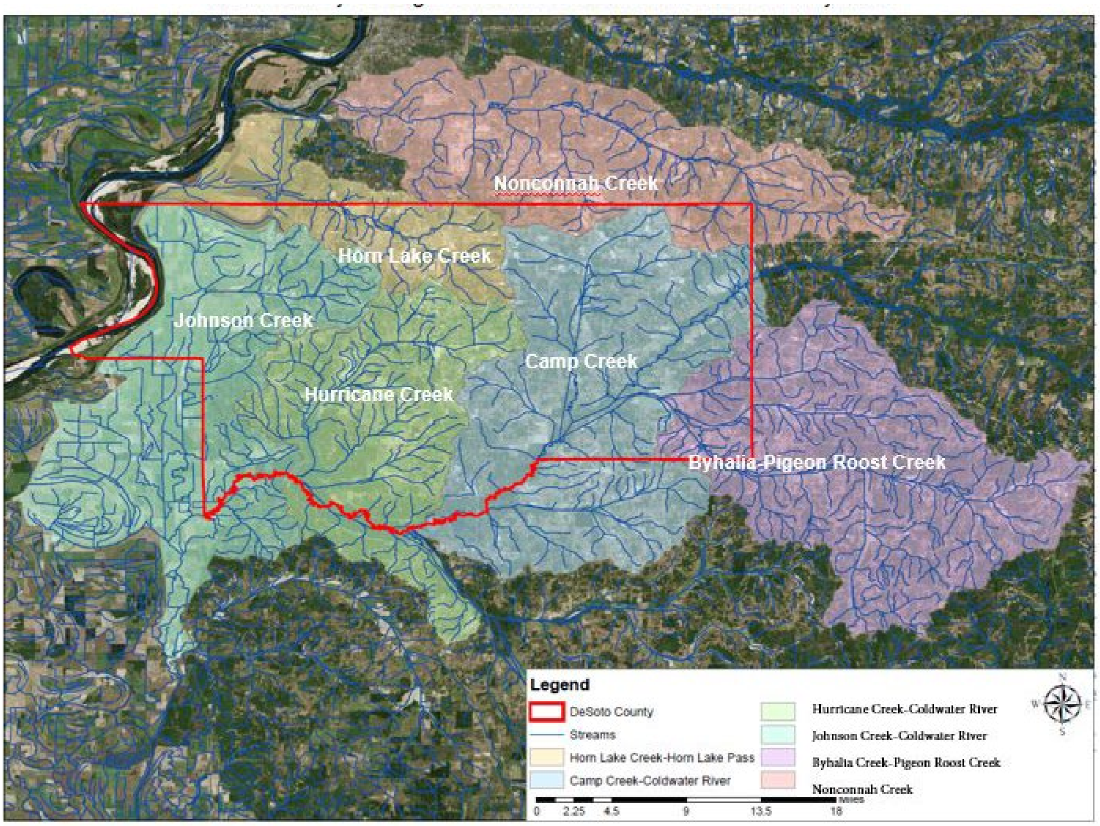
Memphis District
Mississippi River Valley Division

Appendix G: Hydrologic and Hydraulic Appendix



Memphis Metropolitan Stormwater-North Desoto County, Mississippi

December 2022

CONTENTS

LIST OF TABLES

LIST OF FIGURES

Section 1
General Overview

1.1 GENERAL DESCRIPTION OF WORK

The U.S. Army Corps of Engineers (USACE), Memphis District (MVM), Hydraulics and Hydrology (H&H) Branch performed hydrologic and hydraulic modeling for the North Desoto County Feasibility Study. The major basins studied are Horn Lake Creek and Coldwater River. The purpose of this hydrologic and hydraulic modeling effort is to evaluate various design alternatives for Flood Risk Management (FRM) and National Ecosystem Restoration (NER). Hydrologic and Hydraulic models of the Horn Lake Creek Basin were developed by the Memphis District and modified to reflect development in the basin since the previous study in 2005.

Models for the Coldwater River Basin were provided by the USACE Vicksburg District (MVK). Information was also provided by the Sponsors engineering support firm Waggoner Engineering located in Hernando, Mississippi.

Modeling was performed for the 0.99, 0.5, 0.2, 0.1, 0.04, 0.02, 0.01, and 0.002 Annual Exceedance Probability (AEP) rainfall events for existing conditions (year 2019), multiple design alternatives (year 2026), and future without project (FWOP, year 2070). The years 2026 and 2076 represent the base year and the 50-year design life of the project as determined by the economic analysis performed to assess project lifecycle benefits and damages. 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.

1.2 SOFTWARE

1.2.1 HEC-HMS 4.3

USACE Hydraulic Engineering Center’s (HEC) Hydrologic Modeling System (HMS) version 4.3 was the active version at the time of this study and was utilized for the hydrologic modeling.

1.2.2 HEC-RAS 5.0.7

USACE HEC River Analysis System (RAS) version 5.0.7 was the active version at the time of this study and used for the updated hydraulic modeling.

1.3 MODEL DEVELOPMENT

The hydrologic and hydraulic models of the Horn Lake Creek Basin were originally developed for the Desoto County Flood Insurance Study (FIS) dated 1993, updated for the Memphis Metro Study dated 1997 and most recently for the Horn Lake Creek General Re-Evaluation Study dated 2005. Although the 2005 study resulted in an economically justified project, a final report was never completed. To expedite this study process, the previous

2005 hydrologic and hydraulic information were utilized, where possible, and updated to reflect 2018 conditions.

The 2005 HEC-1 model was imported into HEC-HMS. Runoff characteristics in the imported model were based on 2002 land use. The HEC-2 model used for the 2005 study was imported into a HEC-RAS 1D steady flow model. Channel conditions for the model are based on 2002 field surveys. Overbank geometry for the 2005 model was based on LiDAR flown in 2001. A field reconnaissance was conducted and coordinated with the sponsors to ensure any major construction in the streams and floodplain, not included in the previous studies, are captured in this analysis.

Several areas within the Horn Lake Creek basin experience complex flow conditions in the overbank areas and it was determined the 1D/2D unsteady flow capabilities of the HEC-RAS program were needed to simulate and capture specific flooding information. The primary location for HEC-RAS 1D/2D application starts at the IC&G Railroad and extends approximately 1.5 miles to just upstream of Goodman Road. The approach roadways and the intersection of Highway 51 and Goodman Road experience relatively frequent flooding. Documentation and history revealed that after overtopping Highway 51, the flow usually results in flooding of the southwestern quadrant of the intersection. It was determined that a HEC-RAS 1D/2D analysis will capture flows and depths better for this area.

Other study requirements, primarily related to life safety and economic evaluations, were identified that also promoted the use of HEC-RAS 1D/2D modeling. The analysis of detention basins and the respective consequences of a failure prompted the need for a more detailed or robust analysis. Additional analysis was needed to ensure the benefits derived in the 1D analysis were adequately assessed and not overestimated. HEC-HMS lacks the capabilities to evaluate tailwater impacts to storage areas. This fact created some uncertainty in relationship to pond releases and associated benefits. Unsteady flow analysis was used to ensure some of the performance uncertainties related to detention available storage, tailwater conditions, and inflows were assessed using more detail.

As stated above, the Coldwater River Basin information was provided by the Vicksburg District and external Project Delivery Team member (Waggoner Engineering). Most of the RAS model results are shown in the Desoto County Flood Insurance Study (FIS) dated 2006. Information was readily available for the 0.1, 0.02 0.01 and 0.002 AEP events (10-year, 50-year, 100-year, and 500-year return periods). Additional analysis and modeling was conducted in this study to develop information related to the 0.99, 0.50, 0.20, and 0.04 AEP (1-year, 2-year, 5-year, and 25-year return periods) intermediate flood events. Pertinent studies and reports are shown in Table 1.

Table 1. Prior Reports and Studies

Project Year	Study/Report/Environmental Document Title	Document Type
1981	Memphis Metropolitan Area Urban Study, (led to next GDM report)	Urban Study
1986	Horn Lake Creek and Tributaries, Phase I General Design Memorandum (GDM)	General Design Memorandum (GDM)
1988	The Horn Lake Creek and Tributaries Including Cow Pen Creek, General Design Memorandum Re-evaluation	General Design Memorandum Re-evaluation
1999	The Memphis Metro Area, Tennessee, and Mississippi Reconnaissance Report	Reconnaissance Report
2005	Horn Lake Creek and Tributaries Tennessee and Mississippi, General Reevaluation Report	General Reevaluation Report
2018	*Flood Insurance Study Desoto County, Mississippi	Flood Insurance Study

*Original Flood Insurance Study was conducted in 1993.

Section 2
HYDROLOGY

2.1 BASIN DESCRIPTION

The study area lies in the Horn Lake Creek-Nonconnah and Coldwater River Basins in DeSoto County, Mississippi. This includes Horn Lake Creek and tributaries, Nonconnah Creek, Camp Creek and Tributaries, Hurricane Creek, Johnson Creek, and numerous tributaries of the Coldwater River watershed in northern DeSoto County, Mississippi.

The Horn Lake Creek drainage basin is in the north central part of DeSoto County, Mississippi, and the southwest part of Shelby County, Tennessee. Horn Lake Creek, with a total drainage area of 54 square miles at the lower study limits, is a tributary of the Mississippi River. Horn Lake Creek and its tributaries serve as the primary drainage outlets for the cities of Horn Lake and Southaven, Mississippi. Tributaries in the basin include Rocky Creek, Cow Pen Creek, Lateral D, and Lateral E. The slope of Horn Lake Creek above Interstate Highway 55 is approximately 1.8 feet per stream mile. This slope steepens to approximately 5.9 feet per stream mile downstream between Interstate 55 and the Illinois Central Gulf Railroad.

Coldwater River Watershed encompasses 612.5 square miles and lays within portions of DeSoto, Marshall, and Tate counties. Tributaries in the watershed include Beartail Creek, Beartail Tributary, Buttermilk Creek, Byhalia Creek, Camp Creek, Chew Creek, Cuffawa Creek, Lick Creek, Little Beartail Creek, Nolehoe Creek, Nunnally Creek, Pigeon Roost Creek, and Red Banks Creek. The slopes within the watershed vary due in part to approximately 40 in-stream grade control structures installed by Vicksburg District as part of the “Mississippi Delta Headwaters Project.”, previously referred to as the “Demonstration Erosion Control” Project (DEC).

The primary streams identified with flood risk were Upper Coldwater River, Licks Creek, Nolehoe Creek, and Camp Creek. Camp Creek, Nolehoe Creek and Lick Creek are major tributaries in the town of Olive Branch. Camp Creek is 63.6 square miles and has an approximate basin slope of 8.4 feet per mile. The drainage area of Nolehoe Creek is 9.9 square miles and the drainage area of Lick Creek is approximately 10.0 square miles. Basins slopes are 15.6 feet per mile and 18.1 feet per mile for Lick Creek and Nolehoe Creek, respectively. Three study tributaries located in Olive Branch drain into Nonconnah, Creek located in Shelby County, TN.

Three tributaries within the Coldwater Basin provide drainage for the City of Hernando. Hurricane Creek drains the northwestern portion of the City. Short Fork Creek drains the eastern side of the community generally east of Interstate 55. Mussacuna Creek system drains the southwest portion of Hernando.

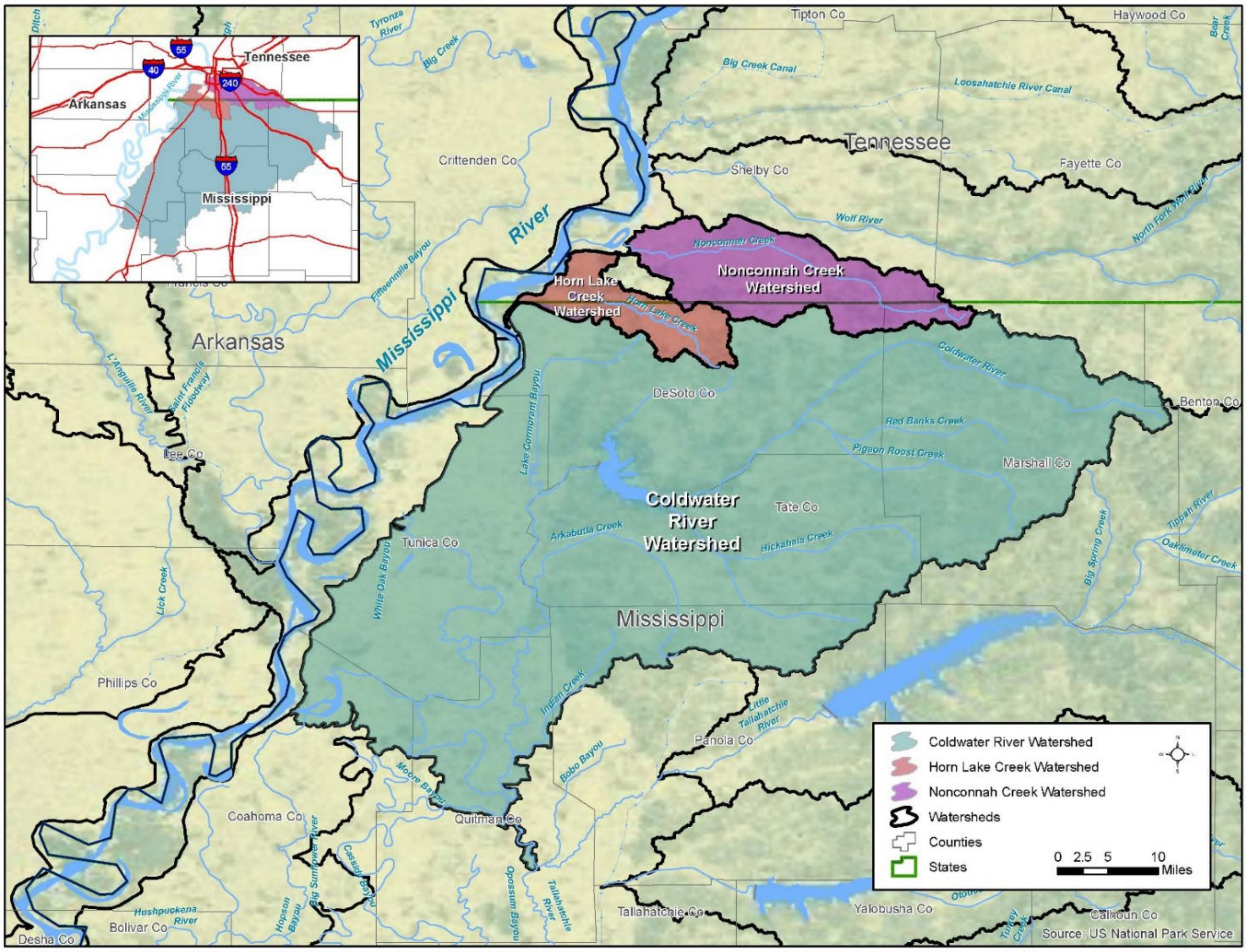


Figure 1. Horn Lake Creek, Nonconnah Creek and Coldwater River Basins

2.2 PRECIPITATION

Eight precipitation events were evaluated for the 0.99, 0.5, 0.2, 0.1, 0.04, 0.02, 0.01, and 0.002 Annual Exceedance Probabilities (AEP) or 1.01- year, 2-year, 5-year, 10-year, 25-year, 50-year, 100-year, and 500-year average recurrence intervals, respectively. The storm duration was for a 24-hour time period. 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. Table 2 shows annual series frequency estimates of precipitation intensity for Southaven, Mississippi from NOAA Atlas 14.

Table 2. Annual Precipitation Frequency Estimates from NOAA Atlas 14-Desoto County, MS

AMS-based point precipitation frequency estimates with 90% confidence intervals (in inches/hour)[1]									
Duration	Annual exceedance probability (1/years)								
	1/2	1/5	1/10	1/25	1/50	1/100	1/200	1/500	1/1000
5-min	**6.30** (5.34-7.48)	**7.75** (6.55-9.23)	**8.83** (7.43-10.5)	**10.2** (8.33-12.4)	**11.3** (9.00-13.9)	**12.3** (9.53-15.4)	**13.2** (9.94-17.0)	**14.5** (10.5-19.1)	**15.5** (11.0-20.6)
10-min	**4.61** (3.91-5.48)	**5.68** (4.80-6.76)	**6.47** (5.44-7.72)	**7.49** (6.10-9.11)	**8.24** (6.59-10.2)	**8.98** (6.97-11.3)	**9.70** (7.27-12.5)	**10.6** (7.71-14.0)	**11.3** (8.04-15.1)
15-min	**3.75** (3.18-4.45)	**4.62** (3.90-5.50)	**5.26** (4.42-6.28)	**6.09** (4.96-7.40)	**6.70** (5.36-8.25)	**7.30** (5.67-9.16)	**7.89** (5.91-10.1)	**8.65** (6.27-11.3)	**9.20** (6.54-12.3)
30-min	**2.49** (2.11-2.96)	**3.08** (2.61-3.67)	**3.53** (2.97-4.21)	**4.10** (3.33-4.98)	**4.52** (3.61-5.56)	**4.93** (3.83-6.19)	**5.33** (4.00-6.85)	**5.85** (4.24-7.68)	**6.24** (4.43-8.32)
60-min	**1.58** (1.34-1.88)	**1.94** (1.64-2.31)	**2.21** (1.86-2.64)	**2.56** (2.09-3.13)	**2.83** (2.27-3.50)	**3.10** (2.41-3.90)	**3.37** (2.53-4.33)	**3.72** (2.70-4.89)	**3.99** (2.83-5.32)
2-hr	**0.958** (0.818-1.13)	**1.17** (0.994-1.38)	**1.33** (1.12-1.57)	**1.54** (1.27-1.87)	**1.70** (1.37-2.09)	**1.87** (1.46-2.34)	**2.04** (1.54-2.60)	**2.26** (1.65-2.95)	**2.43** (1.74-3.22)
3-hr	**0.718** (0.616-0.841)	**0.868** (0.743-1.02)	**0.986** (0.839-1.16)	**1.15** (0.949-1.39)	**1.27** (1.03-1.55)	**1.40** (1.10-1.74)	**1.53** (1.16-1.95)	**1.71** (1.25-2.23)	**1.85** (1.32-2.43)
6-hr	**0.436** (0.377-0.507)	**0.533** (0.459-0.620)	**0.609** (0.522-0.712)	**0.714** (0.596-0.857)	**0.796** (0.651-0.968)	**0.882** (0.700-1.09)	**0.970** (0.742-1.23)	**1.09** (0.807-1.41)	**1.18** (0.855-1.55)
12-hr	**0.260** (0.226-0.299)	**0.326** (0.283-0.377)	**0.378** (0.326-0.438)	**0.449** (0.377-0.534)	**0.504** (0.415-0.607)	**0.561** (0.448-0.689)	**0.619** (0.477-0.778)	**0.698** (0.520-0.897)	**0.759** (0.552-0.987)
24-hr	**0.154** (0.135-0.176)	**0.196** (0.171-0.224)	**0.228** (0.199-0.262)	**0.273** (0.231-0.322)	**0.307** (0.255-0.367)	**0.343** (0.275-0.418)	**0.379** (0.294-0.473)	**0.428** (0.321-0.546)	**0.466** (0.341-0.602)
2-day	**0.090** (0.080-0.102)	**0.113** (0.100-0.128)	**0.131** (0.115-0.149)	**0.156** (0.133-0.183)	**0.175** (0.147-0.208)	**0.196** (0.159-0.237)	**0.216** (0.169-0.268)	**0.245** (0.185-0.310)	**0.267** (0.197-0.343)
3-day	**0.066** (0.058-0.074)	**0.082** (0.073-0.093)	**0.095** (0.083-0.107)	**0.112** (0.096-0.131)	**0.126** (0.106-0.149)	**0.141** (0.115-0.170)	**0.156** (0.122-0.192)	**0.176** (0.133-0.222)	**0.192** (0.142-0.245)
4-day	**0.053** (0.047-0.060)	**0.066** (0.058-0.074)	**0.076** (0.067-0.085)	**0.089** (0.077-0.104)	**0.100** (0.084-0.118)	**0.111** (0.091-0.134)	**0.123** (0.097-0.151)	**0.139** (0.105-0.174)	**0.151** (0.112-0.192)
7-day	**0.036** (0.032-0.040)	**0.044** (0.039-0.049)	**0.050** (0.044-0.056)	**0.058** (0.050-0.067)	**0.064** (0.054-0.075)	**0.071** (0.058-0.084)	**0.077** (0.061-0.094)	**0.087** (0.066-0.108)	**0.094** (0.070-0.119)
10-day	**0.029** (0.026-0.032)	**0.035** (0.031-0.038)	**0.039** (0.035-0.043)	**0.045** (0.039-0.052)	**0.050** (0.042-0.058)	**0.054** (0.045-0.064)	**0.059** (0.047-0.072)	**0.066** (0.050-0.082)	**0.071** (0.053-0.089)
20-day	**0.019** (0.017-0.021)	**0.023** (0.021-0.025)	**0.026** (0.023-0.029)	**0.030** (0.026-0.034)	**0.033** (0.028-0.037)	**0.035** (0.029-0.042)	**0.038** (0.031-0.046)	**0.042** (0.033-0.052)	**0.045** (0.034-0.056)
30-day	**0.016** (0.014-0.017)	**0.019** (0.017-0.020)	**0.021** (0.019-0.023)	**0.024** (0.021-0.027)	**0.026** (0.022-0.030)	**0.028** (0.023-0.033)	**0.030** (0.024-0.036)	**0.033** (0.026-0.041)	**0.035** (0.027-0.044)
45-day	**0.013** (0.012-0.014)	**0.015** (0.014-0.017)	**0.017** (0.016-0.019)	**0.019** (0.017-0.022)	**0.021** (0.018-0.024)	**0.023** (0.019-0.026)	**0.024** (0.019-0.029)	**0.026** (0.020-0.032)	**0.027** (0.021-0.034)
60-day	**0.011** (0.010-0.012)	**0.013** (0.012-0.014)	**0.015** (0.014-0.016)	**0.017** (0.015-0.019)	**0.018** (0.016-0.020)	**0.019** (0.016-0.022)	**0.020** (0.016-0.024)	**0.022** (0.017-0.026)	**0.023** (0.017-0.028)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of annual maxima series (AMS).

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 annual exceedance probability) 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.

Section 3

Climate Change Assessment for Desoto County, Mississippi

3.1 INTRODUCTION

In 2016, USACE issued Engineering and Construction Bulletin No. 2016-25 (USACE, 2016) (hereafter, ECB 2016-25), which mandated that climate change be considered for all federally funded projects in planning stages. This guidance was updated with ECB 2018-14, Rev 2 (USACE, 2022). A qualitative analysis of historical climate trends, as well as assessment of future projections was provisioned by ECB 2018-14, Rev 2. An extensive analysis was conducted for study, in accordance with the cited guidance, and presented in Climate Change Appendix. The following paragraphs briefly summarize the data used.

Detailed information presented in the appendix is related to the Horn Lake Creek basin since the primary Flood Risk Management project measures in this study lie within this watershed. It is assumed the Climate Change results and indicators would be representative of conditions in the Coldwater River Basin, if needed. Non-structural measures were investigated in both the Horn Lake Creek and Coldwater Basins and climate change assumptions and results were included in the Appendix.

3.2 LITERATURE REVIEW

As mandated in the guidance, a literature review was performed to summarize climate change literature relevant to the study area and highlight both observed and projected assessments of relevant climate change variables. As this is a flood risk management study, the primary relevant variable is streamflow. This variable is also affected by precipitation and air temperature. Therefore, this review focuses on observed and projected changes in air temperature, precipitation, and hydrology.

3.2.1 Temperature Precipitation

The Institute of Water Resources' (IWR's) Climate Change Literature Review notes that there is a statistically significant increasing trend in the number of one day extreme minimum temperatures in the Lower Mississippi Region. The consensus from the Climate Change Literature Review indicates only mild increases in annual temperature in the region over the past century with significant variability. However, there is consensus that the extreme minimum daily air temperatures are increasing.

Similar warming trends have been noted in the project area. The longest running gage in the area, located at the Memphis International Airport (MEM) has continuous records going back to the 1940s and is located seven miles south of the headwaters of the study area, as shown in Figure 2.

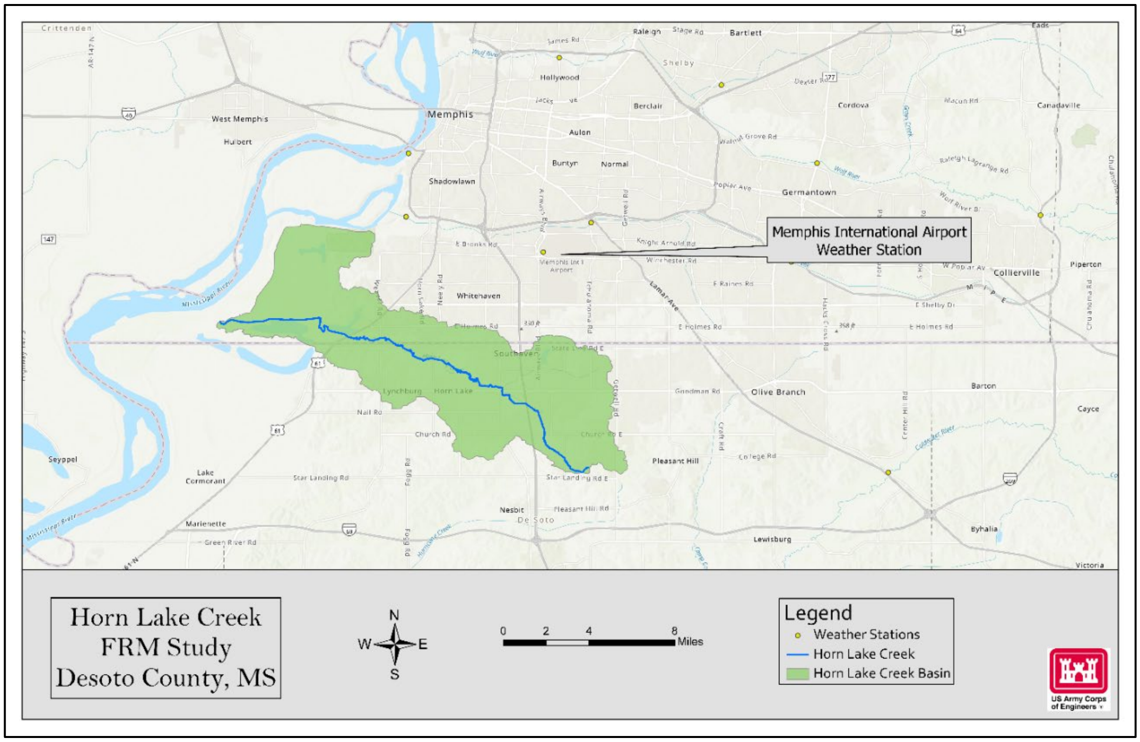


Figure 2. Study area and Memphis International Airport (MEM) Weather Station Statistical Temperature and precipitation Analysis for the Horn Lake Creek Basin

From 1940 to 1970, the average annual temperature at the gage followed no noticeable trend but transitioned to a consistent increase starting in the 1970s. The temperature period of record is shown in Figures 3. Figure 4 highlights the upward trend in temperate during the time period of 1970-2018.

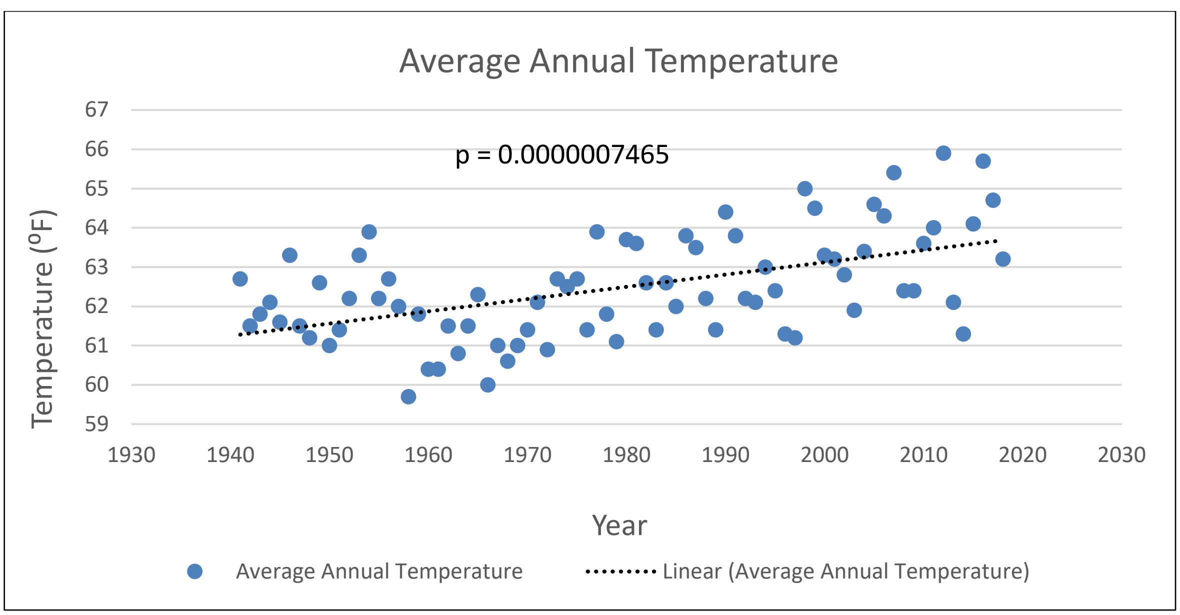


Figure 3. Annual Average Temperature and P-Value from 1940 – 2018 (MEM)

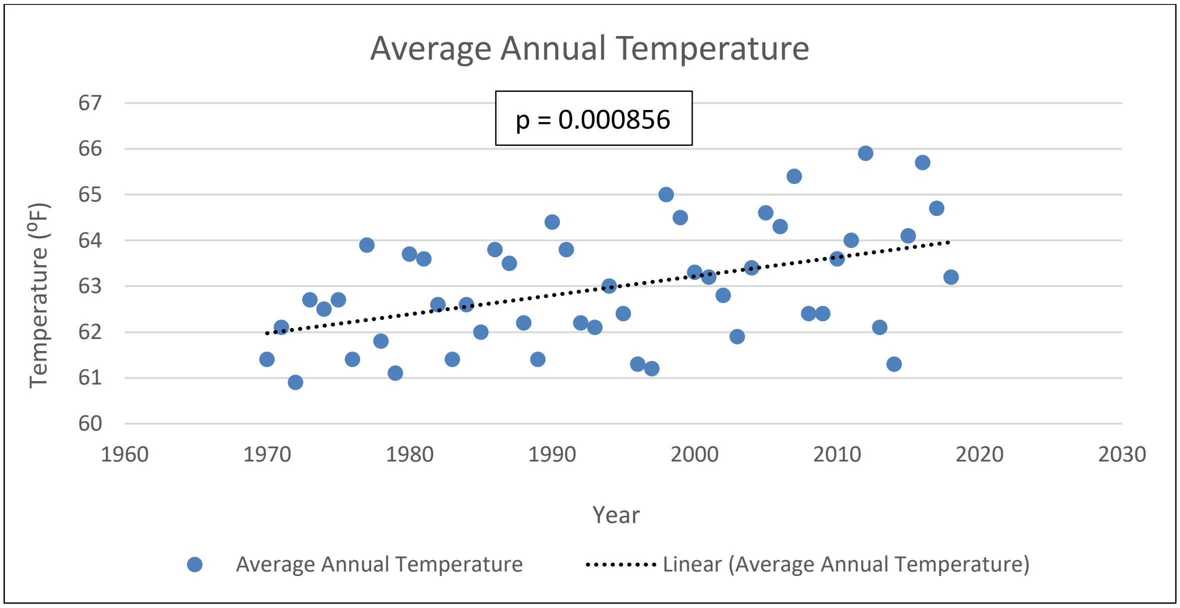


Figure 4. Annual Average Temperature and P-Value from 1970 – 2018 (MEM)

3.2.2 Precipitation

The MEM Airport weather station shows variable annual average precipitation since 1940. The results in Figure 5 show no statistically significant upward trend. Visually, it appears that extremes at either end are becoming more severe since the 1970s. An attempt to analyze the extremes was not undertaken since preliminary results showed no major concerns.

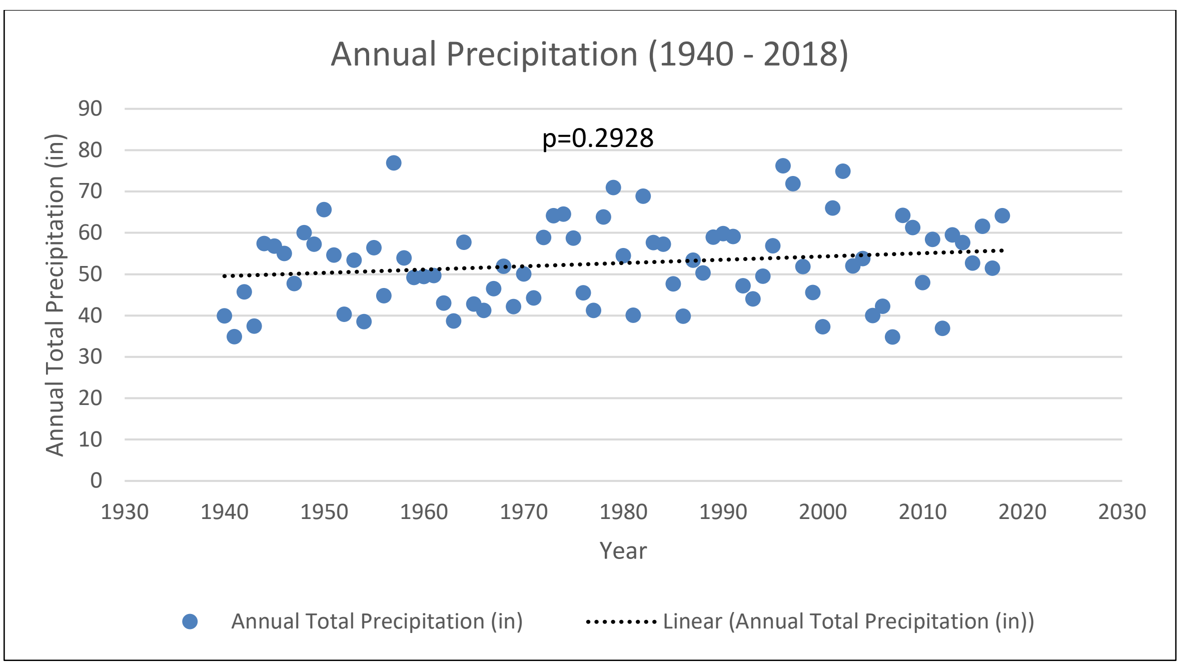


Figure 5. Annual Total Precipitation and P-Value from 1940 – 2018 (MEM)

3.3 HYDROLOGY

3.3.1 Observed Streamflow

Generalized observations of streamflow trends in the Lower Mississippi River Region lack a clear consensus, with some models showing positive trends in some areas and others showing negative trends for areas in the southeast. Generally, most studies in the Lower Mississippi River Region indicated an increasing trend in streamflow.

For the study area, there is no noticeable trend for streamflow in the Horn Lake Creek area. Horn Lake Creek does not have a discharge gage, but a USGS gage is located on the Coldwater River near Olive Branch, MS. The USGS gage 07275900 on the Coldwater River near Olive Branch, MS which has a 21-year period of record was available for select climate change assessments. Figure 6 also shows the gages in the Nonconnah Creek watershed located north of the study area in Tennessee which contained a longer period of record. The gage at Germantown, TN was used for several Climate Change Assessments as required by Corps guidance. As stated above, detailed information is presented in Climate Change Appendix.

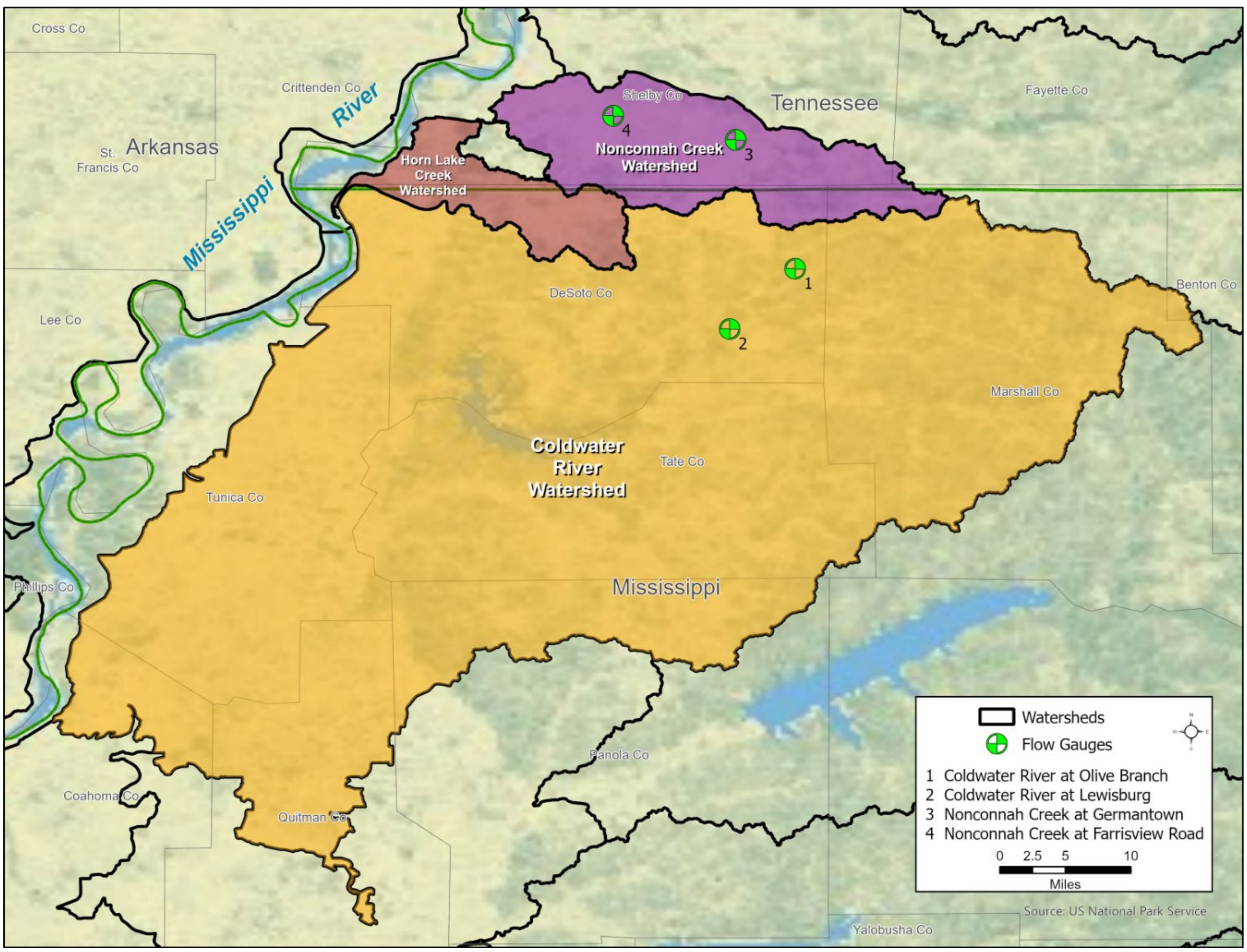


Figure 6. Horn Lake Creek Basin and Gages in Adjacent Basins

Peak flows for the Coldwater River gage in Olive Branch are shown below. The period of record for the gage is 25 years.

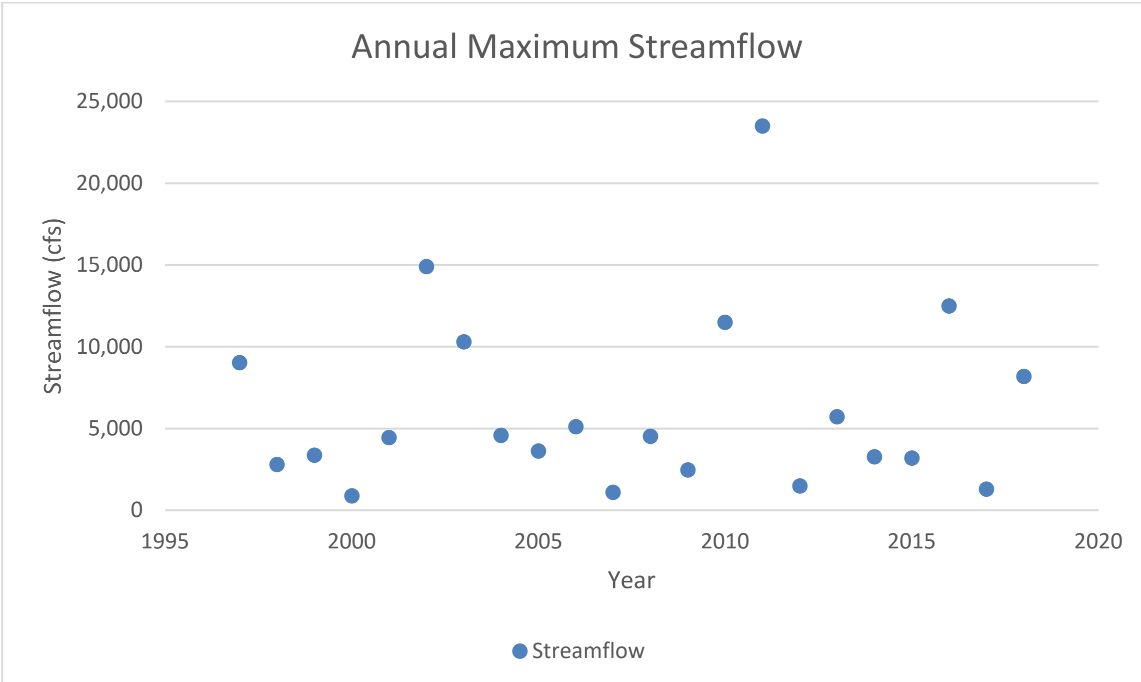


Figure 7. Annual Peak Streamflow at USGS 07275900 Gage Coldwater River near Olive Branch, MS

3.4 NON-STATIONARITY ASSESSMENT

In accordance with ECB 2018-14 rev 2, a stationarity analysis was performed to determine if there are long-term changes in peak streamflow statistics within the Horn Lake Creek basin and its vicinity. Assessing trends in peak streamflow is considered appropriate as one of the primary purposes of this feasibility study is to assess and reduce flooding in the Horn Lake Creek Basin.

The alternatives reviewed for flood risk management measures include a floodwall/levee, channel enlargement, both inline storage and off-channel storage. All measures have the potential to be significantly affected by changes in peak streamflow.

3.4.1 USACE Non-Stationarity Tool

The USACE Nonstationary Tool was used to assess possible trends and change points in peak streamflow in the region. Since the Horn Lake basin does not possess a stream gage, the Nonconnah Creek gage at Germantown, Tennessee (USGS gage 7032200) was used for the analysis. The gage is located on Nonconnah Creek, approximately 8.6 miles northeast of the Horn Lake Creek Watershed boundary. The Nonconnah Creek gage was chosen as its topography and basin size are comparable to Horn Lake Creek.

Additionally, this gage is the only site with similar basin characteristics in the area and at least 30 continuous years of record, which is the minimum recommended years for this tool to detect non-stationarities. For this assessment, the continuous period of water years 1970 – 2014 was analyzed. Figure 8 displays the annual peak streamflow for the gage.

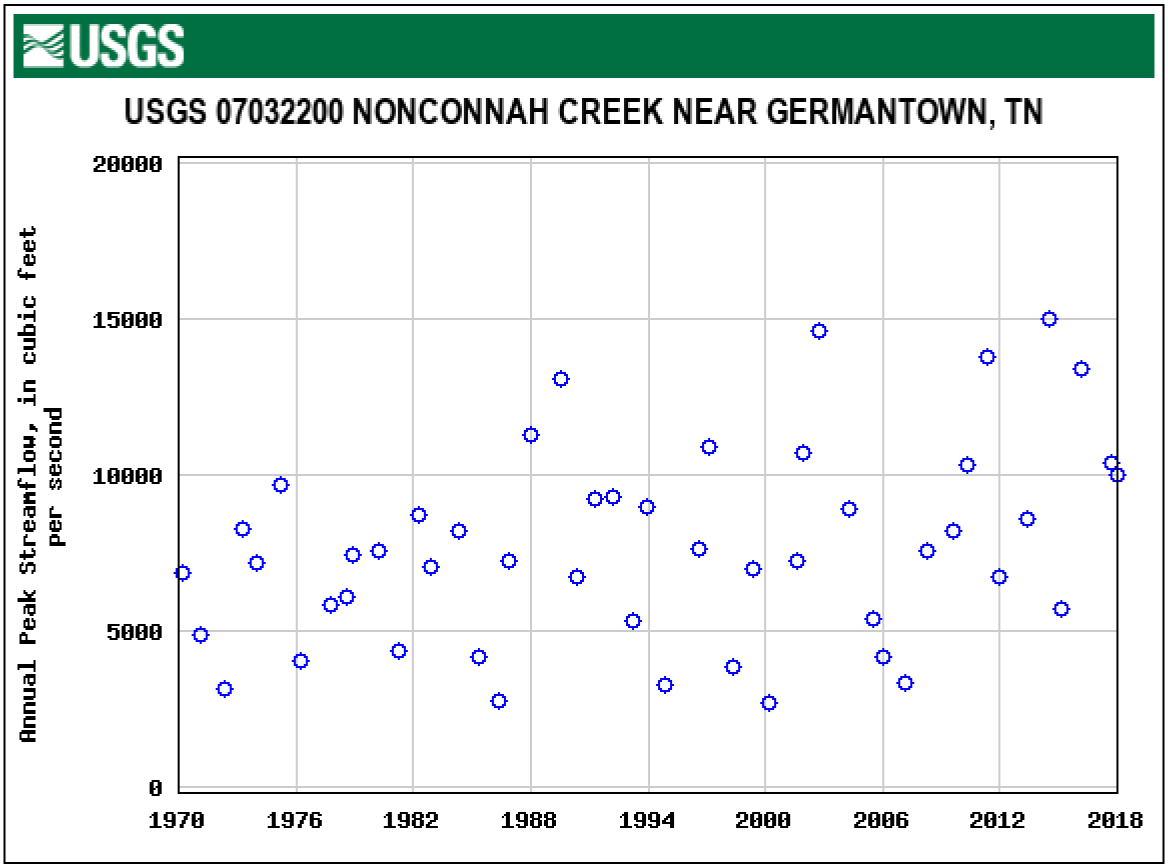


Figure 8. APF at USGS 07032200 Nonconnah Creek near Germantown, TN

3.4.2 Climate Hydrology Assessment

In addition to the stationarity assessment, the USACE Climate Hydrology Assessment Tool (CHAT) was used to assist in the determination of future streamflow conditions. For this assessment, the USGS 07032200 (Nonconnah Creek) was also used. The Nonconnah Creek basin continues to experience development and is projected to continue this growth for the near future. This basin development was a major consideration in quantifying the anticipated impacts from climate change.

3.5 VULNERABILITY ASSESSMENT

To understand potential climate change effects and to increase resilience/decrease vulnerability of flood risk management alternatives to climate change, the relative vulnerability of the basin to such factors was analyzed. In accordance with ECB 2018-14 rev 2, the USACE Watershed Climate Vulnerability Assessment tool was used to identify vulnerabilities to climate change on a HUC-4 watershed scale relative to other HUC-4 basins across the nation. As this study is an assessment of flood risk management alternatives, vulnerability with respect to the Flood Risk Reduction business line is presented in the

Climate Change Appendix H. It should be noted that the Ecosystem Restoration business line was also assessed.

The Lower Mississippi-Hatchie HUC-4 Basin was used for this assessment. To address vulnerabilities due to climate change, the Vulnerability Assessment tool utilized two 30-year epochs centered on 2050 (2035-2064) and 2085 (2070-2099) as well as a base epoch. This provided two scenarios (wet and dry) for each of the two epochs, excluding the base epoch. Consideration of both wet and dry scenarios reveals some of the uncertainties associated with the results produced using the climate changed hydrology and meteorology used as inputs to the vulnerability tool.

Section 4
Hydrologic Methodology and Modeling

4.1 HORN LAKE CREEK HEC-1 TO HEC-HMS CONVERSION

As stated in Section 1.3, hydrologic modeling for this study was performed by importing the 2005 HEC-1 model into HEC-HMS version 4.3. The HEC-1 model was calibrated to conditions in the Horn Lake watershed in 2002. Efforts under this feasibility phase assume that the calibrated HEC-1 model is accurate and reproduction of results using HEC-HMS provide a calibrated model to continue analysis of plan alternatives. The HEC-1 model used an initial and constant rate lose and Snyder’s unit hydrograph method for developing peak flows and flowlines for rural and urban areas. This method has historically been used to great success by the Memphis District and was determined to be maintain through this study to develop existing flows and future flows as a function of increased urbanization in the Horn Lake Creek Basin. The initial and constant rate loss method produces higher peak flows for intense frequency storm events as opposed to the NRCS CN method based on previous study experience developing FEMA FIS flows. The NRCS CN method has been shown that it can under-predict peak flow for intense storms when modeled for subbasins containing silt-loam and heavy soils which dominate this region and study location.

Pertinent hydrologic and hydraulic information related to the 2005 study is shown in the report entitled “Horn Lake Creek and Tributaries Tennessee and Mississippi, General Reevaluation Report”. These parameters remained unchanged in the development of the models used for this study aside from that mentioned previously in Section 1.3. The 2005 study divided the Horn Lake Creek basin into subareas to simulate the runoff process. The same subbasin delineation for this study was used to ensure consistency and allow for comparison to the 2002 model.

Figure 9 shows a delineation of the subareas for the Horn Lake Creek drainage basin developed in the original HEC-1 model. It should be noted the model extends to Highway 6, located in Shelby County, Tennessee.

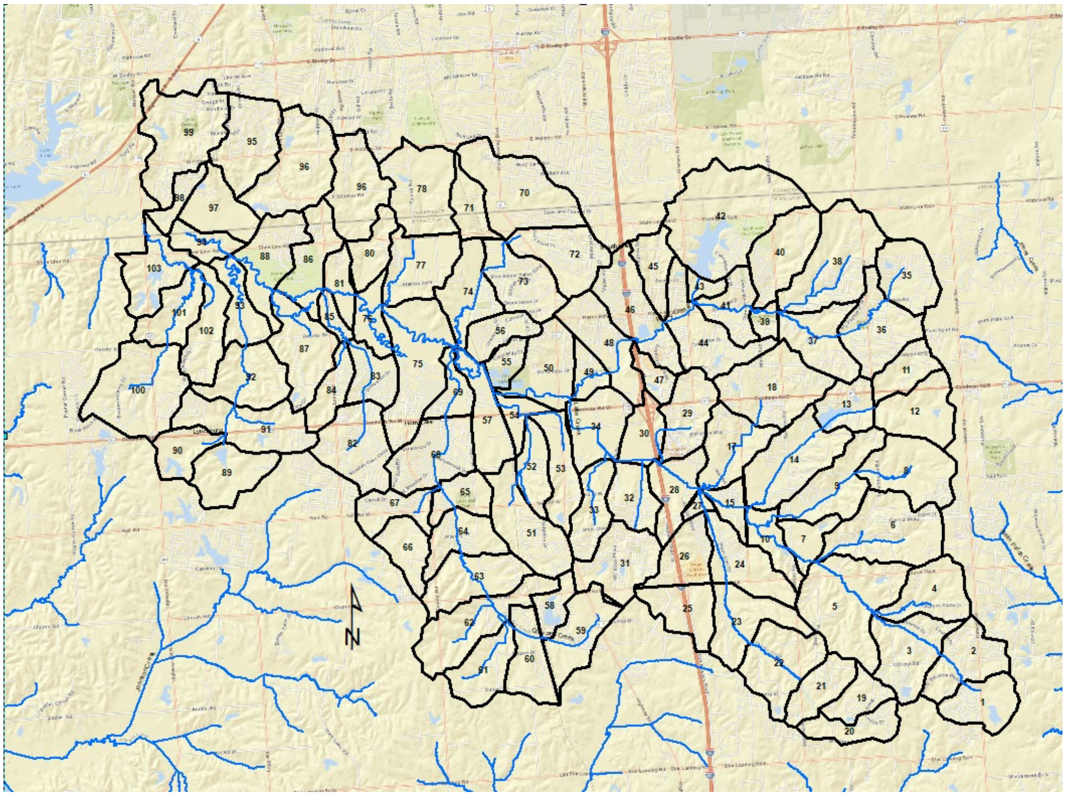


Figure 9. Horn Lake Creek Subarea Delineation

The import process created a basin model, meteorological model and control specifications as needed for HEC-HMS. All features in the HEC-1 were successfully imported and only required minor modifications to reproduce similar results.

4.1.1 Horn Lake Creek Basin Unit Hydrograph Parameters

Synthetic unit hydrographs were developed for each subarea using Snyder's Unit Hydrograph method. Coefficients required in Snyder's relationship varied depending on individual drainage basin characteristics. Atlas 14 hypothetical rainfall was applied to the synthetic unit hydrograph to develop a flood hydrograph for each subarea. To develop composite hydrographs at all pertinent points within the basin, the flood hydrographs were combined and routed using the modified Puls and normal depth methods. Modified Puls volume versus discharge relationships were derived from the 2002 HEC-2 model and updated using the 2018 HEC-RAS model to reflect improvements and alterations to channel geometry and roughness. All hydrologic coefficients, except for Snyder's time to peak, T_p, were unchanged and parameters (i.e., watercourse lengths) were not altered from the present study. The criteria used to select C_t and C_p values were developed during a previous analysis of several gaged basins in the Memphis District. C_t and C_p values are regional coefficients dependent upon basin slopes, stream patterns, shapes, and other properties. Snyder's lag time, T_p, was calculated for each subarea from measured values of L and L_{ca} based on a weighted stream slope. The equation for Snyder's T_p is shown below:

$$T_p = C_t(L \times L_{ca})^{0.3}$$

Where:

L = length in miles of the primary watercourse from the sub-basin outlet to the watershed divide.

L_{ca} = length in miles of the primary watercourse from the sub-basin outlet to the center of gravity of the basin.

The method for updating the 2002 time to peak values to 2018 values are detailed below and shown in Table 3 below.

4.1.2 Coldwater River Basin (Desoto County FIS)

Hydrologic information for this basin is presented in the Desoto County Flood Insurance Study dated March 6, 2018. Peak flows for the streams studied were developed by detailed and limited details methods. Detailed methods and flows in several basins were developed using HEC-HMS. The SCS Curve Number method was used, and average antecedent moisture conditions were assumed. Time of Concentration (TC) values were calculated based on the SCS Lag method, using subbasin slope, CN and hydraulic length. Regression equations were used for the remaining basins. Rural regression values were updated to reflect stream gage weighting. The Upper Camp Creek watershed is shown on Figure 10 highlighting the Nolehoe and Lick Creek basins.

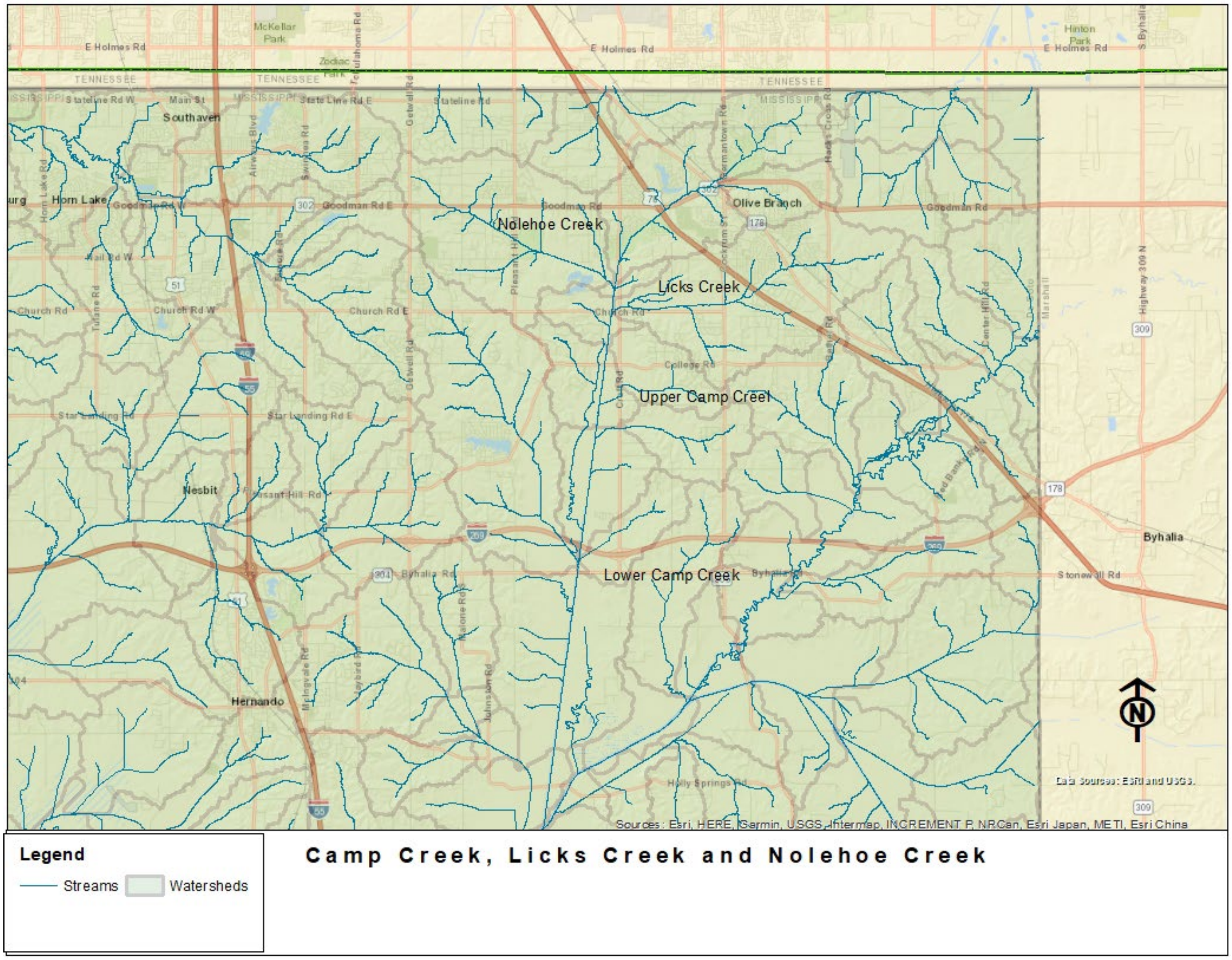


Figure 10. Camp Creek Watershed

4.2 HORN LAKE CREEK BASIN URBAN GROWTH ESTIMATES

Significant urban growth has occurred throughout the Horn Lake Creek watershed. An accurate estimation of this is needed to calculate changes in the time to peak and runoff volumes from the identified subbasins. Desoto County GIS department provided land use information showing subdivisions built prior to the 2005 study and the growth that transpired from 2002 to 2018. Figure 20 shows an estimate of the residential growth and development.

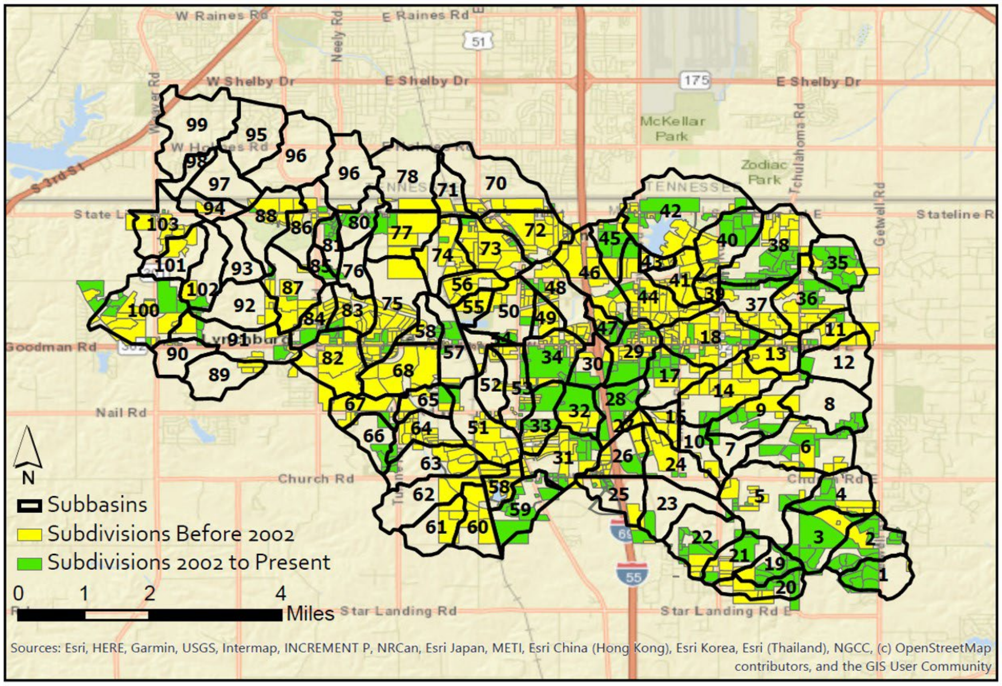


Figure 11. Horn Lake Creek HEC-HMS Hydrologic Subareas

Urban growth which is primarily commercial and industrial is present adjacent to Horn Lake Creek at the intersection of Goodman Road and Hwy 51 in subbasins 28, 32, 33, and 34. There is also noticeable growth along the outer edges of watershed which is primarily residential subdivision of varying lot sizes. Increasing the percent urban area for the subbasins should produce a quicker time-to-peak and greater runoff. It is also anticipated to see small initial losses.

Combining aerial photography with the Desoto County subdivision land use map showed that further refinement to percent urbanization was needed. It is noted that large subdivision lots have not been fully developed or show greater greenspace than typical to the zoning type. Based on the aerial overlays, reductions in total percent urban for subbasins were made to exclude those areas identified as undeveloped. Additional consideration was given to residential lots greater than or equal to 0.75-acres. It was assumed that larger residential lots contain more green open space than smaller lots and would lessen the urbanization impacts to runoff volumes. Rural areas identified in 2002 were converted to urban areas if development was identified in the aerial photography. Subarea parameters were then modified to account for these changes. The following bar-charts show the total urbanized growth as applied to each individual subarea.

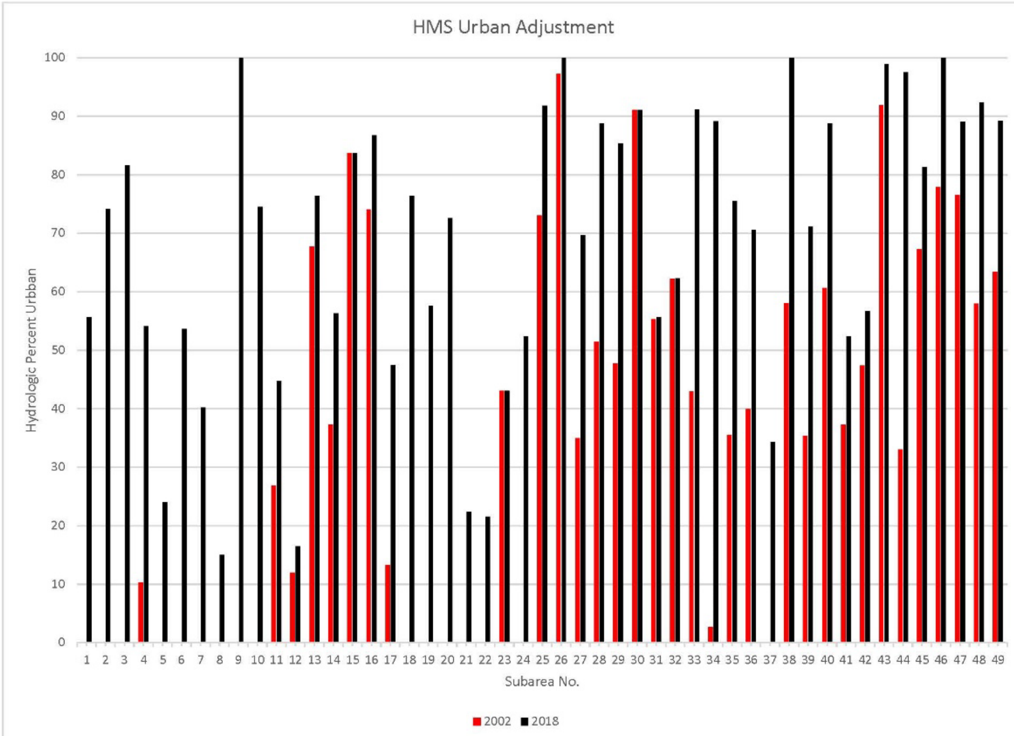


Figure 12. Subarea HEC-HMS Percent Urban Growth from 2002to 2018

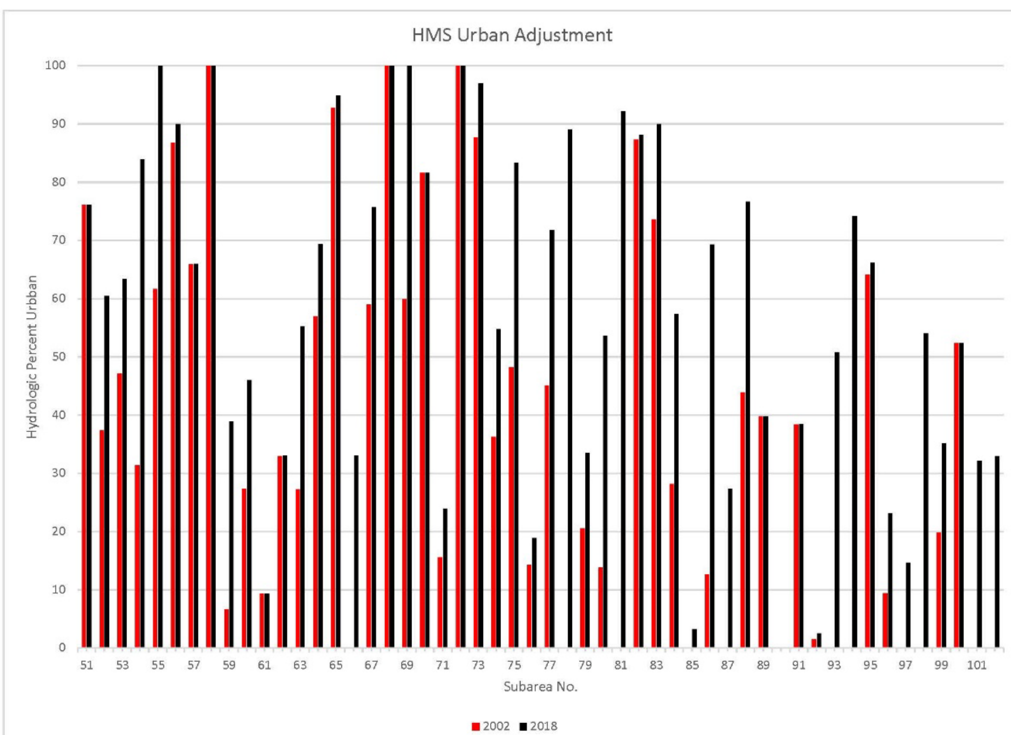


Figure 13. Subarea HEC-HMS Percent Urban 2002 vs. 2018

Subbasins showing urbanized growth were adjusted to reflect the accelerated runoff associated with urbanization and development. The adjustments are reflected in the Snyders T_p and initial and uniform loss rate parameters. The percent of urban area was applied to the unit hydrograph parameter Snyder's T_p by using the following relation:

$$T_{pu} = {}^{T_p}\!/_{(1 + \%urban/100)}$$

where:

T_{pu} = adjusted time to peak based on percent urban area.
T_p = HEC-1 initial T_p.
%urban = specific years percent urban area.

This relationship assumes that maximum urbanization will result in a 50% reduction in the time to peak of a subarea hydrograph. In the initial and constant loss method, the initial loss represents all losses occurring before infiltration losses occur, including the sticking of water droplets to vegetation and other exposed surfaces, settling of dust, and depression storage in puddles and furrows. The constant loss rate represents the maximum rate at which water can infiltrate the soil. Throughout past FIS studies, the Memphis District has developed a relationship of initial loss to constant loss rate for subbasins based on the dominant soils found in the region and percent urban. Values ranging from 1.0 inch to 0.5 inch and 0.1 in/hr and 0.05 in/hr have historically been used to model high intensity frequency storms for heavy silt loams and Sharkey clay respectively to great success. Undeveloped land uses included woods, cropland, pasture, non-grazed hayland, and extensive grassed areas such as parks, which are all assigned the same initial loss.

Initial losses were set at 1.0 inches and uniform loss rates were set at 0.1 inch/hour.

The initial and uniform loss rates were also adjusted to reflect progressive urbanization. Typically, losses for urbanized land ranged between the value of losses for rural land (maximum) to a value of one-half of that for rural land (minimum). The following equation provides a linear interpolation of the loss adjustment factor as a function of percent urbanized area. The equations to adjust initial and uniform loss rates are shown below:

$$Loss\ adjustment\ factor = 1 - (0.5 \times \%urban/100)$$

where:

%urban = specific years percent urban area.

The calculated adjustment factor is applied to both the rural initial loss and the rural constant loss rate. Zero percent urbanization is associated with an adjustment factor of 1.0. One-hundred percent urbanization is associated with a factor of 0.5.
Table 3 shows the resulting adjustments and parameters used in the HMS model. The 2002 and 2018 percent urban area is taken from NRCS land use GIS layers. The future percent urban was assumed to be 100% to generate the highest runoff flows.

Table 3. Subarea Hydrologic Parameters

Sub Area	Area (sq. mi)	2002 % Urban	2018 % Urban	Future % Urban	2002 Initial Loss (in)	2018 Initial Loss (in)	Future Initial Loss (in)	2002 Uniform Loss (in/hr)	2018 Uniform Loss (in/hr)	Future Uniform Loss (in/hr)	Rural Tp(hrs)	2002 Synders' Tp(hrs)	2018 Synders' Tp(hrs)	Future Synders' Tp(hrs)
1	0.41	0.0	55.6	100	1.00	0.72	0.50	0.10	0.07	0.05	0.50	0.50	0.32	0.25
2	0.51	0.0	74.1	100	1.00	0.63	0.50	0.10	0.06	0.05	0.48	0.48	0.28	0.24
3	0.67	0.0	81.6	100	1.00	0.59	0.50	0.10	0.06	0.05	0.97	0.97	0.53	0.49
4	0.47	10.3	54.1	100	0.95	0.73	0.50	0.09	0.07	0.05	0.86	0.78	0.56	0.43
5	0.97	0.0	24.0	100	1.00	0.88	0.50	0.10	0.09	0.05	0.94	0.94	0.76	0.47
6	0.82	0.0	53.6	100	1.00	0.73	0.50	0.10	0.07	0.05	0.96	0.96	0.63	0.48
7	0.23	0.0	40.2	100	1.00	0.80	0.50	0.10	0.08	0.05	0.82	0.82	0.58	0.41
8	0.61	0.0	15.0	100	1.00	0.93	0.50	0.10	0.09	0.05	0.79	0.79	0.69	0.40
9	0.58	0.0	100.0	100	1.00	0.50	0.50	0.10	0.05	0.05	1.07	1.07	0.54	0.54
10	0.28	0.0	74.5	100	1.00	0.63	0.50	0.10	0.06	0.05	0.91	0.91	0.52	0.46
11	0.29	26.9	44.7	100	0.87	0.78	0.50	0.09	0.08	0.05	0.73	0.58	0.50	0.37
12	0.51	12.0	16.5	100	0.94	0.92	0.50	0.09	0.09	0.05	0.65	0.58	0.56	0.33
13	0.33	67.7	76.3	100	0.66	0.62	0.50	0.07	0.06	0.05	0.56	0.33	0.32	0.28
14	0.72	37.2	56.3	100	0.81	0.72	0.50	0.08	0.07	0.05	1.02	0.74	0.65	0.51
15	0.18	83.7	83.7	100	0.58	0.58	0.50	0.06	0.06	0.05	0.81	0.44	0.44	0.41
16	0.76	74.0	86.8	100	0.63	0.57	0.50	0.06	0.06	0.05	0.70	0.40	0.37	0.35
17	0.5	13.3	47.4	100	0.93	0.76	0.50	0.09	0.08	0.05	1.05	0.93	0.71	0.53
19	0.27	0.0	76.4	100	1.00	0.62	0.50	0.10	0.06	0.05	0.63	0.63	0.36	0.32
20	0.3	0.0	57.6	100	1.00	0.71	0.50	0.10	0.07	0.05	0.60	0.60	0.38	0.30
21	0.35	0.0	72.5	100	1.00	0.64	0.50	0.10	0.06	0.05	0.72	0.72	0.42	0.36
22	0.62	0.0	22.4	100	1.00	0.89	0.50	0.10	0.09	0.05	0.74	0.74	0.60	0.37
23	0.53	0.0	21.5	100	1.00	0.89	0.50	0.10	0.09	0.05	0.62	0.62	0.51	0.31
24	0.5	43.1	43.1	100	0.78	0.78	0.50	0.08	0.08	0.05	1.34	0.94	0.94	0.67
25	0.4	0.0	52.3	100	1.00	0.74	0.50	0.10	0.07	0.05	0.59	0.59	0.39	0.30
26	0.39	73.0	91.8	100	0.64	0.54	0.50	0.06	0.05	0.05	0.75	0.43	0.39	0.38
27	0.07	97.2	100.0	100	0.51	0.50	0.50	0.05	0.05	0.05	0.27	0.14	0.14	0.14
28	0.4	34.9	69.6	100	0.83	0.65	0.50	0.08	0.07	0.05	0.73	0.54	0.43	0.37
29	0.45	51.4	88.8	100	0.74	0.56	0.50	0.07	0.06	0.05	0.75	0.50	0.40	0.38
30	0.3	47.8	85.4	100	0.76	0.57	0.50	0.08	0.06	0.05	1.04	0.70	0.56	0.52
31	0.57	91.0	91.0	100	0.55	0.55	0.50	0.05	0.05	0.05	0.65	0.34	0.34	0.33
32	0.34	55.3	55.7	100	0.72	0.72	0.50	0.07	0.07	0.05	1.46	0.94	0.94	0.73
33	0.37	62.1	62.3	100	0.69	0.69	0.50	0.07	0.07	0.05	0.82	0.51	0.51	0.41
34	0.48	43.0	91.1	100	0.79	0.54	0.50	0.08	0.05	0.05	1.51	1.06	0.79	0.76
35	0.45	2.7	89.1	100	0.99	0.55	0.50	0.10	0.06	0.05	0.60	0.58	0.32	0.30
36	0.6	35.5	75.5	100	0.82	0.62	0.50	0.08	0.06	0.05	0.75	0.55	0.43	0.38
37	0.49	39.9	70.5	100	0.80	0.65	0.50	0.08	0.06	0.05	0.86	0.61	0.50	0.43
38	0.87	0.0	34.3	100	1.00	0.83	0.50	0.10	0.08	0.05	0.97	0.97	0.72	0.49
39	0.11	58.1	100.0	100	0.71	0.50	0.50	0.07	0.05	0.05	0.60	0.38	0.30	0.30
40	0.54	35.4	71.1	100	0.82	0.64	0.50	0.08	0.06	0.05	0.85	0.63	0.50	0.43
41	0.33	60.6	88.7	100	0.70	0.56	0.50	0.07	0.06	0.05	0.88	0.55	0.47	0.44
42	1.34	37.3	52.3	100	0.81	0.74	0.50	0.08	0.07	0.05	1.54	1.12	1.01	0.77
43	0.1	47.4	56.6	100	0.76	0.72	0.50	0.08	0.07	0.05	0.46	0.31	0.29	0.23
44	0.63	91.8	98.9	100	0.54	0.51	0.50	0.05	0.05	0.05	0.87	0.45	0.44	0.44
45	0.3	33.0	97.5	100	0.84	0.51	0.50	0.08	0.05	0.05	0.69	0.52	0.35	0.35
46	0.61	67.3	81.3	100	0.66	0.59	0.50	0.07	0.06	0.05	0.64	0.38	0.35	0.32
47	0.17	77.9	100.0	100	0.61	0.50	0.50	0.06	0.05	0.05	0.55	0.31	0.28	0.28
48	0.53	76.5	89.0	100	0.62	0.56	0.50	0.06	0.06	0.05	0.57	0.32	0.30	0.29
49	0.19	58.0	92.3	100	0.71	0.54	0.50	0.07	0.05	0.05	0.64	0.41	0.33	0.32
50	0.52	63.4	89.2	100	0.68	0.55	0.50	0.07	0.06	0.05	1.39	0.85	0.73	0.70
51	0.61	76.0	76.0	100	0.62	0.62	0.50	0.06	0.06	0.05	0.90	0.51	0.51	0.45
52	0.3	37.3	60.4	100	0.81	0.70	0.50	0.08	0.07	0.05	0.95	0.69	0.59	0.48
53	0.42	47.1	63.4	100	0.76	0.68	0.50	0.08	0.07	0.05	1.23	0.84	0.75	0.62
54	0.16	31.3	83.9	100	0.84	0.58	0.50	0.08	0.06	0.05	0.89	0.68	0.48	0.45

Sub Area	Area (sq. mi)	2002 % Urban	2018 % Urban	Future % Urban	2002 Initial Loss (in)	2018 Initial Loss (in)	Future Initial Loss (in)	2002 Uniform Loss (in/hr)	2018 Uniform Loss (in/hr)	Future Uniform Loss (in/hr)	Rural Tp(hrs)	2002 Synders' Tp(hrs)	2018 Synders' Tp(hrs)	Future Synders' Tp(hrs)
55	0.18	61.6	100.0	100	0.69	0.50	0.50	0.07	0.05	0.05	0.40	0.25	0.20	0.20
56	0.28	86.7	89.9	100	0.57	0.55	0.50	0.06	0.06	0.05	0.71	0.38	0.37	0.36
57	0.44	65.9	65.9	100	0.67	0.67	0.50	0.07	0.07	0.05	1.93	1.16	1.16	0.97
58	0.19	100.0	100.0	100	0.50	0.50	0.50	0.05	0.05	0.05	0.61	0.31	0.31	0.31
59	0.49	6.7	38.9	100	0.97	0.81	0.50	0.10	0.08	0.05	0.91	0.85	0.66	0.46
60	0.41	27.3	46.0	100	0.86	0.77	0.50	0.09	0.08	0.05	0.83	0.65	0.57	0.42
61	0.41	9.3	9.3	100	0.95	0.95	0.50	0.10	0.10	0.05	0.80	0.73	0.73	0.40
62	0.66	33.0	33.0	100	0.84	0.84	0.50	0.08	0.08	0.05	0.62	0.47	0.47	0.31
63	0.55	27.2	55.1	100	0.86	0.72	0.50	0.09	0.07	0.05	0.57	0.45	0.37	0.29
64	0.38	56.9	69.4	100	0.72	0.65	0.50	0.07	0.07	0.05	0.60	0.38	0.35	0.30
65	0.27	92.7	94.8	100	0.54	0.53	0.50	0.05	0.05	0.05	0.44	0.23	0.23	0.22
66	0.31	0.0	33.1	100	1.00	0.83	0.50	0.10	0.08	0.05	0.55	0.55	0.41	0.28
67	0.33	59.0	75.7	100	0.71	0.62	0.50	0.07	0.06	0.05	0.77	0.48	0.44	0.39
68	0.62	100.0	100.0	100	0.50	0.50	0.50	0.05	0.05	0.05	0.81	0.41	0.41	0.41
69	0.23	59.9	100.0	100	0.70	0.50	0.50	0.07	0.05	0.05	2.74	1.71	1.37	1.37
70	0.88	81.6	81.6	100	0.59	0.59	0.50	0.06	0.06	0.05	1.49	0.82	0.82	0.75
71	0.28	15.6	23.9	100	0.92	0.88	0.50	0.09	0.09	0.05	1.03	0.89	0.83	0.52
72	0.54	100.0	100.0	100	0.50	0.50	0.50	0.05	0.05	0.05	1.04	0.52	0.52	0.52
73	0.51	87.7	96.9	100	0.56	0.52	0.50	0.06	0.05	0.05	0.63	0.34	0.32	0.32
74	0.46	36.2	54.7	100	0.82	0.73	0.50	0.08	0.07	0.05	1.62	1.19	1.05	0.81
75	0.87	48.2	83.3	100	0.76	0.58	0.50	0.08	0.06	0.05	1.65	1.11	0.90	0.83
76	0.7	14.2	18.8	100	0.93	0.91	0.50	0.09	0.09	0.05	0.89	0.78	0.75	0.45
77	0.54	45.1	71.7	100	0.77	0.64	0.50	0.08	0.06	0.05	1.30	0.90	0.76	0.65
78	0.26	0.0	89.0	100	1.00	0.56	0.50	0.10	0.06	0.05	1.70	1.70	0.90	0.85
79	0.61	20.5	33.5	100	0.90	0.83	0.50	0.09	0.08	0.05	0.84	0.70	0.63	0.42
80	0.3	13.8	53.6	100	0.93	0.73	0.50	0.09	0.07	0.05	0.92	0.81	0.60	0.46
81	0.2	0.0	92.1	100	1.00	0.54	0.50	0.10	0.05	0.05	0.33	0.33	0.17	0.17
82	0.62	87.3	88.1	100	0.56	0.56	0.50	0.06	0.06	0.05	0.88	0.47	0.47	0.44
83	0.32	73.6	89.9	100	0.63	0.55	0.50	0.06	0.06	0.05	1.14	0.66	0.60	0.57
84	0.29	28.1	57.3	100	0.86	0.71	0.50	0.09	0.07	0.05	0.83	0.65	0.53	0.42
85	0.12	0.0	3.3	100	1.00	0.98	0.50	0.10	0.10	0.05	0.94	0.94	0.91	0.47
86	0.48	12.6	69.3	100	0.94	0.65	0.50	0.09	0.07	0.05	1.23	1.09	0.73	0.62
87	0.57	0.0	27.3	100	1.00	0.86	0.50	0.10	0.09	0.05	1.01	1.01	0.79	0.51
88	0.53	43.9	76.6	100	0.78	0.62	0.50	0.08	0.06	0.05	0.63	0.44	0.36	0.32
89	0.46	39.7	39.8	100	0.80	0.80	0.50	0.08	0.08	0.05	0.62	0.44	0.44	0.31
90	0.3	0.0	0.0	100	1.00	1.00	0.50	0.10	0.10	0.05	0.72	0.72	0.72	0.36
91	0.68	38.4	38.4	100	0.81	0.81	0.50	0.08	0.08	0.05	0.57	0.41	0.41	0.29
92	0.53	1.5	2.5	100	0.99	0.99	0.50	0.10	0.10	0.05	1.10	1.08	1.07	0.55
93	0.35	0.0	50.8	100	1.00	0.75	0.50	0.10	0.07	0.05	1.99	1.99	1.32	1.00
94	0.23	0.0	74.2	100	1.00	0.63	0.50	0.10	0.06	0.05	1.76	1.76	1.01	0.88
95	0.6	64.1	66.1	100	0.68	0.67	0.50	0.07	0.07	0.05	0.94	0.57	0.57	0.47
96	0.82	9.4	23.1	100	0.95	0.88	0.50	0.10	0.09	0.05	1.39	1.27	1.13	0.70
97	0.44	0.0	14.6	100	1.00	0.93	0.50	0.10	0.09	0.05	1.20	1.20	1.05	0.60
98	0.21	0.0	54.0	100	1.00	0.73	0.50	0.10	0.07	0.05	0.84	0.84	0.55	0.42
99	0.87	19.8	35.2	100	0.90	0.82	0.50	0.09	0.08	0.05	1.05	0.88	0.78	0.53
100	0.87	52.3	52.3	100	0.74	0.74	0.50	0.07	0.07	0.05	1.01	0.66	0.66	0.51
101	0.38	0.0	32.1	100	1.00	0.84	0.50	0.10	0.08	0.05	1.22	1.22	0.92	0.61
102	0.42	0.0	32.9	100	1.00	0.84	0.50	0.10	0.08	0.05	0.94	0.94	0.71	0.47
103	0.48	0.0	52.8	100	1.00	0.74	0.50	0.10	0.07	0.05	1.01	1.01	0.66	0.51

4.3 RESULTS AND FLOW VERIFICATION

Figures 14 thru 17 show the comparison of results from the HEC-1 model and the new HEC-HMS model. These results indicate that the new model produces results that are in line with the previous study for the 2002 existing conditions simulation. The results of the 2018 existing conditions are also shown on these plots. Flows for less frequent events in 2018 are approximately equivalent to those shown in 2002 while more frequent events have higher flows than those in 2002. These results prove that the conversion from the 2002 HEC-1 model to the 2018 HMS model was successful and depicts an increase in flows due to increased urbanization across the Horn Lake Creek basin. Since the efforts of this study rely on calibration and validation to previous modeling efforts rather than recorded gage data it is highly recommended that design efforts in PED phases of this project continue to calibrate and validate to the latest flow data available or seek to obtain measured flows for the hydrology models.

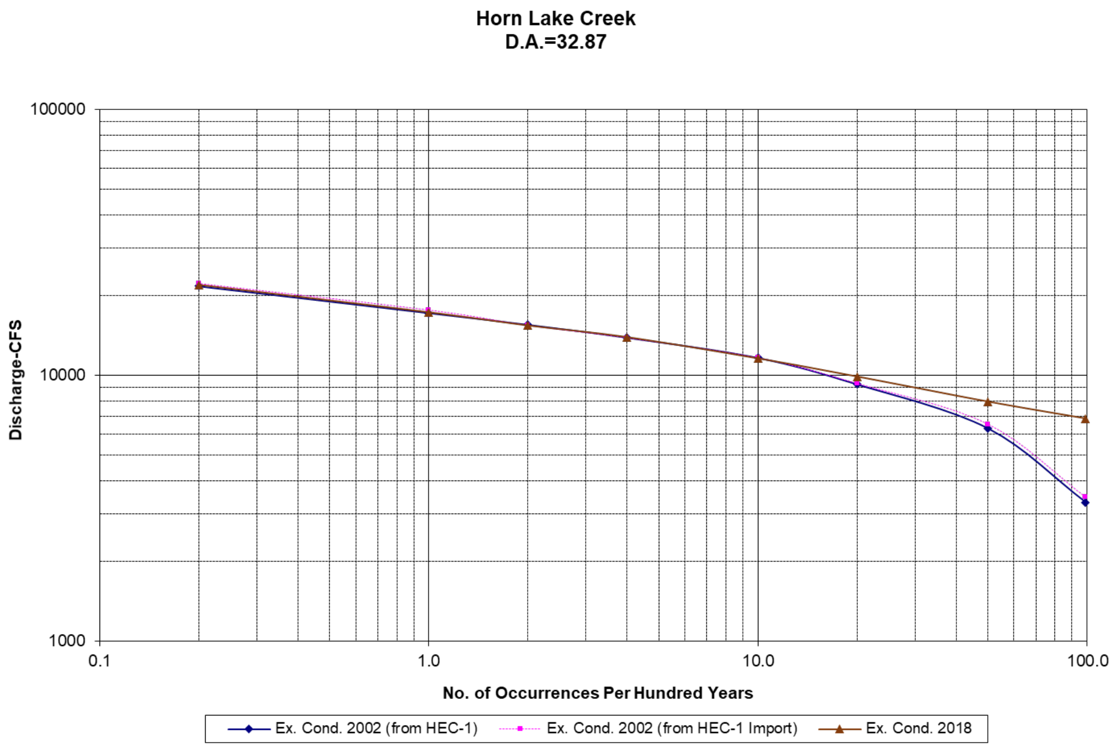


Figure 14. Horn Lake Creek HEC-1 to HEC-HMS Comparison (DA=32.87 Sq.Mi.)

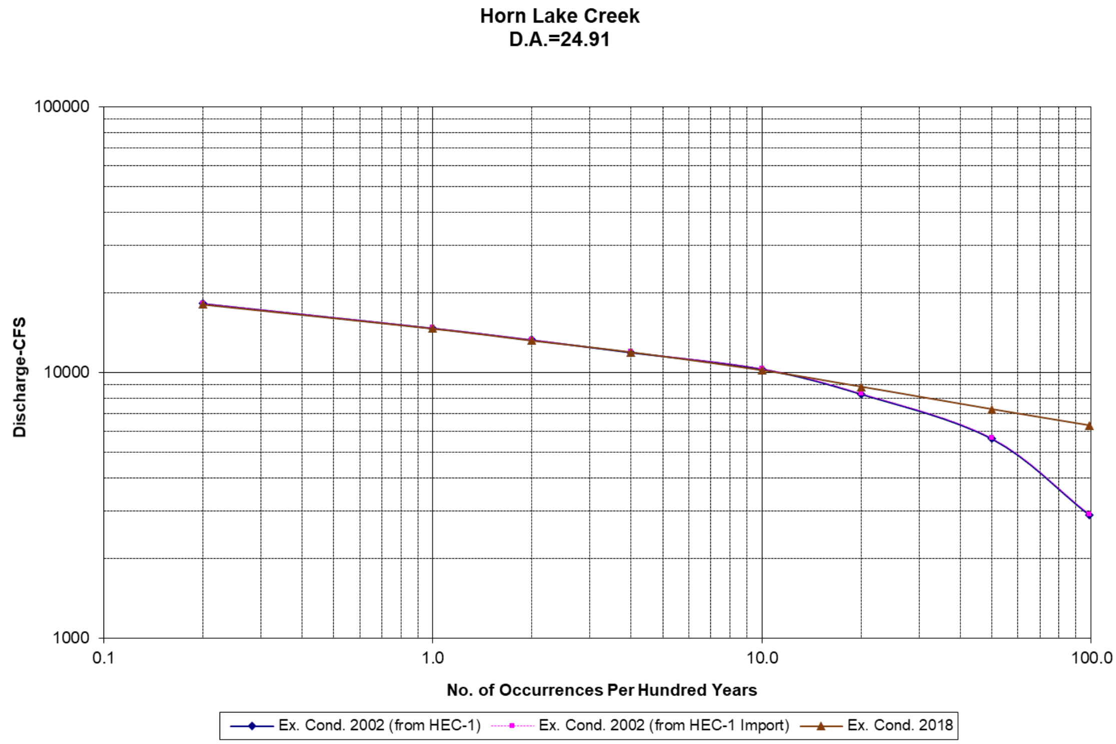


Figure 15. Horn Lake Creek HEC-1 to HEC-HMS Comparison (DA=24.91 Sq.Mi.)

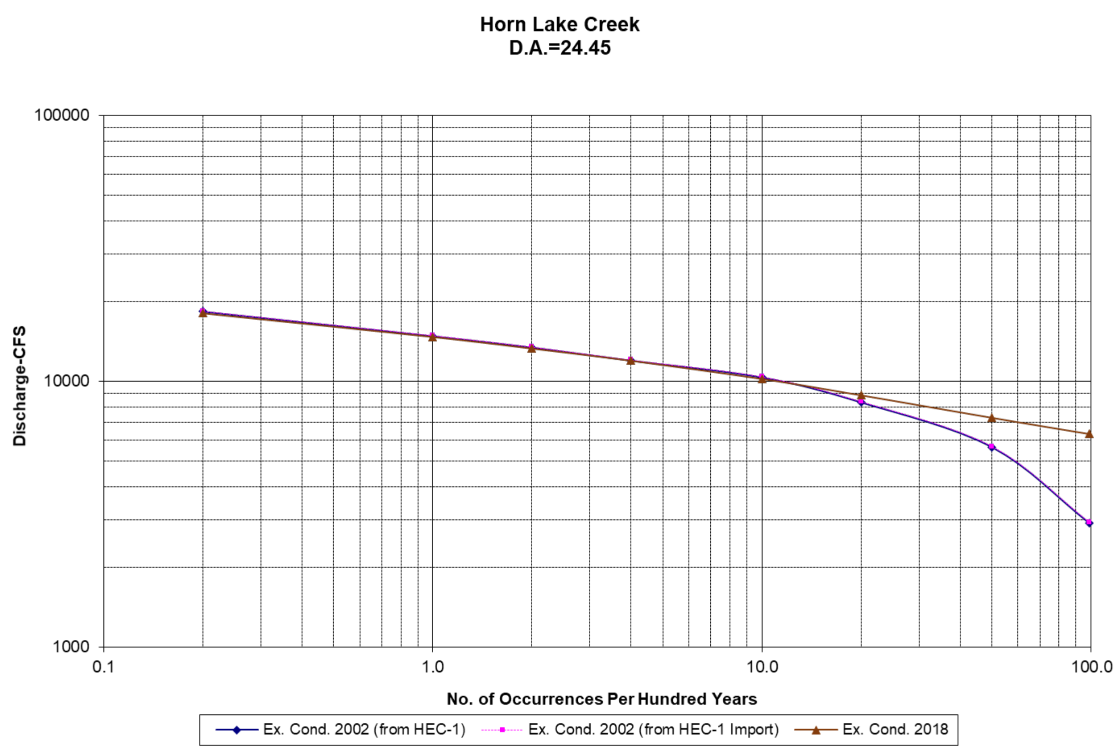


Figure 16. Horn Lake Creek HEC-1 to HEC-HMS Comparison (DA=24.45 Sq.Mi.)

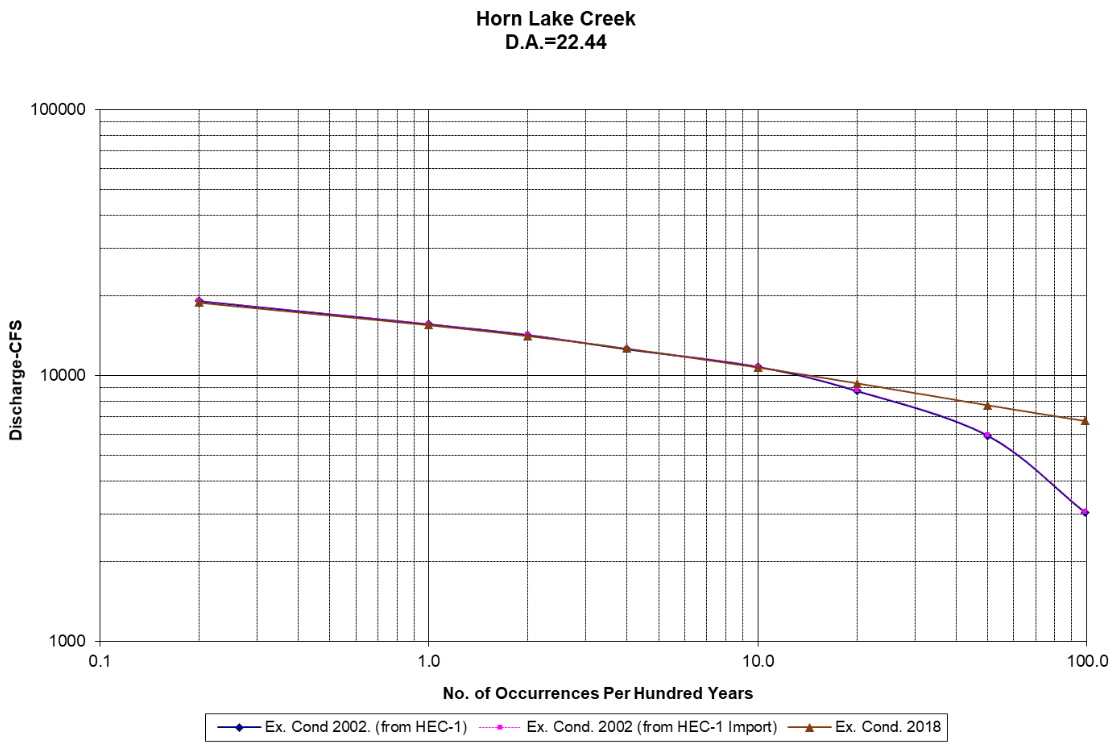


Figure 17. Horn Lake Creek HEC-1 to HEC-HMS Comparison (DA=22.44 Sq.Mi.)

As noted in the previous section Horn Lake Creek watershed does not contain a stream gage station. This limited the ability of the team to develop and calibrate flows for the basin. Due to the absence of stream gage information, the original 2005 project attempted to verify flow using USGS regression equations. However, this method produced results with standard errors that were deemed unacceptable. A secondary method was used which relied upon using the gaged adjacent Coldwater River basin. This basin has similar runoff characteristics as Horn Lake Creek and streams that the Vicksburg District monitors. A 1990 study entitled “Hydrologic Analysis of the Coldwater River Watershed” conducted by Lenzotti and Fullerton Consulting Engineers, Inc. developed flow vs. frequency for numerous streams in north Desoto County. The stream flows developed for Upper Camp Creek are considered reliable for comparison and useful for verification of the Horn Lake Creek basin flows. Figure 18 below shows the proximity of the two basins.

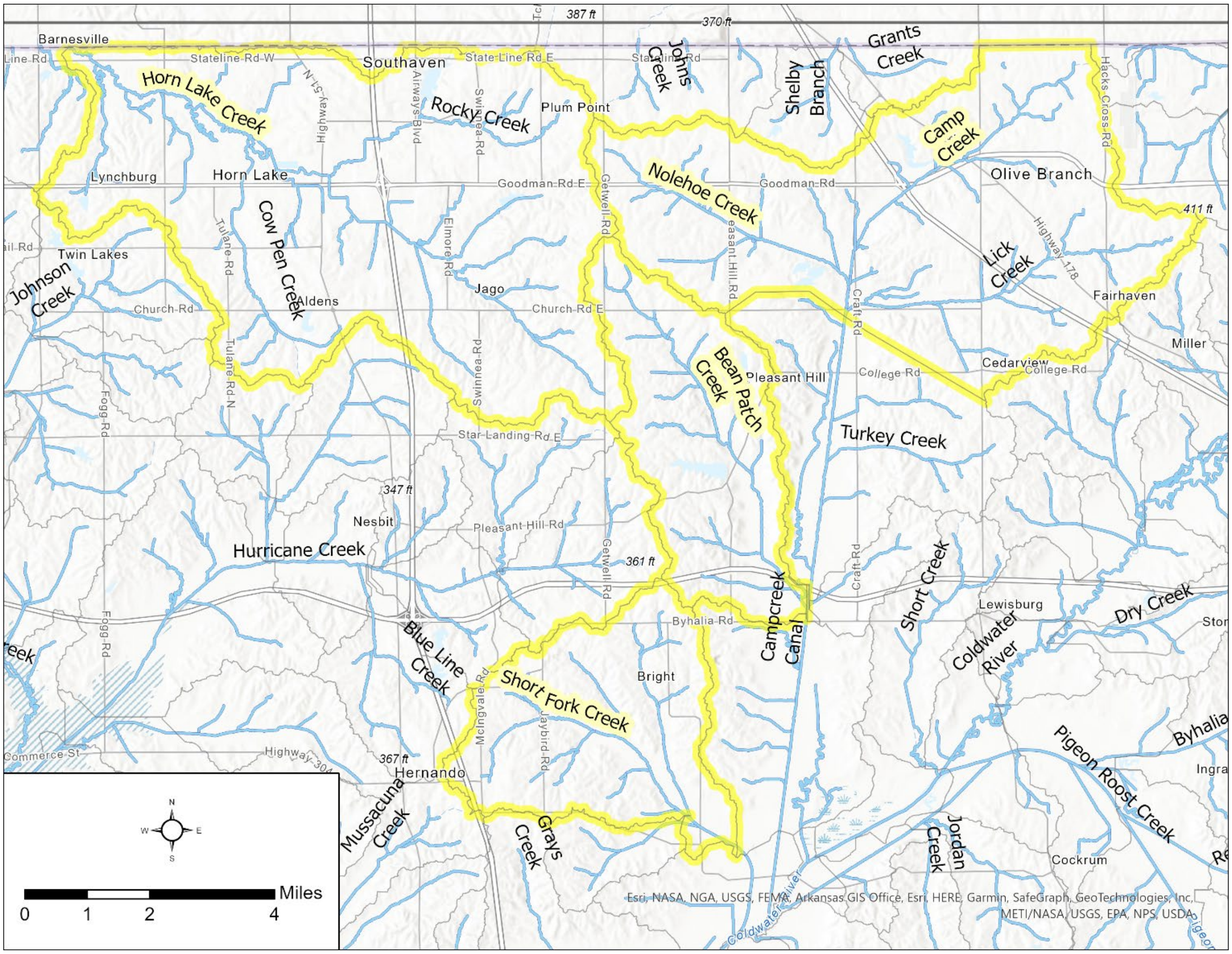


Figure 18. Horn Lake Creek Flow Validation-Upper Camp Creek Adjacent Basin

The 2018 FIS contains a table with updated HMS flows for Nolehoe and Upper Camp Creek. These flows were used for verification. Comparison plots of similarly sized drainage basins for both the Coldwater tributaries and the Horn Lake Creek flows for 2005 and 2018 are shown in Figures 19 to 23. The drainage areas for both streams are of similar land use and geography. By comparing the flow frequencies for the Coldwater River and Horn Lake Creek the team has confidence in the results from the HEC-HMS model for Horn Lake Creek.

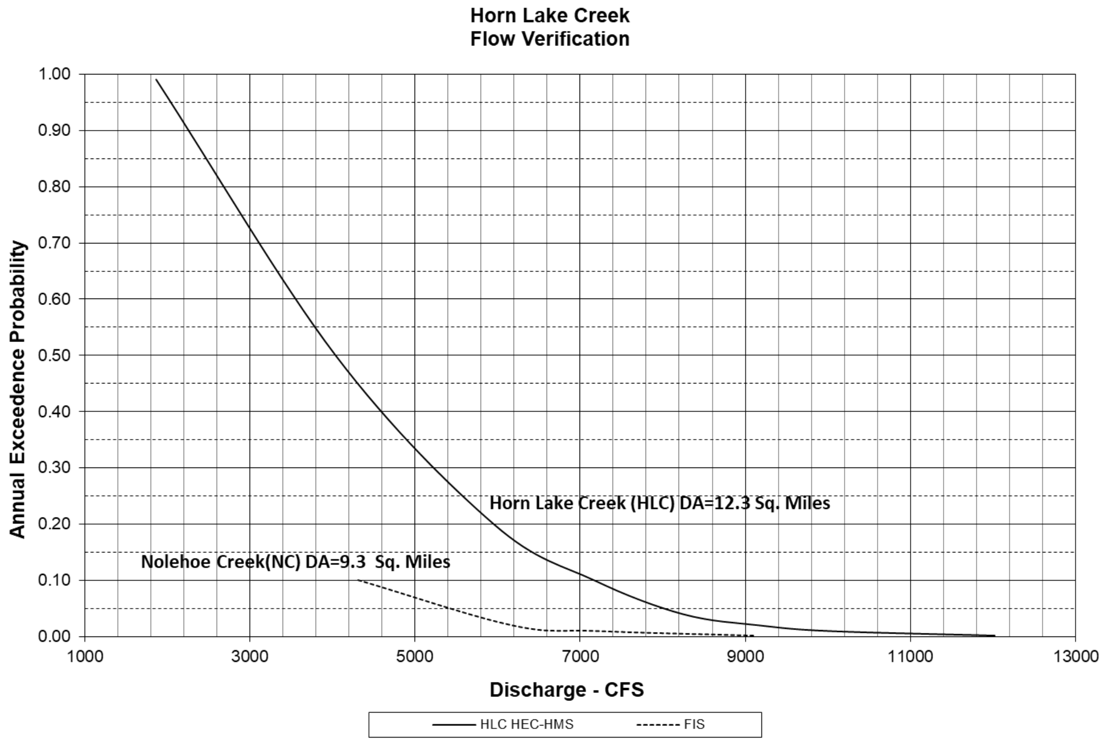


Figure 19. Horn Lake Creek Flow Verification Approximate DA~11 square miles

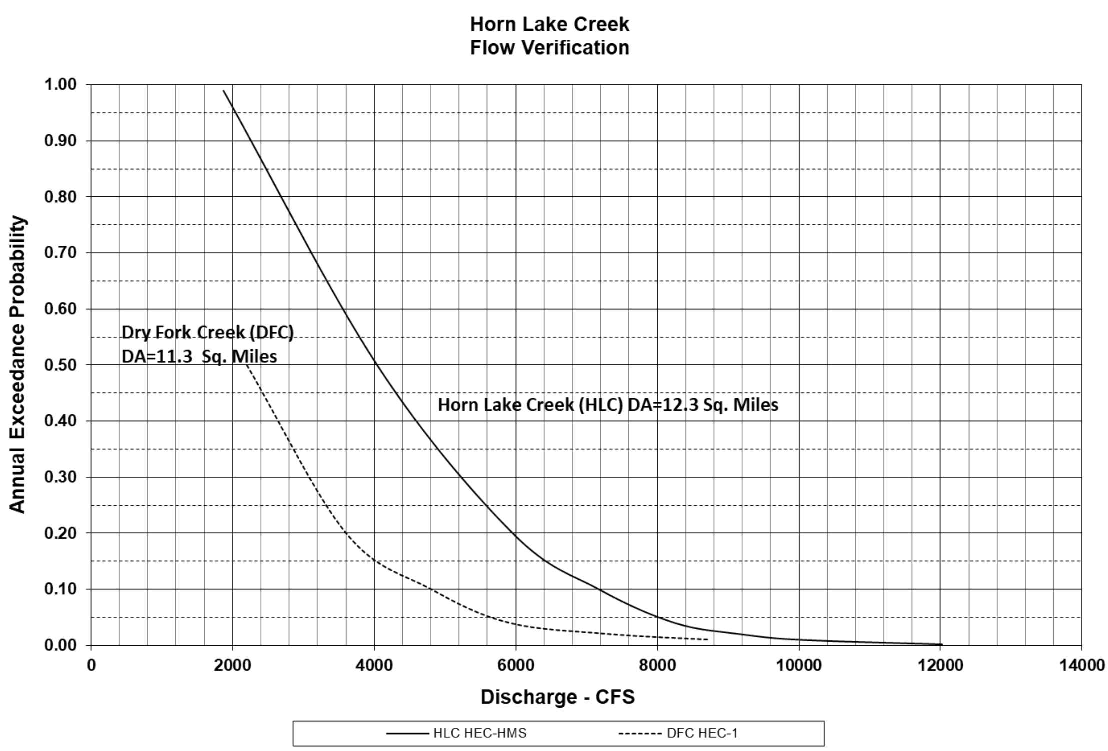


Figure 20. Horn Lake Creek Flow Verification Approximate DA~12 square miles

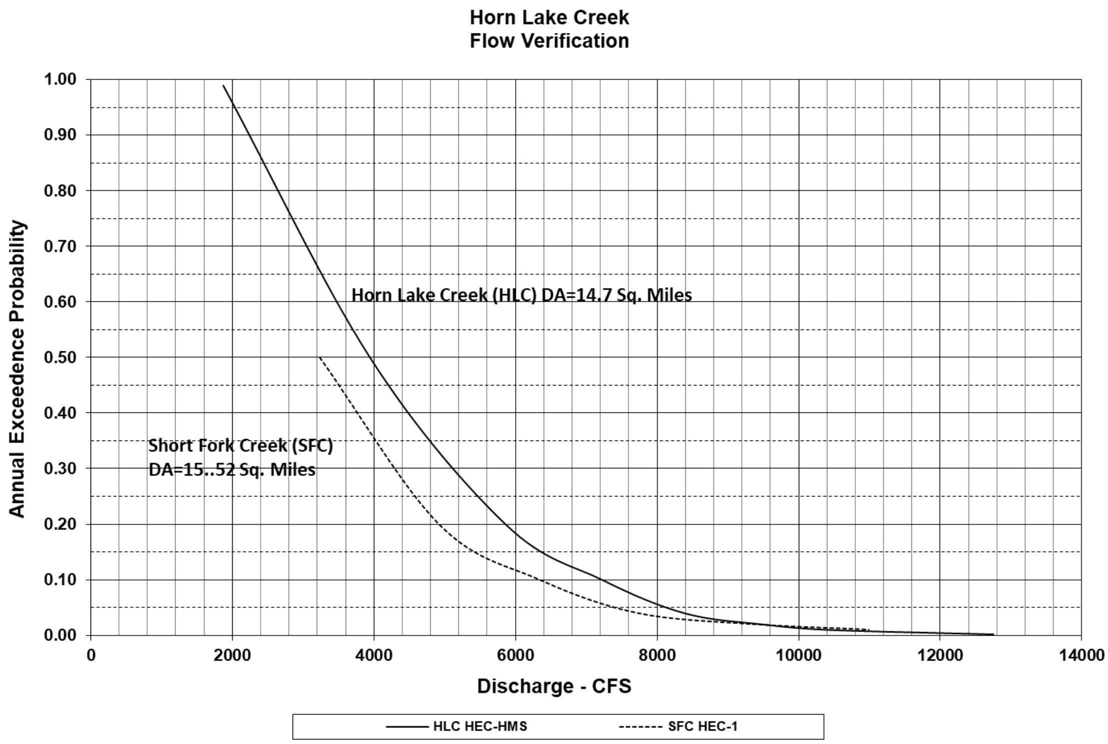


Figure 21. Horn Lake Creek Flow Verification Approximate DA~15 square miles

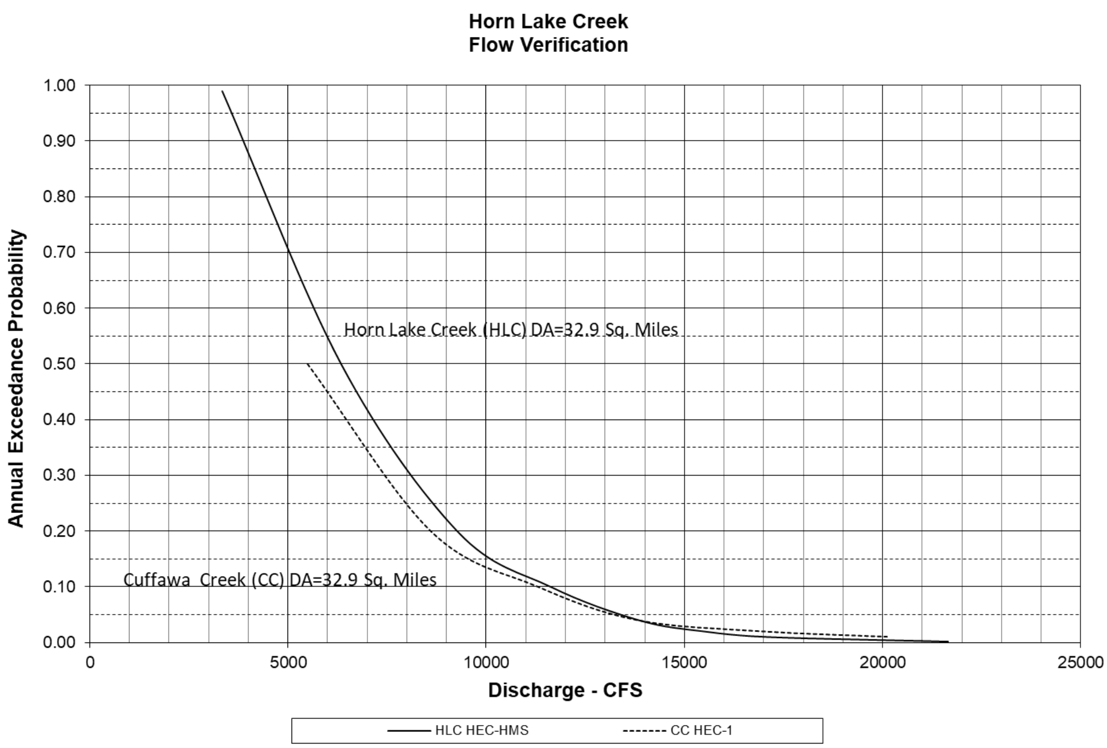


Figure 22. Horn Lake Creek Flow Verification Approximate DA~31 square miles

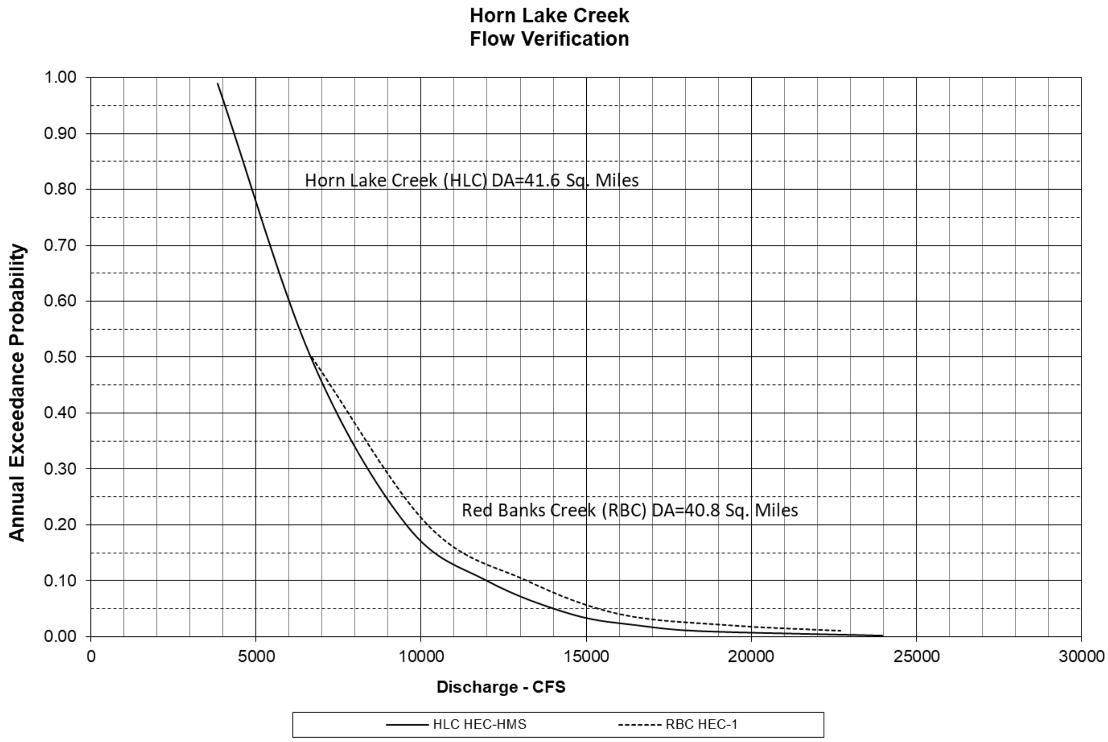


Figure 23. Horn Lake Flow Verification Approximate DA~40.square miles

Section 5
Risk Based Analysis

The HEC-FDA Flood Damage Reduction Analysis was used to compute flood damages. This assessment is conducted by the Economics Project Team member and the results of the analysis are presented in Appendix L.

As a part of the economic analysis, an assessment of the quality of data used for hydrologic and hydraulic data must be conducted. The Horn Lake Creek Basin does not possess stream gages and flow calibration/verification was challenging. The lack of stream gage data makes a direct analytical approach of evaluating flood damage reduction impossible. Due to this, uncertainty parameters for this study are based on a simplistic procedure which uses an estimated period of record. Guidance for using this procedure is presented in the HEC-FDA Technical Reference entitled “Uncertainty Estimates for Graphical Frequency Curves and EM 1110-2-1619, entitled “Risk-Based Analysis for Flood Damage Reduction Studies” dated 1996. This guidance also recommends that stage-discharge relationships be evaluated for each of the 8 frequencies listed in the study. The FDA model for this study does not currently utilize a rating curve. This is a known issue and the PDT plans to improve uncertainty estimates. This study is currently not using standard deviation for uncertainty and is instead using a gage record length to estimate hydrologic and hydraulic uncertainty or error surrounding the stage-duration probability relationships. Further refinement of uncertainty in the H&H modeling will be readdressed and may impact the initial cost estimates and inundation presented in this report.

A fully calibrated model was unable to be produced due to the lack of any gaging station in the Horn Lake Creek watershed. Although flows were compared and verified to the hydrologically similar Coldwater Creek watershed, the team took a conservative approach to selecting an equivalent record length for an ungagged watershed. Guidance in EM 1110-2-1619 states that an Equivalent Record length ranging from 10 and 30 years should be used when flows are derived using an uncalibrated rainfall-runoff model (HEC-HMS). EM 1110-2-1619 guidance is shown below.

Table 4. EM 1110-2-1619, Table 4.5 Equivalent Record Length Guidelines

Table 4-5
Equivalent Record Length Guidelines

Method of Frequency Function Estimation	Equivalent Record Length[1]
Analytical distribution fitted with long-period gauged record available at site	Systematic record length
Estimated from analytical distribution fitted for long-period gauge on the same stream, with upstream drainage area within 20% of that of point of interest	90% to 100% of record length of gauged location
Estimated from analytical distribution fitted for long-period gauge within same watershed	50% to 90% of record length
Estimated with regional discharge-probability function parameters	Average length of record used in regional study
Estimated with rainfall-runoff-routing model calibrated to several events recorded at short-interval event gauge in watershed	20 to 30 years
Estimated with rainfall-runoff-routing model with regional model parameters (no rainfall-runoff-routing model calibration)	10 to 30 years
Estimated with rainfall-runoff-routing model with handbook or textbook model parameters	10 to 15 years

[1] Based on judgment to account for the quality of any data used in the analysis, for the degree of confidence in models, and for previous experience with similar studies.

A period of record of 20 years was adopted for the study. Shorter periods of record produce a larger uncertainty in the flow information and an estimate of 20 years would produce results based on an upper uncertainty level and was considered acceptable for the economic analysis.

Following guidance in EM1110-2-1619, an estimate of expected standard deviation for ungagged stream reaches is given as equation 5-5 in the document and is listed below.

$$S = \left[0.07208 + 0.04936\, I_{Bed} - 2.2626 \times 10^{-7} A_{Basin} + 0.02164 H_{Range} + 1.4194 \times 10^{-5} Q_{100}\right]^2$$

Where:

S = standard deviation of uncertainty in meters,
H_{Range} = maximum expected or observed stage range,
A_{Basin} = basin area in square kilometers,
Q_{100} = estimated discharge in cubic meters per second

Calculating the standard deviation for the main study reach, Horn Lake Creek, which is adjacent to the project site, a standard deviation in stage is 0.15m or 0.51ft.

The same EM gives additional ways to estimate standard deviation in stage based on confidence in Manning’s n values. Table 5-2 of the EM, entitled “Minimum Standard Deviation of Error in Stage” states that if reliability in the Manning’s n value is a “fair to good” estimate and modeled cross sections are based on field data or Aerial Spot Elevations, the standard deviation should be 0.3 to 0.7 feet. Both the HEC-RAS 1D and 1D/2D overbank geometries were developed using Aerial Spot Elevations (i.e., LiDAR). While the channel geometries remained unchanged from the 2002 HEC-2 model, the overbanks were updated to the most current 2018 datasets for land use and topography. Because water surface elevations were matched throughout the updating of the model, a standard deviation of error in stage of 0.5 feet was selected based on the grid spacing and relative accuracy of the terrain dataset and calibration to corresponding flows generated from the Coldwater River tributary gages.

Table 5. EM 1110-2-1619 Minimum Standard Deviation of Error in Stage

Table 5-2
***Minimum* Standard Deviation of Error in Stage**

	Standard Deviation (in feet)	
Manning's n Value Reliability[1]	**Cross Section Based on Field Survey or Aerial Spot Elevation**	**Cross Section Based on Topographic Map with 2-5' Contours**
Good	0.3	0.6
Fair	0.7	0.9
Poor	1.3	1.5

[1] Where good reliability of Manning's *n* value equates to excellent to very good model adjustment/validation to a stream gauge, a set of high water marks in the project effective size range, and other data. Fair reliability relates to fair to good model adjustment/ validation for which some, but limited, high-water mark data are available. Poor reliability equates to poor model adjustment/validation or essentially no data for model adjustment/validation.

The 2002 study model also used a standard deviation in water surface profiles of 0.5 feet. This standard deviation was derived using guidance from HEC Technical Paper 114 (TP-114) "Accuracy of Computed Water Surface Profiles" dated 1986. This paper provides regression equations that use flood depths, stream slopes and the confidence in the Manning's roughness estimate. Parameters were extracted from 1D HEC-RAS results to estimate stage uncertainty. The equations are shown below.

$$E_{mean} = 0.076 \times HD^{0.6} \times S^{0.11}(5 \times N_r + S_n)^{0.65}$$

And

$$E_{max} = 2.1 \times (E_{mean})^{0.8}$$

where:
E_{mean} = mean reach absolute profile error in feet
E_{max} = absolute reach maximum profile error in feet
HD = reach mean hydraulic depth in feet
S = reach average channel slope in feet per mile
Nr = reliability of estimation of Manning's coefficient on a scale of 0 to 1.0
Sn = the standardized survey accuracy interval 2-, 5-, 10-feet divided by 10

The equation for E_{mean} was applied at select HEC-RAS River Stations that coincided with key economic evaluation locations. Results from these calculations affirmed the use of a standard deviation in stage error of 0.5 feet. Coordination was conducted with the Economic PDT member to ensure data was compatible with the economic evaluation. Table 6 shows results from river mile 19.19 as an example.

Table 6. Results from Accuracy of Computed Water Surface Profiles for RM19.19

Horn Lake Creek Water Surface Stage Error at RM 19.19						
AEP	HD	S	Nr	Sn	Emean	Emax
500	2.8	0.001	1	0.1	0.19	0.56
100	2.5	0.001	1	0.1	0.18	0.53
50	2.5	0.001	1	0.1	0.18	0.53
25	2.4	0.001	1	0.1	0.17	0.52
10	2.2	0.001	1	0.1	0.16	0.50
5	2.1	0.001	1	0.1	0.16	0.48
2	1.6	0.001	1	0.1	0.14	0.43
1	1.1	0.001	1	0.1	0.11	0.36

Further analysis of stage and flow frequencies is planned by this PDT to refine risk and uncertainty estimates as the study moves into the PED phase. It is understood that this may impact structure heights and extents to which damages are induced. Additionally, the design team plans to collect current channel survey cross sections. It is anticipated that the channel has degraded and stage estimates in this report may be higher than what is currently being observed in the streams.

Section 6

Hydraulic Modeling and Methodology

6.1 HORN LAKE CREEK EXISTING CONDITIONS-HEC-RAS 1D STEADY FLOW

Hydraulic modeling was performed using HEC-RAS River Analysis System (RAS) version 5.0.7 computer software. The HEC-2 model, previously developed for the original 1993 Desoto County Flood Insurance Study (FIS) and updated in the 2005 Horn Lake Creek Feasibility study, was imported into HEC-RAS Steady Flow module for this study. Figure 24 shows Horn Lake Creek, Cow Pen Creek, Rocky Creek and Lateral D currently modeled in a HEC-RAS 1D steady flow environment.

Figure 24. HEC-RAS 1D Modeled Streams

The need and decision to use the previous 1D HEC-RAS information at the initiation of the project was driven by processes within the plan formulation process to expedite the study.

6.1.1 Channel and Overbank Updates

The imported model was basically unchanged for this study except for the inclusion of the HEC-HMS 2019 updated flows and revisions to channel roughness. It should be noted additional surveys were not obtained for this study and the channel dimensions still reflect channel 2002 conditions.

Initial study limitations dictated the use of the 2002 sections and any concerns in water surface accuracy is captured in the Risk and Uncertainty assessments conducted as a part of the economic analysis. Overbank topography, originally developed for the 2005 feasibility study by use of Light Detection and Radar (LIDAR) flown in 2003, was not altered for the updated 2018 analysis.

6.1.2 Channel Roughness Coefficients

Manning roughness coefficients were updated and based on field reconnaissance conducted in 2018, at the initiation of the current study. Channel roughness coefficients (n values) were estimated to range from 0.040 to 0.07. Overbank n values ranged from about 0.080 for cropland, 0.105 for wooded areas, and 0.055 for overbank urban areas. Manning's roughness coefficients were not altered for future conditions. As stated earlier, any uncertainty related to channel and overbank roughness changes will be included in the standard deviation adopted in the Risk and Uncertainty assessment and formally documented in the study "Risk Register", if considered significant.

6.1.3 Bridge Modeling

Bridge modeling parameters from the original HEC-2 was also imported into the 2018 HEC-RAS model and modified as necessary to reproduce previous water surface elevations. Yarnell's Method was originally used in the HEC-2 model and for consistency, was also used in the HEC-RAS model. After the import process, computed head losses were reviewed at each bridge to ensure the import process did not introduce any errors or results that were significantly different than the 2005 results. Original 1993 survey data, with the bridge piles and other structure information, were available if needed to complete the update. This information is discussed in more detail in the following sections.

6.1.4 Downstream Boundary Conditions

The downstream boundary conditions for Horn Lake Creek were established using the slope-area method. Downstream boundary conditions for Cow Pen Creek, Rocky Creek, and Lateral D were based on Horn Lake Creek frequency water surface elevations computed by HEC-RAS at each respective junction location.

6.2 COLDWATER RIVER BASIN EXISTING CONDITIONS-HEC-RAS 1D STEADY FLOW (DESOTO COUNTY FIS)

The Coldwater River Basin was analyzed using models developed for the Desoto County FIS. Cross section geometries were obtained from a combination of terrain data and field surveys. Bridges and culverts located with the detailed study and limited detailed study limits were field surveyed to obtain elevation and data and structural geometry. Manning's roughness coefficients for Camp Creek are 0.04 for the channel and 0.05-0.10 for the overbanks.

Downstream boundary conditions for the hydraulic models were set to normal depth using the starting slope calculated from values taken from topographic data, or where applicable, derived from the water surface elevations. Water-surface profiles were computed using HEC-RAS version 4.1.

Information presented in this report was supplemented with data from the Vicksburg District Corps of Engineers. The COE data is the most recent information and utilized where deemed appropriate. Figure 25 shows the main streams of investigated for flood risk management and reduction measures.

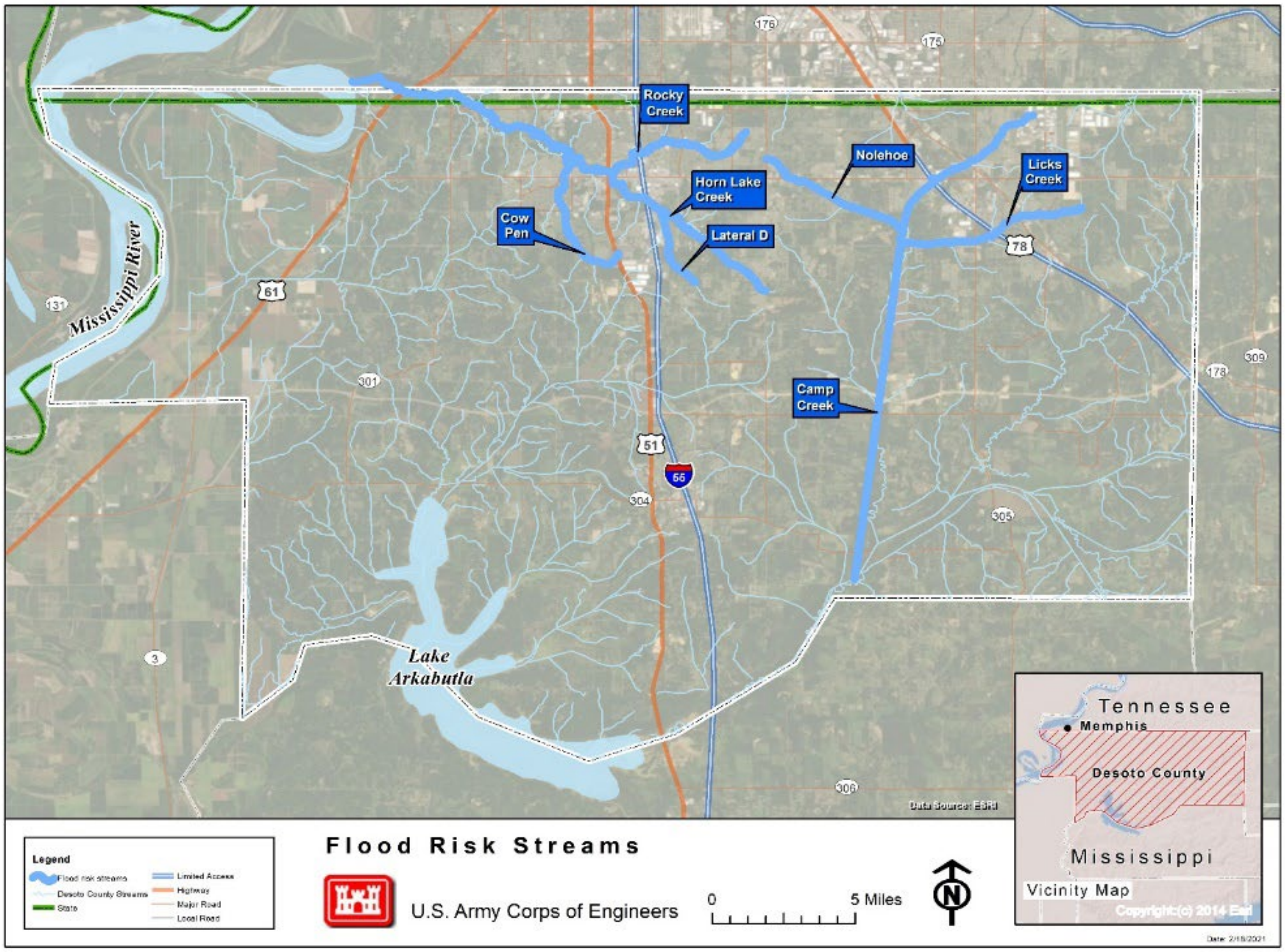


Figure 25. Coldwater River Basin Flood Risk Management Streams of Interest

6.3 EXISTING CONDITIONS

The RAS model used the FIS flows and was calibrated to the profiles and cross-sections presented in the FIS study for the 10-, 2-, 1-, & 0.2-percent annual chance floods. Comparison back to the HEC-2 model used in the FIS studies was done as a check on calibration of the new RAS model. The updated flows from the existing conditions HMS model were then used to create the existing conditions hydraulic RAS model. Since the watershed is ungaged the FIS model presents the best available flow and stage data to compare this projects hydrologic and hydraulic model too. Since the model is calibrated to a previous study and may not reflect current conditions due to system degradation, PED phases should consider methods to ensure accurate modeling of all future work.

6.3.1 Water Surface Profiles

Water surface profiles and stage frequency (elevation vs. probability) curves were produced for the Annual Exceedance Probability (AEP) events developed by Atlas 14 (0.99 to 0.002 AEPs) for baseline without project (2026) and FWOP (2076). Project conditions evaluations were developed for the same probabilistic events. The following figures show resulting HEC-RAS water surface profile plots.

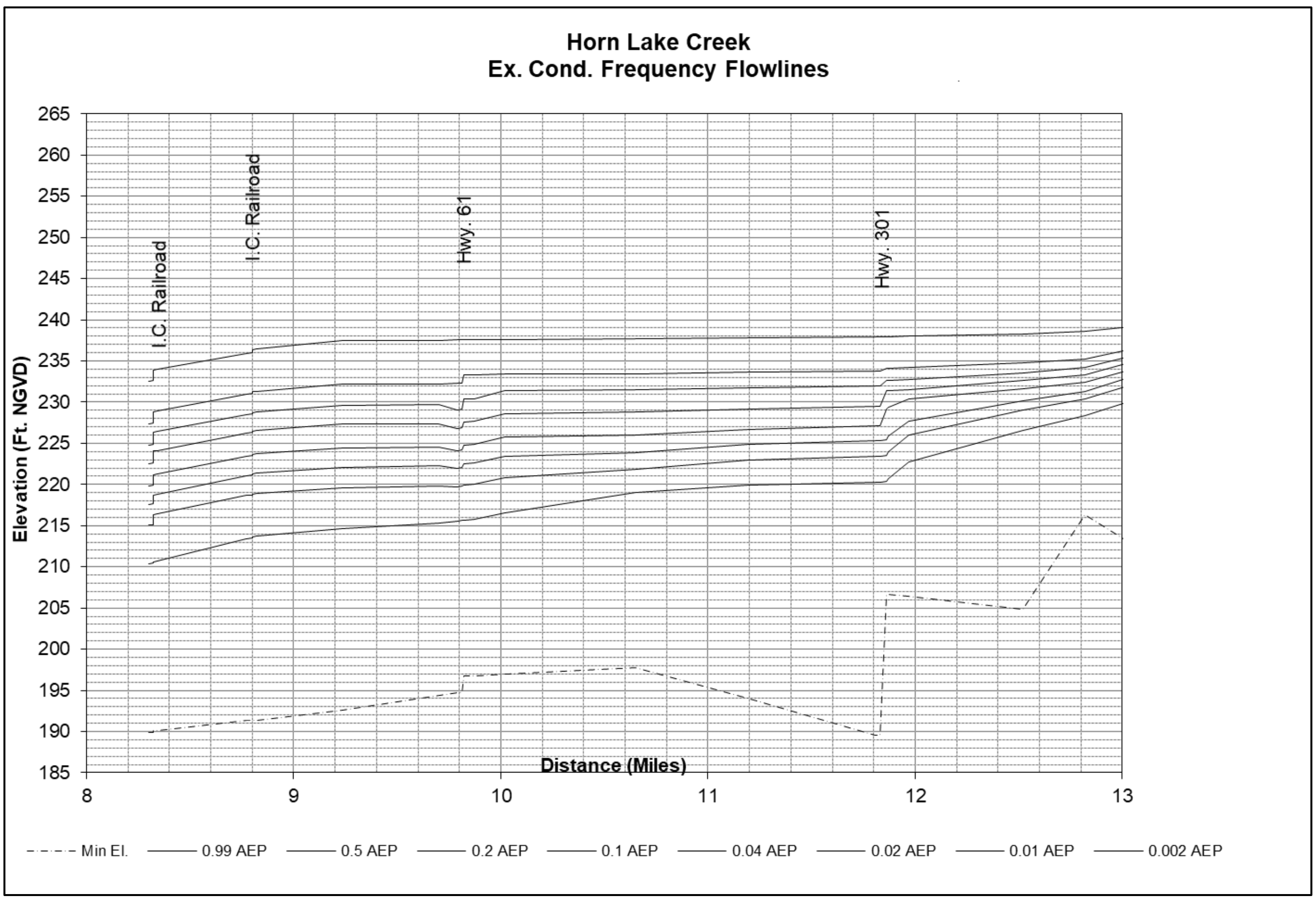


Figure 26. Horn Lake Creek Frequency Flowlines

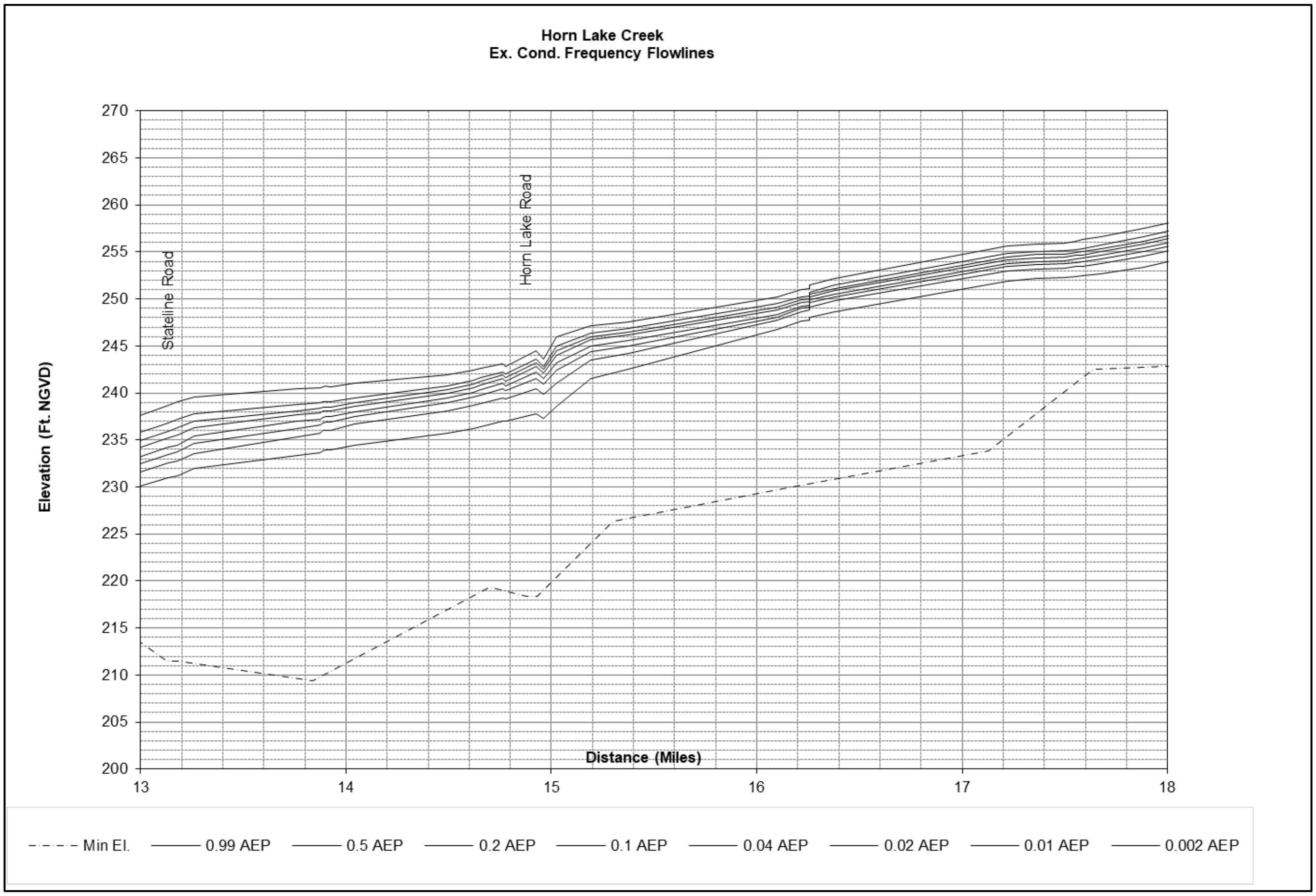


Figure 27. Horn Lake Creek Frequency Flowlines

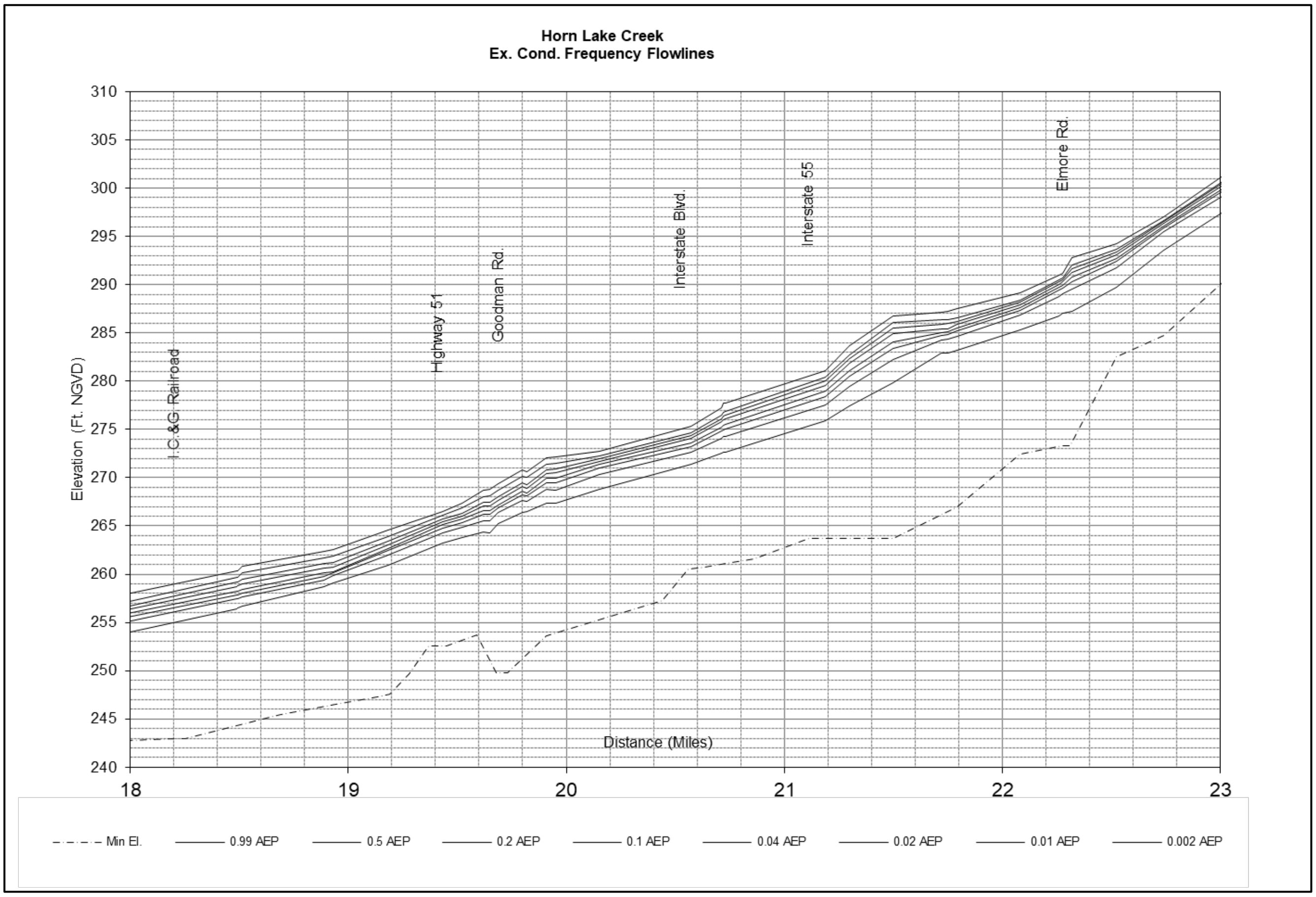


Figure 28. Horn Lake Creek Frequency Flowlines

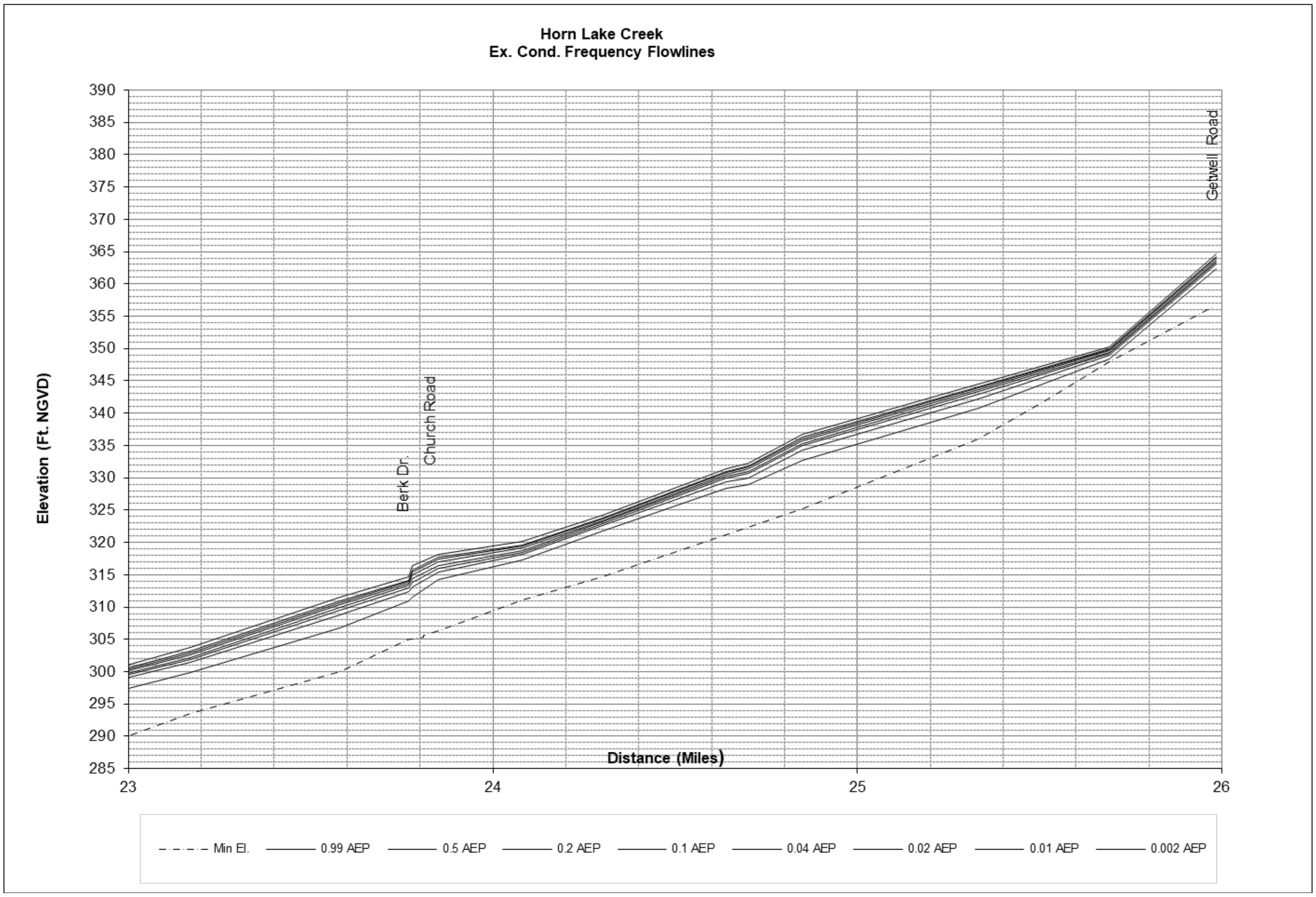


Figure 29. Horn Lake Creek Frequency Flowlines

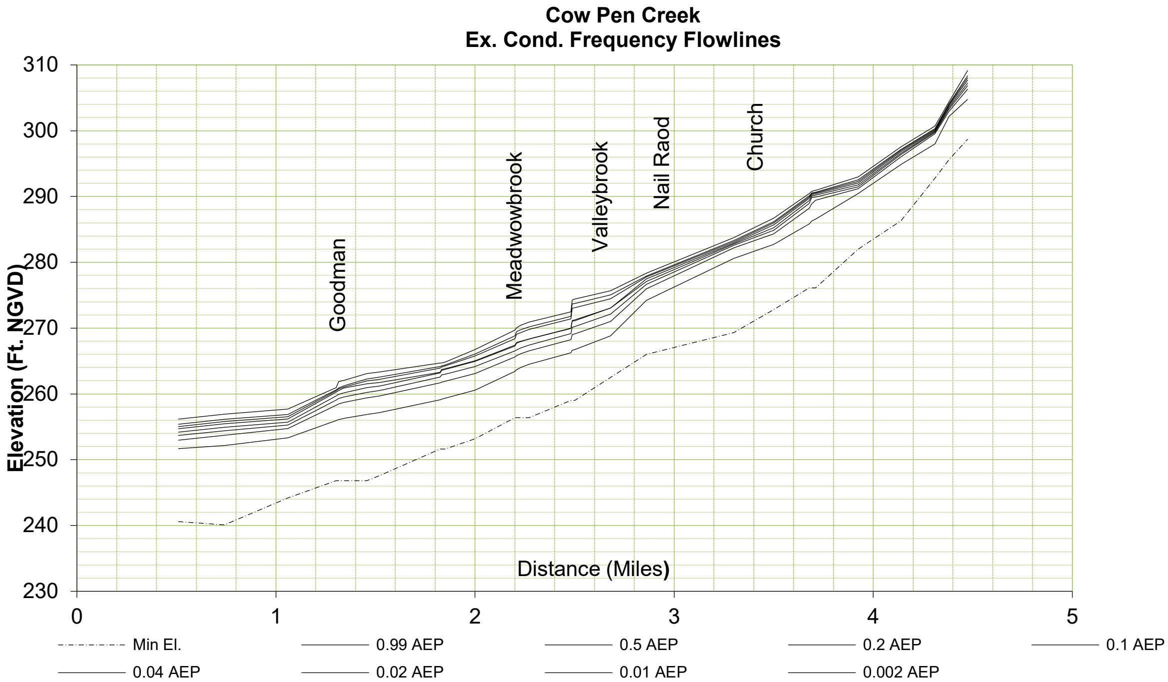


Figure 30. Cow Pen Creek Frequency Flowlines

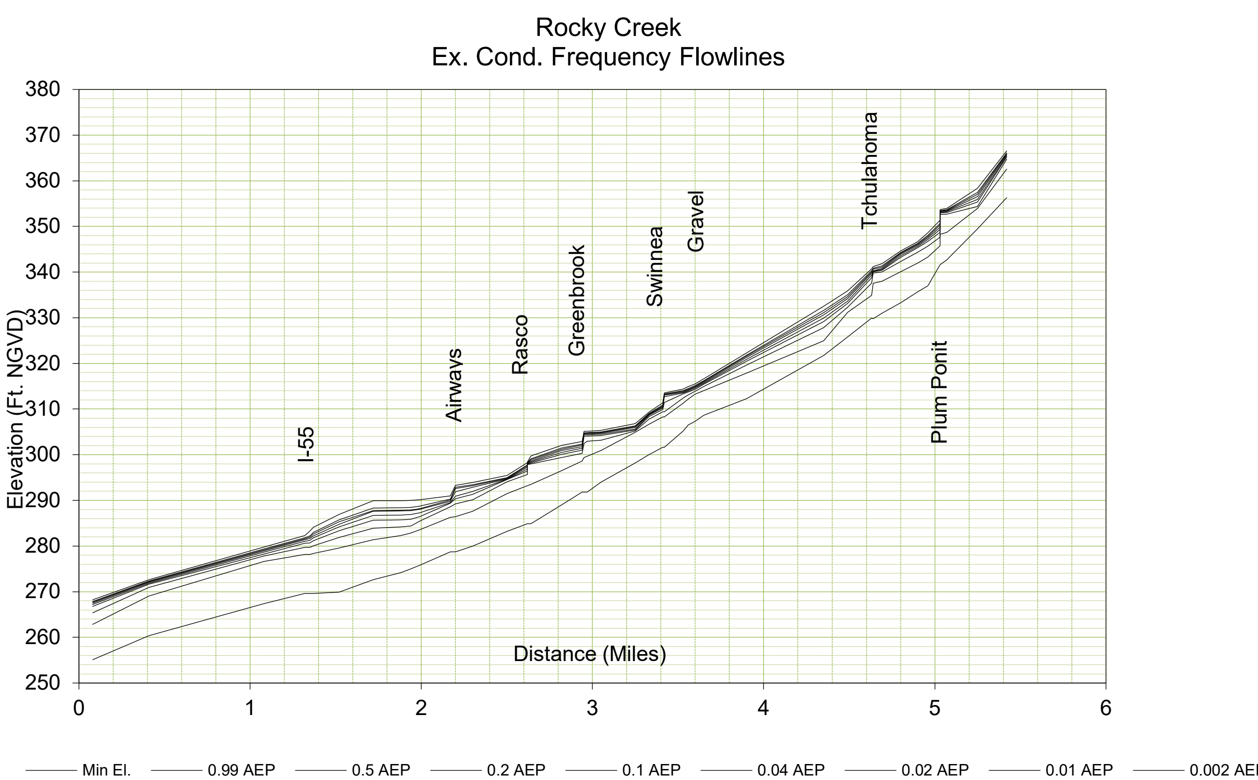


Figure 31. Rock Creek Frequency Flowlines

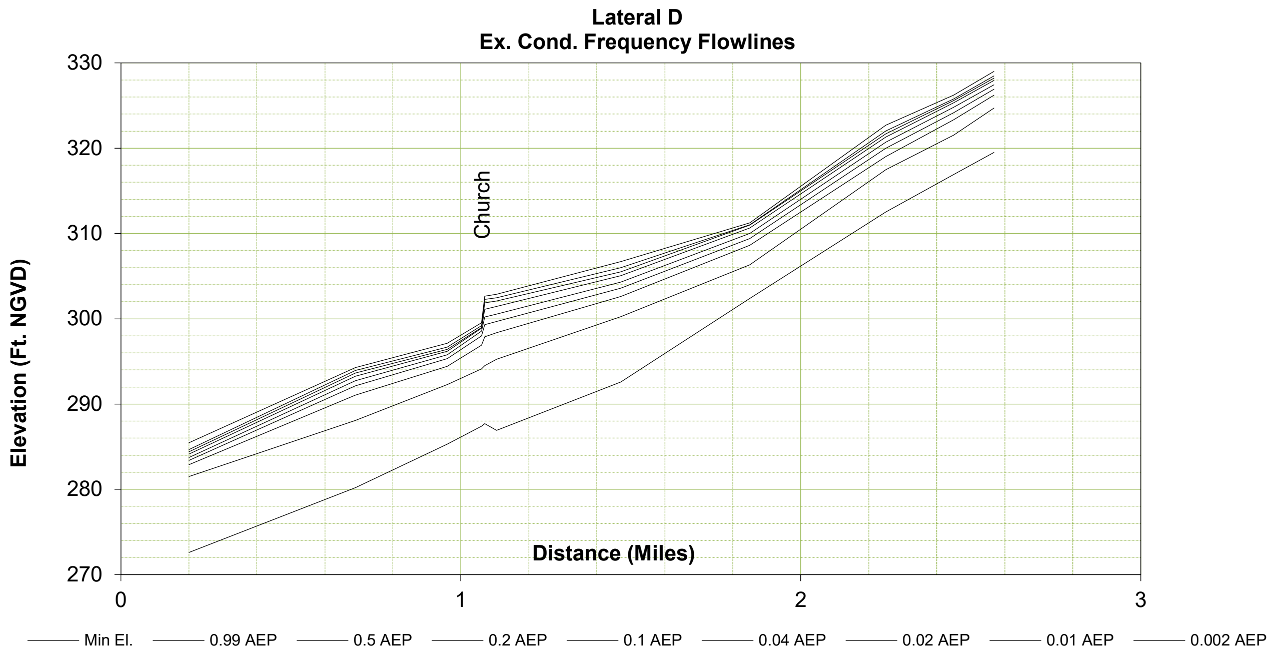


Figure 32. Lateral D Frequency Flowlines

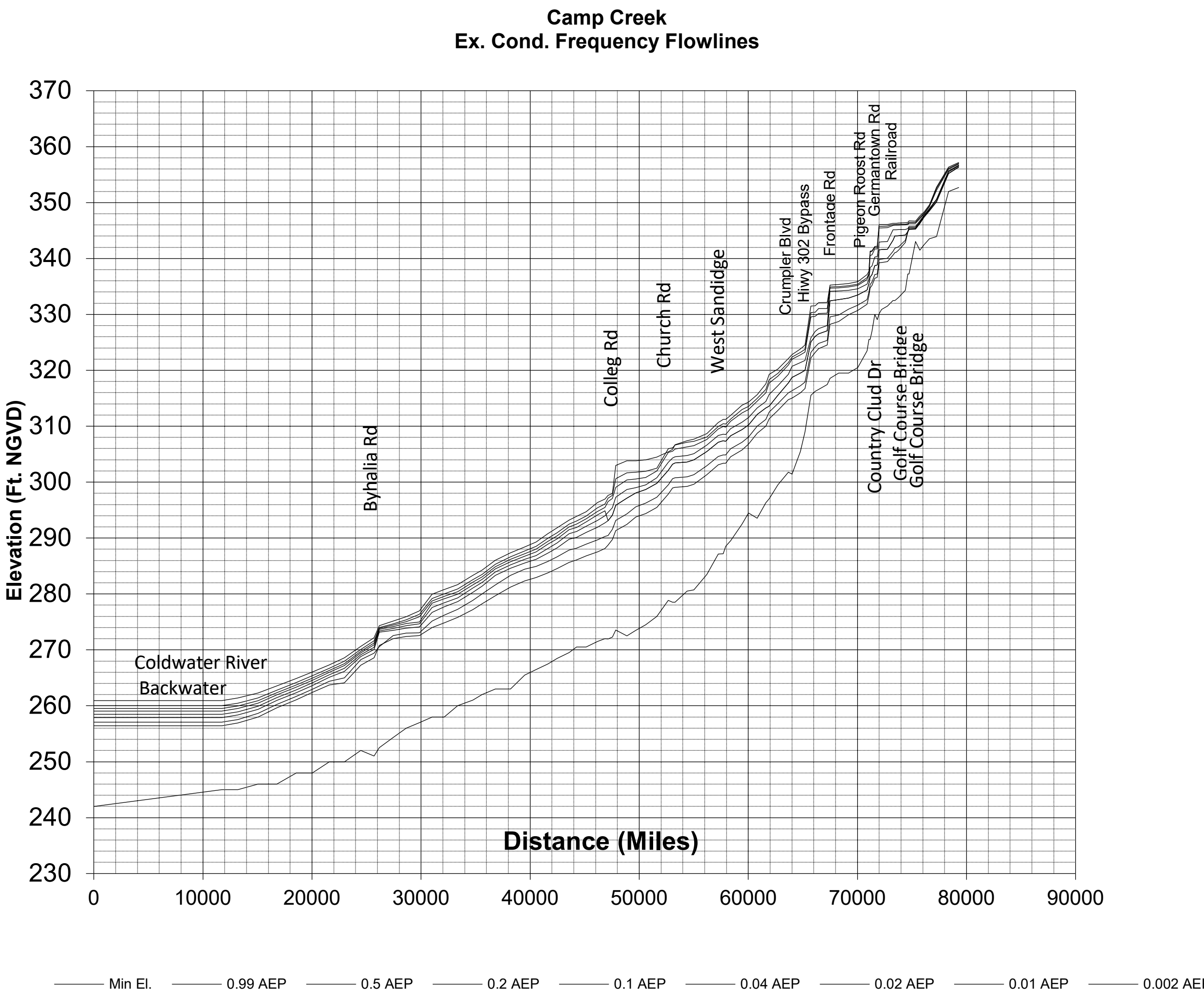


Figure 33. Camp Creek Frequency Flowlines

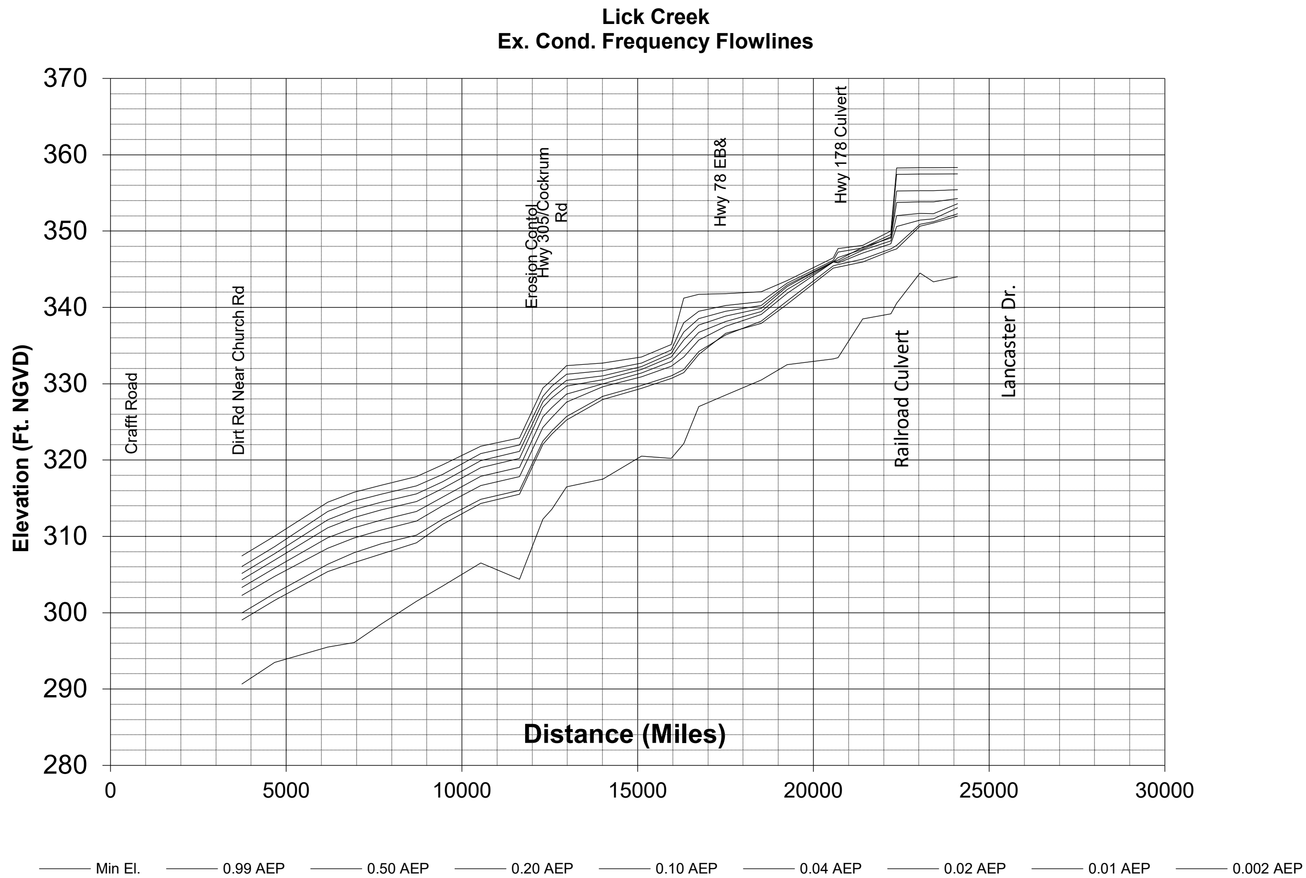


Figure 34. Lick Creek Frequency Flowlines

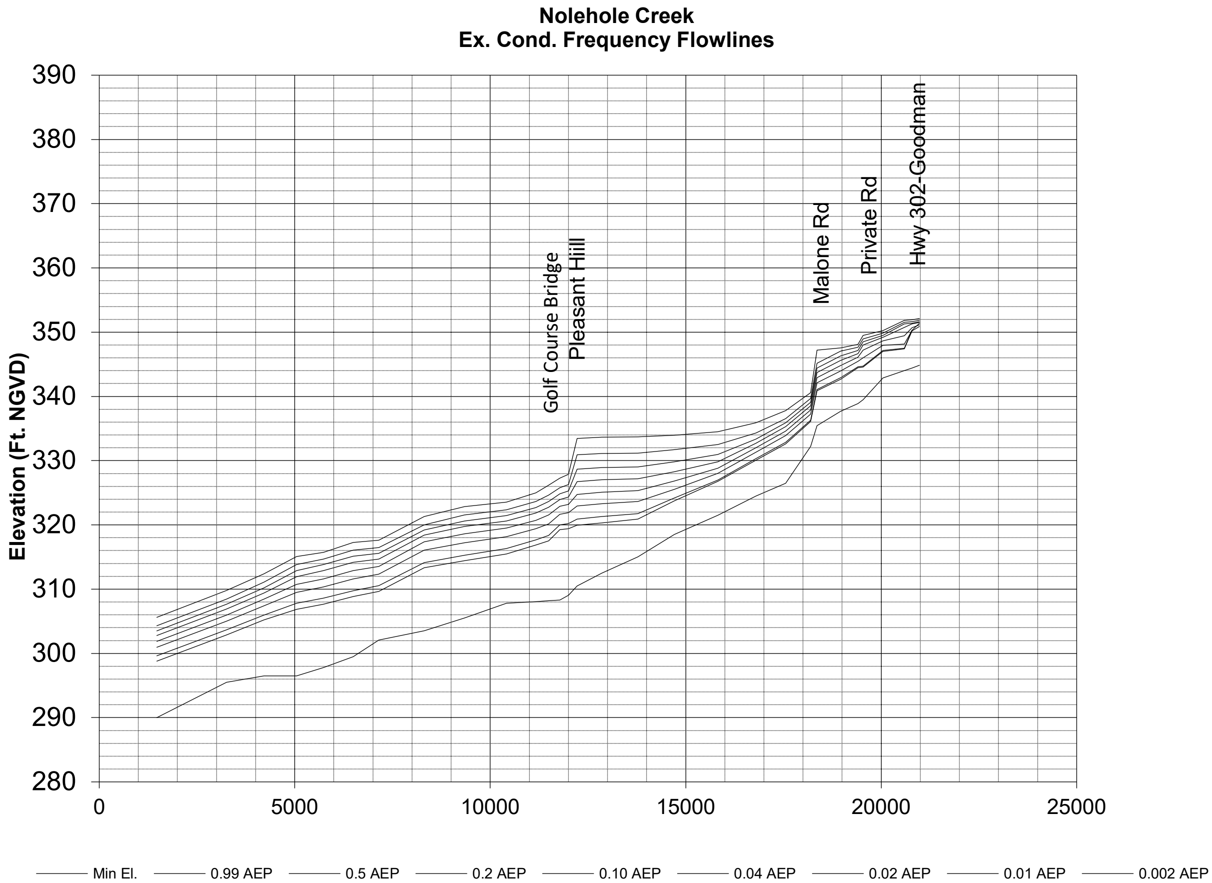


Figure 35. Nolehoe Creek Frequency Flowlines

6.4 HORN LAKE CREEK ALTERNATIVES-HEC-RAS 1D STEADY FLOW

Several alternatives were investigated to address flood risk in the Coldwater River and Horn Lake Creek basins. The primary measures and alternatives investigated included channel enlargement, detention basins, levees, and pump stations. Non-structural (NS) alternatives were also investigated. To assess the alternatives, modifications were made to the HEC-HMS and HEC-RAS models. A list of the various alternatives investigated is shown in Table 7. It should be noted, the study included non-structural features and are also included in the Table 7.

Table 7. Initial Array of Alternatives

Alt ID	Description	Measures Included
NS -25yr	0.04 AEP Nonstructural Aggregation	Elevating Residential and Flood proofing Commercial Structures
NS-50yr	0.02 AEP Nonstructural Aggregation	Elevating Residential and Flood proofing Commercial Structures

Alt ID	Description	Measures Included
NS-100yr	0.01 AEP Nonstructural Aggregation	Elevating Residential and Flood proofing Commercial Structures
6	Basin Wide Bermless Detention	All Detention Combined (alt ID 9-12)
7	2005 Plan	Combination of channel enlargement, diversion, berm and weir, and detention
9	Rocky Creek Detention	Detention Basin on Rocky Creek
10	Horn Lake Creek Detention at Elmore	Upstream detention basin at Elmore Road
11	Lateral D Detention	Detention on Lateral D. near Airways
12	Cow Pen Creek Detention	Detention on Cow Pen Creek near Nail and Hurt Rd.
14	Horn Lake Creek Berm Drainage Ditch Levee	Drainage ditch, small levee blocking water from entering stormwater drainage ditch south of Bullfrog Corner
16	Horn Lake Creek Drainage Ditch Levee and Detention Combo 1	Drainage Ditch Levee, Horn Lake Detention and Rocky Creek Detention
17	Multi Detention with Drainage Ditch Levee Combo 2	Levee+ 4Detention: Bullfrog Levee, HLC detention at Elmore, Rocky Creek Detention, Cow Pen detention, Lat D detention
18	Horn Lake Creek Channel Enlargement	River mile 18.86-19.41
19	Multi Detention without Levee Combo 3	4 Detention only: Horn Lake Detention, Rocky Creek Detention, Cow Pen Creek Detention and Lateral D Detention
20	Three Detention sites	Rocky Creek Detention, Cow Pen Creek Detention and Lateral D Detention
21	Three Detention sites+ Horn Lake Creek Channel Enlargement 18.86-19.41	Rocky Creek Detention, Cow Pen Creek Detention and Lateral D Detention+ HLC Channel Enlargement with Rip Rap
22	Extended Horn Lake Creek Channel Enlargement	Extended Channel Enlargement with Rip Rap (18.60-19.41)
23	Horn Lake Creek Channel Enlargement +Lateral D detention	Extended HLC Channel Enlargement +Lateral D Detention (Plan 11+22)
24	Extended Horn Lake Channel Enlargement with Cow Pen Detention	Extended HLC Channel Enlargement +Cow Pen Detention (Plan 12+22)
25	Extended Horn Lake Channel Enlargement with Rocky Detention	Extended HLC Channel Enlargement +Rocky Creek Detention (Plan 9+22)
26	Extended Horn Lake Channel Enlargement with 2 detention basins	Extended HLC Channel Enlargement +Cow Pen Detention + Lateral D Detention (Plan 11+12+22)

Based on the measures and alternatives examined during the HEC-RAS 1D analysis, channel enlargement and detention basins resulted in the most feasible and economical plans in the Horn Lake Creek basin. Structural flood risk reduction measures in the Coldwater River Basin were not economically justified but non-structural alternatives were examined. Information related to non-structural alternatives are shown in the Economic Section of the Report, Appendix L. The primary streams in which flood risk reduction measures were investigated are shown in Figure 36.

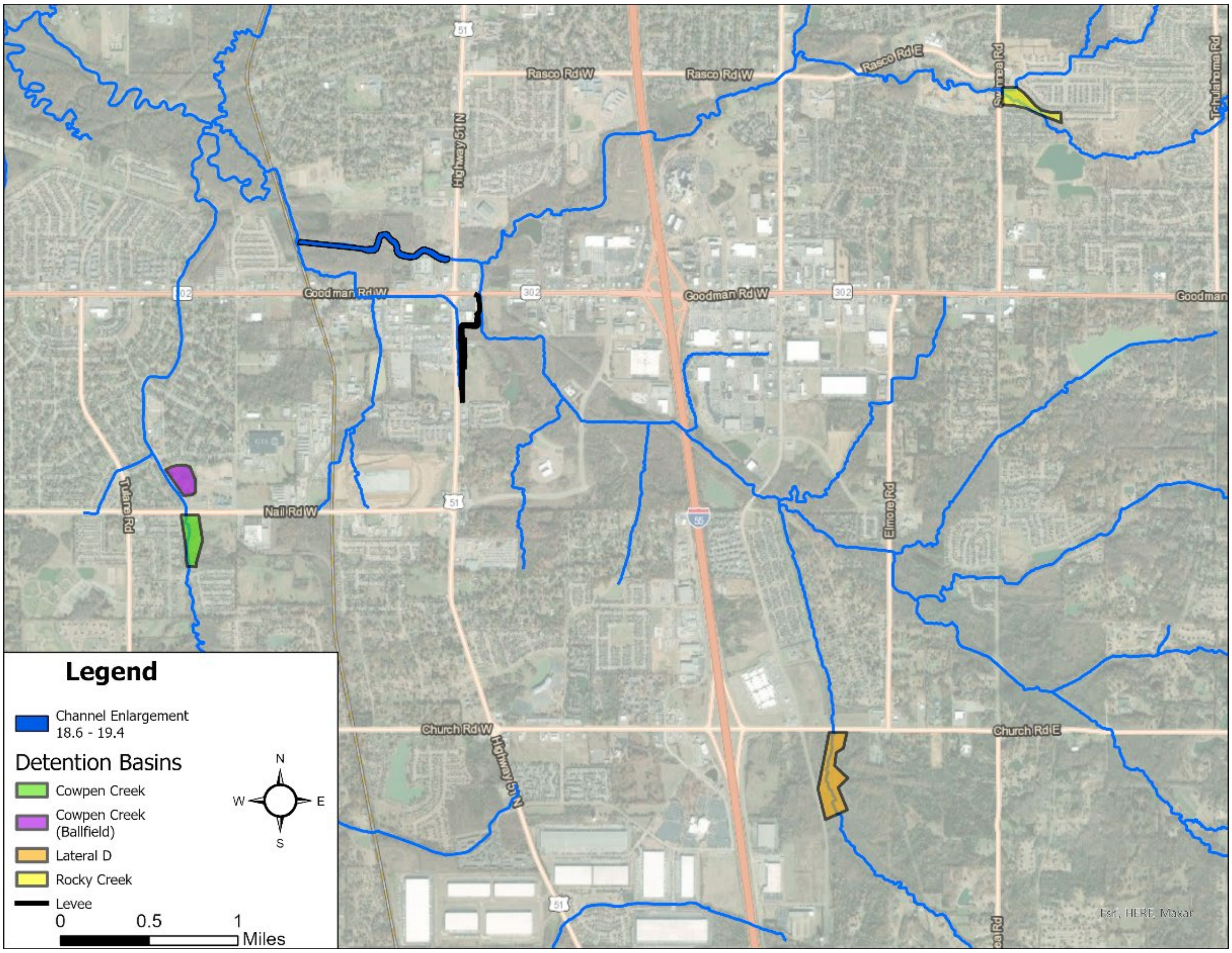


Figure 36. Flood Risk Management Alternative Sites

6.4.1 Horn Lake Creek Channel Enlargement (Alt. ID 18)

The Horn Lake Creek HEC-RAS model was modified to assess the channel enlargement measure. Horn Lake Creek is currently experiencing bed degradation and channel widening. Due to this instability, the design attempted to increase conveyance but disturb the existing channel as little as possible. Channel deepening will be avoided. Channel enlargement on Horn Lake Creek will consist of a 40-foot bottom width with 1V on 3H channel side slopes and designed to the existing thalweg to avoid channel deepening. It should be noted the Modified Puls Routing relationship in HEC-HMS was altered to reflect the increase channel area and conveyance. Manning's roughness values for the channel were reduced to 77% of the original roughness from 0.045 to 0.035 to account for smoother channel bottom and sides after excavation activities. The downstream impacts of the channel enlargement are presented section 6.4.3.

The enlargement will start at River Station 18.6 and extend to 19.4. Currently, both banks of the improved channel are based on a 1V to 3H slope for stability but one-sided enlargement is the desired plan and will be finalized in the detailed analysis. A riprap "blanket" is needed on the lower channel to prevent erosion. The riprap blanket will be placed in a three-foot deep layer on the bottom and extend 5 feet up both banks. The upper banks will be protected with turf reinforcing mat. Figure 37 shows a plan view of the enlargement reach.

Figure 37. Horn Lake Creek Channel Enlargement

6.4.2 Horn Lake Creek Enlargement-Frequency vs. Elevation Curves

Frequency/probability water surface elevations were also developed for all measures investigated as needed for economic analysis. Rather than providing water surface profile comparisons, it was felt “stage/elevation-probability curves, would present the information more clearly. These comparisons are shown in Figures 38 to 40.

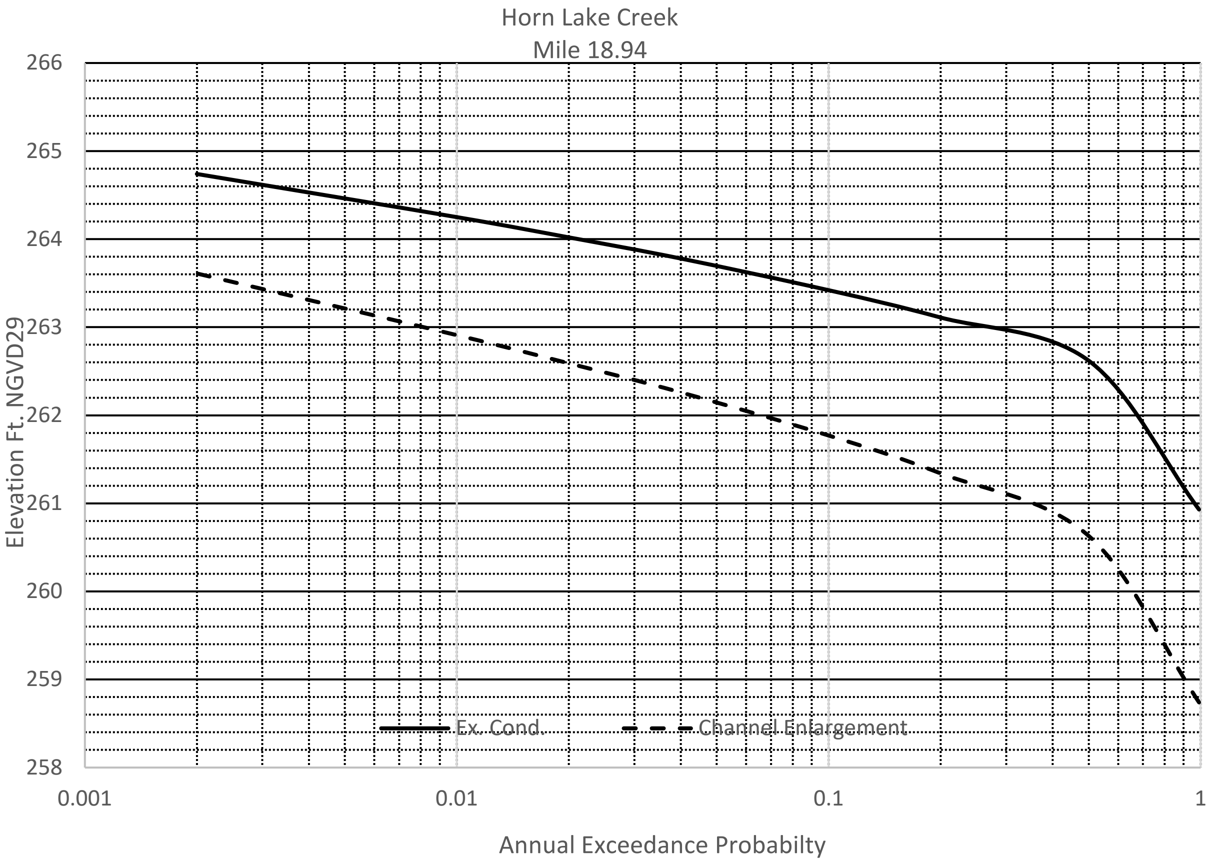


Figure 38. Water Surface Elev. vs. Probability Curve

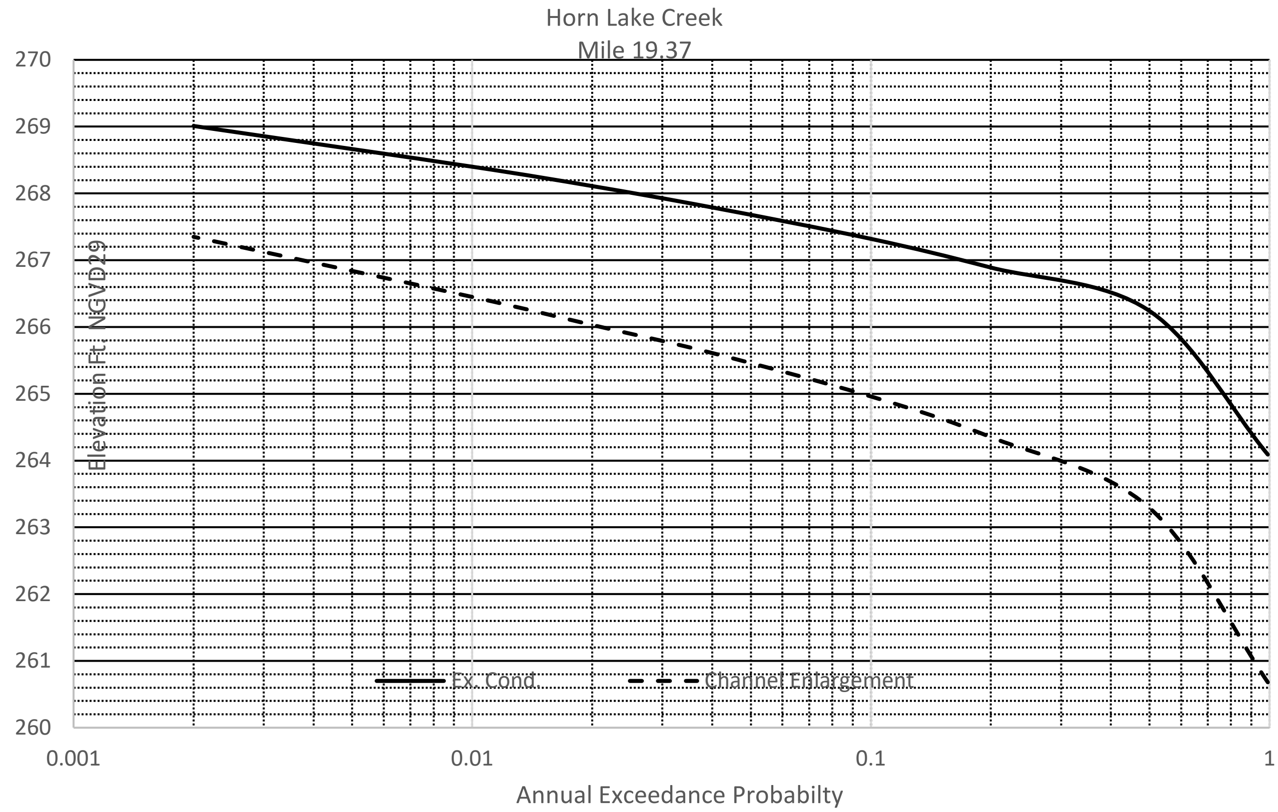


Figure 39. Water Surface Elev. Vs. Probability Curve

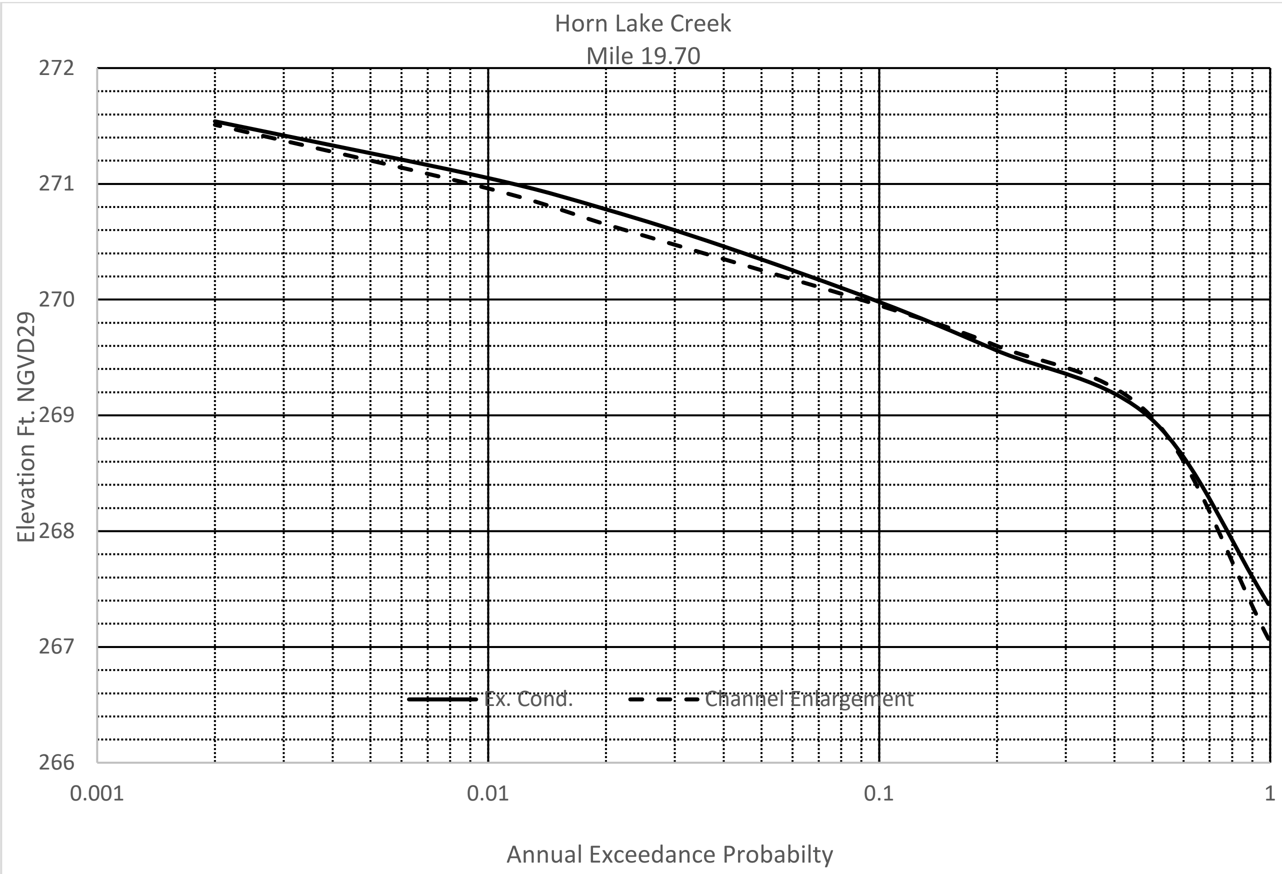


Figure 40. Water Surface Elev. Vs. Probability Curve

6.4.3 Horn Lake Creek Enlargement Downstream Impacts

The channel enlargement reach extends from Mile 18.6 to 19.4. Peak discharges downstream of the enlargement reach will increase, due the improved conveyance and increased capacity. The HEC-HMS routing information was altered to reflect the improved conditions and the resulting increased peak flows were input into the HEC-RAS model. The relative impacts to existing conditions are shown in Figure 38. The Original TSP is composed of the subject channel enlargement combined with a detention basin on Lateral D at Church Road. It should be noted the Original TSP did not eliminate the downstream impacts but reduced the average increase in frequency flood elevations from an average of 0.2 feet to 0.1 feet. The locally preferred plan (LPP) did not induce damages. Downstream induced damages, resulting from the Horn Lake Creek enlargement alternative, are mitigated as a project cost and are explained in the Economic Appendix.

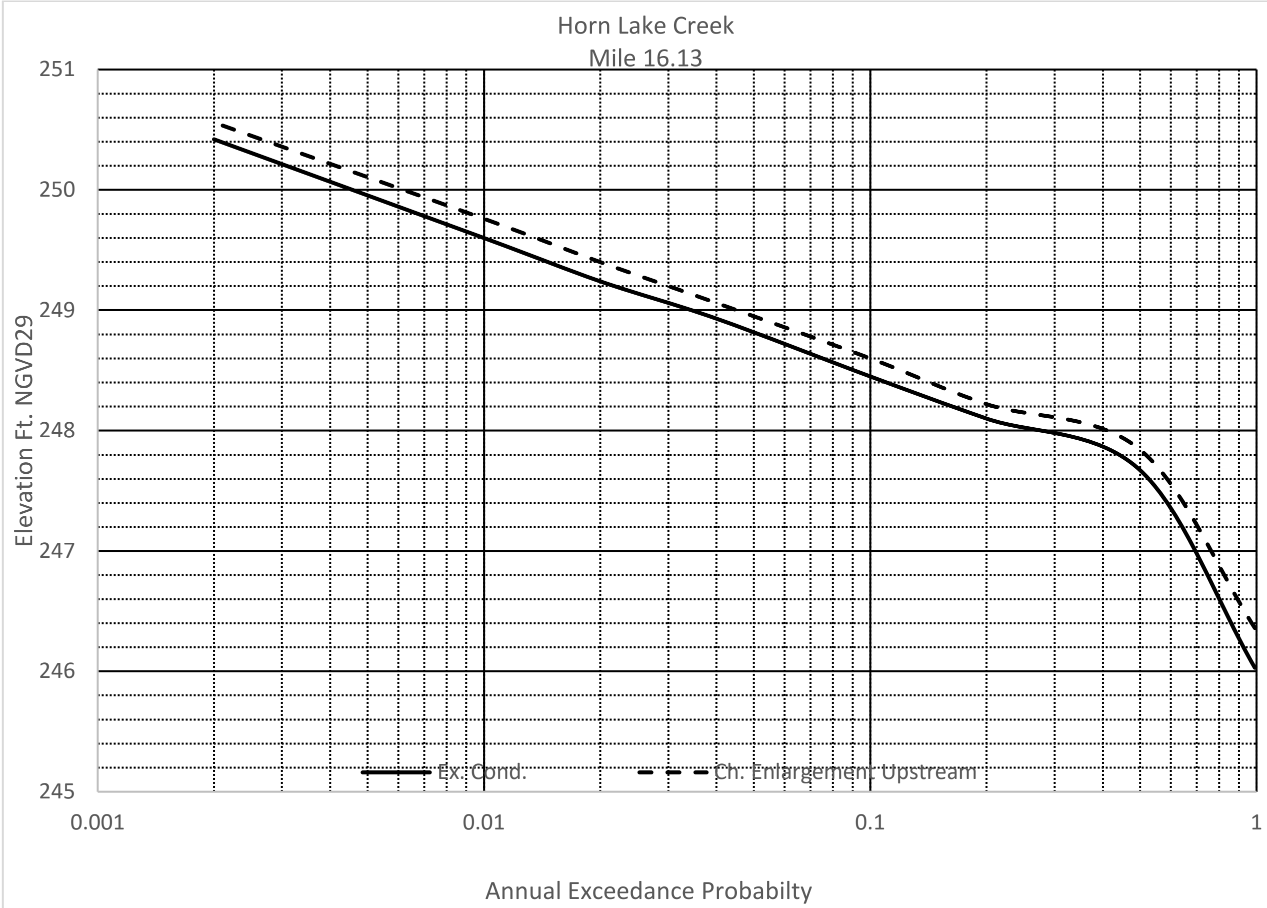


Figure 41. Water Surface Elev. Vs. Probability Curve

6.4.4 Horn Lake Creek Stream Stability

The enlargement reach is relatively short and is protected with riprap to prevent erosion of the channel bottom and lower channel slopes. Since any form of channel enlargement constitutes a change in regime, it is logical to assume Horn Creek will adjust itself after completion of the project.

Horn Lake is bedded in a silty clay soil with several large trees located within the channel. Typically, there are little or no grasses covering the bed and banks, but the channel remains essentially stable. Field reconnaissance indicated that, historically headcutting may have occurred. The channel currently experiences localized erosion, primarily resulting from the significant quantities of debris or artificial hard points that were observed. Recent field reconnaissance conducted by the US Corps Engineering Research Development Center (ERDC) and documented in Appendix C states the riprap protecting various crossings has aided in maintaining some stability.

The short duration and flashy nature of Horn Lake Creek has been considered in the determination of protection requirements. If the design flows were longer in duration, the level of bank protection would be increased accordingly. Horn Lake Creek is undergoing continuous changes in hydrology due to increased urbanization and construction activities within the basin and will remain in a state of flux with or without this channel enlargement project. Without the proposed project, localized problems will continue in the vicinity of bridges and bendways.

With the proposed improvements, velocities will be increased within the riprapped reach, but HEC-RAS 1D modeling results indicate they transition back to existing conditions upstream of the railroad. The project will not provide a channel system that is totally stable, but the overall stability of the project should be enhanced by the incorporated protection measures and the relatively short enlargement reach should not increase the current overall stream instability.

6.5 DETENTION ANALYSIS-INFLOW DESIGN FLOODS

The available acres and sites, where detention basins could be constructed, were provided by the sponsor. This determined the streams analyzed for potential detention sites. Depths of the detention pond(s) and bottom elevations are based on the approximate elevations of the outlet ditches and/or adjacent detention ponds. Basin storage will be provided primarily by excavation and berms/embankments construction will be kept to a minimum. The structures will be designed as dry ponds and built to not hold a permanent pool.

According to Corps guidance, a dam is defined as a barrier usually built across a stream that obstructs, directs, retards, or stores water that exceeds 15 acre-feet in volume or has an embankment height that exceeds 6 feet. Based on this directive, ER-1110-8-2(FR), entitled "Inflow Design Flood for Dams and Reservoirs," will be used to ensure the design adheres to current standards and guidance.

Since the structures are in an urban area, the selection of Inflow Design Floods (IDFs) and the design of dam/structure elements will conform to Corps of Engineers Safety Standard No. 1, which is applicable to a high hazard flood retarding structure. Any detention pond which is determined to economically be justified, would be evaluated and subject to the following specific design criteria:

Inflow design flood (IDF) computed using the Probable Maximum Flood.

Inflow unit hydrograph peaked 25 to 50 percent.

Runoff ratio should be 90 percent or higher.

Minimum starting water surface elevation for routing the IDF will either be the full flood control pool (100 year) or an elevation prevailing five days after a storm that produced one-half the IDF.

Regulating outlets assumed to be inoperable.

Freeboard above the maximum IDF elevation is based on either a minimum of three feet or five feet if the IDF pool hydrograph is within three feet of the maximum pool for 36 hours of longer.

The above criteria will be assessed using the unsteady HEC-RAS 1D/2D flow model to establish final embankment heights, simulate breaches and assess life safety concerns. A dry reservoir is less desirable to the sponsor but since it will not have a permanent pool, the downstream risk will be reduced. A dry detention pond will also have a higher probability of satisfying the Corps of Engineers risk requirements and perhaps the inflow/spillway design flood standard criteria can be minimized.

6.5.1 Cow Pen Creek Basin Detention Analysis (Alt. ID 12)

Cow Pen Creek Detention South

As stated in Section 6.1, HEC-HMS was used to model detention ponds. A 12-acre inline detention basin will be located on Cow Pen, south of Nail Road (River Station 2.5). The dry detention basin will have a bottom elevation of 262.0, bottom surface area of 10 acres, and the pond banks will be sloped back up to grade at 1V to 3H. The locations of detention basins are shown in Figure 42.

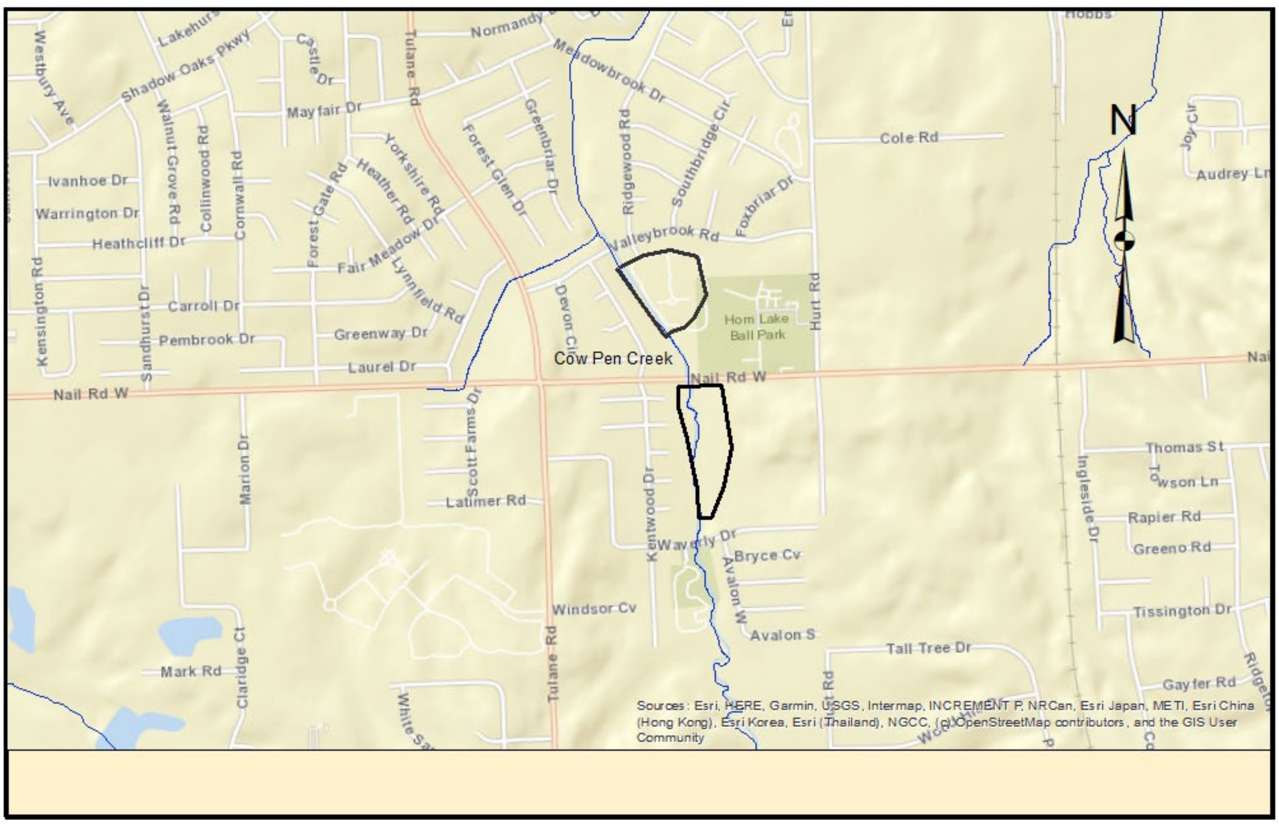


Figure 42. Cow Pen Creek Detention Basins

To compute the effects of the detention ponds, a volume vs. elevation relationship was developed for the detention pond. Areas and volumes above the detention ponds were computed using ArcMap and 2010 LiDAR. The two volumes were combined, and the relationships were input into HEC-HMS and the reduced outlet peak flows were computed and input into HEC-RAS. The final storage relationship is shown below.

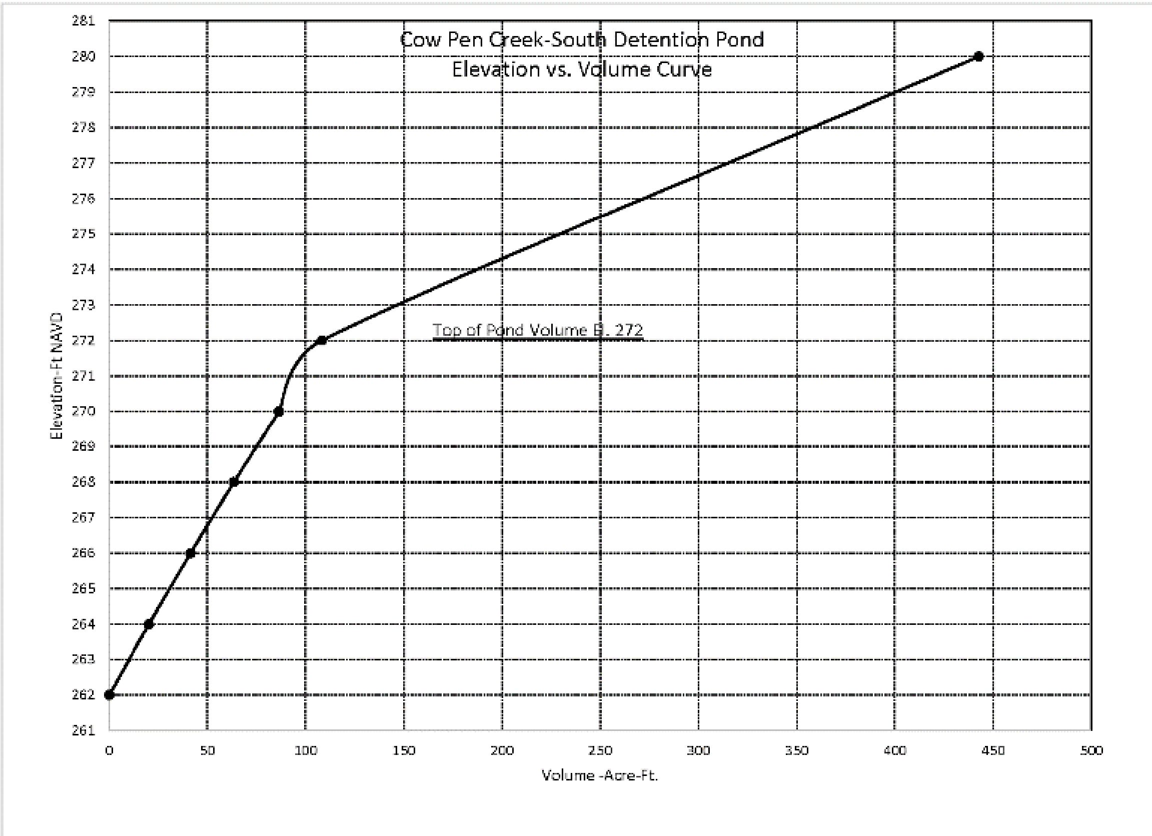


Figure 43. Cow Pen Creek South-Volume Curve

A 500-foot-long outlet embankment will include a 48 in. reinforced concrete pipe outlet and 100-foot-wide overflow spillway. The maximum storage of 108 acre-feet requires approx. 175,000 cubic yards (CY) of excavation. The basin has the approximate capacity to contain the 0.99 AEP event.

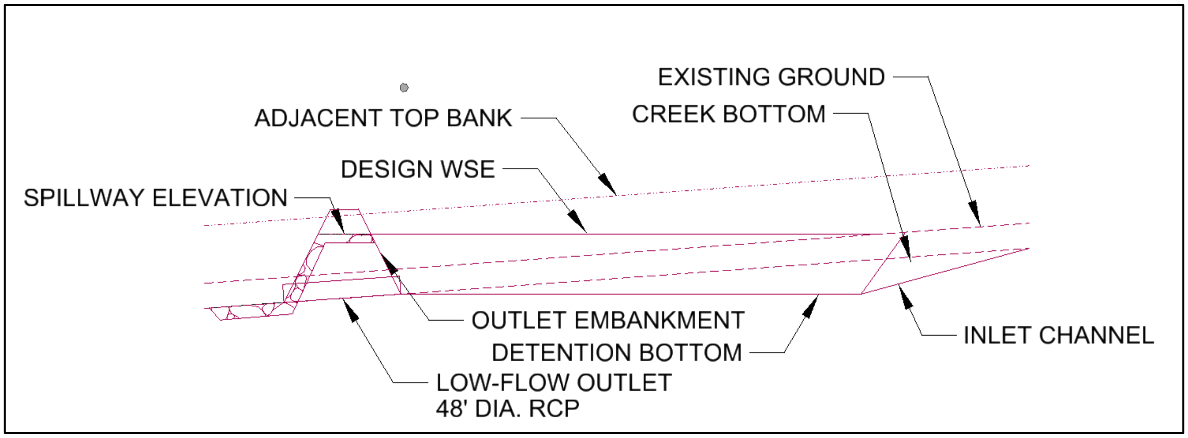


Figure 44. Typical Detention Basin

The 100-foot overflow spillway will be set at the maximum top of pond elevation of 272.0 Ft. NAVD. Topography will dictate the final configuration, but the spillway will be grouted and designed for concentrated flows. To examine potential overflow velocities, the 0.02 AEP was routed through the basin. The overflow velocities at the crest were estimated to be 6.2 fps for a depth of 5 feet. The recommended gradation is R200 max riprap placed in a minimum 24-inch blanket. Riprap design is based on the Isbach Equation and guidance shown in the MVD report entitled “MVD Report on Standardization of Riprap Gradations” dated January 1982.

The basins capacities are relatively small, and overtopping will be frequent. Both the height, length and other spillway parameters design will be optimized during feasibility-level design and final inflow design determination.

Cow Pen Creek Detention North

An 8-acre offline detention basin will be located adjacent to Cowpen Creek north of Nail Road in Horn Lake, MS. The dry detention basin will have a bottom elevation of 258.0, bottom area of 6 acres, and shall be sloped back up to grade at 3H to 1V. A 500-foot-long outlet embankment will include a 48 in. reinforced concrete pipe outlet and 100-foot-wide overflow spillway armored with riprap on the downstream side. The 100-foot-wide spillway will operate at elevation 268.0 which is approximately the 0.50 AEP event. The maximum storage of 68 acre-feet requires approx. 115,000 cubic yards of excavation.

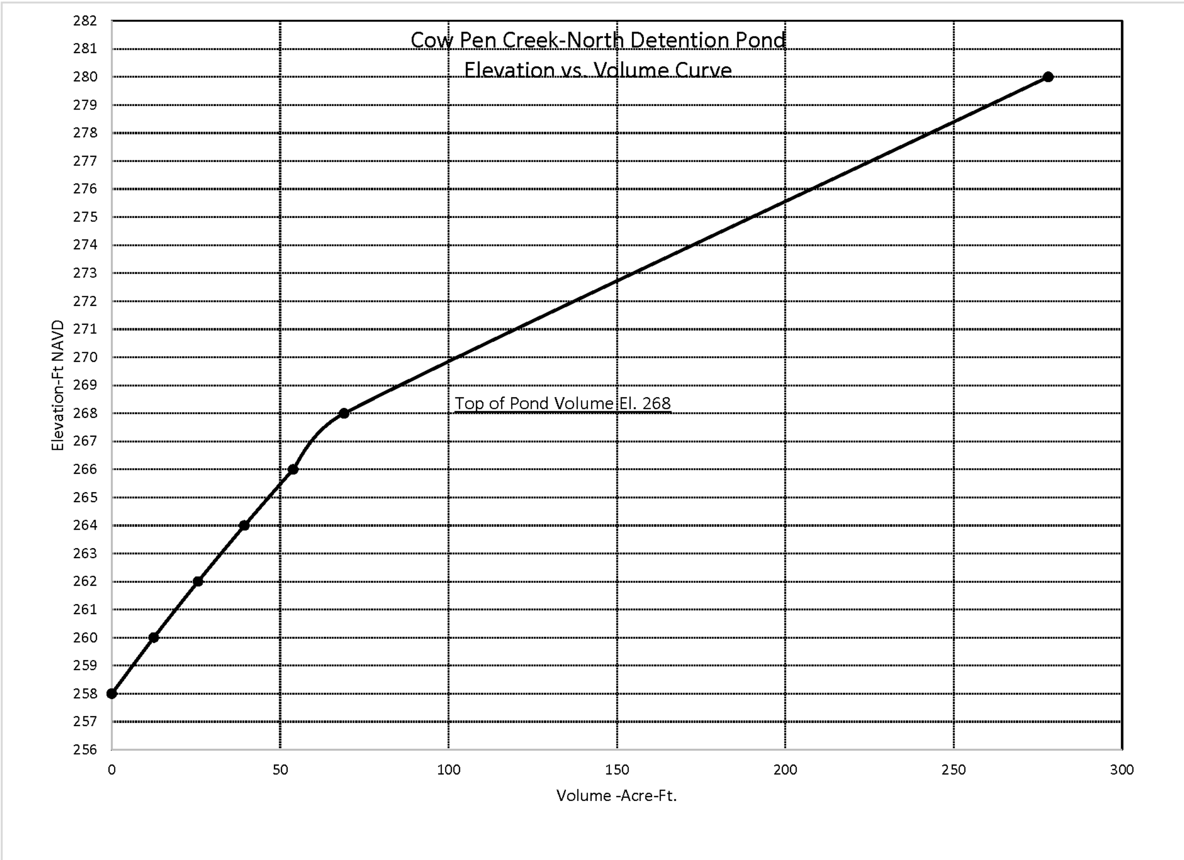


Figure 45. Cow Pen Creek North-Volume Curve

The 100-foot spillway will be set at the maximum top of pond elevation of 268.0 Ft. NAVD. Topography will dictate the final configuration, but the overflow will be designed to occur at the downstream portion of the basin. To examine potential overflow velocities, the 0.02 AEP was routed through the basin. The overflow velocities at the crest were estimated to be 7.1 fps for an approximate depth of 6 feet. It should be noted the height, length and other spillway parameters will be optimized during feasibility-level design.

6.5.2 Cow Pen Creek Frequency vs. Elevation Curves

The reduction in flows is primarily a benefit to Cow Pen Creek. The HEC-HMS flows were adjusted for the entire Horn Lake Basin and the corresponding water surfaces on the Cow Pen Creek were computed. Comparisons of the results are shown below in Figures 46 and 47.

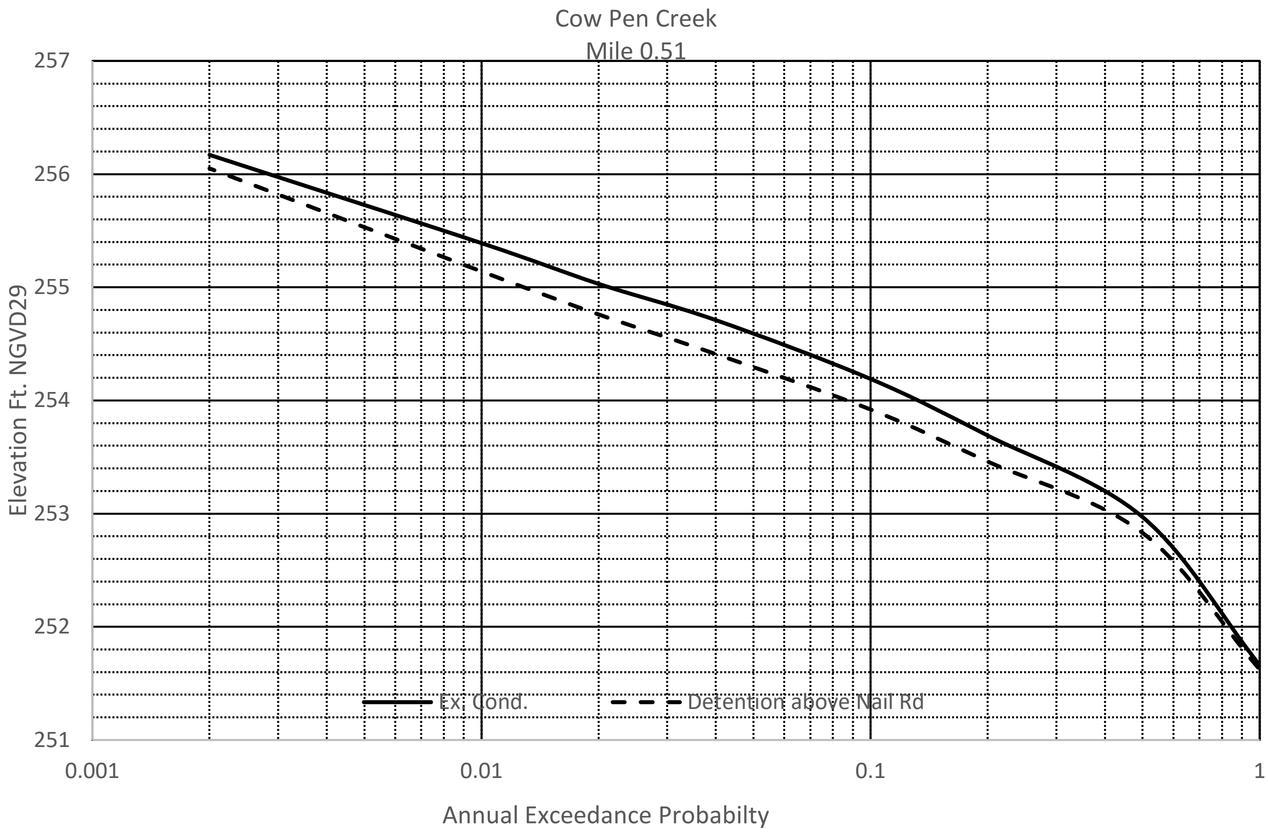


Figure 46. Water Surface Elev. Vs. Probability Curve

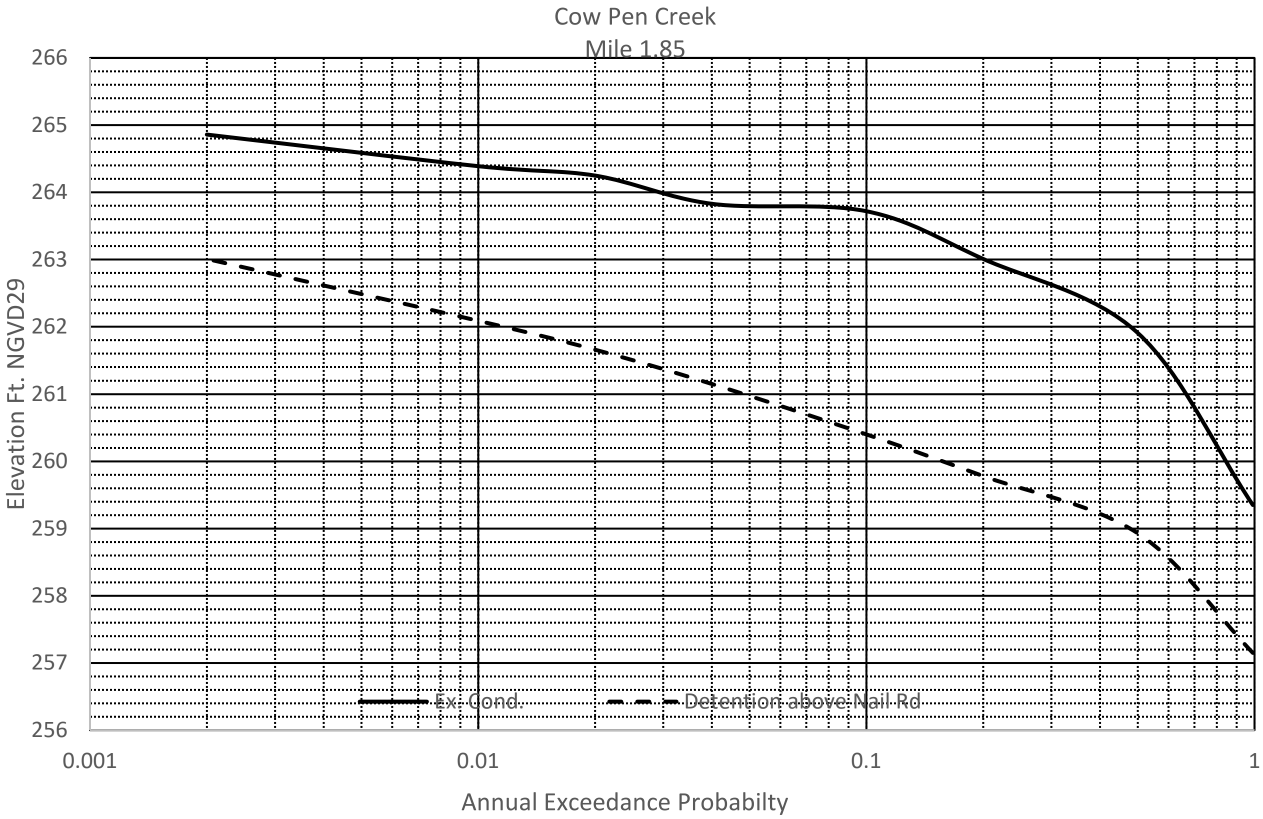


Figure 47. Water Surface Elev. Vs. Probability Curve

6.5.3 Rocky Creek Basin Detention Analysis (Alt. ID 9)

A nine-acre inline detention basin will be located on Rocky Creek (River Station 3.42) east of Swinnea Road in Southaven, MS. The dry detention basin will have a single pool elevation 302.0 ft. The pool bottom area is six acres. All slopes back up to grade shall be 3H to 1V. The site is shown in Figure 48.

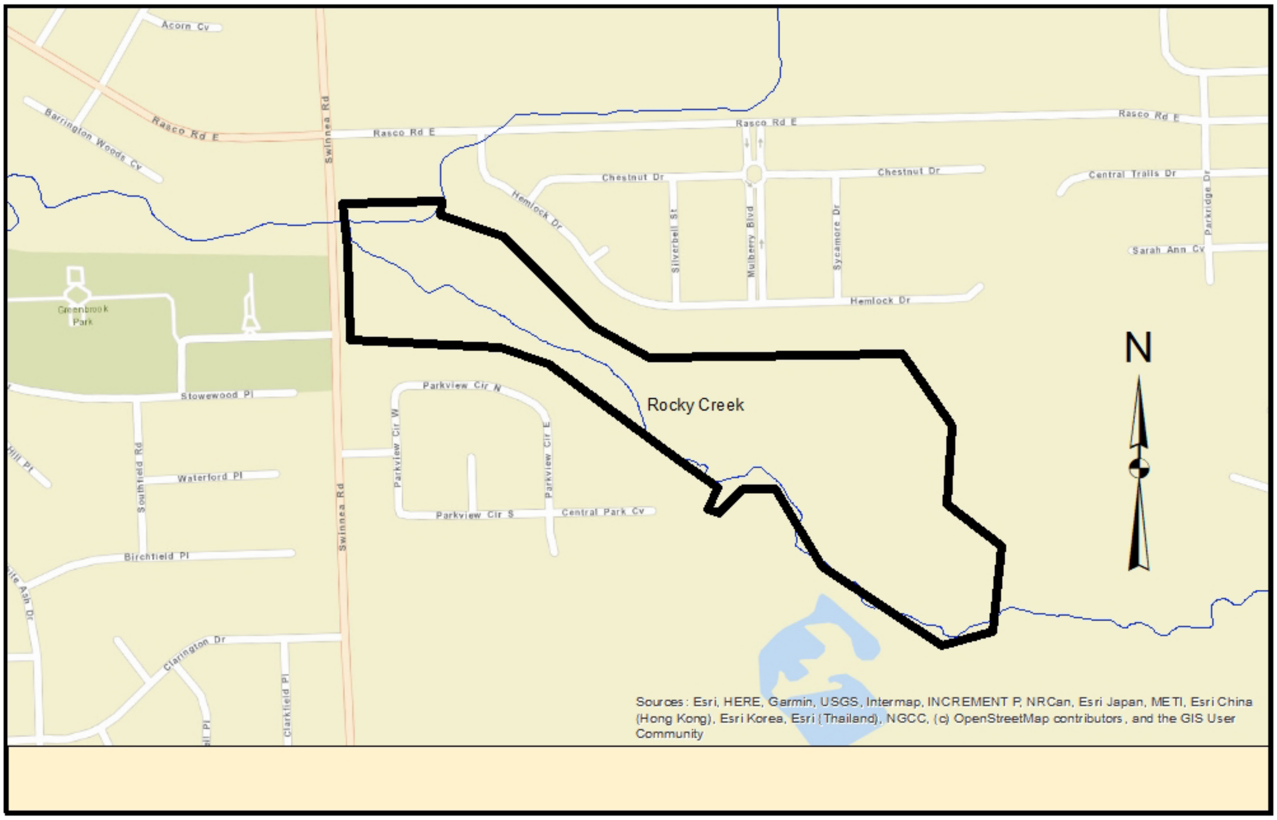


Figure 48. Rocky Creek Detention Basin

Downstream embankment is 500 linear feet and will include a 48 in. reinforced concrete pipe outlet and 100-foot-wide overflow spillway armored with riprap on the downstream side the 100-foot-wide spillway will operate at elevation 312.0 ft at the 0.50 AEP event. The maximum storage of the detention pond is 72 acre-ft. The total storage curve used in HEC-HMS is shown in Figure 49.

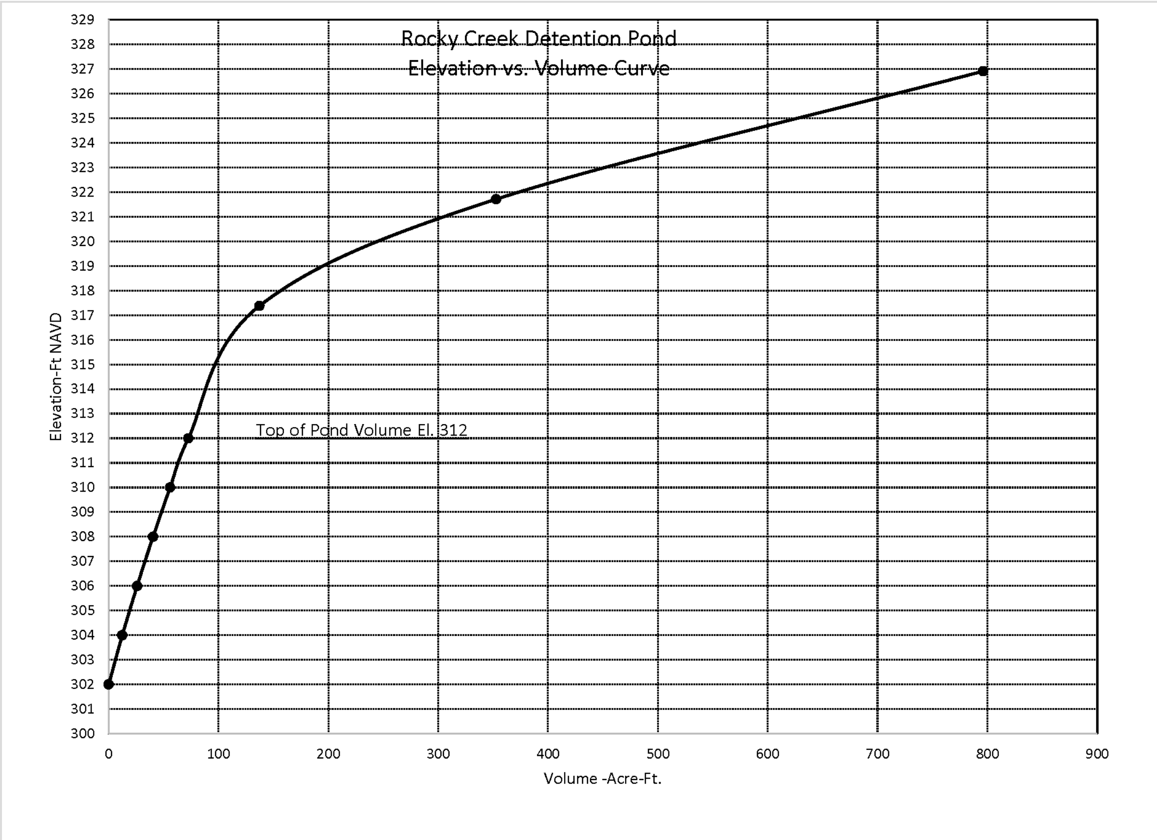


Figure 49. Rocky Creek-Volume Curve

The original detention design of Rocky Creek consisted of 4 smaller ponds constructed in sequence. During the latter phases of the study, several utilities were identified that altered the pond design. This resulted in the abandonment of multiple basins and the construction of one larger basin is recommended for final design.

Original overflow velocity estimates for Rocky Creek detention pond were based on 4 detention ponds in-line and are no longer valid. Riprap quantities and gradations used for Rocky Creek were based on Cow Pen Creek estimates. It is assumed structural design will be similar (i.e., 100 feet spillway crest, 4 to 5 feet crest depths). Final heights, lengths and other spillway parameters will be optimized during feasibility-level design.

6.5.4 Lateral D Detention Basin (Alt. ID 11)

A 22-acre inline detention basin will be located on Lateral D (River Station 1.06), south of Church Road. The dry detention basin will have a bottom elevation of 290 ft, bottom area of 16 acres, and shall be sloped back up to grade at 3H to 1V. The site is shown in Figure 50.

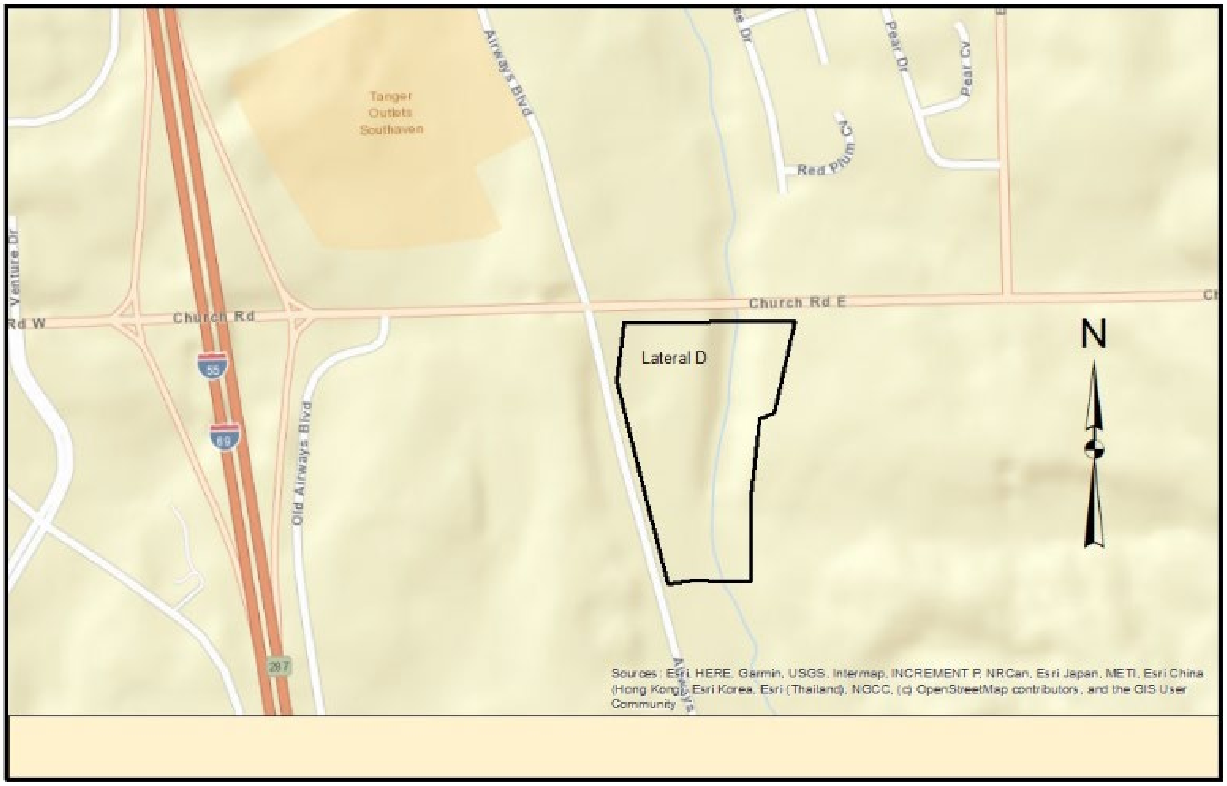


Figure 50. Lateral D Detention Basin

A 500-foot-long outlet embankment will include a 48 in. reinforced concrete pipe outlet and 100-foot-wide overflow spillway armored with approx. 2,000 tons riprap on the downstream side. The 100-foot-wide spillway will operate at elevation 300.0 ft, at the 0.50 AEP event. The maximum storage of 177 acre-feet requires approx. 350,000 CY of excavation. Figure 51 shows the total volume curve.

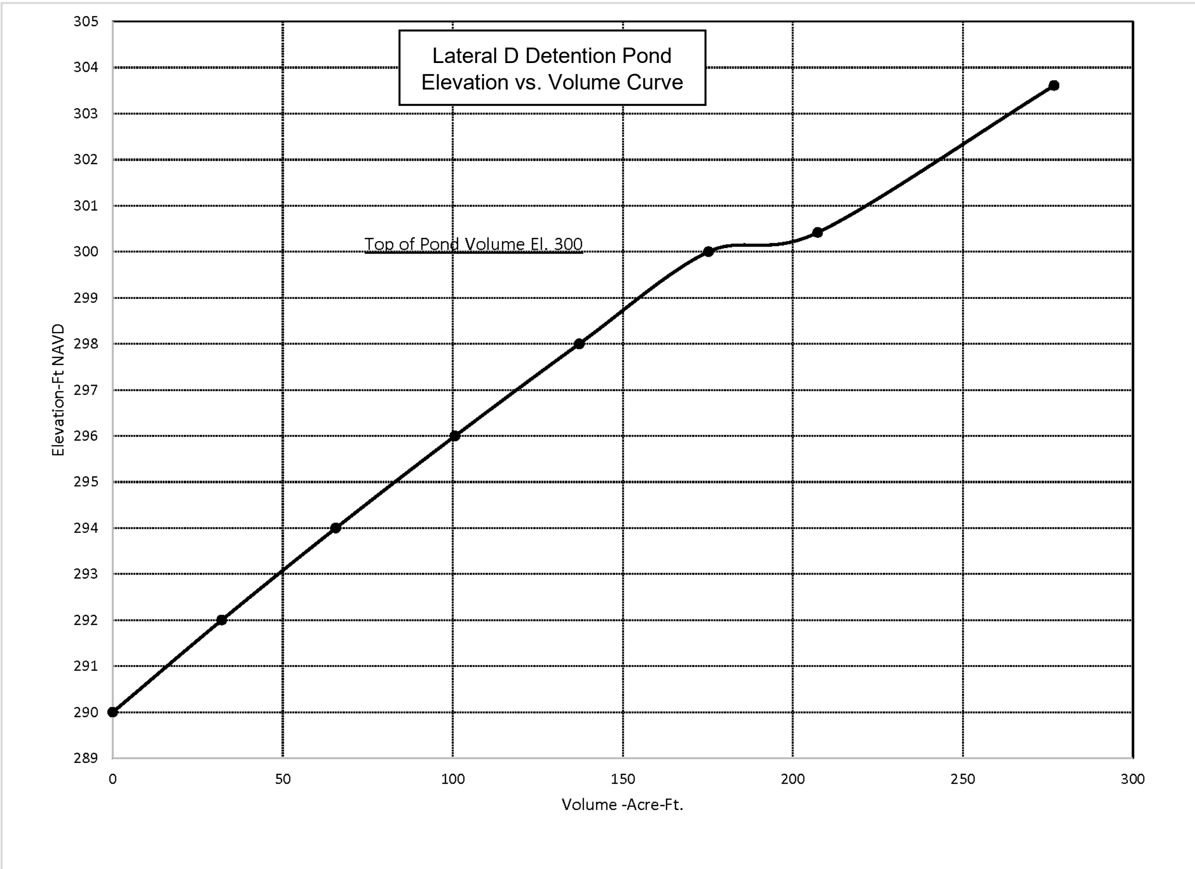


Figure 51. Lateral D-Volume Curve

Calculated velocities for the spillway crest were 6.2 fps at a depth of approximately 5 feet and were used to assess riprap design. The basins capacities are relatively small, and overtopping will be frequent. Both the height, length and other spillway parameters design will be optimized during feasibility-level design.

Peak steady flows, resulting from the detention pond, were input in the HEC-RAS 1D models for Lateral D and project water surface profiles were developed. The following Elevation vs. Probabilities relationships are shown for existing conditions and projection conditions in figures 52 and 53.

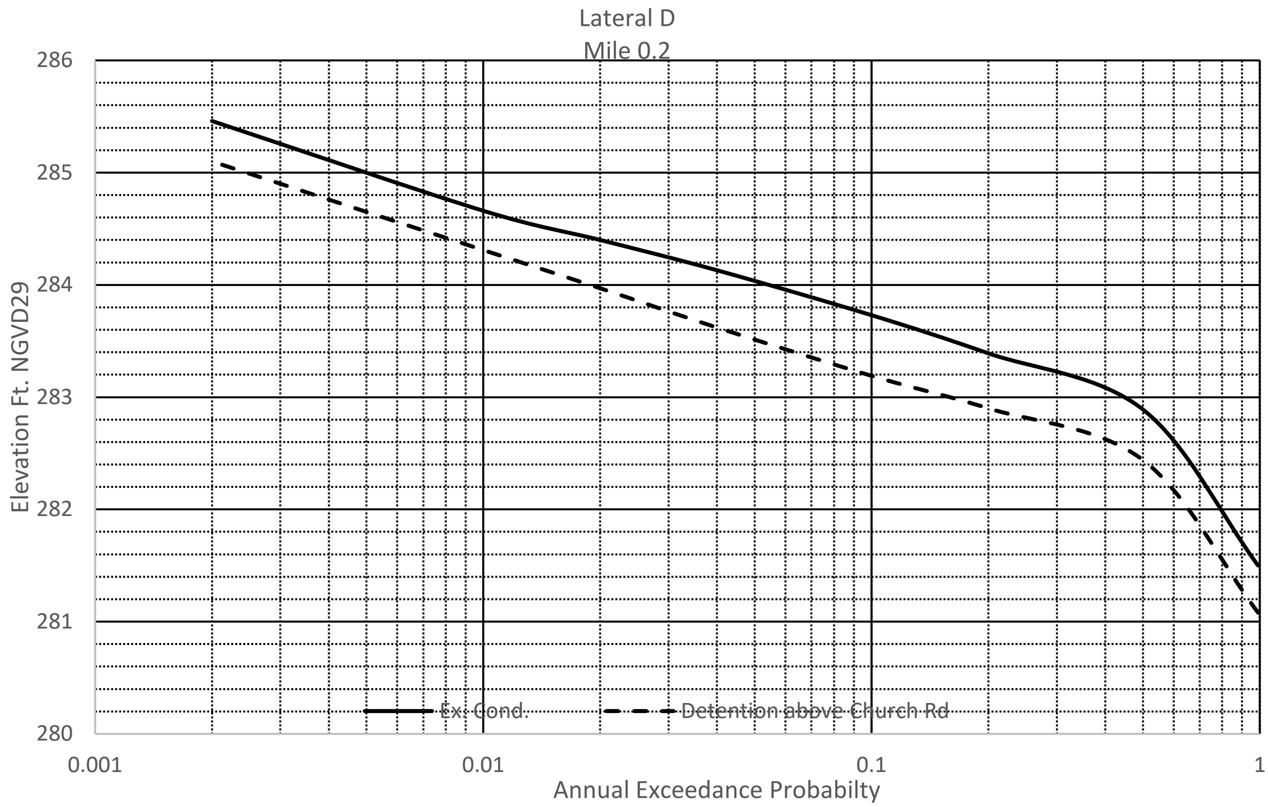


Figure 52. Water Surface Elev. Vs. Probability Curve

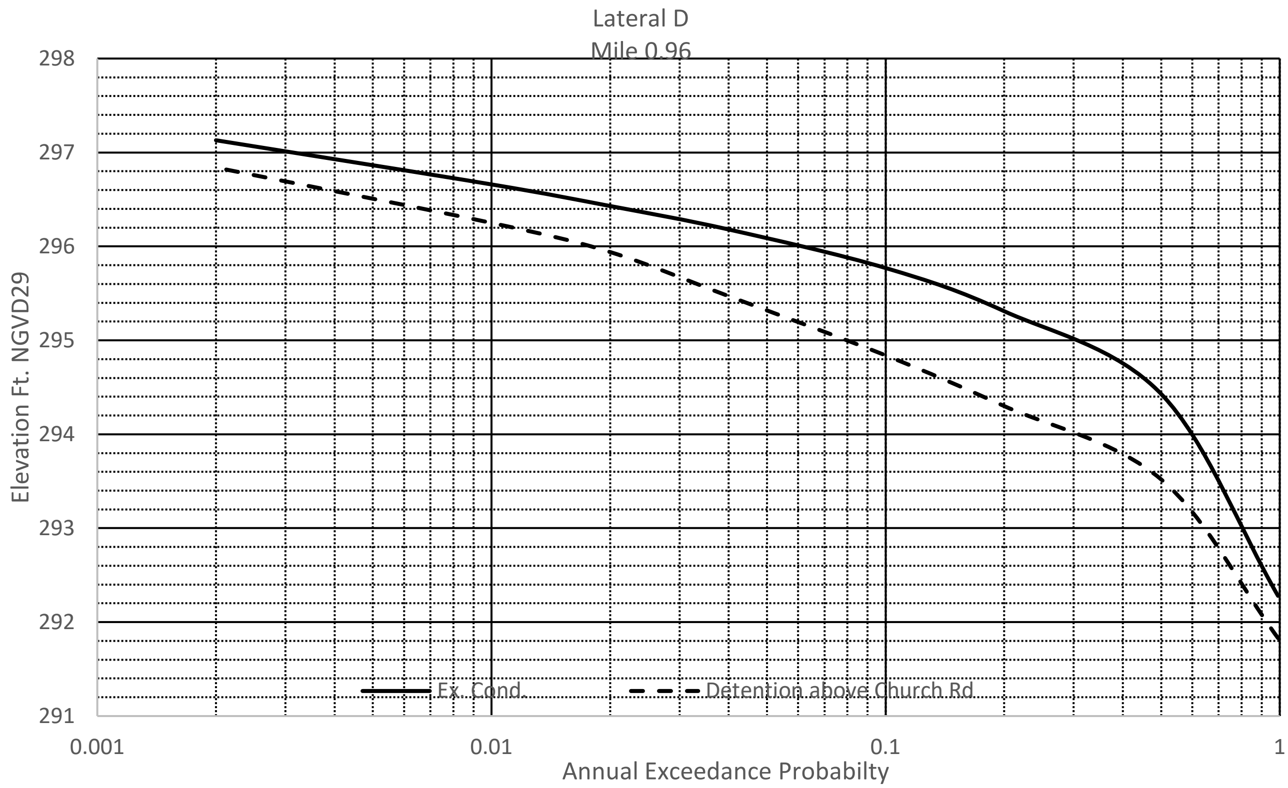


Figure 53. Water Surface Elev. Vs. Probability Curve

6.6 FREQUENCY VS. ELEVATION CURVES TENTATIVELY SELECTED PLAN (ORIGINAL TSP)

The initial National Economic Development Plan (NED) identified from the final array of Flood Risk Management alternatives is a combination of the Horn Lake Creek Channel Enlargement (RM 18.6-19.4) combined with the Lateral D Detention basin, and an optimized nonstructural plan. This is explained in more detail in the Economic Appendix L.

The Original TSP is not the National Economic Development (NED) Plan. The non-federal sponsor identified a combination of the above measures as the locally preferred plan. This plan includes all component measures included in the NED plan (Horn Lake Creek Channel Enlargement (RM 18.6-19.4) combined with the Lateral D Detention basin, as well as two additional detention basins (Cow Pen and Rocky Creek Detention basins). Stage vs. Frequency curves on Horn Lake Creek are shown on Figures 54 to 56.

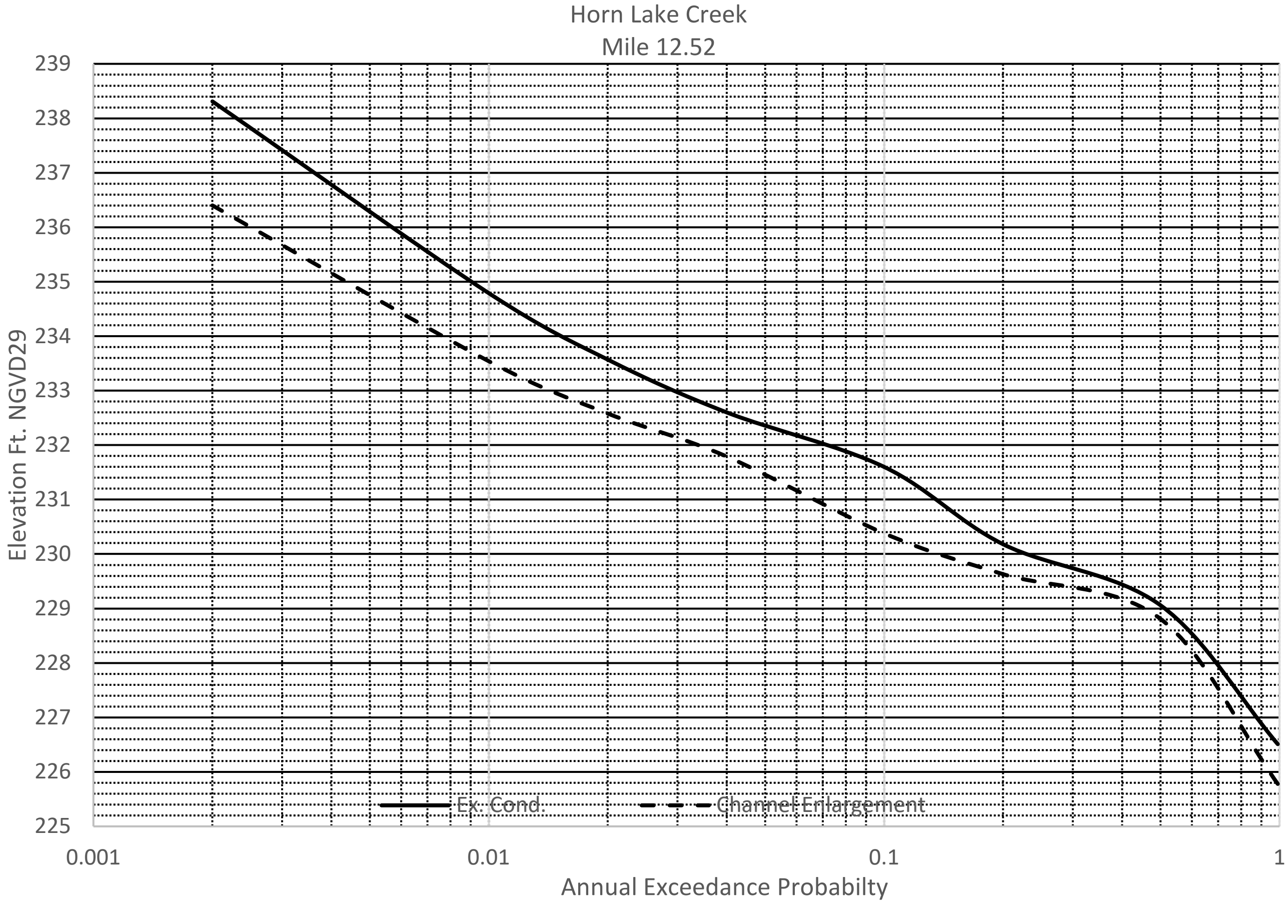


Figure 54. Water Surface Elev. Vs. Probability Curve

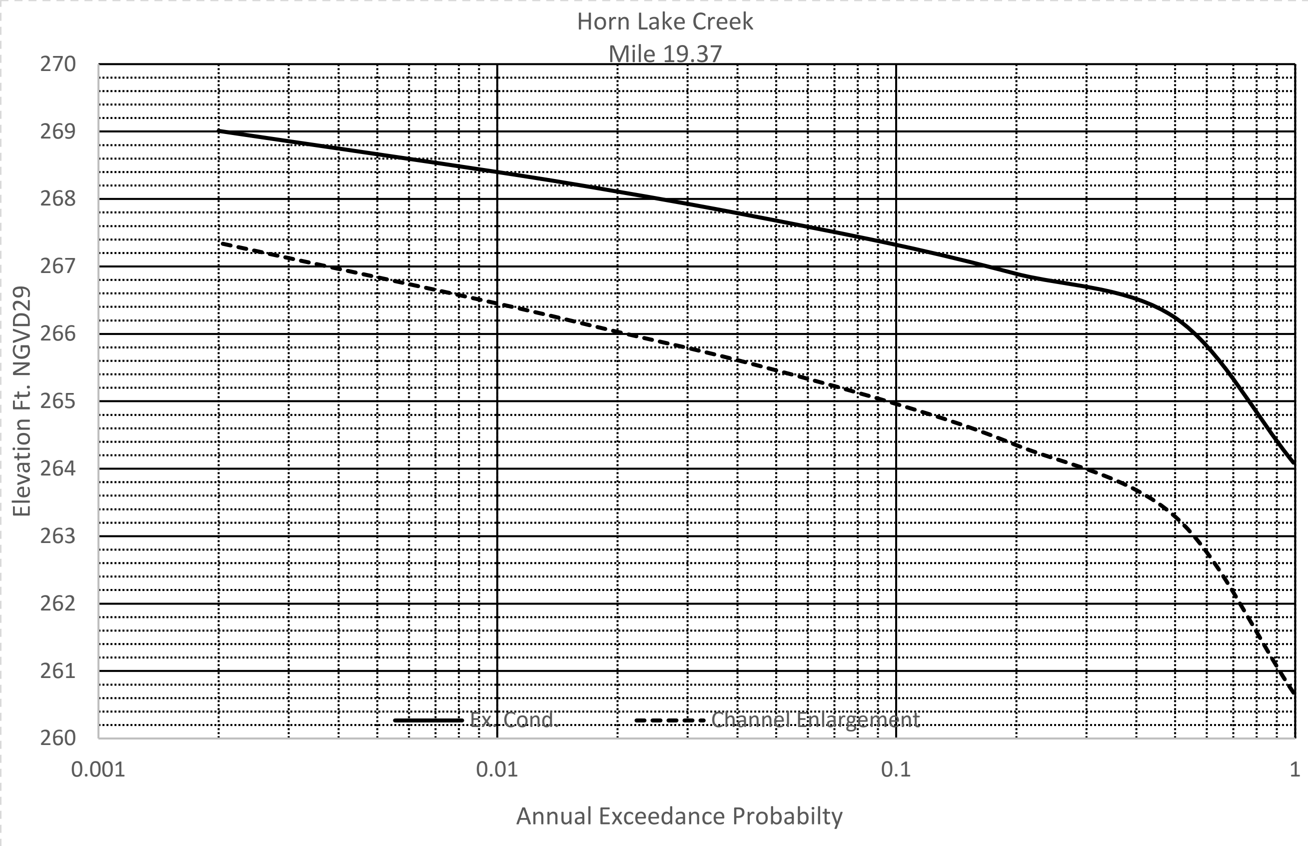


Figure 55. Water Surface Elev. Vs. Probability Curve

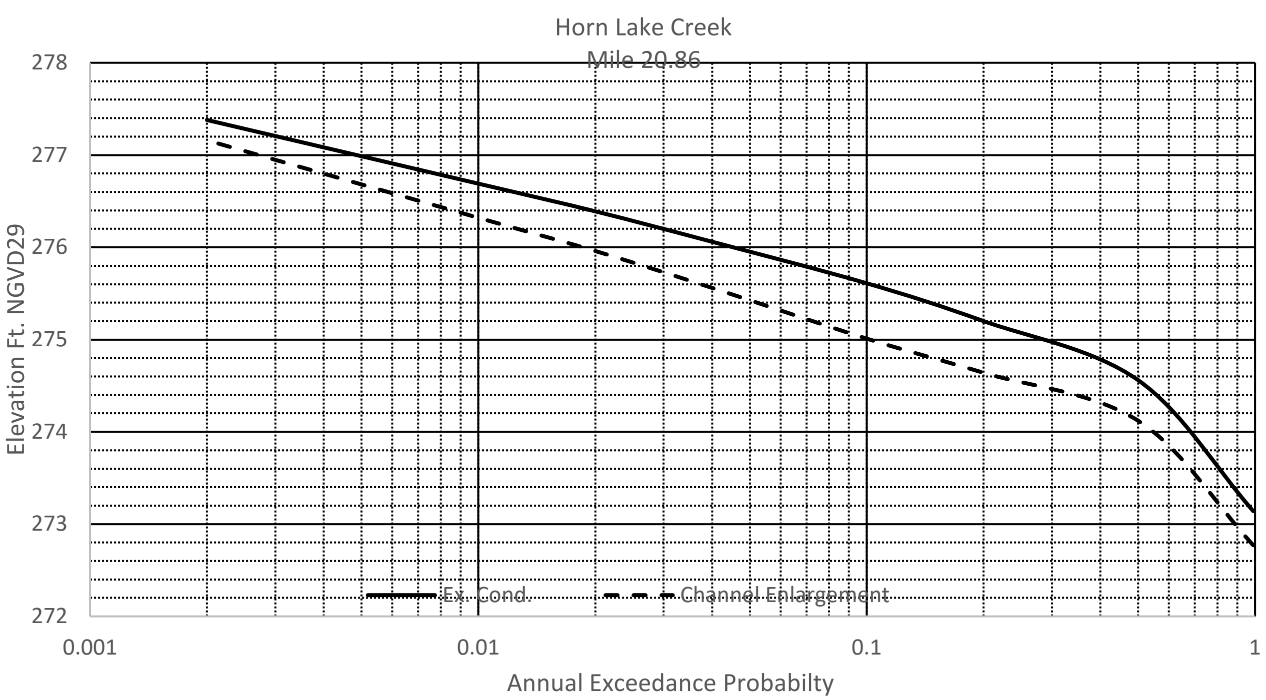


Figure 56. Water Surface Elev. Vs. Probability Curve

6.7 HORN LAKE CREEK BASIN MODELING - HEC-RAS 1D/2D UNSTEADY FLOW DEVELOPMENT

A couple of areas within the Horn Lake Creek basin experience complex flow conditions and it was determined the 1D/2D unsteady flow capabilities of the HEC-RAS program would be needed to simulate and capture specific information. The primary location for HEC-RAS 1D/2D application is the intersection of Highway 51 and Goodman Road, also known as Bullfrog Corner. A review of historical flooding documentation indicates Horn Lake Creek typically exceeds its current capacity upstream of Goodman Road, flows westward overtopping Highway 51 and inundates the southwestern quadrant of Bullfrog Corner.

The alignment of Horn Lake Creek, combined with the railroad crossing embankment, appear to create a major constriction across the floodplain potentially resulting in a significant backwater effect upstream. HEC-RAS 1D/2D was able to simultaneously assess both complex conditions and quantify the flooding in this study's major damage area.

Other study requirements were identified that promoted the use of HEC-RAS 1D/2D. The analysis of detention basins and the respective consequences of a failure prompted the need for a more detailed analysis to ensure the benefits derived in the 1D analysis presented in the previous section were adequately assessed. Unsteady flow analysis was needed to ensure storage capacity was available during the storm and benefits derived by using the standard HEC-HMS routing techniques were not overestimated since they don't account for tailwater conditions.

Development of the 1D/2D model altered the assessment of the Original TSP from the 1D analysis. The 2D analysis of the overbank areas removed damages from the project, thereby dropping the BCR of the Original TSP. Comparisons of the 1D and 1D/2D modeling results showed that water surface elevations were maintained and consistent across the models, validating the results of the new hydraulic modeling approach. The change in BCR was reflected in the economic calculations moving from a traditional cross-section approach to a grid-based approach. These updates to the economic analysis along with the new hydraulic modeling approach resulted in a new TSP from the existing alternatives which is a levee and floodwall along the southwest corner of Hwy 51 and Goodman Road. The development and results from the 1D/2D analysis are discussed further in this section. The preferred final recommended plan is now identified as the construction of a levee and floodwall described later in this appendix as the Horn Lake Creek Levee Alternative.

6.8 EXISTING CONDITION UNSTEADY FLOW – HEC-HMS

6.8.1 HEC-HMS-Upstream Inflow Hydrographs

HEC-HMS flow hydrographs were used as inflow boundary conditions for the 2018 HEC-RAS 1D/2D Unsteady flow model. Three types of inflow runoff hydrographs were extracted from HMS and used as HEC-RAS 1D/2D model inputs or boundary conditions: 1D upstream inflow hydrographs, lateral inflow, and uniform later inflow hydrographs.

The upstream boundary conditions of the 1D reaches of Lateral D, Rocky Creek, Cow Pen Creek and Horn Lake Creek were based on HEC-HMS runoff hydrographs. On streams or creeks that were not individually modeled as a 1D reach, hydrographs were input into Horn Lake Creek as Lateral Inflow or Uniform Lateral Inflow Hydrographs.

Figure 57-63 show the locations of the upstream boundaries of the Horn Lake Creek, Rocky Creek, Cow Pen Creek and Lateral D and their respective upstream inflow hydrographs.

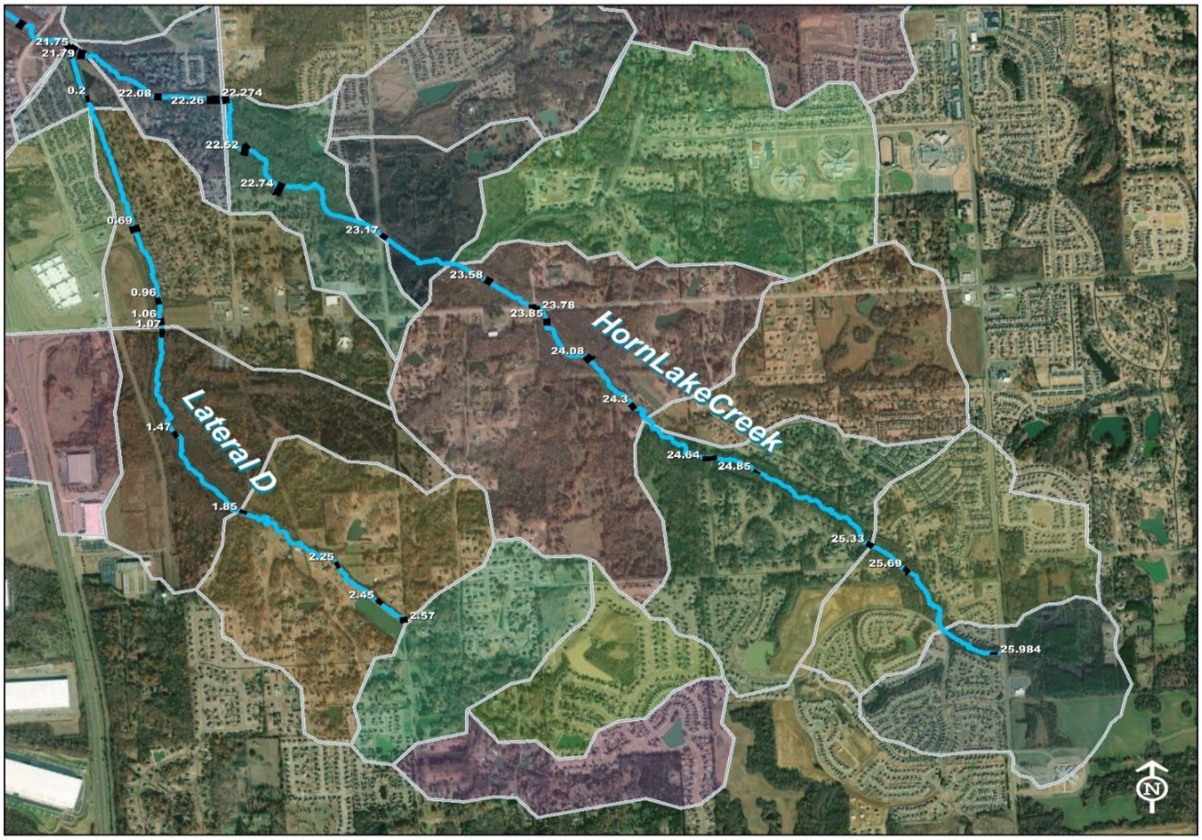


Figure 57. Horn Lake Creek (Right) and Lateral D (Left) Upstream Boundary Locations

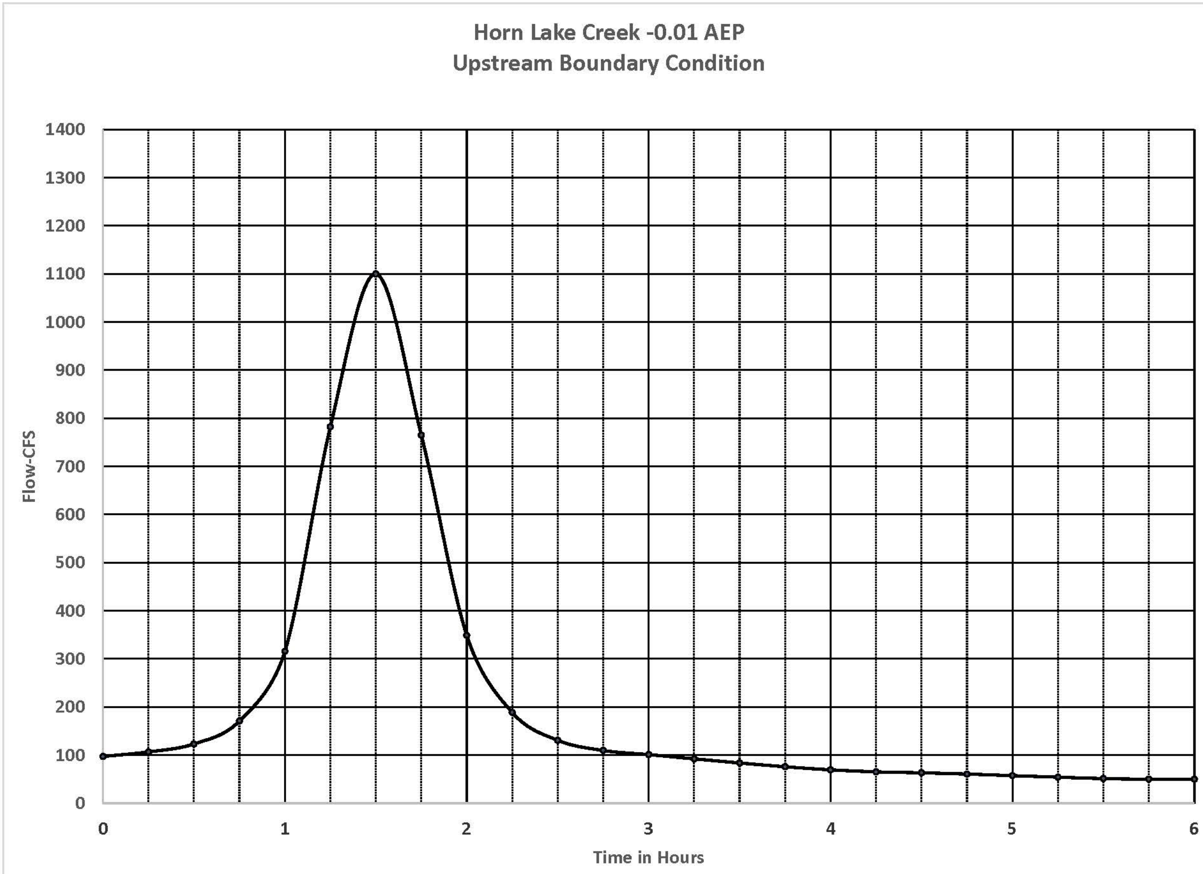


Figure 58. Horn Lake Creek and Lateral D 0.01 AEP (100 Year) Inflow Hydrographs

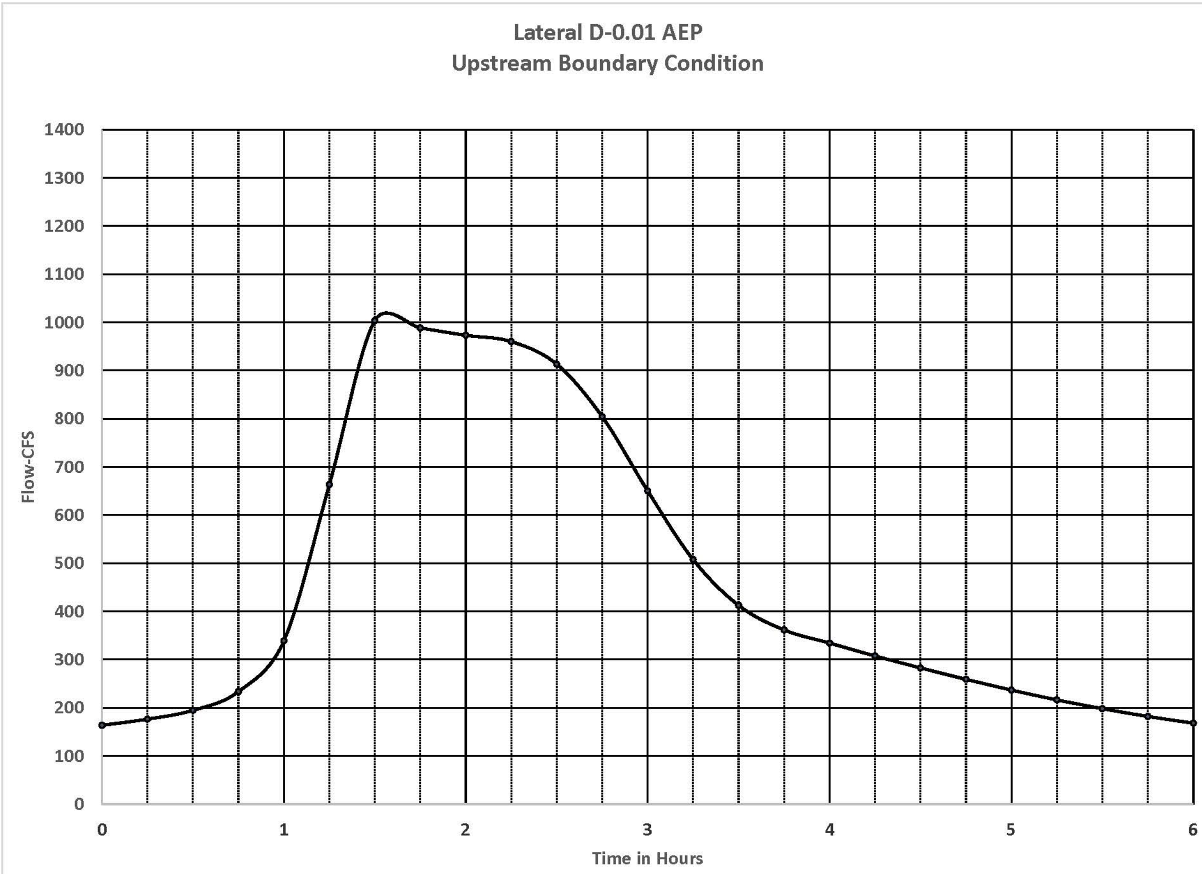


Figure 59. Horn Lake Creek and Lateral D 0.01 AEP (100 Year) Inflow Hydrographs

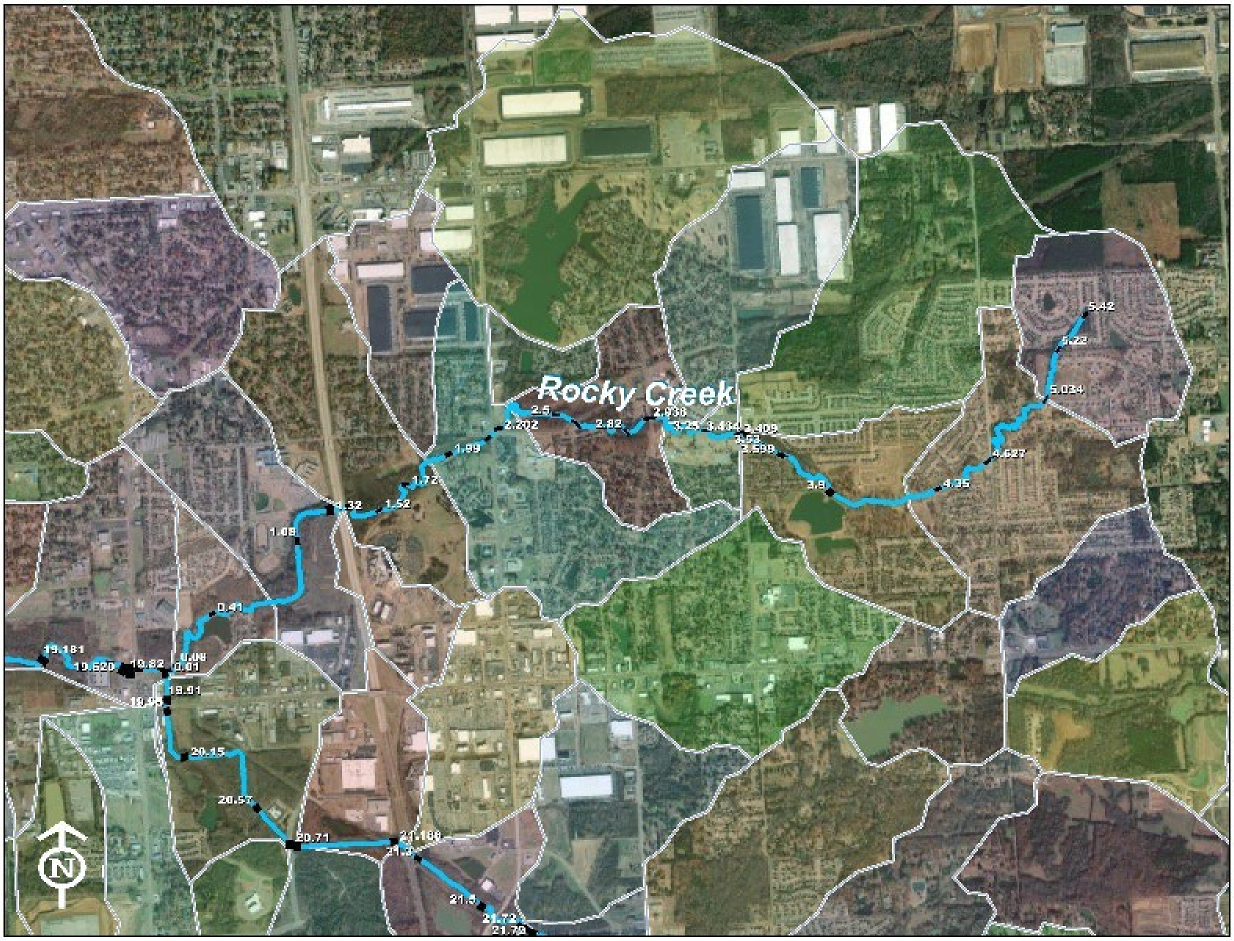


Figure 60. Rocky Creek Upstream Boundary Location and 0.01 AEP (100 Year) Inflow Hydrograph

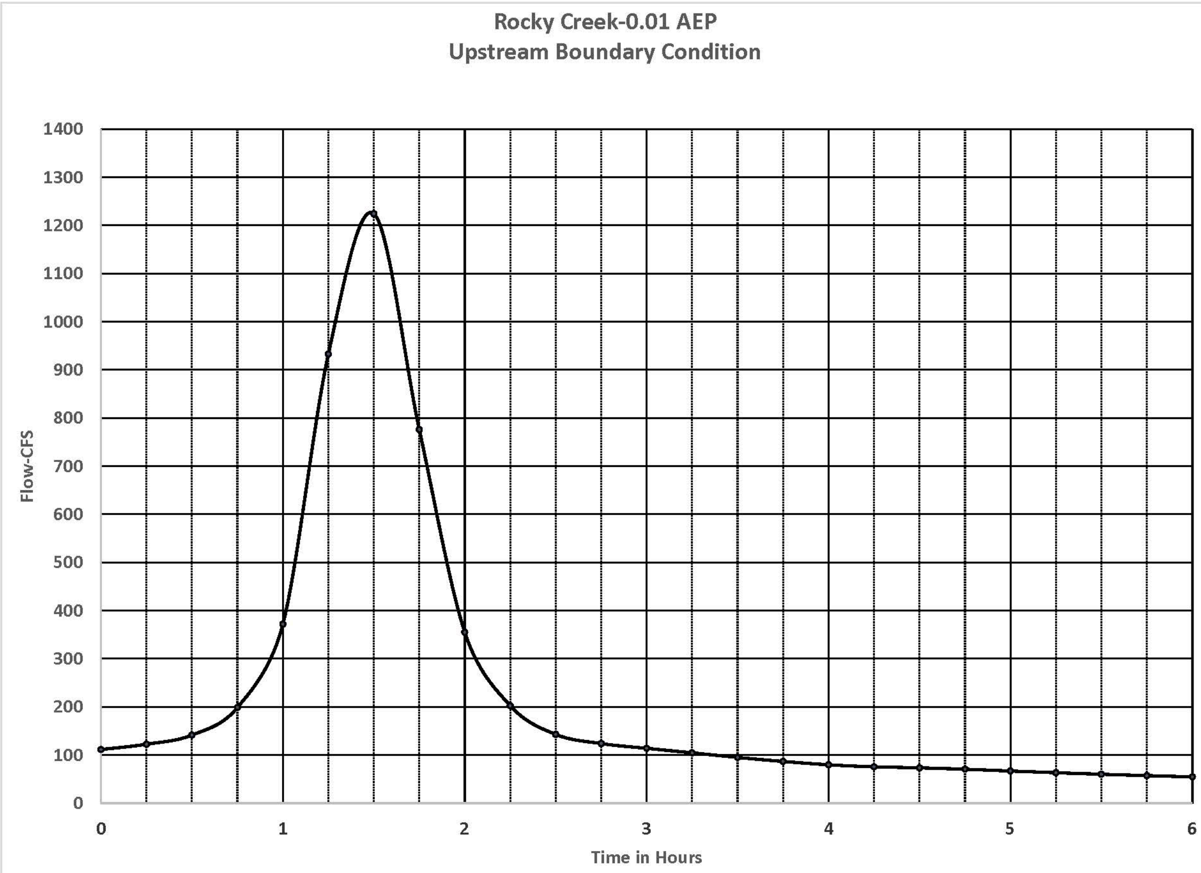


Figure 61. Rocky Creek Upstream Boundary Location and 0.01 AEP (100 Year) Inflow Hydrograph

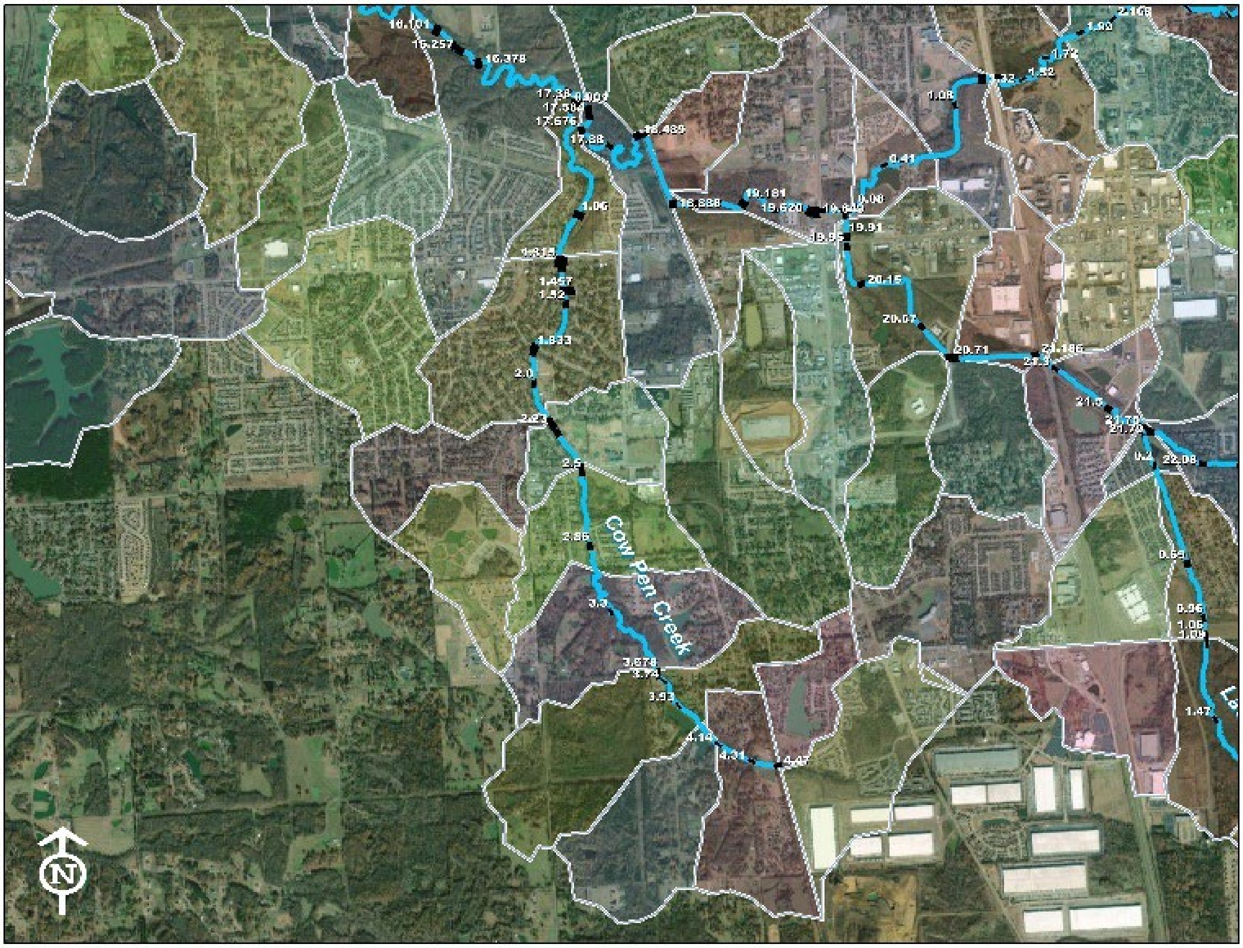


Figure 62. Cow Pen Creek Upstream Boundary Location and 0.01 AEP (100 Year) Inflow Hydrograph

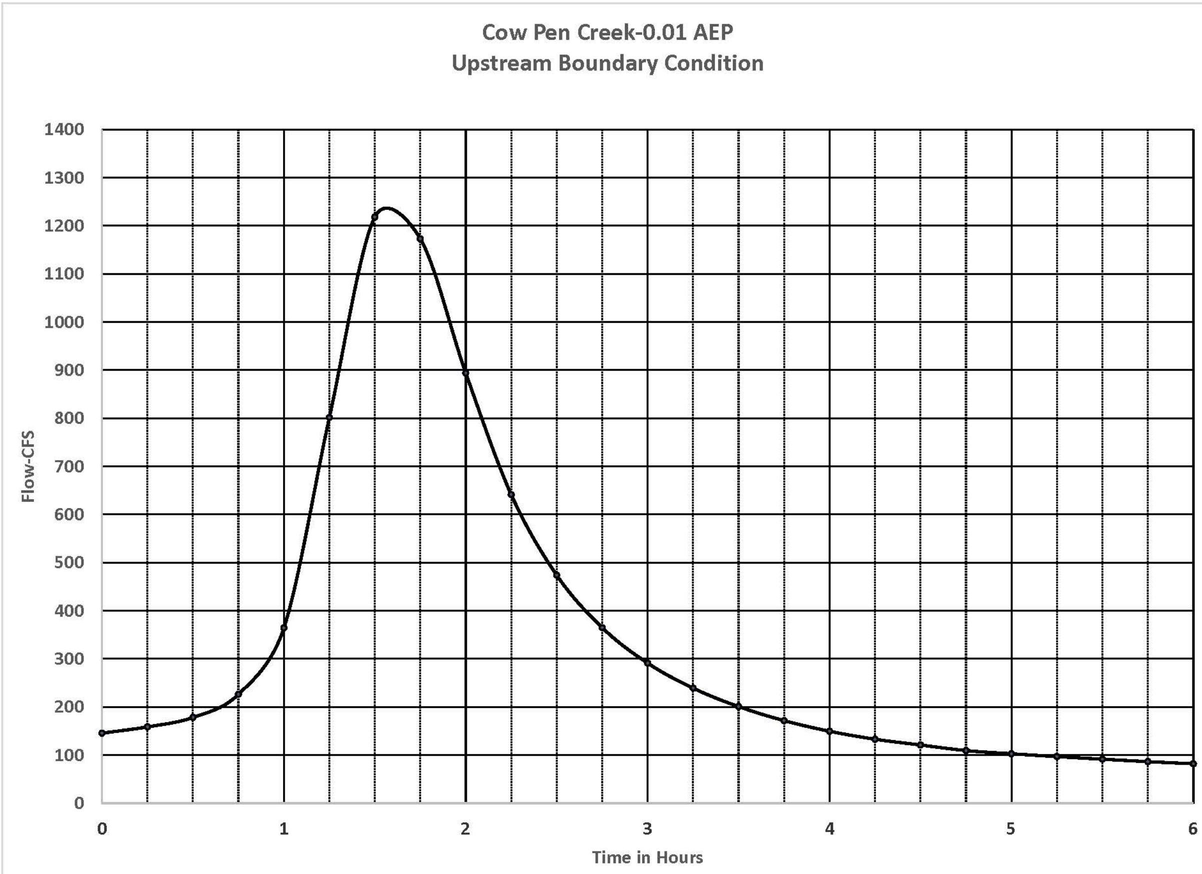


Figure 63. Cow Pen Creek Upstream Boundary Location and 0.01 AEP (100 Year) Inflow Hydrograph

6.8.2 HEC-HMS Lateral Inflow Hydrographs

Inflow hydrographs were also applied to 1D portions of the HEC-RAS model in the form of lateral inflow hydrographs. These hydrographs represent contributing flow from basins that are not individually modeled as a 1D reach. Minor tributaries inflows were captured by inflow hydrographs and intervening runoff between computational nodes was input as a uniform inflow hydrograph. There are approximately 50 individual lateral inflow hydrographs in the unsteady flow model.

6.9 EXISTING CONDITIONS-HEC-RAS 1D/2D

Updating the hydraulic model from a 1D analysis to a 1D/2D unsteady flow model provided the team with detailed simulations and analysis of flow conditions at the major constriction point of Horn Lake Creek and the study's major damage area.

Modeling the overbank and proposed detention areas from the 1D model's Original TSP allowed the team to assess the consequence of failure and the adequacy of available storage during storm events when backwater is present in the system which can be a common occurrence. Modeling in this way helps to attenuate flow from HMS since HMS's simplified routing overestimates flows during backwater conditions.

The 2002 channel sections for the 1D reaches overbank of Horn Lake Creek, Cow Pen Creek, Rocky Creek and Lateral D were replaced using LiDAR data acquired in 2010. Unlike the previous studies, the 1D cross section were georeferenced for the 1D/2D analysis. As stated previously, bridge data was imported from the 2005 HEC-RAS 1D steady flow model. Several minor adjustments were necessary during the import process and the original 1993 bridge survey was available and used to complete the effort. Pertinent import information is presented in Paragraph 6.9.3. With an elevated interest in flow conditions at the Railroad, more accurate modeling of the pile configuration, alignment impacts, and overflow areas were necessary.

Like the previous 1D model, the new 1D/2D model was calibrated to the stages established in the current FIS study. Profile elevations were checked along the 1D channel and overbank cross sections were checked at the same FIS cross section locations which are now being modeled as a 2D overbank. As stated previously, this basin is ungaged and calibration and validation of the model was done using a previous study. It is noted that geomorphic changes have likely occurred during the time of the FIS study and this modeling effort. These geomorphic changes are not reflected in the model's current terrain and cross section data. PED phases should consider collection of supplementary channel and overbank survey to ensure model accuracy for future work and final construction level design of the preferred final recommended plan.

Structures that were constructed after the previous studies (i.e., Interstate Boulevard) were included in the study update. The 2D areas were constructed using 100-foot by 100-foot computational cells on the landward side of the lateral structures. Figure 64 shows the general layout of the 1D/2D geometric model.

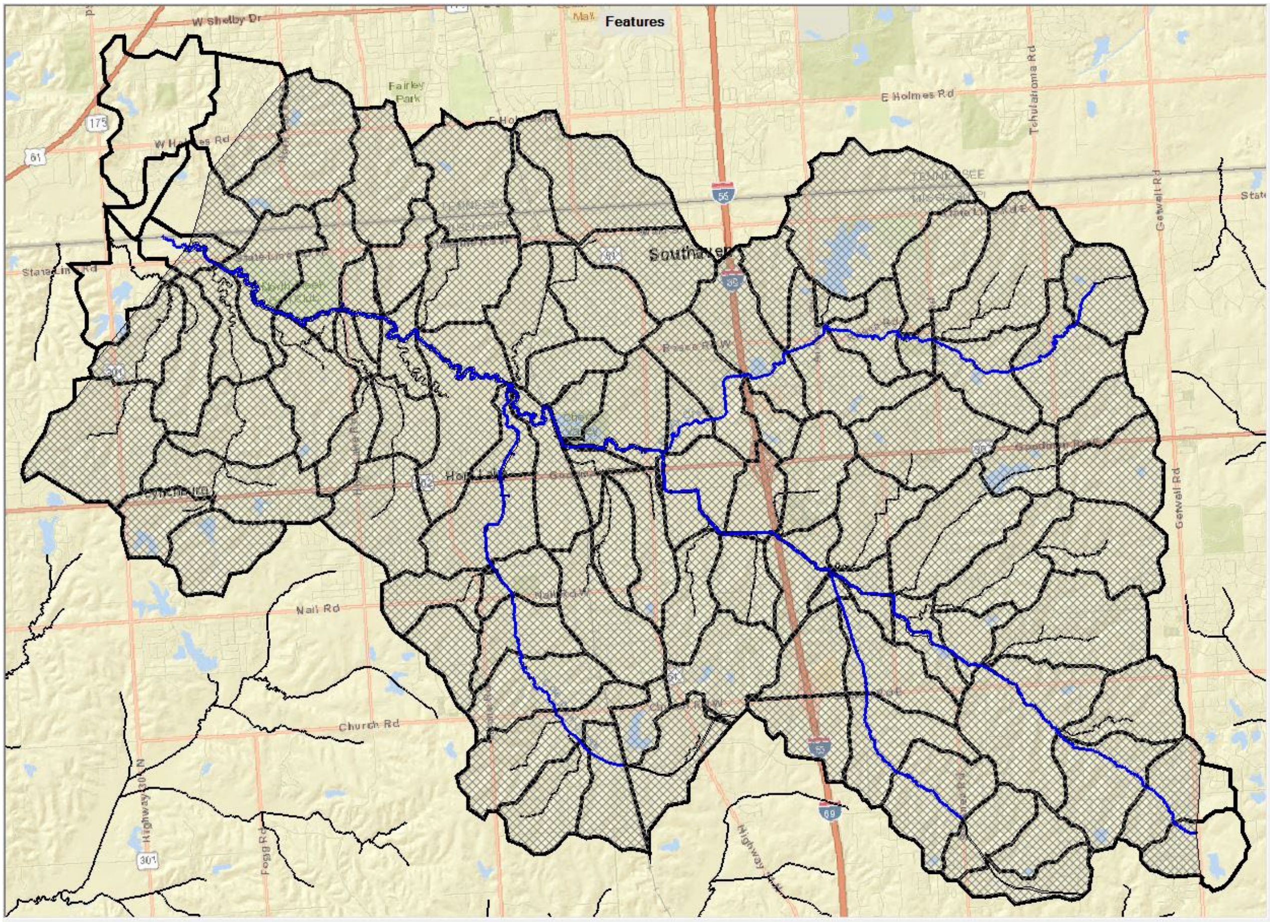


Figure 64. HEC-RAS 1D and 2D Geometric Characteristics

6.9.1 Terrain Data

The 2010 LIDAR was used to the develop the Terrain Elevation model for the 2D areas. Lateral structures were constructed to simulate the transfer of flow from the 1D reaches to the 2D areas. Elevations of the Lateral Structure were also obtained from the 2010 LiDAR. All lateral structures computed overflow using the weir equation as opposed to the Normal 2D equations. Weir coefficients were estimated to be 0.1 to simulate un-elevated ground at the 1D/2D interface, as stated in HEC-RAS guidance.

6.9.2 Channel and Overbank Roughness Coefficients

Manning's roughness coefficients, updated in the 1D HEC-RAS reaches, remained constant for the 1D/2D assessment. Overbank roughness coefficients were computed using the National Land Cover Database (NLCD) dated 2016 developed by the United States Geological Survey (USGS). Manning's roughness coefficients were not altered for future conditions. Table 5 shows the final adopted roughness coefficients used in the 2D areas. It should be noted some land cover descriptions did not match the current land use in the basin. Land uses with discrepancies were matched with the most appropriate manning's n.

Table 8. Manning's Roughness Summary

Land Cover Name	Manning's n
Developed, Open Space	0.11
Developed, Low Intensity	0.11
Developed, Medium Intensity	0.09
Developed, High Intensity	0.08
Barren Land	0.08
Deciduous Forest	0.12
Evergreen Forest	0.09
Mixed Forest	0.09
Shrub/Scrub	0.12
Grassland, Herbaceous	0.09
Pasture/Hay	0.09
Cultivated Crops	0.12
Woody Wetlands	0.12
Emergent Herbaceous Wetlands	0.09

6.9.3 Bridge and Roadway Crossings

The decision to construct a 1D/2D model as opposed to a complete 2D model was based on the 1D HEC-RAS capabilities to perform complex bridge hydraulic calculations. At the time of model development, the HEC-RAS 2D computation engine lacked the ability model bridges in the 2D environment. Since the available version of HEC-RAS lacks a concrete option to analyze bridges in a 2D environment, it was decided to model Horn Lake Creek basin and its tributaries in the 1D environment of HEC-RAS. Modification of the terrain data to simulate bridge piles has been recommended in certain cases, but emphasis on the Railroad and it's 32 piles necessitated the use of more standard bridge modeling options. Field reconnaissance, Desoto County GIS data review, and sponsor coordination were conducted to ensure any major alteration to stream crossings and other structures were captured in the updated model(s).

All overbanks flow conditions, including roadway overtopping, were modeled in a 2D environment. Breaklines and 2D Connections were used in the 2D areas to model barriers of flows such as natural high ground or elevated roadway embankments. Although bridge modeling is limited with HEC-RAS version 5.0.7, the program has the capability to model culverts in 2D connections and was utilized when information was readily available and if the overflow area was considered significant to results.

6.9.4 Existing Conditions -HEC-RAS 1D/2D Results

The hydrographs from HEC-HMS were input in the HEC-RAS 1D/2D model and results were developed for the same probabilistic events analyzed for HEC-1 1D Steady flow analysis. Inundation maps showing the depth of flooding were created from the 1D/2D analysis. Figures 65 to 72 show the extent of existing conditions flooding for the watershed above the Mississippi-Tennessee Stateline and near Bull Frog Corner for the 0.10, 0.02, 0.01, and 0.002 AEPs.

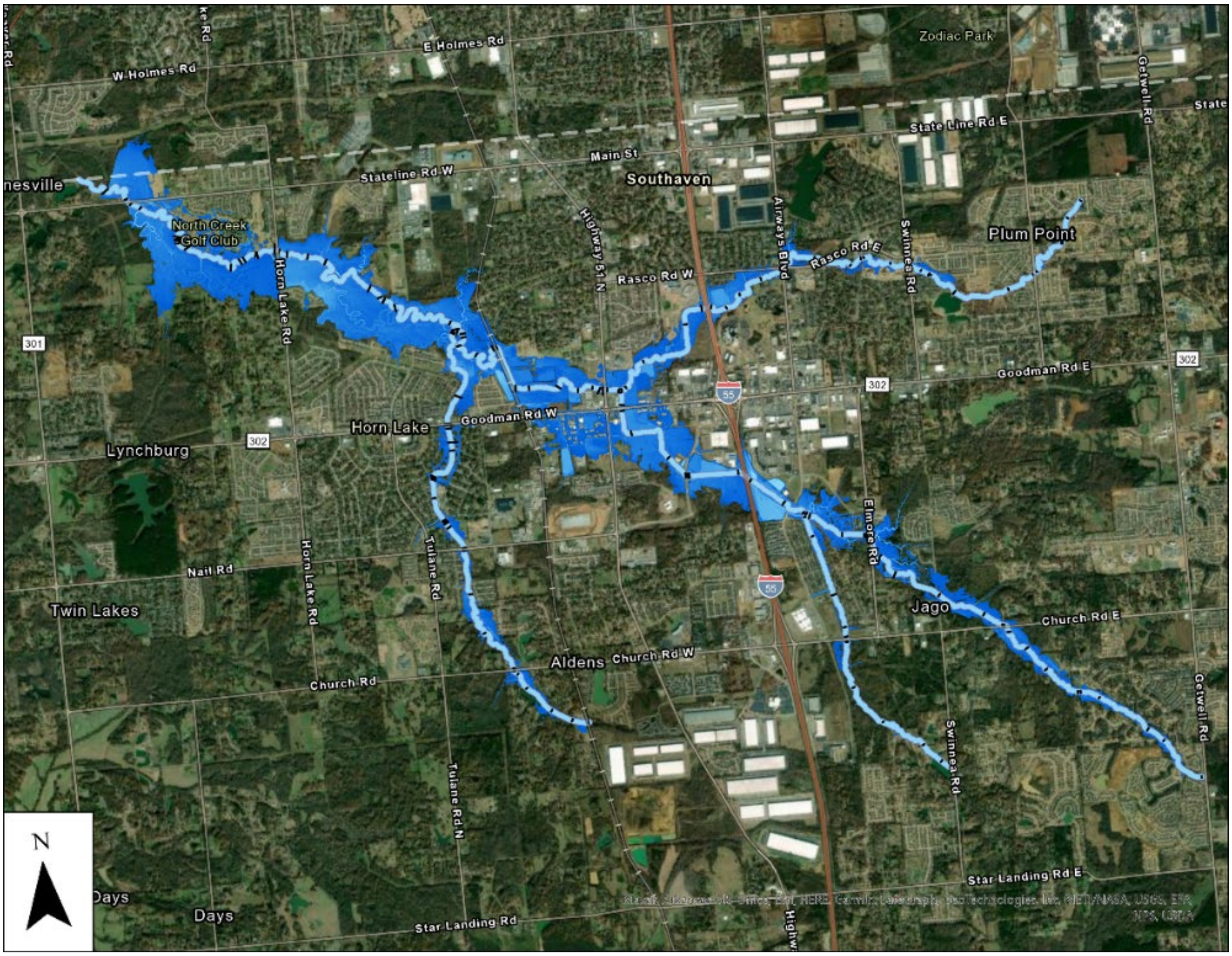


Figure 65. 0.10 AEP Inundation Map-Horn Lake Creek above Mississippi-Tennessee Stateline

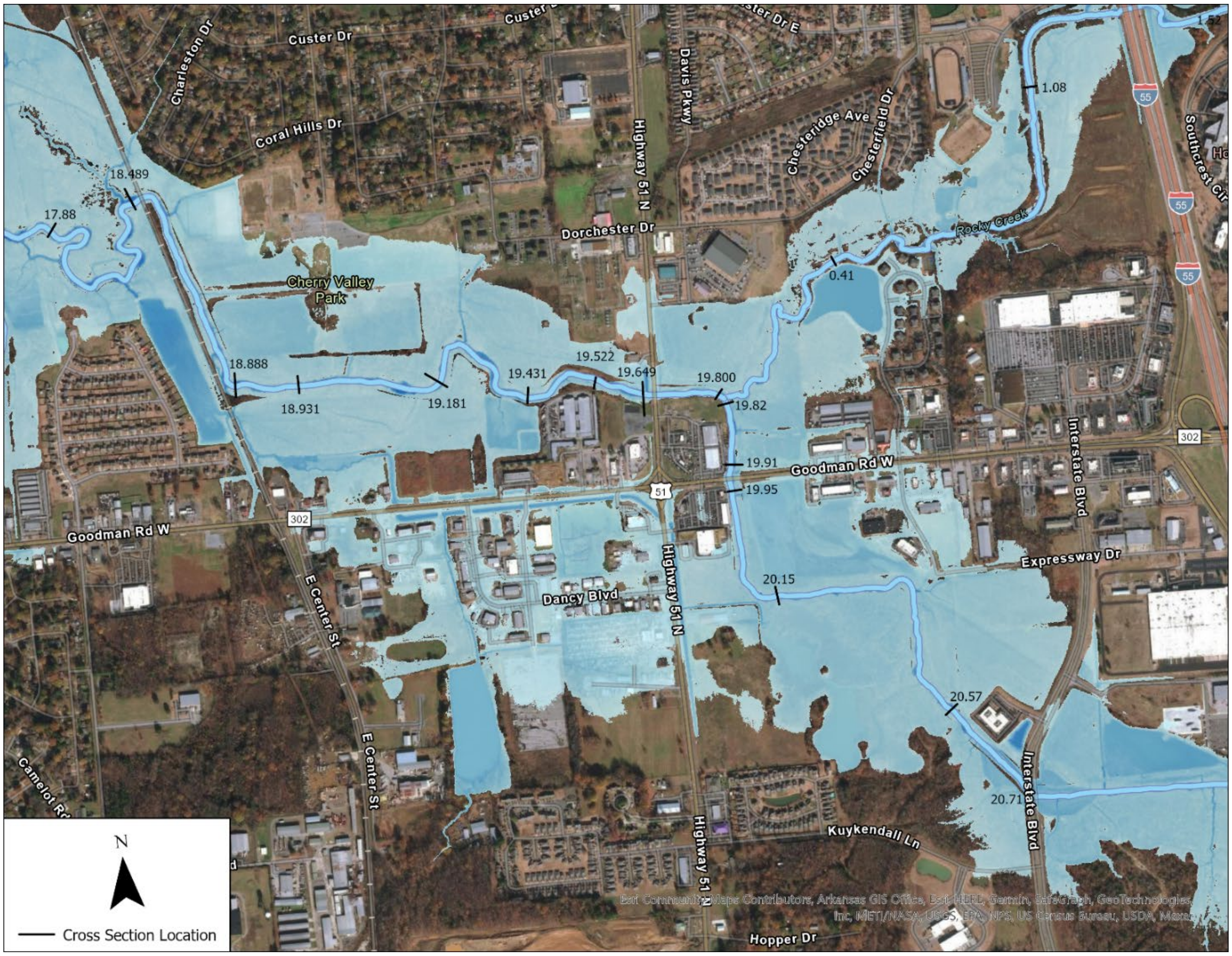


Figure 66. 0.10 AEP Inundation Map-Bullfrog Corner

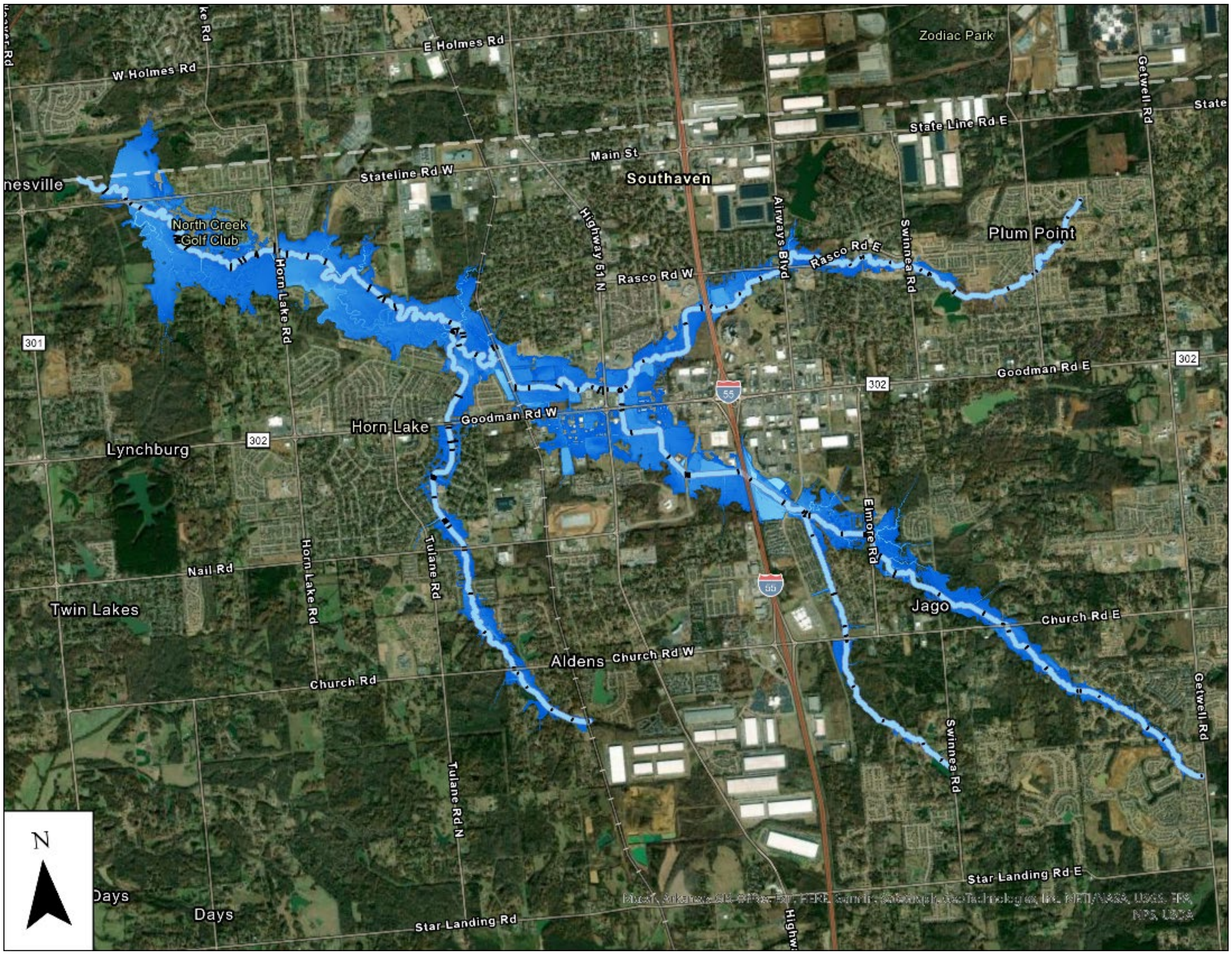


Figure 67. 0.02 AEP Inundation Map-Horn Lake Creek above Mississippi-Tennessee Stateline

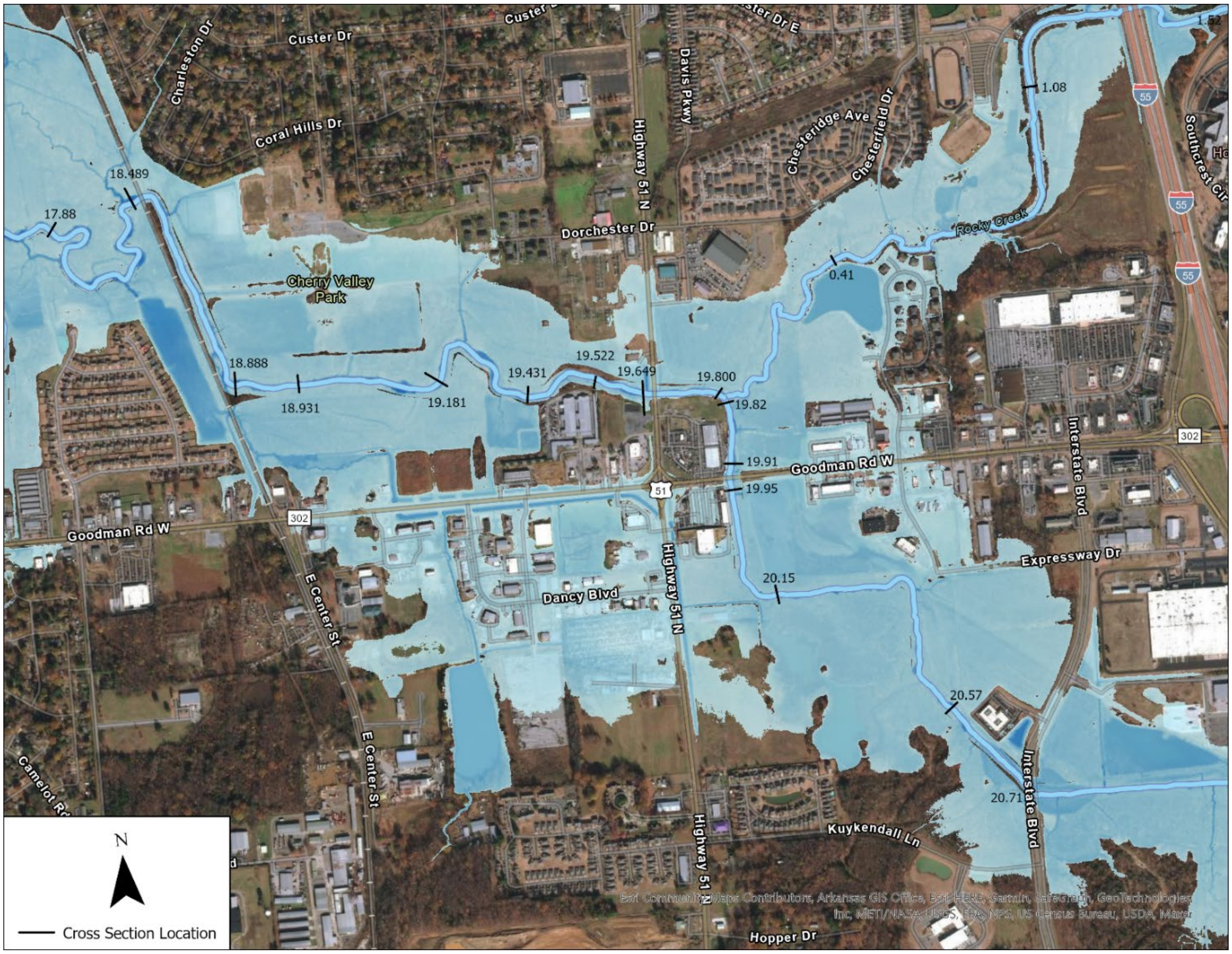


Figure 68. 0.02 AEP Inundation Map-Bullfrog Corner

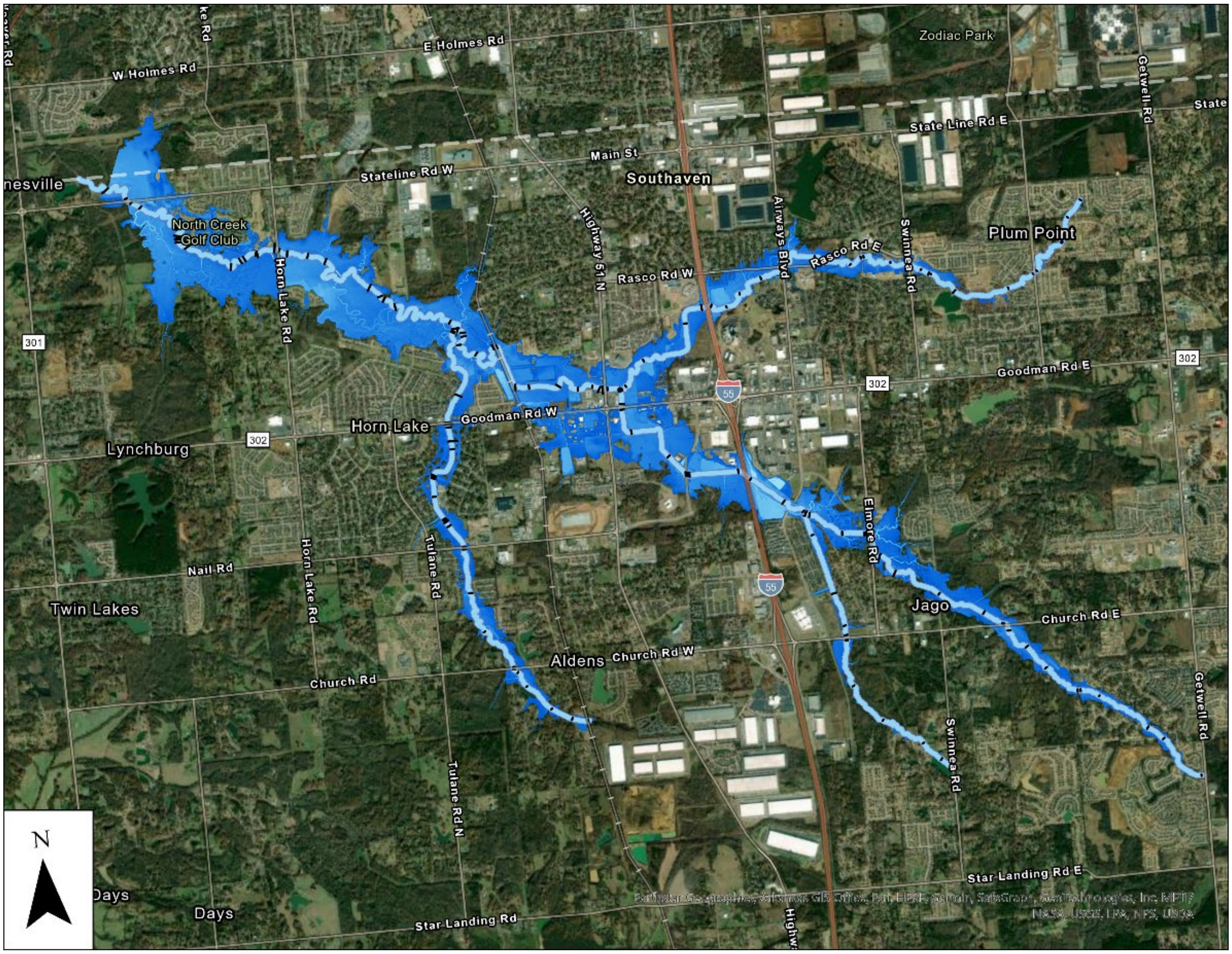


Figure 69. 0.01 AEP Inundation Map-Horn Lake Creek above Mississippi-Tennessee Stateline

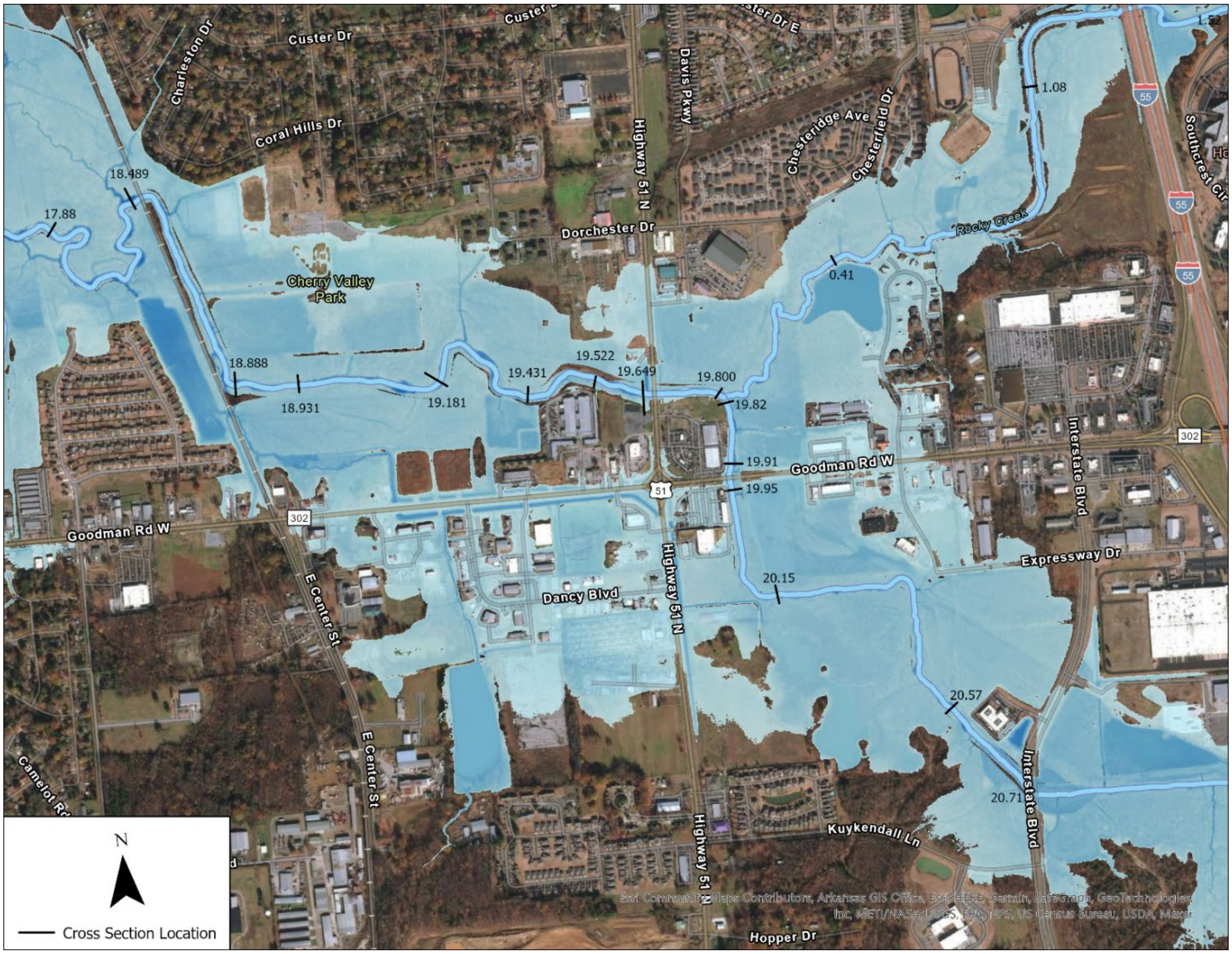


Figure 70. 0.01 AEP Inundation Map-Bullfrog Corner

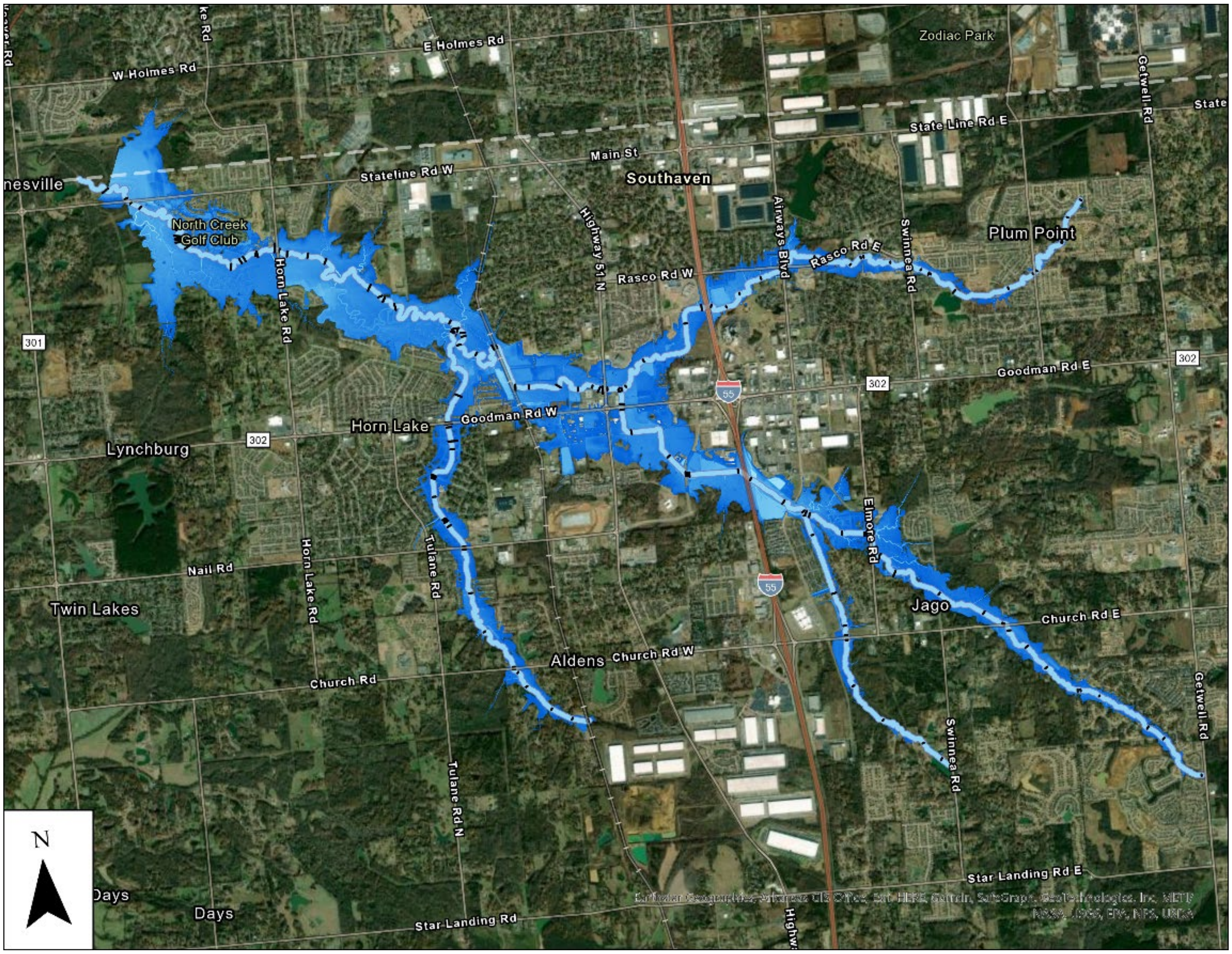


Figure 71. 0.002 AEP Inundation Map-Horn Lake Creek above Mississippi-Tennessee Stateline

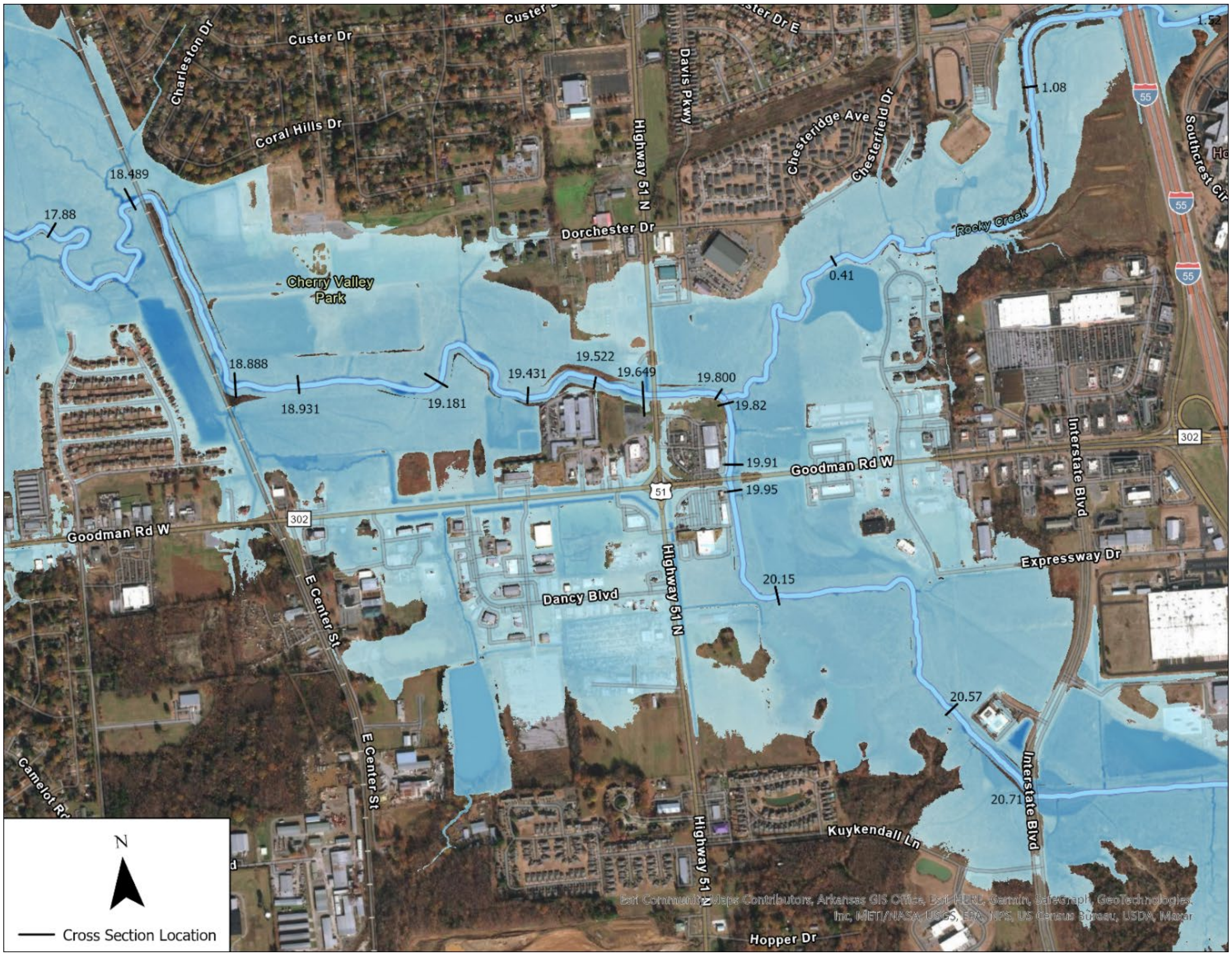


Figure 72. 0.002 AEP Inundation Depth Map-Bullfrog Corner

6.10 HEC-RAS 1D/2D ANALYSIS-ALTERNATIVES

The Horn Lake Creek HEC-RAS 1D/2D model was modified to assess the channel enlargement and detention measures identified in the 1D modeling original TSP selection. Actions necessary to complete this task are very similar to the HEC-RAS 1D modeling efforts. The Channel Improvement option within the HEC-RAS Geometric Editor was applied to the 1D reach Horn Lake Creek using the channel design developed for the original TSP. Inundation depth grids produced by HEC-RAS model were provided to the Economics Project Development Team member and reduction of water surface elevations were quantified. Figure 73 shows the 100-foot 2D cells and overbank topography as developed in the terrain data.

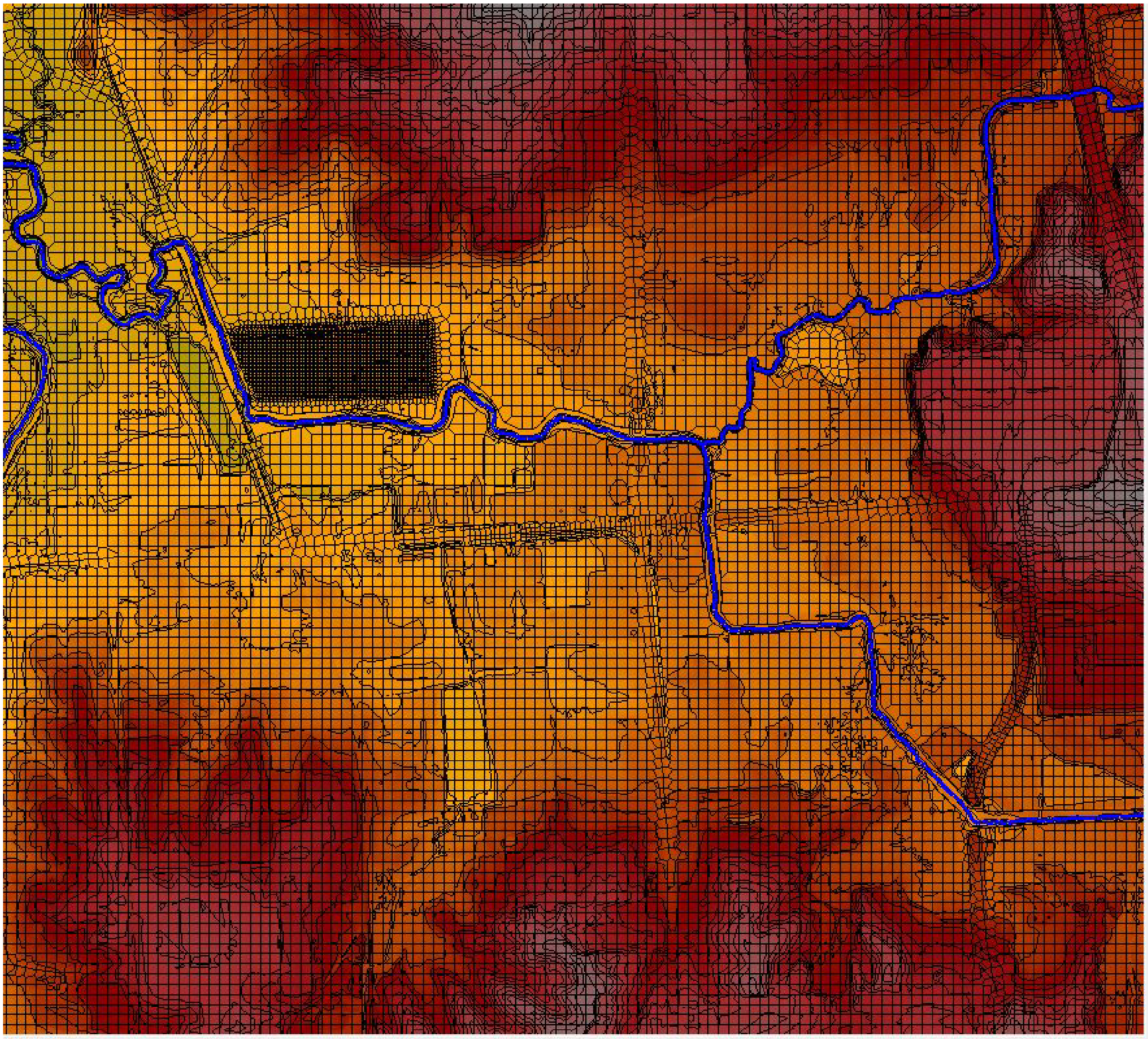


Figure 73. Horn Lake Creek 1D/2D Channel Enlargement Area

6.10.1 Cow Pen Creek Detention Ponds at Nail Road

The Terrain file was altered to simulate detention on both the North and the South Detention ponds. This was accomplished using ArcMap and storage volumes used in the 1D analysis. Figure 74 shows the results of the modifications.

A revised 1D reach was extended through the North Detention Pond to simulate low flows at the approximate bottom Elevation of 258.0 ft. The lateral structure was used to divert flow out of the low flow channel into the adjacent ponding surface area.

The In-line structure option was used to regulate the outlet. Overtopping is expected at elevation 267 and a 4-foot diameter concrete culvert releases low flows.

The South Detention Pond provides side detention. Flow is diverted into pond by a side earthen weir and is modeled by a lateral structure. The pond is slowly evacuated by a 5-foot culvert at the lower end of the pond.

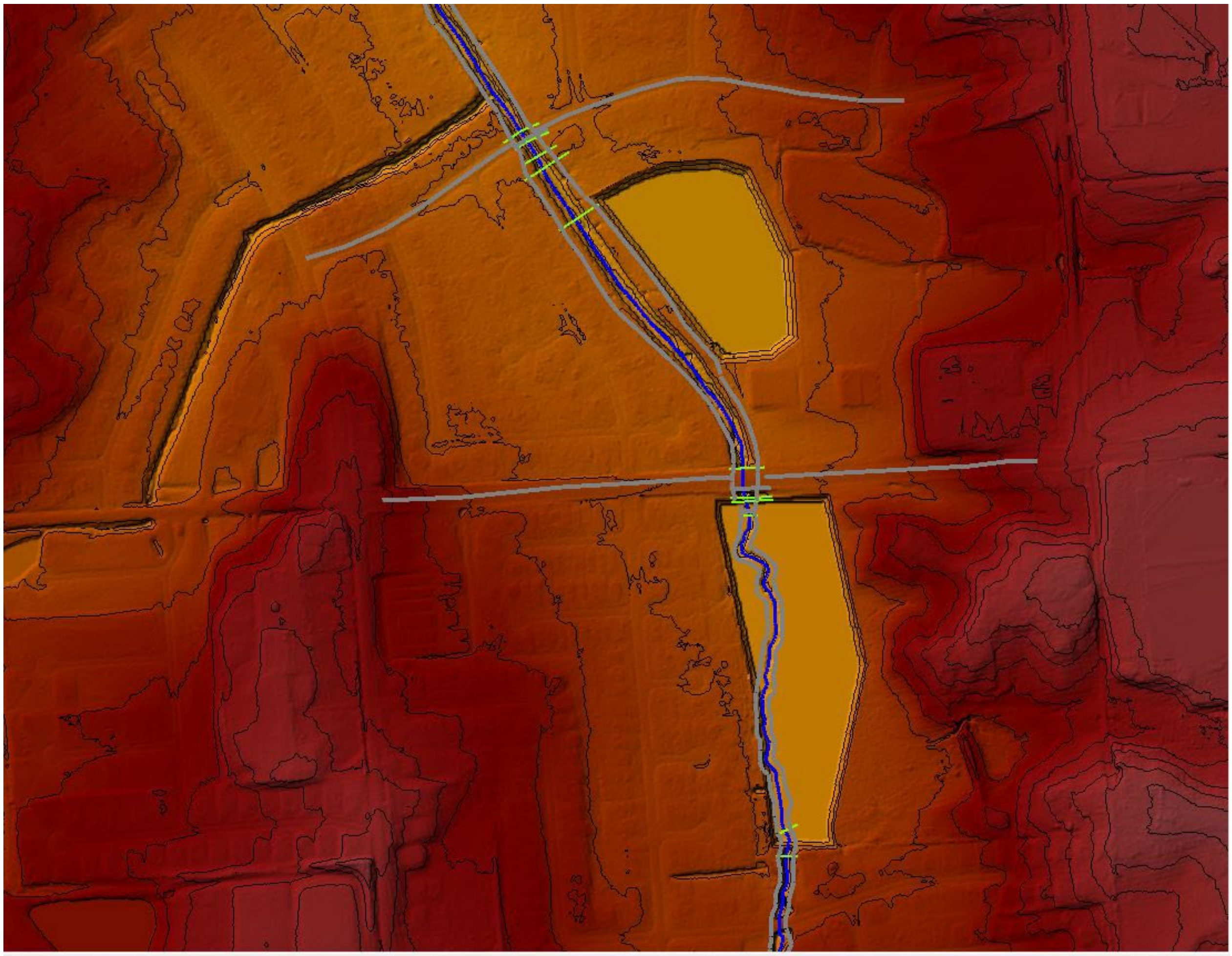


Figure 74. Cow Pen Creek Detention Pond Terrain Modification

6.10.2 Rocky Creek Detention Ponds at Swinnea Road

The Terrain file was altered to simulate detention on Rocky Creek. This was accomplished using ArcMap and storage volumes used in the 1D analysis. Figure 75 shows the results of the terrain modifications.

Figure 75. Rocky Creek Detention Pond Terrain Modification

A 1D reach extended through the detention pond to simulate low flows. The In-line structure option was used to regulate the outlet. Overtopping is expected at elevation 310 and a 4-foot diameter culvert releases low flows.

6.10.3 Lateral D Detention at Church Road.

The Terrain file was altered to simulate detention on Lateral D. A 1D reach extended through the detention pond. The lateral structure was used to divert flow into the ponds and the in-line structure option was used to regulate the outlet. A view of the modified Terrain data, stream alignment and structures used to model Lateral D are shown in Figure 76.

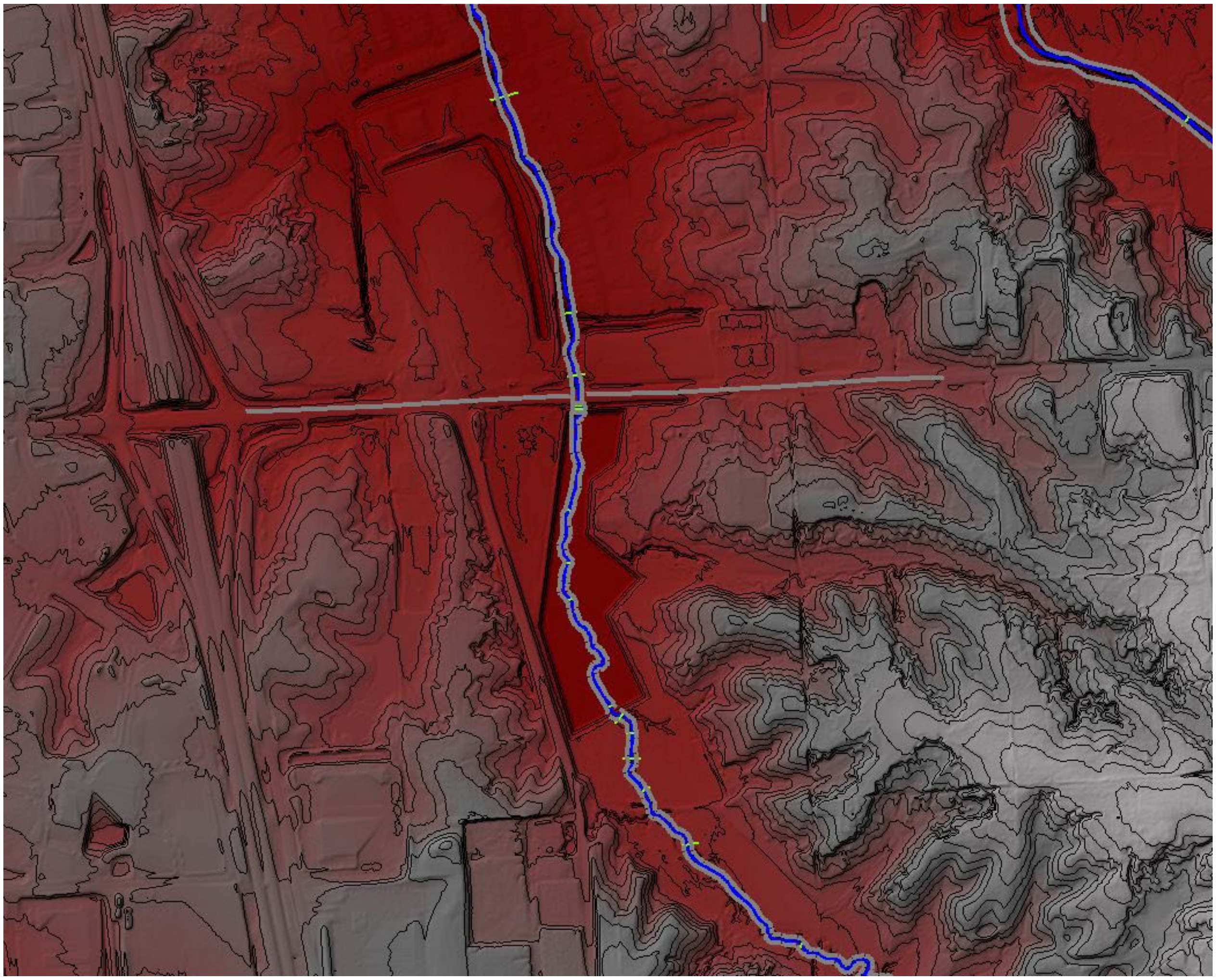


Figure 76. Lateral D Detention Pond Terrain Modification

The In-line structure option was used to regulate the outlet. Overtopping is expected at elevation 298 and a 4-foot diameter concrete culvert releases low flows.

6.10.4 Horn Lake Creek Levee Alternative (Preferred Recommended Plan)

The levee alternative was studied in more detail during the HEC-RAS 1D/2D analysis. The proposed levee and floodwall combination is approximately 3,000 feet and will primarily protect structures on the left-bank of Horn Lake Creek and west of Highway 51. As stated earlier, a review of historical flooding documentation indicates Horn Lake Creek typically exceeds its current capacity upstream of Goodman Road, flows westward overtopping Highway 51 and inundates the southwestern quadrant of “Bullfrog Corner”. Construction of the combination levee/floodwall will reduce the frequency and magnitude of inundation in this flood prone area. This alternative is the preferred recommended plan. The location of the levee is shown below in Figure 77.

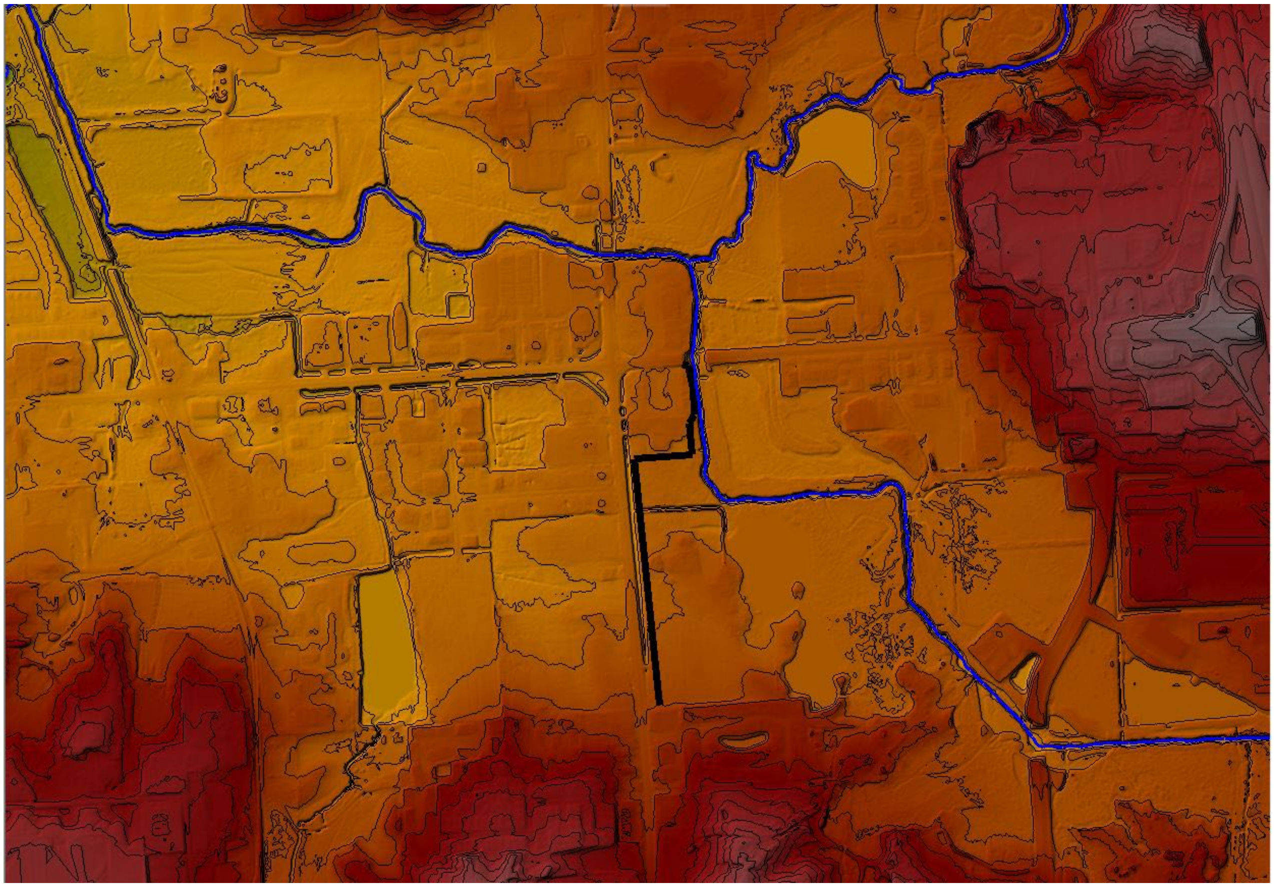


Figure 77. Levee Approximate Location

Cross sections of the floodwall and levee are shown below in Figures 78 and 79.

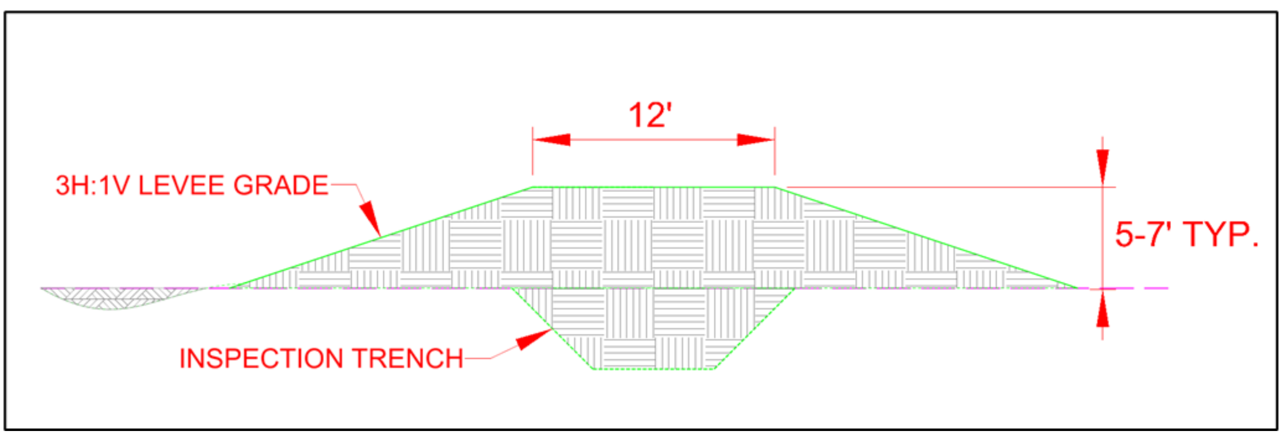


Figure 78. Levee Cross Section

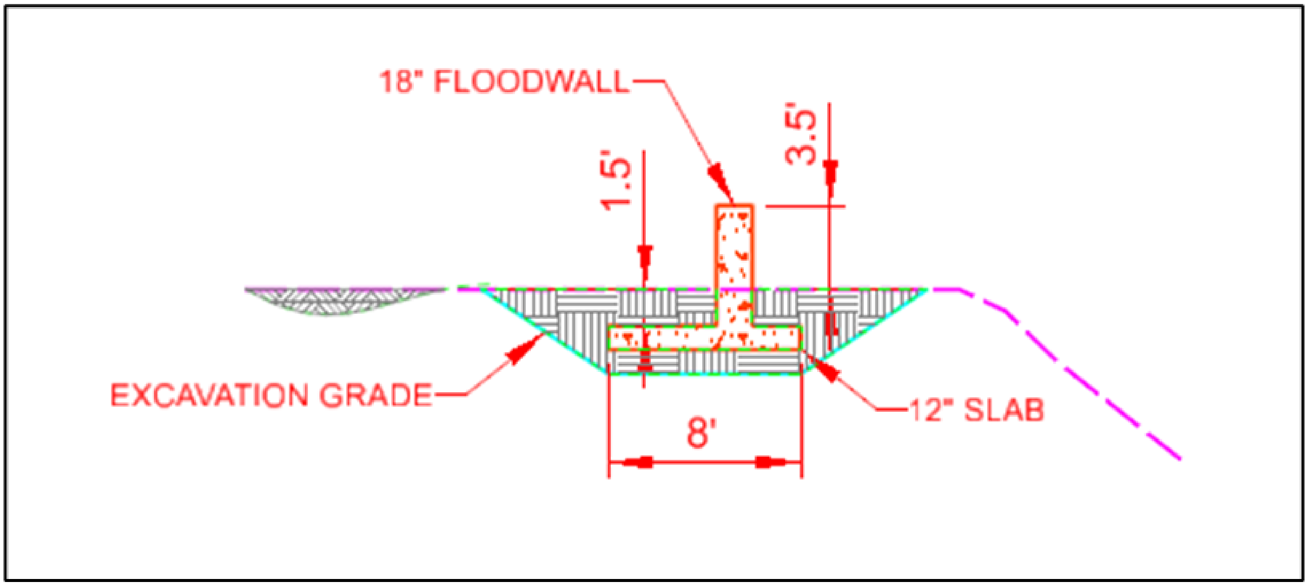


Figure 79. Floodwall Cross Section

6.10.4.1 Levee Height Optimization

The height of the proposed levee and floodwall was set to an elevation just above the 0.2% AEP flowline to provide the most reasonable level of protection. The levee height then underwent optimization in accordance with the process outlined in ECB 2019-8 *Managed Overtopping of Levee Systems*. The first optimization analysis considered 300-foot-wide managed overtopping set an elevation approximately equal to the 0.02 AEP flowline. This elevation was determined to be 273.3ft. The location of the overtopping was placed at a location that ensured overtopping flows would be conveyed through existing open channels downstream of the proposed levee. Managed overtopping of the 0.02 AEP flow is approximated to be at 50cfs and stay primarily confined to the existing conveyance channels on the protected side of the levee. AEP flowlines and levee profiles are shown in Figure 80 below. The red dashed line represents the levee crown elevation of Optimization Run #1.

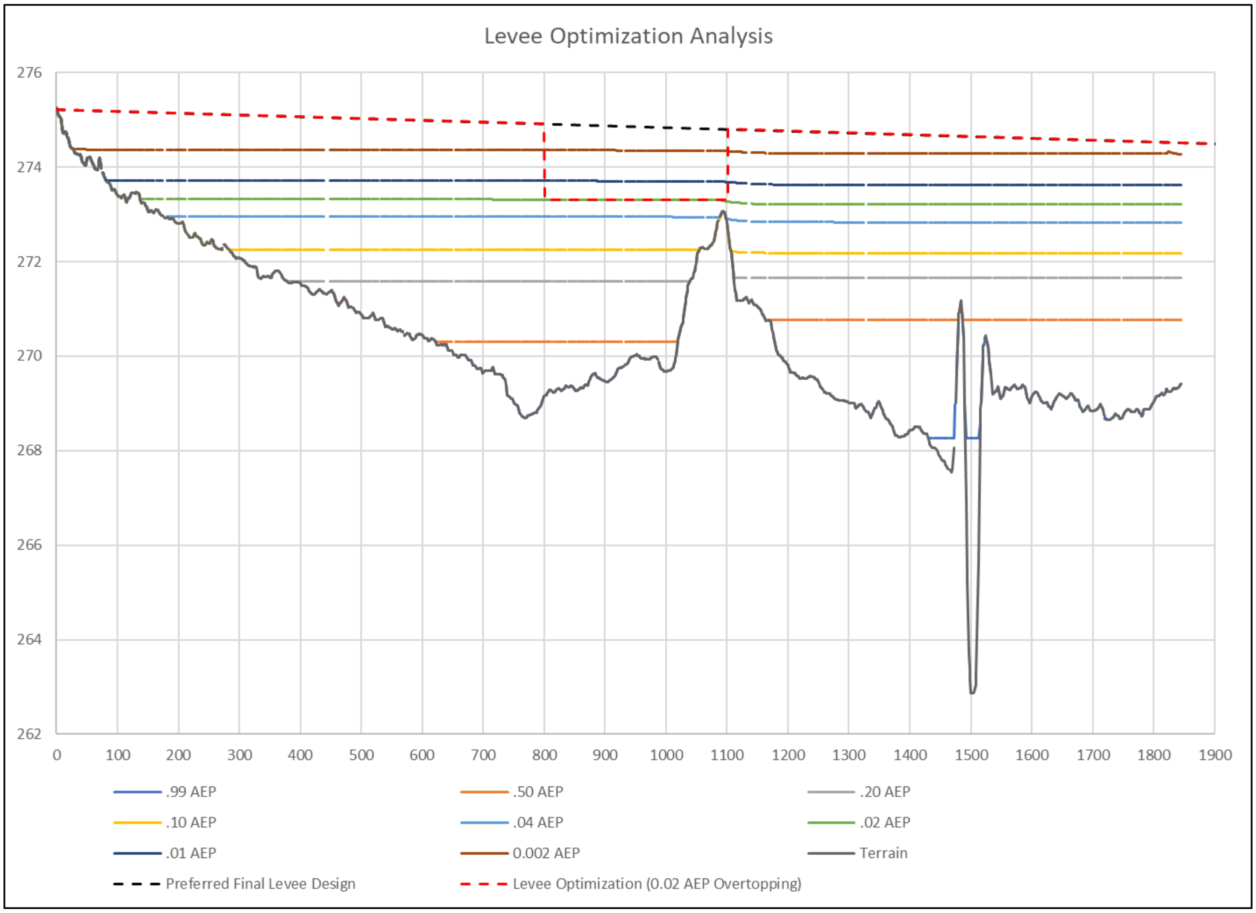


Figure 80. Levee Optimization Analysis.

Inundation maps produced from this optimization showed that managed overtopping at the 0.02 AEP flowline provided the most protection to the left bank downstream of the levee but did not reduce inundation along the right bank. At lower frequency flowlines the benefits were reduced as the managed overtopping inundated more of the protected left bank area. Comparison of results for the 0.04, 0.02, 0.01 and 0.002 AEP flows are shown in Figure 81. Continued optimization was determined to not be necessary after review of these results proved that the baseline BCR could not be exceeded with managed overtopping of the levee. These results and their impacts on costs and economic damages and benefits are discussed further in App. I (Design) and Appx. L (Economics).

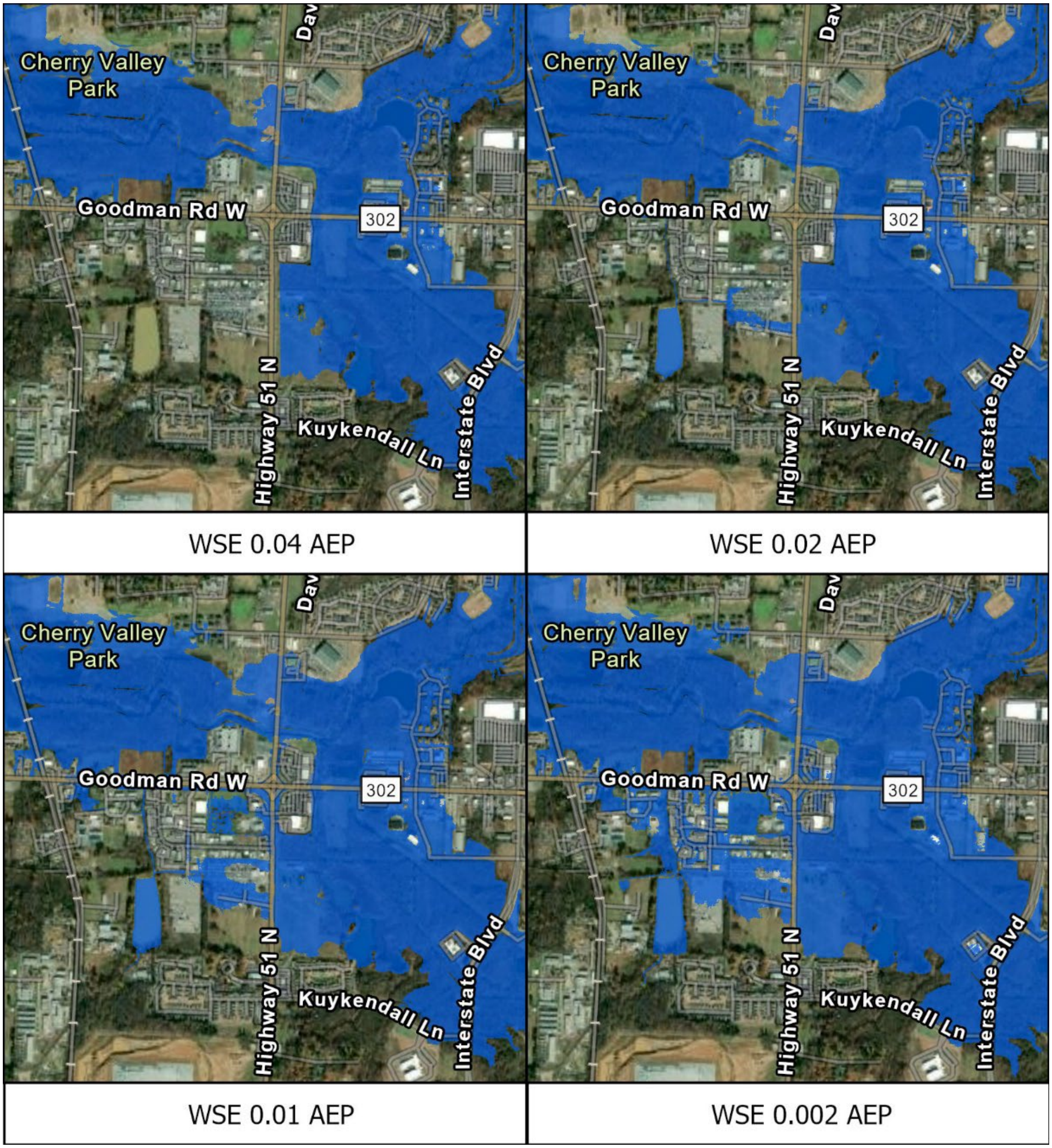


Figure 81. Comparison of Optimization Run #1 WSE for 0.04, 0.02, 0.01, and 0.002 AEP Flows.

6.11 FUTURE WITH AND WITHOUT PROJECT-HEC-RAS 1D/2D

A future condition with and without a constructed project was analyzed in order to evaluate impacts due to increases in urbanization which produce higher flows and shorter time to peaks for subbasin hydrographs. In order to estimate future flows, HEC-HMS was utilized to account for a 100 percent urbanized basin. These input flows were analyzed in HEC-RAS using the same geometry as the existing conditions model for a future without project and the project geometry to estimate a future with project. Inundation depth grids were provided to the Economics Project Delivery Team member. Increases in water surface were noticed in certain areas of the model but overall, the rise in water surface elevation was not seen as substantially different from the existing conditions model and the future with project model. This is since the majority of the Horn Lake Creek basin is urbanized leaving little area to increase runoff coefficients.

6.11.1 Future With and Without Project – HEC-RAS 1D/2D Results

Across all AEP events water surface elevations increase in both the future with and without project scenarios. Despite these increases, there is minimal increase to additional areas inundated by these flood events. Depicted below is a comparison of results from the most extreme event to show the minimal increase in area inundated and stage rise in the future scenario. Figure 82 shows the difference between the existing condition and the future without project results. Figure 83 shows the difference in the future with and without project results.

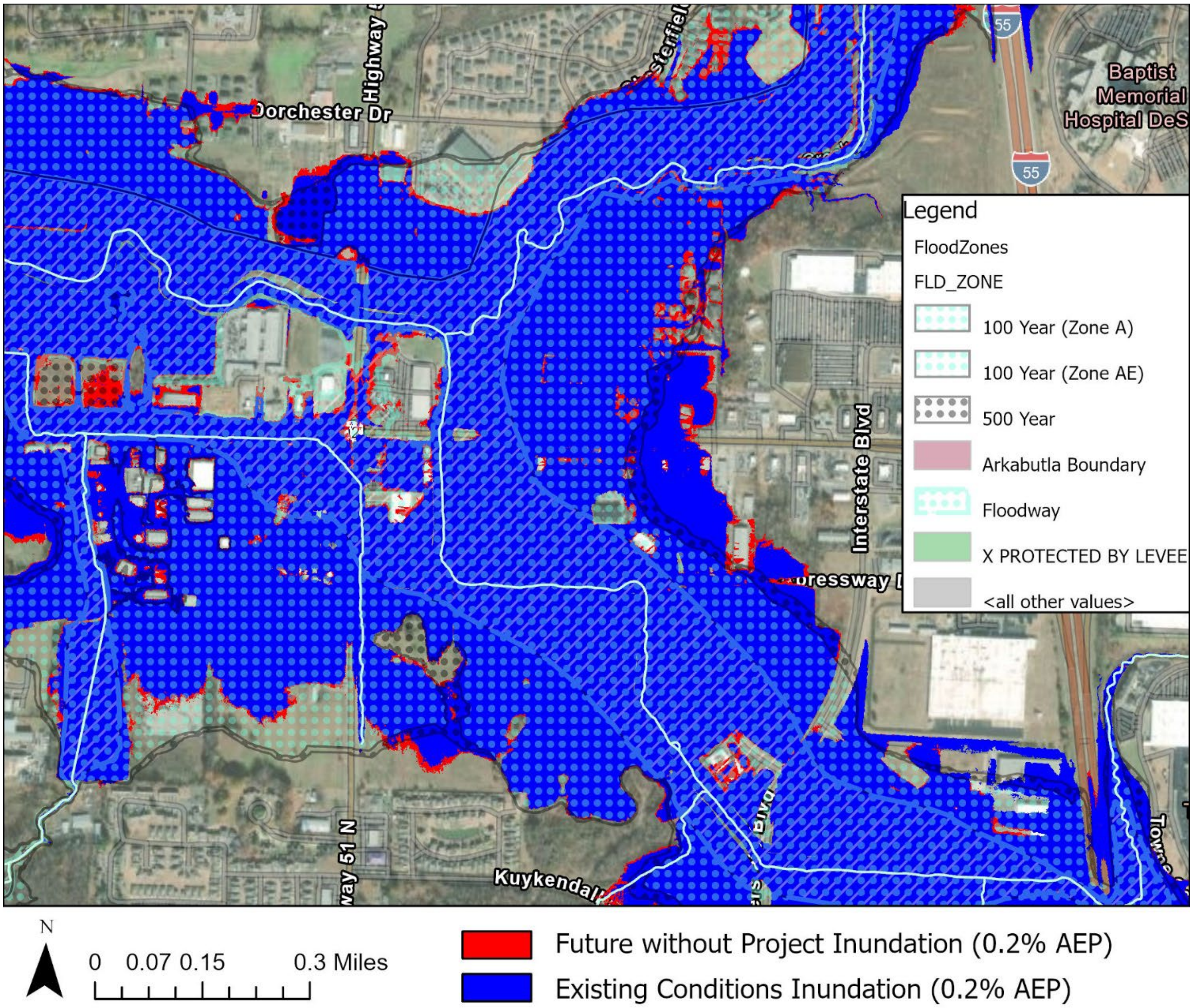


Figure 82. Comparison of Existing Conditions vs. Future Without Project WSE

As expected, the future conditions show increase in areas inundated. The stage increases along the edges of the inundation show increases ranging between 0 to 1ft.

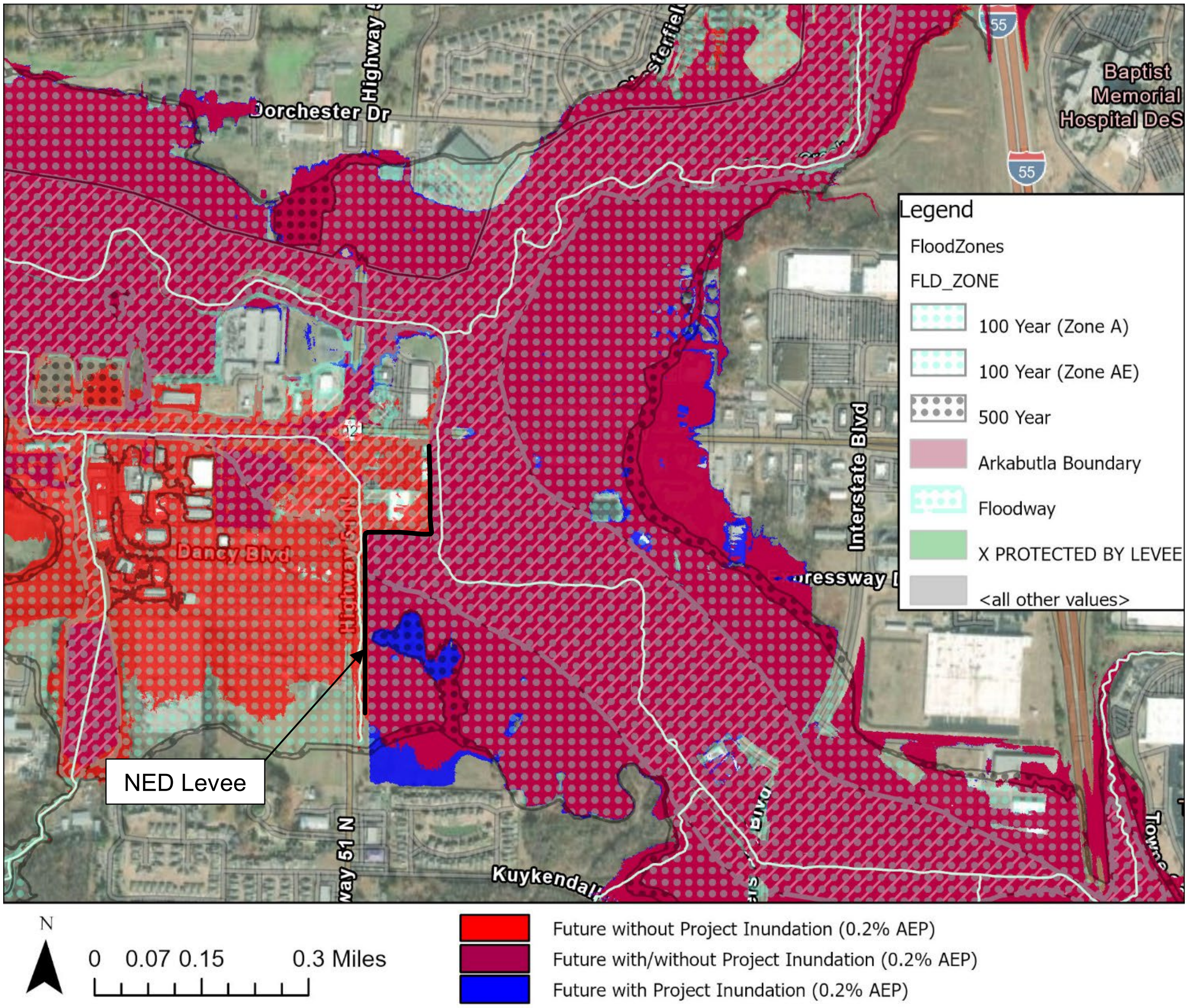


Figure 83. Comparison of Future With vs. Future Without Project WSE

Visually the differences in the future with the preferred plan levee show that water surface elevations are significantly higher along the proposed levee but are not impacting built out parcels along the left bank. The red area west of the levee and south of Goodman Road is the area seeing the greatest protection and benefits. Areas along the right bank are seeing slightly higher stages ranging from 0.25 to 0.8-feet increases. These depth grids are further analyzed in the economics analysis to determine damages and benefits over the project lifecycle.

6.12 RECOMMENDED PLAN (NED)-HORN LAKE CREEK LEVEE ALTERNATIVE

When the plan formulation process upgraded the analysis from the HEC-RAS 1D steady flow model to the HEC-RAS 1D/2D unsteady flow model, a more effective NED plan was determined to include only a levee and floodwall. The final recommended plan consists of a

3,000 linear foot levee and floodwall feature combined with a nonstructural aggregation to address residual flooding.

6.12.1.1 Stage Frequency for NED Plan Levee and Floodwall

The non-federal sponsor was briefed on all the changes to the original TSP selected and the determination of this alternative to be the best solution for reducing flooding at the target site of Bullfrog Corner. The non-federal sponsor was also made aware of potential increases in stage frequencies upstream of the proposed levee and floodwall and along Rocky Creek. Non-structural measures including dry-flood proofing were communicated to the non-federal sponsor and included in cost, schedule, and risk assessments that are discussed in the main report and economics and cost appendix. The figures below show the changes in stage frequency along Horn Lake Creek and Rocky Creek in the immediate vicinity of the Bullfrog Corner study site.

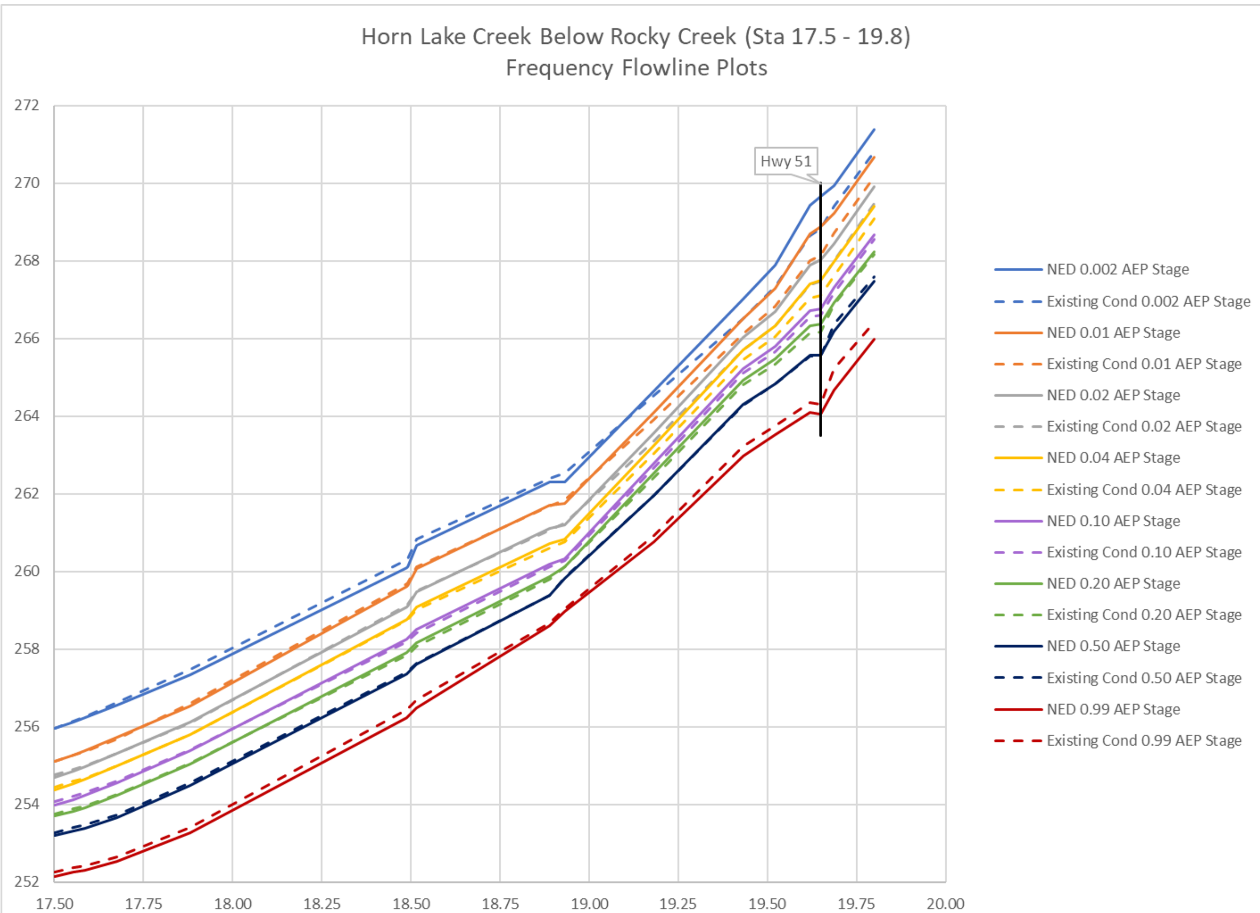


Figure 84. Frequency Flowline Plots for all modeled AEP events. Horn Lake Creek Sta 17.5 to 19.8 downstream of Rocky Creek.

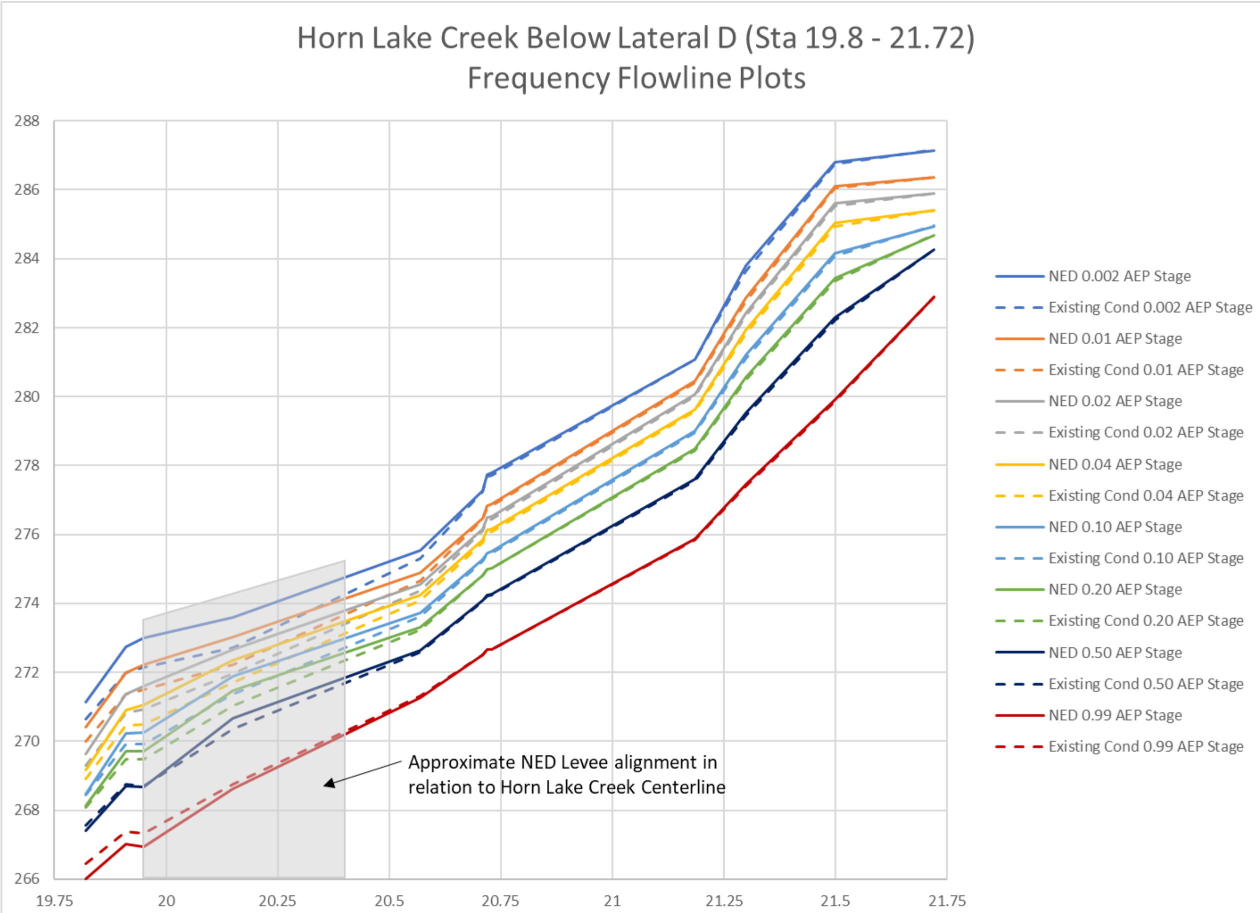


Figure 85. Frequency Flowline Plots for all modeled AEP events. Horn Lake Creek Sta 19.8 to 21.72 downstream of Lateral D.

The NED plan initiates a rise in water surface elevations due to prevention of floodwaters from accessing the left overbank on the southwest corner of Goodman Road and Hwy 51. This rise is discussed in the following sections inundation maps and in the Economics Appendix where impacts from the NED plan are discussed. The impacts are seen along the right descending bank of Horn Lake Creek opposite the NED levee and partially along Rocky Creek just upstream of its confluence with Horn Lake Creek. These areas and properties are currently prone to flooding during low frequency events. The NED levee potentially raises the water surface elevations on these structures over the current condition on the order of magnitude from 0.0-0.5 feet. Standard construction practices for structures in flood prone areas is to raise the finish floor elevations to be above the flood plain elevation. The LiDAR overbank used to map inundation has scrubbed building finish floor elevations, a standard procedure, and is not providing an accurate accounting of potential damages as many structures may be raised above modeled stages. The team rectified this by obtaining surveyed finish floor elevations and conducting interviews with property owners and store workers who were present during the September 2014 flood of record. The results of the finish floor survey and interviews removed many structures from potential damages. The structures that remained in this area designated as prone to flooding will be eligible to receive voluntary dry floodproofing.

Stages downstream of Hwy 51 return to existing conditions levels relatively quickly, within 0.5 miles. Overbank flooding has been reviewed to show that there are no potential damages to structures in this area. There is no anticipated transfer of risk to properties downstream of the NED levee.

The residual risk was assessed and determined to be minimal - and appropriate for the study phase- though there are accuracy limitations that must be better understood and addressed prior to implementation. The residual risks are accounted for in cost contingencies and non-structural remediations strategies.

Error in the LiDAR dataset elevations used for the hydraulic modeling and economic calculations were determined to be accurate to ±0.5ft averaged over a wide area, but error increases as the area of concern decreases or as vegetation and structures increase. Appendix I provides greater detail regarding the analysis of damages and additional survey data that was obtained to calculate damages and mitigation strategies.

Further refinement of the 2d overbank and more accurate survey data in this area should be considered during PED to further identify inducements and remediation costs. Prior to implementation, economic calculations should consider finish floor elevation surveys to accurately determine structures' risk of flooding.

6.12.1.2 Inundation Mapping for NED Plan Levee and Floodwall

Since the channel enlargement and detention storage are no longer incorporated in the preferred plan, only inundation pertinent to the levee and floodwall alternative is presented below. The following figures show the inundation maps developed for the NED plan for the 0.10, 0.02, 0.01, and 0.002 AEP events.

The NED levee and floodwall were designed to divert stages and flows up to the 0.002 AEP event. Any event greater than the 0.002 poses risk to the design and protection of property and structures behind the levee and floodwall. This residual risk has been communicated to the non-federal sponsor. During the PED phase of this project further analysis should be performed to determine the level or residual risk and if additional and alternative measures should be taken to add further protection to properties and structures protected by the NED levee and floodwall.

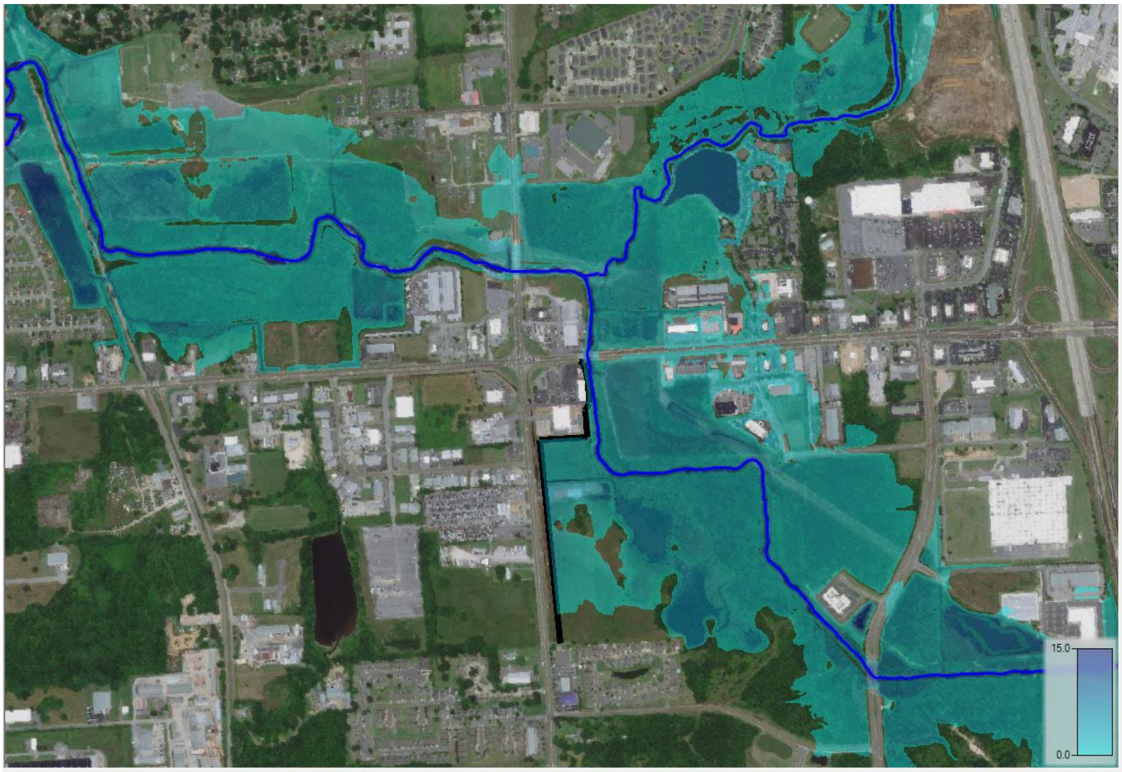


Figure 86. 0.10 AEP Levee Alternative Inundation Depth Map-Bullfrog Corner

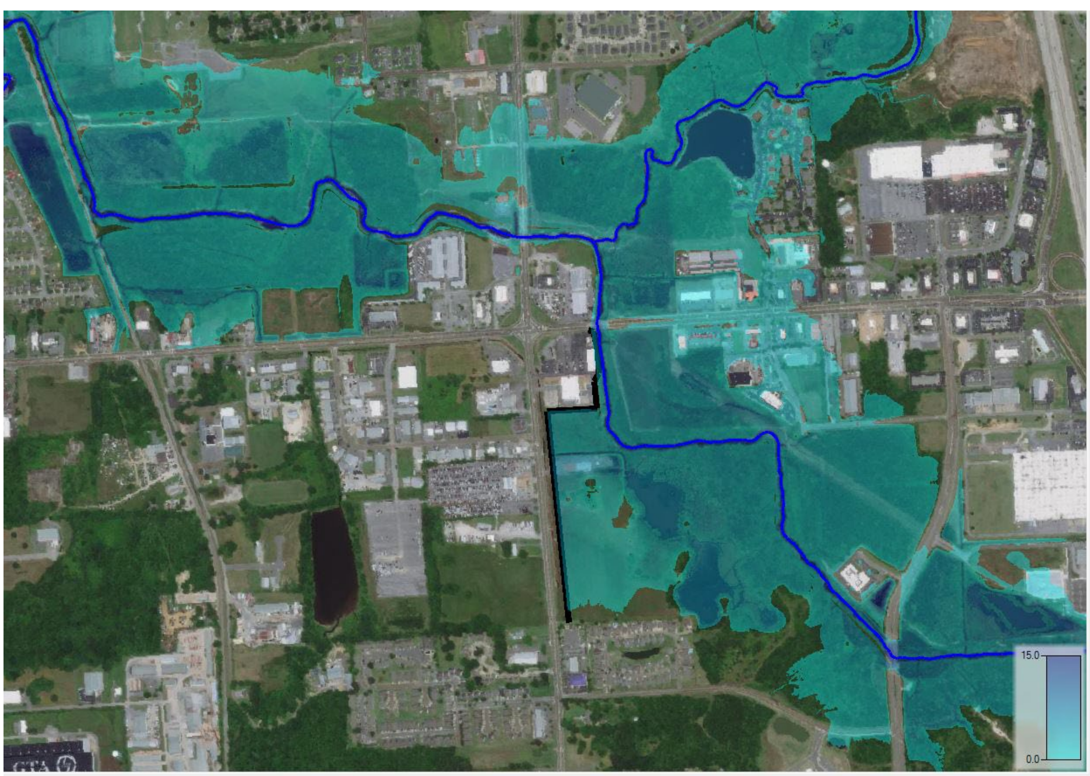


Figure 87. 0.02 AEP Levee Alternative Inundation Depth Map-Bullfrog Corner

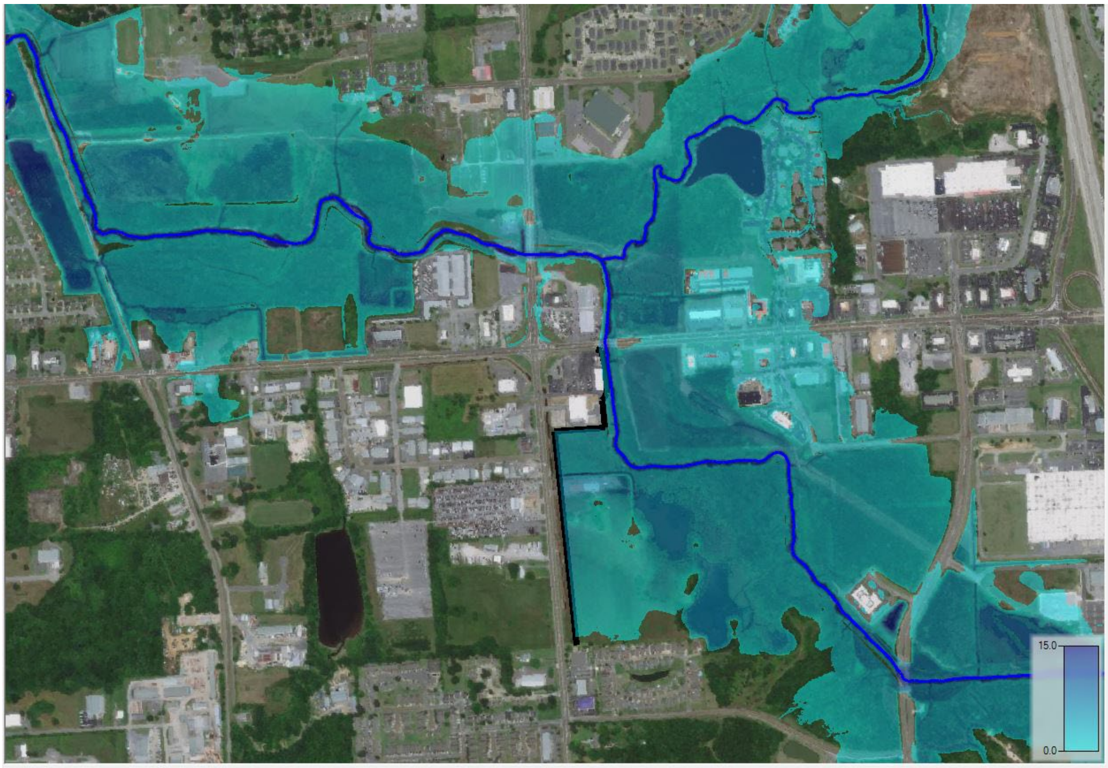


Figure 88. 0.01 AEP Levee Alternative Inundation Depth Map-Bullfrog Corner

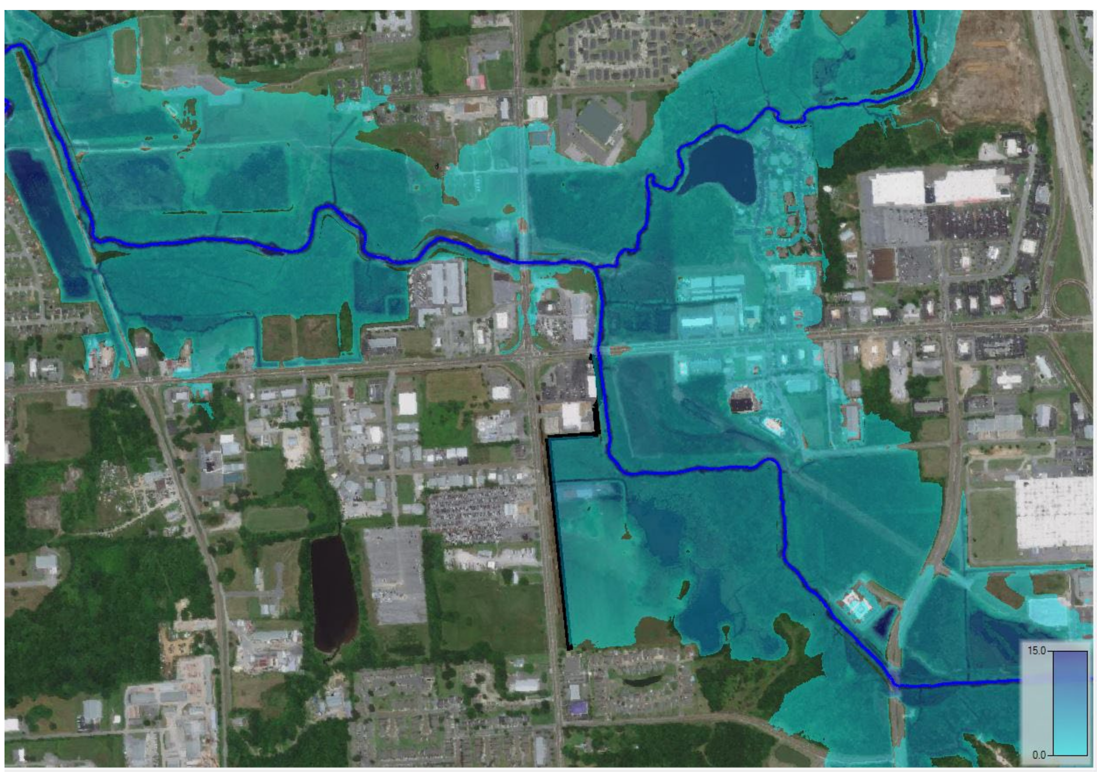


Figure 89. 0.002 AEP Levee Alternative Inundation Depth Map-Bullfrog Corner

6.12.1.3 Areas Requiring Additional Analysis

Review of the hydraulic model results along Rocky Creek between Rasco Road and Swinnea Road indicates inconsistency and low confidence in the inundation mapping of the area. Several attempts were made to edit the geometry by adding and removing culverts that connect Rocky Creek with the overbank flow areas. Little improvement was seen. The model results indicated that at certain index locations the future with project stages were significantly lower than the future without project stages. This is contrary to what would be expected in an area that is over 1.5 miles upstream of a tributary from the project location. The expected result would be no change or a slight increase. During Agency Technical Review the stage results were noticed and related to a group of structures in the area being included in the inventory of structures receiving benefits. The economics team reviewed this and determined that their exclusion would provide insignificant changes to benefits and no change to the BCR calculations.

During hydraulic modeling this area showed insignificant impact to the results in and around the NED project site which is of the highest concern and has the highest economic impact. Seeing minimal benefit from further refinement of this portion of the model to the evaluation of the NED plan efforts were halted. This report notes a low level of confidence in model results and inundation for this portion of the model along Rocky Creek. Overall and at the NED project site a high confidence remains as the results fall in line with expected results showing incremental increases in stages as AEP events become less frequent and between with and without project conditions. It is recommended that the portion of the model along Rocky Creek received further refinement as the project moves to PED. Checks should be made to determine the impact of Rocky Creek on the recommended NED levee and floodwall heights.

Section 7

Ecosystem Restoration-Engineering, Research and Development Center (ERDC) Assessment

The ecosystem restoration analysis was conducted by Engineering Research and Developmental Center (ERDC) in coordination with the Memphis District PDT Environmental member. Pertinent and detailed information of their efforts and findings are presented in the Appendix A, B and C of this report.

These Appendices outline restoration alternatives based on the field site and "FluvialGeomorph" (FG) assessments. The assessments provided background for developing the watersheds stabilization plans. According to the reports, the plans are developed in two phases, Phase I-Stabilization Alternatives and Phase II-Adaptive Management options for further bank stabilization and habitat enhancements. The Channel Evolution Model (CEM) was also used to supplement the analysis. Details related to its application and theory are explained in more detail in the references.

The primary ecosystem restoration objective is to restore and protect aquatic and riparian ecosystems by decreasing channel slopes and stabilizing bank lines which would improve transport of stream flows and sediment. The initial screening criteria was to retain, for further evaluation, those streams that were considered as degradational. Streams were evaluated using LIDAR and Geographic Information Systems (GIS) data. If a stream was identified stable (i.e., stable plan form geometry), it was screened out for ecosystem restoration.

Initial discussions with the sponsor and field visits allowed the PDT to identify nine streams that were degradational. The NER plan includes a bank stabilizing system of 74 grade control structures (GCS) coupled with 328 acres of riparian restoration on eleven streams (Camp, Cane, Hurricane, Johnson, Lick, Mussacuna, Nolehoe, Nonconnah, Red Banks, and Short Fork Creeks) as shown in Figure 86 below.

The efforts performed as part of this study were preliminary in nature. Some limitation of the current analysis includes limited field investigations which forced the PDT and experts to rely heavily on interpretation of LiDAR. Previous modeling efforts allowed for analysis of grade control structures on Horn Lake Creek during this study. A more rigorous evaluation of the impacts to overall stability of the system and water surface profiles is highly encouraged for future phases of this project on both the grade control structures presented in this report as well as the grade control structures proposed for all other streams.

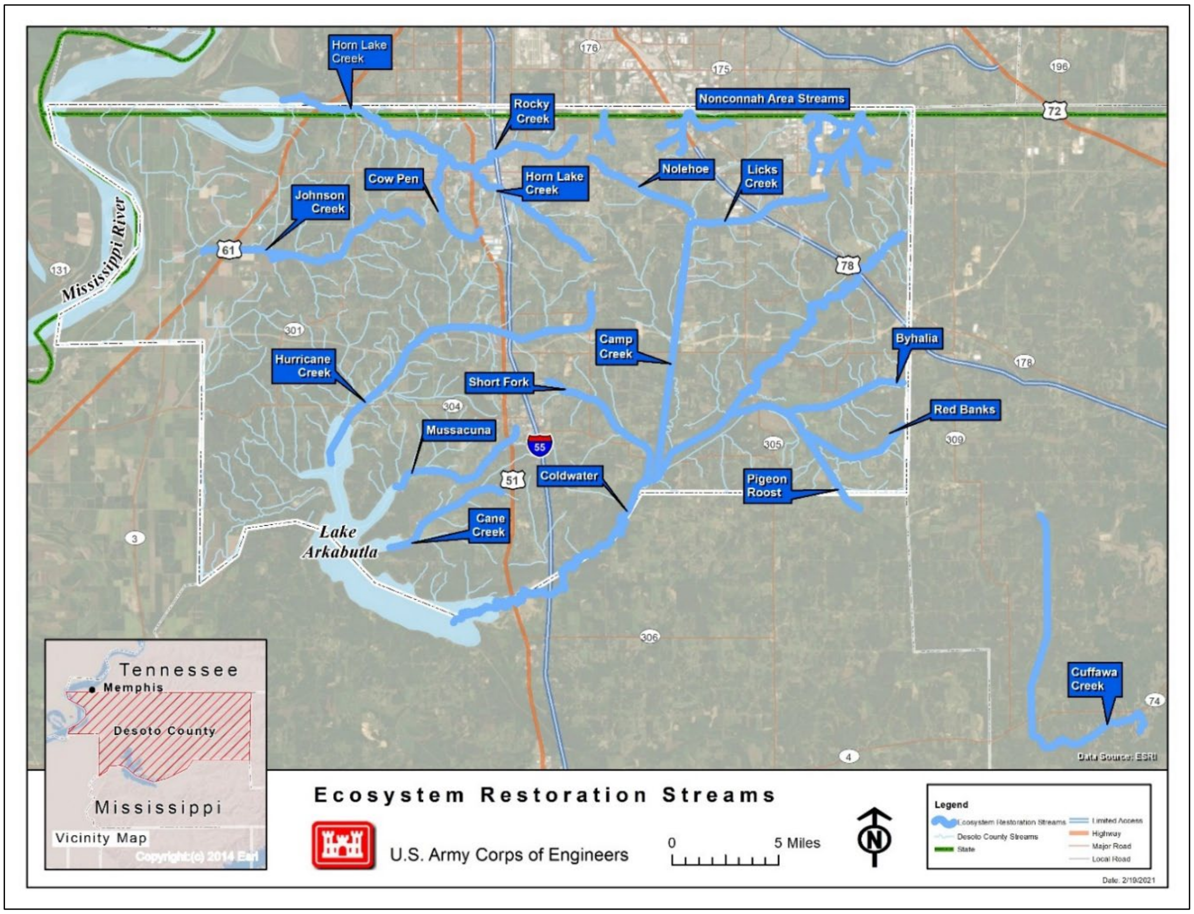


Figure 90. Ecosystem Restoration Streams

7.1 HORN LAKE CREEK GRADE CONTROL STRUCTURE LOCATION DETERMINATION

Originally 82 structures were proposed as part of the NER, 14 of those were located on Horn Lake Creek. Analysis was performed on these structures and is presented in this report. After environmental evaluation was performed on these structures it was determined that more tree clearing was necessary for construction than benefits gained.

Since HEC-RAS models were available on Horn Hake Creek and the GCSs have a potential to impact features of the FRM plan formulation process, it became the model stream to assess NER impacts. The locations of proposed and existing Horn Lake Creek GCSs are shown in Figure 88 below.

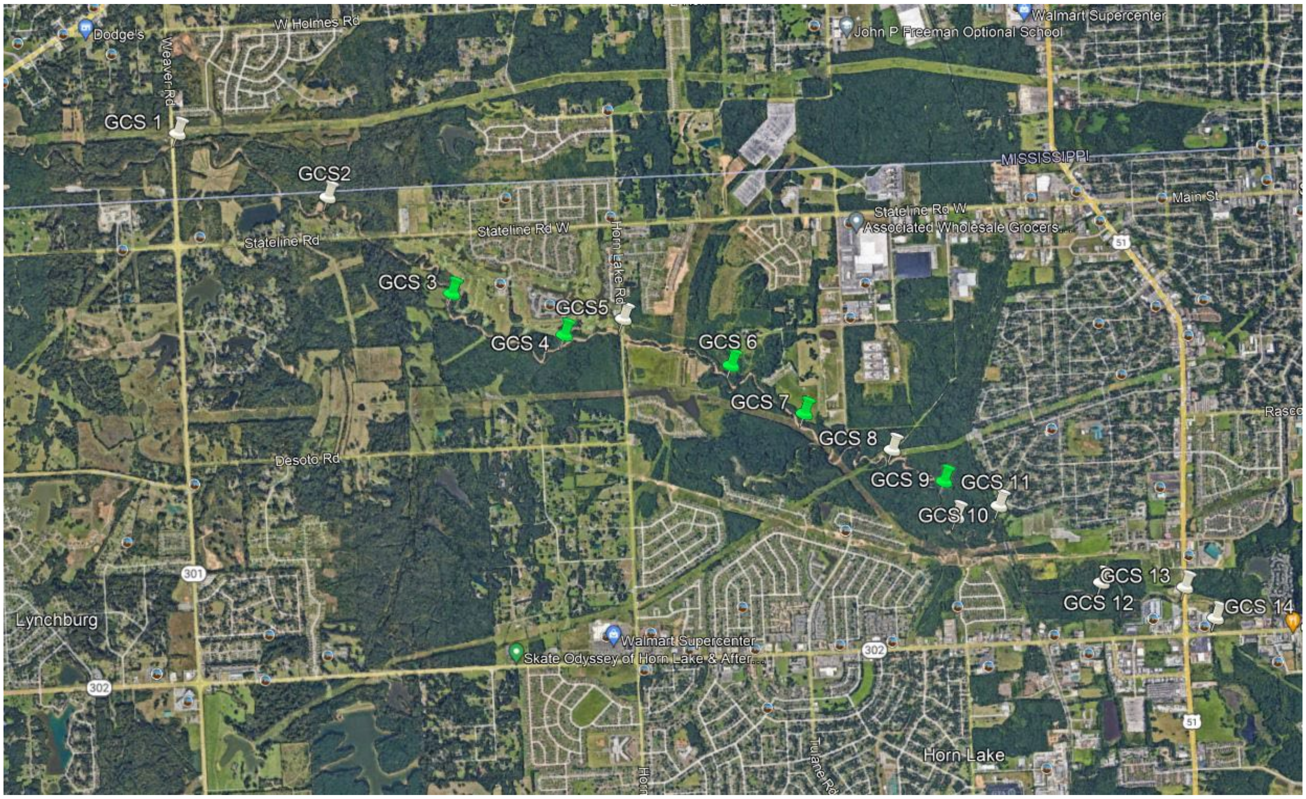


Figure 91. Horn Lake Creek Grade Control Structures

One of the primary goals of the NER feature is to minimize channel degradation, channel erosion, and sedimentation to support aquatic ecosystem form and function. The preliminary field investigations along Horn Lake Creek suggest that the long-term stability of the creek is directly dependent on the continued functionality of the existing grade control structures along the channel.

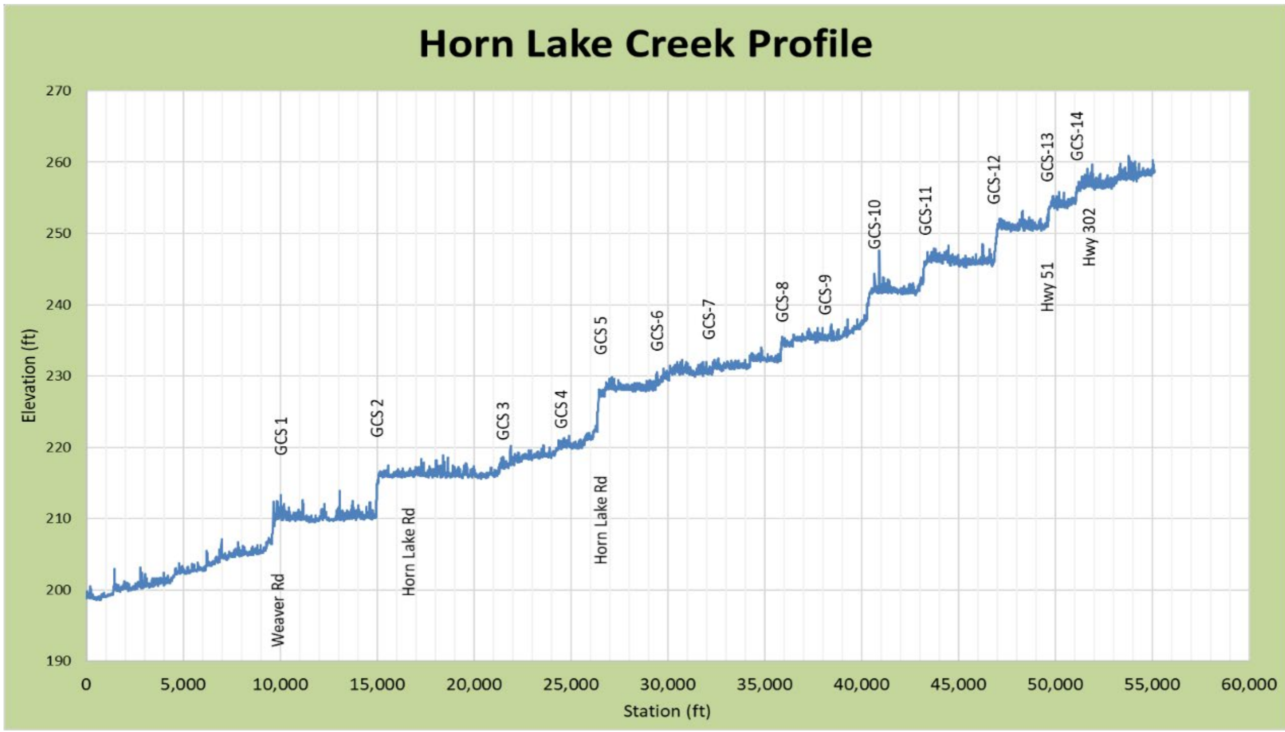


Figure 92. Profile View of Horn Lake Creek GCSs

Figure 89 shows the Horn Lake Creek profile is controlled by numerous grade control type structures. Most of these structures are components of bridges and culverts along the stream, but several appear to be associated with pipeline crossings. While these structures are currently controlling the grade of the channel system, many of these appear to have been designed without adequate regard for engineering and geomorphic considerations. As a result, ERDC feels many have a higher likelihood of failure, which could be catastrophic to the geomorphic and environmental character of the channel system.

Therefore, the primary recommendations for Horn Lake Creek should include rehabilitation or replacement of these existing structures. Stabilization of meanders that could endanger these structures should also be an important feature of the Horn Lake Creek Plan.

7.2 HEC-RAS 1D/2D GRADE CONTROL STRUCTURE ASSESSMENT

Over the 50-year project life, the structures will retain/detain sediment resulting in a decrease in the available channel capacity and potentially increase the frequency and magnitude of flooding. To ensure the proposed new structures did not result in negative impacts, they were modeled in HEC-RAS 1D/2D using the Inline Structure option. Additional cross sections were added upstream and downstream of the In-Line Structure to capture the velocities and other pertinent information near the structure (see schematic below).

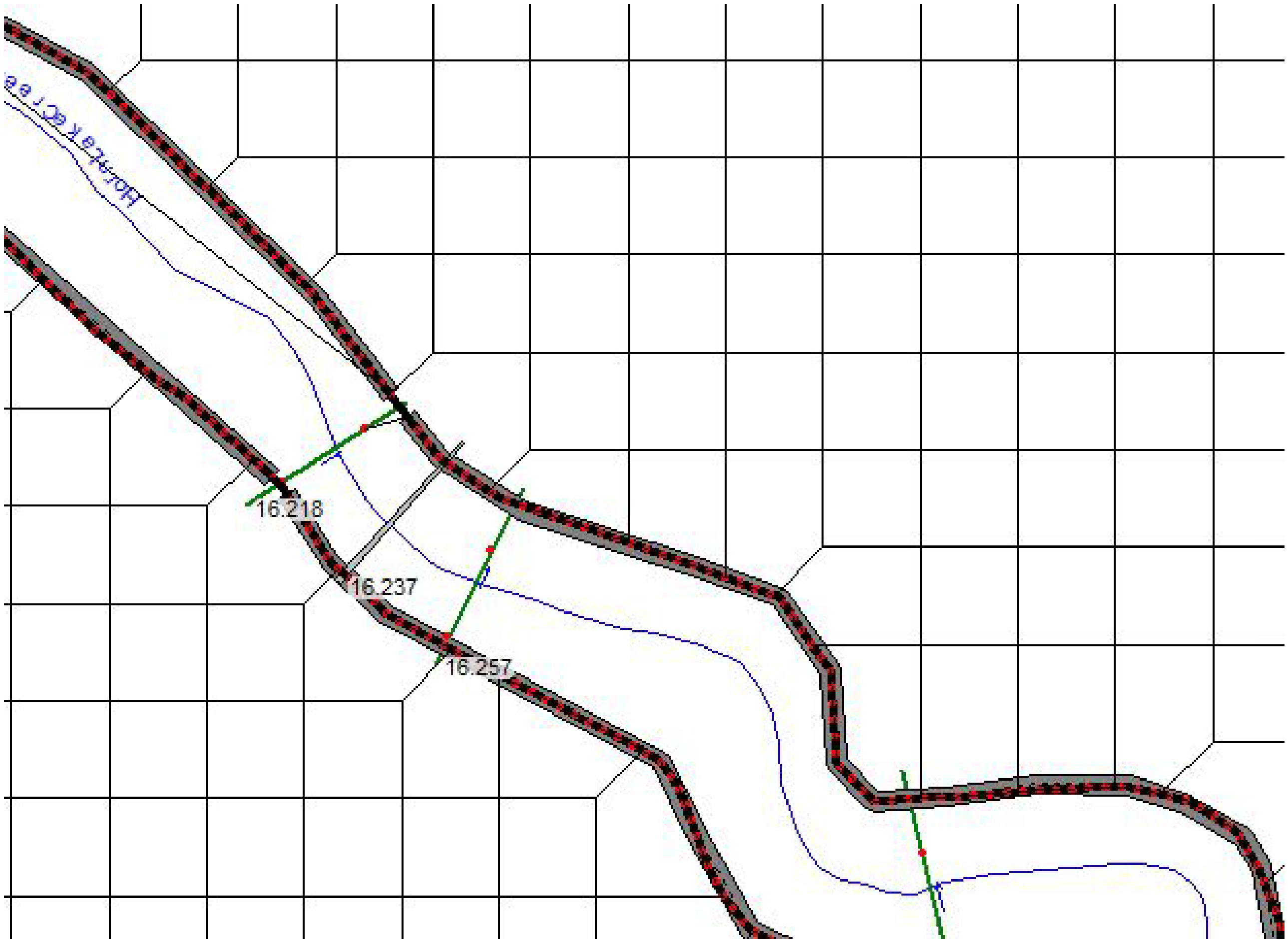


Figure 93. Addition of cross sections for GCS analysis

Since the current model was a HEC-RAS 1D/2D, the Inline Structure option was easily applicable in the 1D environment. The profile below shows a depiction of the sediment accumulation at the In-Line structure after the 50-year period and how the bed slope might adjust.

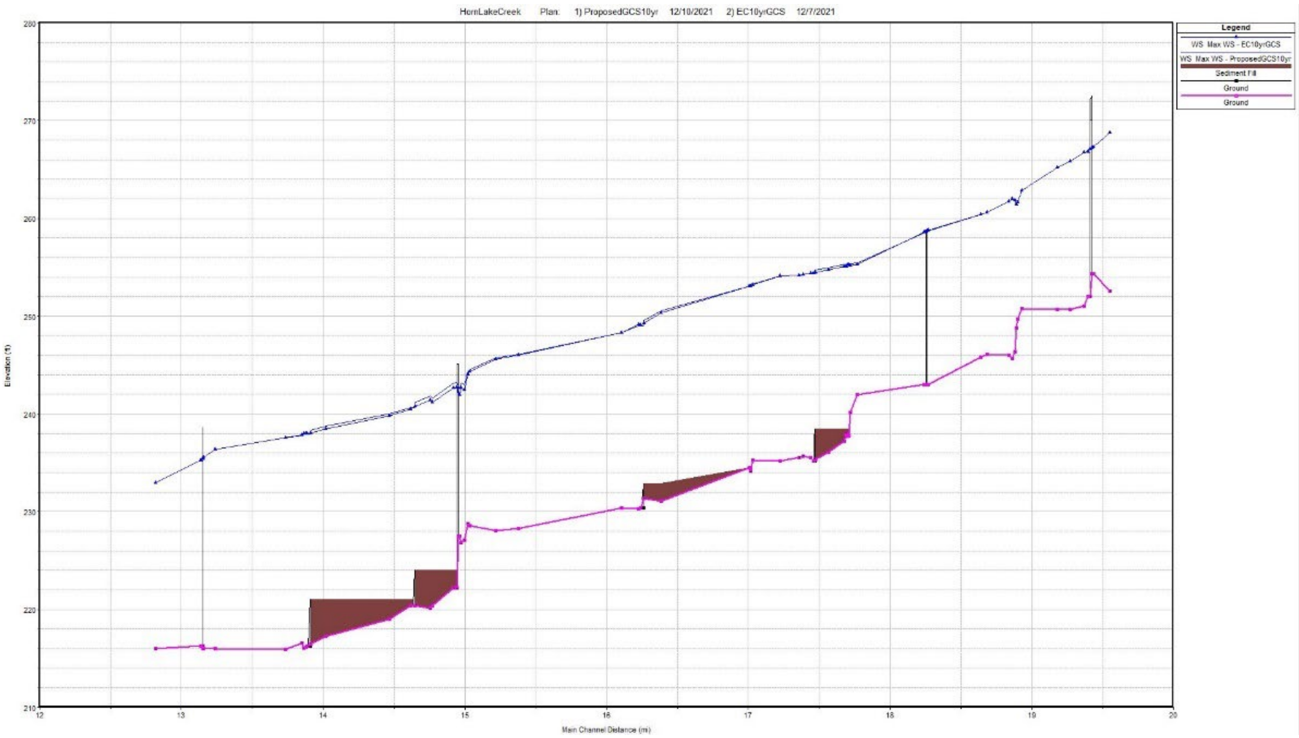


Figure 94. HEC-RAS Inline Structure Locations w/Simulated Sediment Deposition

HEC-RAS model modification was not conducted at the existing stream crossings and existing riprap protection. With the perceived stability issues at the existing structures, it was decided to model them “as-is” using the current LiDAR. The final design of these 9 structures will be undertaken in PED phases and will be based on updated field surveys. Existing condition HEC-RAS 1D/2D hydrographs, used for the FRM analysis, were input into the model and a comparison of “with” and “without” GCSs conditions was conducted.

The impacts of the structures are shown on the inundation maps in Figures 92 to 94. Based on the preliminary results, which reflects the 50-year sediment accumulation, the only areas showing a significant increase in water surface elevations were above GCS 3 and GCS 4. As shown in the figures, this magnitude ranges from 0.2 to 0.4 feet for the 0.50 AEP event. The magnitudes for the other AEPs were computed to be less.

Table 9. Change in WSE at GCS 3 and 4 for 0.50 to 0.01 AEP events.

Horn Lake GCS 3 and 4 WSE Change			
Frequency	Existing	With GCS	Difference
2 year	237.71	238.01	0.3
5 year	238.5	238.58	0.08
10 year	239.08	239.27	0.19
25 year	239.84	240	0.16
50 year	240.55	240.7	0.15
100 year	241.08	241.22	0.14

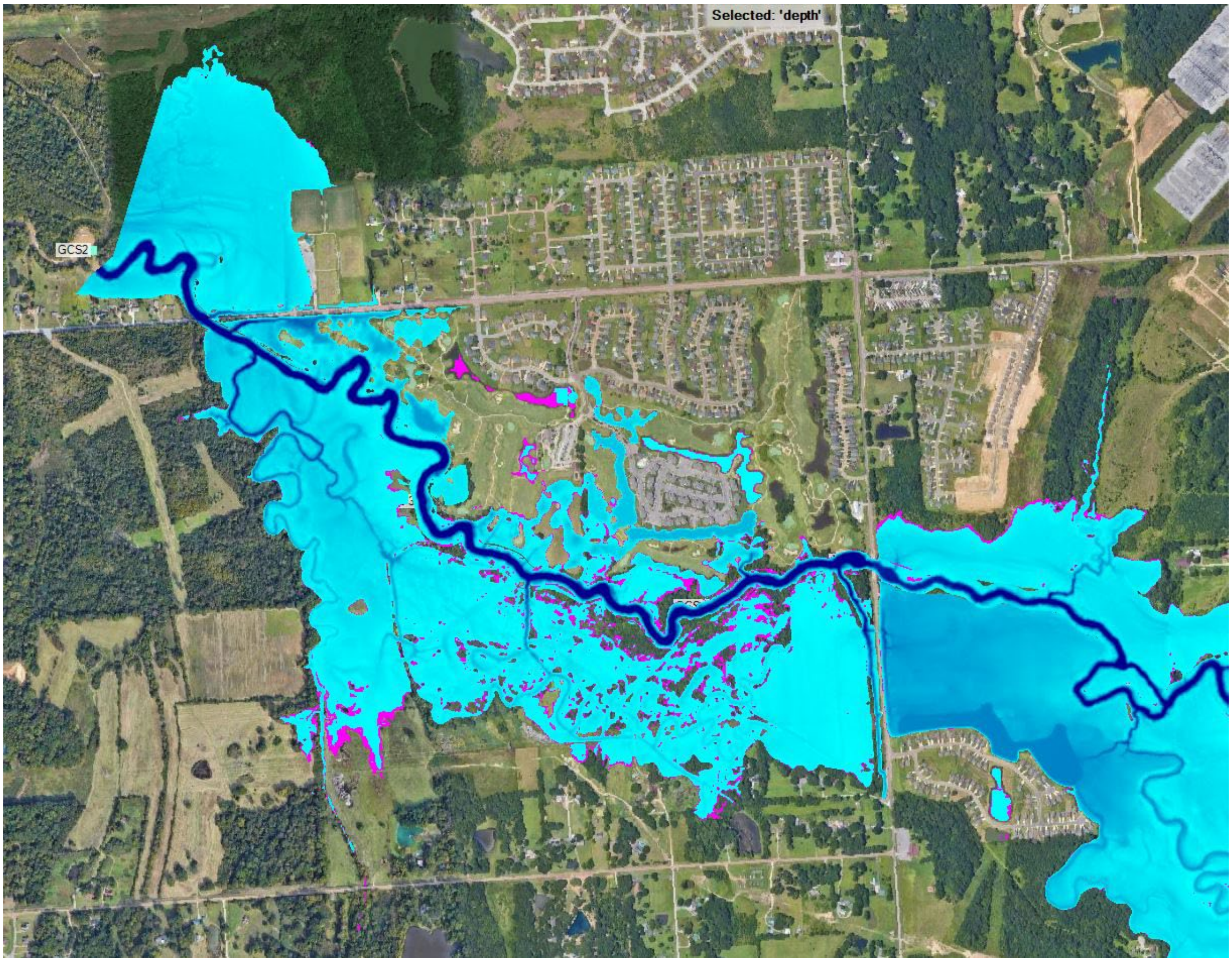


Figure 95. 0.50 AEP Existing (Blue) vs. GCS (Pink) Inundation Comparison Above GCS 3 and 4

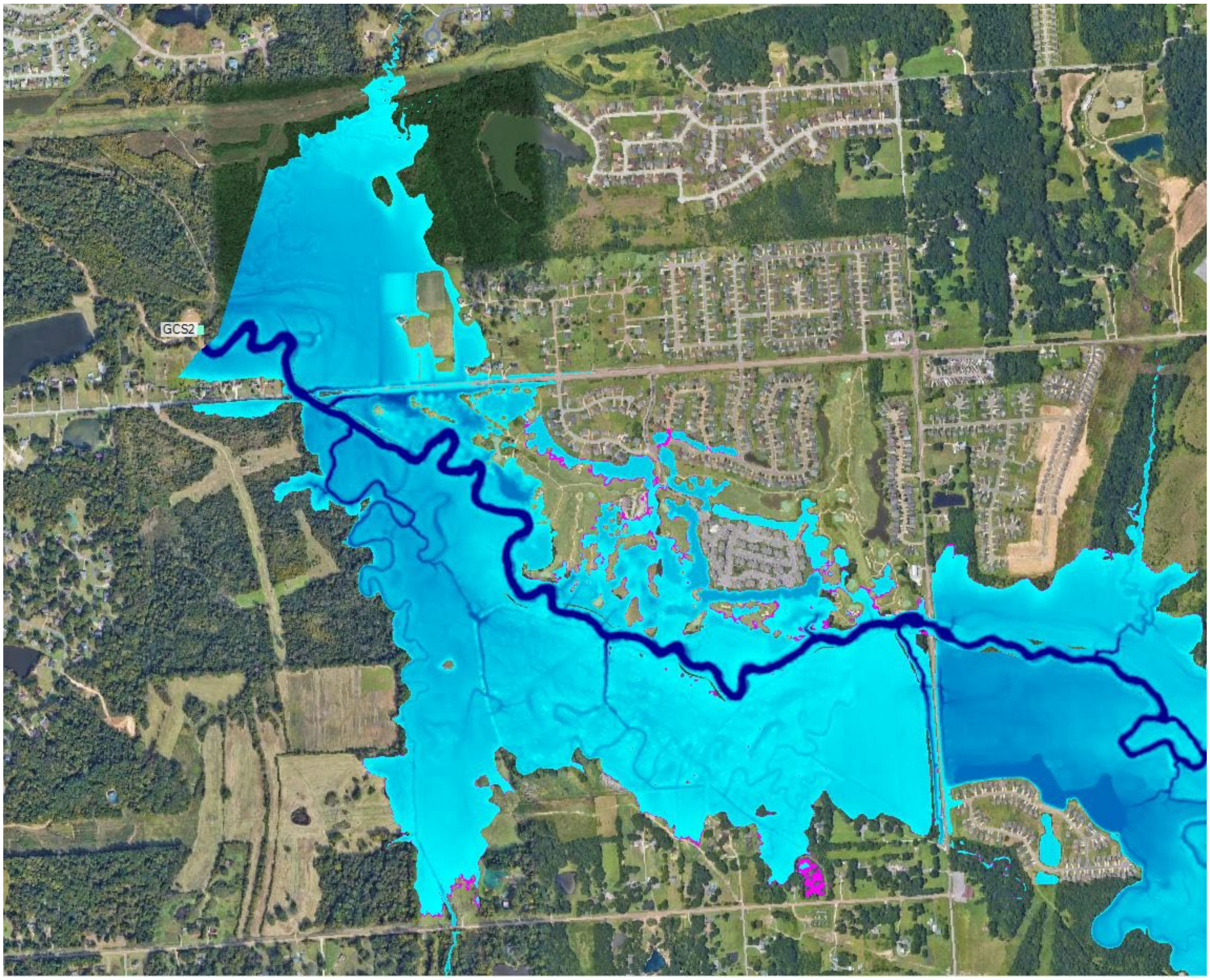


Figure 96. 0.10 AEP Existing (Blue) vs. GCS (Pink) Inundation Comparison Above GCS 3 and 4

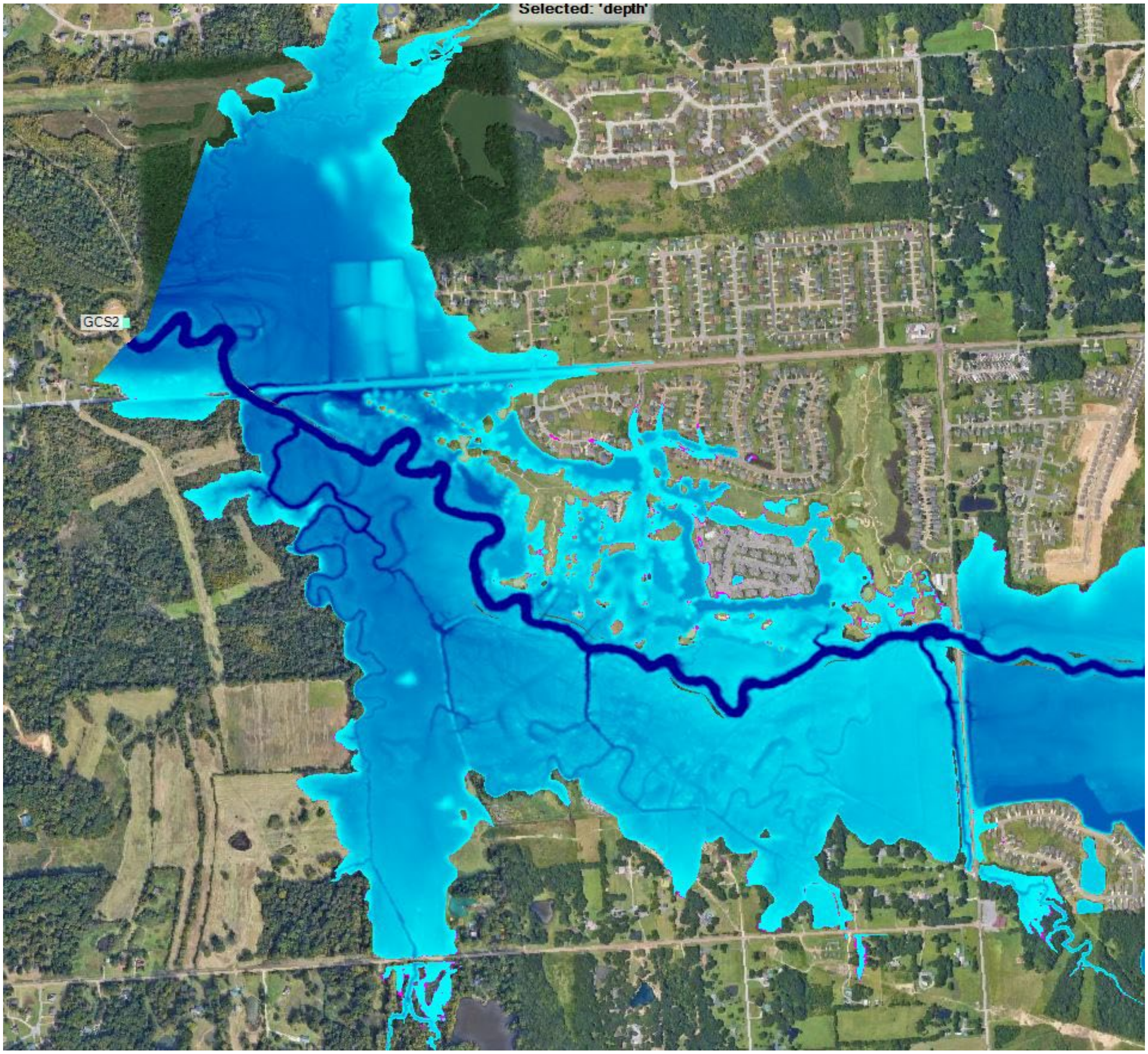
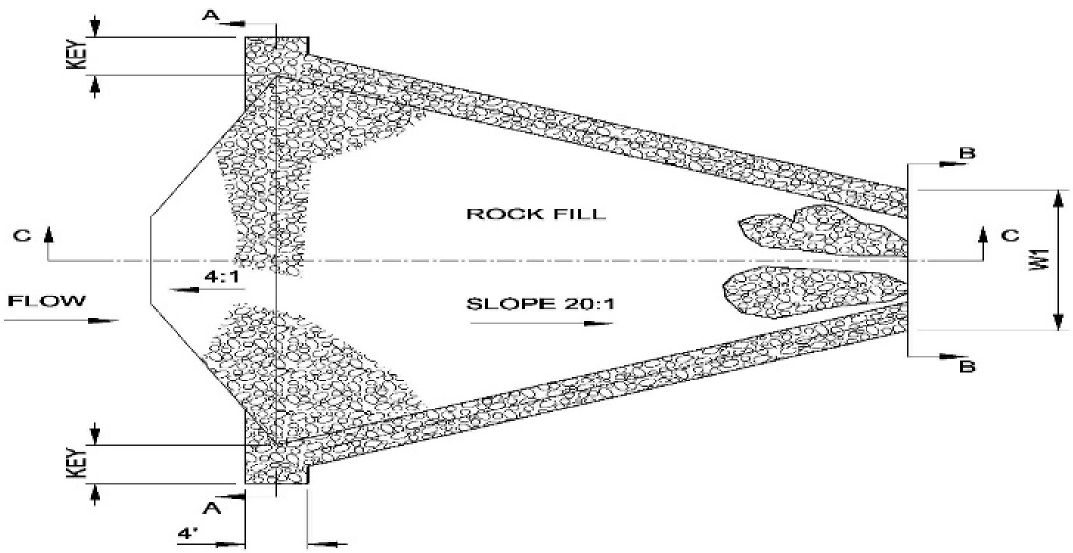


Figure 97. 0.01 AEP Existing (Blue) vs. GCS (Pink) Inundation Comparison Above GCS 3 and 4

7.3 GRADE CONTROL DESIGN CRITERIA

All proposed grade control structures will be sloping riprap structures. Each ramp will be supplemented with bank stabilization treatments consisting of longitudinal stone toe protection with tiebacks or keys to prevent flanking and undermining. These structures have recently been constructed by ERDC for several ecosystem restoration projects and details can be found in the Appendices A, B, and C. A plan view of a typical ERDC structure is shown below in Figure 95.



NOT TO SCALE

ERDC-CHL
ROCK RIFFLE DETAILS

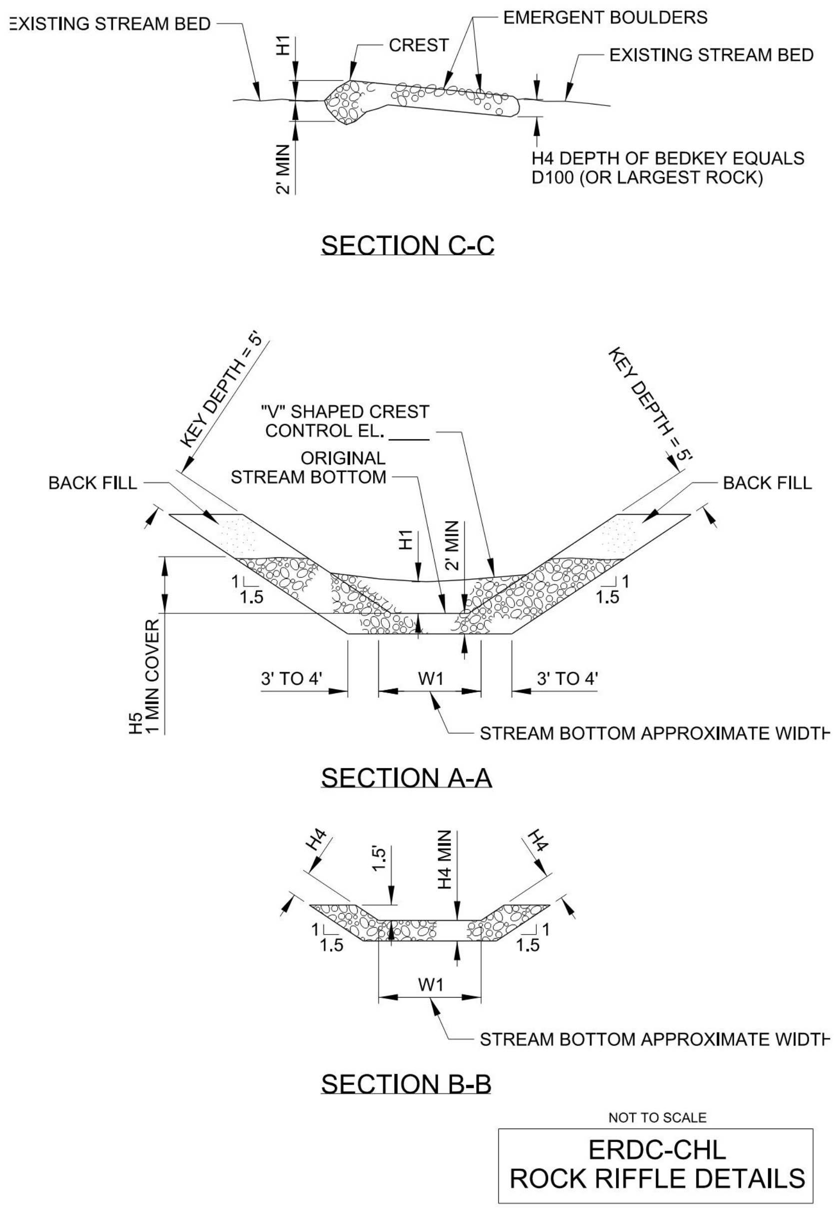


Figure 98. Plan View- ERDC Typical Rock Riffle Structure

This design was adapted from ERDC loose rock riffle, with additional slope armor and keys to account for the erodibility of local soils. Final structures will be approximately 3.5 feet above the channel bottom at the time of construction. Larger stone will face upstream, with smaller 200-pound stone protecting the downstream side. Side slope armoring and keys will reduce the risk of flanking or undercutting the structure.

HEC-RAS 1D channel results, combined with RAS-mapper velocity outputs, indicate the velocities are less than 9.1 feet per second. The design riprap gradation recommended for the GCSs is R650 for the overflow section of the structure. According to the Isbash

Equation, R650 is stable for velocities less than 9.1 feet per second. Upper bank paving will consist of R200 gradation. Tables 9 and 10 below show the flow frequencies and velocities anticipated at each new grade control structure. For the feasibility design these results have been extrapolated out to all proposed grade control structures for the purposes of design and cost estimation.

Table 10. Flow Frequencies at New Grade Control Structures

Grade Control Structure Name	Frequency Flows					
	2 yr	5 yr	10 yr	25 yr	50 yr	100 yr
GCS 3	6,295.00	6,753.22	6,632.58	3,632.42	6,161.27	6,020.48
GCS 4	7,297.20	8,747.96	9,146.03	7,568.32	10,315.41	10,141.62
GCS 6	5,605.93	6,213.88	6,521.06	7,021.05	7,502.71	7,690.31
GCS 7	6,199.14	6,633.34	6,997.02	9,841.22	8,140.18	8,350.26
GCS 9	2,998.72	3,377.54	3,436.60	6,578.91	3,657.40	4,449.33

Table 11. Maximum Velocities Anticipated at New Grade Control Structures

Grade Control Structure Name	Frequency Velocities					
	2 yr	5 yr	10 yr	25 yr	50 yr	100 yr
GCS 3	5.26	5.54	5.45	5.49	5.55	5.46
GCS 4	6.42	7.08	7. 26	7.57	7.78	7.91
GCS 6	5.86	5.88	6.04	6.19	6.05	5.91
GCS 7	7.18	7.62	7.9	8.14	8.25	8.29
GCS 9	4.59	4.9	4.88	4.87	4.78	4.8

Table 11 presents the Isbash standard riprap gradations that were used to determine the appropriate riprap sizing for the grade control structures. The structures will be a minimum of 3.5 feet high off the channel bottom. More information of the design of these structures is presented in Appendix I.

Table 12. Isbash Standard Riprap Gradations

	GRADATION NORMALLY PRODUCED MECHANICALLY							GRADATIONS NORMALLY REQUIRING SPECIAL HANDLING					
			R90		R200	R400	R650		R1500	R2200			R7400
Layer Thickness in Inches HIGH TURBULENT FLOW	12	15	18	21	24	30	36	42	48	54	63	72	81
Layer Thickness in Inches Low Turbulent Flow			12	14	16	20	24	28	32	36	42	48	54
Percent Lighter by Weight													
100	25 10	50 20	90 40	140 60	200 80	400 160	650 260	1000 400	1500 600	2200 900	3500 1400	5000 2000	7400 3000
50	10 5	20 10	40 20	60 30	80 40	160 80	280 130	430 200	650 300	930 440	1500 700	2200 1000	3100 1500
15	5 2	10 5	20 5	30 10	40 10	80 30	130 40	210 60	330 100	460 130	700 200	1100 300	1500 500
Velocity, fps, HIGH TURBULENCE	5.3	5.9	6.7	7.1	7.5	8.4	9.1	9.8	10.5	11.1	12.0	12.8	13.7
Velocity, fps, low turbulence	7.4	8.3	9.3	9.9	10.4	11.7	12.7	13.6	14.6	15.6	16.8	17.8	19.1

Approximate riprap quantities are based on the configurations shown in Figures 96 and 97.

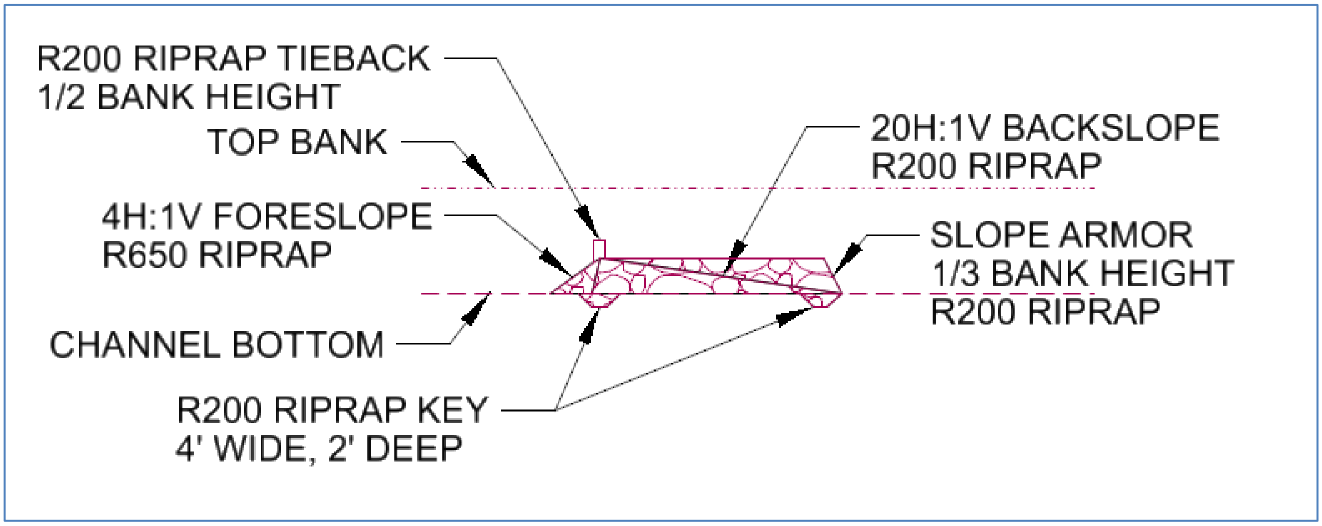


Figure 99. Conceptual Profile View-Grade Control Structure

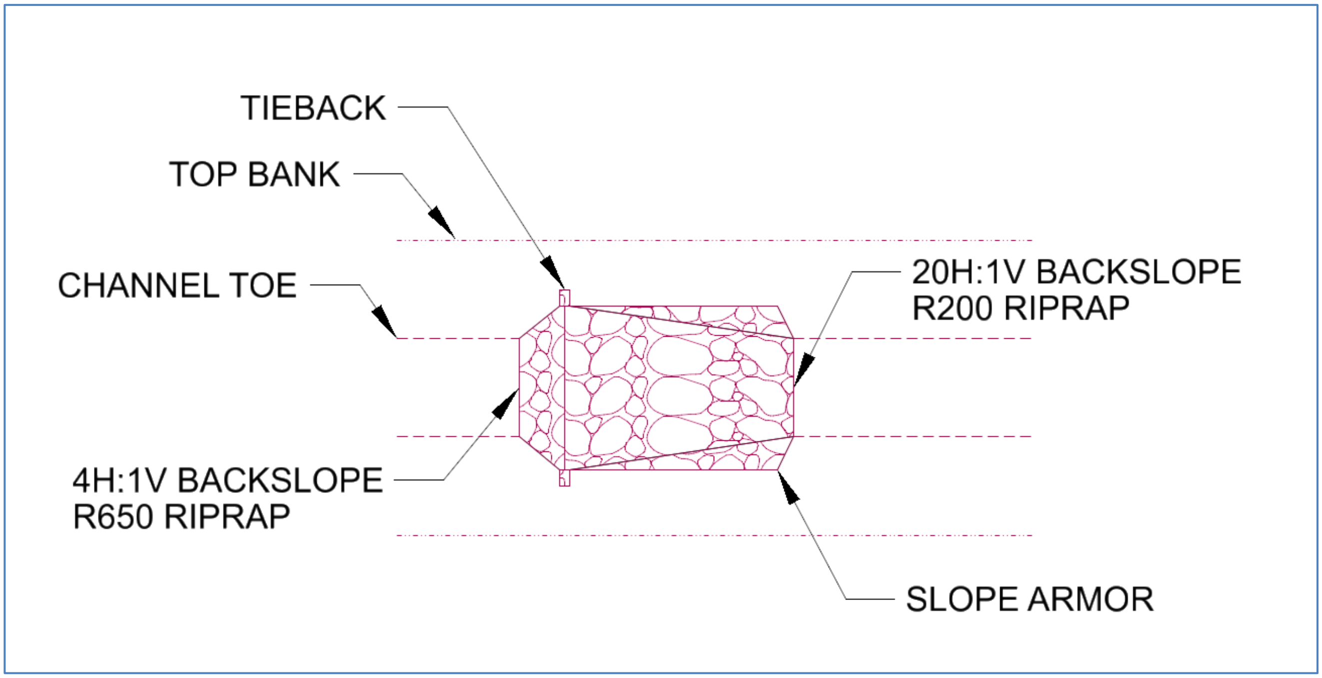


Figure 100. Conceptual Plan View-Grade Control Structure

A total of 5 new structures and the rehabilitation of 9 structures are proposed. The total bank stabilization reach is approximately 20,000 feet. A summary of riprap details for these structures are shown in Table 12.

Table 13. Grade Control Structure Riprap Details

Control Site	Type of Construction	Linear feet of bank stabilization
GCS-1	Rehab existing structure	0
GCS-2	Replace existing structure	2,000
GCS-3	**New Structure**	1500
GCS-4	**New Structure**	1500
GCS-5	Rehab existing structure	200
GCS-5a	Replace existing structure	800
GCS-6	**New Structure**	1,500
GCS-7	**New Structure**	1,500
GCS-8	Replace existing structure	2,200
GCS-9	**New Structure**	1,200
GCS-10	Replace existing structure	2,500
GCS-11	Rehab existing structure	1,000
GCS-12	Replace existing structure	1,500
GCS-13	Minor rehab of existing structure	1,000
GCS-14	Rehab existing structure	1,500
Total		19,900

Section 8
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