Mississippi River Valley Division
Regional Planning and Environmental Division South

Appendix G-1: Hydrologic and Hydraulic Models

Amite River and Tributaries Study East of the Mississippi River, Louisiana Feasibility Study with Integrated Environmental Impact Statement

November 2019

CONTENTS

1.0 GENERAL DESCRIPTION OF WORK

The US Army Corps of Engineers (USACE), New Orleans District (MVN), Hydraulics, Hydrology, and Coastal Engineering Branch (HH&C) performed hydrologic and hydraulic modeling for the Amite River and Tributaries (AR&T) Flood Risk Management (FRM) project. The purpose of this hydrologic and hydraulic modeling effort is to evaluate various design alternatives for FRM in the Amite River Basin. Hydrologic and Hydraulic models of the Amite River Basin were provided by the Louisiana Department of Transportation (LaDOT), and modified by the HH&C for use in this modeling effort. Modeling was performed for the 0.5, 0.2, 0.1, 0.04, 0.02, 0.01, 0.005, and 0.002 Annual Exceedance Probability (AEP) rainfall events for existing conditions (year 2026), three design alternatives (year 2026), and Future without Project (FWOP, year 2076). Maximum water surface elevation results were extracted for each model run, and provided to the Project Delivery Team (PDT) for use in economic, environmental, and engineering analyses.

2.0 SOFTWARE

2.1 HEC-HMS 4.3

The latest version of the USACE Hydraulic Engineering Center's (HEC) Hydrologic Modeling System (HMS) that was available at the time of model development was used for the hydrologic modeling.

2.2 HEC-RAS 5.0.6

The latest version of the HEC's River Analysis System (RAS) that was available at the time of model development was used for the hydraulic modeling.

3.0 MODEL DEVELOPMENT

The hydrologic and hydraulic models of the Amite River Basin were provided to the MVN HH&C Branch by the (LaDOT). Development, calibration, and validation of the models are discussed in the LaDOT's Amite River Basin Numerical Model Project Report, however some discussion is provided in this appendix. The LaDOT report is included in this document as Appendix G-2.

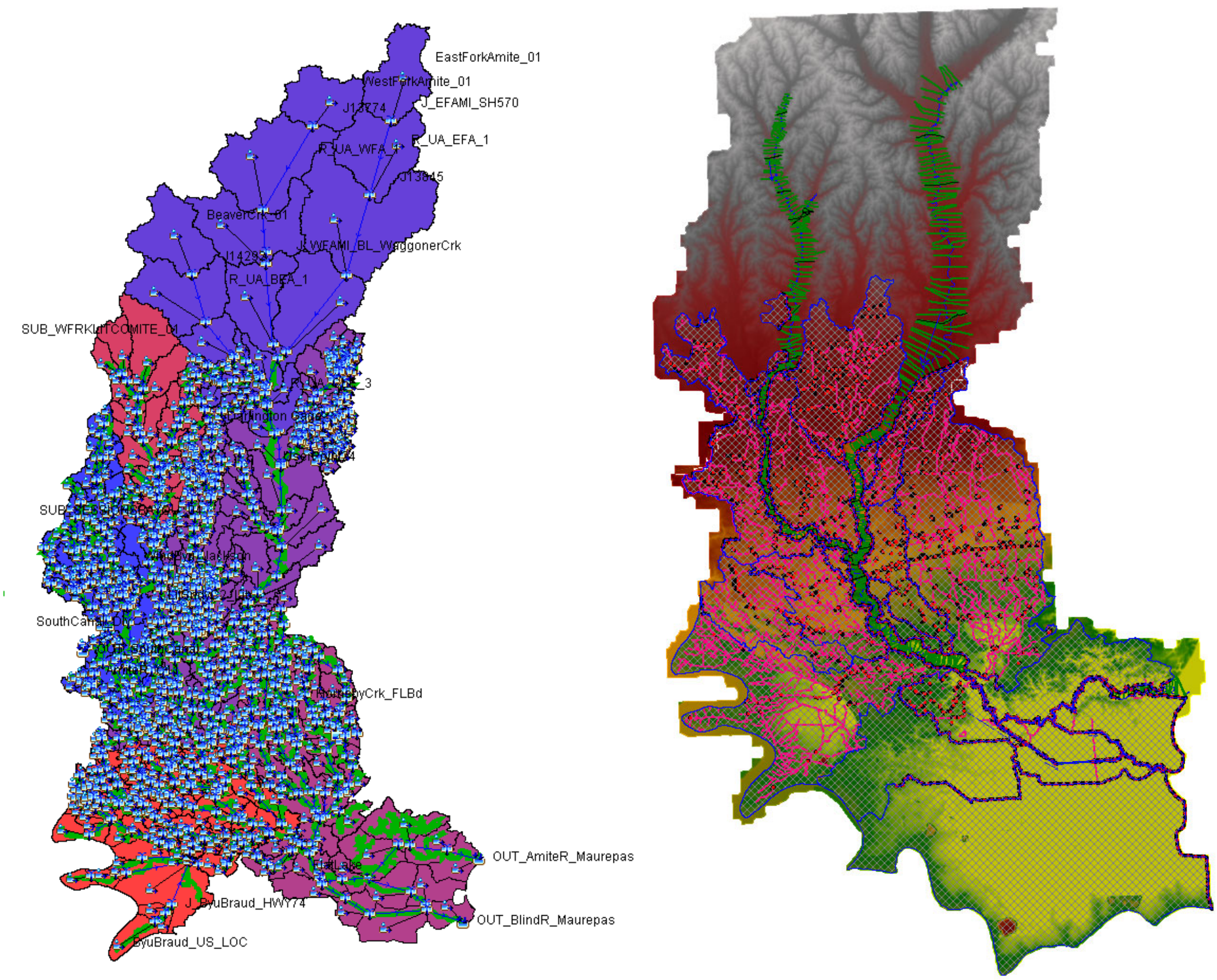


Figure G-1 shows the geometry for the HMS and RAS models.

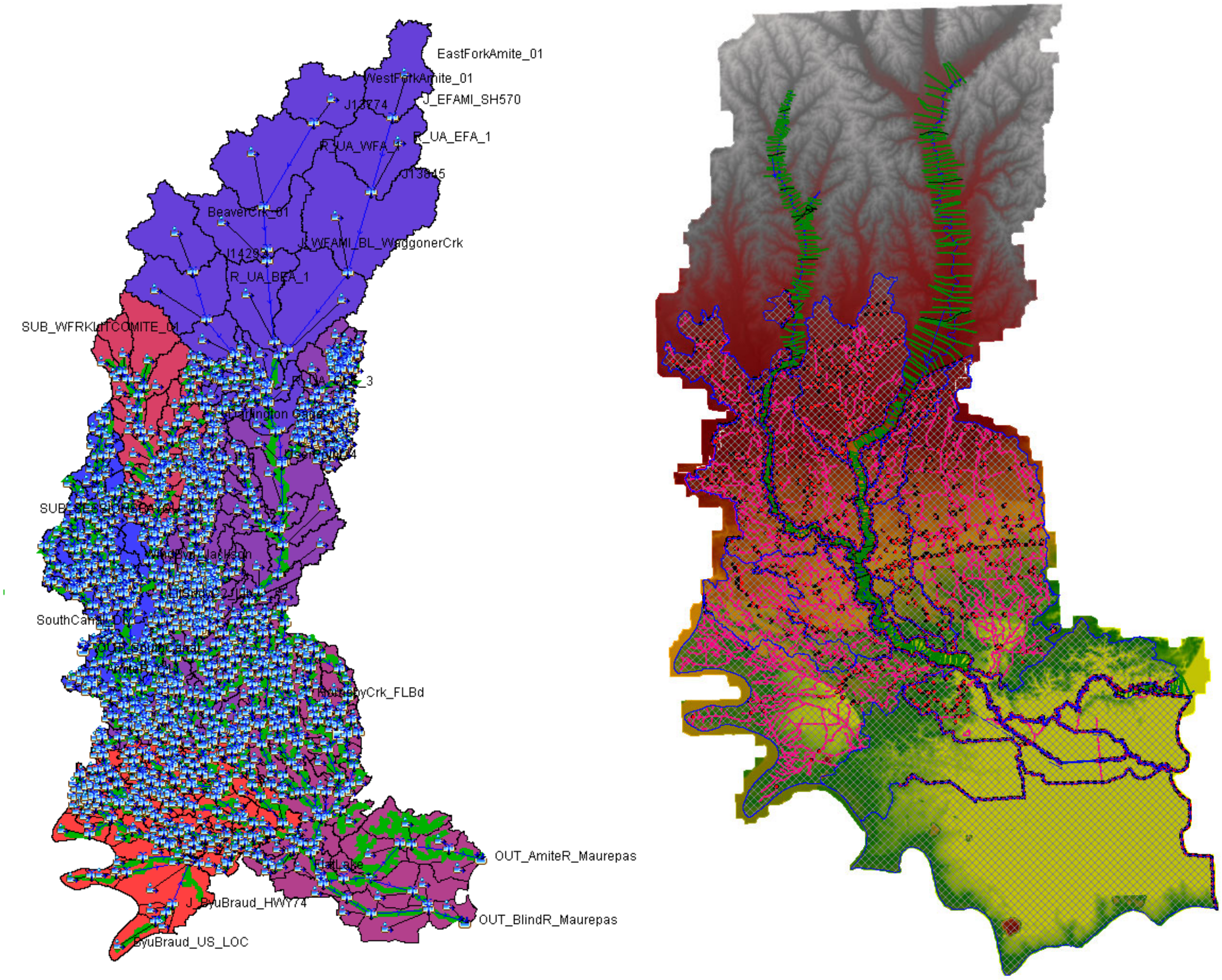


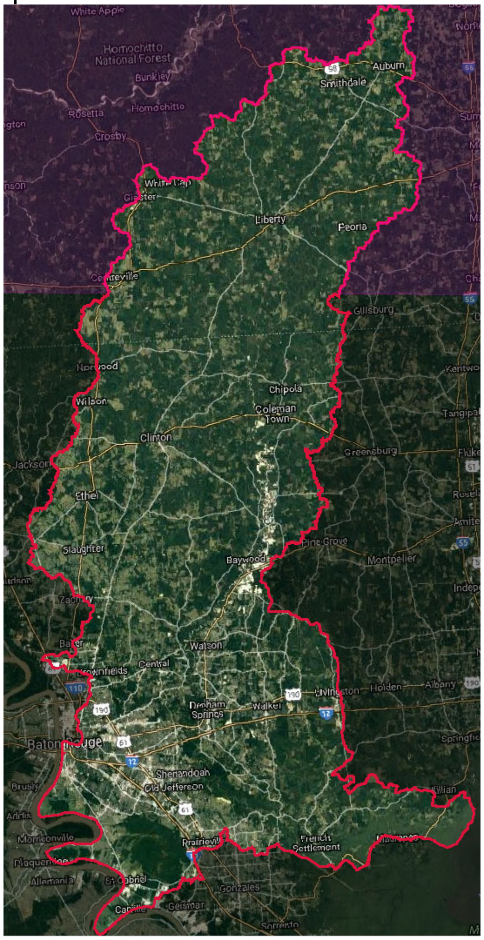
Figure G-1 – HEC-HMS Model Geometry (left) and HEC-RAS Model Geometry (right)

4.0 HYDROLOGY, CLIMATE CHANGE, AND STORM SURGE

4.1 Basin Hydrology

The Amite River Basin covers approximately 2,200 square miles in Mississippi and Louisiana. The Amite River runs for approximately 117 miles in a mostly southerly direction through Mississippi and Louisiana.

The Amite River begins with an East Fork and a West Fork in southwest Mississippi, both starting at elevations of over 450 feet. These forks are the steepest portions of the Amite River, with elevations dropping to approximately 200 feet and lengths of approximately 49 miles. The forks merge just south of Mississippi's border with Louisiana. The middle portion of the Amite River runs for approximately 61 miles and drops approximately 180 feet between the confluence of the upper forks and the confluence with the Comite River. The Comite River, a right bank tributary that meets the Amite River near Denham Springs, is the Amite's largest tributary. The lower portion of the Amite River runs for approximately 54 miles and discharges into Lake Maurepas. This is the flattest portion of the Amite River, dropping from approximately 20 feet to nearly sea level. Near French Settlement, Downstream of Port Vincent, the Amite River Diversion Canal splits off from the Amite River, sending a portion of the river's water southwest



to the Blind River, which also flows into Lake Maurepas. Figure G-2shows the boundary of the Amite River Basin.

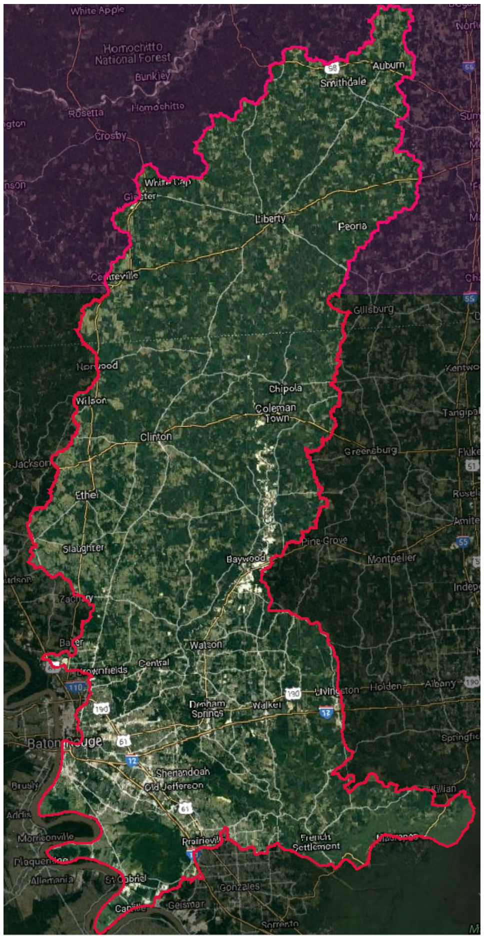


Figure G-2 – Amite River Basin in Louisiana and Mississippi

4.2 Precipitation and Runoff

Eight precipitation events were evaluated: the 2-year, 5-year, 10-year, 25-year, 50-year, 100-year, 200-year, and 500-year average recurrence interval, 24-hour duration events. Precipitation hyetographs were developed for each of those events based on rainfall intensities from the National Oceanic and Atmospheric Administration's (NOAA) Atlas 14 Point Precipitation Frequency Estimates. Figure G-3 shows frequency estimates of precipitation intensity for the Amite River Basin from NOAA Atlas 14.

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]										
Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.534 (0.430-0.662)	0.607 (0.488-0.754)	0.730 (0.585-0.907)	0.834 (0.664-1.04)	0.979 (0.756-1.25)	1.09 (0.825-1.41)	1.21 (0.882-1.59)	1.33 (0.931-1.79)	1.49 (1.00-2.05)	1.62 (1.06-2.25)
10-min	0.781 (0.629-0.969)	0.889 (0.715-1.10)	1.07 (0.857-1.33)	1.22 (0.973-1.52)	1.43 (1.11-1.83)	1.60 (1.21-2.07)	1.77 (1.29-2.33)	1.95 (1.36-2.62)	2.19 (1.47-3.00)	2.37 (1.55-3.29)
15-min	0.953 (0.767-1.18)	1.08 (0.872-1.35)	1.30 (1.05-1.62)	1.49 (1.19-1.86)	1.75 (1.35-2.24)	1.95 (1.47-2.52)	2.16 (1.58-2.85)	2.38 (1.66-3.20)	2.67 (1.80-3.66)	2.89 (1.90-4.02)
30-min	1.43 (1.15-1.78)	1.63 (1.31-2.03)	1.97 (1.58-2.44)	2.25 (1.79-2.80)	2.64 (2.04-3.38)	2.95 (2.23-3.82)	3.27 (2.38-4.31)	3.60 (2.52-4.84)	4.04 (2.72-5.55)	4.38 (2.87-6.09)
60-min	1.93 (1.55-2.39)	2.20 (1.77-2.73)	2.64 (2.12-3.29)	3.02 (2.41-3.77)	3.54 (2.73-4.53)	3.95 (2.98-5.11)	4.37 (3.19-5.75)	4.80 (3.36-6.45)	5.38 (3.62-7.38)	5.82 (3.82-8.09)
2-hr	2.43 (1.97-2.99)	2.76 (2.24-3.40)	3.32 (2.68-4.10)	3.79 (3.04-4.69)	4.44 (3.45-5.63)	4.95 (3.76-6.35)	5.47 (4.02-7.14)	6.00 (4.23-8.00)	6.71 (4.55-9.15)	7.25 (4.79-10.0)
3-hr	2.73 (2.23-3.35)	3.12 (2.54-3.82)	3.76 (3.04-4.61)	4.29 (3.46-5.29)	5.04 (3.93-6.37)	5.63 (4.29-7.18)	6.22 (4.59-8.09)	6.83 (4.84-9.08)	7.65 (5.21-10.4)	8.28 (5.50-11.4)
6-hr	3.26 (2.67-3.95)	3.75 (3.07-4.55)	4.57 (3.73-5.56)	5.27 (4.28-6.44)	6.26 (4.93-7.87)	7.05 (5.42-8.95)	7.86 (5.84-10.2)	8.70 (6.21-11.5)	9.84 (6.76-13.3)	10.7 (7.18-14.6)
12-hr	3.77 (3.12-4.55)	4.40 (3.63-5.31)	5.48 (4.51-6.62)	6.42 (5.25-7.78)	7.77 (6.18-9.73)	8.87 (6.88-11.2)	10.0 (7.51-12.9)	11.2 (8.07-14.7)	12.9 (8.92-17.3)	14.2 (9.56-19.2)
24-hr	4.33 (3.60-5.18)	5.10 (4.24-6.10)	6.44 (5.33-7.72)	7.62 (6.28-9.16)	9.36 (7.51-11.7)	10.8 (8.44-13.5)	12.3 (9.30-15.7)	13.9 (10.1-18.2)	16.1 (11.3-21.5)	17.9 (12.2-24.1)
2-day	4.97 (4.17-5.89)	5.85 (4.90-6.94)	7.39 (6.17-8.79)	8.77 (7.28-10.5)	10.8 (8.74-13.4)	12.5 (9.85-15.6)	14.3 (10.9-18.1)	16.2 (11.8-21.0)	18.8 (13.3-25.0)	21.0 (14.3-28.0)
3-day	5.42 (4.56-6.39)	6.35 (5.34-7.50)	7.98 (6.69-9.44)	9.43 (7.86-11.2)	11.6 (9.41-14.3)	13.3 (10.6-16.6)	15.2 (11.7-19.3)	17.2 (12.7-22.3)	20.0 (14.2-26.5)	22.3 (15.3-29.6)
4-day	5.82 (4.91-6.84)	6.78 (5.72-7.97)	8.45 (7.11-9.96)	9.94 (8.31-11.8)	12.1 (9.89-14.9)	13.9 (11.1-17.2)	15.9 (12.2-20.0)	17.9 (13.2-23.0)	20.8 (14.7-27.3)	23.0 (15.9-30.5)
7-day	6.90 (5.87-8.06)	7.90 (6.71-9.24)	9.63 (8.15-11.3)	11.2 (9.40-13.1)	13.4 (11.0-16.3)	15.2 (12.2-18.7)	17.2 (13.3-21.5)	19.2 (14.3-24.6)	22.1 (15.8-28.9)	24.4 (16.9-32.1)
10-day	7.84 (6.69-9.12)	8.88 (7.57-10.3)	10.7 (9.06-12.4)	12.2 (10.3-14.3)	14.5 (11.9-17.5)	16.4 (13.1-20.0)	18.3 (14.2-22.8)	20.4 (15.2-25.9)	23.2 (16.7-30.2)	25.5 (17.8-33.5)
20-day	10.4 (8.97-12.0)	11.6 (10.00-13.4)	13.7 (11.7-15.8)	15.4 (13.1-17.9)	17.9 (14.8-21.4)	19.9 (16.1-24.0)	22.0 (17.2-27.1)	24.1 (18.1-30.4)	27.1 (19.5-34.9)	29.4 (20.6-38.2)
30-day	12.7 (11.0-14.6)	14.1 (12.2-16.2)	16.5 (14.2-19.0)	18.4 (15.8-21.3)	21.2 (17.6-25.1)	23.3 (18.9-27.9)	25.5 (20.0-31.1)	27.7 (20.8-34.6)	30.7 (22.2-39.2)	32.9 (23.2-42.7)
45-day	15.7 (13.6-17.9)	17.5 (15.2-20.0)	20.3 (17.6-23.3)	22.7 (19.5-26.0)	25.8 (21.4-30.2)	28.1 (22.8-33.3)	30.3 (23.8-36.8)	32.6 (24.6-40.4)	35.5 (25.8-45.0)	37.6 (26.7-48.5)
60-day	18.4 (16.0-20.9)	20.5 (17.9-23.4)	23.9 (20.7-27.3)	26.5 (22.9-30.4)	30.0 (24.9-34.9)	32.4 (26.4-38.3)	34.8 (27.4-41.9)	37.0 (28.0-45.6)	39.7 (28.9-50.1)	41.6 (29.6-53.6)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

Figure G-3 – Point Precipitation Frequency Estimates from NOAA Atlas 14 for the Amite River Basin

Infiltration and initial abstraction hydrologic losses were calculated by the hydrologic model based on land use and imperviousness. Discussion of those parameters can be found in the Amite River Basin Numerical Model Project Report. That report is included as Appendix G-2. Forecasts of the Amite River Basin over the project life show an expected increase in urban development. Urban development correlates with an increase in impervious area, which leads to increases in runoff. A forecast of urban growth provided by the project delivery team showed an expected 35% increase over the project life. HH&C utilized this forecast to increase the impervious area by 35% for future conditions in the hydrological calculations.

4.3 Hydrology Non-Stationarity

In order to evaluate potential impacts to project performance in the future due to climate-based changes in hydrology, the USACE Non-Stationarity Detection Tool was used. According to the Trend Analysis for the Amite River at Port Vincent between 1985 and 2015 (Figure G-4), there has been a statistically significant downward trend in annual peak streamflow. Additionally, according to the Projected Annual Maximum Monthly Streamflow from the Climate Hydrology Assessment Tool (Figure G-5), there is an expected downward trend in annual maximum monthly streamflow. Because of this expected decrease in peak flow rates in the Amite River due to climate change, project performance is not expected to be adversely affected by climate change-induced hydrologic non-stationarity.

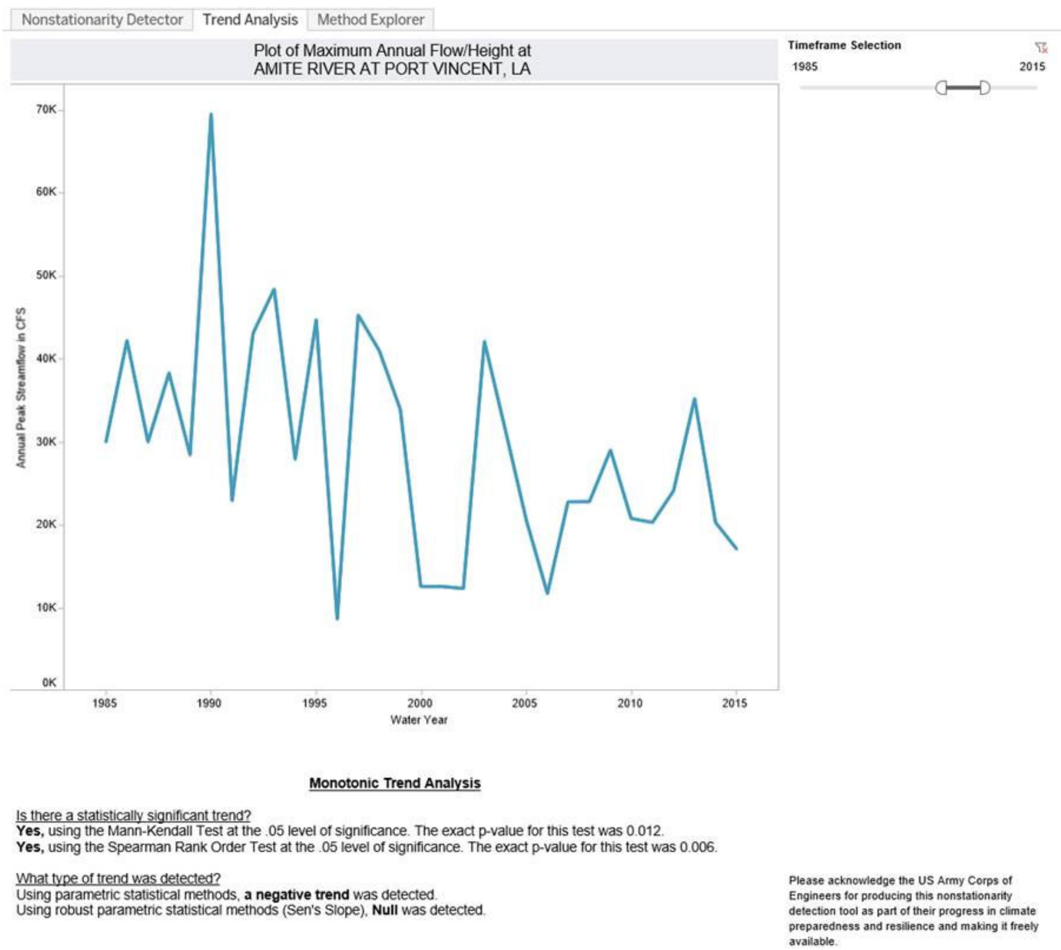


Figure G-4 – Peak Streamflow Trend Analysis for the Amite River at Port Vincent

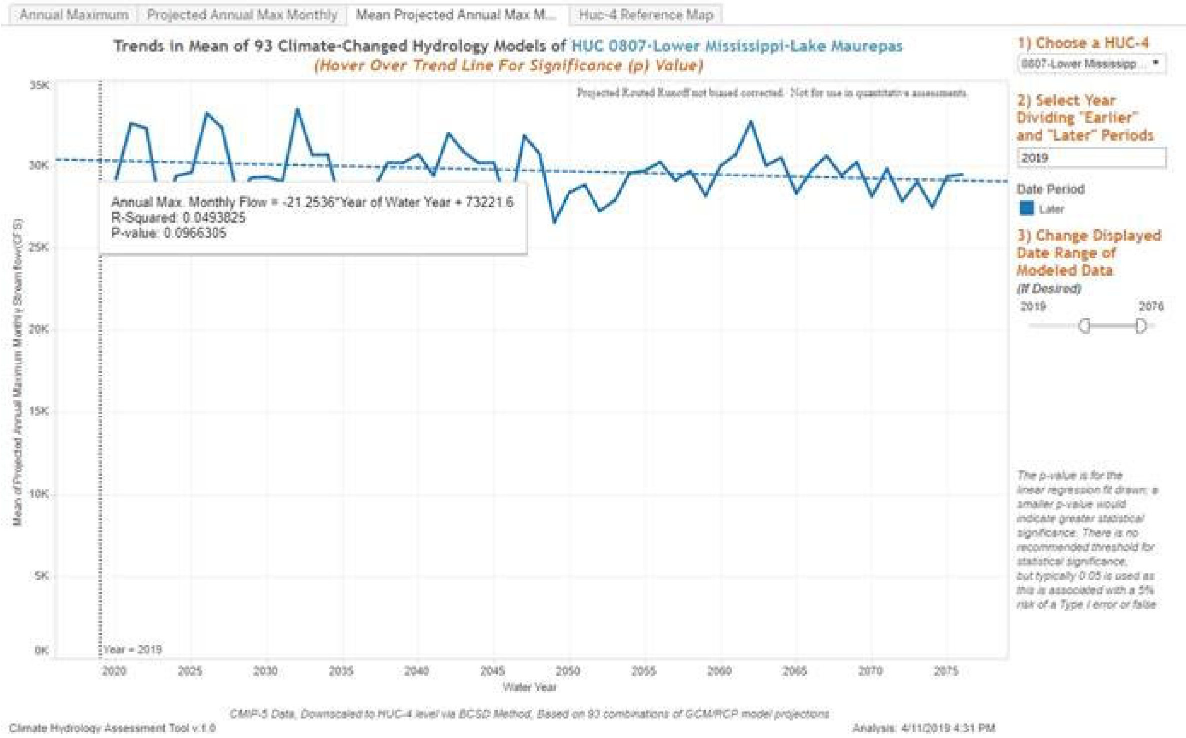


Figure G-5 – Mean Projected Annual Maximum Monthly Streamflow for the Amite River at Port Vincent

While climate-based changes are not expected to adversely affect project performance, population growth and urban development is expected to affect the Amite River Basin. An analysis of future growth by the economics team forecast approximately 35% growth in the Amite River Basin. HH&C translated that projected growth to projected increases in runoff by increasing the amount of impervious area in the hydrology model. Future conditions model runs have increased flow rates at all flow boundaries. Thus, projected increases in runoff flow rates have been considered in evaluation of project performance.

4.4 Storm Surge

The lower portion of the Amite River Basin experiences impacts from storm surge, which propagates through Lake Maurepas. Recent ADCIRC storm surge modeling was performed using a refined grid in the Lake Pontchartrain and Lake Maurepas region. Results from that modeling were obtained and reviewed by HH&C for potential impacts to this project.

At this time, results from storm surge modeling have not been incorporated into the hydraulic analysis for this project. For future milestones of this project, results from storm surge modeling will be coupled with results from the hydraulic modeling for each AEP event. This will be done during post processing, by layering maximum storm surge modeling results with maximum hydraulic modeling results, and taking the larger water surface elevation of the two results grids. HH&C compared the maximum water surface elevation grids for storm surge modeling and hydraulic modeling, and determined that only in the region within near Lake Maurepas would the storm surge results have a higher maximum water surface elevation. In that region, there are very few structures, and thus impacts to project performance and TSP selection are not expected to be significant. Figure G-6 shows the 100 year maximum water surface elevations from storm surge modeling.

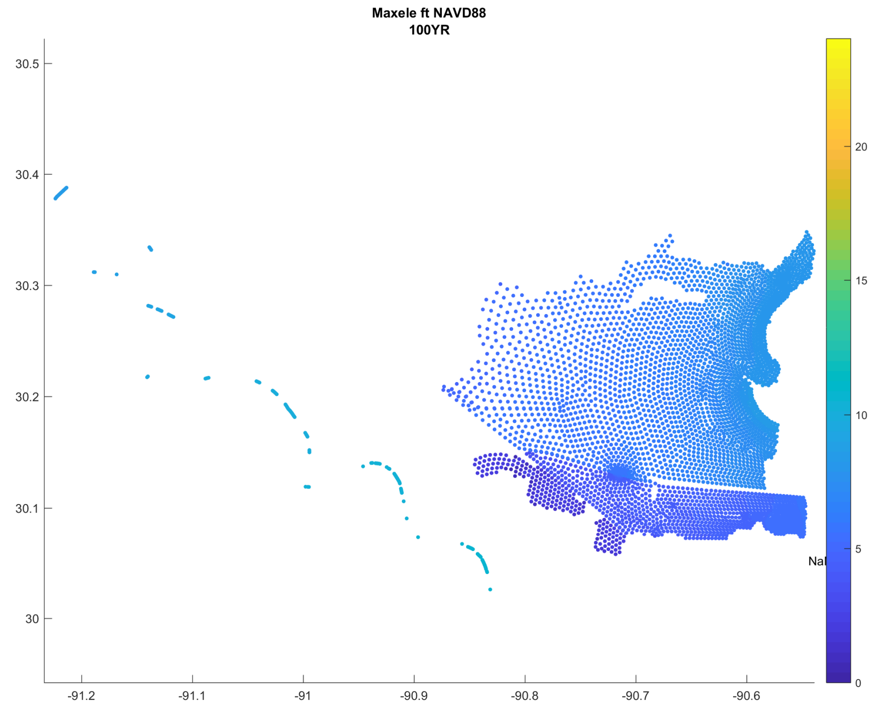


Figure G-6 – 100-year maximum water surface elevations from storm surge modeling

4.5 Sea Level Rise

In order to evaluate potential future changes in project performance due to sea level rise, the USACE Sea-Level Calculator was used. The Lake Pontchartrain gage at Frenier is the closest gage to the AR&T study area, and thus was selected for this analysis. The Sea-Level Calculator provides three rates of sea level change: low, intermediate, and high. Between the latest full year of recorded stages for Lake Maurepas (2018) and the project baseline year (2026), the low, intermediate, and high estimates of sea level rise are 0.2 ft, 0.2 ft, and 0.4 ft, respectively.

Between the project baseline year (2026) and the 50-year project life (2076), the low, intermediate, and high estimates of sea level rise are 1.37 ft, 1.90 ft, and 3.56 ft, respectively. The AR&T Project Delivery Team (PDT) determined that the intermediate rate of sea level rise should be used in this project for future conditions model runs.

USACE Curves computed using criteria in USACE EC 1165-2-212

Gauge 85550: Lake Pontchartrain at Frenier: Jan 1950 to Dec 2002 All values are in feet			
Year	USACE Low	USACE Int	USACE High
2018	0.7	0.8	1.0
2019	0.7	0.8	1.0
2020	0.8	0.8	1.1
2021	0.8	0.9	1.1
2022	0.8	0.9	1.2
2023	0.9	0.9	1.2
2024	0.9	1.0	1.3
2025	0.9	1.0	1.3
2026	0.9	1.0	1.4

USACE Curves computed using criteria in USACE EC 1165-2-212

Gauge 85550: Lake Pontchartrain at Frenier: Jan 1950 to Dec 2002 All values are in feet			
Year	USACE Low	USACE Int	USACE High
2026	0.94	1.04	1.37
2031	1.07	1.21	1.64
2036	1.21	1.38	1.93
2041	1.35	1.56	2.24
2046	1.49	1.75	2.57
2051	1.63	1.94	2.92
2056	1.76	2.13	3.28
2061	1.90	2.32	3.67
2066	2.04	2.53	4.07
2071	2.18	2.73	4.49
2076	2.31	2.94	4.93

Figure G-7 shows the estimates of sea level rise for Lake Pontchartrain at Frenier.

USACE Curves computed using criteria in USACE EC 1165-2-212

Gauge 85550: Lake Pontchartrain at Frenier: Jan 1950 to Dec 2002 All values are in feet			
Year	USACE Low	USACE Int	USACE High
2018	0.7	0.8	1.0
2019	0.7	0.8	1.0
2020	0.8	0.8	1.1
2021	0.8	0.9	1.1
2022	0.8	0.9	1.2
2023	0.9	0.9	1.2
2024	0.9	1.0	1.3
2025	0.9	1.0	1.3
2026	0.9	1.0	1.4

USACE Curves computed using criteria in USACE EC 1165-2-212

Gauge 85550: Lake Pontchartrain at Frenier: Jan 1950 to Dec 2002 All values are in feet			
Year	USACE Low	USACE Int	USACE High
2026	0.94	1.04	1.37
2031	1.07	1.21	1.64
2036	1.21	1.38	1.93
2041	1.35	1.56	2.24
2046	1.49	1.75	2.57
2051	1.63	1.94	2.92
2056	1.76	2.13	3.28
2061	1.90	2.32	3.67
2066	2.04	2.53	4.07
2071	2.18	2.73	4.49
2076	2.31	2.94	4.93

Figure G-7 – Estimated Sea Level Change from Sea-Level Calculator for Lake Pontchartrain at Frenier

Lake Maurepas is connected to Lake Pontchartrain via Pass Manchac and marshes. Lake Pontchartrain is connected to the Gulf of Mexico via The Rigolets and Chef Menteur Pass, as well as marshes. Through this connection of Lake Maurepas to the Gulf of Mexico, there is some tidal influence in Lake Maurepas. From review of the USACE gage 85420 Pass Manchac near Pontchatoula, which is located in the eastern end of Lake Maurepas, the tidal range is approximately 0.2 feet from peak to trough. From analysis of the sensitivity of the Amite River basin to small differences in downstream boundaries, this difference is negligible.

Additional hydraulic modeling is planned for after the TSP is selected, for purposes of checking project performance sensitivity against the low and high estimates of sea level rise. The differences between intermediate and low, and intermediate and high are approximately 1.5 feet each. There is fairly low sensitivity of the Amite River Basin to differences in the Lake Maurepas stage, especially for areas with a significant number of structures, which are mostly in the middle portion of the basin. Because of the relatively small differences between sea level rise forecasts, and the low sensitivity of the Amite River Basin to stages in Lake Maurepas, future modeling of low and high sea level rise estimates is not expected to have a significant impact on project performance.

4.6 Climate Vulnerability

Climate vulnerability was assessed to determine if the USACE's mission of flood risk management is vulnerable to climate change in the Amite River Basin. USACE's Screening-Level Climate Change Vulnerability Assessment Tool at the Watershed_Scale, which assesses vulnerabilities to climate change for USACE's missions, was used for this assessment. For the Lower Mississippi-Lake Maurepas watershed (hydrologic unit code-4 (HUC-4) watershed 0807), which includes the Amite River basin, no vulnerability to Flood Risk Reduction was found. The only vulnerability found for HUC-4 watershed 0807 was for the Recreation business line for the Dry – 2085 scenario & Epoch, as shown in

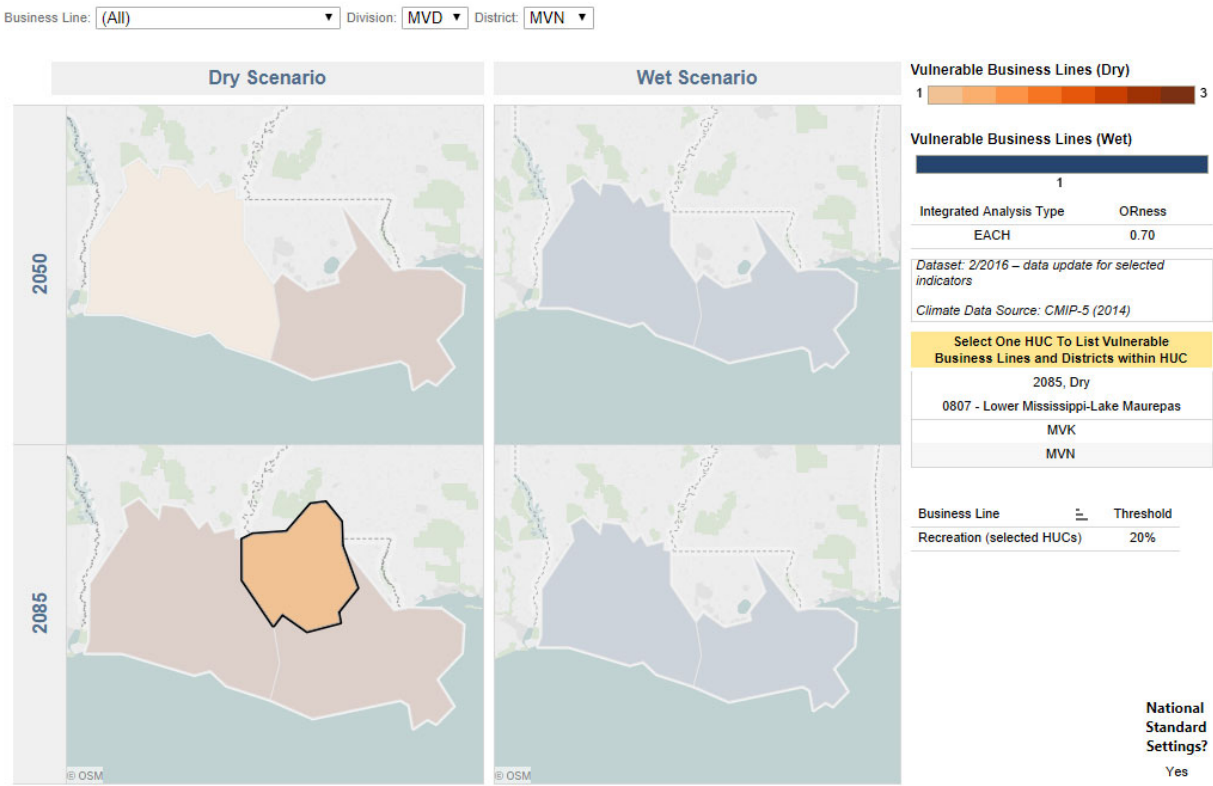


Figure G-8.

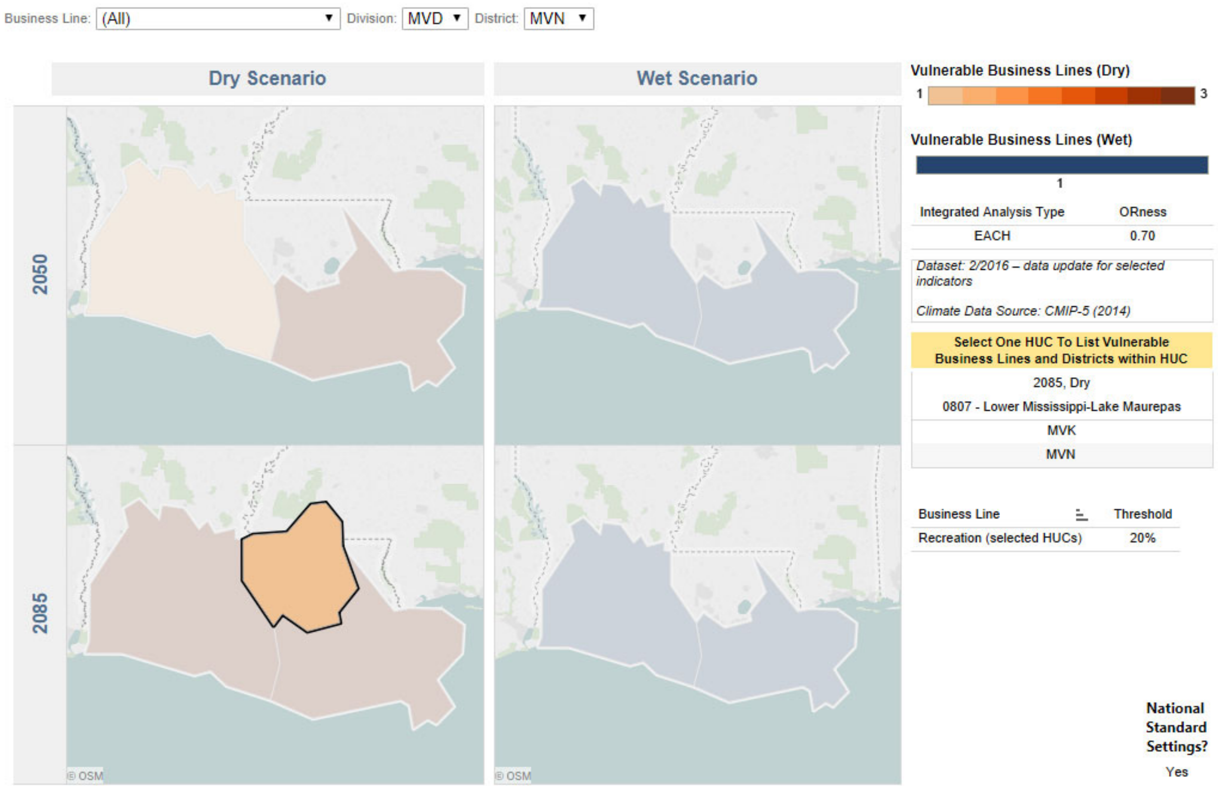


Figure G-8 – Scenario Comparison Over Time map for MVN. The only vulnerability shown for HUC-4 watershed 0807 is for recreation.

4.7 Hydrologic Modeling

Hydrologic modeling was performed using the HEC-HMS model provided by the LaDOTD. The hydrologic model domain covers the entire Amite River Basin, from southern Mississippi to southeast Louisiana. Figure G-9 shows the geometry of the hydrologic model.

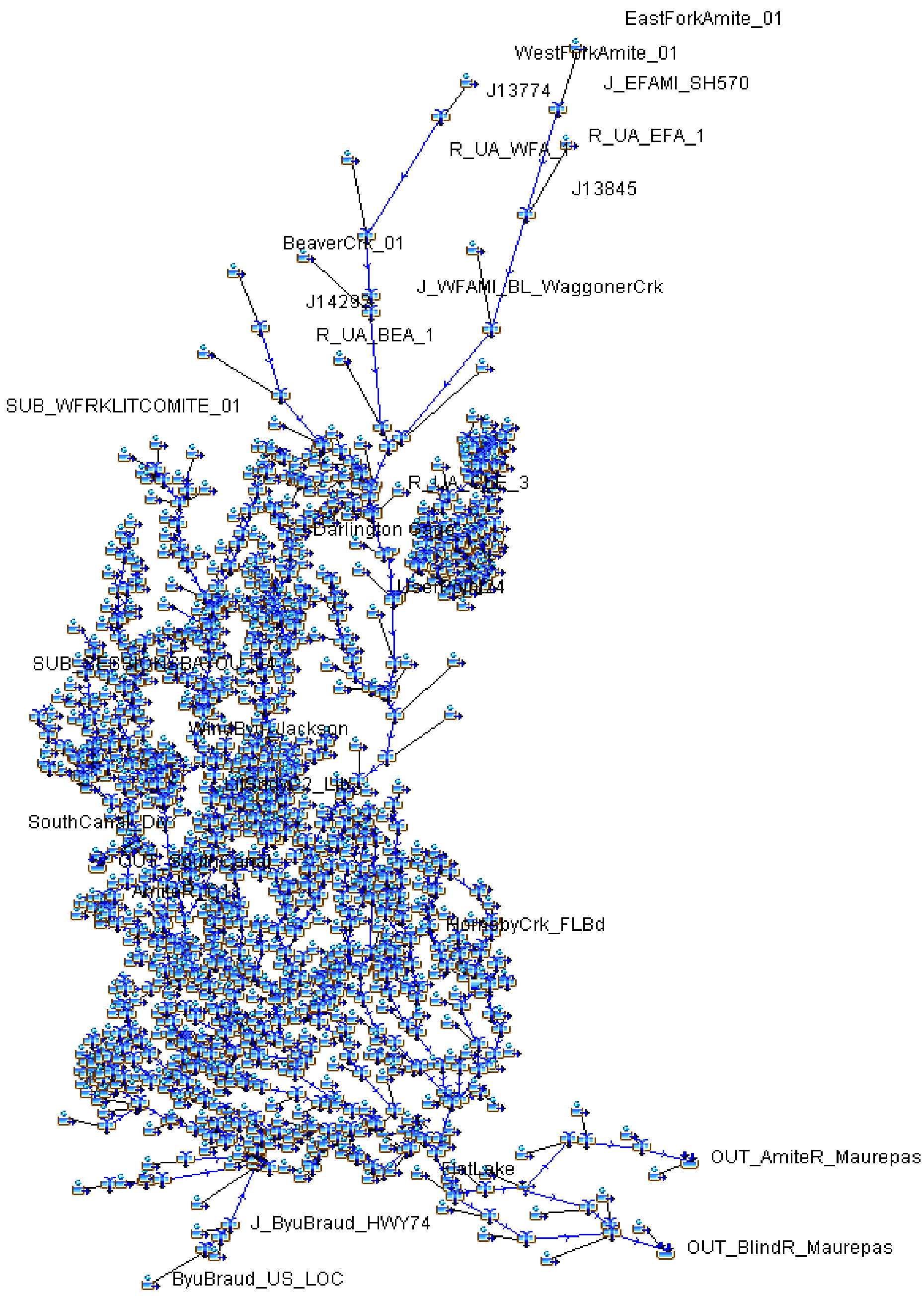


Figure G-9 – Hydrologic Model Geometry

Initial abstraction and infiltration losses were calculated by the hydrologic model based on runoff coefficients and imperviousness parameters. The model routed the runoff and spatially distributed it to 422 riverine output locations that were utilized as unsteady inflow boundary conditions in the hydraulic model. Figure G-10 shows the sub-basins and junctions for Claycut

Bayou, a tributary of the Amite River. A portion of those hydrologic nodes are used as model output locations.

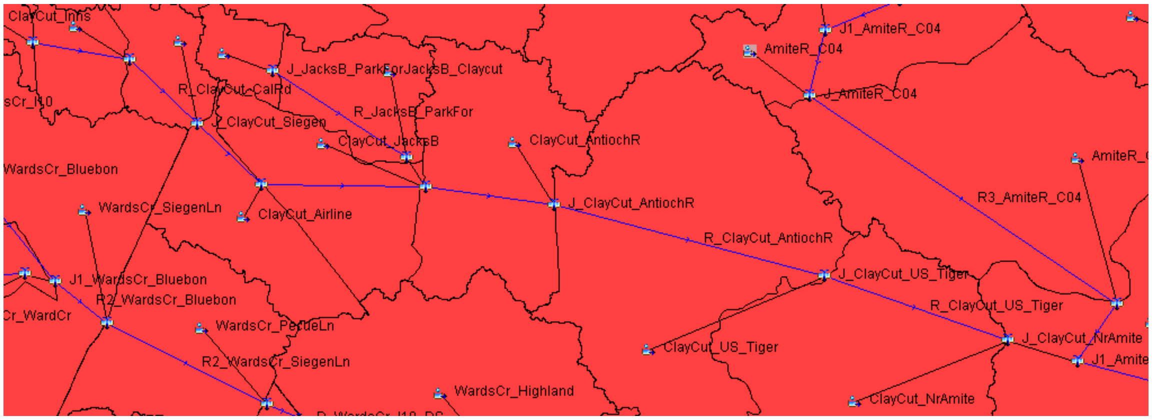


Figure G-10 – Example Hydrologic Nodes for Claycut Bayou

Each of the 24-hour AEP precipitation events was applied to the entire Amite River Basin in the HMS model. This was done with the existing model for the baseline year (2026), and with an adjusted imperviousness parameter for the future conditions (2076). Figure G-11 shows the 200 year precipitation hyetograph and flow output hydrograph for Sandy Creek near Mahoney Road.

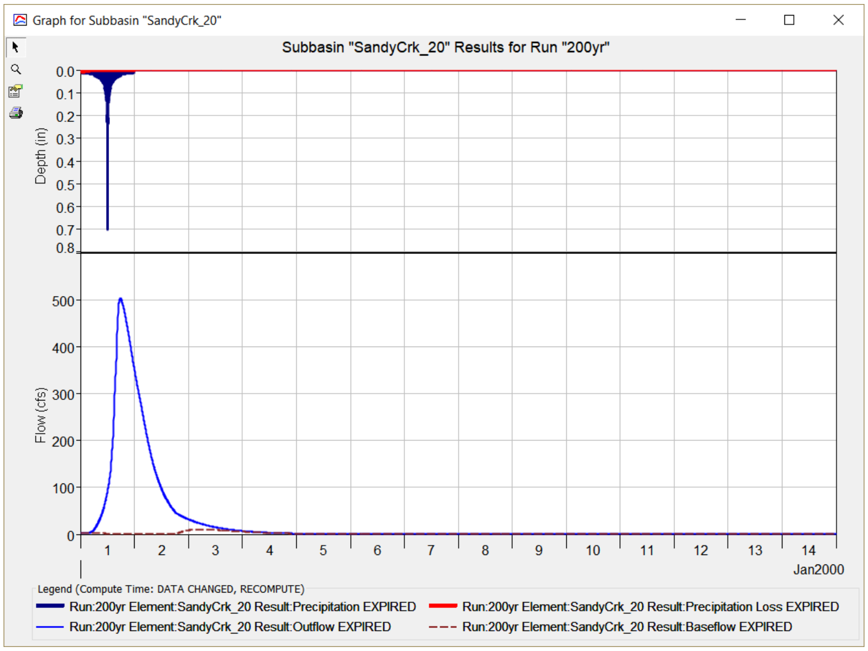


Figure G-11 – Example Precipitation Hyetograph and Flow Output Hydrograph

5.0 HYDRAULIC MODELING

5.1 Overview

Hydraulic modeling was performed using the HEC-RAS model obtained from the LaDOTD. The model is a one-dimensional/two-dimensional (1D/2D) unsteady flow hydraulic model. The model covers the Amite River Basin near the Louisiana/Mississippi border to the outlet of Amite River at Lake Maurepas. The model does not cover the portion of the Amite River Basin that is north of the state border. The datum of the model is NAVD 1988 (Geoid 12B).

Two versions of the model geometry were utilized in this modeling effort. One model geometry represents the Amite River Basin baseline conditions. That geometry was used for baseline runs, FWOP runs, and all alternative runs except for Darlington Dam. The second model geometry represents the Amite River Basin with Darlington Dam. That geometry was used for the Darlington Dam alternative runs. Figure G-12 shows the two model's domains.

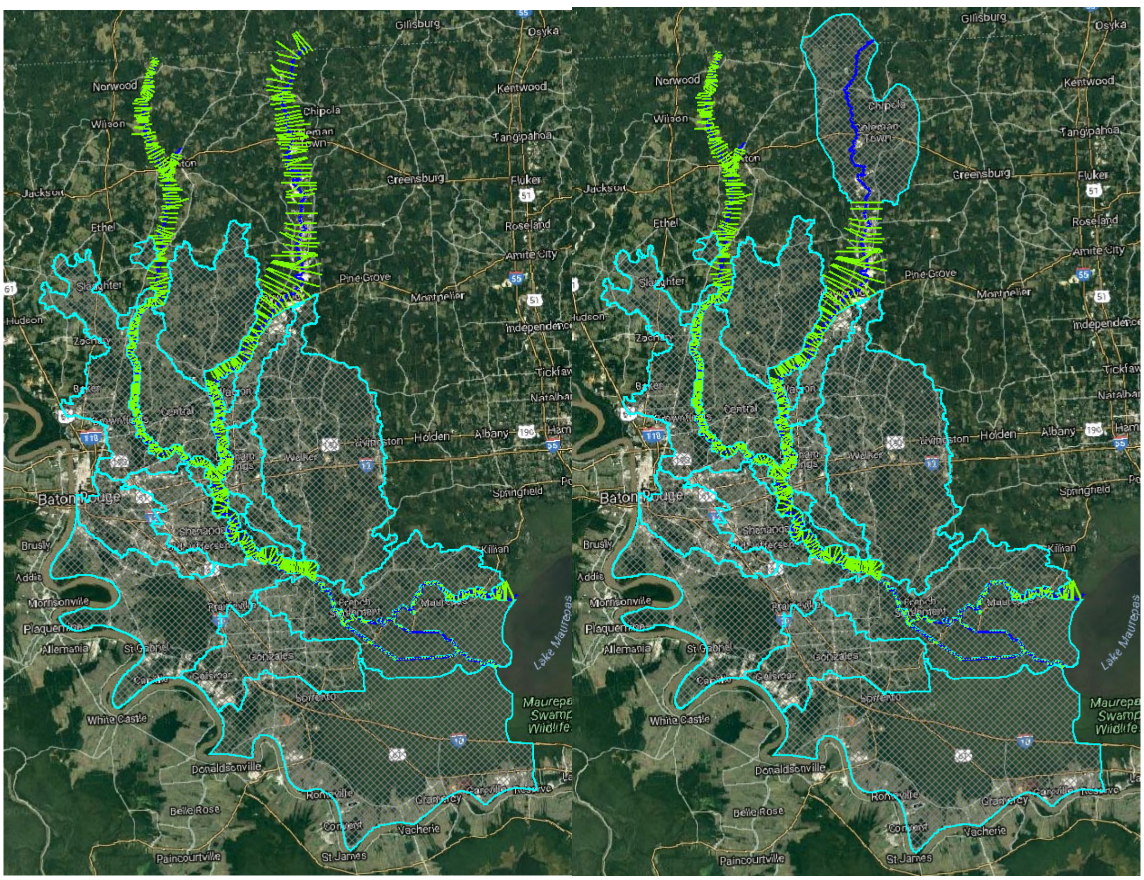


Figure G-12 – Baseline (left) and with Darlington Dam (right) HEC-RAS Model Domains

5.2 Boundary Conditions

Inflow boundary conditions to the hydraulic model were imported from results of the hydrologic model. There are three types of inflow boundary conditions in this hydraulic model: 1D inflow hydrographs, lateral inflow hydrographs, and 2D inflow hydrographs. There are two types of downstream boundary conditions in this hydraulic model: 1D stage hydrographs and 2D stage hydrographs.

5.2.1 1D Inflow Hydrographs

The upstream boundaries of the 1D portion of the hydraulic model are the Amite River and the Comite River near the Mississippi-Louisiana border, as well as Pretty Creek approximately 3 miles upstream of the Comite River. Inflow hydrographs are applied at those locations to represent flow from the portion of their basins that are upstream of the boundaries. Figure G-13, Figure G-14Figure G-15 show the locations of the upstream boundaries of the Amite River, Comite River, and Pretty Creek, as well as the upstream inflow hydrographs for those rivers for the 25 year baseline conditions.

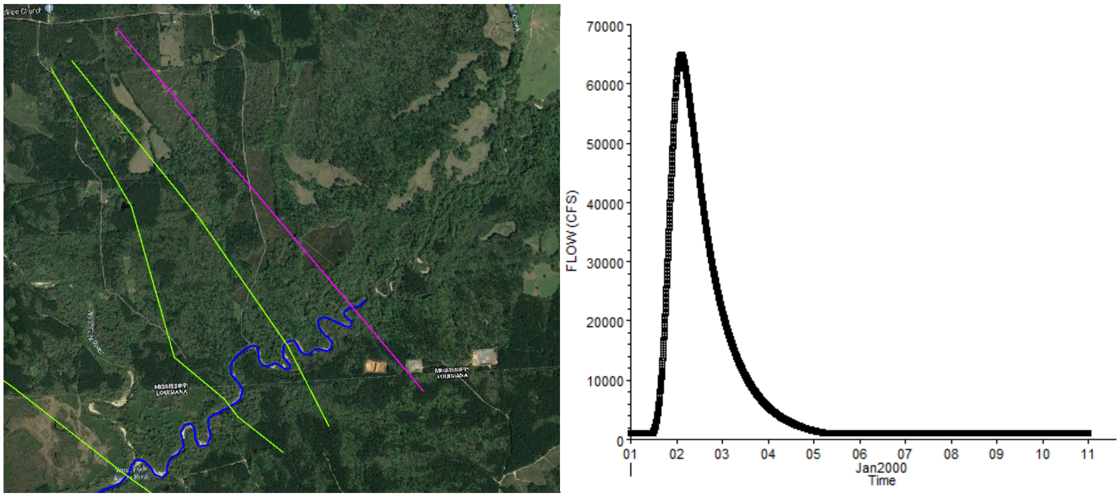


Figure G-13 – Amite River Upstream Boundary Location and 25 Year Baseline Inflow Hydrograph

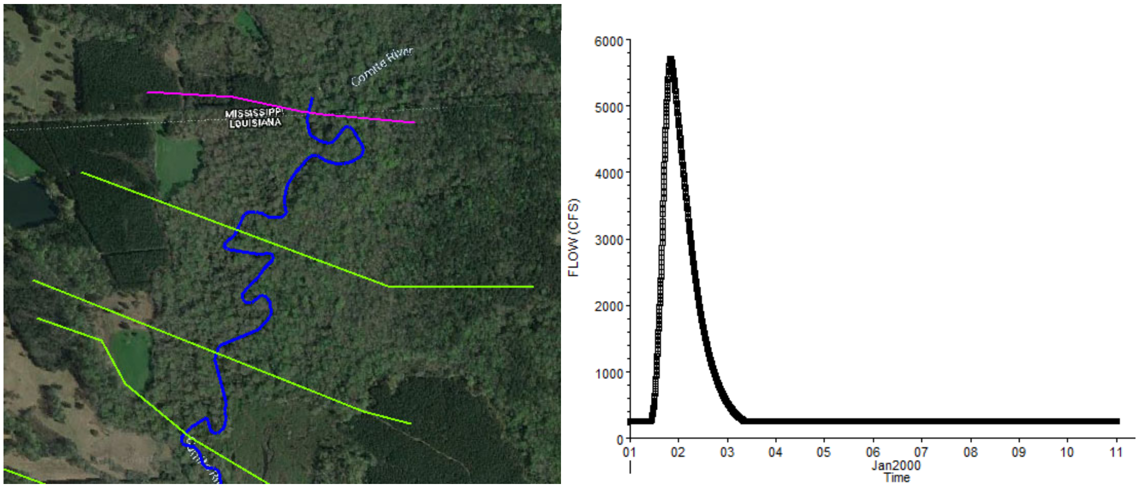


Figure G-14 – Comite River Upstream Boundary Location and 25 Year Baseline Inflow Hydrograph

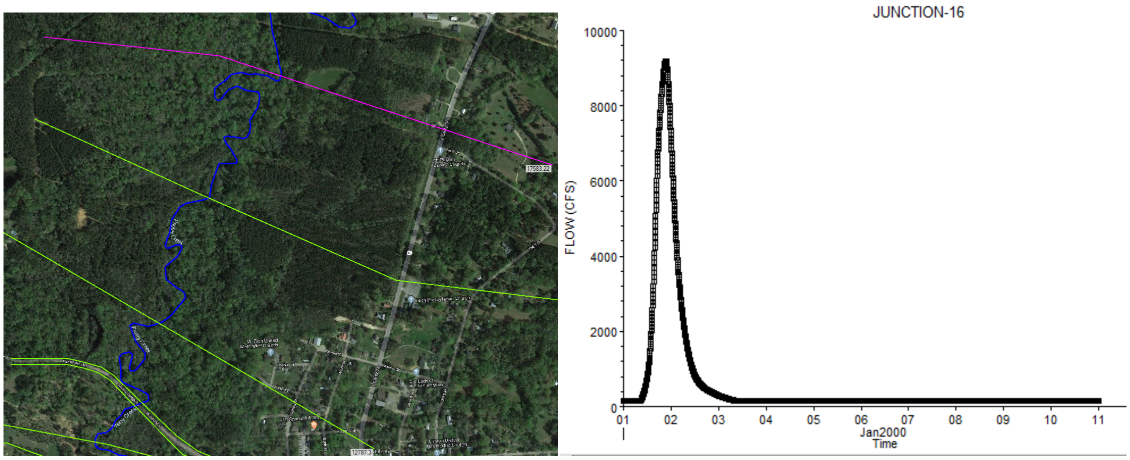


Figure G-15 – Pretty Creek Upstream Boundary Location and 25 Year Baseline Inflow Hydrograph

5.2.2 Lateral Inflow Hydrographs

Inflow hydrographs are also applied to 1D portions of the model in the form of lateral inflow hydrographs. These hydrographs represent flow from basins that are either not included in the 2D domain or that are near intersections of the 1D and 2D domains. There are 99 lateral inflow hydrographs in the baseline model, and 91 in the Darlington Dam model. Figure G-16 shows the location of the lateral inflow hydrograph that represents flow from Bluff Creek into the Amite River. Figure G-17 shows the lateral inflow hydrograph for the Amite River at Bluff Creek for 25 year baseline conditions.

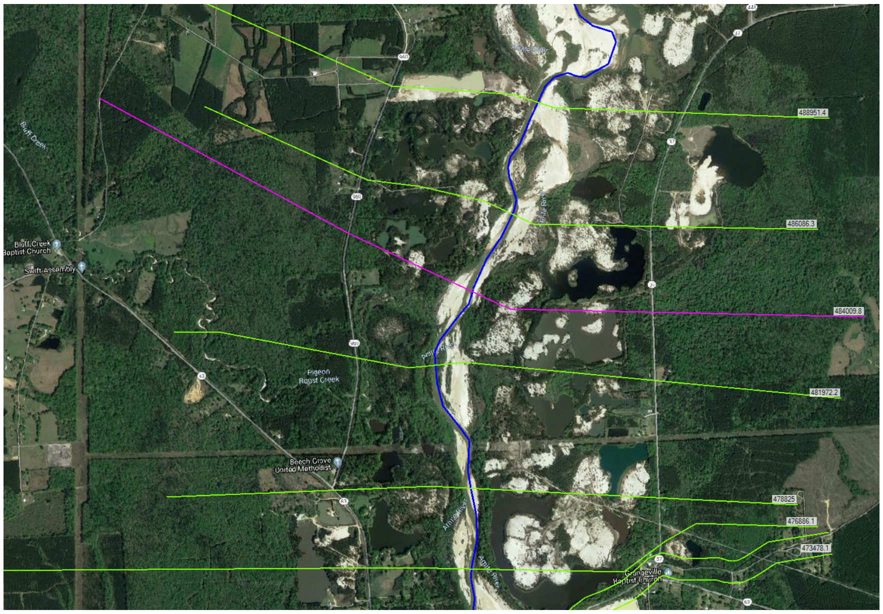


Figure G-16 – Lateral Inflow Location Representing Flow from Bluff Creek into the Amite River

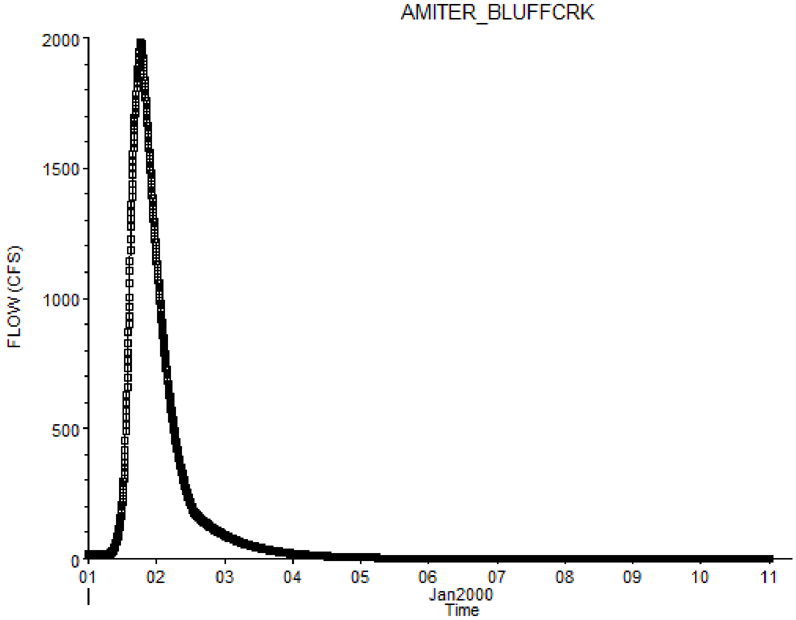


Figure G-17 – Lateral Inflow Hydrograph for the Amite River at Bluff Creek (25 Year Baseline)

5.2.3 2D Inflow Hydrographs

Inflow hydrographs are applied to the 2D portions of the model at 2D boundary condition lines. 2D boundary condition lines are located at intervals along tributaries of the Amite and Comite Rivers, as well as smaller streams that flow to those tributaries. These hydrographs represent the runoff from local rainfall, as well as rainfall from areas upstream that is not captured at another boundary condition line. There are 320 2D boundary condition lines in the baseline model, and 328 2D boundary condition lines in the Darlington Dam model. Figure G-18 shows the location of the 2D inflow hydrograph that inputs flow to Claycut Bayou near Airline Highway. Figure G-19 shows the inflow hydrograph for runoff into Claycut Bayou near Airline Highway for 25 year baseline conditions.

Figure G-18 – 2D Boundary Condition Line for flow into Claycut Bayou near Airline Highway

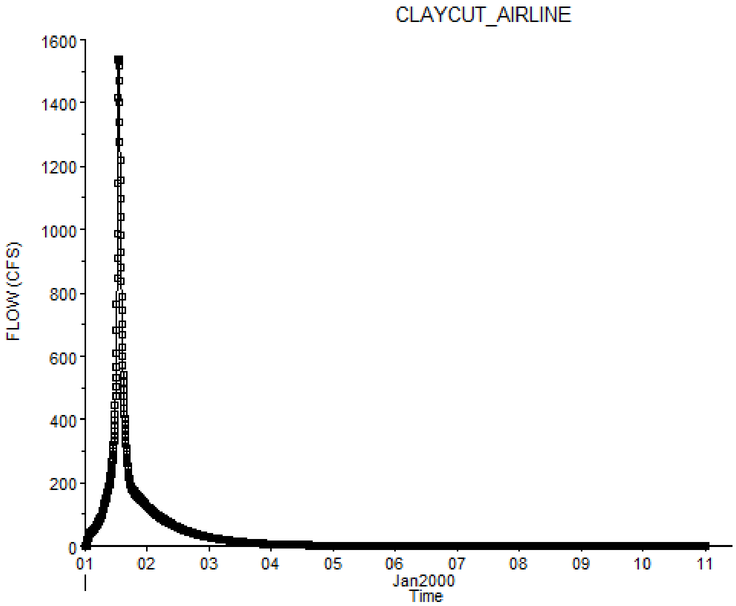


Figure G-19 – 2D Inflow Hydrograph for flow into Claycut Bayou near Airline Highway (25 Year Baseline)

5.2.4 Stage Hydrographs

The downstream boundaries of the hydraulic model are stage boundaries that represent the water surface elevation of Lake Maurepas. Stage boundaries are used where the Amite River and Blind River enter Lake Maurepas. Stage boundaries are also used where the 2D domain interacts with Lake Maurepas. For baseline (year 2026) model runs, a high Lake Maurepas was determined from the USACE gage 85420 Pass Manchac near Pontchatoula, which is located in the eastern end of Lake Maurepas. An analysis of that gage for the year 2018 showed a high stage at that gage to be approximately 1.5 feet, as that stage was exceeded approximately 15% of the time. 0.2 feet of sea level rise (from the intermediate sea level rise estimate from 2018 to 2026) was added to that 1.5 feet, to produce a stage boundary of 1.7 ft. For future conditions (year 2076) model runs, 1.9 feet of sea level rise (from the intermediate sea level rise estimate from 2026 to 2076) was added to the Lake Maurepas stage, resulting in a stage boundary of 3.6 feet. Figure G-20 shows the locations of the downstream stage boundaries of the 1D reaches, and Figure G-21 shows the locations of the 2D stage boundary condition lines.

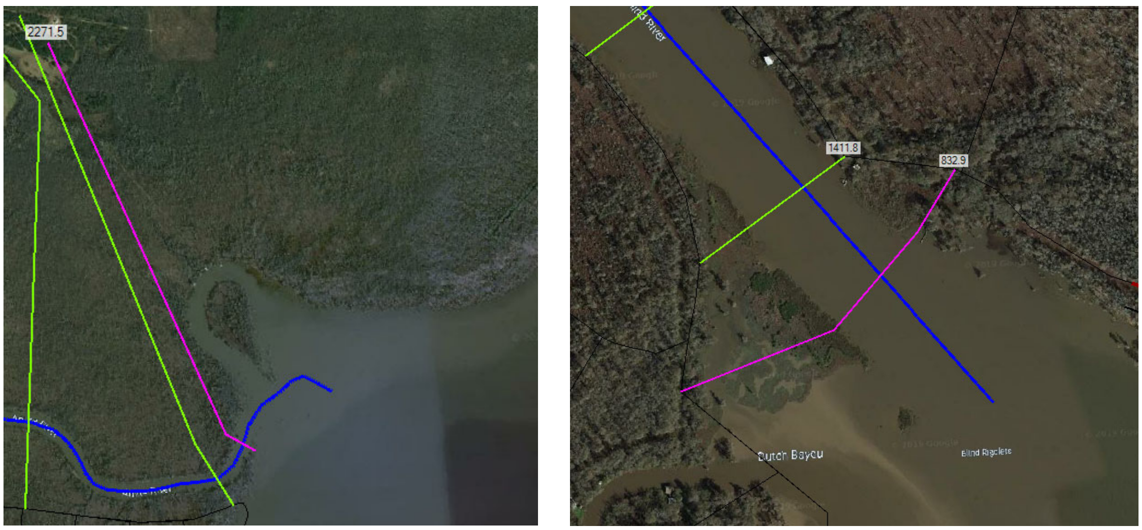


Figure G-20 – Stage Boundary Locations at Lake Maurepas for Amite River (left) & Blind River (right)

Figure G-21 – 2D Stage Boundary Locations at Lake Maurepas

5.3 Incorporation of Comite River Diversion and East Baton Rouge FRM Projects

Two other authorized projects in the Amite River Basin are projected to be complete prior to the baseline year of the Amite River and Tributaries FRM project (2026). Those projects are the Comite River Diversion (CRD) project and the East Baton Rouge (EBR) FRM project. The impacts of those projects were incorporated into this hydraulic modeling. The locations of those projects in East Baton Rouge Parish are shown in Figure G-22.

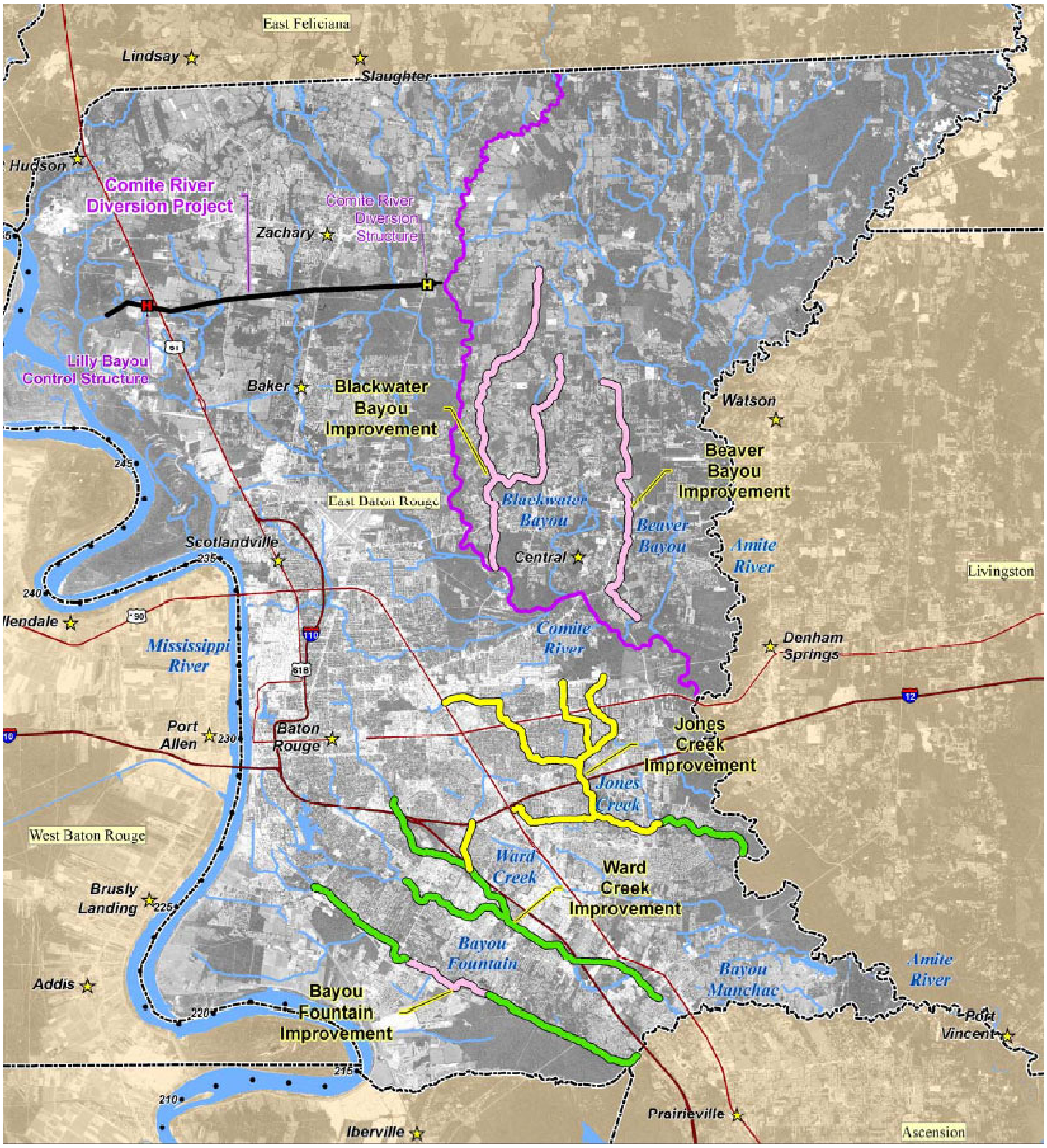


Figure G-22 – Locations of CRD and EBR Projects

5.3.1 Comite River Diversion Project

The Comite River Diversion will be located approximately 20 river miles upstream of the confluence of the Comite and Amite Rivers. Figure G-23 shows the expected location of the Comite River Diversion relative to the hydraulic model. The project will divert water from the Comite River west to the Mississippi River, between the cities of Zachary and Baker. The authorized diverted flows are based on flow rates in the Comite River immediately upstream of the diversion. To incorporate the impacts of the Comite River Diversion into this hydraulic modeling, a lateral diversion feature was implemented at the location of the diversion. The lateral diversion removes water from the Comite River based on a flow-flow rating curve. Figure G-24 shows the flow-flow rating curve. At the time of the writing of this HH&C Appendix, construction of the Comite River Diversion project has not been completed.

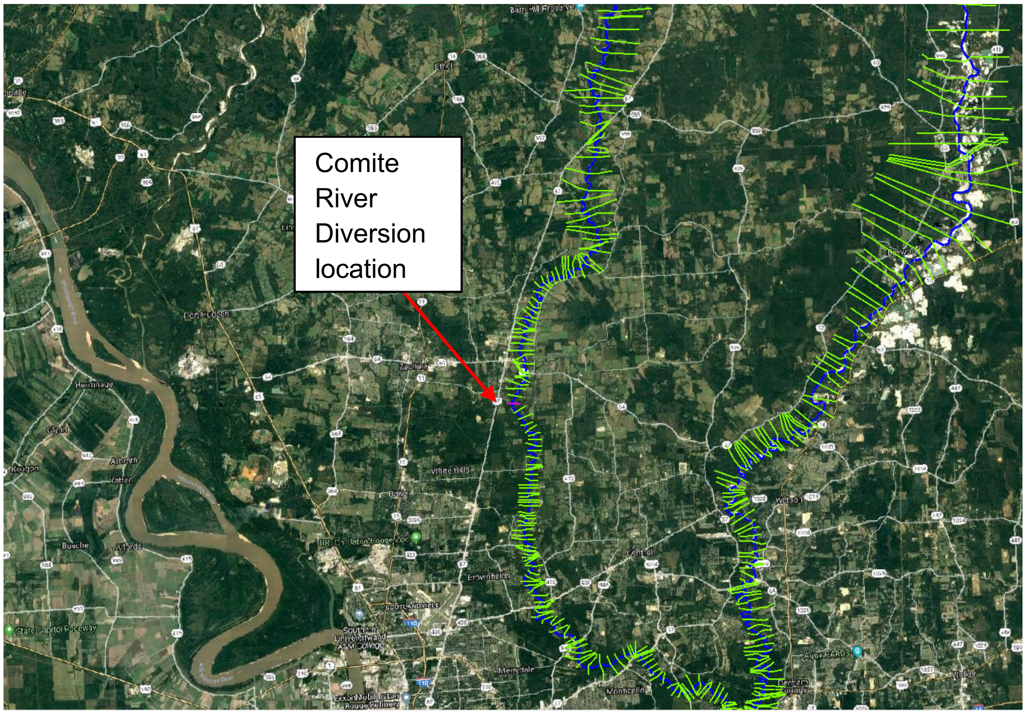


Figure G-23 – Location of Incorporation of Comite River Diversion Project into Hydraulic Model

Outlet Rating Curve

US Flow	Outlet Flow
0	0
6850	4450
10700	6150
16200	9300
22100	12700
28400	16800
37500	20800
45800	23900
50300	24900
56200	25800

Figure G-24 – Authorized Flow-Flow Rating Curve for Comite River Diversion

5.3.2 East Baton Rouge FRM Project

The authorized East Baton Rouge (EBR) FRM project includes projects on five separate streams: Beaver Bayou, Blackwater Bayou, Jones Creek, Ward Creek, and Bayou Fountain. The feasibility study for the EBR project reported flow rates that are expected at the downstream ends of the five streams with the authorized EBR projects in place. Because updated hydraulic modeling for the EBR projected has not yet been completed, the flow rates from the EBR feasibility study were used in this study's modeling. Figures Figure G-25, Figure G-26, and Figure G-27 show where the inflow hydrographs for the five EBR streams were applied to the hydraulic model. Table 1 lists the location in the hydraulic model where the flow for each EBR stream was applied.

Table 1
Hydraulic Model Locations for Application of EBR Stream Outflow

EBR Stream	1D River and Reach	Cross Section
Beaver Bayou	ComiteRiver Abv_AmiteR	22408.94
Blackwater Bayou	ComiteRiver Abv_AmiteR	52579.85
Jones Creek	AmiteRiver Blw_ComiteR	258117.4
EBR Stream	*2D Flow Area*	*Boundary Condition Line*
Wards Creek	BayouManchac	WardsCr_Manchac
Bayou Fountain	BayouManchac	BFount_ByuManch

The EBR feasibility study only reported maximum flows. Unsteady inflow hydrographs were needed for this study's hydraulic modeling. To created inflow hydrographs, HH&C used hydrographs from initial updated EBR modeling for each stream and scaled them to make their maximum flow equal to the flow from the feasibility study. An example of this scaling is shown in Figure G-28 for the Jones Creek 25 year baseline flow.

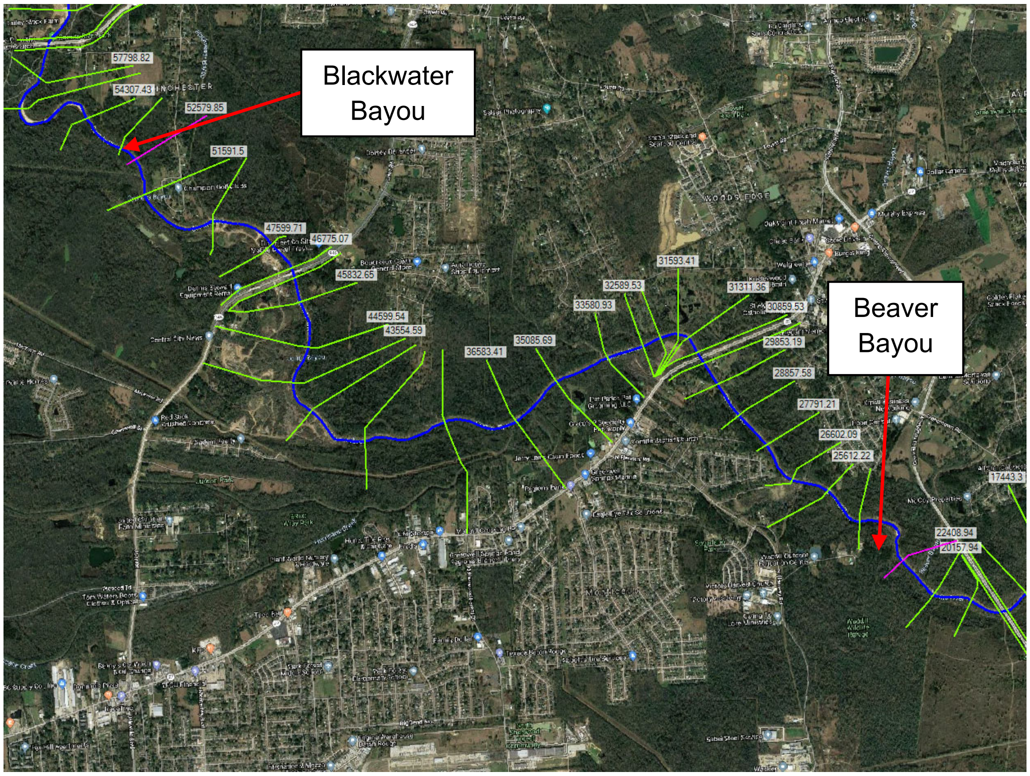


Figure G-25 – Cross Sections where Blackwater Bayou and Beaver Bayou EBR flows were applied

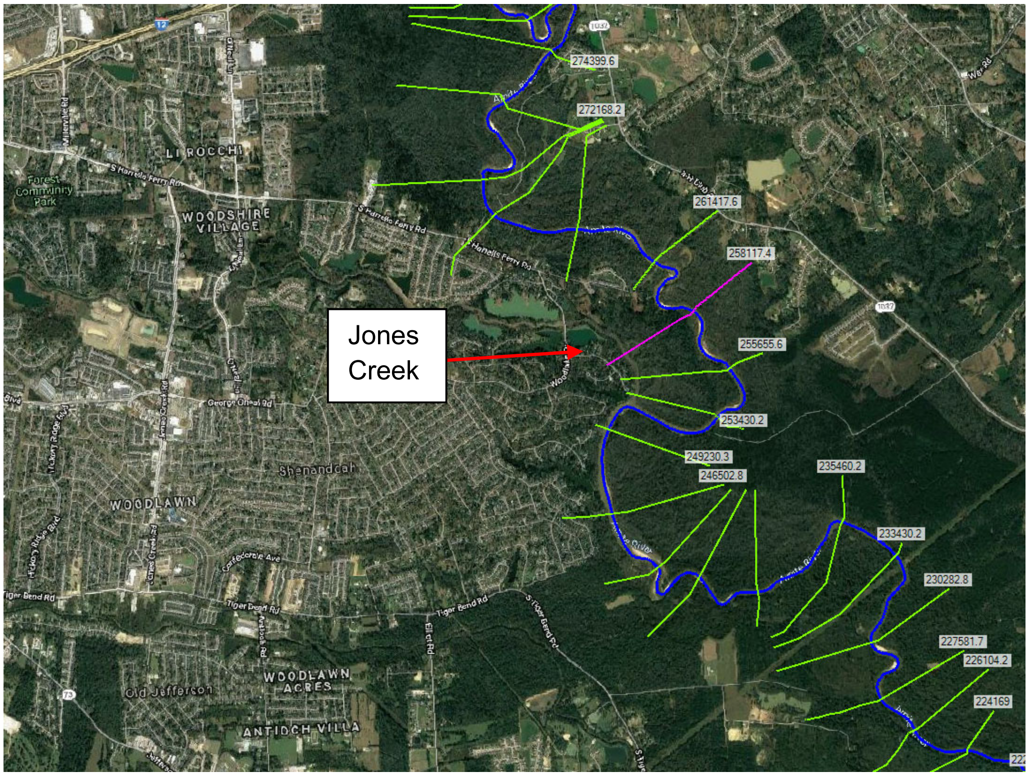


Figure G-26 – Cross Section where Jones Creek EBR Flows were applied

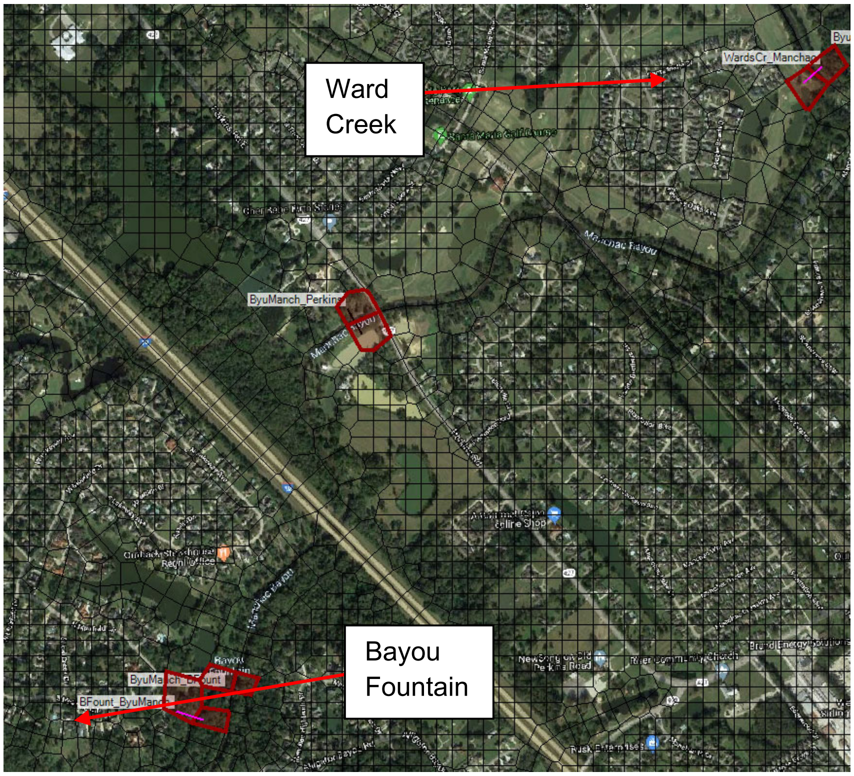


Figure G-27 – Cross Sections where Ward Creek and Bayou Fountain EBR Flows were applied

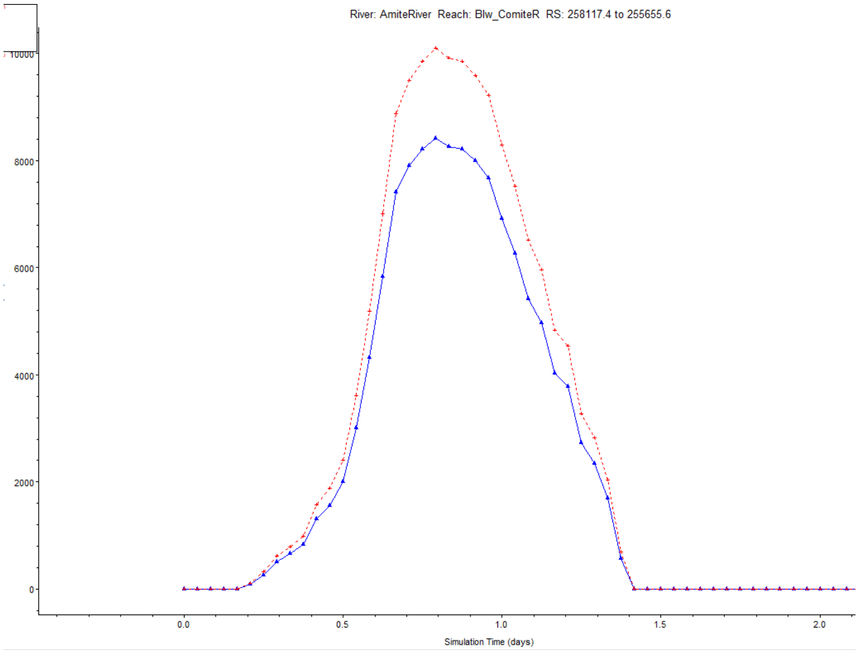


Figure G-28 – 25 Year Baseline Flow from initial Jones Creek H&H modeling (blue), Scaled to Match EBR Feasibility Flow (red)

5.4 Alternatives

5.4.1 Darlington Dam

Darlington Dam is a proposed dam on the Amite River near Darlington, Louisiana. The dam would provide FRM benefits by attenuating floodwater in its impoundment, and releasing water for an extended time at a lower rate, thus saving downstream areas from the peak flows of the upper Amite River.

This alternative was considered potentially effective for providing significant FRM benefits, so it was selected as an alternative to model. The Darlington Dam was modeled as a Dry Dam, meaning that it began with no water in the impoundment. This allowed for maximum storage capacity for purposes of evaluating potential effectiveness.

The Darlington Dam model obtained from LaDOTD utilized a 100-year dam design. For this modeling effort, HH&C was tasked with modeling the 25-year dry dam. HH&C edited the 2D area connection of the Darlington Dam to represent the 25-year dry dam. Those edits included lowering the dam crest and the emergency spillway elevation. When the water surface elevation in the impoundment is below the elevation of the emergency spillway, water flows through the dam via the low level outlet, which is three 10-ft by 10-ft culverts at the base of the dam. When the water surface is higher than the emergency spillway, the low level outlet is closed. In order to properly represent this operation of the dam outlets in the model, stage-flow rating curves were calculated from model results for both the low level outlet and the emergency spillway. Those curves were combined into a single stage-flow rating curve that was applied to the 2D area connection of the Darlington Dam.

5.4.2 Lily Bayou, Bluff Creek, and Darlington Creek Dry Detention Ponds (Alternative 8A)

The Lily Bayou, Bluff Creek, and Darlington Creek dry detention ponds are dams on three tributaries of the upper Amite River. The dams would provide FRM benefits by attenuating floodwater in their impoundments, and releasing water for an extended time at lower rates, thus saving the Amite River Basin from the peak flows of the three streams.

This alternative was considered potentially effective for providing significant FRM benefits, so it was selected as an alternative to model. This alternative was modeled by assuming that all of the flow upstream of each detention pond would be stored in the ponds for every flood event.

5.4.3 Sandy Creek Dry Detention Pond (Alternative 8C)

Sandy Creek Dry Detention Pond is a dam on Sandy Creek, a right bank tributary of the Amite River. The dam would provide FRM benefits by attenuating floodwater in its impoundment, and releasing water for an extended time at a lower rate, thus saving the lower Sandy Creek Basin and the lower Amite River Basin from the peak flows of upper Sandy Creek.
This alternative was considered potentially effective for providing significant FRM benefits, so it was selected as an alternative to model. This alternative was modeled by assuming that all of the flow upstream of the detention pond would be stored in the pond for every flood event.

5.4.4 Spanish Lake Pump Station and Gate Operation

The Spanish Lake area and surrounding bayous (Bayou Fountain and Bayou Manchac) historically flood due to backwater from the Amite River. A pump station that collects water from the northwest portion of Spanish Lake and pumps to the Mississippi River was originally considered to divert incoming floodwaters flowing upstream up Bayou Manchac. That alternative was modeled with the 100 year event, and it was determined that the influence area of a pump station in that location could not have significant FRM benefits to the Spanish Lake area. A pump station located nearer to the confluence of Bayou Fountain and Bayou Manchac (near the entrance to Spanish Lake) was considered, as that could have a more significant influence area. But that pump station location was several miles from where it would pump water to in the Mississippi River, and thus was screened out due to cost.

This alternative was considered not economically feasible for FRM, and thus was not modeled for all AEP events.

5.4.5 Highway 22

Highway 22 crosses the Amite River Diversion approximately 3 miles downstream from the Amite River. For large events where there is significant flow out of the banks of the Amite River Diversion, Highway 22 acts as a barrier to flow. This causes backup of water upstream of Highway 22. Adding additional drainage underneath Highway 22, or turning Highway 22 into a short causeway, was considered as a way to mitigate the flow blockage. Both of these options were modeled with the 100 year event. Water levels were able to be lowered upstream of Highway 22, but it was determined that there were not enough structures in the region that could see benefit from this project.

This alternative was considered not beneficial enough to be modeled for all AEP events.

5.4.6 Port Vincent Bridge

Highway 42 crosses the Amite River at Port Vincent, Louisiana. The Port Vincent Bridge has several piers and a bridge deck that were assumed to act as a restriction to flow, causing an increase in water levels upstream of the bridge. Replacing the existing bridge with a clear span bridge and raising the bridge deck were considered as an alternative to mitigate the flow blockage. Evaluation of the impacts of the existing bridge for the 500 year event shows that water levels do not reach the elevation of the bridge deck. Several bridge piers are in the flow path, so conceivably a clear span bridge could show FRM benefits. But water levels upstream of the bridge could only be expected to be lowered by approximately one foot at the 500 year event, and by less than that for higher frequency events.

Based on the small expected hydraulic impact of the bridge, this alternative was not modeled for the suite of AEP events.

5.4.7 Amite River Re-meandering

Adding meanders to the Amite River above the Comite River was an alternative suggested recently by other federal agencies. The potential benefit is that there would be additional length in the river, and thus additional storage capacity, and floodwaters would be slowed down on their journey to inundate populated areas downstream. There are potential benefits from this alternative, especially at higher frequency events where the Amite River is still in its banks.

There are design and feasibility challenges with this alternative and the true potential for FRM benefits is quite unclear. At lower frequency events, the Amite River is out of its banks, and mostly flowing as sheet flow across the entire flood plain. In those cases, the shape and length of the river channel is less significant. Also, there would be difficulty in “adding” meanders to the river in a stable way. Man-made shaping of rivers in a “natural” manner requires a thorough understanding of river morphodynamics, and significant erosion control measures would need to be taken.

While this alternative could yield FRM benefits downstream of the re-meandered region, it would likely be significant for only the higher frequency events. For lower frequency events, the potential benefits would be negligible. The total benefits from this project would likely be on a smaller order of magnitude than the benefits from the various dam alternatives. Due to the low expected relative benefits from this alternative, and the significant engineering challenges associated with the restoration of meanders, this alternative was not modeled for the suite of AEP events.

5.4.8 Highway 16

Highway 16 crosses Colyell Creek south of Port Vincent, Louisiana, approximately one mile upstream from the confluence with the Amite River. The Highway 16 Bridge has several piers and a bridge deck that are assumed to act as a restriction to flow, causing an increase in water levels upstream of the bridge. Due to the relative small size of Colyell Creek, the Highway 16 Bridge was not included in the hydraulic model that was used for this modeling effort. Analysis of the potential impacts of this bridge for the 200 year event show that the likely elevation of the bridge deck is above the peak water surface. The bridge deck is likely not a restriction to flow to any of the model events except for the 500 year. In order to model this alternative, a survey of the existing Highway 16 Bridge would be required, as well as further refinement of the hydraulic model.

There is a low density of structures in the region where water backs up behind the Highway 16 Bridge. Based on the low density of structures in the region, the lack of survey data for the bridge, and the small expected hydraulic impact of the bridge deck, this alternative was not modeled for the suite of AEP events.

5.5 Results

Hydraulic model runs were made for the full suite of eight 24-hour AEP events (0.5, 0.2, 0.1, 0.04, 0.02, 0.01, 0.005, and 0.002) for baseline without project (2026) and FWOP (2076). Model runs were also made for the full suite of eight 24-hour AEP events for three alternatives: Darlington Dam, Alternative 8A, and Alternative 8C. All alternative model runs were made using the baseline (2026) hydrology. Figures G-29 and G-30 show stages for the six lower frequency events (0.1, 0.04, 0.02, 0.01, 0.005, and 0.002) for baseline and alternative runs at two relevant locations on the Amite River: Denham Springs and Port Vincent.

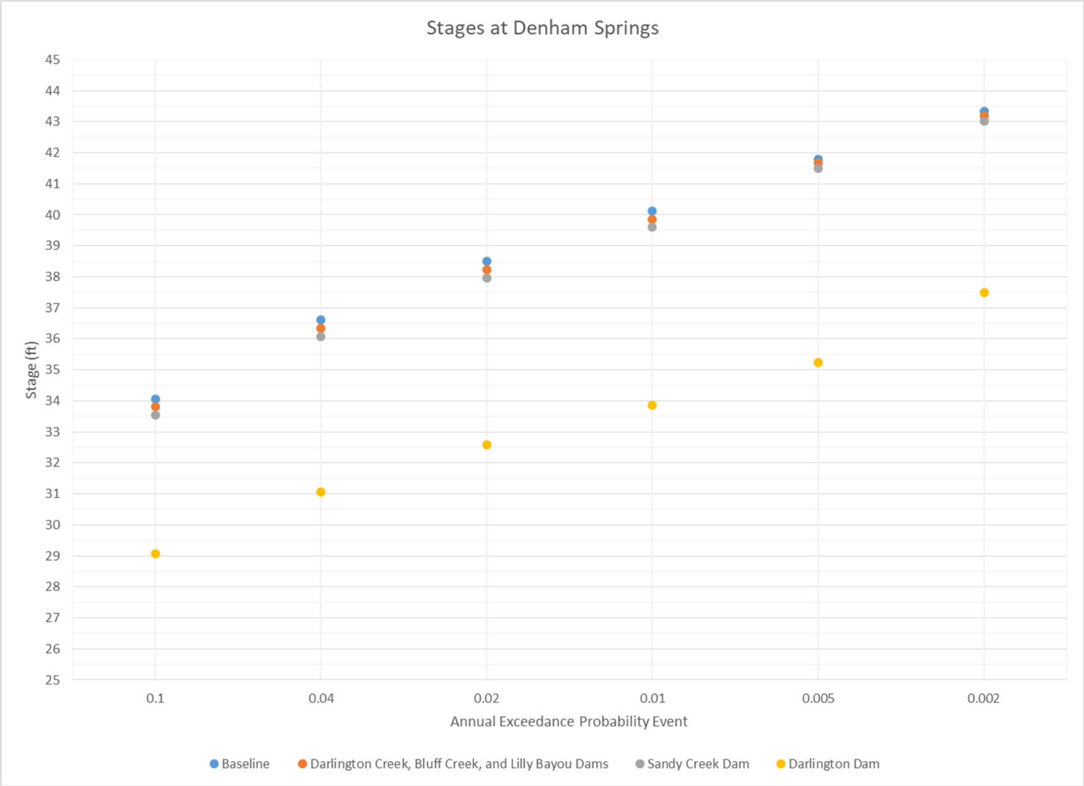


Figure G-29 – Stages at Denham Springs

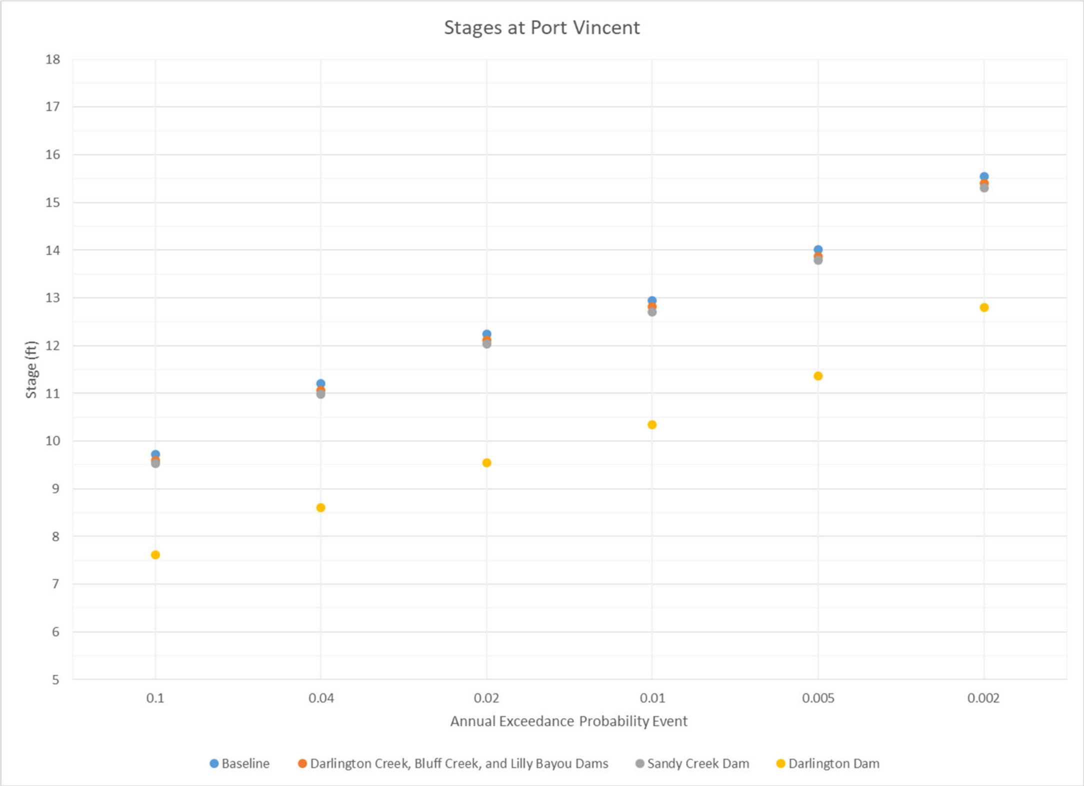


Figure G-30 – Stages at Port Vincent

This section shows results of the 10 year and 500 year model runs for each of the three modeled alternative and FWOP, compared against baseline results. The 10 year (Figure G-29 – Figure G-40) and 500 year (Figure G-41 – Figure G-52) results were selected for presentation in this document because they represent a higher frequency event (10 year) and lower frequency event (500 year). Water surface elevation profiles are shown on the Amite River, because that is where the most significant impacts are seen. Maximum inundation maps for the entire hydraulic model domain are also included.

Results of hydraulic modeling were used to generate water surface elevation and depth grids for every alternative for the full suite of eight 24-hour AEP events. Those results grids were provided to the GIS and Economics branches for use in developing economics analyses.

10 Year Darlington Dam

Maximum Water Surface Profile: Amite River above Comite River

Figure G-29 – Darlington Dam (green) and Baseline (blue) on Amite River above Comite

10 Year Darlington Dam
Maximum Water Surface Profile: Amite River below Comite River

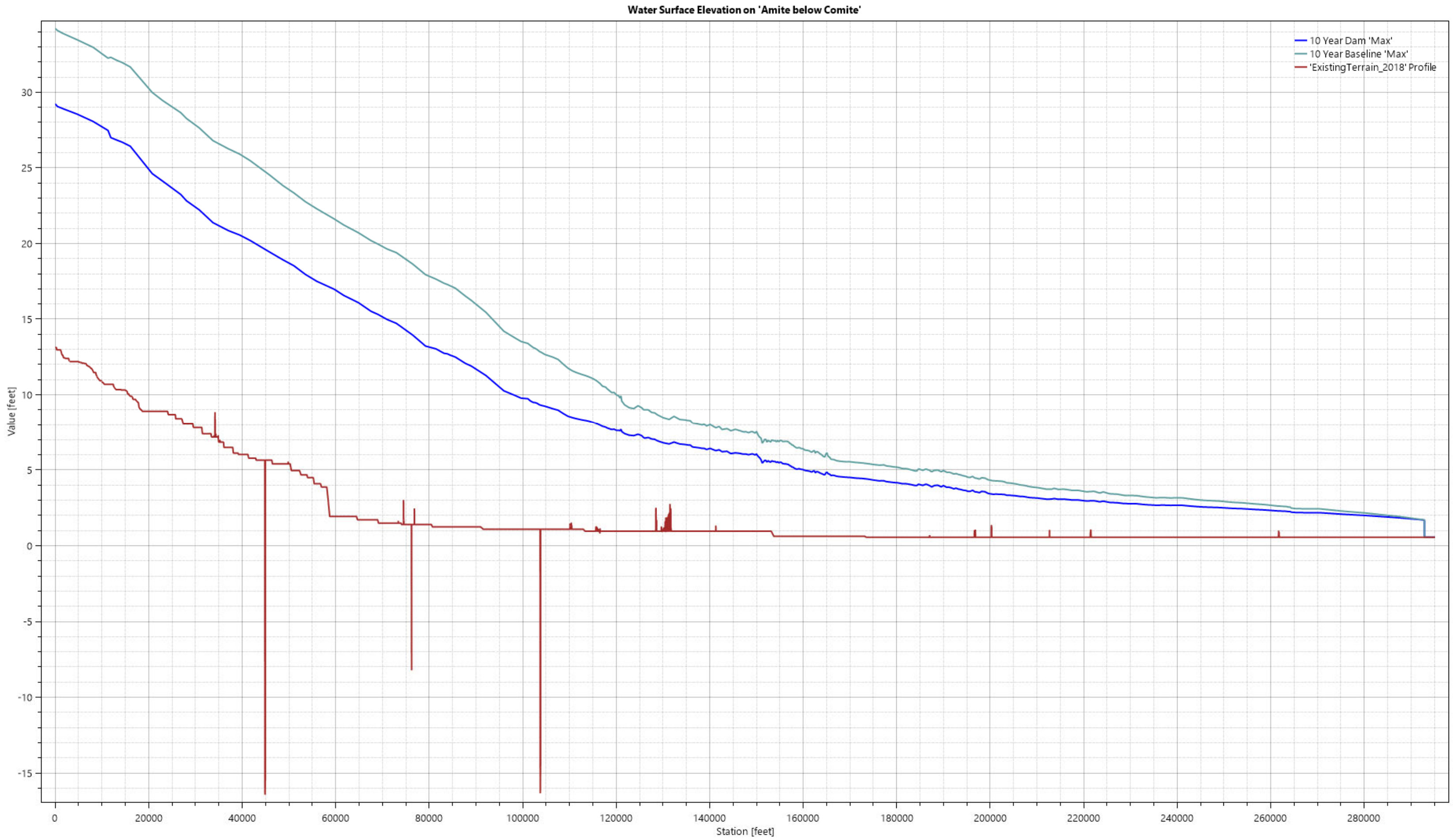


Figure G-30 – Darlington Dam (blue) and Baseline (green) on Amite River below Comite

10 Year Darlington Dam
Maximum Inundation

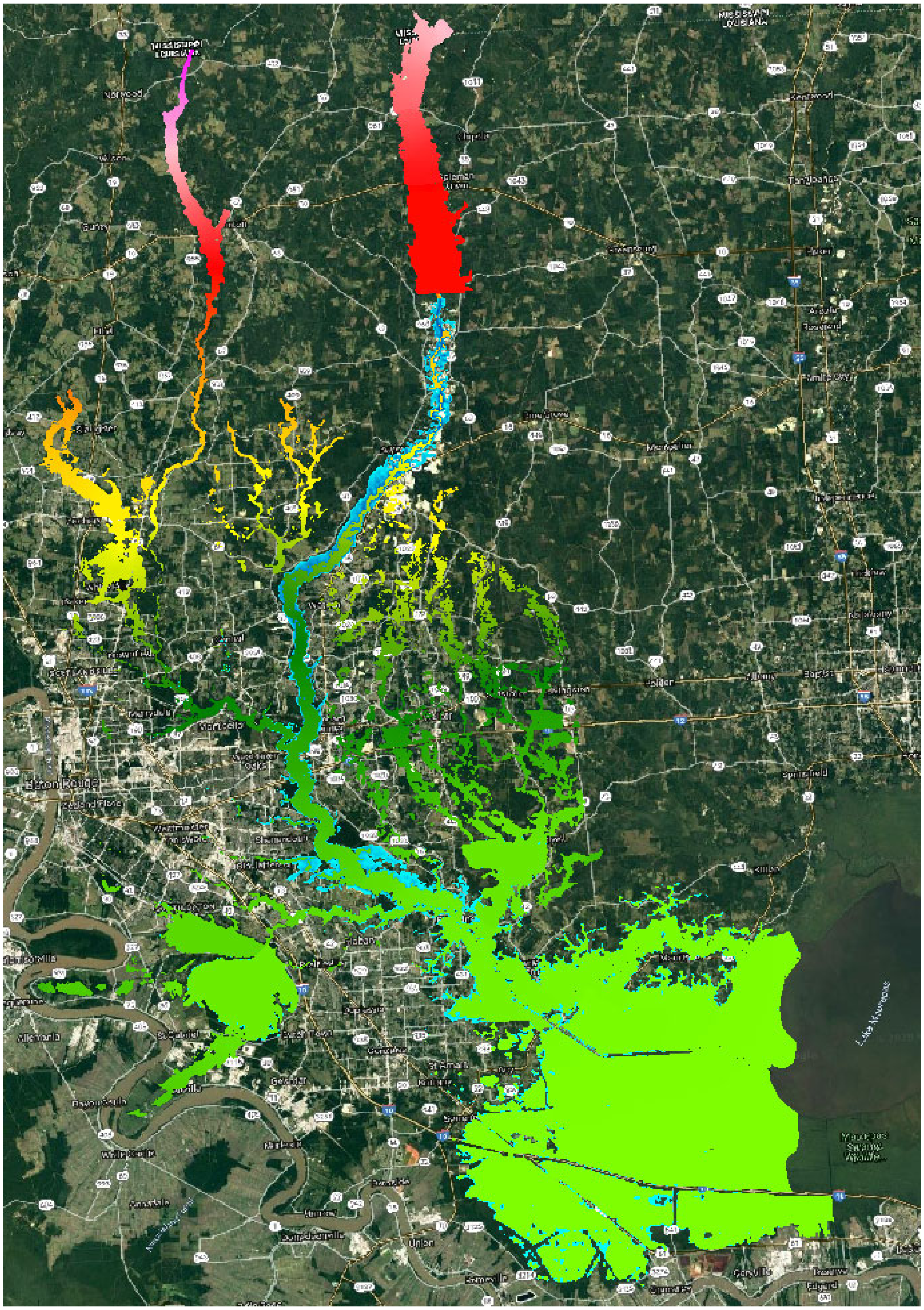


Figure G-31 – Darlington Dam (red-green scale) and Baseline (blue) Maximum Inundation Area

10 Year Alternative 8A

Maximum Water Surface Profile: Amite River above Comite River

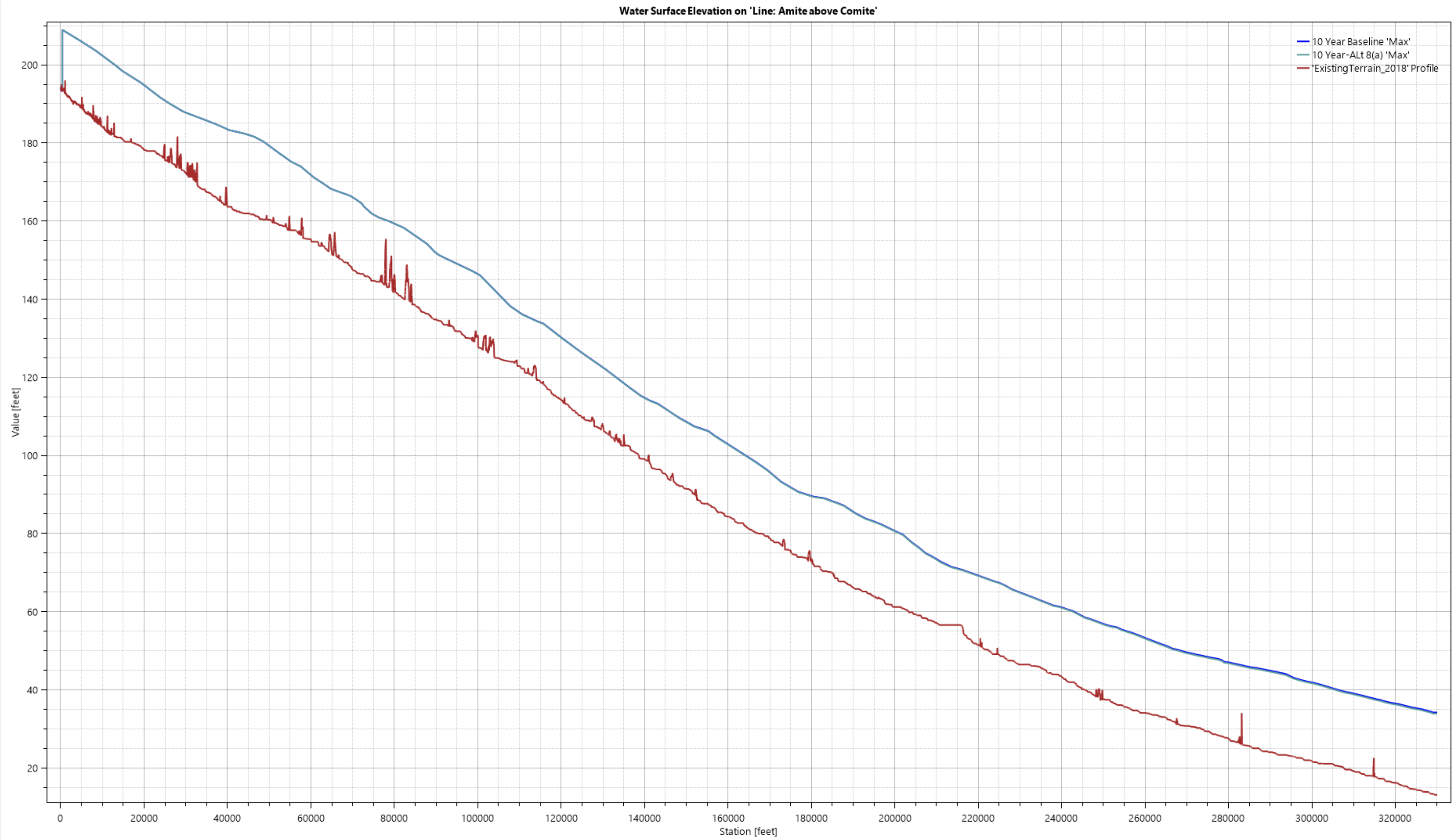


Figure G-32 – Alternative 8A (green) and Baseline (blue) on Amite River above Comite

10 Year Alternative 8A

Maximum Water Surface Profile: Amite River below Comite River

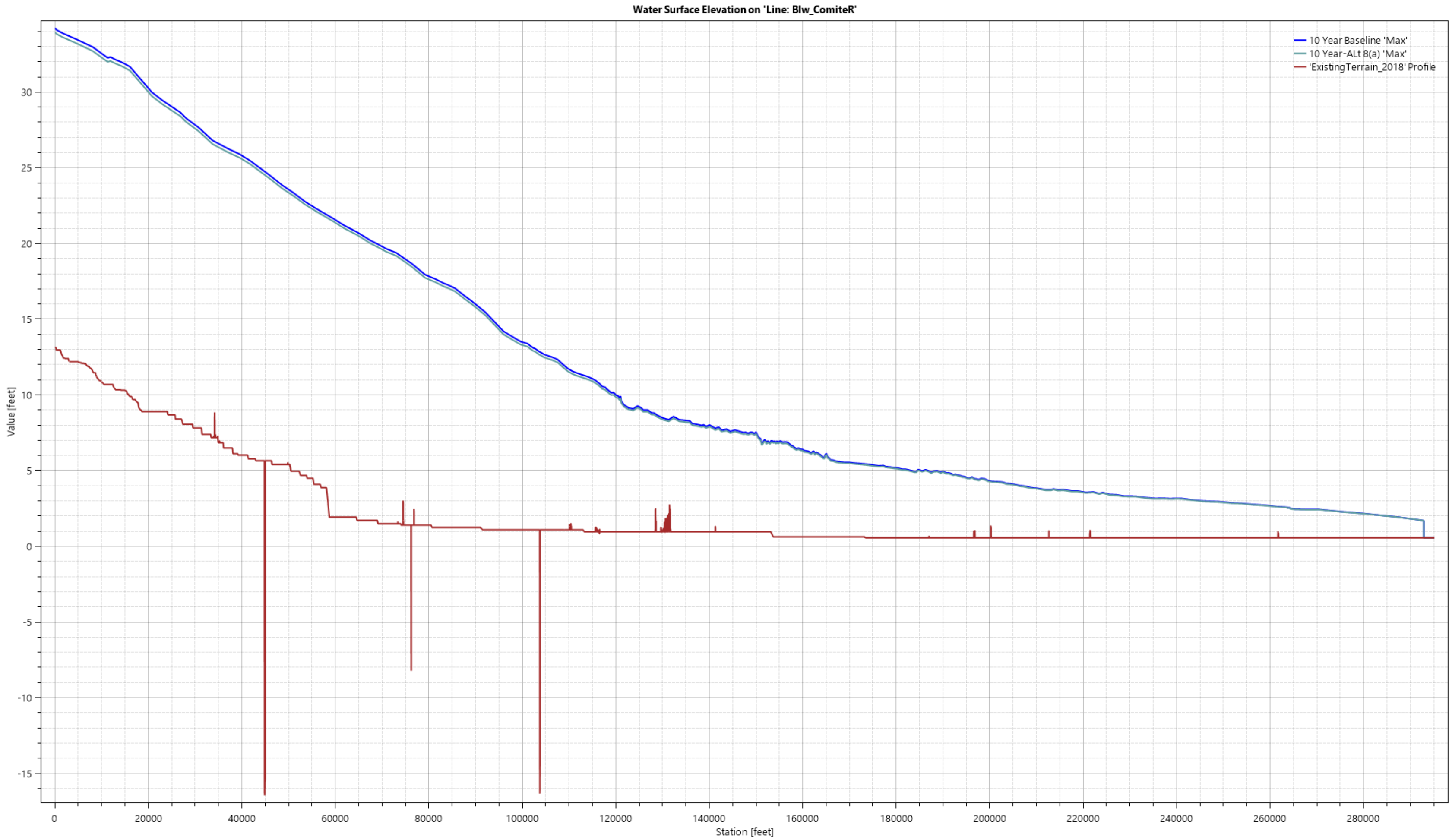


Figure G-33 – Alternative 8A (green) and Baseline (blue) on Amite River below Comite

10 Year Alternative 8A
Maximum Inundation

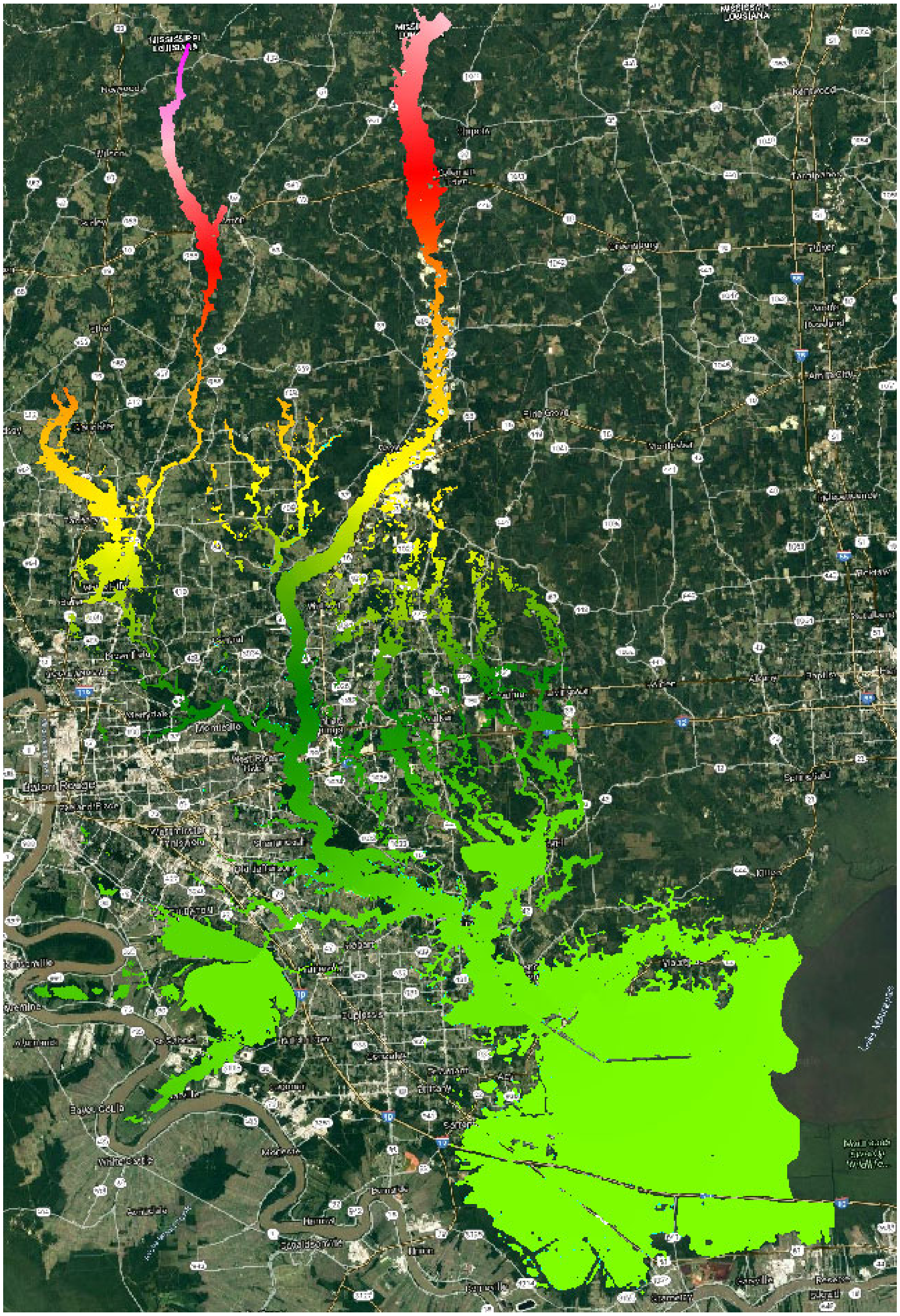


Figure G-34 – Alternative 8A (red-green scale) and Baseline (blue) Maximum Inundation Area

10 Year Alternative 8C

Maximum Water Surface Profile: Amite River above Comite River

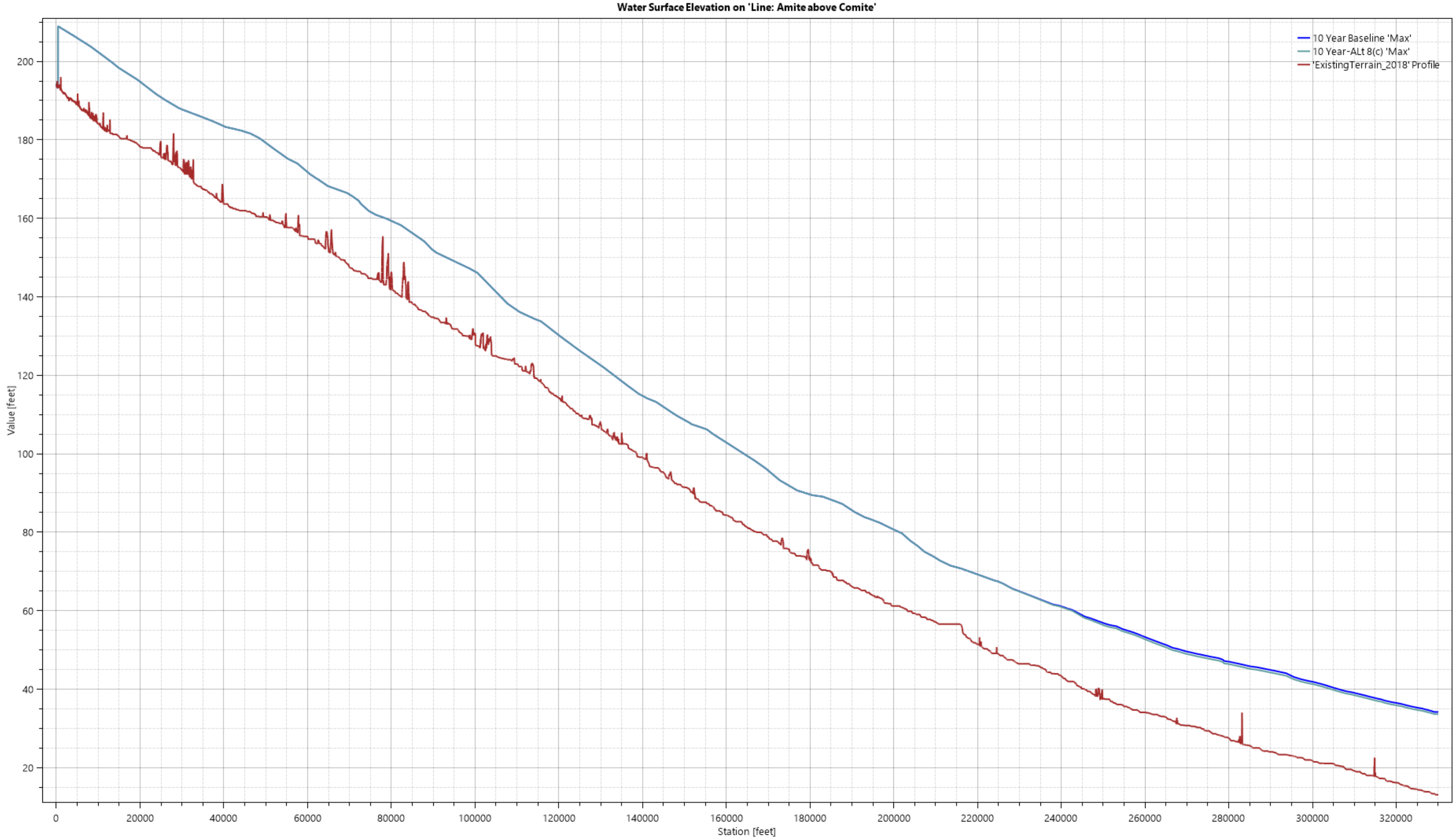


Figure G-35 – Alternative 8C (green) and Baseline (blue) on Amite River above Comite

10 Year Alternative 8C

Maximum Water Surface Profile: Amite River below Comite River

Figure G-36 – Alternative 8C (blue) and Baseline (green) on Amite River below Comite

10 Year Alternative 8C
Maximum Inundation

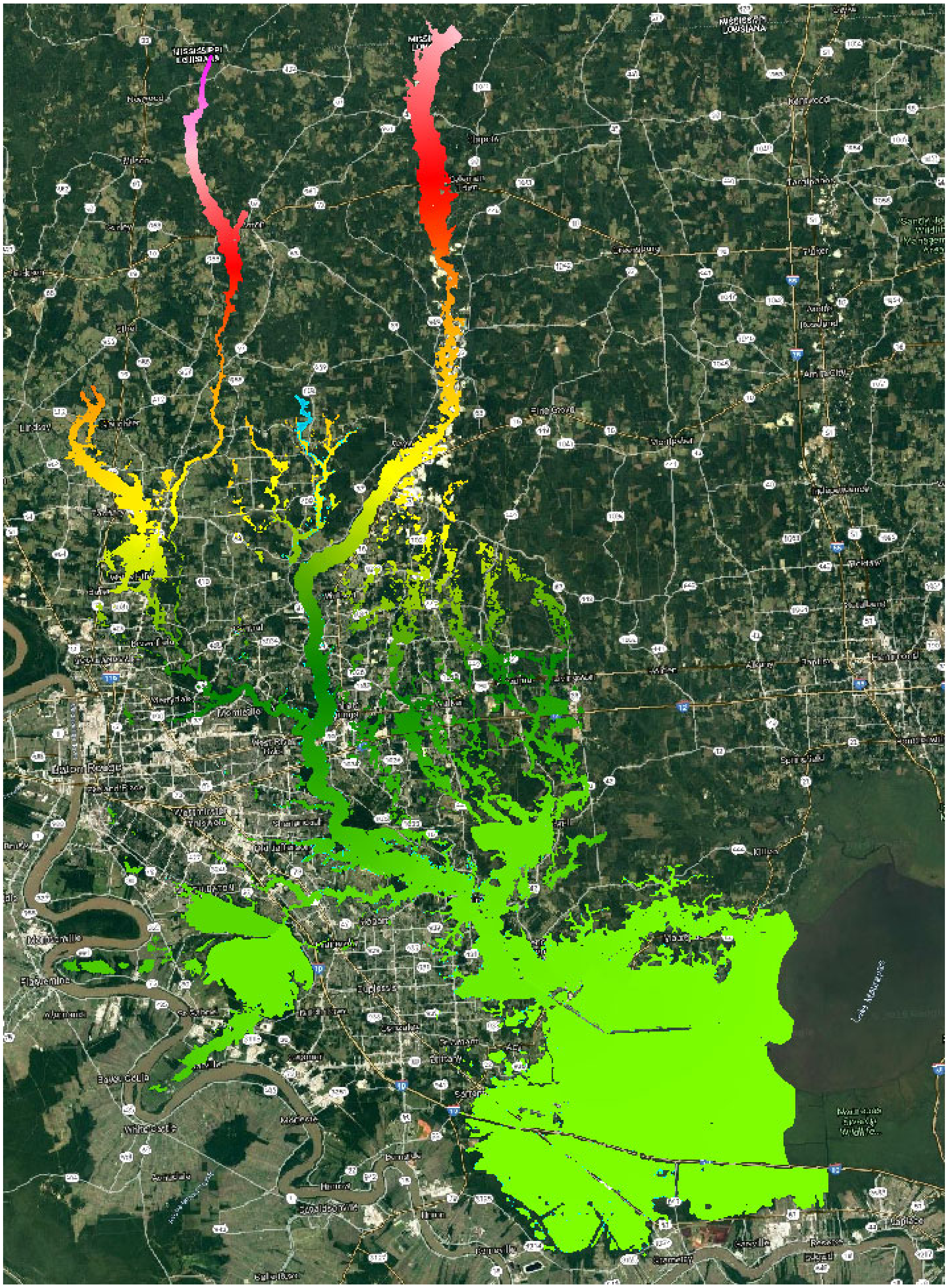


Figure G-37 – Alternative 8C (red-green scale) and Baseline (blue) Maximum Inundation Area

10 Year FWOP
Maximum Water Surface Profile: Amite River above Comite River

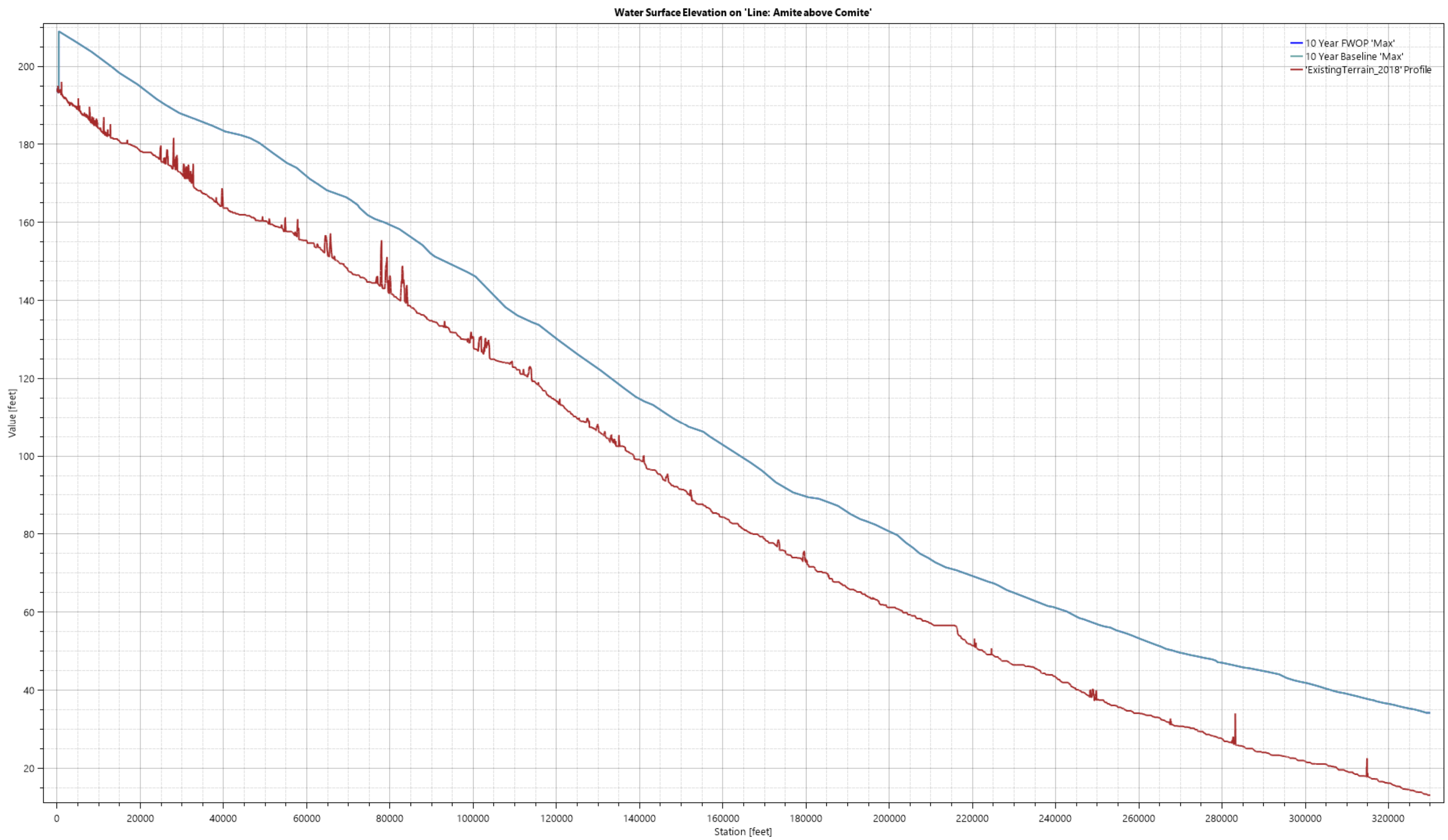


Figure G-38 – FWOP (blue) and Baseline (green) on Amite River above Comite

10 Year FWOP

Maximum Water Surface Profile: Amite River below Comite River

Figure G-39 – FWOP (blue) and Baseline (green) on Amite River below Comite

10 Year FWOP
Maximum Inundation

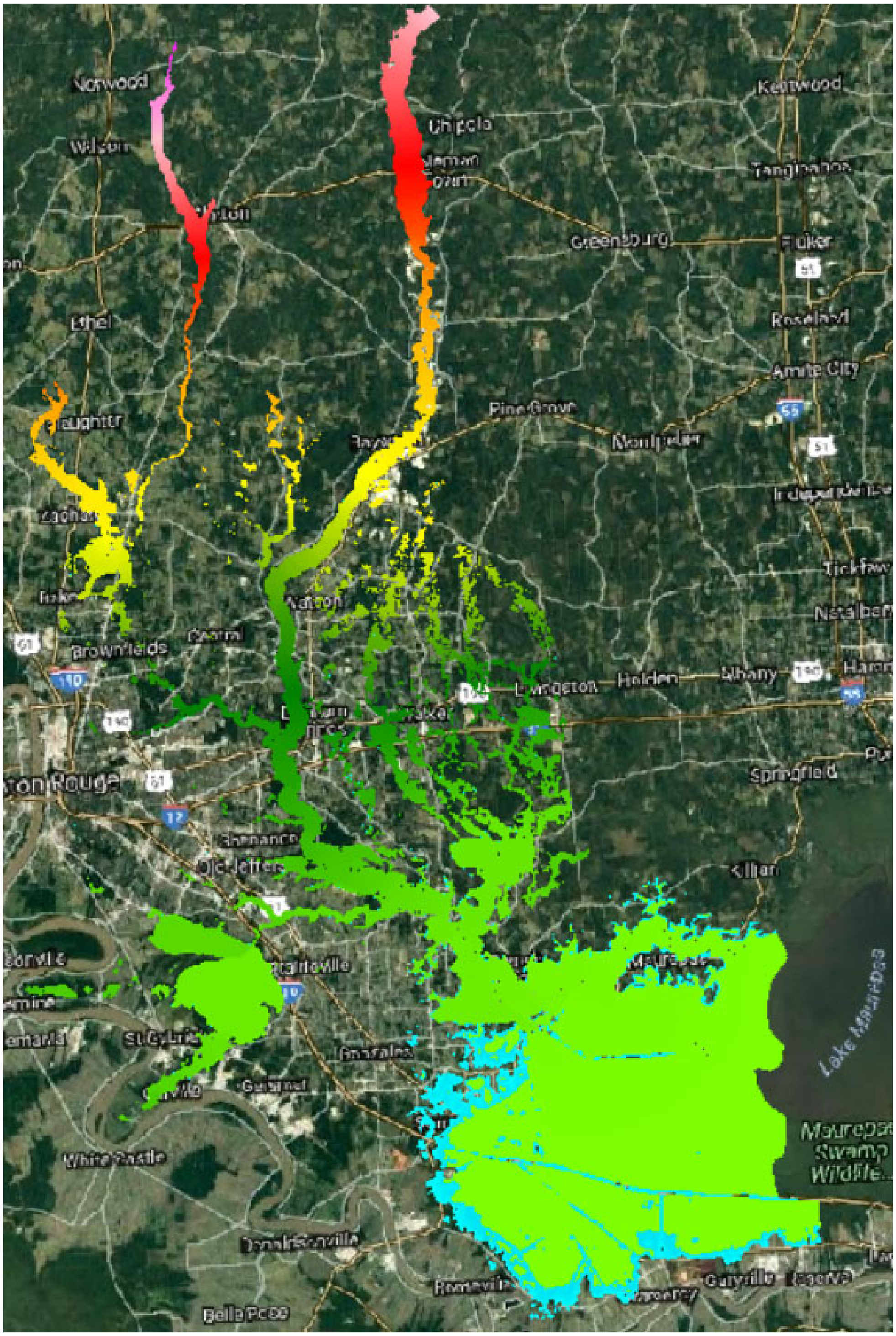


Figure G-40 – FWOP (blue) and Baseline (red-green scale) Maximum Inundation Area

500 Year Darlington Dam

Maximum Water Surface Profile: Amite River above Comite River

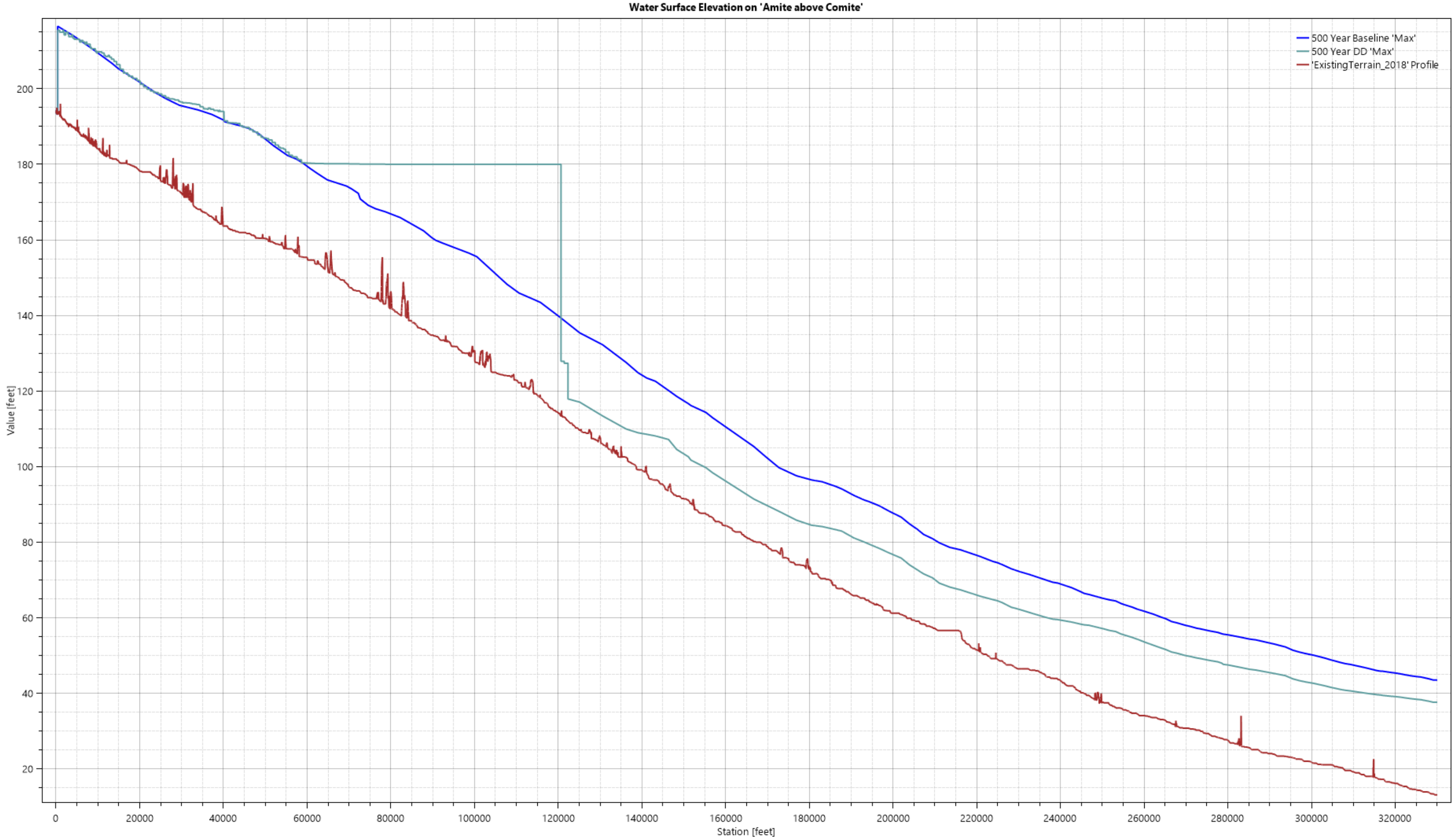


Figure G-41 – Darlington Dam (green) and Baseline (blue) on Amite River above Comite

500 Year Darlington Dam

Maximum Water Surface Profile: Amite River below Comite River

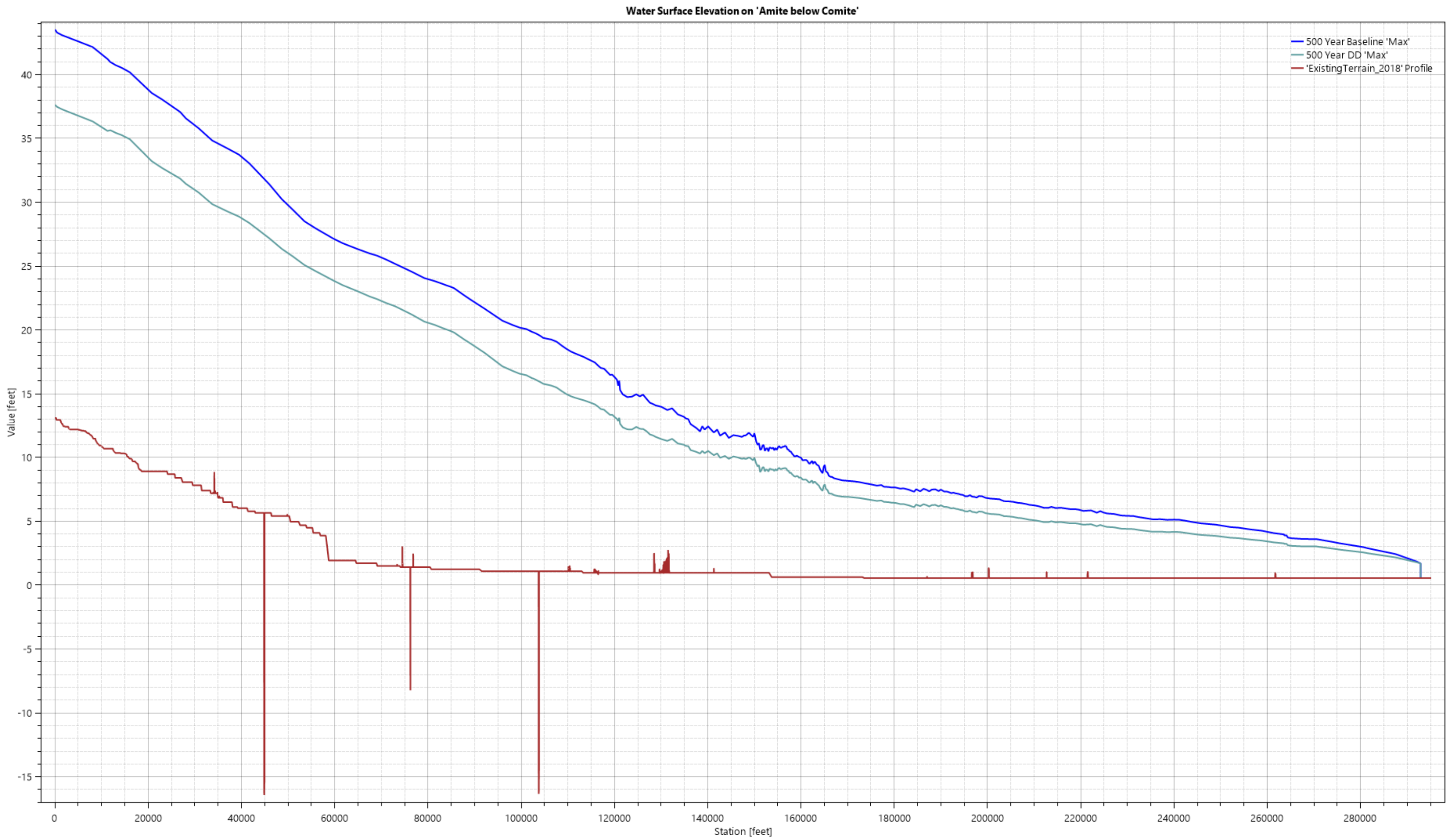


Figure G-42 – Darlington Dam (green) and Baseline (blue) on Amite River below Comite

500 Year Darlington Dam
Maximum Inundation

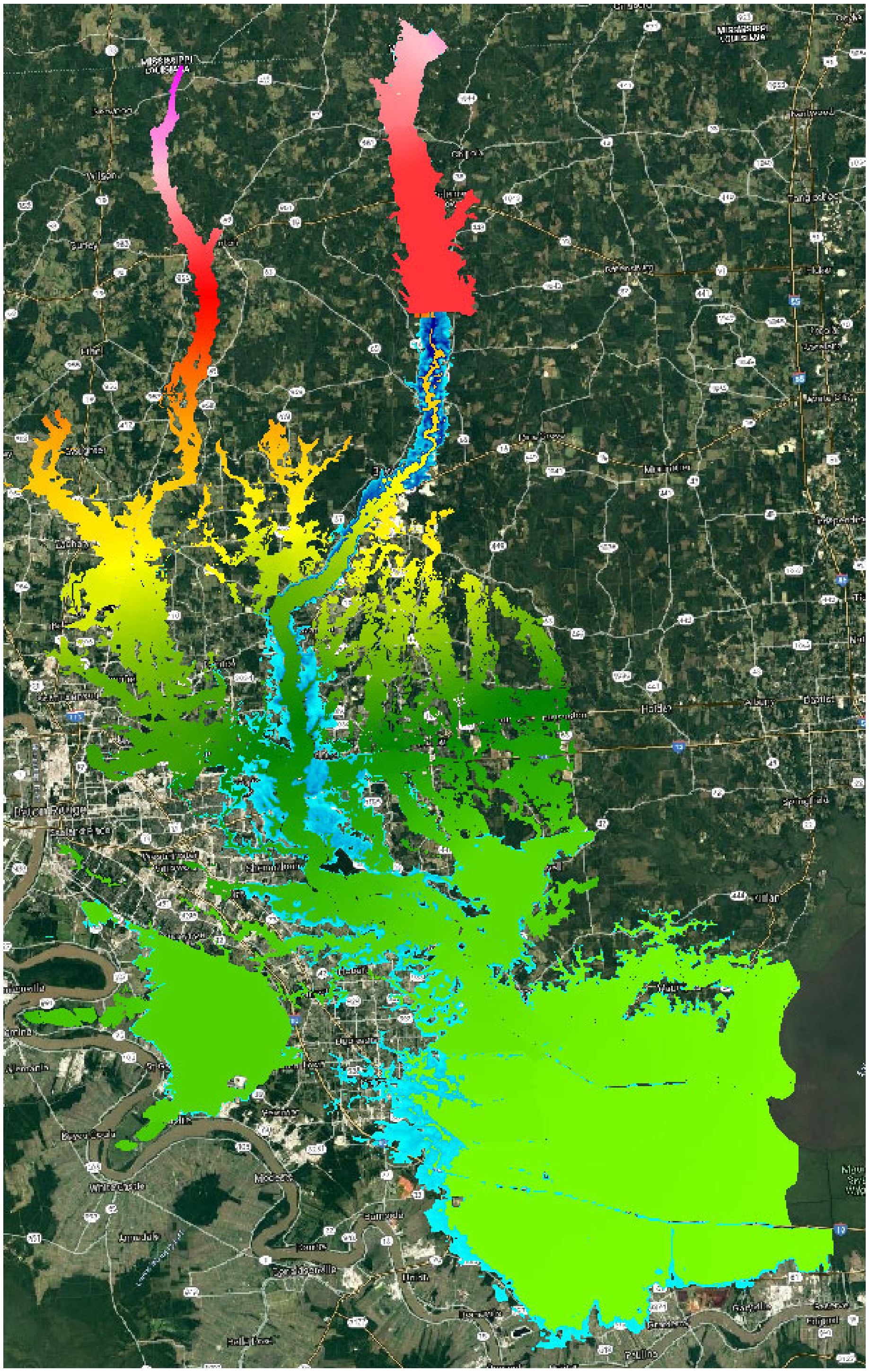


Figure G-43 – Darlington Dam (red-green scale) and Baseline (blue) Maximum Inundation Area

500 Year Alt 8A

Maximum Water Surface Profile: Amite River above Comite River

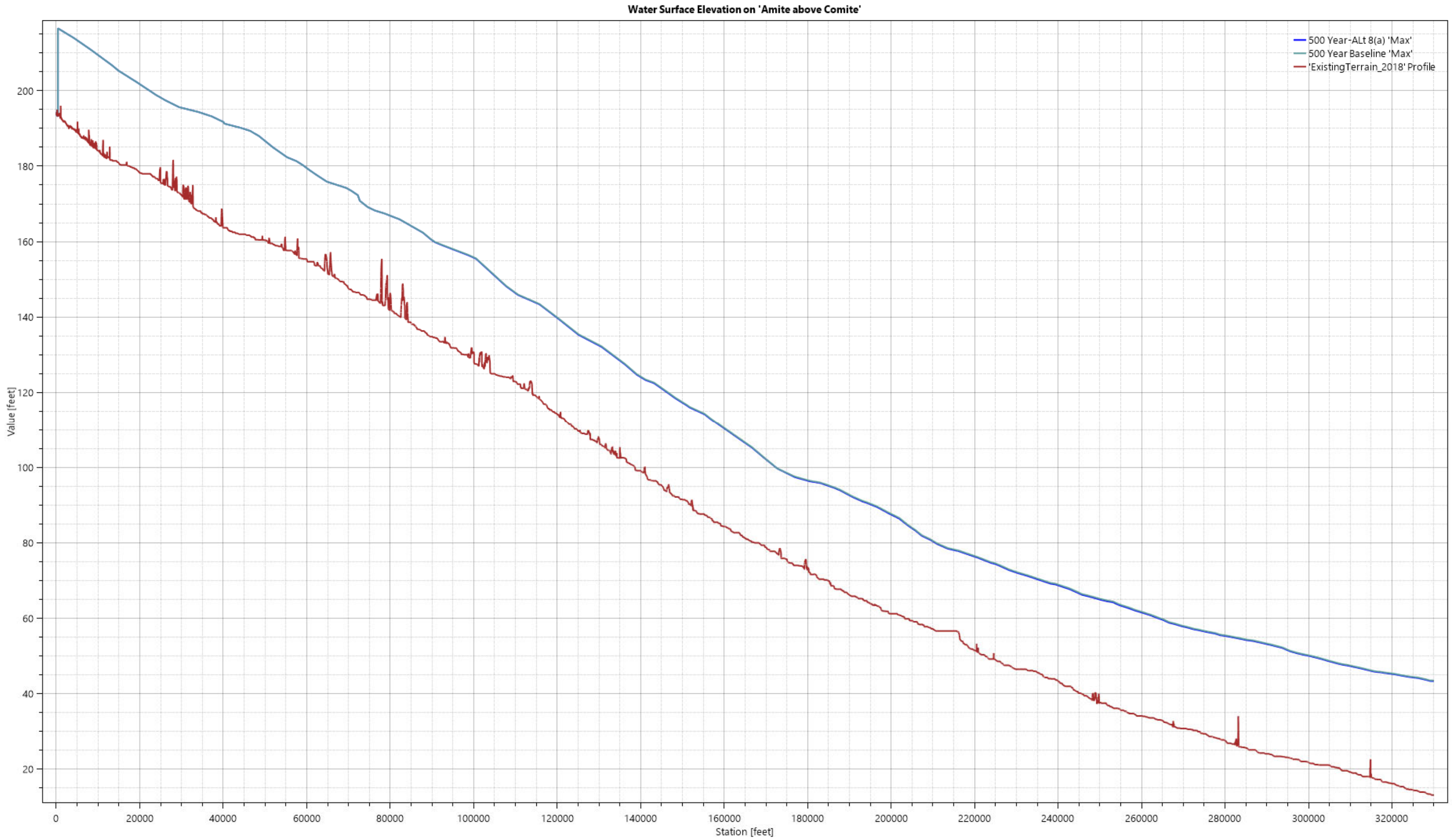


Figure G-44 – Alternative 8A (blue) and Baseline (green) on Amite River above Comite

500 Year Alt 8A

Maximum Water Surface Profile: Amite River below Comite River

Figure G-45 – Alternative 8A (blue) and Baseline (green) on Amite River below Comite

500 Year Alt 8A
Maximum Inundation

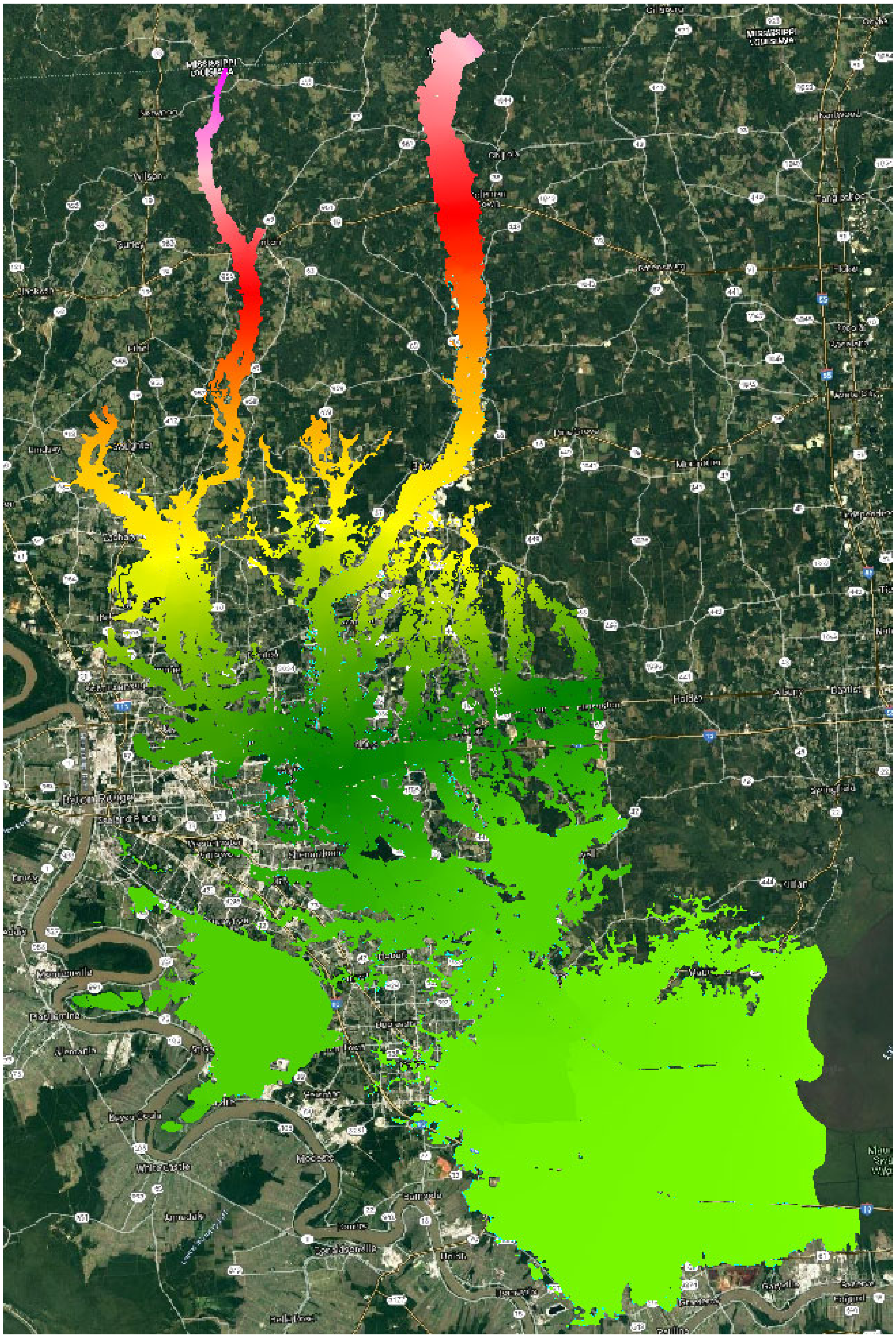


Figure G-46 – Alternative 8A (red-green scale) and Baseline (blue) Maximum Inundation Area

500 Year Alt 8C

Maximum Water Surface Profile: Amite River above Comite River

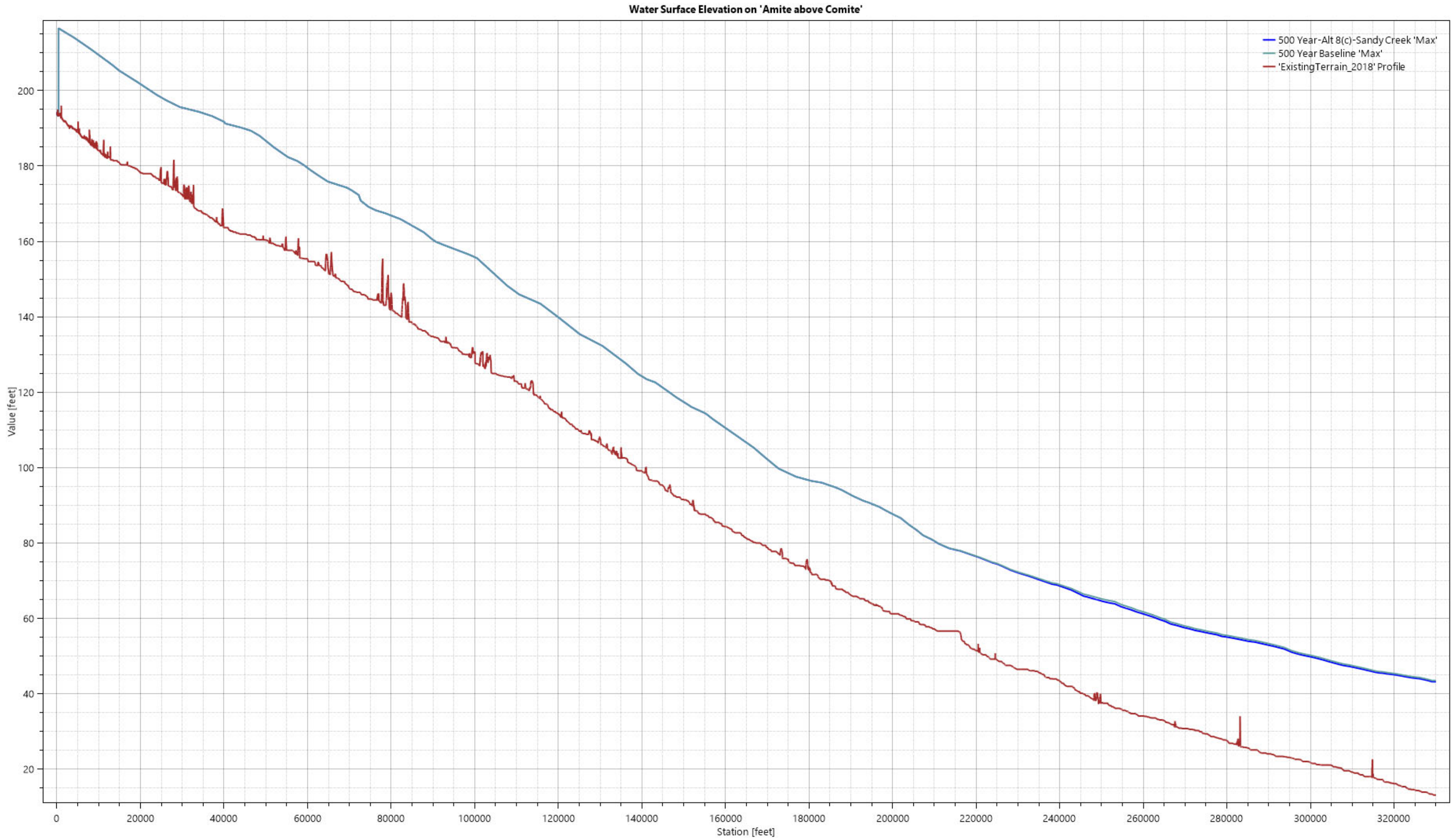


Figure G-47 – Alternative 8C (blue) and Baseline (green) on Amite River above Comite

500 Year Alt 8C

Maximum Water Surface Profile: Amite River below Comite River

Figure G-48 – Alternative 8C (blue) and Baseline (green) on Amite River below Comite

500 Year Alt 8C
Maximum Inundation

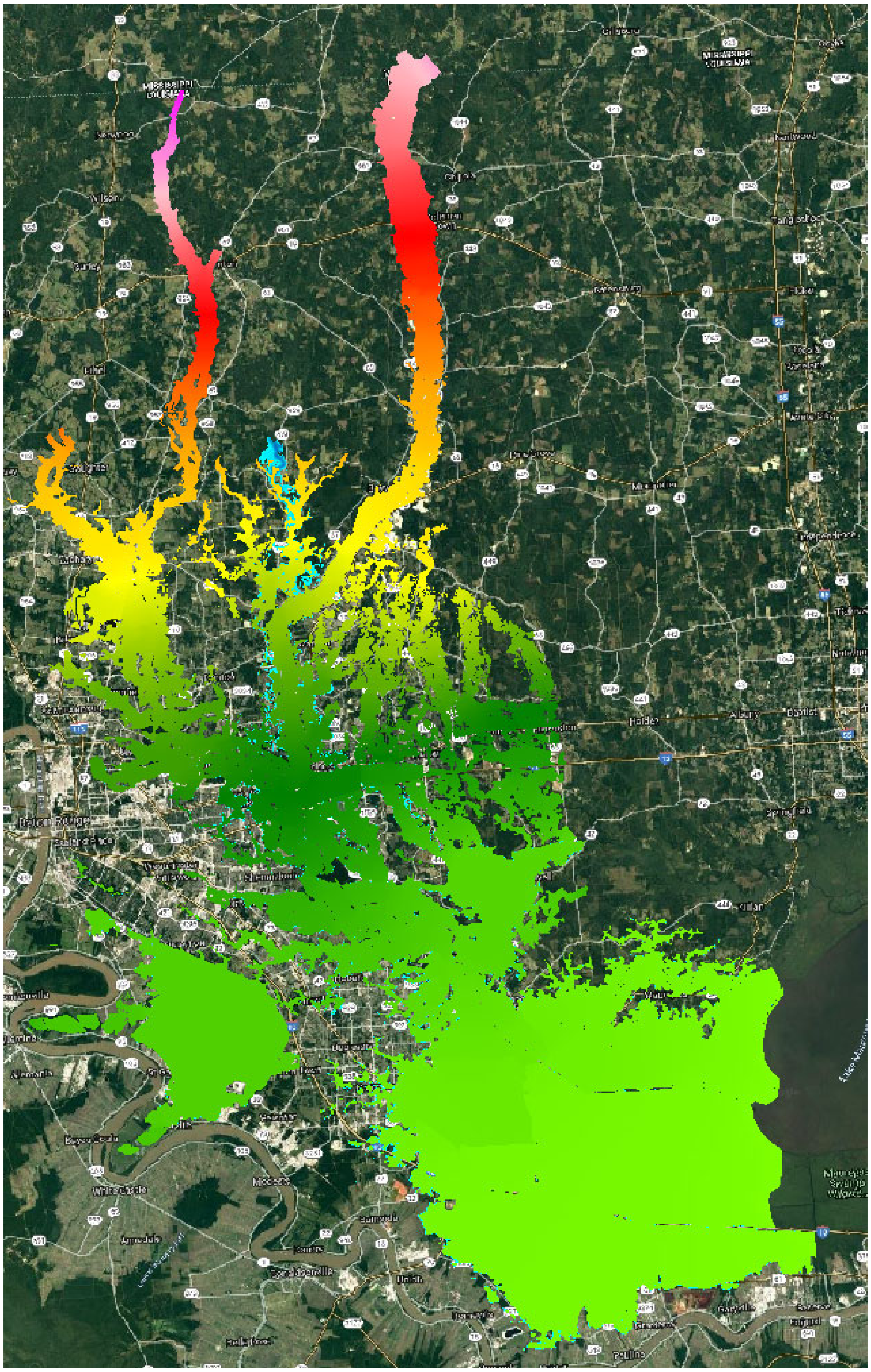


Figure G-49 – Alternative 8C (red-green scale) and Baseline (blue) Maximum Inundation Area

500 Year FWOP

Maximum Water Surface Profile: Amite River above Comite River

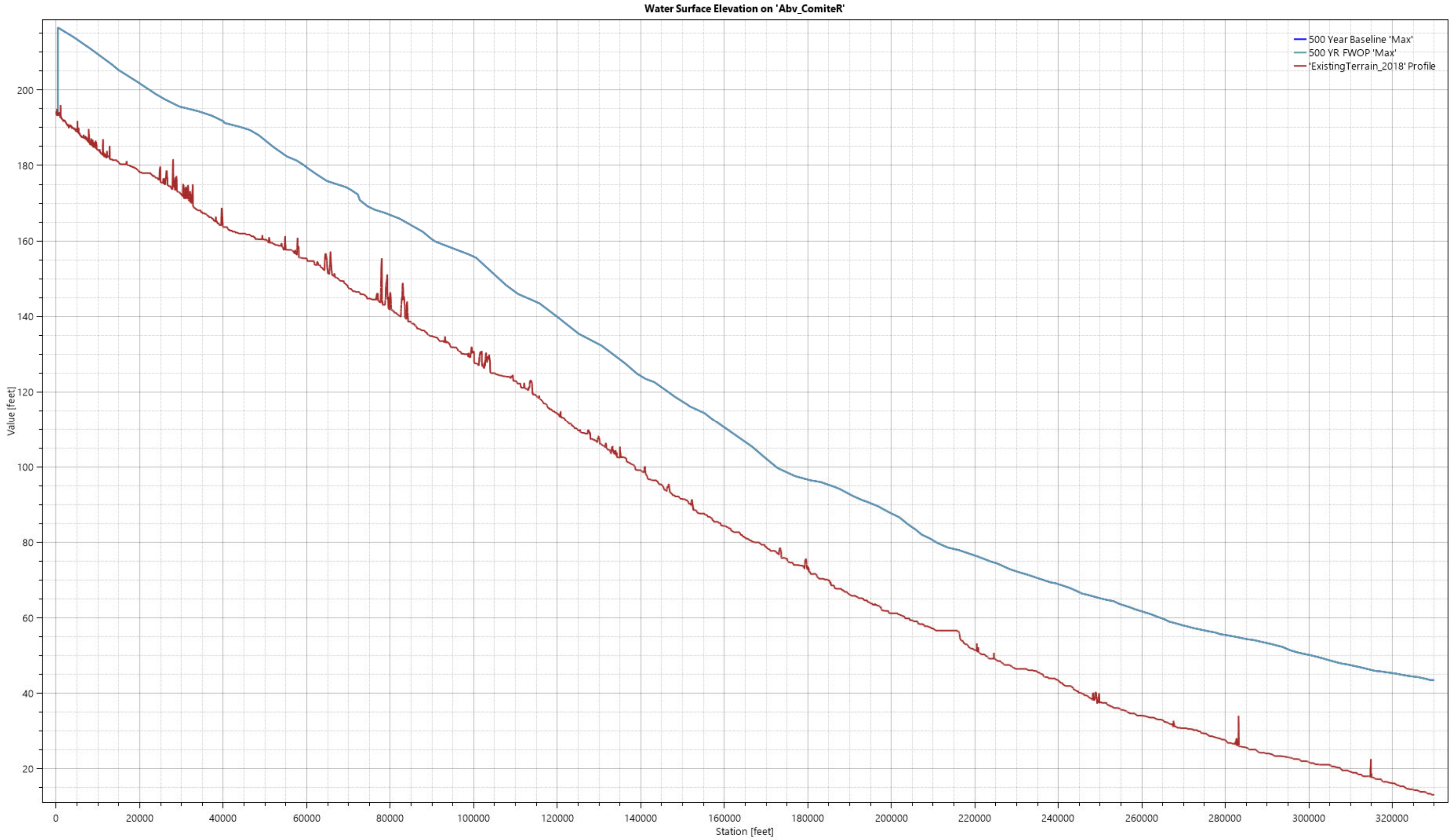


Figure G-50 – FWOP (green) and Baseline (blue) on Amite River above Comite

500 Year FWOP

Maximum Water Surface Profile: Amite River below Comite River

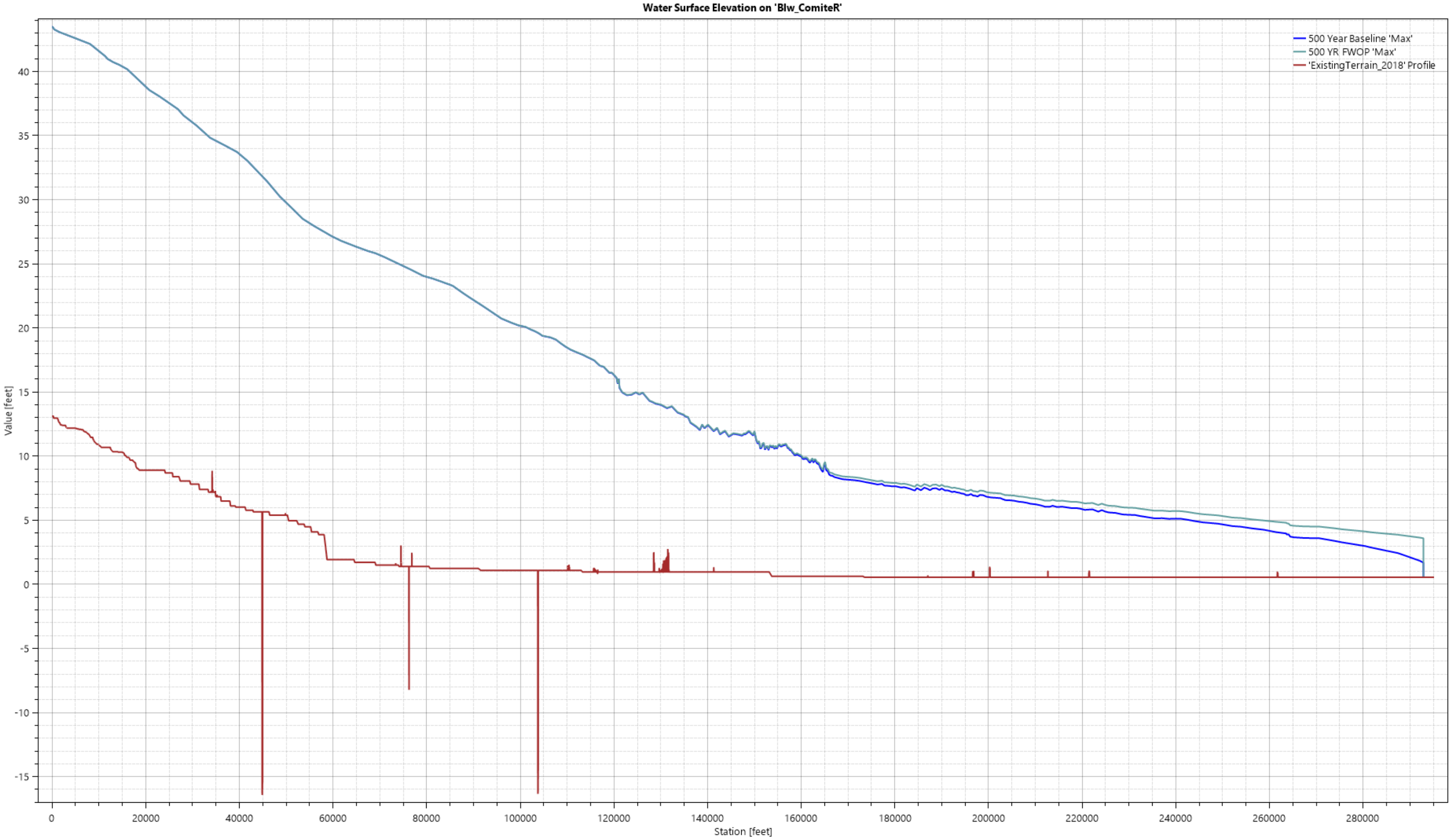


Figure G-51 – FWOP (green) and Baseline (blue) on Amite River below Comite

500 Year FWOP
Maximum Inundation

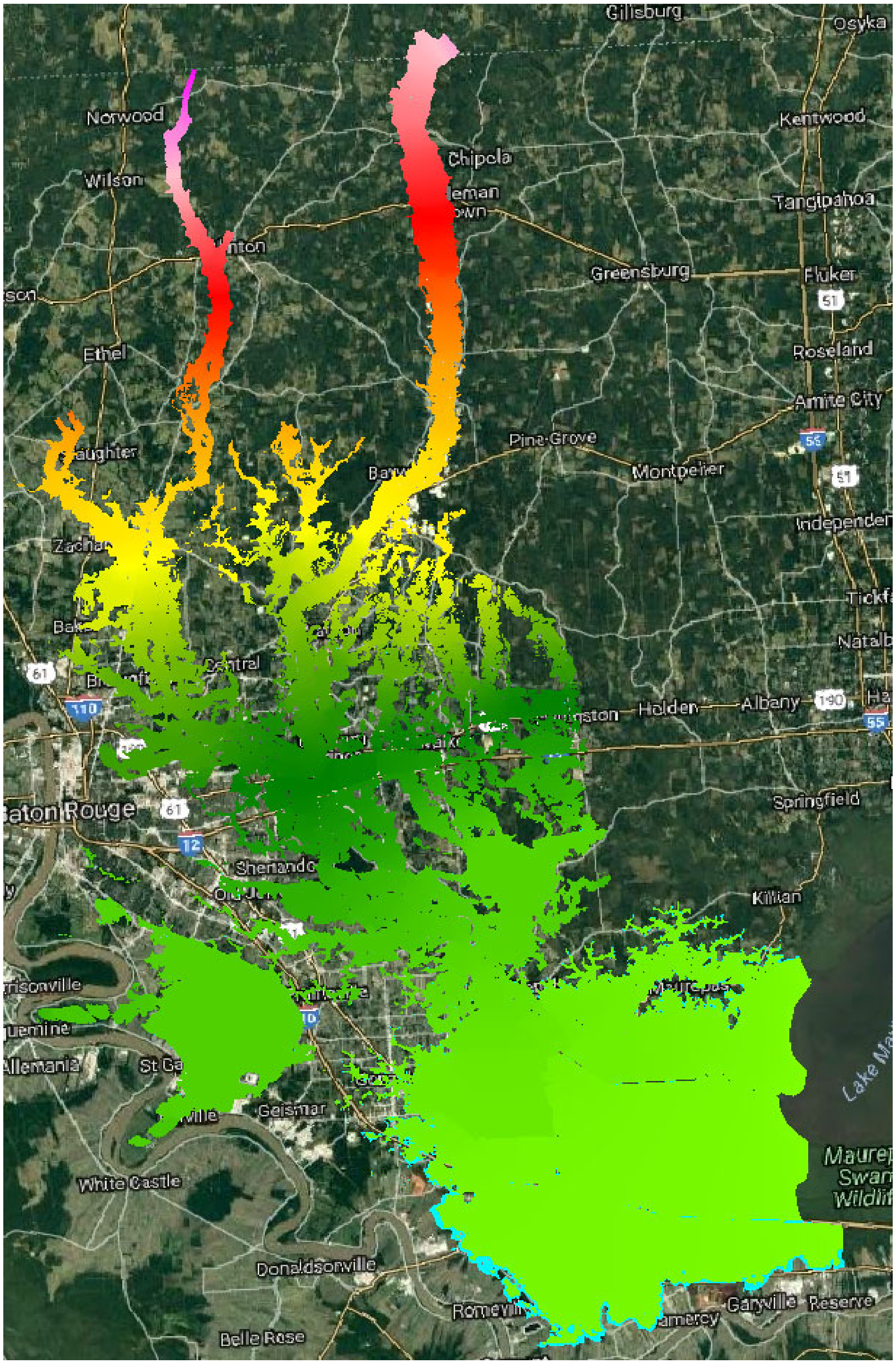


Figure G-52 – FWOP (red-green scale) and Baseline (blue) Maximum Inundation Area