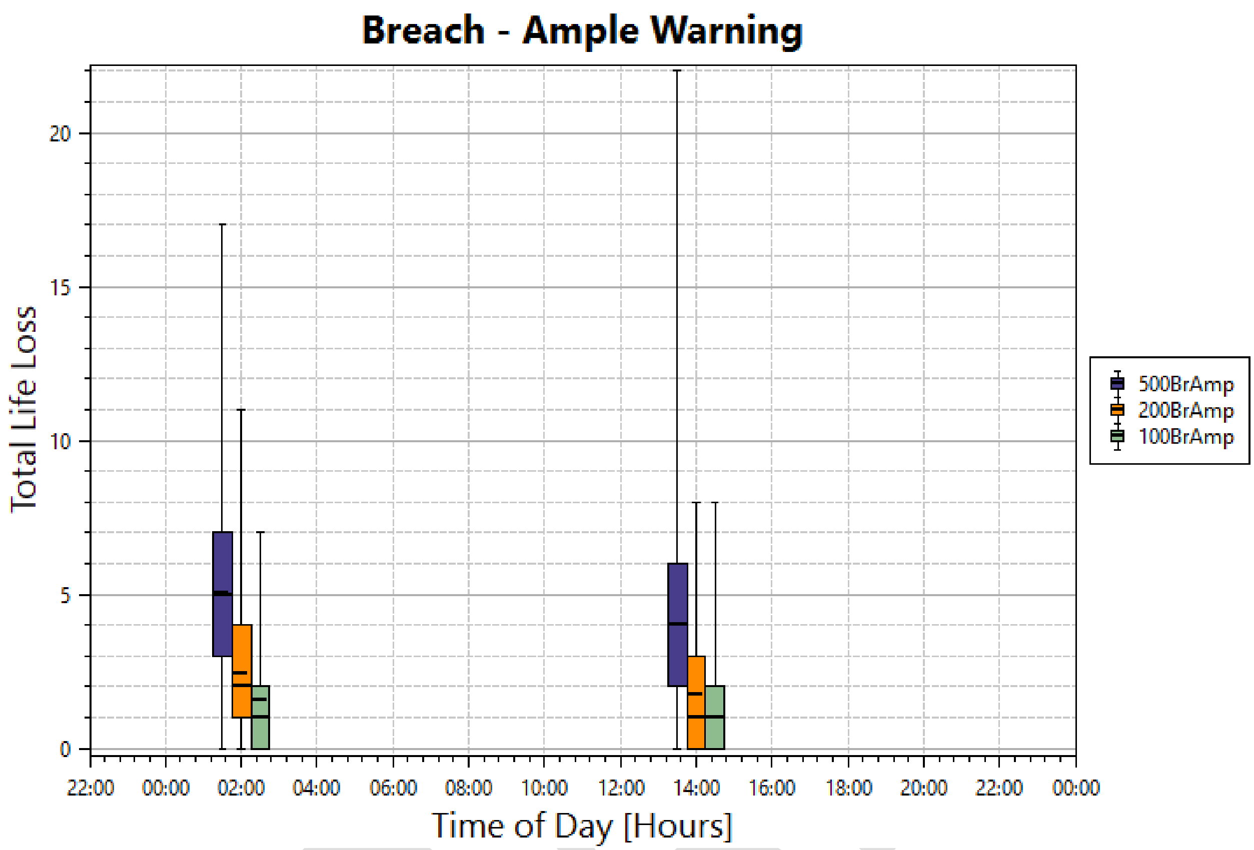


Figure 17. Life Loss with Ample Warning, With Project, Breach Scenario.

3.4. Incremental Life Loss

Incremental life loss is summarized below in Table 11 and Table 12. In the minimal and ample warning scenario, median incremental life loss is approximately zero. Life loss quantiles are in Table 13 and Table 14.

Minimal Warning

Table 11. Incremental Life Loss with Minimal Warning.

Minimal Warning - Incremental Life Loss					
Scenario Name	Structures Inundated	Population at Risk		Median Total Life Loss	
		Day	Night	Day	Night
.2% AEP Event	20	0	100	1	0
.5% AEP Event	20	0	100	0	0
1% AEP Event	10	100	100	0	0

Ample Warning

Table 12. Incremental Life Loss with Ample Warning.

Ample Warning – Incremental Life Loss					
Scenario Name	Structures Inundated	Population at Risk		Median Total Life Loss	
		Day	Night	Day	Night
500-Year Event (0.2% AEP)	20	0	100	1	0
200-Year Event (0.5% AEP)	20	0	100	0	0
100-Year Event (1% AEP)	10	0	0	0	0

Table 13. Estimated Life Loss Quantiles for Existing Condition

Existing Condition - .2% AEP	**Minimal Warning**		**Ample Warning**	
Statistic	Day	Night	Day	Night
95th Percentile	15	14	12	12
75th Percentile	10	11	8	8
Median	7	8	5	6
25th Percentile	5	6	2	3
5th Percentile	3	4	0	1

Table 14. Estimated Life Loss Quantiles for With-Project Condition

Breach Scenario - .2% AEP	**Minimal Warning**						**Ample Warning**					
	Breach		Non-Breach		Incremental		Breach		Non-Breach		Incremental	
Statistic	Day	Night	Day	Night	Day	Night	Day	Night	Day	Night	Day	Night
95th Percentile	12	12	10	12	2	0	9	10	8	10	1	0
75th Percentile	8	9	7	9	1	0	6	7	5	7	1	0
Median	6	7	5	7	1	0	4	5	3	5	1	0
25th Percentile	4	5	4	5	0	0	2	3	2	3	0	0
5th Percentile	2	3	2	3	0	0	0	0	0	0	0	0

3.5. Key Limitations / Lessons Learned

For this model, evacuation simulation on roads was not included. Duration of flooding is short and therefore roadway inundation is minimal, but some uncertainty exists since evacuation is limited to vertical movement (i.e., to the attic or second floor) within structures. Additionally, the modeling parameters related to warning and protective action were all given distributions with the greatest uncertainty, absent data indicating otherwise. Finally, population estimates are based on NSI 2.0 values, without more detailed information of the study area demographics.

3.6. Conclusions

Life loss in the with-project condition (breach and non-breach) is less than the without-project condition, so there is unlikely to be any additional risk of life loss from the detention basin. Indeed, the risk of life loss is likely reduced from the presence of the proposed detention basin. Furthermore, incremental life loss is approximately zero, suggesting there is little-to-no additional risk of life loss due to failure of the detention basin. Table 15 and Table 16 summarize the minimal and ample warning scenarios respectively.

Table 15. Minimal Warning Scenario

Minimal Warning Scenario						
Without Project						
Scenario Name	Structures Inundated	Population at Risk		Median Total Life Loss		
		Day	Night	Day	Night
.2% AEP Event	980	3200	3100	7	8
.5% AEP Event	850	2900	2700	4	5
1% AEP Event	720	2500	2300	2	2
With-Project Breach					
Scenario Name	Structures Inundated	Population at Risk		Median Total Life Loss	
		Day	Night	Day	Night
.2% AEP Event	950	3200	3000	6	7
.5% AEP Event	800	2800	2600	2	3
1% AEP Event	650	2400	2200	1	2
With-Project Non-Breach					
Scenario Name	Structures Inundated	Population at Risk		Median Total Life Loss	
		Day	Night	Day	Night
.2% AEP Event	930	3200	2900	5	7

.5% AEP Event	780	2800	2500	2	3
1% AEP Event	640	2300	2100	1	2

Table 16. Ample Warning Scenario

Ample Warning Scenario					
Without Project					
Scenario Name	Structures Inundated	Population at Risk		Median Total Life Loss	
		Day	Night	Day	Night
.2% AEP Event	980	3200	3200	5	6
.5% AEP Event	850	2900	2800	2	3
1% AEP Event	720	2500	2400	1	2
With-Project Breach					
Scenario Name	Structures Inundated	Population at Risk		Median Total Life Loss	
		Day	Night	Day	Night
.2% AEP Event	950	3200	3100	4	5
.5% AEP Event	800	2800	2700	1	2
1% AEP Event	650	2300	2200	1	1
With-Project Non-Breach					
Scenario Name	Structures Inundated	Population at Risk		Median Total Life Loss	
		Day	Night	Day	Night
.2% AEP Event	930	3200	3000	3	5
.5% AEP Event	780	2800	2600	1	2
1% AEP Event	640	2300	2200	1	1

4. POTENTIAL FAILURE MODE ANALYSIS (PFMA)

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. A facilitator guided the team members in developing the potential failure modes, based on the team's understanding of the project vulnerabilities resulting from the data review and current field conditions.

A PFMA was conducted by the following personnel (Table 17).

Table 17. Personnel Conducting the PFMA.

Name	Role	Organization
Troy Cosgrove, PE	Facilitator	MVD Levee Safety Center, Branch Chief
Jose Lopez, PE	Geotechnical Engineer	MVS, Geotech Design Section Chief
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On January 20, 2022, a scaled-down Potential Failure Mode Analysis (PFMA) was performed to inform the design of a detention basin for the River Des Peres in University City, MO. The scaled-down nature of the analysis was used to meet project requirements while being commensurate with the size and scope of the study. Risks associated with a structure breach do not exist now because the project still has not been built. The intention of the PFMA session is to mitigate future risk by identifying key items of concern that should be addressed during design and cost risks in development of the total project cost.

4.1. Design Background

The proposed structure is a detention basin that works more like a dam than a levee. It behaves like a dam by containing water to reduce the peak of flow downstream during certain storms and then releasing it after the peak has passed when stages are lower.

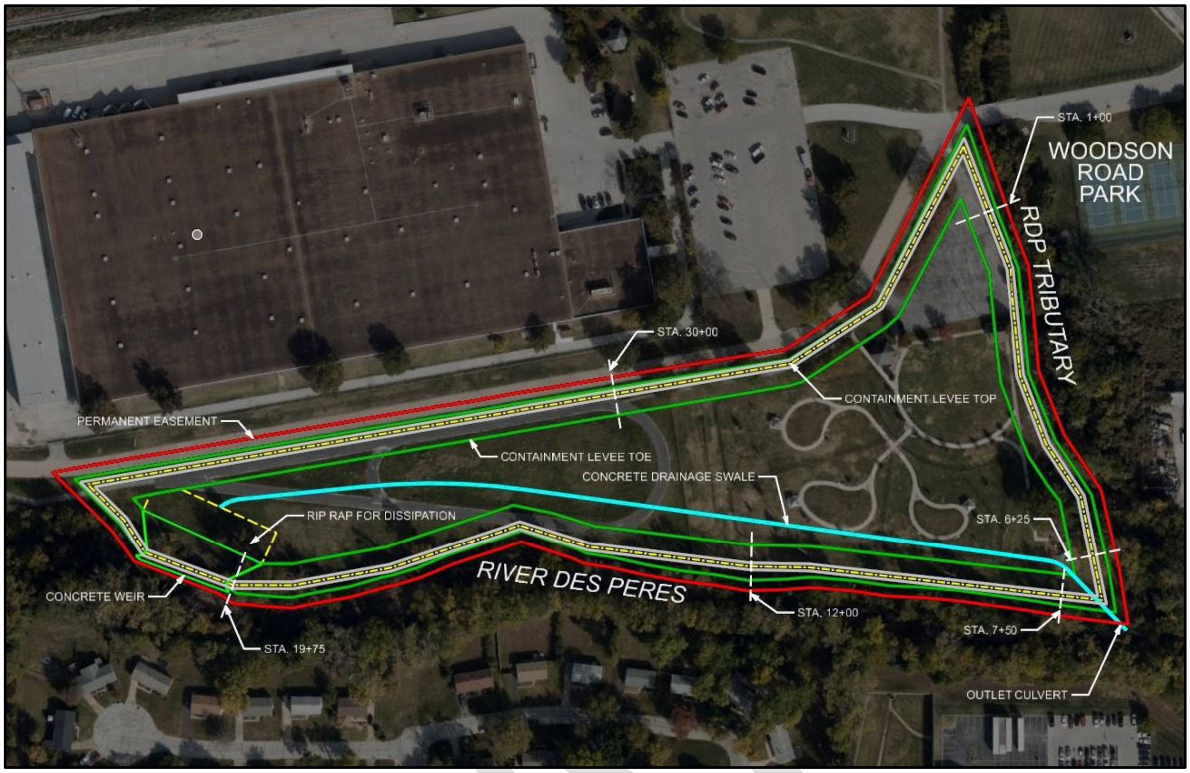

Figure 18. River Des Peres - Detention Basin 4 proposed geometry.

The detention basin design has a rectangular weir inlet, and a swale that channels water to the outlet pipe at the downstream end. It is primarily an earthen embankment with a substantial amount of cut occurring within the detention basin footprint to generate the required hydraulic volume. The majority of the structure will consist of some earthen compacted fill containment dikes. Currently the assumption is that the excavated material for the basin will be reused for the embankments. It's likely that the material will be sufficient and suitable as there is much more cut than fill in the current plan. The proposed geometry can be seen in Figure 18.

The detention basin was conceptualized to reduce flood risk for events starting at the 50% Annual Exceedance Probability (AEP) flood event. The weir-controlled inflow is designed to occur at the upstream end of the detention basin. The weir elevation is set to approximately fill the detention basin at the 10% AEP level. The inlet control weir is set at elevation 557.50 feet NAVD 88. The detention basin remains dry until the river stage reaches this elevation. Top of the detention basin embankments is at elevation 563.0 feet NAVD 88. At the downstream end, the unregulated pipe is sized to be a 36" diameter reinforced concrete pipe (RCP) with a flap gate and no valve. Invert elevation at the downstream end is 549.5 feet NAVD 88 with invert elevation at the upstream end of 550.0 feet NAVD 88 over 62 feet.

Approximately 30 to 45 minutes elapse from the center of mass of rainfall to when the River Des Peres reaches peak stage at the detention basin. In the detention basin, water will be retained and recede over a period of one to two days. Failure at the outlet pipe on the downstream end of the basin would be the worst failure for downstream locations. This is because the highest head difference between the retained water and the River Des Peres is at the outlet pipe.

The upstream end of the detention basin will overtop during a 1% AEP storm event. At its peak stage (561.59 ft NAVD88) the basin contains about 11.6 feet of impounded water above the outlet pipe invert. When the River Des Peres drops, because impounded water is higher than the river, it will backflow over the weir for events over exceeding 10% AEP flood levels. The basin will empty normally at the time water levels in the basin reach the weir control inlet elevation.

The peak head difference of approximately 5.0 feet between the river level at the outlet and detention basin water level occurs during the 0.5% AEP event. This coincides with a 5.3 ft head difference at the weir inlet. The head difference is lower for the 0.2% AEP. The head differences more gradually decrease for the frequencies below the 0.5% AEP. A peak weir head of 7.0 ft occurs during the 0.2% AEP event. The Figure 14 shows the river flood profiles for different return periods. Figure 19 shows the hydrograph for those return periods on the river side of the weir inlet.

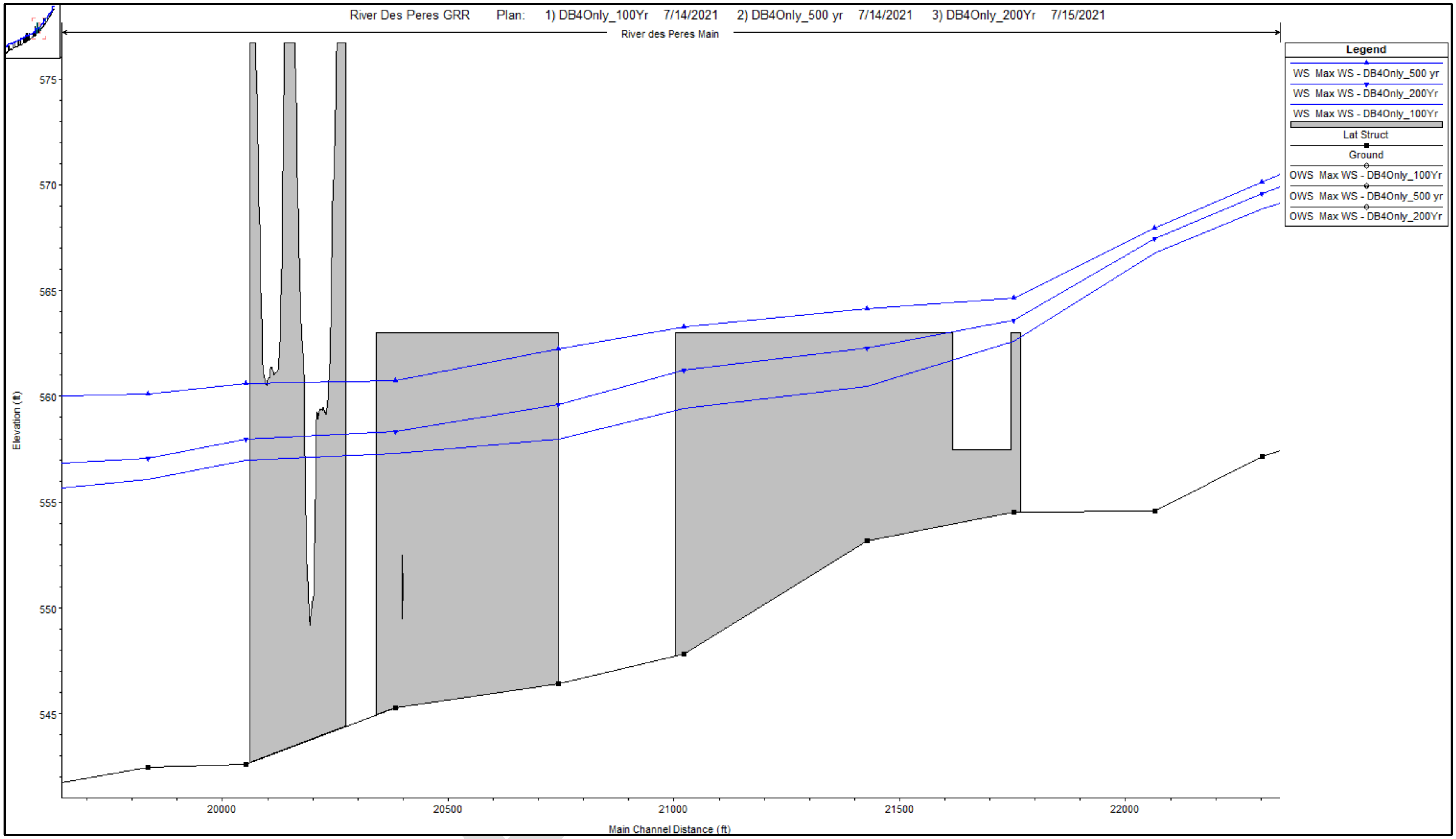

Figure 19. 1%, 0.5%, and 0.2% AEP flood profiles.

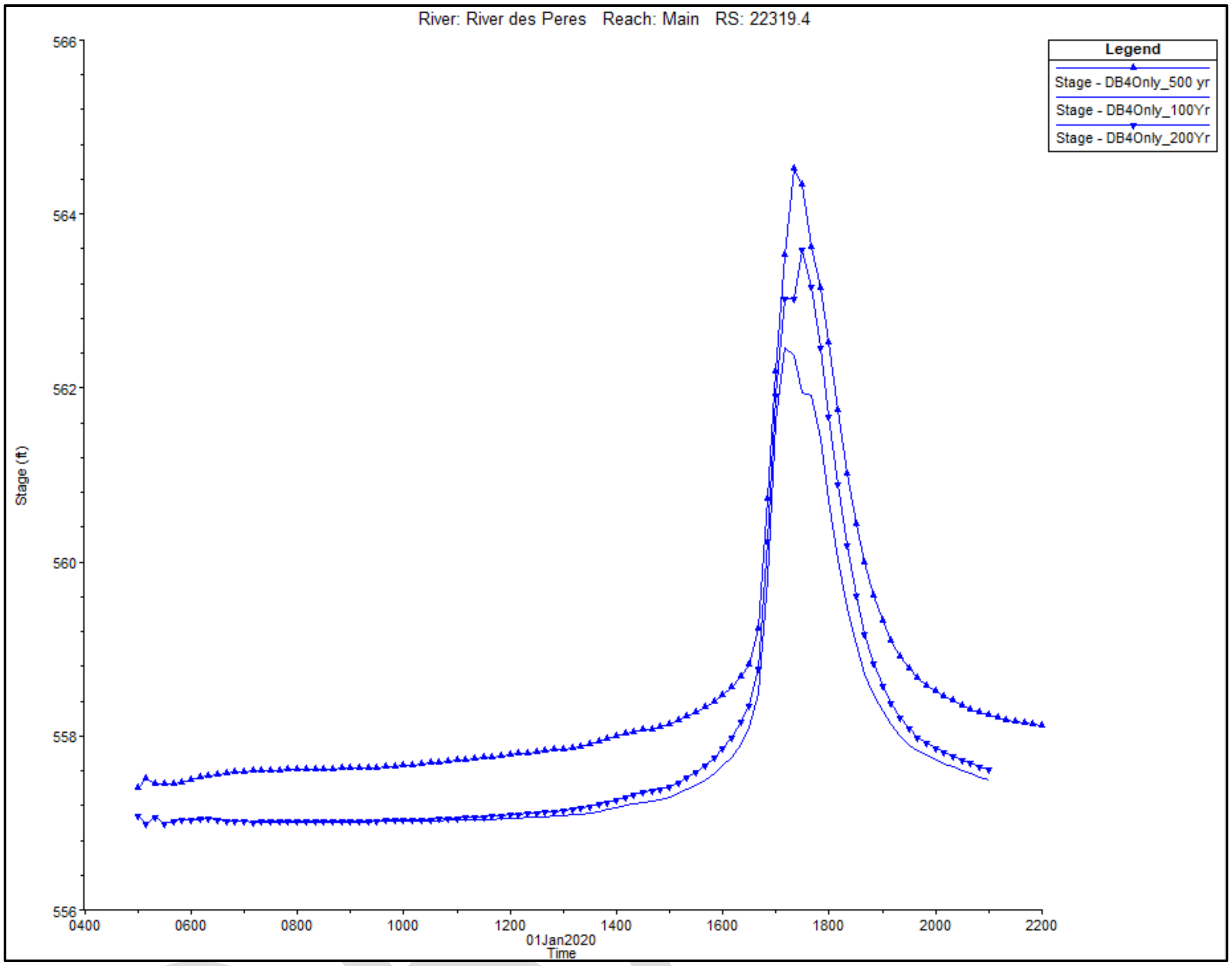

Figure 20. Hydrograph for the 1%, 0.5%, and 0.2% AEP flood at the upstream end.

Worst-case scenario for loading conditions would be when there is no water outside the basin but there is water inside at the weir location. The maximum possible head difference at the outlet pipe would be 7.5 feet. The flood of record on the River Des Peres occurred in September 2018. It was between the 4 and 10% AEP flood. Water surface reductions attributed to the construction of the detention basin are the highest in the area immediately downstream of the basin. Those benefits are diminished the further away you get downstream from the detention basin.

For the failure scenarios, breaching at the downstream side was analyzed because that's the location with the highest head difference. For the 1% AEP breach modeling, the breach was modeled to be 75 feet wide and was triggered at the outlet pipe (location with the highest head). All frequency storm events analyzed show that the without-project condition is worse than the breach with-project condition.

It was determined that highest with-project reduction immediately downstream of the detention basin is seen at the 0.5% AEP frequency (3.48 ft). The average reduction in water surface below the 1% AEP is 1.9 feet. Below the 2% AEP, water surface reduction trends similar differences for those more frequent events. The benefit stops once the weir control level is reached. The highest reduction in water level being the 0.5% AEP event is most likely due to the embankment having less of a constricting effect as it is overtopped during this and the less frequent events.

4.2. Brainstorming PFMs

- PFM 01: Overtopping of the detention basin embankment leads to breach.
- PFM 02: Internal erosion of the embankment through the foundation due to Backward Erosion Piping (BEP) causes breach.
- PFM 03: Internal erosion at the weir through the foundation due to BEP causes breach.
- PFM 04: Concentrated Leak Erosion (CLE) through the embankment due to construction shut down or transition zone leads to breach.
- PFM 05: CLE along the conduit drain leads to breach.
- PFM 06: CLE into the conduit drain at a pipe joint leads to breach.
- PFM 07: Wave wash of the embankment leads to breach.
- PFM 08: Scour/erosion along the River Des Peres embankment toe leads to breach.
- PFM 09: CLE along the interface of the embankment and natural ground leads to breach.
- PFM 10: Conduit is blocked with woody debris, sediment or flap gate fails to operate, basin fails to drain, and a subsequent storm leads to premature overtopping.
- PFM 11: Global stability failure of the detention basin weir leads to breach (sliding, overturning, or bearing).
- PFM 12: Scour at the interior weir apron leads to failure and breach.
- PFM 13: Scour along the River Des Peres at the entrance to the weir leads to stability failure.
- PFM 14: Slab jacking of the spillway slope leads to instability, failure of the weir, and breach.
- PFM 15: Structural failure of the reinforced concrete wingwalls leads to adjacent embankment erosion and breach.
- PFM 16: Reverse head leads to scour along the weir wingwall which leads to loss of embankment and breach.
- PFM 17: Seismic failure of the concrete weir leads to failure and breach.
- PFM 18: Static slope instability of the embankment leads to breach.
- PFM 19: CLE along the weir and the embankment contact/transition leads to breach.
- PFM 20: Seismic event causes slope instability or liquefaction and leads to breach.

- PFM 21: Discharge from the submerged outlet conduit leads to scour and erosion of the embankment leading to breach.
- PFM 22: Structural collapse of the conduit causes cracking in the embankment and leads to CLE which leads to breach.
- PFM 23: Improper compaction in the conduit excavation leads to settlement and cracking of the embankment and CLE which leads to breach.
- PFM 24: CLE along the weir/foundation interface leads to breach.
- PFM 25: Sinkhole forms due to karst within the foundation which leads to collapse of the embankment and breach.
- PFM 26: CLE through the embankment due to an animal burrow leads to breach.
- PFM 27: CLE through the embankment due to tree roots leads to breach.
- PFM 28: Tree collapse or excessive vegetation causes turbulence and erosion of the embankment leading to breach.

4.3. Evaluating PFMs

Many of the brainstormed PFMs are typically managed with designed defensive measures, adhering to published engineering standards, construction Quality Assurance (QA), or Emergency Action Plans (EAP). A more thorough risk assessment (i.e., Semi-Quantitative Risk Assessment – SQRA) will occur during the pre-construction engineering and design (PED) phase of the project.

For this screening-level assessment, qualitative methods were used to determine life loss likelihoods if that failure mode occurred. This evaluation did not consider the actual failure likelihood (i.e., reliability) from this level of design. The ease of prevention via design considerations was evaluated, and a decision was made if further evaluation was required. Even if the potential for failure was high, if the evaluation states that it is a typical design consideration, no additional evaluation is required at this stage.

Table 18. Potential Failure Modes Analysis.

Failure Modes	Evaluation/Factors	Mitigation	Uncertainties	Other Considerations
PFM 01: Overtopping of the detention basin embankment leads to breach.	Long-term settlement of the embankment lowers crown elevation leading to overtopping. Embankment will overtop for any event greater than the 1% AEP event.	Turf reinforcement mat. Proper overbuild considerations based on site specific subsurface information and settlement analysis.	Current results are from hydraulic computer models. There is instability in the models which could lead to different results. Current subsurface compressibility is unknown due to lack of exploration.	Further evaluation will be needed to determine velocities on downstream slope during an overtopping event. Most likely overtopping location is at the upstream end (from the creek into the basin. Somewhere between the 0.5% and 0.2% AEP flood; not a big differential [tenths of a foot to maybe half a foot]). But the downstream end will have the larger head differentials (from the basin into the creek. About 4 to 5 feet of differential head).

Failure Modes	Evaluation/Factors	Mitigation	Uncertainties	Other Considerations
				Evaluate if duration of overtopping is short (1 to 2 hours), and velocities may not be enough to erode a good grass cover.
PFM 02: Internal erosion of the embankment through the foundation due to BEP causes breach.	Depending on the foundation conditions, it will make this failure mode less or more likely.	Needs to be evaluated during design. Site specific exploration program should be conducted, to classify foundation materials and evaluate susceptibility to this failure mode.	Currently, no site-specific subsurface information has been obtained. Unsure what the makeup of the foundation soils is. We do know there is silty loam at shallow depth due to NRCS maps.	Likely short duration loadings are expected within the basin, which may be a factor to consider for likelihood evaluation.
PFM 03: Internal erosion at the weir through the foundation due to BEP causes breach.	Depending on the foundation conditions, it will make this failure mode less or more likely.	Needs to be evaluated during design. Site specific exploration program should be conducted. Classify foundation materials. Evaluate the need for a seepage cutoff at structure (e.g. sheet pile)	Currently, no site-specific subsurface information has been obtained. Unsure what the makeup of the foundation soils is. We do know there is silty loam at shallow depth due to NRCS maps.	Likely short duration loadings are expected within the basin, which may be a factor to consider for likelihood evaluation. Sheet pile cut-off likely needed regardless from a structural/foundation standpoint.
PFM 04: CLE through the embankment due to construction shut down or transition zone leads to breach.	Depending on the borrow source, it will make this failure mode less or more likely. The embankment should be able to be constructed within one season, making shutdowns and transition zones less likely.	Needs to be evaluated during design. Site specific exploration program should be conducted to classify borrow materials or identify offsite borrow. Specifications should address any type of shutdown or transition zones.	Currently, no site-specific subsurface information has been obtained. Unsure if the foundation soils will be acceptable as embankment fill. We do know there is silty loam at shallow depth due to NRCS maps.	
PFM 05: CLE along the conduit drain leads to breach.	Type of backfill used around the conduit. Width of the trench needs to be sufficient for proper compaction. Improper compaction techniques or other issues could contribute to likelihood.	Special compaction may be needed around the pipe. Consideration of flowable fill and or filter at the downstream end of the pipe.	Material type for embankment and/or foundation is currently unknown.	
PFM 06: CLE into the conduit drain at a pipe joint leads to breach.	Ensure placement of the pipe and joint are constructed in a way to prevent separation of the pipe joint.	Wrap pipe joints with geotextile. Use pipes with bell and spigot	Foundation conditions and construction of the pipe.	

Failure Modes	Evaluation/Factors	Mitigation	Uncertainties	Other Considerations
		with some type of O-ring. Consider cast-in-place if cost effective.	Material type for embankment and/or foundation is currently unknown.	
PFM 07: Wave wash of the embankment leads to breach.	Low likelihood of high winds at the peak of the flood. Duration of the waters at the peak is very short. River is very flashy and will go up and down rather quickly, reducing likelihood of wave wash. There is not a big fetch.	Evaluate during design.	Durations and level of flooding based on hydraulic computer models.	Consider upstream revetment as with most dams.
PFM 08: Scour/erosion along the River Des Peres embankment toe leads to breach.	Steep streambed will yield high velocities which could potentially erode the toe of the embankment.	Evaluate during design if rip rap or other type of slope protection is needed. Evaluate channel velocities and geometry of channel and embankment.	Velocities within the stream channel, turbulence, and erosion resistance of foundation and embankment materials.	
PFM 09: CLE along the interface of the embankment and natural ground leads to breach.	Need to ensure proper interface between embankment materials and foundation to prevent improperly compacted interface.	Specifications should be written to ensure scarification of the foundation prior to placement of embankment to ensure good contact between the two materials. Ensure proper compaction and clearing and grubbing is performed.	Foundation conditions and construction of the embankment.	Typically, a non-issue and addressed with clearing and grubbing and foundation prep specs. Consider need for inspection trench. Proper site-specific investigation should be conducted.
PFM 10: Conduit is blocked with woody debris, sediment or flap gate fails to operate, basin fails to drain, and a subsequent storm leads to premature overtopping.	There is trash, debris, and sediment within the stream channel that could potentially cause blockage of the conduit. If the flap gate is not properly maintained, it may not operate as intended. There may be enough time before subsequent storm to fix the problem.	Trash rack. Inspect and maintain clean after every event. Proper O&M of the flap gate.	The amount of trash, debris, and sediment that will accumulate and make it past the weir.	
PFM 11: Global stability failure of the detention basin weir leads to breach (sliding, overturning, or bearing).	No foundation information nor designs have been completed.	Global stability will be designed and analyzed during the design phase to ensure meeting appropriate factors of safety.	Uncertainty on foundation conditions and sub-surface information. Unsure of rock foundation, shallow or deep, etc.	

Failure Modes	Evaluation/Factors	Mitigation	Uncertainties	Other Considerations
PFM 12: Scour at the interior weir apron leads to failure and breach.	Scour of soils at the interior toe of the weir during initial filling of detention basin due to high energy environment.	Design should consider the appropriate stilling basin and the need to use rock to prevent scour during the filling of the basin.	No layout yet developed. Stilling basin not designed yet.	Potential for hydraulic modeling to establish cost effective weir design.
PFM 13: Scour along the River Des Peres at the entrance to the weir leads to stability failure.	Velocities from the stream and turbulence created by the structure location.	Consider the need for a riverside cutoff, apron, and/or rip rap at the entrance.	Velocities and turbulence generated at the interface of the stream and the weir. Erosion resistance of foundation materials.	
PFM 14: Slab jacking of the spillway slope leads to instability, failure of the weir, and breach.	Even if the slab is lost, due to the short duration, it may not progress to weir instability and breach.	If there is an apron, the design should consider the potential for slab jacking or flotation.	No layout yet developed. Stilling basin not designed yet.	
PFM 15: Structural failure of the reinforced concrete wingwalls leads to adjacent embankment erosion and breach.	If there is a wall failure occurring, it may be separate from a flood event, which could allow for time to mitigate the problem.	Wingwalls will be designed for the anticipated soil loads.	Geometry of the structure is still not final. Wingwall configuration not yet designed/developed/	
PFM 16: Reverse head leads to scour along the weir wingwall which leads to loss of embankment and breach.	Velocities from the stream and turbulence created by the structure location. There will be reverse flow over the weir and out of the detention basin as the flood is passing during extreme events.	Consider the need for a riverside cutoff, apron, and/or rip rap at the entrance to mitigate for exit velocity and turbulence created by flow out of the detention basin.	Velocities and turbulence generated at the interface of the stream and the weir. Erosion resistance of foundation materials.	
PFM 17: Seismic failure of the concrete weir leads to failure and breach.	Seismic potential may be low for this area. A seismic event is independent of a flood event, potentially allowing time to mitigate structural damages.	Seismic loading will be considered during design of the weir structure. Consideration for a post-seismic mitigation may be needed in the O&M manual.	Seismic potential for the area has not been evaluated.	Need to consider this carefully. This will not impound water “permanently” like most USACE dams. Coincident probabilities should inform the rigor that is needed for this evaluation and design.
PFM 18: Static slope instability leads to breach.	Need to evaluate slope stability utilizing properties of embankment and foundation materials.	Evaluate during design and determine embankment slopes and any ground improvements that may need to be performed.	Currently, no site-specific subsurface information has been obtained. Unsure what the makeup of the foundation and embankment soils is. We do know there is silty loam at shallow depth due to NRCS maps.	
PFM 19: CLE along the weir and the embankment	Need to evaluate the potential for CLE along the weir and embankment interface.	Specifications should address proper	Weir layout and embankment soils.	

Failure Modes	Evaluation/Factors	Mitigation	Uncertainties	Other Considerations
contact/transitions leads to breach.		compaction, and design should address seepage lengths and battered walls to ensure that CLE is not an issue. Sheet pile transitions should also be considered.		
PFM 20: Seismic event causes slope instability or liquefaction and leads to breach.	Seismic potential may be low for this area. A seismic event is independent of a flood event, potentially allowing time to mitigate structural damages.	Seismic loading will be considered during design of the embankment. Consideration for a post-seismic mitigation may be needed in the O&M manual.	Seismic potential for the area has not been evaluated. Currently, no site-specific subsurface information has been obtained. Unsure what the makeup of the foundation and embankment soils is. We do know there is silty loam at shallow depth due to NRCS maps. Uncertain if liquefiable soils are on-site.	Need to consider this carefully. This will not impound water “permanently” like most USACE dams. Coincident probabilities should inform the rigor that is needed for this evaluation and design.
PFM 21: Discharge from the submerged outlet conduit leads to scour and erosion of the embankment leading to breach.	Discharge from submerged outlet conduit may cause turbulence that may erode embankment foundation materials.	Evaluate the need for protection around the conduit during design.	Unsure about velocities and turbulence of the discharge from the submerged conduit. Unsure about the erosion resistance of the embankment and foundation soils.	
PFM 22: Structural collapse of the conduit causes cracking in the embankment and leads to CLE which leads to breach.	Conduit may collapse under load from the trench backfill soils. When the conduit collapses, that may lead to settlement and cracking of the embankment. Cracking of the embankment may lead to CLE of the embankment materials. If RCP pipe is selected, it has long durability.	Design the pipe for the loading caused by the trench backfill soils.	Construction methods and backfill soils.	
PFM 23: Improper compaction in the conduit excavation leads to settlement and cracking of the embankment and CLE which leads to breach.	Backfill materials within the excavation trench for the conduit are not properly compacted, leading to settlement and cracking of the embankment, and eventually CLE through the embankment.	Ensure proper backfill soils, trench excavation size, and compaction are specified for the conduit trench.	Construction methods and backfill soils.	
PFM 24: CLE along the weir/foundation interface leads to breach.	The interface between the weir and foundation can form a pathway for CLE to occur.	Seepage cutoff under the weir during design.	The weir layout and the foundation soils are unknown. Proper subsurface information and characterization can better inform this.	

Failure Modes	Evaluation/Factors	Mitigation	Uncertainties	Other Considerations
PFM 25: Sinkhole forms due to karst within the foundation which leads to collapse of the embankment and breach.	Depending on the foundation conditions it will make this failure mode less or more likely.	Needs to be evaluated during design. Site specific exploration program should be conducted, confirm the presence of karstic materials.	Currently, no site-specific subsurface information has been obtained. Unsure of the presence of karstic features at the site.	Consider geophysical survey and regional data for evaluation during design.
PFM 26: CLE through the embankment due to an animal burrow leads to breach.	If animal control is not performed, burrows could lead to pathways for CLE to develop.	Ensure that the O&M manual accounts for animal control plan and the treatment of animal burrows if they are observed.	None.	
PFM 27: CLE through the embankment due to tree roots leads to breach.	If trees are allowed to grow in embankment, tree roots may serve as pathways for CLE to develop.	Ensure that the O&M manual accounts for vegetation control plan.	None.	
PFM 28: Tree collapse or excessive vegetation causes turbulence and erosion of the embankment leading to breach.	If a tree collapses or vegetation is allowed to grow in the embankment, it could create turbulence that would erode the embankment.	Ensure that the O&M manual accounts for vegetation control plan.	None.	

While none of the failure modes evaluated stood out as particularly "risk driving", these failure modes should and will be considered during design of the project and will be re-evaluated once the design is more substantial.

5. TYPICAL RISKS

Since the designs are still relatively conceptual in nature a more rigorous risk assessment (e.g., Semi-Quantitative Risk Assessment, Quantitative Risk Assessment) was not performed at this point. Having subsurface data and design at least at the 35-65% level would reduce the uncertainties to the point that the risk assessment may further inform what measures will be needed to ensure compliance with USACE Dam Safety guidelines, so that incremental risks are properly mitigated and managed as low as practicable.

6. KEY LIMITATIONS

The limitation of the PFMA session and any risk analysis methodology is primarily driven by the availability and the completeness of the information used to assess the risk. With due regards for uncertainty at this point it is recommended that further design is conducted and that at least an SQRA session is completed between the 35-65% design level.

The methodology for the scaled down PFMA seems appropriate for this level of study. It identifies the potential for risks but cannot fully quantify the risk until more information is available on the design and existing conditions.

7. CONCLUSIONS

At the feasibility phase of the project, the screening level risk assessment did not identify any potential failure modes that would favor one alternative significantly over the other or that would lead to elimination of the DB4 alternative. Additional information, including modeled life loss evaluations, subsurface investigations, and advancing design will allow for a more thorough and quantitative evaluation.