Appendix B – Hydraulic Analysis

Whitney Lake Reallocation Study
Bosque and Hill Counties, Texas

DRAFT

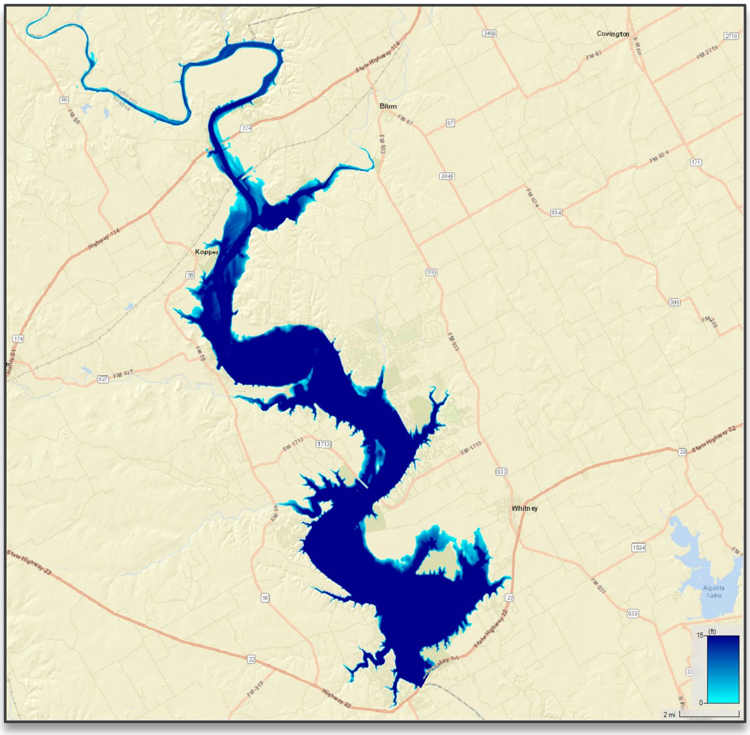
Integrated Feasibility Report and Environmental Assessment

July 2025

US Army Corps
of Engineers®
Fort Worth District

Whitney Lake Reallocation Study, Bosque and Hill Counties, Texas Hydraulic Analysis

DRAFT Appendix B



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May 27, 2025

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1. Introduction

The U.S. Army Corps of Engineers, Ft. Worth District has prepared a hydrologic and hydraulic analysis to inform the risks associated with the eight alternatives under evaluation for the Whitney Lake Reallocation Study, Hill and Bosque Counties, Texas Draft Integrated Feasibility Report and Environmental Assessment (IFR/EA). The following sections will cover the general project information, alternatives evaluated, and the hydrologic and hydraulic analysis for this study.

1.1. General Project Information

Lake Whitney is a flood control reservoir on the main stem of the Brazos River in Texas. It is located on river mile 442.4 and controls a drainage area of 26,606 square miles which 17,656 square miles of contributing and 8,950 square miles are noncontributing. The reservoir encompasses a surface area of more than 23,500 acres under normal operating conditions. The reservoir has a maximum depth of 108 ft and 225 miles of shoreline when the pool is at conservation. The reservoir in the headwaters of Whitney's watershed is De Cordova Bend dam/Lake Granbury which occupies the 8,950 square miles of Whitney's noncontributing area. Lake Granbury does not have flood storage therefore it is treated as a run of the river reservoir. Once the Lake reaches conservation pool, the project's releases match the inflow which in turn increases the contributing area for Whitney.

Whitney Dam is one of many reservoirs in a system which includes Morris Sheppard Dam (Brazos River Authority [BRA] Project), De Cordova Bend Dam (BRA Project), Waco Dam, Aquilla Dam, Sterling C Robertson Dam (BRA Project), Proctor Dam, Belton Dam, Stillhouse Hollow Dam, North San Gabriel Dam, Granger Dam, and Somerville Dam. To provide the maximum reduction of floods in the Middle and Lower Brazos River Basin. Releases from Whitney Dam need to be coordinated with the other projects to ensure downstream constraints do not exceed normal channel capacity.

The embankment consists of an 8,201 feet long rolled earthfill embankment a 1,674 feet concrete gravity section, and 7,820 feet of dikes. The maximum height of the concrete section of the embankment is 159 feet. The top of concrete section width is 38 feet of which 28 feet is roadway. The top of the concrete dam section is at elevation 584.0 feet and the top of the earthen embankment is at elevation 580.0 feet. The spillway is a 680 feet long gate-controlled ogee spillway. The spillway is controlled by seventeen 40-foot by 38-foot tainter gates, with a crest at elevation 533.0 feet. The discharge capacity of the gated spillway is 661,000 cubic feet per second (cfs) when the water surface elevation is at 573.0 feet.

The Kopperl Levee surrounds and protects the town of Kopperl, Texas in Bosque County. The Levee is located on the perimeter of Whitney Lake. In the event of a large storm event, Kopperl levee will protect the town up to elevation 577.1 feet. The levee consists of six segments and has a crown elevation of 577.1 feet. The top width is 10 feet wide, with a total length of 7,744.12 feet.

1.2. Pertinent Data

Table 1-1 lists pertinent project data for Whitney Dam.

Table 1-1. Whitney Dam Pertinent Project Data

Dam Physical Data	
Dam General Description	Earthfill with controlled concrete spillway
Dam Type	Earth
Dam Crest Length including spillway and dyke (feet)	17,695
Dam Crest Width (feet)	34
Dam Crest Elevation (feet)	580.10 – Earth Embankment 584.10 – Concreate Dam 579.92 (Low spot on the embankment left of the spillway–STA 41+00)
Spillway General Description	680-foot long (net length), Tainter gate-controlled ogee weir
Spillway Type	Controlled
Spillway Crest Elevation (feet)	533.10
Spillway Width (feet)	680
Spillway Maximum Design Discharge (cubic feet per second [cfs])	676,345
Spillway Maximum Design Discharge Elevation (feet)	573.10
Spillway Gates	17 Tainter gates (40 feet by 38 feet)
Outlet Works General Description	Sixteen 5 by 9 feet conduits through the base of the concrete dam
Outlet Gates	Sixteen sluice gates (5 by 9 feet)
Outlet Works Discharge Capacity (cfs)	46,400
Outlet Works Discharge Capacity Elevation (feet)	571.10
Dam Hydrology	
Drainage Area (square miles)	26,606 (17,656 Contributing and 8,950 noncontributing)
Maximum Historic Release (cfs)	50,000 (05/29/1957)
Maximum Historic Pool Elevation (feet)	570.25 (05/29/1957)
Brazos River Channel Capacity (cfs)	60,000
Datum adjustment from NGVD 29 to NAVD 88 (feet)	+0.10

Note: All elevations in this report are expressed in NAVD 88. Elevations in this table are rounded to two decimal places. All subsequent elevations in this report will be rounded to one decimal place.

2. Hydraulic Modeling

The following sections summarize the hydraulic model that was used for the Whitney reallocation, breach modeling, and reassessing the downstream control points.

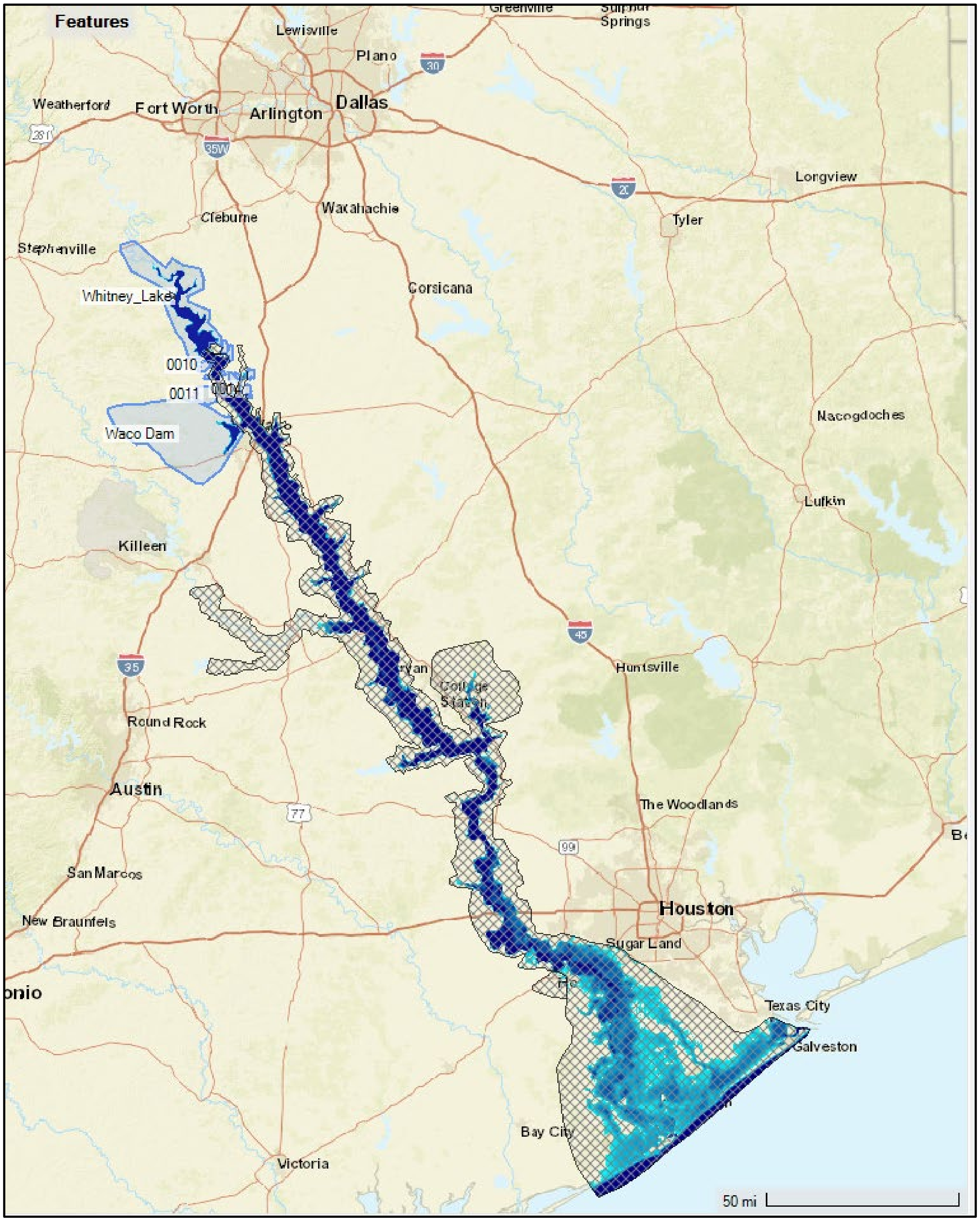

Figure 2-1: Extents of the HEC-RAS Model Used in this Study

2.1. Hydraulic Model Development

A previous 2D model was used from the Waco PA in 2023. The model was tested and approve for the 2023 study. Modifications were made to the model to repurpose for the Whitney Breach analysis. A storage area was added for the Whitney reservoir and a surveyed embankment profile was used for the embankment (low spot at the left of the spillway of 579.9 ft-NAVD88). The dike left of the spillway was also added to the model. Terrain modifications were made to best represent the structure and a 2D flow area was added to model the overflow at that section. On the perimeter of the reservoir, an additional 2D mesh was added in the town of Kopperl to represent the Kopperl levee. When the lake elevation reaches 576.1 ft NAVD88, the levee is overtopped. Downstream of Whitney Lake, additional storage areas were added to accommodate the 1,5 PMF run.

The model was run using diffusion-wave equation. A sensitivity was performed on the TAS scenario utilizing the full-momentum equation for comparison. Using the City of Waco, Texas, as an index point for comparison, the TAS non-breach scenario produces a peak water surface elevation that was 1.2 feet lower (413.2 feet versus 414.4 feet) for the diffusion-wave run compared to the full-momentum run. As for the timing and peak flow, the diffusion wave arrived 1 hour and 20 minutes earlier and had a higher peak inflow of 13,758 cfs (diffusion wave equation 544,455 cfs at 06 Feb 11:30 verses the Full momentum at 530,697 cfs at 06 Feb 12:50). The diffusion wave equation was a more conservative value on the peak flow and timing of the event; therefore, it was used for the final equation for the breach modeling.

Terrain Data and Layers

Several types of terrain data and data layers were used to perform the hydraulic analysis of the study area. Table 2-1 summarizes terrain data and its source. The Corps Water Management System (CWMS) model was provided by the Fort Worth District and was deemed inadequate for purposes of the Periodic Assessment (PA). A full two-dimensional (2D) model was developed for evaluation of a dam breach for the 2023 PA. Detailed discussions of their use are provided in other sections of this report.

Table 2-1: Summary of Geospatial Data

Data	Name of File	Description	Source
Digital Elevation Model (DEM)	Waco_DEM_MMC_Merged_10ft_2023.Waco_DEM_MMC_Merged_10Ft_2023.tif	The area to the south is from National Oceanic and Atmospheric Administration (NOAA) bathymetry data; Continuously updated Digital Elevation Model (CUDEM). The data was originally 3-meter resolution, resampled to 10-meter resolution.	The area to the south is from NOAA bathymetry data; CUDEM: https://coast.noaa.gov/dataviewer/#/lidar/search/where:ID=8483
	Waco_DEM_MMC_Merged_30ft_2023.tif	The area to the north was provided by the Fort Worth District and was originally obtained from the Texas Water Development Board, which contracted Aqua Strategies that resampled mixed images of resolutions, geographic projections, and file type. The data was resampled to 3-meter and 10-meter resolution.	Texas Water Development Board (data provided by the Fort Worth District)
Land Cover	LandCover.tif	2016 National Land Cover Database (NLCD) raster data	NLCD

Cross-Section Elevation Data

The DEM data for this study was obtained from the Fort Worth District CWMS model and supplemented with DEM data provided by the MMC. The source of the CWMS terrain data was unavailable and has been carried forward from previous models. The supplemental terrain data was from NOAA and the Texas Natural Resources Information System (TNRIS). Cross-section elevation data was not used for this study as the model was fully 2D.

Two-dimensional Flow Area

This model was completed using a 2D flow area for the Brazos River downstream of the [Subject] in the studied area. The 2D flow area was used in place of 1D cross sections due to model instabilities in the 1D model and a complex confluence near the city or Waco. The NLCD was used to assign Manning's "n" values as appropriate for the 2D flow area based on the land uses. The terrain data did not contain information below the water surface, so the terrain modification tool in RAS Mapper was used to incorporate the channel geometry downstream of Whitney into the terrain within the 2D flow area.

Manning's N Values

Manning's "n" values for the main channel and overbanks were obtained from the 2016 NLCD.

Structures

The Bosque River has several bridges downstream of Whitney Dam and the confluence with the Brazos River. Further downstream, there are several bridges crossing the Brazos River as it flows to the Gulf of Mexico. Bridge and culvert data was not available in the existing CWMS model, therefore was not used.

Storage Areas

Storage areas were added for the 1.5 PMF run to accommodate the large breach flows that exceeded the 2D RAS mesh.

Tributaries

Tributaries that experience backwater flooding or a lateral transfer of flow during the flood wave were modeled as part of the 2D domain. Tributaries modeled included portions of the Navasota, San Gabriel, Little, Brazos, Aquilla rivers, and Yegua Creek.

Whitney Dam and Downstream Structures

Whitney Dam was modeled using a storage area/2D connection. The location of the structure was extracted from aerial photography. The 2019 survey was used to input the station-elevation data. The reservoir pool behind the dam was modeled with a storage area. The current elevation-storage relationship is shown in the WCM, and was used in the HEC-RAS model for the reservoir. The controlled spillway was modeled as a gate with a known rating curve provided in the 2019 Whitney Dam WCM. The model includes gate operation based on pool elevation.

Whitney Dam is operated to control floods between the dam and mouth of the Brazos River. It is also one of the systems of reservoirs operated to provide flood damage reduction in the Brazos River Basin. Other dams in this system are the Morris Sheppard, De Cordova Bend, Waco, Aquilla, Sterling C Robertson, Proctor, Belton, Stillhouse Hollow, North San Gabriel, Granger, and Somerville dams. The only dam that was included model domain was Waco Lake. This reservoir already existed in the 2D model and was left in.

The Lake Brazos Dam is located downstream of Whitney Dam on the main stem of the Brazos River. The dam is owned and operated by the City of Waco, Texas. The Lake Brazos Dam was originally completed in 1970 and substantially modified in 1988. In 2007, a labyrinth weir was built to replace the existing dam providing a significantly more reliable dam with considerably less maintenance than the existing structure.

Lake Brazos Dam consists of a 3,000-foot-long labyrinth weir and was constructed within the footprint of the existing dam location on the Brazos River, just downstream of the Baylor University Stadium and east of Interstate-35. An outlet structure was built on the right embankment of the labyrinth weir. The new outlet works consists of a single 10-foot by 10-foot sluice gate.

The purpose of the Lake Brazos Dam is to provide an in-channel recreation reservoir on the Brazos River near downtown Waco, Texas. Lake Brazos Dam creates Lake Brazos, which has a volume of 5,600 acre-feet of water that Waco, Texas, utilizes for municipal purposes. When the river flows over the labyrinth weir, it drops 8.5 feet from the labyrinth platform at elevation 363.5 feet into the river channel at elevation 355.0 feet. The downstream impact slab is 15-feet wide, extending the length of the labyrinth platform area.

Levees

There are several known local levees in the system identified in the National Levee Database (NLD). Levees identifiable in the system were included in the model as either a storage area/2D connection (if elevation data was available in the NLD) or a breakline (if elevation data was not available). Due to lack of elevation data, there were far more levees modeled as breaklines than as storage area/2D connections.

Model Calibration and Downstream Boundary Conditions

The extent of model calibration for this analysis included verifying that non-damaging discharges remained within channel banks as well as checking the model for reasonableness at various downstream gages using available United States Geological Survey (USGS) gage data.

When entering the Gulf of Mexico, flow is generally contained within the Brazos River Basin. The downstream boundary condition was set to mean high-water level, MHW (elevation 0.88 feet) for the breach analysis and mean tide level, MTL, for the frequency, 1957 event, and the controlled flows. USGS gage ID 8772471, Freeport Harbor, TX.

For the Max High scenario, the 2015 updated PMF was used as the inflow hydrograph. All other scenarios were scaled to achieve the desired peak pool elevation for the breach analysis. For the Frequency analysis and the 1957 event that were used for the alternatives, RiverWare model outputs were used for each scenario.

Dam performance and operation changes over the life of the project

Whitney reservoir has had three conservation pool raises in the life of the project. The first time was in 1968, two additional feet of flood storage was reallocated for the conservation pool. The second was in 1969 for one additional foot resulting in a pool elevation of 523.1 ft-NAVD88. The last was in 1972, an additional 10 feet was added to the conservation pool raising its current elevation of 533.1 ft-NAVD88. Three years prior, the construction of Granbury Lake upstream of Whitney started on September 15, 1969. One third of Whitneys watershed was transferred to this reservoir with a total capacity of 136,326 acre-feet. As this does reserve some of that runoff from the watershed to be allocated to Granbury Lake, this reservoir does not have flood storage. Therefor during large events, additional flood water will be passed through Granbury Lake and routed to Whitney reservoir essentially becoming a run of river project after the conservation pool. All other changes were temporary deviations that do not apply to today's operations.

3. Alternative Descriptions

3.1. Whitney Reallocation Alternatives

The following sections summarize the alternatives that were modeled in HEC-RAS to determine the risk of reallocating Whitney's reservoir pool for additional water supply.

Alternative Descriptions

8 Alternatives were considered for the Whitney reservoir pool reallocation. Alternative 1 looked at existing conditions without change, Alternative 2, 2a, and 2c looked at redistributing the allocation of hydropower and water supply in a range of percentages within the conservation pool (elevation 520-533 ft-NAVD88), alternative 3 and 5 looked at increasing the conservation pool, alternative 4 looked at lowering the bottom of the power pool to increase water supply, and alternative 6 was a combination of lowering the bottom of the power pool and redistributing the percent allocated to water supply and hydropower. A full list of the alternatives are provided in Table 3-1.

Table 3-1: Alternative Descriptions

Alternative	Description
Alt 1	Existing Condition
Alt 2	Conservation pool: 2/3 water supply and 1/3 hydropower
Alt 2a	Conservation pool: 50/50
Alt 2C	Conservation pool: Energy focused, 34% water supply and 66% Hydropower
Alt 3	Flood Pool: Increase 3ft
Alt 4	Conservation pool: decrease bottom of conservation pool by 1.6 ft
Alt 5	Combo: Conservation pool (50/50) and Flood increase 1.5ft
Alt 6 (2a+4)	conservation pool: 50/50 split and lower the conservation pool by 8 ft

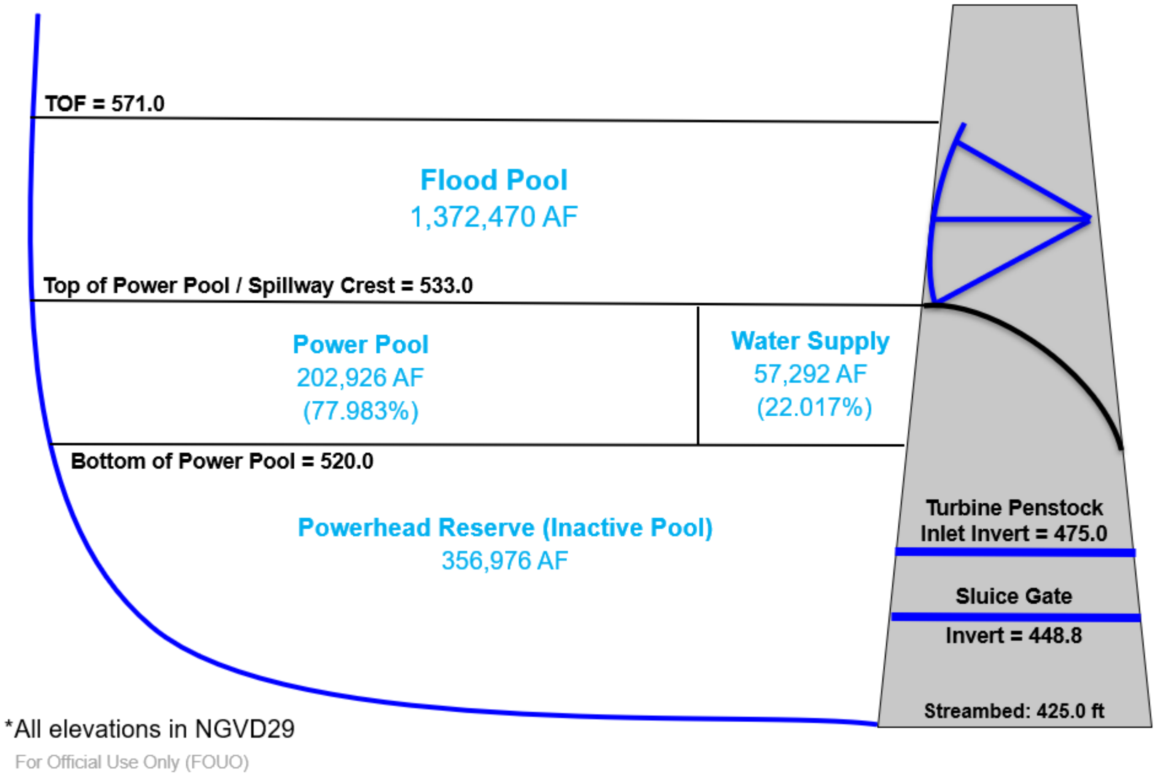

Figure 3-1: Current Pool Allocation

Data

A comprehensive river and reservoir modeling software analysis was used to develop the datasets for the 8 alternatives, RiverWare. Each alternative was set up to optimize the flood, hydropower, and water supply releases based on pool allocations and a set rule curve for the conservation pool. A period of record from 1938 through 2022 and two historical storm events were routed through the reservoir simulation model and used in the 6 hydrologic and hydraulic analysis for this study (Appendix C).

The period of record inflow was used to develop the elevation duration curves, inflow volume frequency curve, and the stage frequency curves for this analysis. The first historical event, November 2015, was scaled to represent a family of frequency inflows events that represented the 10, 1, 0.5, and 0.2 percent annual chance of exceedance (ACE) storm events using the RMC-RFA frequency analysis software program. This was used to determine the performance of the reservoir with each alternative. The second historic storm event used was the 1957 pool of record event. This was a series of events that were equivalent to a 1% ACE inflow volume followed by a 10% ACE, and a 3.6% ACE rain event. This resulted in an outflow greater than the 0.2% ACE event and was a good example to test the system response of the reservoir using the 8 alternatives reservoir operation rules.

To develop the inflow frequency curve a statistical analysis program, HEC-SSP was used. RMC-RFA was also used to develop the stage frequency curve.

4. Hydrologic and Hydraulic Analysis

The following section covers the hydrologic and hydraulic analysis preformed for this study. USACE uses a risk-informed approach where risk and deterministic design standards are used to provide a more informed assessment. For this study, six objectives were reviewed to help inform the expected risk for each of the alternatives.

- Frequency analysis was developed to determine the capacity of the reservoir and when we would expect to reach surcharge releases
- Reservoir response from a historic storm event with the proposed conservation pool operations for each alternative was analyzed to observe the performance of the reservoir
- Stage frequency curve was calculated to determine the probability of expected pool elevations
- Elevation duration analysis was developed to determine percent of time a pool elevation will be at or above with each of the alternatives
- Monthly average pool elevations were calculated to determine impacts on environmental, real estate, and inundation on gates and other structures (includes boat ramps, campgrounds, picknick areas)
- Breach analysis to determine risk of failure and normal operations in high pools, surcharge releases

These objectives will help evaluate the proposed alternatives to determine if it is an acceptable risk to push forward to the final selected alternatives

4.1. Frequency Analysis

This frequency analysis was used to estimate the response of Whitney Reservoir to various magnitude precipitation events. A large historic storm event from 2015 was chosen for this evaluation. The inflow volume was scaled to represent a 10, 1, 0.5, and 0.2 percent annual chance of exceedance (ACE) event, as seen in Figure 4-1. The inflow hydrograph was first tested in a Hydrologic model, HEC-HMS to determine if it would reach a critical elevation in the flood pool at or above 571.1 ft-NAVD. Once the critical elevation is reached, operations will switch from controlled releases based on downstream capacity to uncontrolled based on inflows. This initial simulation filtered the events that had the potential to cause damaging flows from the ones that would not. From there, the critical frequency hydrographs were routed through the reservoir simulation model, RiverWare. This model simulated a system response with multiple reservoirs and the operations for each alternative to provide a more detailed analysis. Once the operations were established from the reservoir operation model, they were routed through a hydraulic model to define the resulting inundation flooding extents.

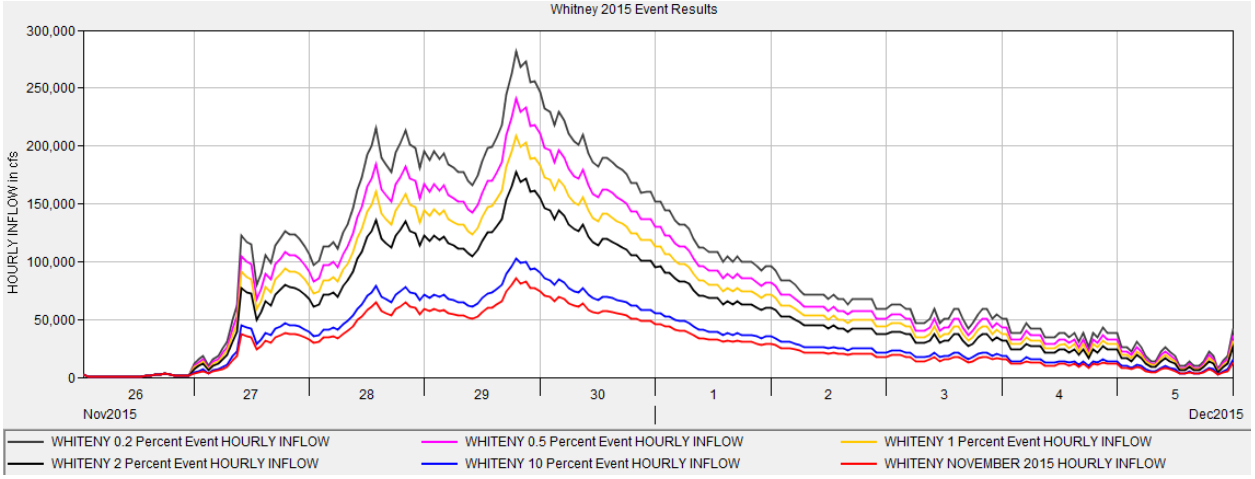

Figure 4-1: 2015 Storm Event Scaled to fit the 10-0.2 ACE Frequency Volumes

Detailed Results for the Frequency Analysis

The preliminary check in a hydrologic model, HEC-HMS, was used to determine the general capacity of the reservoir. The reservoir was set at the highest proposed pool elevation, 536.1 ft-NAVD88 alternative 3, and then the frequency storm events were routed into the reservoir to determine if a critical elevation was reached resulting in damaging flows. Any resulting pool elevation below the surcharge pool 571.1 ft-NAVD88 would have controlled releases below the channel capacity downstream of 25,000 cfs. Anything above this elevation would result in uncontrolled flows equivalent to inflow calculations to maintain the pool elevation without exceeding the top of surcharge pool 573.1 ft-NAVD88 (max design pool elevation). Table 4-1 shows the results from the HEC-HMS model. Events below the 500-year event stayed below the critical elevation of 571.1, controlled non-damaging releases. The o.2 ACE event exceeded the critical elevation causing releases above the downstream control point of 25,000cfs.

Table 4-1: Hydrologic Results from the Preliminary HMS Model

Inflow Frequency Event (ACE)	Peak Pool Elevation (ft-NAVD88)	Peak Inflow (cfs)	Peak Release (cfs)	Surcharge Duration (hr)
10	545.6	103,153	25,000	0
1	563.1	208,923	25,000	0
0.5	567.7	240,486	25,000	0
0.2	571.2	281,411	64,775	65

A more detailed analysis on the 0.2 ACE event was conducted using a reservoir simulation model (RiverWare) in the Brazos system and a hydraulic model (HEC-RAS) with the full array of alternatives. The level of detail that this analysis provided that the HEC-HMS model did not, includes operation decisions for each of the alternatives and a response in a system that included eight other reservoir release decisions. Releases are determined by downstream constraints and priority is given to reservoirs with higher percentage of flood storage. Alternatives 1, 2, 2a, 2c and 4 all resulted in releases that were very similar, and the results were negligible. For this reason, they were all represented by alternative 1 in the rest of this section. Alternative 3 and 5 included raising the conservation pool, reallocating flood storage for water supply. Alternative 3 increased the conservation pool by 3 feet and alternative 5 increased the pool by 1.5 feet. The simulation set the starting pool at these elevations (proposed top of conservation pool for each alternative). The result in releases increased by nine percent for alternative 3 and four percent for alternative 5 relative to alternative 1. The full results from the reservoir simulation model can be found in Table 4-2.

Table 4-2: Frequency Whitney Reservoir Results

Model Location	**Scenario**	**Conservation Pool (ft-NAVD88)**	**Peak Pool Elevation (ft-NAVD88)**	**Max Release out of Whitney (cfs)**
Whitney Reservoir (WTYT2)	Alt 1	533.1	571.1	49,200
	Alt 2	533.1	571.1	49,200
	Alt 2a	533.1	571.1	49,200
	Alt 2c	533.1	571.1	49,200
	Alt 3	536.1	571.2	53,700
	Alt 4	533.1	571.1	49,200
	Alt 5	534.6	571.1	51,200
	Alt 6	533.1	571.1	49,200

A hydraulic model was used to create the inundation mapping for the three alternatives 1, 3, and 5. Reservoir releases from all nine reservoirs were added to the hydraulic model to represent system wide operations. For the final inundation extents, Alternative 1, 3 and 5 resulted in very similar results with some variation in the releases out of Whitney. The other reservoirs operated the same with each alternative. Figure 4-2 shows the releases from Whitney Lake for all three alternatives. The largest was 53,700 cfs from alternative 3, the highest proposed conservation pool at elevation 536.1. Alternative 5 resulted in a max release of 51,200 and alternative 1 had a max release of 49,200. The inundation mapping from these three alternatives resulted in a nominal increase. Raising the pool 3 and 1.5 feet for alternative 3 and 5 resulted in a 1.8 and 1.5 percent inundation area increase. Both were considered nominal and are shown in Figure 4-3 through Figure 4-5. Because all three results were very similar, only the minimum and maximum alternatives were mapped, Alt 1 and 3. Pink represents the maximum extents, and light blue represents the minimum.

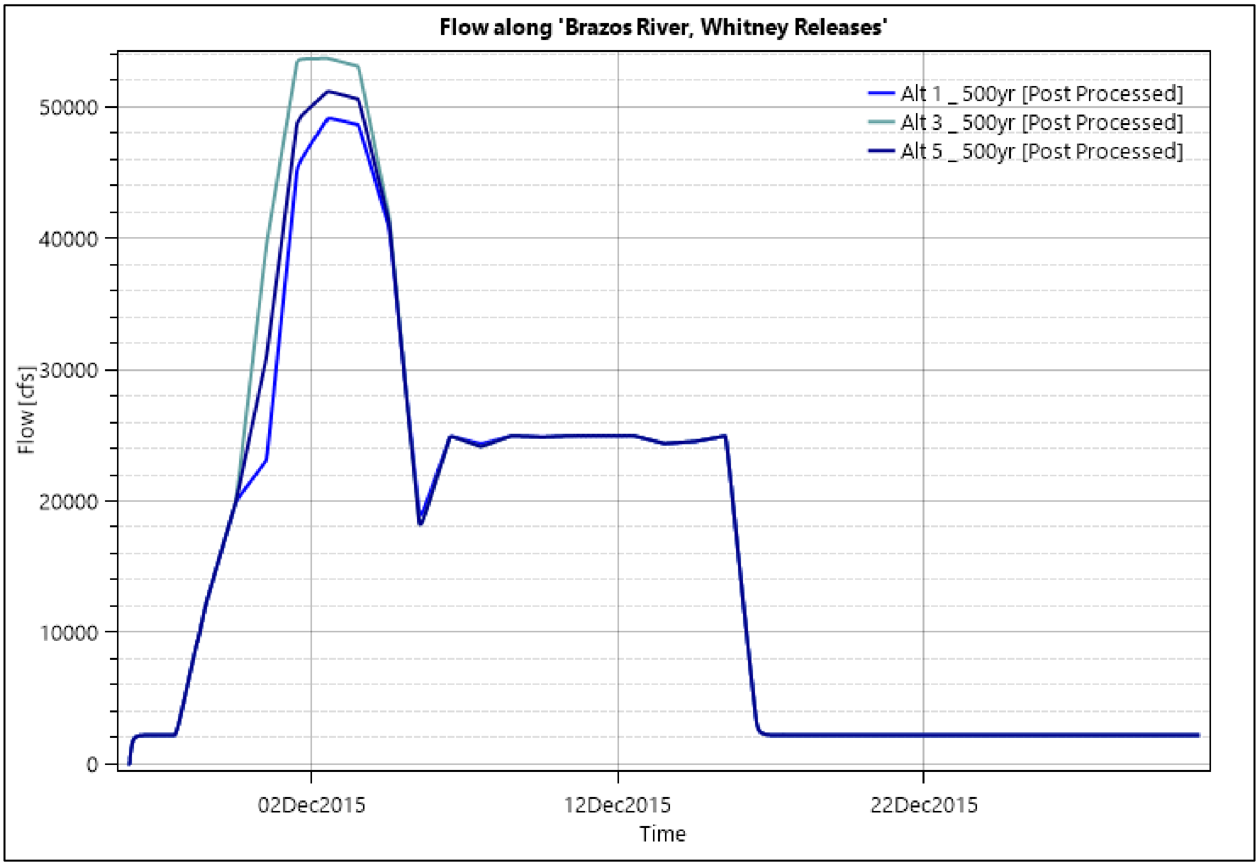

Figure 4-2: Releases from Whitney Reservoir.

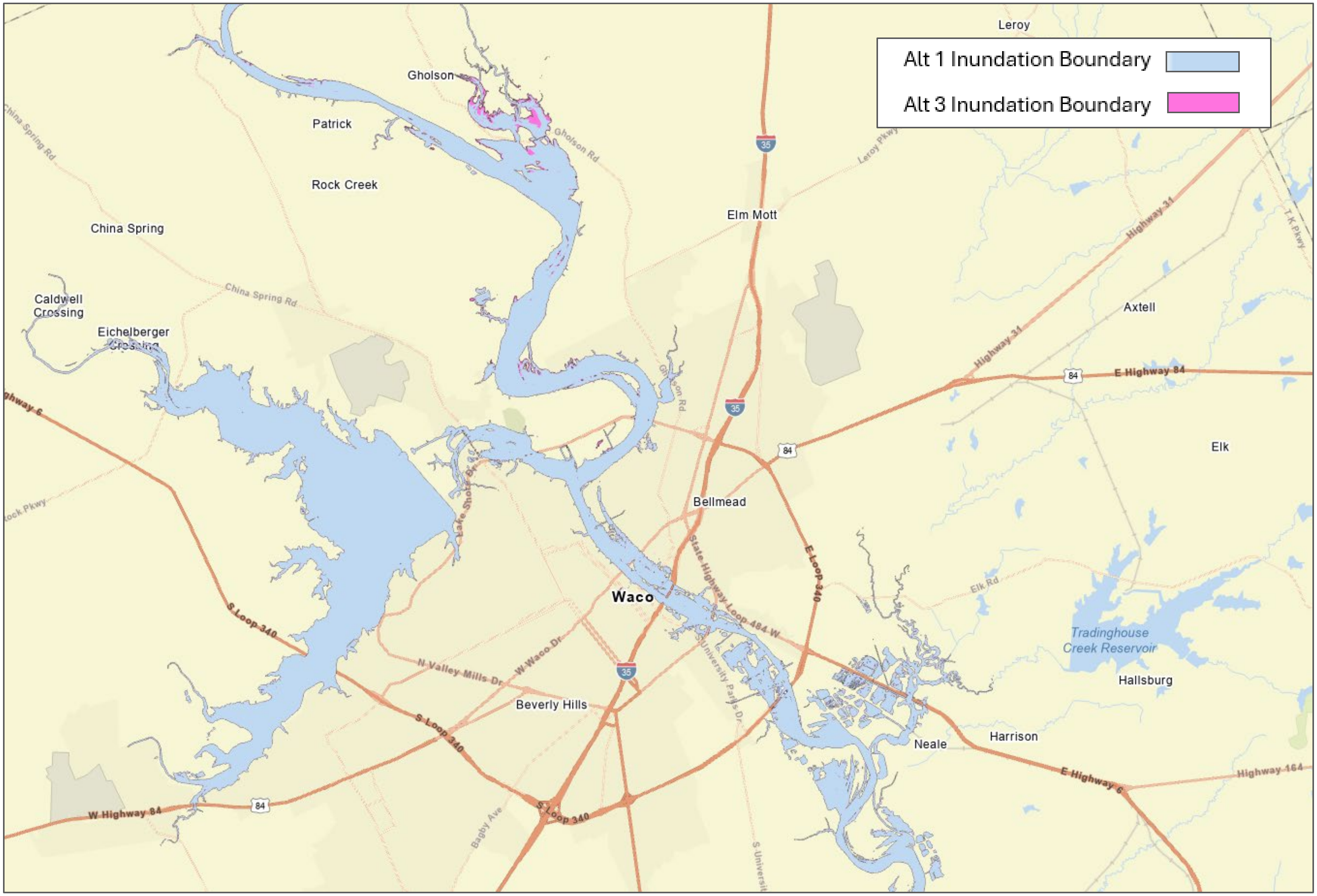

Figure 4-3: Inundation Map for the City of Waco, Frequency Analysis

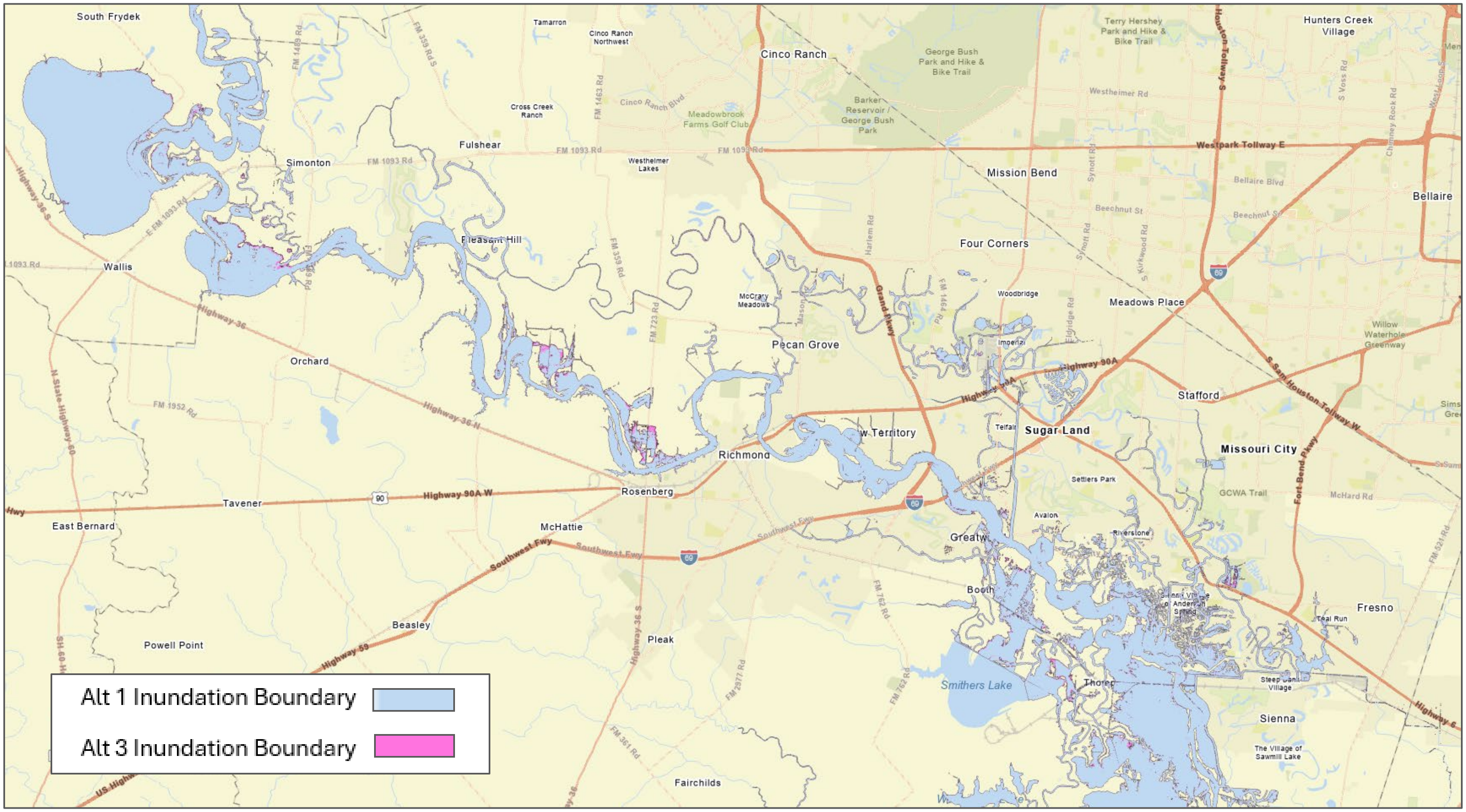

Figure 4-4: Inundation Map for the City of Sugarland, Frequency Analysis

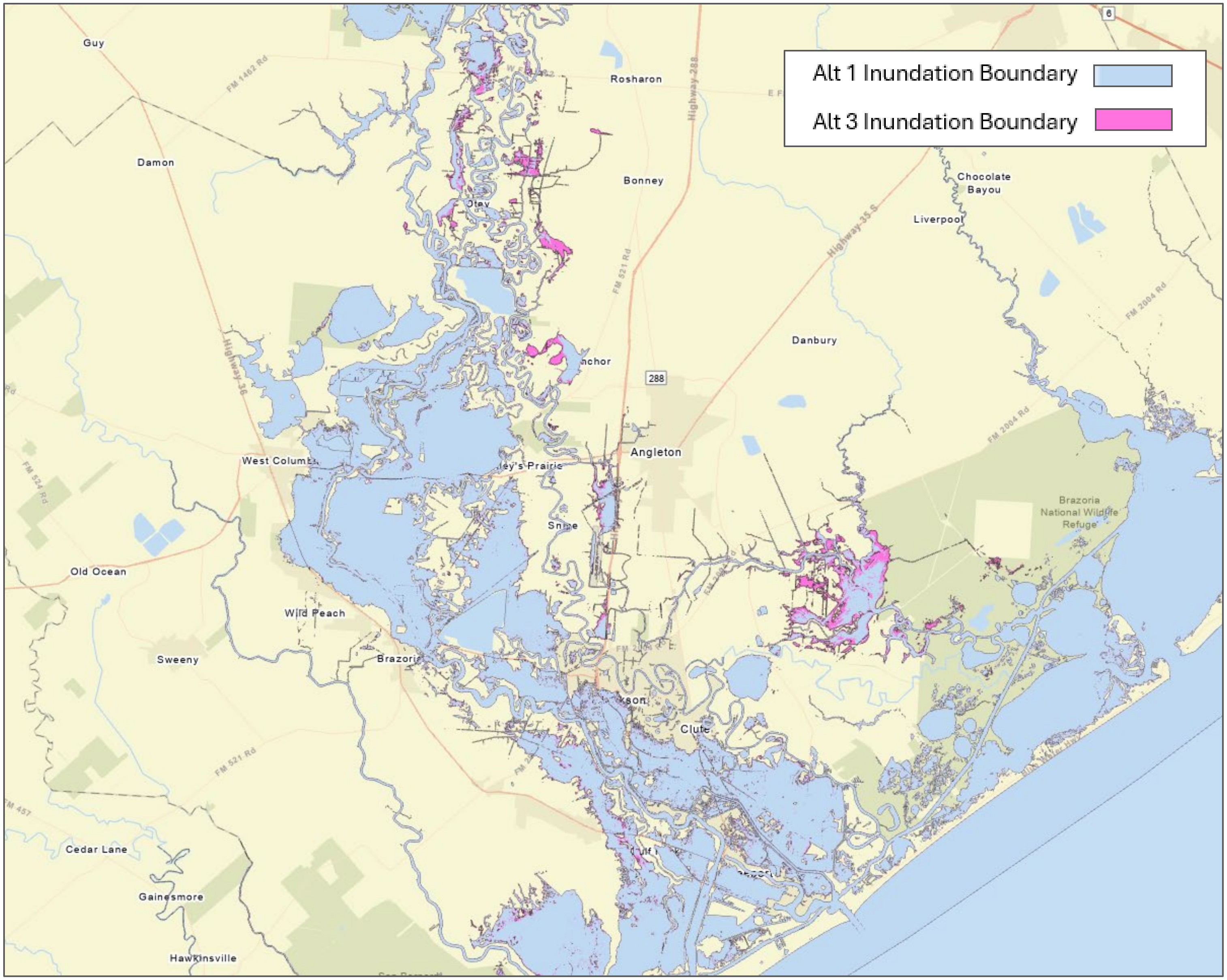

Figure 4-5: Inundation Map for the Coastal Region, Frequency Analysis

4.2. 1957 Event

This Alyases evaluates the response of the reservoir to the period of record historical event in 1957. A series of precipitation events that were less than the 0.2% annual chance of exceedance (ACE) return intervals filled the capacity of flood storages causing uncontrolled surcharge releases in 1957. This is very common in the spring to have multiple events therefore was added to this study to better define capacity of Whitney reservoir. The 1957 series of events included a 1% ACE followed by a 10% ACE, and then a 3.6% ACE event with a 9-day average inflow of 82,000, 42,000, and 63,000 respectively (based on the inflow volume frequency curves as seen in Figure 4-6 and Figure 4-7).

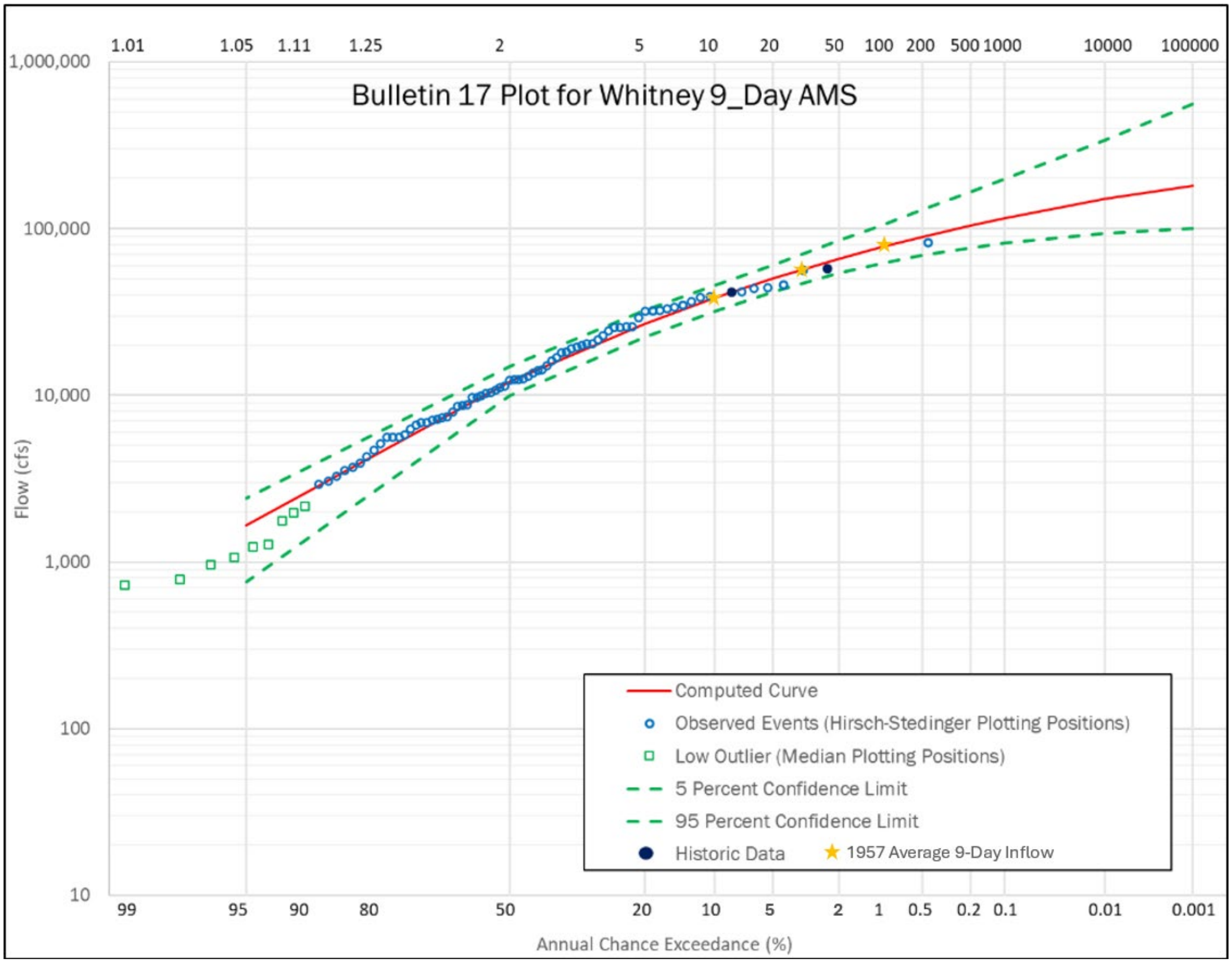

Figure 4-6: Inflow Frequency Curve

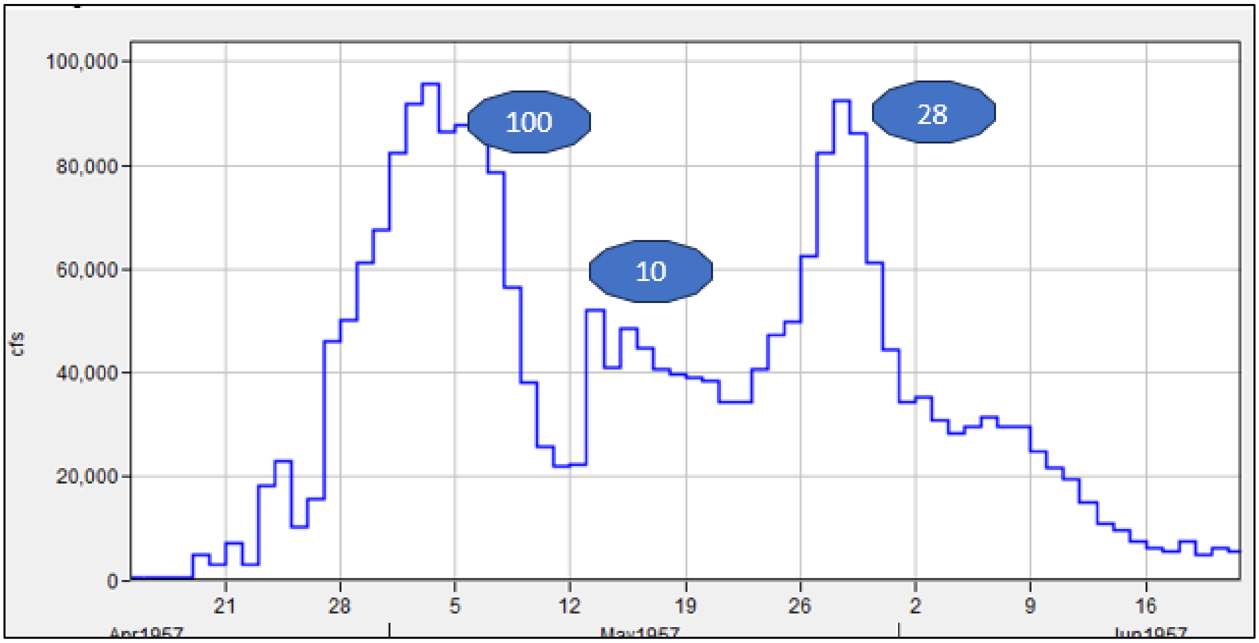

Figure 4-7 Inflow hydrograph for the 1957 historic event with corresponding frequencies for each storm event

The historic event was routed through the Brazos RiverWare model using the operations set up for the 8 alternatives. This model included 11 reservoirs in the Brazos basin to represent a full system response. Starting pool elevation were set to the same condition as what was observed in 1957. Releases from each reservoir were routed downstream using boundary conditions in the HEC-RAS model to simulate this event.

Detailed results for the 1957 Event

Table 4-3 shows the response downstream and within the reservoir from the 8 alternitives. From the period of record, the largest event was used to evaluate each alternatives, May 1957 event. This event pushed the pool into surcharge operations resulting in large releases that exceeded the control point downstream. Due to the unique operations of this reservoir, the results for each scenario were very similar. Alternative 1, 2, 2a, 2c, 4, and 6 have the same operations resulting in the same releases and flooding extents. Alternatives 3, and 5 represent the two proposed pool elevation raises for additional water supply storage. Both of these were very similar in releases and flooding extents. Figure 4-8 though Figure 4-10 illustrate anticipated inundation in the key location for alternatives 1 and 3. Alternative 5 was not shown, all three alternatives were very similar with nominal results, therefore only the maximum and minimum were shown.

Table 4-3: 1957 Whitney Reservoir Results

Model Location Lake Code	Scenario	Conservation Pool (ft-NAVD88)	Peak Pool Elevation (ft-NAVD88)	Max Release out of Whitney (cfs)
Whitney Reservoir (WTYT2)	Alt 1	533.1	571.3	78,296
	Alt 2	533.1	571.3	78,296
	Alt 2a	533.1	571.3	78,296
	Alt 2c	533.1	571.3	78,296
	Alt 3	536.1	571.3	79,707
	Alt 4	533.1	571.3	78,296
	Alt 5	534.6	571.3	78,907
	Alt 6	533.1	571.3	78,296

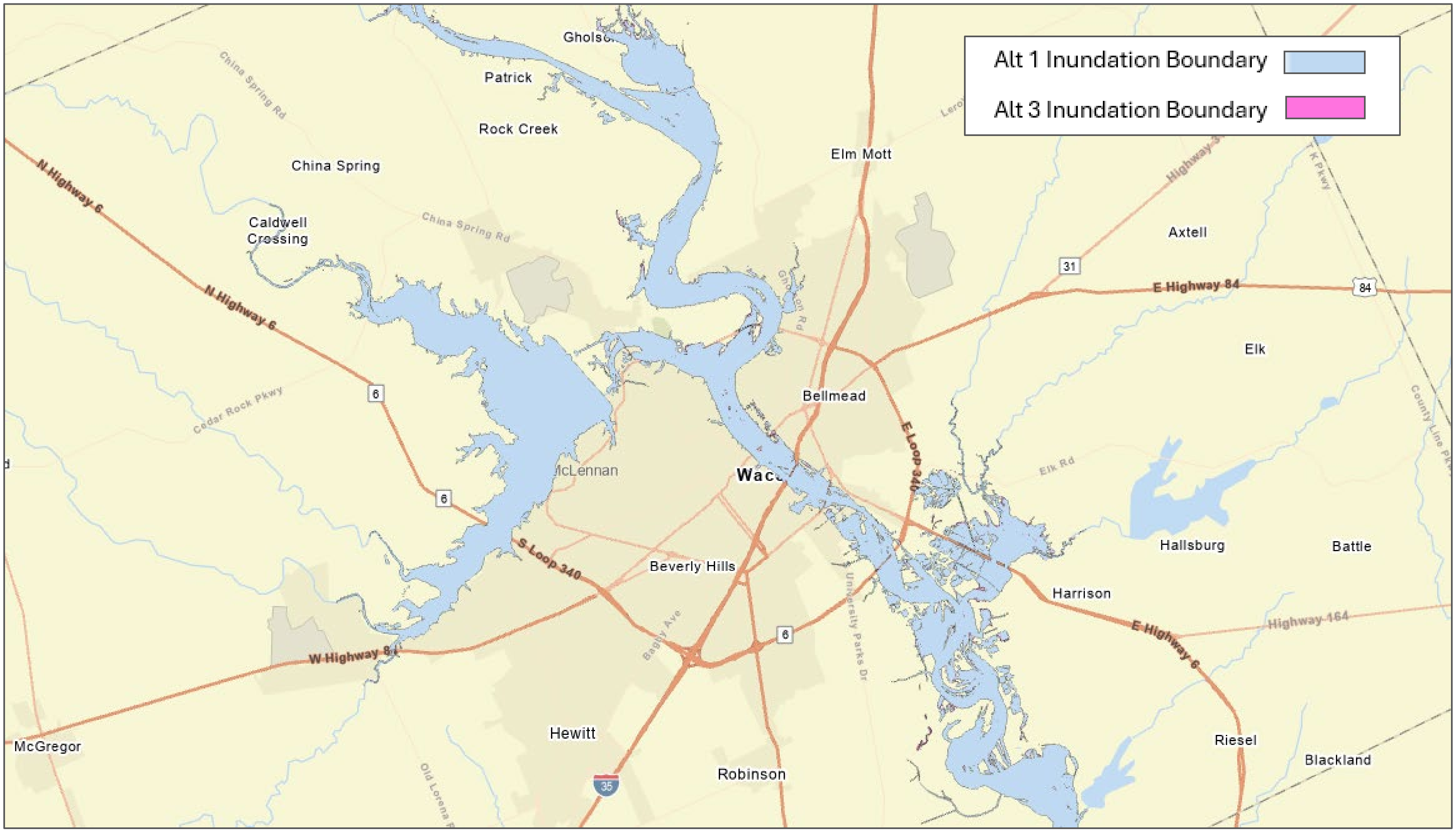

Figure 4-8: Inundation Map for the City of Waco, 1957 Storm Event

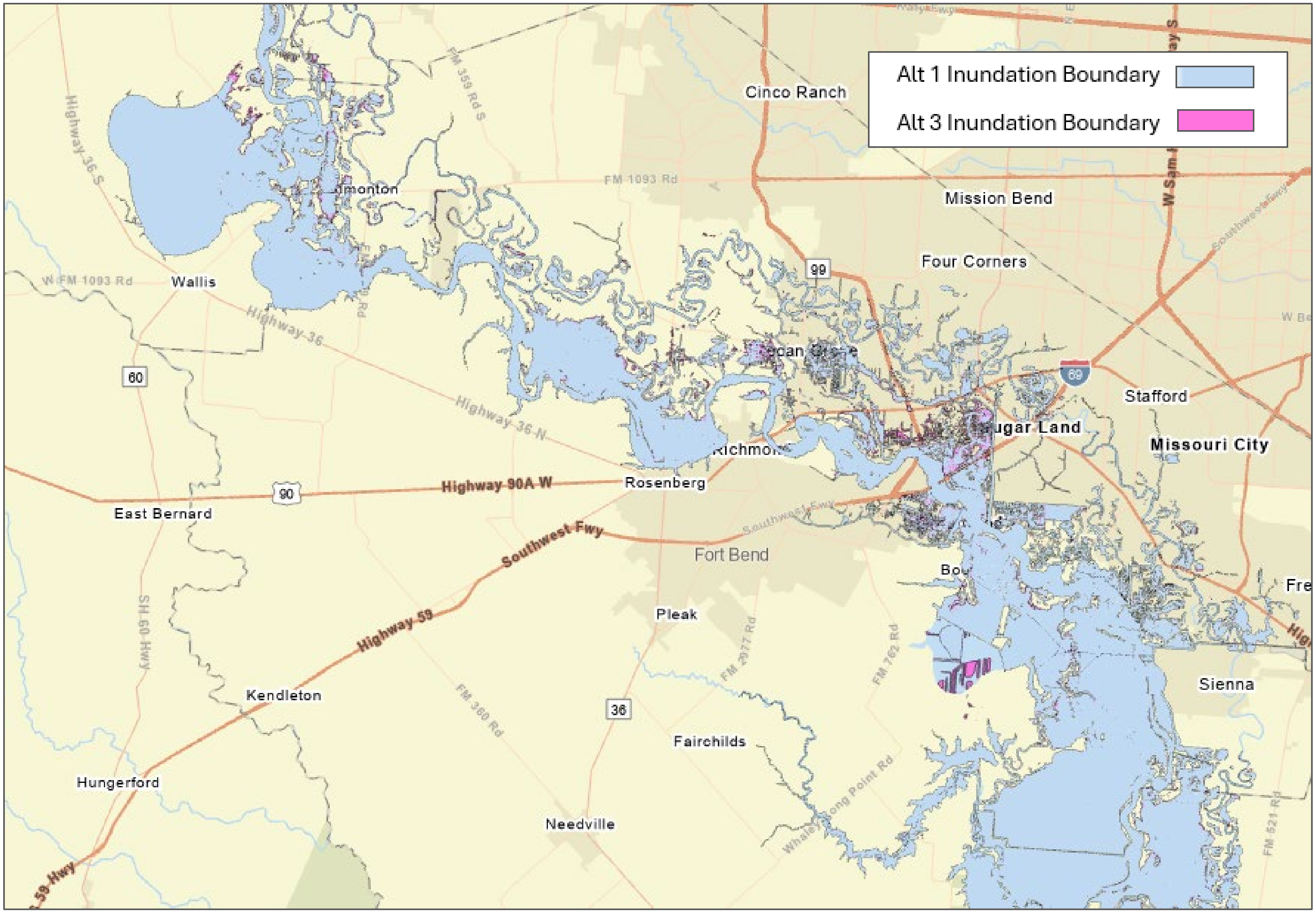

Figure 4-9: Inundation Map for the City of Sugarland, 1957 Storm Event

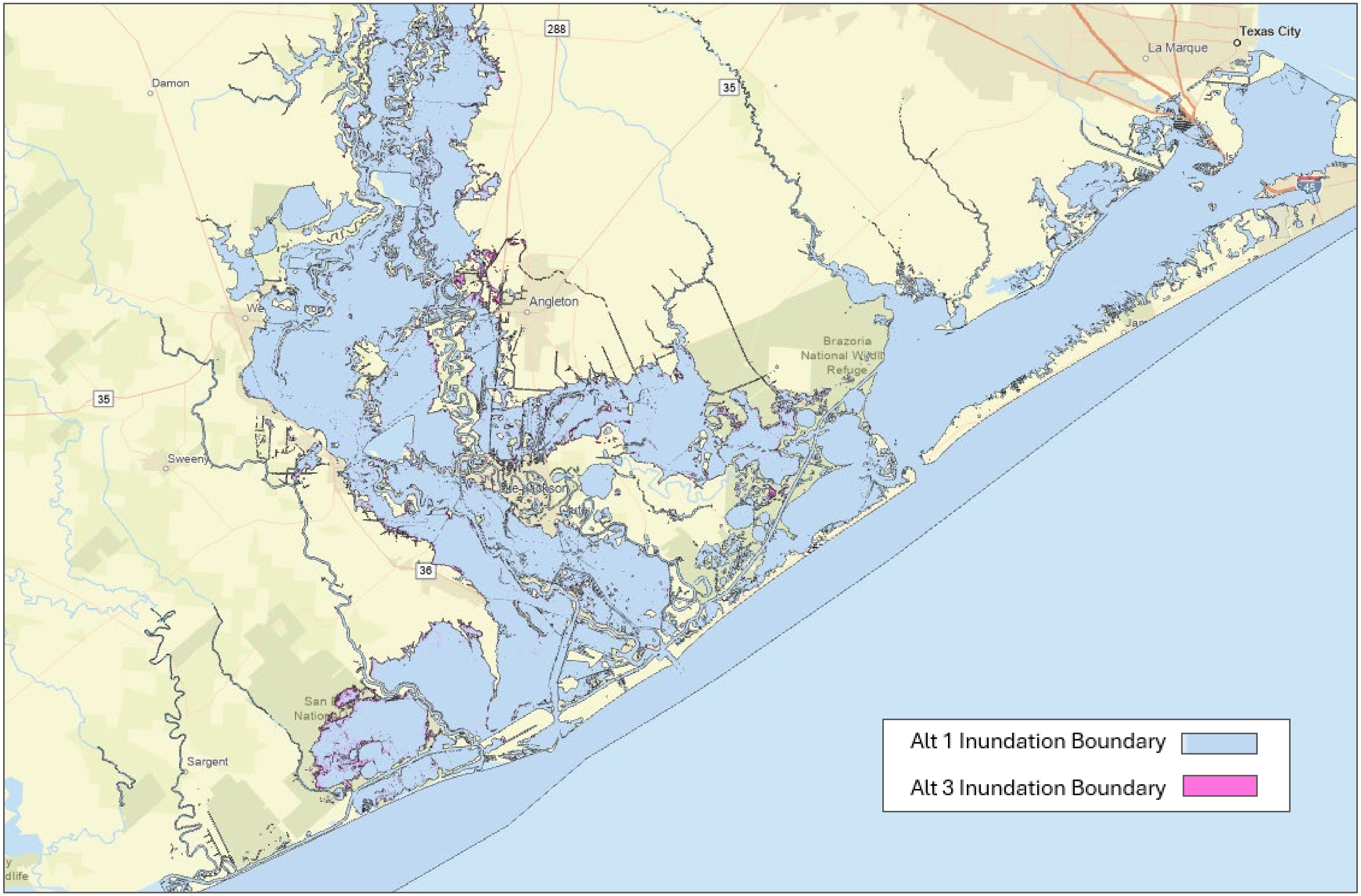

Figure 4-10: Inundation Map for the Coastal Region, 1957 Storm Event

4.3. Stage Frequency Curve

A stage frequency curve was developed for this study to assess the likelihood of the pool elevation (stage) reaching critical levels under each alternative. The operational rules from the Whitney Reservoir Water Control Manual were incorporated into the RiverWare reservoir simulation model for each scenario. This enabled the generation of elevation data for Whitney Reservoir for the period of record from 1938 through 2022. In addition to flood operations, current water supply and hydropower operations were integrated into the simulation to reflect existing conditions. From the RiverWare simulation results, Alternatives 1, 2, 2a, 2c, and 4 all had similar results that were concluded to have nominal variations, therefore were all represented as Alternative 1.

For Alternatives 1, 3, and 5, the conservation pool elevations were set at 533.1, 536.1, and 534.6 ft-NAVD88, respectively. RiverWare simulated reservoir elevations based on these conservation pool levels and the associated rule curves, which govern the release of flood storage above the conservation pool and manage water supply and hydropower operations below it.

The simulated daily elevation data from 1938 to 2022 period of record were used to develop the stage frequency curve. This dataset was converted into an Annual Maximum Series (AMS), which was ranked and plotted using the Weibull plotting position method within the RMC-RFA tool.

Figure 4-11 presents the resulting stage frequency curve. Below elevation 570.1 ft, the pool elevations vary across the different alternatives. However, above 570.1 ft, the curves converge into a single frequency line. This convergence occurs due to the surcharge operations of the reservoir, where a controlled percentage of inflows is released to preserve the structural integrity of the dam. This value is consistent across the alternatives.

Table 4-4:Frequency Analysis Results

Scenario	Elevation (ft-NGVD88)	Annual Exceedance Probability (AEP)		
		Alt 1	Alt 3	Alt 5
Overtopping Failure	585.1	1/378,144	1/373,700	1/374,700
Top Concrete Dam	584.1	1/282,025	1/278,600	1/278,900
1.5 x PMF	584	1/273,466	1/270,100	1/270,400
Top Embankment	580.1	1/92,292	1/93,500	1/94,000
Low Spot of Dam	579.51	1/82,921	1/84,100	1/84,500
PMF	577.9	1/62,087	1/63,200	1/63,400
Intermediate High Pool	571.9	1/15,315	1/14,700	1/14,800
Top of Active Storage Pool	571.1	1/3,685	1/3,100	1/3,300
Pool of Record	570.35	1/271	1/145	1/165
Security Pool	544.8	1/5	1/4	1/4
Normal High Pool	533.5	1/2	1/2	1/2

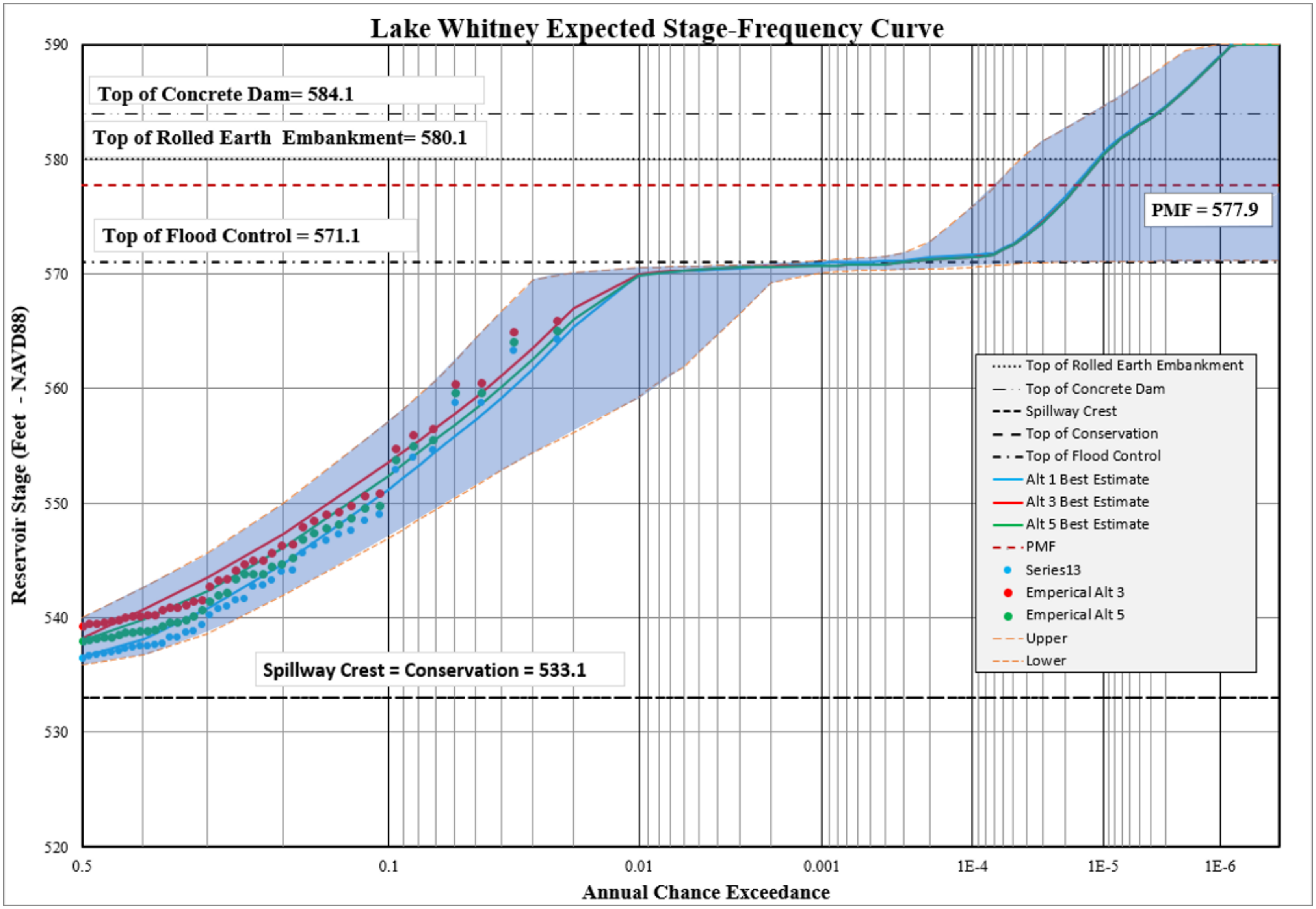

Figure 4-11: Annual Chance of Exceedance Plot

4.4. Elevation Duration Analysis

A pool stage-duration plot was developed for Alternatives 1, 3, and 5. This data shows the percent time an elevation reached or exceeded a specific pool elevation. Operations for the reservoir were simulated for each of the alternatives using RiverWare, reservoir simulation model. The elevations are then sorted from lowest to highest to show percent time the reservoir would be at or exceed that elevation. Alternatives 1, 2, 2a, 2c, and 4 plotted similar enough to be represented by Alt 1, there was not a significant change between those datasets. Figure 4-12 Shows Alternative 1, 3, and 5 with proposed conservation pool elevations, 533.1, 534.6, and 536.1 respectively.

Key elevations that were evaluated were the following:

- Conservation pool elevation for each Alternative (533.1 Alt 1, 536.1 Alt 3 and 534.6 alt 5)
- Spillway crest at 533.1
- Half of a foot of water on the gates, 533.6
- Four boat ramps are closed, and Whitney Ridge Marina is impacted 534.6
- Additional boat ramps, picnic sites, and campsites begin to close 535.6
- Top of flood control pool 571.1

It was found that the three alternatives, 1, 3, and 5 had a significant change in elevations that ranged from 523 to 545 and then converged as it approached the top of the flood control pool at 571.1. For the conservation pools, it was found to be at or above that elevation approximately 22 to 23 percent of the time, as seen in Table 4-5. For the spillway crest, an increase from 23 to 82 percent of the time was observed when the conservation pool was raised from 533.1 to 536.1. This would indicate water would be at the spillway crest touching the back of the gates for 82 percent of the time for Alt 3 and 70 for Alt 5. The potential for boat ramps and picknick areas closures also saw a significant increase for Alternatives 3 and 5, as seen in Table 4-5.

Table 4-5: Elevation Duration Results Percent Time Equal or Exceeded

Elevation	Description	Alt 1	Alt 3	Alt5
Conservation Pool	Proposed conservation pool for each alternative	23%	22%	22%
533.1	Spillway crest	23%	82%	70%
533.6	Impact due to erosion, wave action on banks of the reservoir	18%	80%	64%
534.6	Four boat ramps are closed, and the Ridge Merina is impacted	11%	71%	22%
535.6	Additional boat ramps, picnic sites, and campsites begin to close	4%	54%	14%
571.1	Top of flood control pool	< 0.1%	< 0.1%	< 0.1%

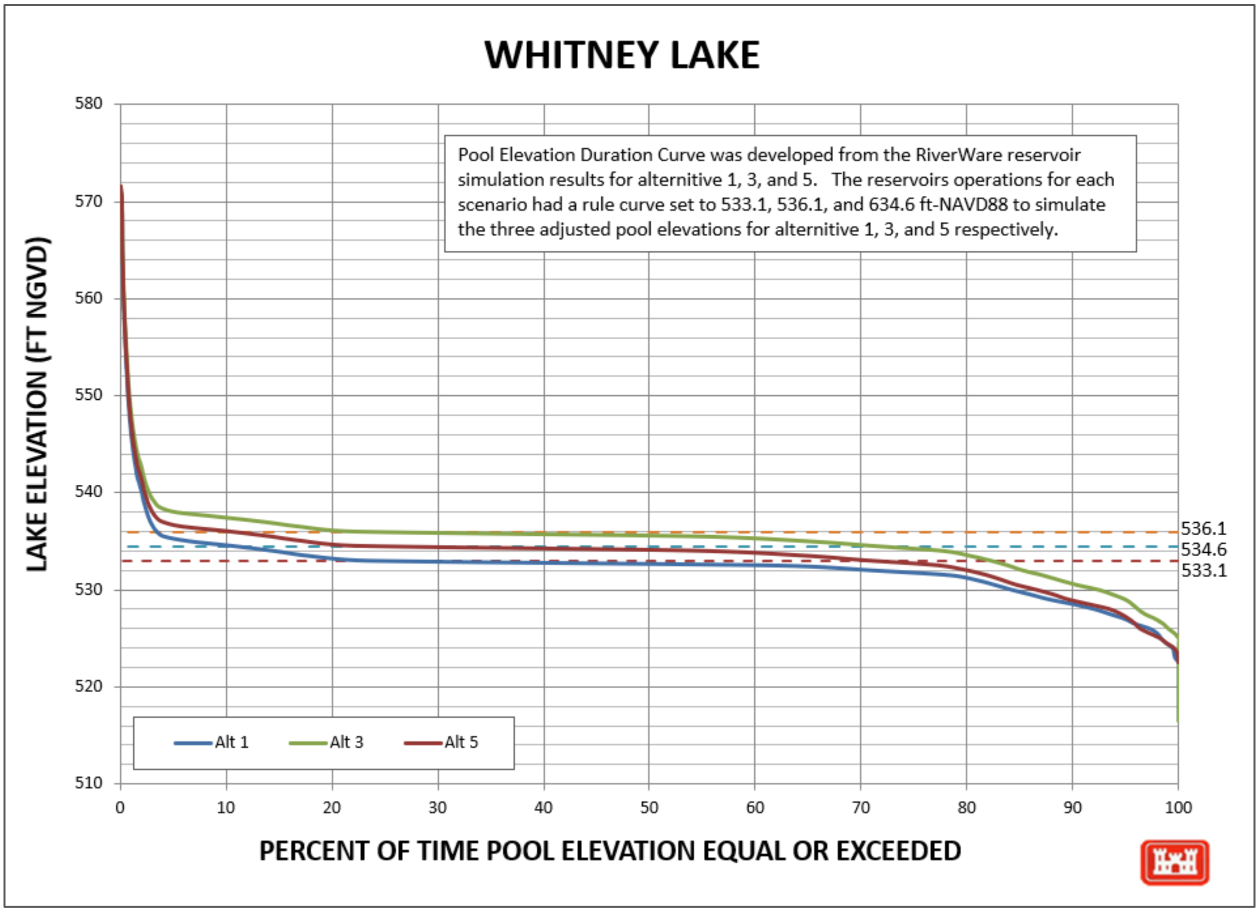

Figure 4-12: Elevation Duration Chart for Alternative 1, 3, and 5

4.5. Monthly Average Elevation

Monthly average reservoir elevations for each of the alternatives were calculated to provide information for environmental, real estate, and operational impacts. From an environmental standpoint, reservoir elevation influences shoreline habitat availability, aquatic ecosystem health, and water quality parameters such as temperature and nutrient distribution. For real estate, consistent elevation data help delineate property lines, manage floodplain risk, and guide development restrictions. In campground and recreational facility operations, elevation affects shoreline accessibility, dock usability, and safety planning, particularly during seasonal fluctuations. The results for Alternative 1, 2, 2a, 2c, 4, and 6 were relatively the same with a slight variation due to operations ranging from a tenth to four tenths of a foot. The two alternatives that had a raise in the pool elevation had a more significant change in the average monthly pool elevations. Alternative 5 had an average pool increase of 1.1 feet and Alternative 3 had an average pool increase of 2.6 feet as seen in Figure 4-13. A full array of the monthly average elevations that includes the Upper and lower 90 percentile can be found in Table 4-6.

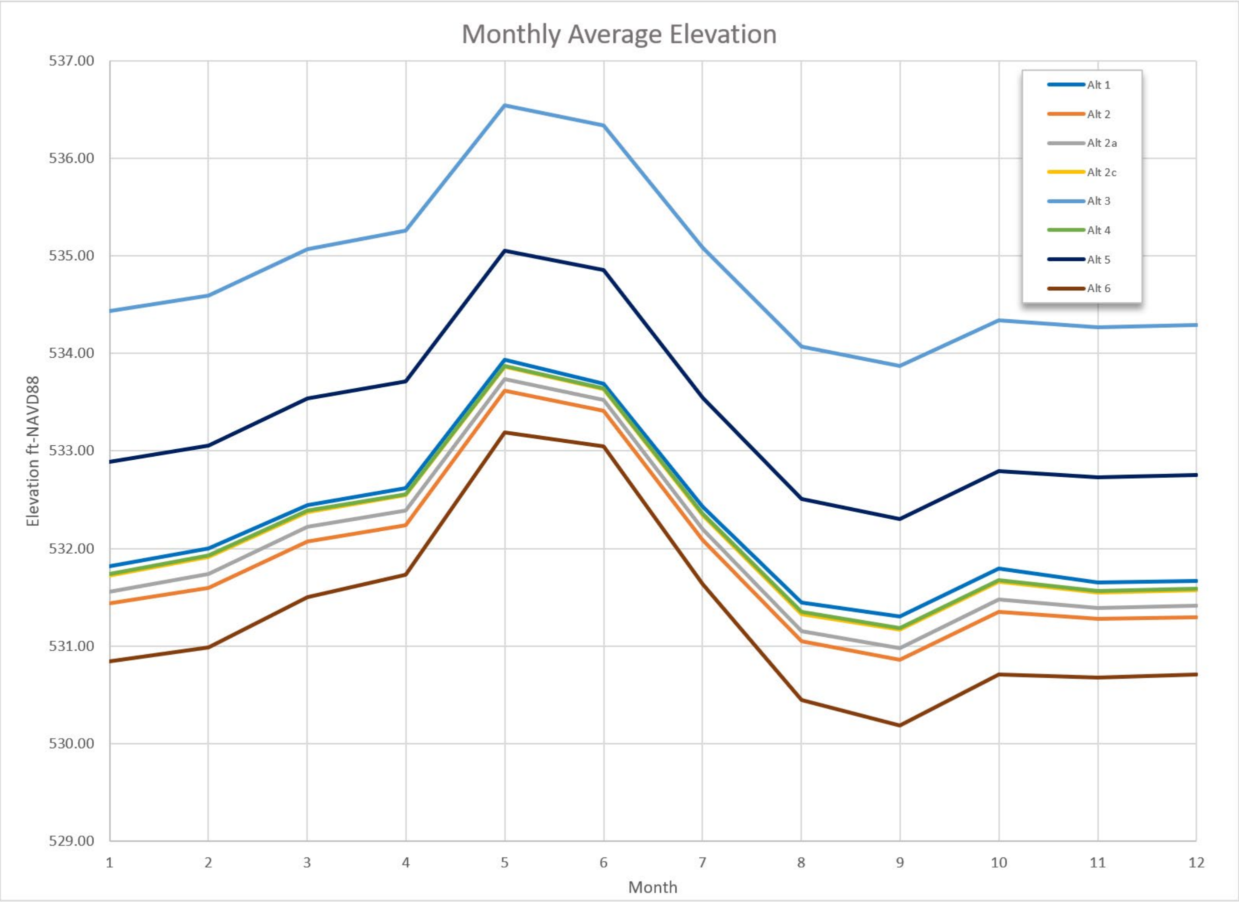

Figure 4-13: Monthly Averages Pool Elevations for Each Alternative

Table 4-6: Average, Standard Deviation, lower 5 percentile and upper 95 percentile Monthly Results for Each Alternative

	Alt 1				Alt 2				Alt 2a				Alt 2c			
Month	Ave	Std	5% LCI	95% UCI	Ave	Std	5% LCI	95% UCI	Ave	Std	5% LCI	95% UCI	Ave	Std	5% LCI	95% UCI
JAN	532.05	3.22	525.61	538.49	531.71	3.45	524.82	538.60	531.83	3.42	524.99	538.66	531.94	3.33	525.28	538.60
FEB	532.28	2.48	527.32	537.24	531.93	2.85	526.22	537.63	532.05	2.74	526.58	537.53	532.18	2.60	526.98	537.38
MAR	532.72	3.04	526.64	538.80	532.37	3.39	525.59	539.15	532.51	3.27	525.97	539.05	532.63	3.14	526.34	538.92
APR	532.80	2.35	528.09	537.50	532.46	2.81	526.85	538.07	532.60	2.66	527.29	537.91	532.71	2.49	527.72	537.69
MAY	534.25	5.31	523.62	544.88	533.95	5.55	522.86	545.05	534.06	5.46	523.13	544.99	534.16	5.38	523.40	544.91
JUN	534.00	4.98	524.04	543.95	533.73	5.20	523.34	544.12	533.82	5.11	523.60	544.05	533.91	5.03	523.85	543.97
JUL	532.58	2.97	526.64	538.52	532.26	3.23	525.79	538.72	532.37	3.18	526.01	538.74	532.48	3.07	526.33	538.63
AUG	531.55	2.14	527.28	535.82	531.16	2.32	526.52	535.80	531.27	2.36	526.55	535.98	531.40	2.26	526.88	535.91
SEP	531.42	2.51	526.39	536.44	531.01	2.65	525.70	536.32	531.12	2.72	525.68	536.56	531.25	2.65	525.96	536.54
OCT	531.91	2.86	526.19	537.63	531.50	3.02	525.46	537.53	531.62	3.06	525.51	537.73	531.75	2.99	525.77	537.72
NOV	531.80	2.59	526.62	536.98	531.45	2.80	525.84	537.05	531.56	2.83	525.90	537.21	531.67	2.74	526.20	537.15
DEC	531.88	2.81	526.26	537.50	531.54	3.04	525.45	537.62	531.64	3.03	525.58	537.71	531.76	2.94	525.87	537.65

	Alt 3				Alt 4				Alt 5				Alt 6			
Month	Ave	Std	5% LCI	95% UCI	Ave	Std	5% LCI	95% UCI	Ave	Std	5% LCI	95% UCI	Ave	Std	5% LCI	95% UCI
JAN	534.70	3.43	527.84	541.55	531.99	3.31	525.37	538.60	533.16	3.51	526.14	540.18	531.74	3.50	524.75	538.73
FEB	534.92	2.75	529.42	540.42	532.22	2.57	527.09	537.36	533.38	2.86	527.66	539.11	531.97	4.49	522.99	540.95
MAR	535.37	3.22	528.93	541.82	532.68	3.12	526.44	538.92	533.85	3.36	527.12	540.58	532.44	4.50	523.44	541.43
APR	535.47	2.69	530.09	540.85	532.76	2.45	527.86	537.66	533.94	2.81	528.32	539.57	532.53	4.45	523.64	541.42
MAY	536.86	5.19	526.47	547.24	534.23	5.37	523.49	544.97	535.39	5.41	524.56	546.21	534.00	4.38	525.25	542.75
JUN	536.65	4.83	526.99	546.31	534.00	5.01	523.99	544.02	535.15	5.04	525.08	545.23	533.77	4.31	525.14	542.40
JUL	535.26	3.10	529.06	541.45	532.59	2.98	526.62	538.56	533.73	3.24	527.25	540.20	532.31	4.26	523.79	540.83
AUG	534.19	2.40	529.39	538.99	531.53	2.09	527.35	535.71	532.62	2.48	527.66	537.59	531.18	4.21	522.76	539.60
SEP	534.02	2.77	528.48	539.56	531.40	2.50	526.39	536.41	532.45	2.83	526.79	538.12	531.02	4.10	522.82	539.23
OCT	534.48	3.08	528.33	540.64	531.79	2.95	525.90	537.68	532.94	3.15	526.65	539.24	531.52	4.01	523.50	539.54
NOV	534.44	2.89	528.66	540.21	531.71	2.69	526.33	537.09	532.90	2.95	527.00	538.79	531.46	3.90	523.66	539.27
DEC	534.53	3.10	528.34	540.72	531.80	2.91	525.99	537.61	532.99	3.16	526.66	539.32	531.56	3.83	523.90	539.21

4.6. Breach Analysis

A breach analysis is a standard assessment for all high hazard dam that has a potential for property damage or loss of life in the event of a failure. A series of hydraulic loading conditions were assessed; normal high, security scenario, top of active storage, intermediate high, max high, and 1.5 PMF overtopping. The following sections summarize the breach analysis for this study and the results.

Hydraulic Loading Conditions

The hydrologic loading conditions found both upstream and downstream of the project are needed to model dam breach and non-breach scenarios. Starting reservoir pool elevations, inflow conditions into the reservoir, and tributary flow into the modeled reach downstream of the dam can significantly impact consequence results. These inputs were determined using the observed hydrologic data, which included historic stream flow, reservoir pool elevation and discharge records, inflow design flood (IDF), and facility operation information from the WCM.

The period between March 7, 1973 and December 31, 2022 was used for the elevation duration curve. The conservation pool was raised on May 17, 1972, from 523.1 to 533.1 ft-NAVD88. March 7, 1973 is the date the raised pool filled for the first time. For the Security Scenario (1% chance of exceedance) and the Normal High pool (10% chance of exceedance) a peak pool of 544.8 and 533.5 was used respectively.

The sunny day breach scenario represents a static pool water surface elevation at the ten-percent exceedance duration elevation (NH pool). The routing was set up such that the inflow was equal to the discharge, resulting in a steady pool. Downstream discharge was 2,200 cfs, which is minimal and well below channel capacity.

The event-based loading conditions are based on inflow conditions determined from the best available IDF or design water elevation information received from the district. The design water elevation represents the maximum reservoir water surface elevation, including surcharge, which a dam is designed to accommodate, excluding any allowance for freeboard, and corresponds to the maximum storage. The IDF is defined as the flood hydrograph used in the design of a dam and its appurtenant works, particularly for sizing the spillway and outlet works, and for determining maximum temporary storage and dam height requirements.

Multiple IDFs may be available for a study dam. Older studies commonly refer to IDF as the spillway design flood (SDF) or reservoir design flood. Some dams may include an updated IDF based on the probable maximum flood (PMF) scenario. The MH pool scenario for Whitney Dam uses the district-provided PMF inflow hydrograph, which was scaled down to 95 percent to match peak pool (April 2019 Probable Maximum Flood Report and provided by Fort Worth District).

The IDF was routed in the HEC-RAS modeling using an unsteady pool routing and utilizing the rating curves for the outlet works and spillway gates shown in the WCM. The routing was started at the top of flood pool, consistent with the design routing from the WCM, and only the spillway rating was used for the routing. The WCM notes that the outlet works will be closed during spillway operations. The peak pool from the model/routing was elevation 516.9 feet, matching the PMF routing and had a peak discharge of 1,151,200 cfs, which is less than 1 percent greater than the design discharge.

The PMF hydrograph was scaled down to develop the IH, TAS, and SS pool scenarios. Figure 4-14shows the PMF inflow hydrograph used for this study.

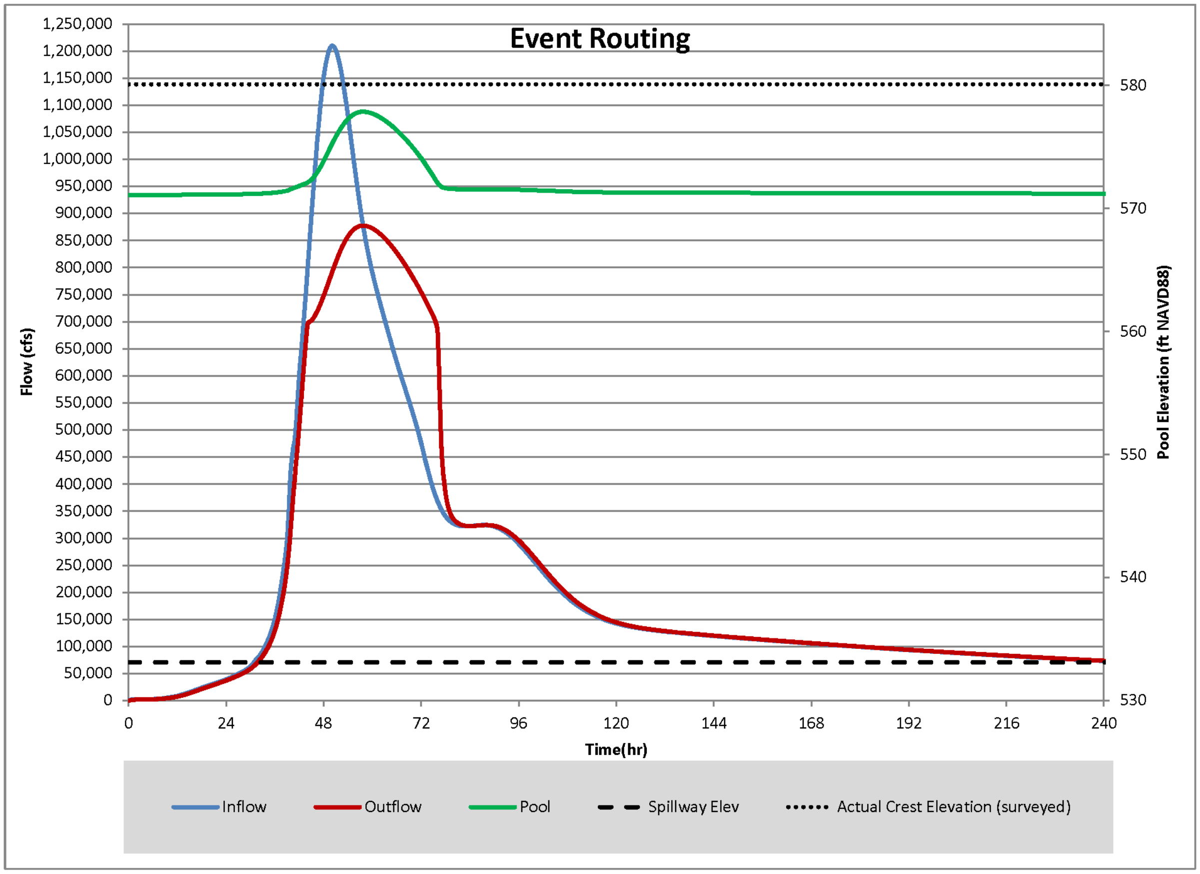

Figure 4-14 Final Recommended Probable Maximum Flood for Whitney Dam from 2015 Periodic Assessment

Minimal inflows were added to the model to incorporate the potential runoff volume being added through tributaries and the local mainstem reach of the Brazos River. Discharges representing tributary low flow were carried forward from the CWMS 1D hydraulic model and varied from 100 cfs to 500 cfs on the Aquilla, San Gabriel, Little, and Navasota rivers and Yegua Creek.

Breach Characteristics

The Whitney Dam breach analysis utilized Froehlich (Froehlich 1995), Von Thun and Gillette, and MacDonald and Langridge-Monopolis regression equations to estimate breach size and development time. Breach parameters are calculated using various factors, including but not limited to, water depth above the bottom of the breach and reservoir volume. For breaches that are assumed to occur with the water level below the top of dam elevation, piping breach through the main embankment was used. For breaches that are assumed to occur with the water level above the top of dam elevation, overtopping breach through the main embankment was used. Table 4-7and Table 4-8 present pertinent data for the breach scenarios.

A sensitivity run was done on the TAS scenario to compare the results from a concrete dam breach versus the earthen embankment breach. The largest estimated monolith breach was used for the concreate failure, 6 failed monoliths at the spillway (50ft width for each monolith, 300 feet in total over an hour breach timing). The concrete breach resulted in a peak flow that was 53 percent of the earth embankment breach. This was tested at the downstream USGS gage AQLT2. The earth embankment failure was used as the final breach mechanism for this analysis.

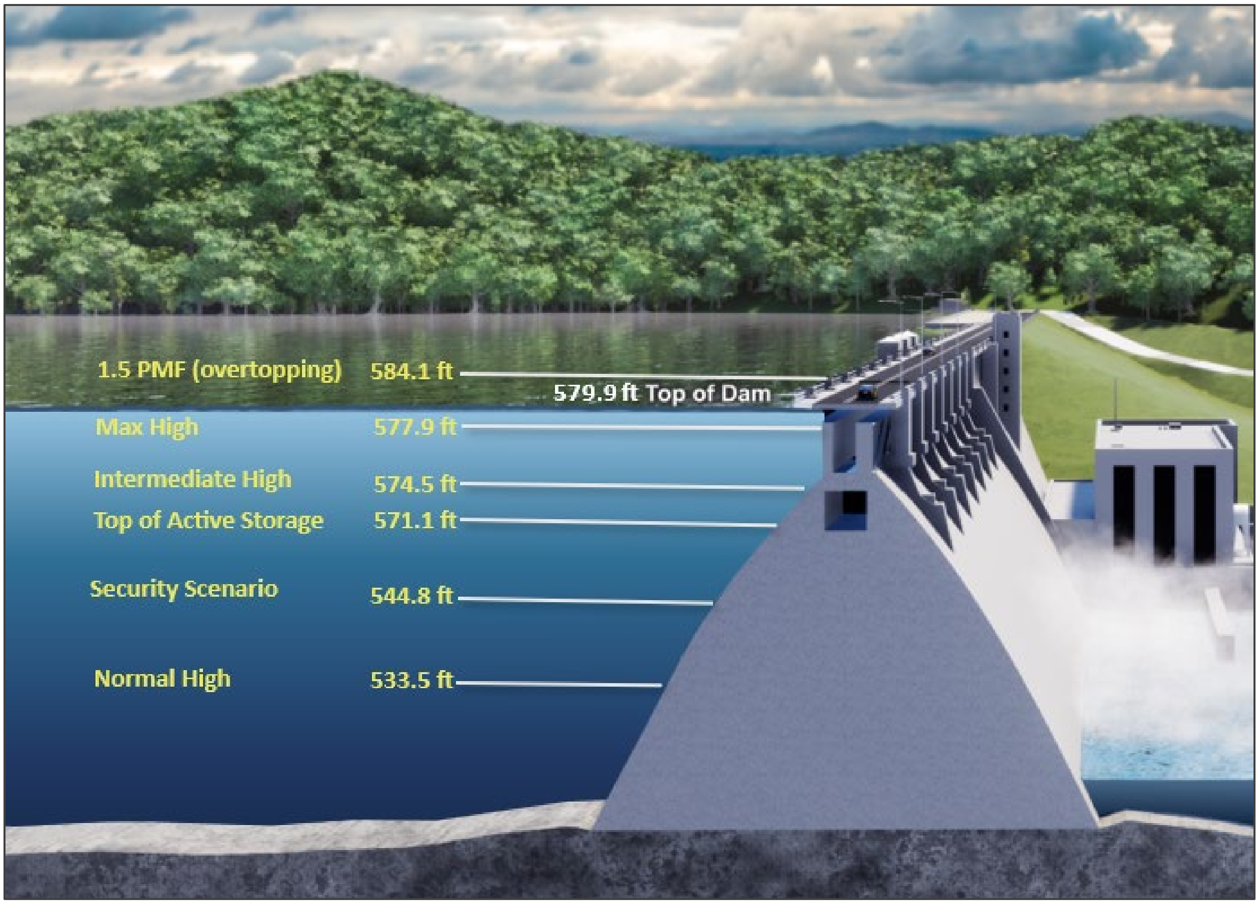

Figure 4-15: Flood Scenarios, Top of Dam represents the low spot on the embankment at station 41+00

Table 4-7 Breach Data (sheet 1 of 2)

Input Data						
Top of Dam (feet)	579.9					
Breach Invert (feet)	425.1					
DS Channel Invert (feet)	425.1					
h_b (feet)	154.5					
US_{ss}	3.27					
DS_{ss}	2.33					
	1.5 PMF	**Maximum High**	**Intermediate High**	**Top of Active Storage**	**Security Scenario**	**Normal High**
Peak Stage (feet)	584.1	577.9	574.50	571.10	544.8	533.5
Volume (acre-feet)	2,643,443	2,220,112	2,097,676	1,926,673	874,244	563,757
h_w (feet)	159.0	152.8	149.4	146.0	119.7	108.4
Failure Mode	Overtopping	Piping	Piping	Piping	Piping	Piping
Results						
MacDonald and Langridge-Monopolis						
W_b (feet)	6221	5262	4929	4532	2098	1369
t_f (hour)	6.5	6.1	6.0	5.8	4.4	3.8
Froehlich (1995)						
K_o	1.4	1.0	1.0	1.0	1.0	1.0
W_b (feet)	1690	1149	1126	1092	817	691
t_f (hour)	8.7	8.0	7.7	7.4	4.9	3.9
Froehlich (2008)						
K_o	1.3	1.0	1.0	1.0	1.0	1.0
W_b (feet)	1333	974	954	926	695	590
t_f (hour)	6.8	6.2	6.0	5.8	3.9	3.1
Von Thun and Gillette						
C_B (feet)	180	180	180	180	180	180
W_b (feet)	500	485	476	468	402	374
t_{f1} (hour)	1.2	1.2	1.2	1.1	1.0	0.9
t_{f2} (hour)	0.7	0.7	0.7	0.7	0.6	0.5

Table 4-8 Breach Data (sheet 2 of 2)

Definitions	
Volume	At peak stage (acre-feet)
h_w	Depth of water above bottom of breach (feet)
h_b	Height from top of dam to bottom of breach (feet)
US_{ss}	Upstream Face Slope(v/h)
DS_{ss}	Downstream Face Slope(v/h)
K_o	Froehlich Constant (1.4 for overtopping breaches, 1.0 for piping—Froehlich 1995) (1.3 for overtopping breaches, 1.0 for piping—Froehlich 2008)
W_b	Bottom width of breach (feet)
t_f	Breach formation time (hours)
t_{f1}	Von Thun and Gillette Breach formation time for an erosion resistant dam (hour)*
t_{f2}	Von Thun and Gillette Breach formation time for an easily erodible dam (hour)*
C_B	Von Thun and Gillette coefficient, which is a function of reservoir size-feet (Table 3-2)

*Note that the Von Thun and Gillette's breach formation times are presented for both erosion resistant and easily erodible. The paper on Von Thun and Gillette states, "It is suggested that these limits be viewed as upper and lower bounds corresponding respectively to well-constructed dams of erosion resistant material and poorly constructed dams of easily eroded materials."

Table 4-9 Von Thun and Gillette Breaches Data C_B Coefficients

Reservoir Size	C_B (feet)
<1,000 acre-feet	20
1,000–5,000 acre-feet	60
5,000–10,000 acre-feet	140
>10,000 acre-feet	180

The breach location was assumed to be near the thalweg of the stream corresponding to the location of the outlet works. The regression equation that produced the most reasonable breach parameters were used for each scenario. The Von Thun regression equation was used to estimate the breach parameters for all scenarios. Table 4-10 displays the breach parameters used for this study. Table 4-11 shows dam breach and non-breach modeling data and results. Table 4-12 and Table 4-13 present model parameters and model timings.

Table 4-10 Dam Breach Parameters

Hydrologic Loading Condition	Peak Reservoir Pool Elevation (feet)	Assumed Breach Mode	Bottom Width of Dam Breach (feet)	Breach Side Slopes (H:1V)	Top Width of Dam Breach (feet)	Final Breach Bottom Elevation (feet)	Full Breach Development Time (hours)
1.5 PMF	584.1	Overtopping	1333	1.0	1643	425.1	6.8
MH Pool	577.9	Piping	974	0.7	1191	425.1	6.2
IH Pool	574.5	Piping	954	0.7	1171	425.1	6.0
TAS Pool	571.1	Piping	926	0.7	1143	425.1	5.8
SS Pool	544.8	Piping	695	0.7	911	425.1	3.9
NH Pool	533.5	Piping	590	0.7	806	425.1	3.1

Table 4-11 Dam Breach and Non-breach Modeling Data and Results

Hydrologic Loading Condition	Starting Reservoir Pool Elevation (feet)	Volume at peak stage (acre-feet)	Inflow Hydrograph (Percent PMF)	Peak Inflow (cfs)	Breach Condition Peak Outflow (cfs)	Non-breach Condition Peak Outflow (cfs)
1.5 PMF	571.1	2,643,443	141	1,790,442	4,972,315	1,134,370
MH Pool	571.1	2,220,112	100	1,330,734	4,096,212	827,630
IH Pool	533.5	1,960870	88	971522	3,844,458	399,478
TAS Pool	533.5	1,872,214	41	475,813	3,468,058	25,000
SS Pool	533.5	874,244	14	163,317	2,023,582	24,721
NH Pool	533.5	563,757	0	Constant 2,200	1,657,582	2,200

Table 4-12 Hydraulic Model Extent

Downstream distance of inundation model (miles)	263
Description of model end location	The inundation was mapped from the outer extents of Whitney Lake to the confluence with the Gulf of Mexico.

Table 4-13 Hydraulic Model Timings

Event	Simulation Start Date		Start of Breach Initiation	
	Date	Time	Date	Time
1.5 PMF	02 FEB 2099	2400	05 FEB 2099	0745
MH Pool	02 FEB 2099	2400	05 FEB 2099	1145
IH Pool	02 FEB 2099	2400	05 FEB 2099	2100
TAS Pool	02 FEB 2099	2400	12 FEB 2099	1330
SS Pool	02 FEB 2099	2400	07 FEB 2099	0035
NH Pool	02 FEB 2099	2400	04 FEB 2099	0005

Hydraulic Results

Table 4-14 and Table 4-15 identify select hydraulic model results at key locations for each modeled scenario. Result locations were chosen at population centers or based on easily identifiable downstream locations such as bridges or significant structures. Model output for breach and non-breach scenarios at selected cross sections are summarized in the tables. Additionally, Table 4-16 and Table 4-17 provides gage elevation comparisons for downstream breach scenarios.

Figure 4-16 through Figure 4-19 illustrate anticipated max high breach inundation from a probable maximum flood event.

Table 4-14: Breach Model Output Summary (sheet 1 of 2)

Community Name or Flooding Location	River/Reach Name	River Station or Cell ID	Distance from Dam (miles)	Scenario	Peak Water Surface Elevation (feet)
Waco, TX	Brazos River at SH 77 in Waco, TX (Balor University)	162016	42.6	1.5 PMF	431.64
				MH	428.19
				IH	425.84
				TAS	422.68
				SS	412.86
				NH	408.25
Bryan, TX	Brazos River at SH 21 near Bryan, TX	373157	113.0	1.5 PMF	259.28
				MH	257.03
				IH	254.89
				TAS	251.31
				SS	245.23
				NH	240.68
Hempstead, TX	Brazos River at SH 290 near Hempstead, TX	140598	152.4	1.5 PMF	175.82
				MH	173.06
				IH	170.38
				TAS	166.30
				SS	160.23
				NH	155.60

Table 4-15: Breach Model Output Summary (sheet 2 of 2)

Community Name or Flooding Location	River/Reach Name	River Station or Cell ID	Distance from Dam (miles)	Scenario	Peak Water Surface Elevation (feet)
Richmond, TX	Brazos River at SH 90 in Richmond, TX	458837	197.4	1.5 PMF	90.43
				MH	88.98
				IH	87.66
				TAS	86.09
				SS	82.16
				NH	77.64
West Columbia, TX	Brazos River at HWY 35 near West Columbia, TX	217767	249.4	1.5 PMF	31.85
				MH	31.28
				IH	30.75
				TAS	30.09
				SS	29.00
				NH	28.39

Table 4-16: Non-breach Model Output Summary (sheet 1 of 2)

Community Name or Flooding Location	River/Reach Name	River Station or Cell ID	Distance from Dam (miles)	Scenario	Peak Water Surface Elevation (feet)
Waco, TX	Brazos River at SH 77 in Waco, TX	162016	42.6	1.5 PMF	416.12
				MH	412.31
				IH	406.85
				TAS	392.88
				SS	388.01
				NH	380.39
Bryan, TX	Brazos River at SH 21 near Bryan, TX	133925	113.0	1.5 PMF	253.10
				MH	250.89
				IH	247.82
				TAS	231.94
				SS	221.24
				NH	208.16

Table 4-17: Non-Breach Model Output Summary (sheet 2 of 2)

Community Name or Flooding Location	River/Reach Name	River Station or Cell ID	Distance from Dam (miles)	Scenario	Peak Water Surface Elevation (feet)
Hempstead, TX	Brazos River at SH 290 near Hempstead, TX	140599	152.4	1.5 PMF	171.27
				MH	167.79
				IH	164.39
				TAS	153.32
				SS	143.12
				NH	129.07
Richmond, TX	Brazos River at SH 90 in Richmond, TX	155092	197.4	1.5 PMF	88.71
				MH	86.99
				IH	85.56
				TAS	78.18
				SS	71.42
				NH	60.26
West Columbia, TX	Brazos River at HWY 35 near West Columbia, TX	217767	249.4	1.5 PMF	31.27
				MH	30.47
				IH	29.92
				TAS	28.75
				SS	26.39
				NH	13.82

Table 4-18: Comparison of Breach Model Results with Downstream Gage Flood Categories (sheet 1 of 2)

Gage Description/ Gage Code	HEC-RAS Model Cross Section or Cell	Gage Datum Elevation (feet)	NOAA AHPS Action Stage (feet)	NOAA AHPS Major Flood Stage (feet)	Scenario	HEC-RAS Maximum Water Surface Elevation (feet)
Brazos River at Waco (WBAT2)	211957	349.39	366.39	386.39	1.5 PMF	413.96
					MH	410.68
					IH	407.62
					TAS	396.47
					SS	395.73
					NH	392.27
Brazos River near Bryan (BBZT2)	373157	188.78	225.78	254.78	1.5 PMF	259.28
					MH	257.03
					IH	254.11
					TAS	246.15
					SS	245.23
					NH	240.68
Brazos River near Hempstead (HPDT2)	140598	107.92	132.9	162.9	1.5 PMF	175.82
					MH	173.06
					IH	169.51
					TAS	161.39
					SS	160.23
					NH	155.60

Table 4-19: Comparison of Breach Model Results with Downstream Gage Flood Categories (sheet 2 of 2)

Gage Description/ Gage Code	HEC-RAS Model Cross Section or Cell	Gage Datum Elevation (feet)	NOAA AHPS Action Stage (feet)	NOAA AHPS Major Flood Stage (feet)	Scenario	HEC-RAS Maximum Water Surface Elevation (feet)
Brazos River at Richmond (RMOT2)	458837	27.92	47.9	77.9	1.5 PMF	90.43
					MH	88.98
					IH	87.46
					TAS	83.71
					SS	82.16
					NH	77.64
Brazos River near West Columbia (WCBT2)	217767	0	20.0	27.0	1.5 PMF	31.85
					MH	31.28
					IH	30.63
					TAS	29.24
					SS	29.00
					NH	28.39

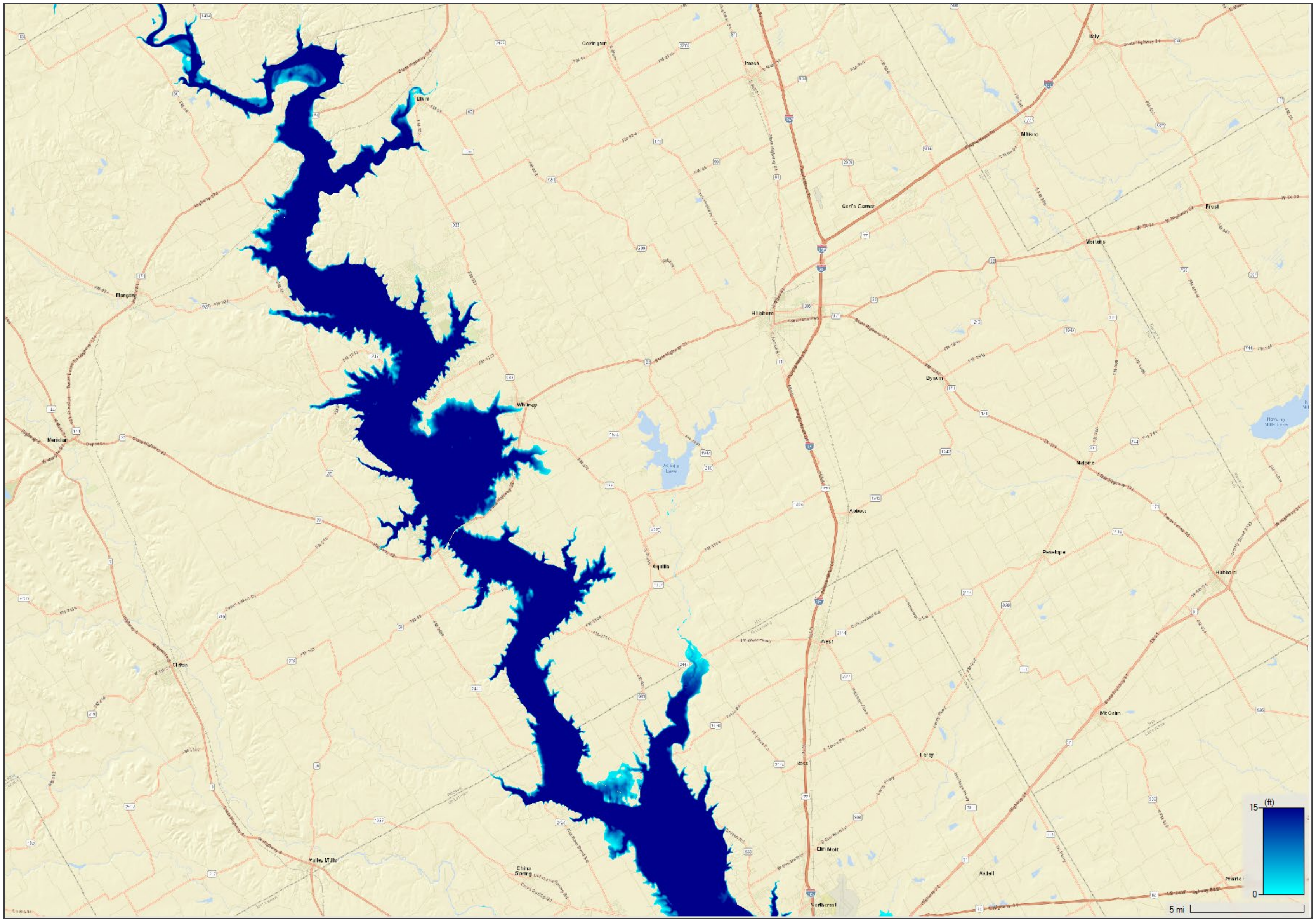

Figure 4-16: Inundation and Depth Map at Whitney Lake and Downstream

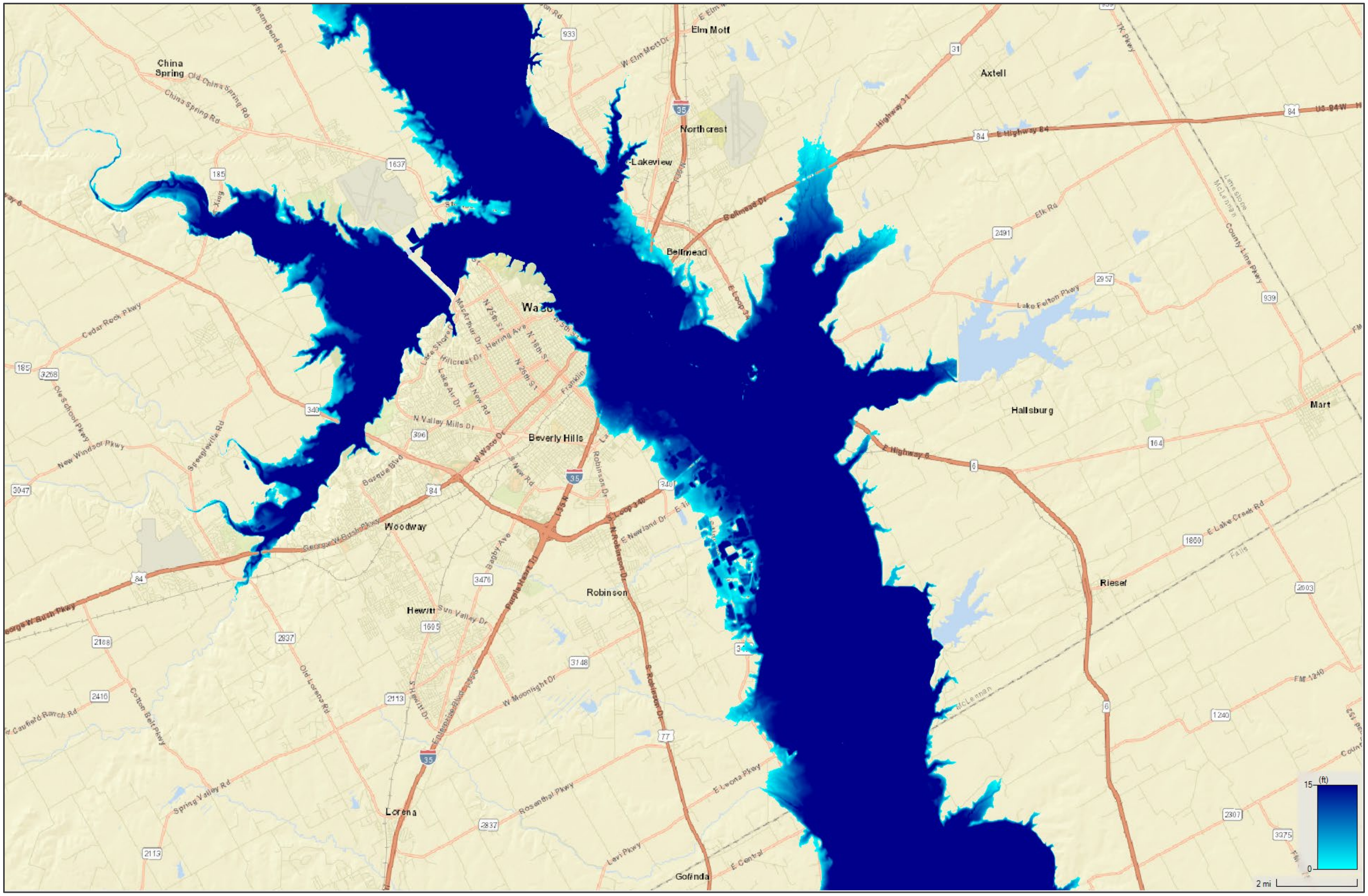

Figure 4-17: Inundation and Depth Map at the City of Waco

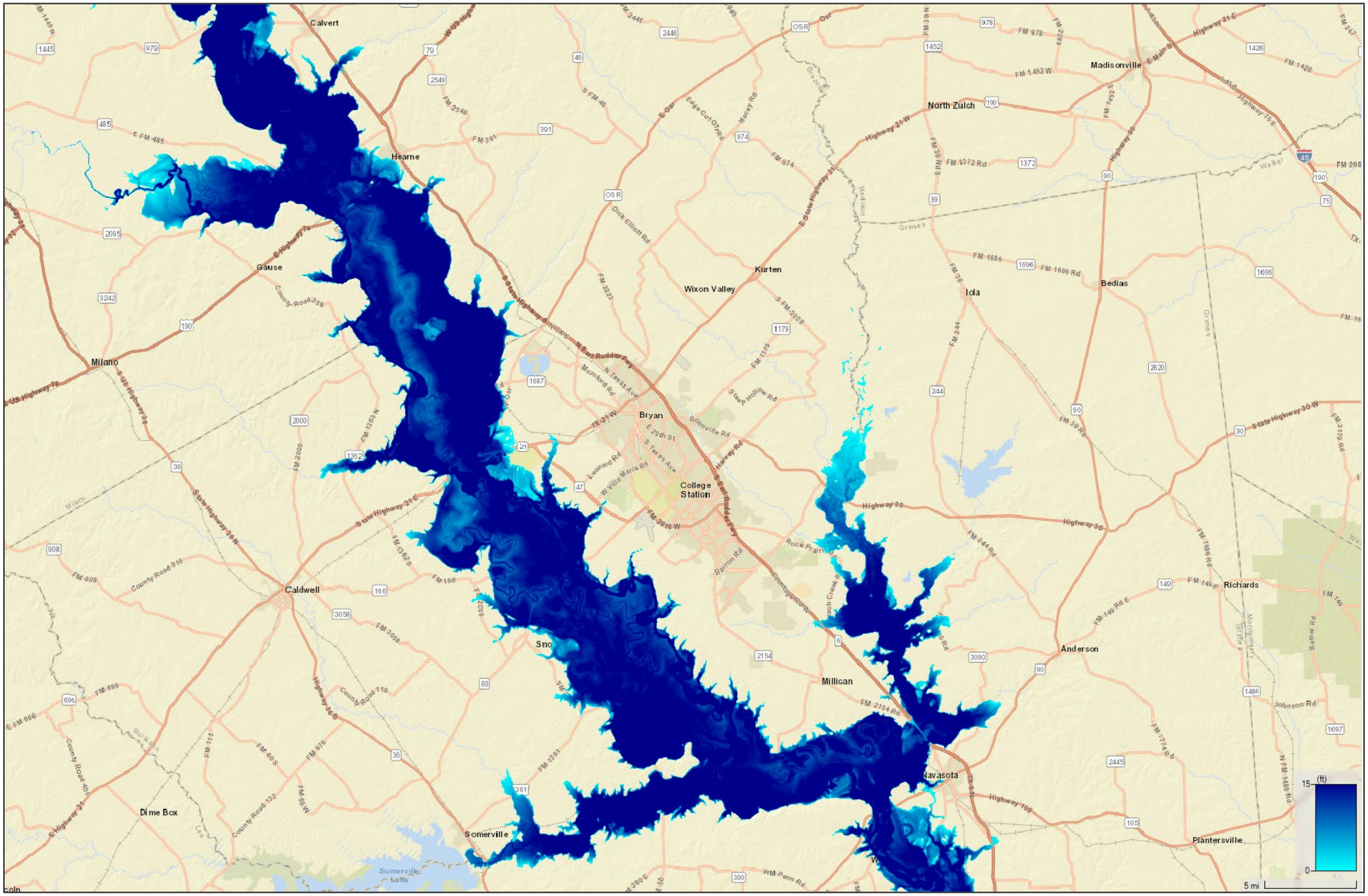

Figure 4-18: Inundation and Depth Map at Bryan TX and Collage Station

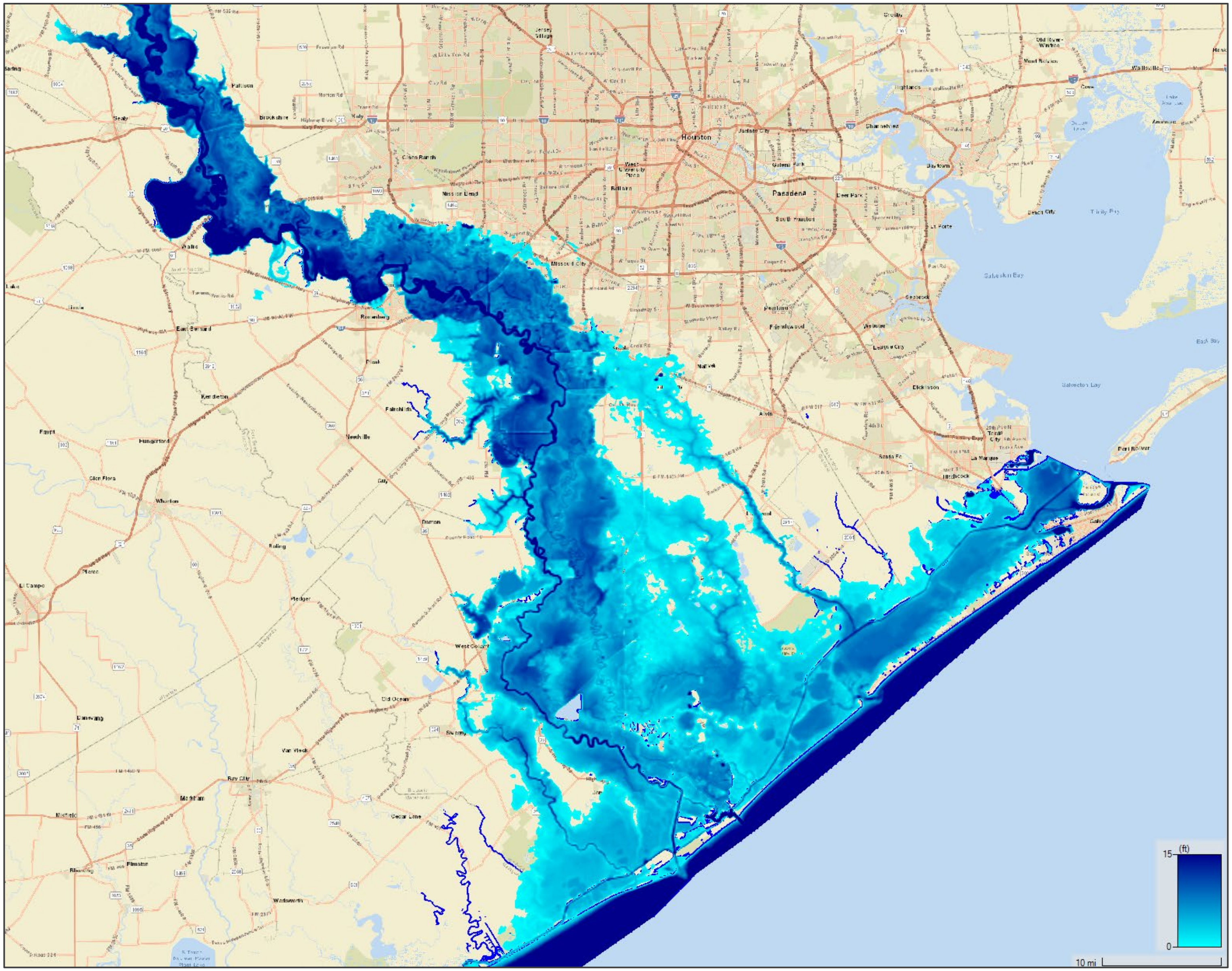

Figure 4-19: Inundation and Depth Map near the City of Houston and the Coast

5. ASSUMPTIONS & LIMITATIONS

Data

RiverWare data makes the assumption operation changes will be made once every 24 hours for 7 days a week. In reality operation changes can be made more frequent than once every 24 hours and will likely be in the normal working business hours between 8am to 5pm. For large storm events and flooding conditions, operational changes will occur any time during the 24-hour period. RiverWare also uses a basin ratio method of calculating inflows into the reservoir. Depending on where the storm fell in the basin, values may be slightly over or underestimated.

Modeling

A 2D hydraulic model was used in the is analysis HEC-RAS. This modeling software uses an implicit finite difference scheme to solve the unsteady flow equations. This approach is computationally efficient but approximate. Exact solutions are not feasible for complex river systems, so HEC-RAS uses a numerical method that can introduce errors.

For the Hydrologic model, HEC-HMS, it is assumed characteristics in the delineated subbasins have uniform properties. This does not fully capture the complexity within each subasins. HEC-HMS also employs a quasi-coupled approach to modeling land surface processes like evapotranspiration and infiltration. This means that the calculations are done sequentially rather than simultaneously, which might not fully capture the interactions between these processes in the real world.

Analysis

1. Frequency Analysis

The frequency analysis used a single storm event from 2015 to represent the family of frequency volumes. System responses are dependent on storm location and intensity. A shorter more intense flooding event that fell near the reservoir will have a different response with the potential of larger releases from the reservoir. This is due to the tainter gate operations that are influenced by lake elevation and inflows. It is possible the 2015 storm event was not the most conservative in the period of record. A sensitivity of other storm patterns might produce a larger consequence from a higher peak of inflow.

2. Historic Storm Event, 1957

This analysis used the period of record storm event that caused surcharge releases in 1957. This was a series of storm events that happened simultaneously involving a 1% ACE, a 10% ACE, followed by a 3.6% ACE event causing releases above the channel capacity for downstream extents. This study is sensitive to the sequence of events. If it was reversed, the reservoir would have had much larger releases from the 1% ACE event (peak inflow occurring after the pool exceeded the top of the flood pool). For this reason, one event would be unique to another. The results from this analysis still provided good information for these types of events but is unlikely to occur the same way in the future with the new alternatives operations.

3. Stage Frequency Curve

This analysis assumes that all rainfall events occur independently. In reality, storm events (especially in the spring) occur in a series causing compounding flooding effects that is not captured in the frequency curve analysis. These events typically fall on the high end of the frequency cure on or beyond the 90 percent confidence line. The current standard for frequency curves assumes a single event system response. To analyze multiple events would take a very complex modeling approach that is not feasible for this level of analysis.

4. Elevation Duration Analysis

This analysis is dependent on the dataset developed by RiverWare and has limitations stated above. It is also limited by the period analyzed, 1938 through 2022. Having additional data would provide more information on the higher and lower end of the frequency curve.

5. Monthly Average Pool Elevations

This analysis is dependent on the dataset developed by RiverWare and has limitations stated above

6. Breach Analysis

The modeling effort is not meant to produce detailed hydraulic models usable for all types of studies. The purpose of this effort is to create a model able to obtain a reasonable estimate of consequences associated with dam breach and non-breach conditions over a full range of hydrologic (reservoir pool) loading conditions and project performance. For this level of risk assessment, modeling procedures have been appropriately simplified. In some cases, modeling challenges led to simplified approximation of model geometry or dam operation. Following is a discussion of model features that have been simplified or excluded, modeling assumptions and known or expected modeling inaccuracies. These items may be addressed in future, more detailed, modeling efforts.

- Coincidental flooding on the tributaries was not modeled and may be considered for future analyses. Minimal low flows were added at several tributaries consistent with the 1D CWMS model; however, this likely does not constitute a true coincident flood analysis.
- Operations assume all 17 Tainter gates can open simultaneously.
- Whitney Dam is part of a system of 12 dams controlling the Middle and Lower Brazos River Basins. Upstream dam breaches were not considered.
- The downstream boundary condition was set to mean sea level.
- A breach of Brazos Lake Dam, which is located downstream of Whitney Dam (volume 5,600 acre-feet), was not considered.
- Private levee system elevation data could be incorporated into the 2D model as storage area/2D connections in the future to accurately model top of levee elevations.
- The sensitivity run using full momentum equations produced higher water surface elevations at a downstream index point as noted in Section 2.2. Future modeling should consider using more conservative results.
- Uncertainty in the levee systems modeled in the Sugar land area is moderate. Not all of the levees in this region had information in the National Levee database. The model was visually aligned to the tops of the levees but might not fully represent the true height. Some of the inundation in this aera might be misrepresented for this reason.

6. CONCLUSION

From the five-objective analysis (Frequency, Historic 1957 Event, Elevation Duration, Monthly average pool elevations), it was found that Alternatives 2, 2a, 2c, 4, and 6 were relatively the same with a slight variation due to operations. The changes between these alternatives were found to be nominal and did not have much variation from the original conditions, Alt 1, when it came to risk due to hydrologic and hydraulic conditions (probability of occurrence, downstream consequences, duration of pool exceedance, monthly pool averages, and capacity of the flood storage within the pool of Whitney Lake), they were all very similar.

For the two alternatives that had a change in the top of the conservation pool elevation (Alt 3 and 5), there was a significant difference in the lower pool elevation that relate to inundation on the gates, boat ramps, and structures between elevation 534.6 and 535.6 (picnic areas and campgrounds). This range was below the surcharge pool starting at elevation 571.1 ft NAVD88 all the way down to the conservation pool. Once the elevation reached the surcharge pool where the releases change from controlled flows based on downstream channel capacity to uncontrolled releases relating to inflows into the reservoir, the differences between the two converged and no longer had a significant change for the six hydrologic and hydraulic analysis performed for this study.

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