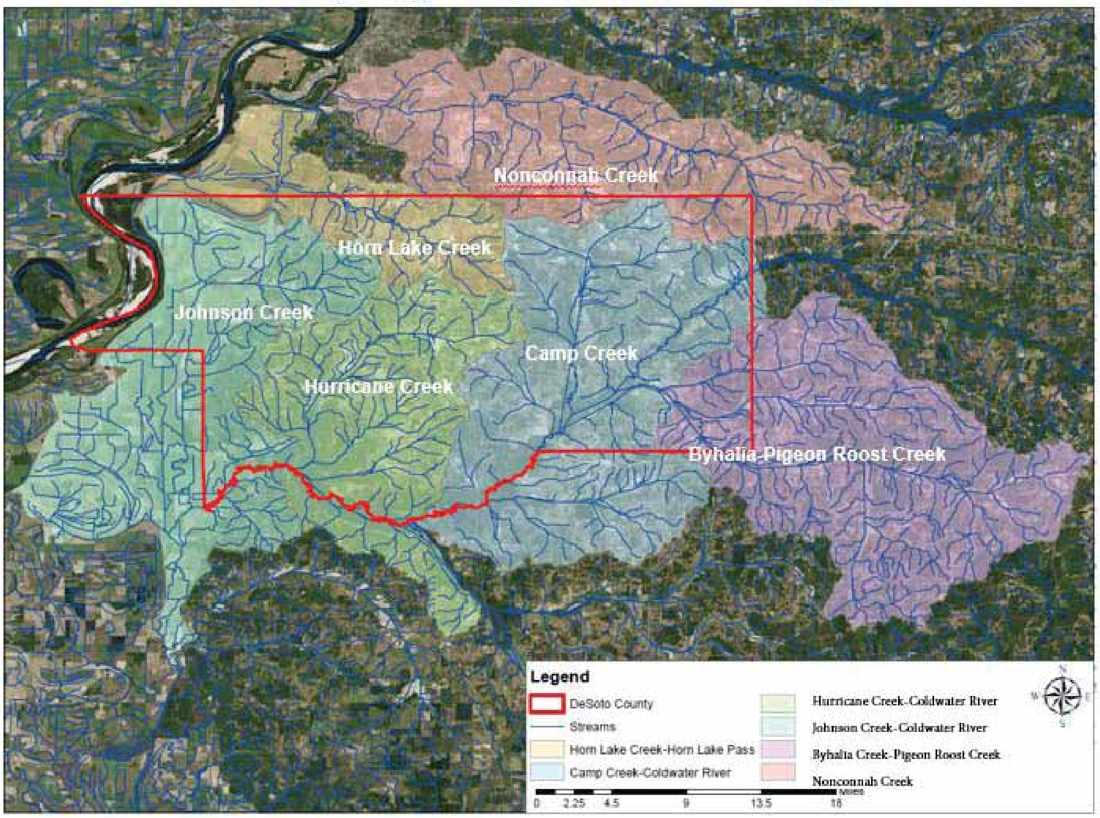
Memphis District
Mississippi River Valley Division

Appendix G: Hydrologic and Hydraulic Appendix



Memphis Metropolitan Stormwater-North Desoto County, Mississippi

May 2022

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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). 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.

SOFTWARE

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.

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.

3.0 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 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.

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 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, and it was felt HEC-RAS 1D/2D analysis will capture this better.

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 with 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 0.001 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 (1.01-year, 2-year, 5-year, and 25-year return periods) intermediate flood events. Pertinent studies and reports are shown in Table 1.

Table1. 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.

4.0 HYDROLOGY

4.1 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 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.

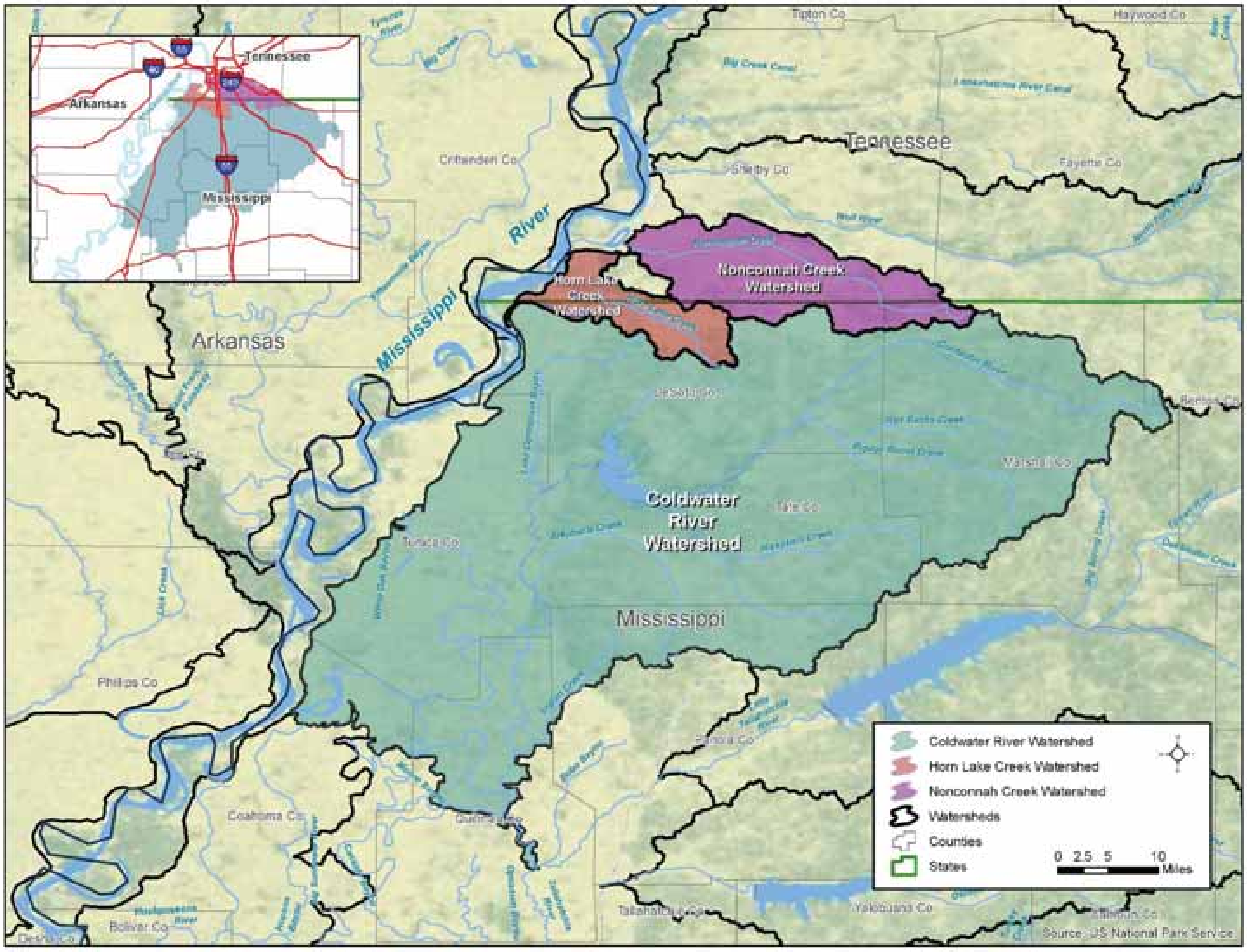

Figure 1 – Horn Lake Creek, Nonconnah Creek and Coldwater River Basins

4.2 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)[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	0.524 (0.439-0.625)	0.645 (0.539-0.771)	0.734 (0.611-0.881)	0.850 (0.687-1.04)	0.936 (0.745-1.16)	1.02 (0.791-1.28)	1.10 (0.829-1.41)	1.21 (0.882-1.58)	1.29 (0.923-1.71)
10-min	0.767 (0.643-0.915)	0.944 (0.789-1.13)	1.08 (0.895-1.29)	1.25 (1.01-1.52)	1.37 (1.09-1.69)	1.50 (1.16-1.88)	1.62 (1.21-2.07)	1.77 (1.29-2.32)	1.89 (1.35-2.51)
15-min	0.935 (0.784-1.12)	1.15 (0.962-1.38)	1.31 (1.09-1.57)	1.52 (1.23-1.85)	1.67 (1.33-2.06)	1.82 (1.41-2.29)	1.97 (1.48-2.52)	2.16 (1.58-2.83)	2.31 (1.65-3.06)
30-min	1.25 (1.05-1.49)	1.55 (1.29-1.85)	1.77 (1.47-2.12)	2.05 (1.66-2.50)	2.26 (1.80-2.79)	2.47 (1.91-3.10)	2.67 (2.01-3.42)	2.94 (2.14-3.84)	3.13 (2.24-4.15)
60-min	1.59 (1.33-1.90)	1.95 (1.63-2.33)	2.22 (1.85-2.66)	2.58 (2.09-3.15)	2.85 (2.27-3.52)	3.12 (2.42-3.92)	3.39 (2.55-4.34)	3.74 (2.73-4.90)	4.01 (2.87-5.32)
2-hr	1.93 (1.63-2.28)	2.35 (1.98-2.78)	2.67 (2.24-3.17)	3.10 (2.53-3.76)	3.43 (2.75-4.21)	3.76 (2.94-4.70)	4.10 (3.11-5.22)	4.55 (3.34-5.92)	4.89 (3.52-6.44)
3-hr	2.17 (1.84-2.55)	2.63 (2.22-3.10)	2.99 (2.51-3.53)	3.47 (2.85-4.20)	3.85 (3.11-4.71)	4.24 (3.33-5.27)	4.63 (3.53-5.89)	5.17 (3.82-6.70)	5.59 (4.04-7.32)
6-hr	2.63 (2.25-3.07)	3.22 (2.75-3.76)	3.69 (3.13-4.32)	4.32 (3.58-5.19)	4.82 (3.92-5.85)	5.33 (4.22-6.59)	5.86 (4.49-7.39)	6.58 (4.89-8.47)	7.14 (5.19-9.29)
12-hr	3.16 (2.72-3.65)	3.97 (3.41-4.60)	4.61 (3.94-5.35)	5.47 (4.56-6.51)	6.14 (5.02-7.38)	6.82 (5.43-8.35)	7.52 (5.80-9.40)	8.46 (6.33-10.8)	9.19 (6.72-11.9)
24-hr	3.73 (3.24-4.27)	4.75 (4.11-5.45)	5.54 (4.77-6.37)	6.60 (5.54-7.79)	7.43 (6.13-8.86)	8.27 (6.64-10.0)	9.13 (7.10-11.3)	10.3 (7.75-13.0)	11.2 (8.25-14.3)
2-day	4.36 (3.82-4.94)	5.46 (4.76-6.20)	6.32 (5.49-7.21)	7.51 (6.36-8.79)	8.44 (7.02-9.99)	9.40 (7.61-11.3)	10.4 (8.14-12.8)	11.7 (8.91-14.8)	12.8 (9.49-16.3)
3-day	4.78 (4.20-5.39)	5.94 (5.21-6.71)	6.85 (5.98-7.77)	8.12 (6.91-9.45)	9.11 (7.61-10.7)	10.1 (8.24-12.2)	11.2 (8.80-13.7)	12.6 (9.63-15.8)	13.8 (10.3-17.4)
4-day	5.14 (4.53-5.77)	6.35 (5.58-7.14)	7.30 (6.39-8.24)	8.60 (7.35-9.98)	9.63 (8.07-11.3)	10.7 (8.71-12.8)	11.8 (9.29-14.4)	13.3 (10.1-16.6)	14.4 (10.8-18.2)
7-day	6.13 (5.44-6.83)	7.41 (6.56-8.27)	8.40 (7.41-9.42)	9.76 (8.38-11.2)	10.8 (9.12-12.6)	11.9 (9.76-14.1)	13.0 (10.3-15.8)	14.5 (11.1-18.0)	15.7 (11.8-19.7)
10-day	6.98 (6.23-7.75)	8.35 (7.43-9.29)	9.41 (8.33-10.5)	10.8 (9.35-12.4)	12.0 (10.1-13.8)	13.1 (10.8-15.4)	14.2 (11.3-17.2)	15.8 (12.2-19.5)	17.0 (12.8-21.2)
20-day	9.32 (8.38-10.2)	11.1 (9.96-12.2)	12.5 (11.1-13.8)	14.3 (12.4-16.1)	15.6 (13.3-17.9)	17.0 (14.1-19.8)	18.4 (14.7-21.9)	20.2 (15.6-24.7)	21.6 (16.4-26.7)
30-day	11.3 (10.2-12.3)	13.5 (12.1-14.8)	15.1 (13.5-16.6)	17.2 (15.0-19.3)	18.8 (16.0-21.3)	20.3 (16.9-23.5)	21.8 (17.5-25.9)	23.8 (18.5-28.9)	25.3 (19.3-31.2)
45-day	13.9 (12.6-15.1)	16.6 (15.0-18.0)	18.5 (16.7-20.2)	20.9 (18.3-23.3)	22.7 (19.5-25.6)	24.4 (20.3-28.1)	26.0 (21.0-30.6)	28.1 (21.9-33.8)	29.5 (22.6-36.2)
60-day	16.1 (14.7-17.5)	19.2 (17.5-20.9)	21.4 (19.4-23.3)	24.1 (21.1-26.6)	26.0 (22.3-29.1)	27.8 (23.2-31.8)	29.4 (23.7-34.4)	31.4 (24.5-37.6)	32.7 (25.1-40.0)

[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.

5.0 Climate Change Assessment for Desoto County, Mississippi

5.1 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 (USACE, 2018). A qualitative analysis of historical climate trends, as well as assessment of future projections was provisioned by ECB 2018-14. 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.

5.2 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.

5.2.1 Temperature Precipitation

The 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..

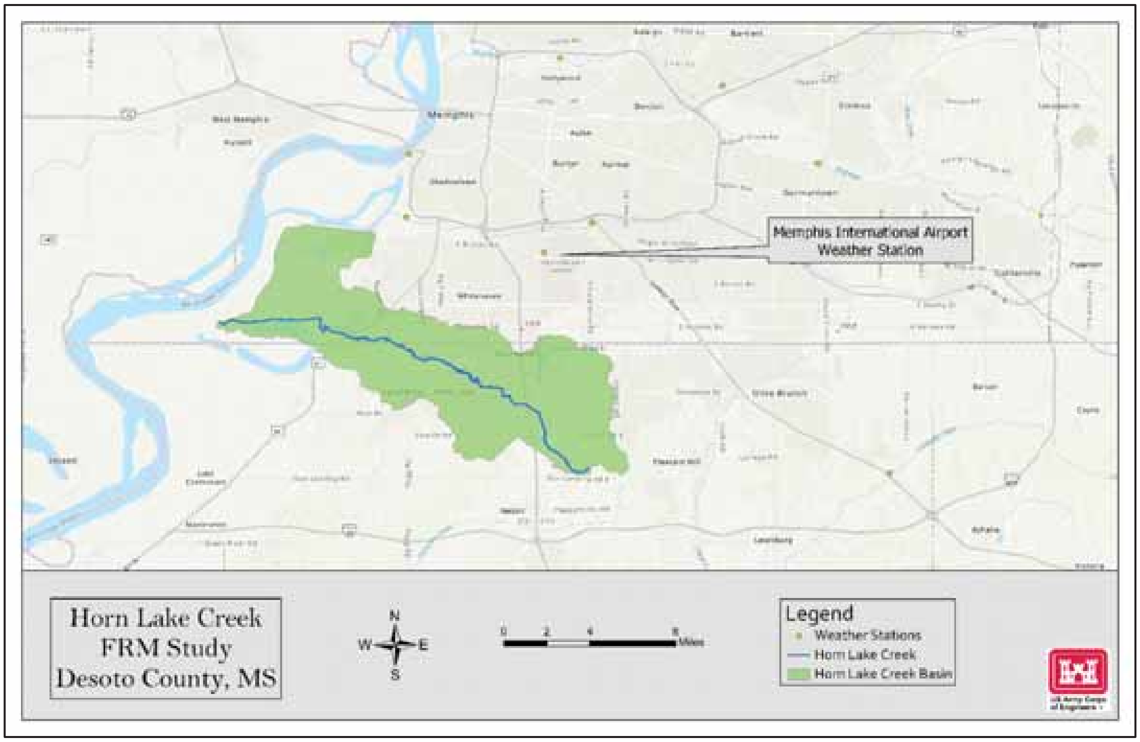

Figure 2. Study area and Memphis International Airport (MEM) Weather Station Statistical Temperature and precipitation Analysis for the Horn Lake Creek Basin

From 1930 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 and 4.

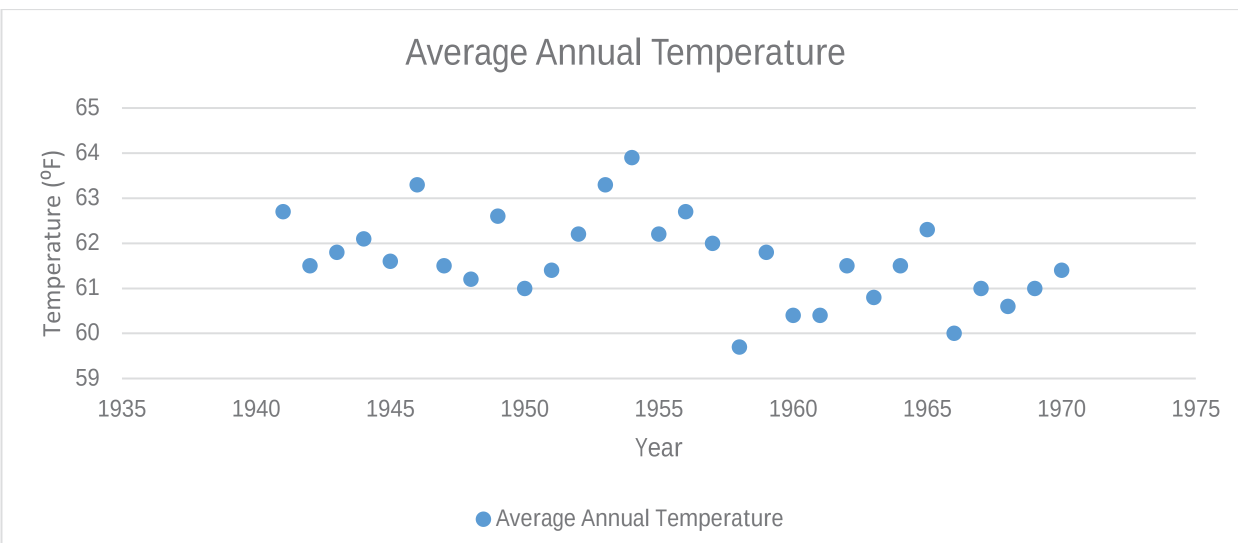

Figure 3: Annual average temperature from 1940 – 1970

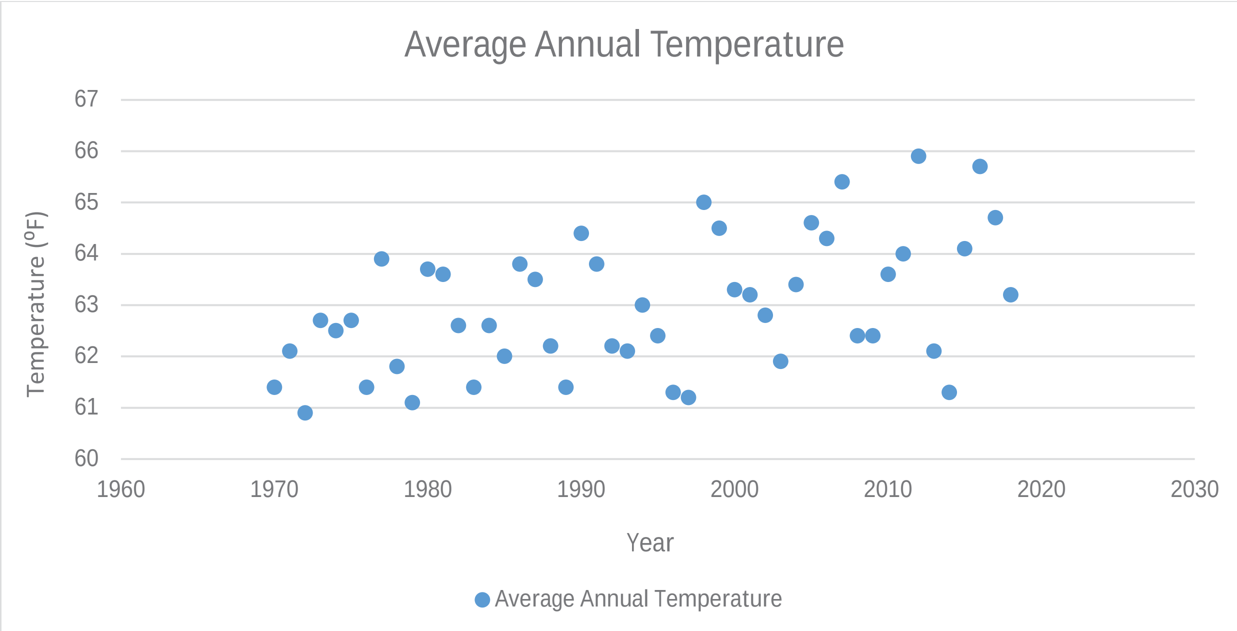

Figure 4: Annual average temperature from 1970 – 2018

5.2.2 Precipitation

The MEM Airport weather station shows variable annual average precipitation since 1940. The results in Figures 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.

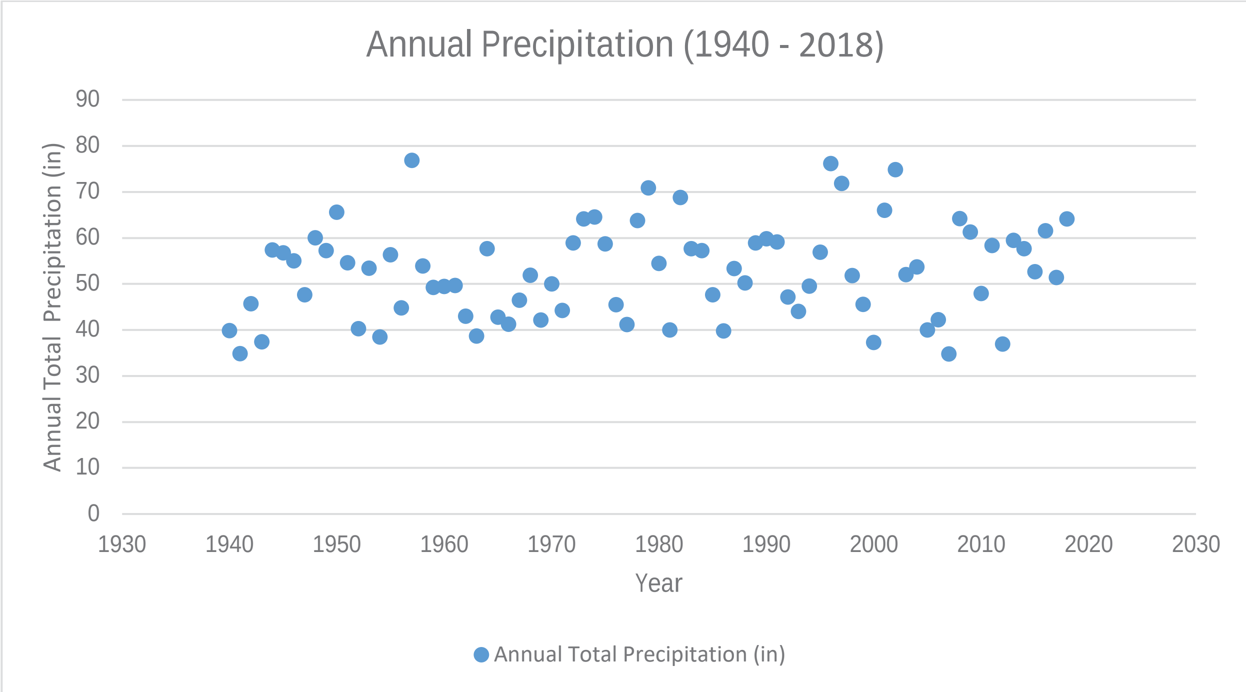

Figure 5. Annual total precipitation 1940 - 2018

5.2.3 Hydrology

5.2.3.1 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. Horn Lake Creek does not have a discharge gage, but a gage is located east in the adjacent Coldwater Basin. The USGS gage 07275900 on the Coldwater River near Olive Branch, MS has 21 years of record was available for select climate change assessments. Figure 7 also shows the gages in the Nonconnah Creek located north in Tennessee which contained a longer period of record and used for several Climate Change Assessments as required by Corps guidance. As stated above, detailed information is presented in Climate Change Appendix.

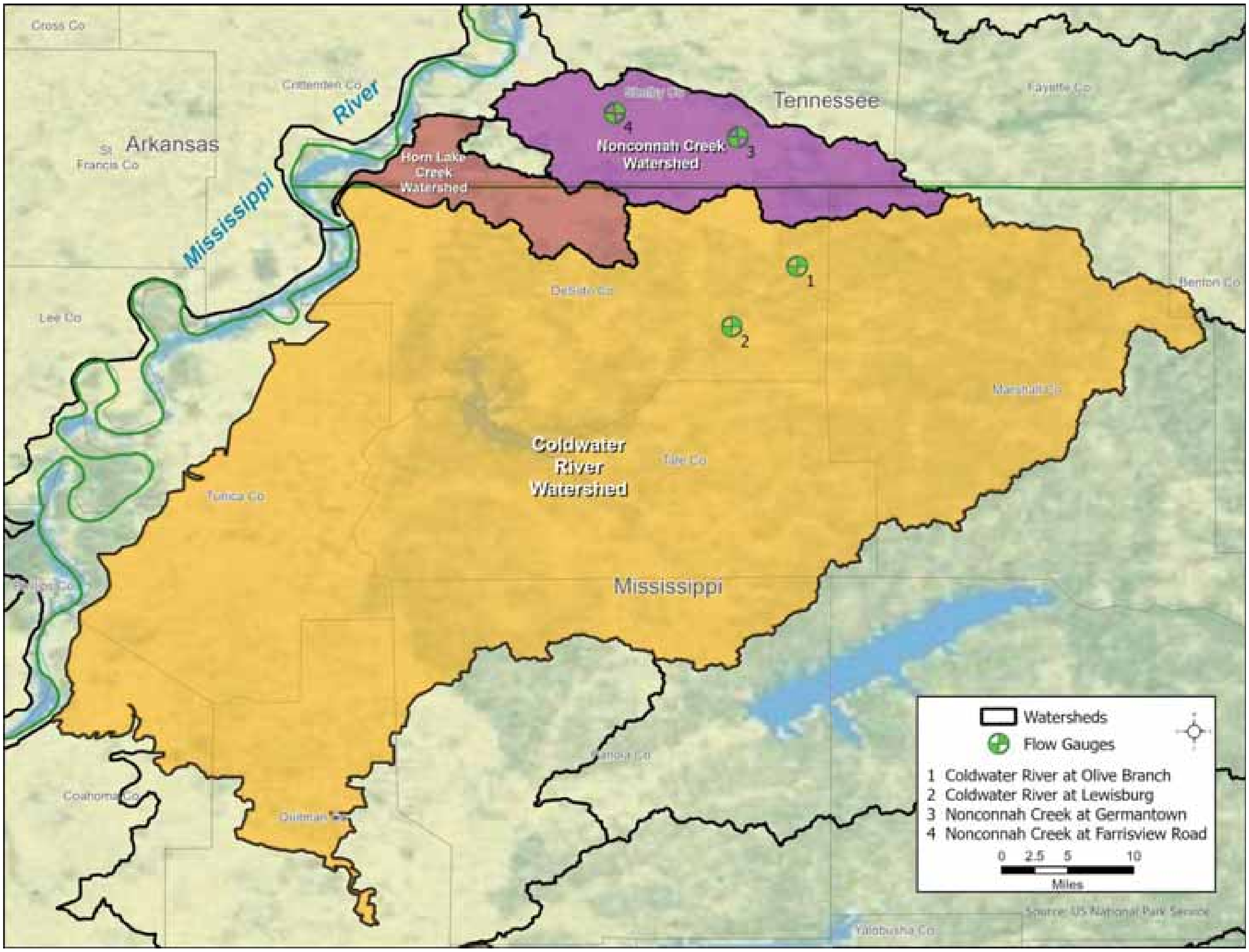

Figure 7. Horn Lake Creek Basin and Gages in Adjacent Basins

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

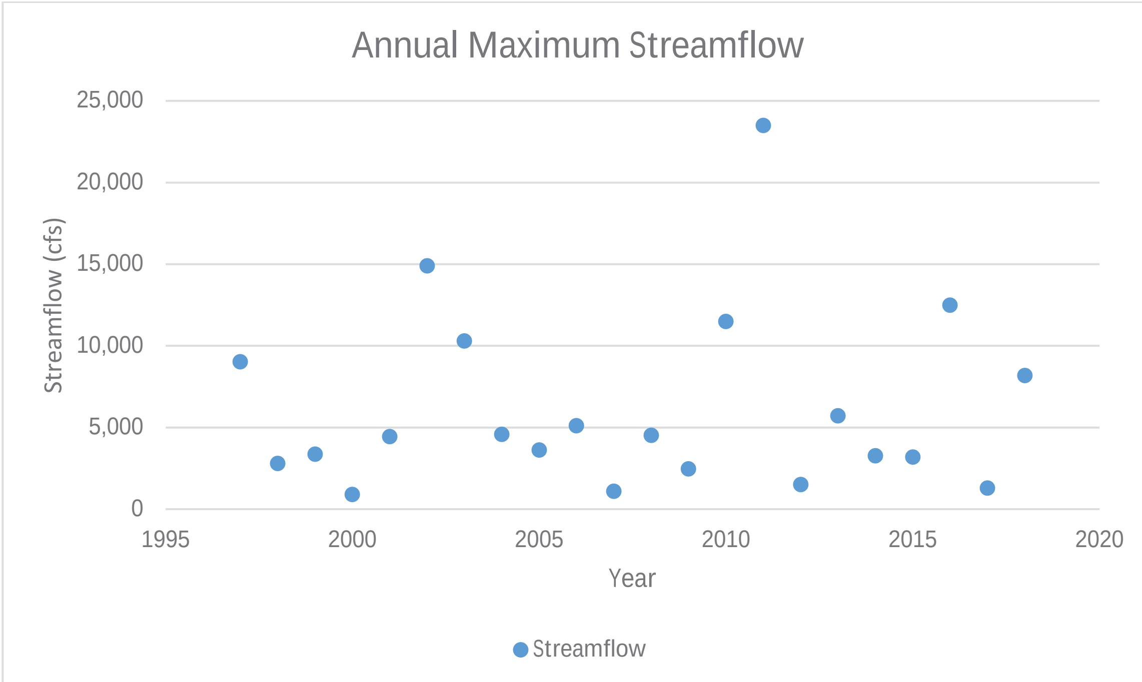

Figure 6: Annual Peak Streamflow at USGS 07275900 Gage Coldwater River near Olive Branch, MS

5.3 Non-Stationarity Assessment

IN ACCORDANCE WITH ECB 2018-14, 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 TENTATIVELY SELECTED PLAN 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.

5.3.1 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 in this analysis 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.

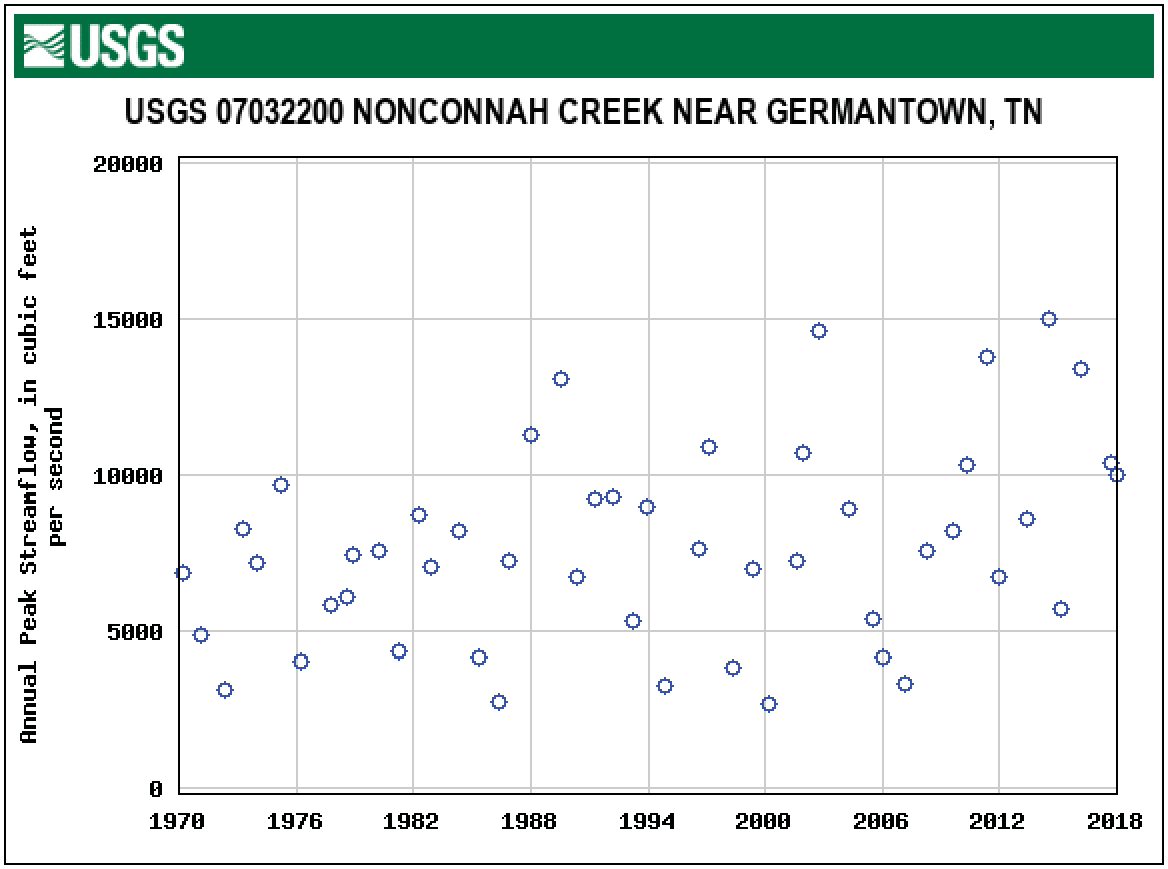

Figure 8: APF at USGS 07032200 Nonconnah Creek near Germantown, TN

5.3.2. 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.

5.4 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, 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 assess.

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.

6.0 Hydrologic Methodology and Modeling

6.1 Horn Lake Creek HEC-1 to HEC-HMS Conversion

As stated in Section 3.0, hydrologic modeling for this study was performed by importing the 2005 HEC-1 model into HEC-HMS version 4.3. 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”. The Horn Lake Creek drainage area was originally divided into subareas to simulate the runoff process. A synthetic unit hydrograph was developed for each subarea using Snyder’s Unit Hydrograph method. 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 of stream flow routing. Modified Puls volume versus discharge relationships were derived using the original HEC-2 model and updated using the 2018 HEC-RAS model. The Modified Puls relationship was adjusted to reflect improved conveyance for channel improvement alternatives.

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.

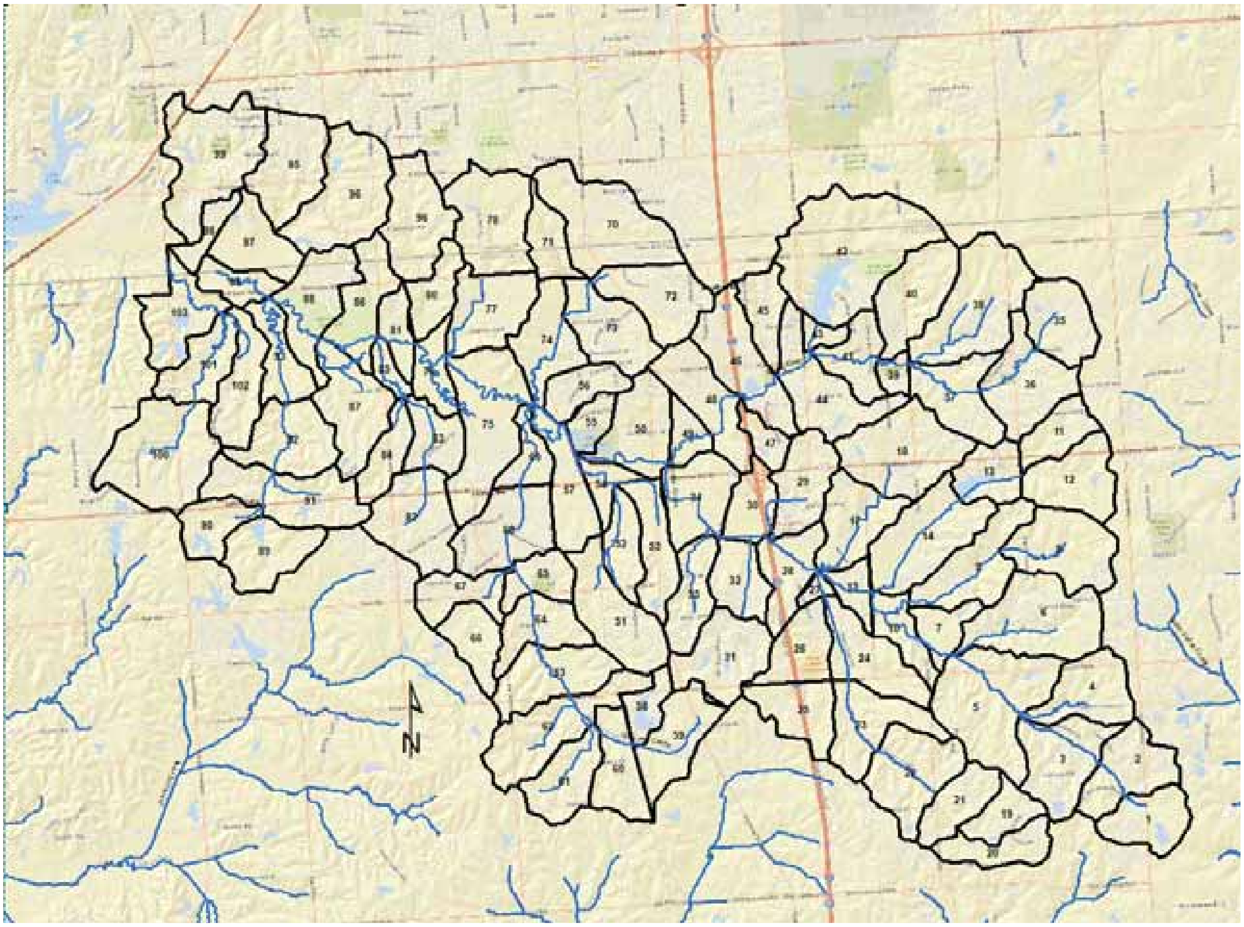

Figure 9. Horn Lake Creek Subarea Delineation

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.

6.1.1 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. All hydrologic coefficients, except for Snyder's time to peak, Tp, were unchanged and parameters (i.e., watercourse lengths) were not altered for 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:

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

Where:

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

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

6.2 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.

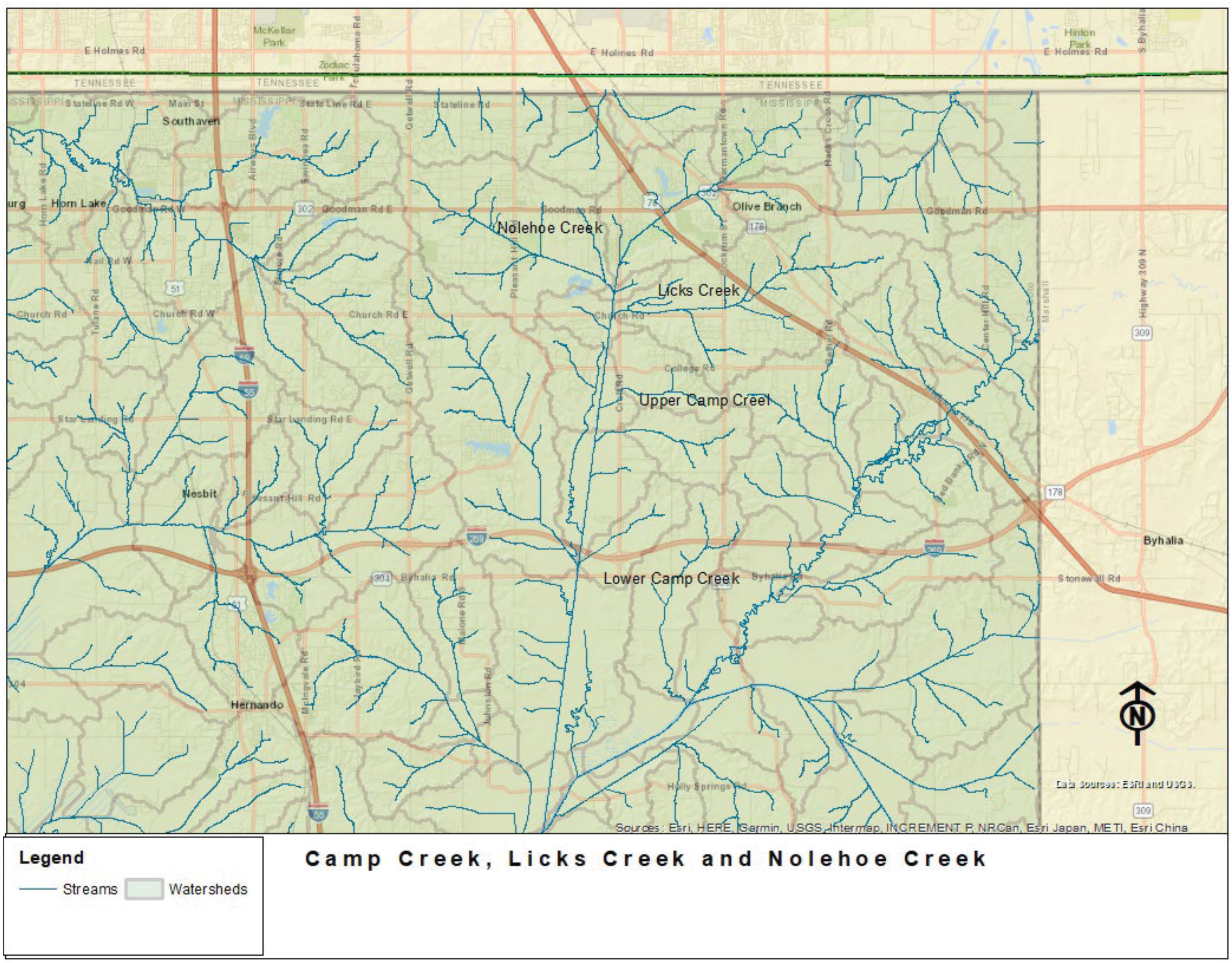

Figure 10. Camp Creek Watershed

6.3 Horn Lake Creek Basin Urban Growth Estimates

Hydrologic parameters in HMS were adjusted to quantify the impacts to peak runoff rates in the Horn Lake Creek basin. Desoto County GIS department provided land use information showing subdivisions built prior to the 2005 study and the residential growth that transpired from 2002 to 2018. Figure 11 shows an estimate of the residential growth and development.

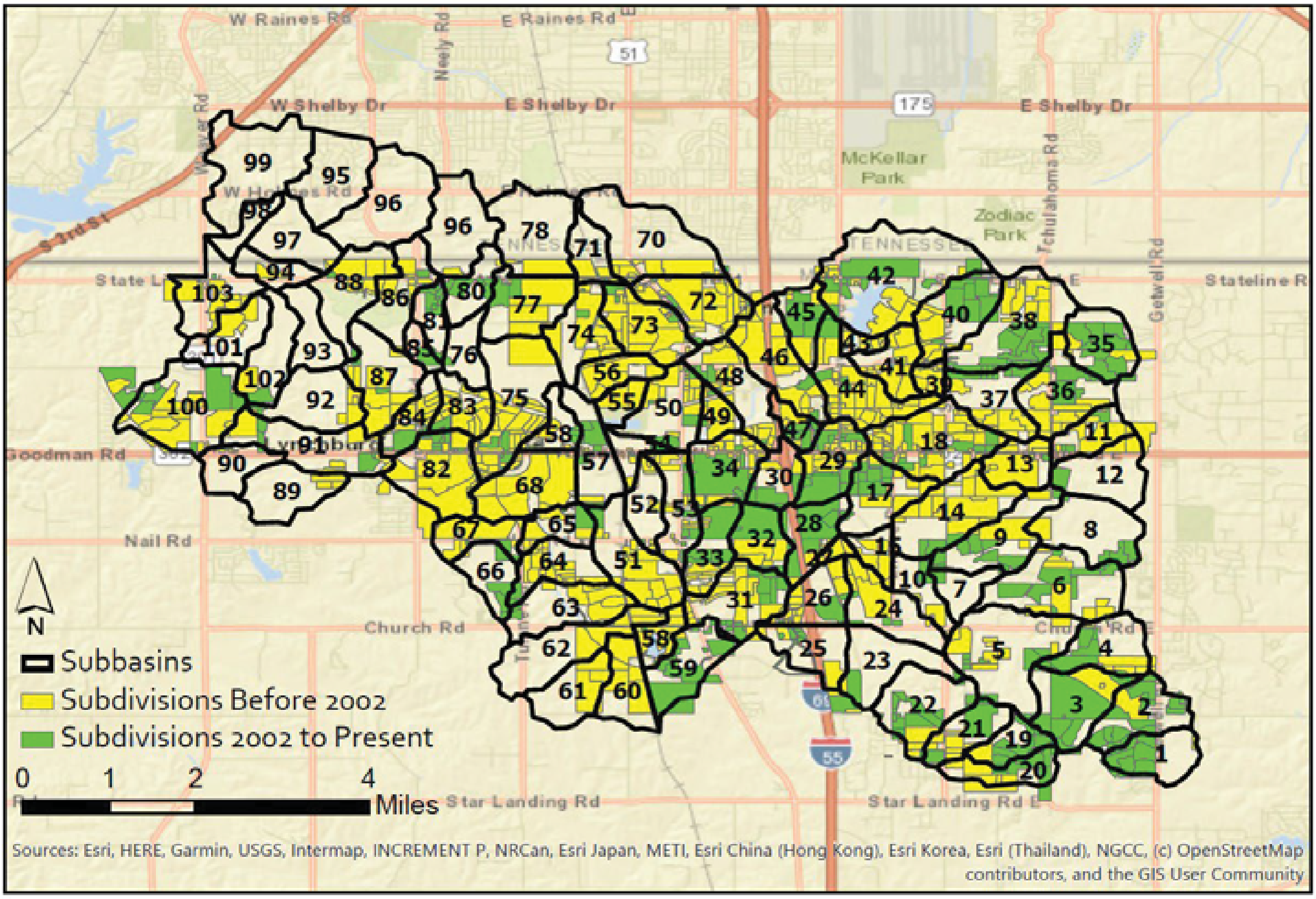

Figure 11. Horn Lake Creek HEC-HMS Hydrologic Subareas

Recent aerial photography was combined with the Desoto County subdivision land use map to approximate the total development that has occurred since the 2005 feasibility study. Subarea parameters were modified if pervious areas were converted to impervious surfaces by the construction of typical commercial, industrial, and office facilities. Residential development was considered to impact runoff if property lot sizes were less than approximately three-fourths of an acre. MVM has traditionally assumed larger lot sizes possess pervious areas that tend to reduce rainfall runoff quantities and lessen urban development impact. The following bar-charts show the total growth, which includes both commercial and residential development, and their resulting changes to land use as applied to each individual subarea.

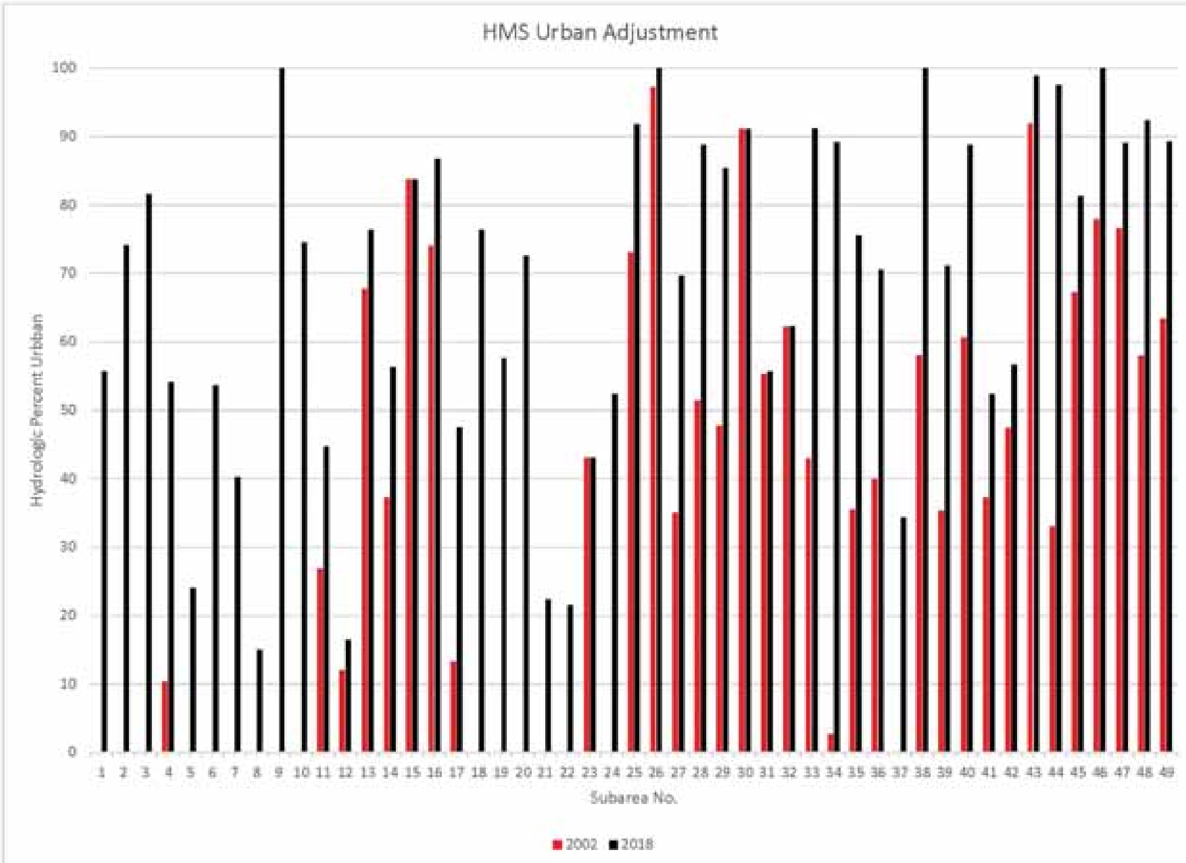

Figure 12. Subarea HEC-HMS Percent Urban 2002 vs. 2018

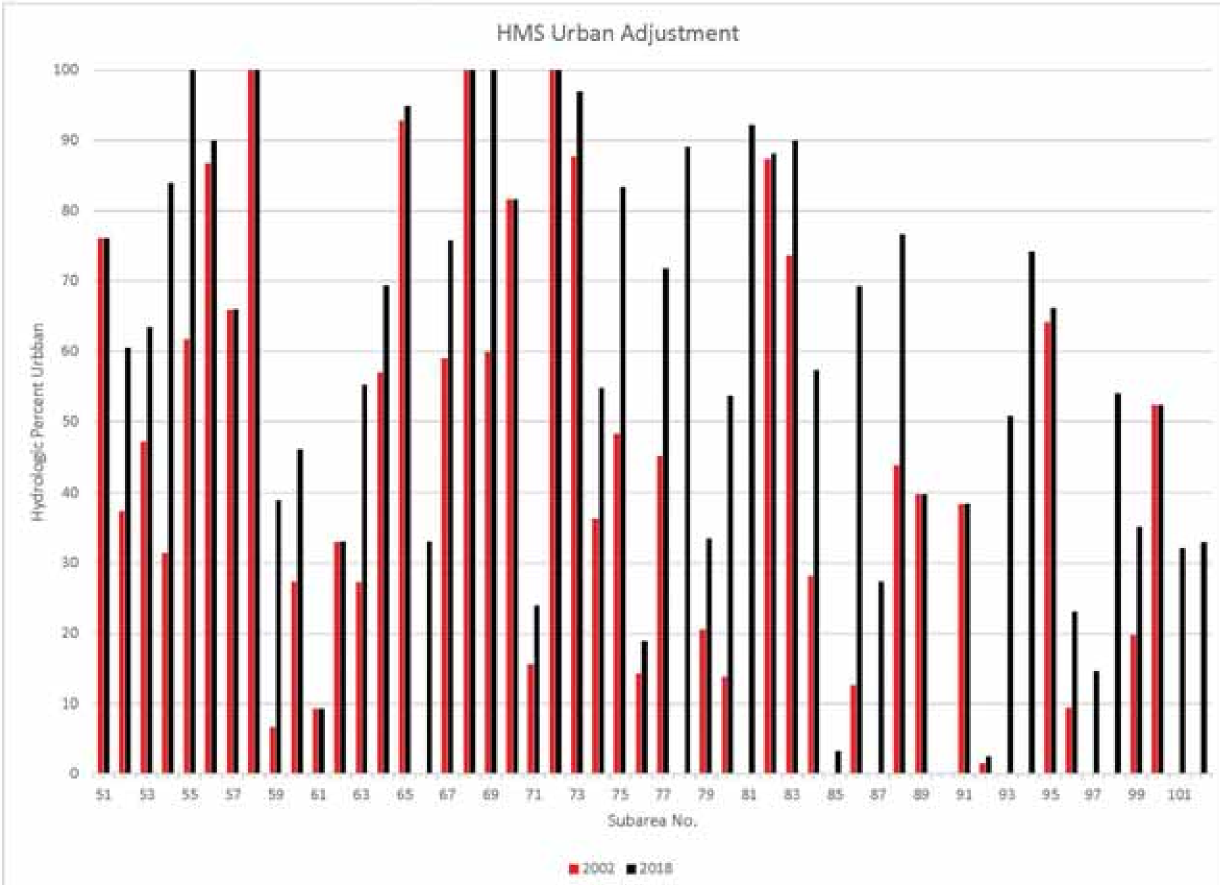

Figure 12 cont. Subarea HEC-HMS Percent Urban 2002 vs. 2018.

Subareas, which contained urbanized or impervious areas, were adjusted to reflect the accelerated runoff associated with urbanization and development. The adjustments are reflected in the Snyders Tp and initial and uniform loss rate parameters. The percent of urban/developed area was applied to the unit hydrograph parameter Snyder's T_p by using the following relation:

$$T_{pu} = 1 + \frac{\%urban}{100}$$

where:

T_{pu} is the adjusted time to peak based on percent impervious area.

This relationship is based on the assumption that maximum urbanization will result in a 50% reduction in the time to peak of a subarea hydrograph.

The initial and uniform loss rates were also adjusted to reflect progressive urbanization. Unadjusted initial losses were assumed to 1.0 inches and uniform loss rates were assumed to be 0.1 inch/hour. The equations to adjust initial and uniform loss rates are shown below:

L_i = initial loss rate*urban percentage/100
L_u = uniform loss rate*urban percentage/100

Table 3 shows the resulting adjustments and parameters used in the HMS model.

Subarea	Drainage Area (Sq. Mi)	2002 % Urban	2018 % Urban		2002 Initial Loss (Inches)	2018 Initial Loss (Inches)		2002 Snyders' Tp (Hours)	2018 Snyders' Tp (Hours)
1	0.41	0	56		1.00	0.72		0.36	0.32
2	0.51	0	74		1.00	0.63		0.68	0.28
3	0.67	0	82		1.00	0.59		0.97	0.53
4	0.47	10	54		1.00	0.73		0.86	0.56
5	0.97	0	24		1.00	0.88		0.94	0.76
6	0.82	0	54		1.00	0.73		0.96	0.62
7	0.23	0	40		1.00	0.8		0.82	0.59
8	0.61	0	15		1.00	0.93		0.79	0.69
9	0.58	0	100		1.00	0.5		1.07	0.54
10	0.28	0	74		1.00	0.63		0.91	0.52
11	0.29	27	45		0.94	0.92		0.58	0.56
12	0.51	12	17		0.87	0.78		0.57	0.5
13	0.33	68	76		0.67	0.62		0.34	0.32
14	0.72	37	56		0.84	0.72		0.77	0.65
15	0.18	84	84		0.59	0.58		0.44	0.44
16	0.76	74	87		0.77	0.57		0.48	0.37
17	0.5	13	47		0.94	0.76		0.94	0.71
19	0.27	0	76		1.00	0.71		0.6	0.38
20	0.3	0	58		1.00	0.62		0.63	0.36
21	0.35	0	73		1.00	0.64		0.72	0.42
22	0.62	0	22		1.00	0.89		0.74	0.6
23	0.53	0	21		1.00	0.89		0.62	0.51
24	0.5	43	43		0.78	0.78		0.92	0.94
25	0.4	0	52		1.00	0.74		0.59	0.39
26	0.39	73	92		1.00	0.54		0.75	0.39
27	0.07	97	100		0.50	0.5		0.14	0.14
28	0.4	35	70		0.80	0.65		0.52	0.43
29	0.45	51	89		0.82	0.56		0.55	0.4
30	0.3	48	85		0.77	0.57		0.71	0.56
31	0.57	91	91		0.55	0.54		0.34	0.34
32	0.34	55	56		0.72	0.72		0.94	0.94
33	0.37	62	62		0.70	0.69		0.51	0.51
34	0.48	43	91		0.80	0.54		1.08	0.79
35	0.45	3	89		1.00	0.55		0.6	0.32
36	0.6	36	76		0.79	0.62		0.52	0.43
37	0.49	40	71		0.80	0.65		0.61	0.5
38	0.87	0	34		1.00	0.83		0.97	0.72
39	0.11	58	100		0.73	0.5		0.39	0.3
40	0.54	35	71		0.83	0.64		0.63	0.5
41	0.33	61	89		0.71	0.56		0.56	0.47
42	1.34	37	52		0.77	0.74		1.05	1.01
43	0.1	47	57		0.80	0.72		0.33	0.29
44	0.63	92	99		0.53	0.51		0.45	0.44
45	0.3	33	97		0.50	0.51		0.35	0.35
46	0.61	67	81		0.50	0.59		0.32	0.35
47	0.17	78	100		0.56	0.5		0.29	0.28
48	0.53	76	89		0.60	0.55		0.31	0.3
49	0.19	58	92		0.87	0.54		0.47	0.33
50	0.52	63	89		0.67	0.55		0.83	0.73
51	0.61	76	76		0.65	0.62		0.53	0.51
52	0.3	37	60		0.94	0.7		0.84	0.59
53	0.42	47	63		0.83	0.68		0.92	0.75
54	0.16	31	84		0.86	0.58		0.69	0.48
55	0.18	62	100		0.72	0.5		0.26	0.2

Table 3. Subarea Hydrologic Parameters

56	0.28	87	90		0.59	0.55		0.39	0.37
57	0.44	66	66		0.67	0.67		1.16	1.16
58	0.19	100	100		0.92	0.81		0.78	0.66
59	0.49	7	39		0.50	0.5		0.31	0.31
60	0.41	27	46		0.81	0.77		0.6	0.57
61	0.41	9	9		1.00	0.95		0.8	0.73
62	0.66	33	33		0.94	0.84		0.55	0.47
63	0.55	27	55		0.85	0.72		0.44	0.37
64	0.38	57	69		0.84	0.65		0.45	0.35
65	0.27	93	95		0.59	0.53		0.24	0.23
66	0.31	0	33		0.58	0.62		0.3	0.41
67	0.33	59	76		0.55	0.83		0.4	0.44
68	0.62	100	100		0.50	0.5		0.41	0.41
69	0.23	60	100		0.70	0.5		1.7	1.37
70	0.88	82	82		0.62	0.59		0.85	0.82
71	0.28	16	24		0.58	0.5		0.56	0.52
72	0.54	100	100		1.00	0.88		1.03	0.83
73	0.51	88	97		0.61	0.52		0.35	0.32
74	0.46	36	55		0.85	0.73		1.25	1.05
75	0.87	48	83		0.74	0.58		1.09	0.9
76	0.7	14	19		0.93	0.91		0.78	0.75
77	0.54	45	72		0.75	0.64		0.87	0.76
78	0.26	0	89		1.00	0.55		1.7	0.9
79	0.61	21	33		0.91	0.83		0.71	0.63
80	0.3	14	54		0.94	0.73		0.81	0.6
81	0.2	0	92		1.00	0.54		0.33	0.17
82	0.62	87	88		0.60	0.56		0.49	0.47
83	0.32	74	90		0.57	0.55		0.61	0.6
84	0.29	28	57		0.82	0.71		0.61	0.53
85	0.12	0	3		1.00	0.98		0.94	0.91
86	0.48	13	69		0.93	0.65		1.07	0.73
87	0.57	0	27		1.00	0.86		1.01	0.79
88	0.53	44	77		0.87	0.62		0.5	0.36
89	0.46	40	40		0.92	0.8		0.53	0.44
90	0.3	0	0		1.00	1		0.72	0.72
91	0.68	38	38		0.85	0.81		0.44	0.41
92	0.53	1	2		1.00	0.99		1.1	1.07
93	0.35	0	51		1.00	0.75		1.99	1.32
94	0.23	0	74		1.00	0.63		1.76	1.01
95	0.6	64	66		1.00	0.67		1.39	1.13
96	0.82	9	23		0.68	0.88		0.57	0.57
97	0.44	0	15		1.00	0.93		1.2	1.05
98	0.21	0	54		1.00	0.73		0.84	0.55
99	0.87	20	35		0.91	0.82		0.89	0.55
100	0.87	52	52		0.68	0.74		0.61	0.78
101	0.38	0	32		1.00	0.84		1.22	0.66
102	0.42	0	33		1	0.84		0.94	0.94
			53						

Table 3 cont. Subarea Hydrologic Parameters

6.3 Flow Verification

The Horn Lake Creek Basin does not have any stream gages, as shown Figure 7 above. Due to the absence of stream gage information, the original 2005 information was verified by use of the U.S.G.S. regression relationships and calibrated HEC-1 models developed in the Coldwater River basin. A 1990 study entitled "Hydrologic Analysis of the Coldwater River Watershed" was conducted by Lenzotti and Fullerton Consulting Engineers, Inc. for the Vicksburg District Corps of Engineers and presents flow vs. frequency for numerous streams in north Desoto County. The current Desoto County FIS references this document as a source for other information and it was felt reliable for flow verification assuming the Horn Lake Creek basin would have relatively compatible runoff characteristics and volumes.

The Coldwater River watershed is an adjacent basin with similar runoff characteristics as Horn Lake Creek. The flow versus frequency relationships in the Coldwater Basin, specifically the Upper Camp Creek basin, are felt to be applicable and were used to aid in verification. The figure below shows the compatible watersheds.

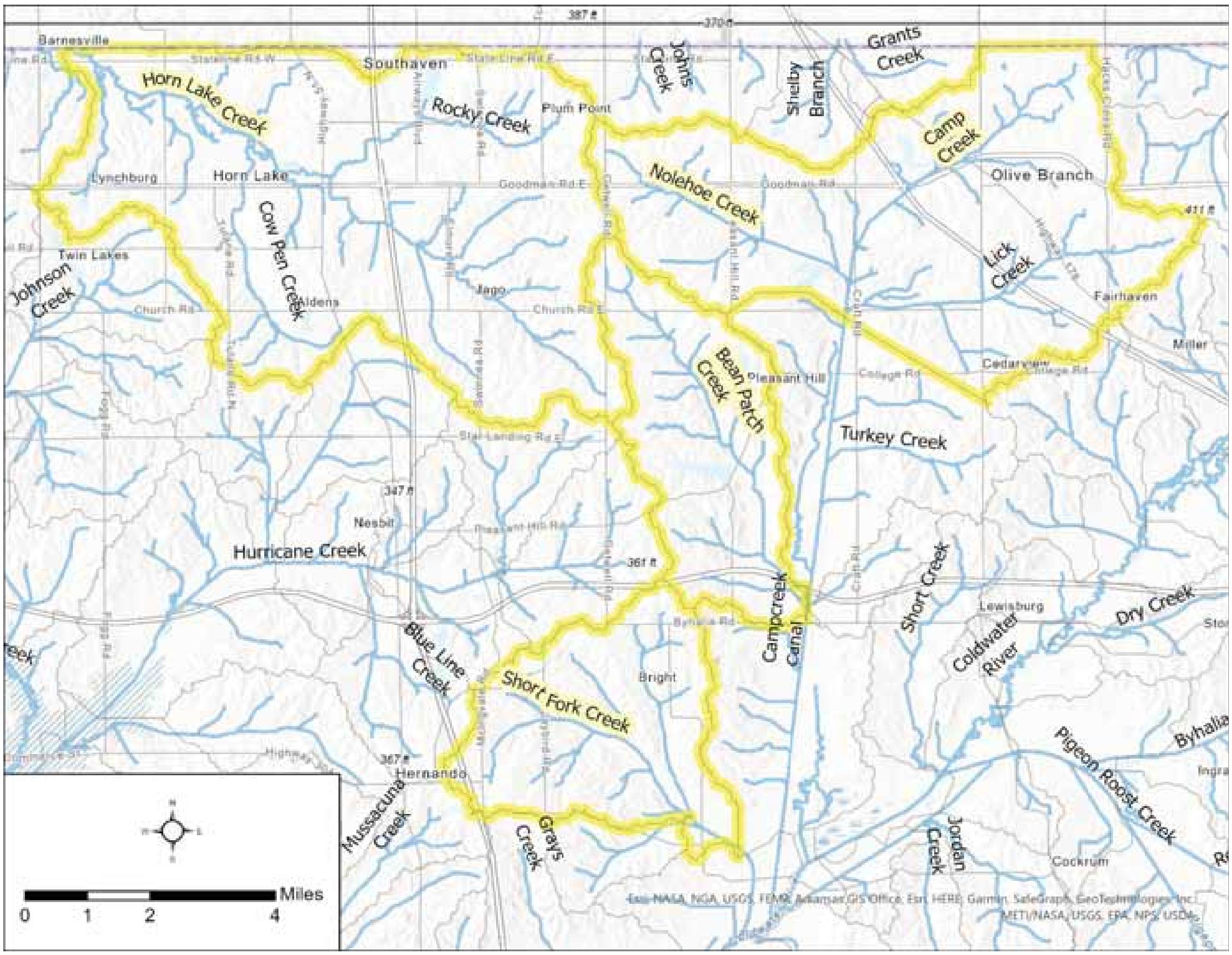

Figure 13. Horn Lake Creek Flow Validation-Upper Camp Creek Adjacent Basin

The Coldwater River Watershed report lacked any data pertinent to Nolehoe and Upper Camp Creek. The FIS contains a table with updated HMS flows for Nolehoe and Upper Camp Creek and used for verification. Comparison plots of the Coldwater tributaries and the Horn Lake Creek flows for 2005 and 2019 are shown in Figures 14 to 17.

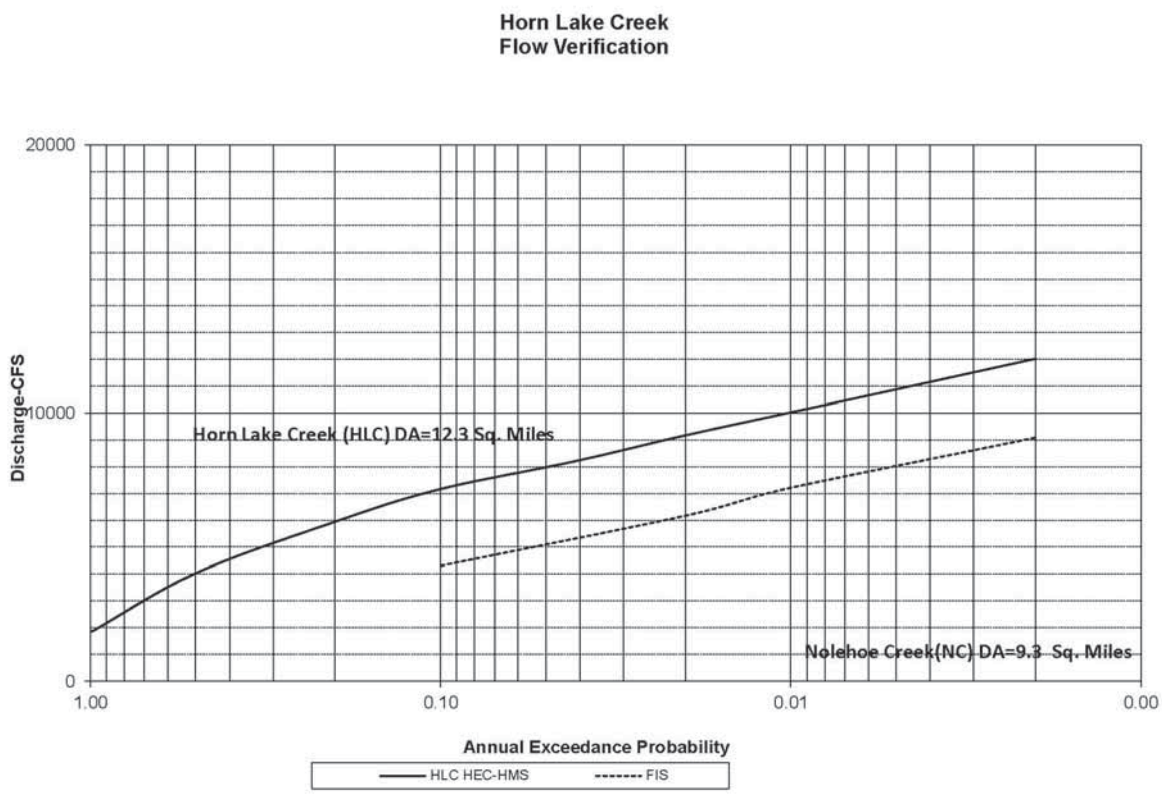

Figure 14. Horn Lake Creek Flow Verification
Approximate DA~11 square miles

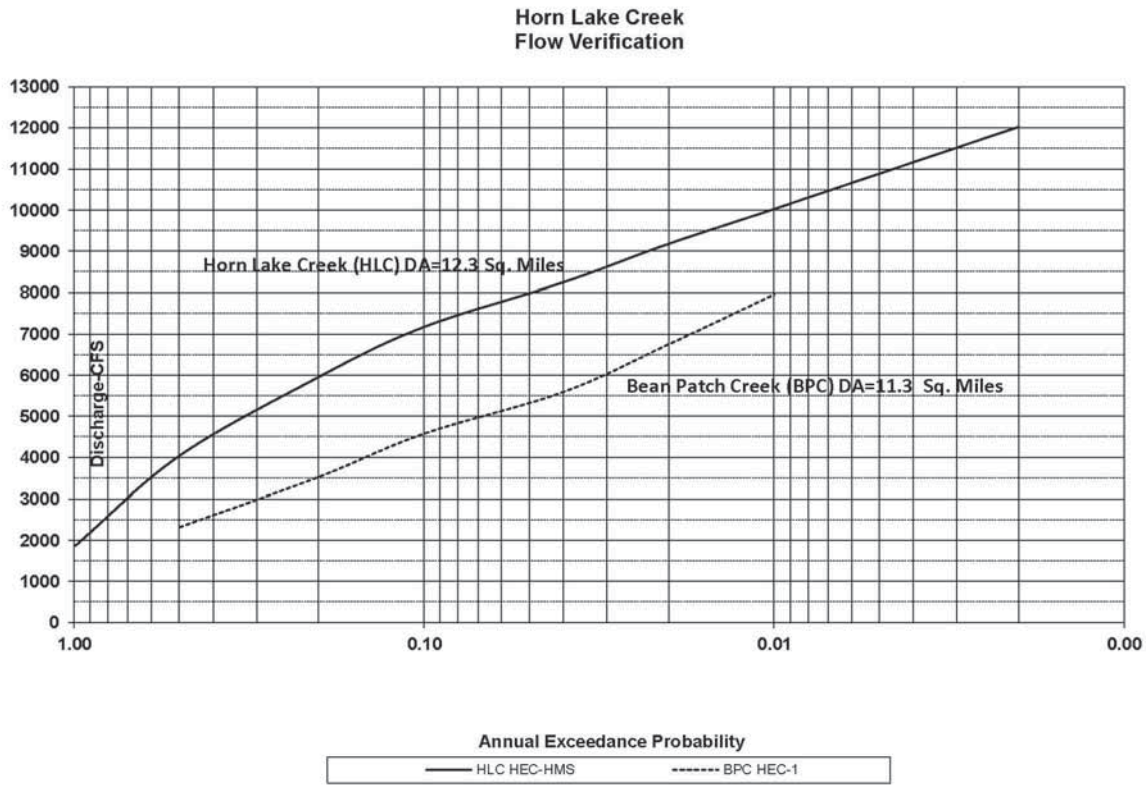

Figure 15. Horn Lake Creek Flow Verification
Approximate DA~12 square miles

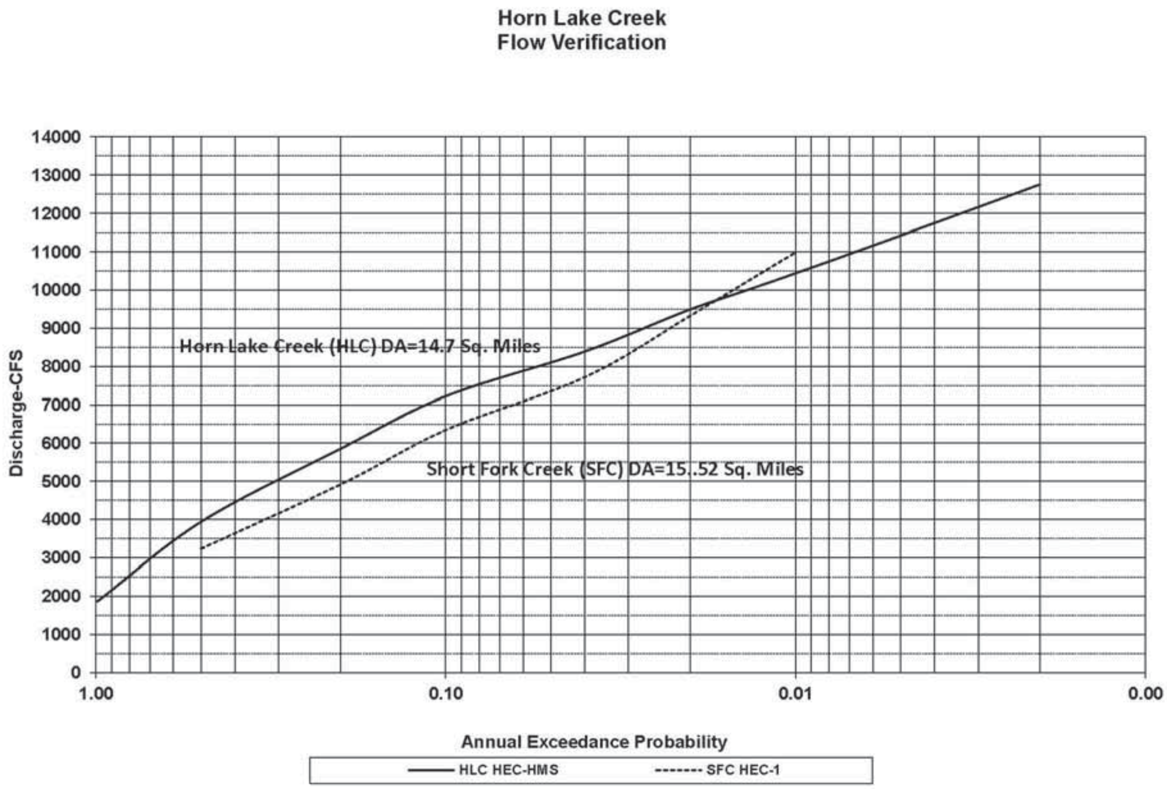

Figure 16. Horn Lake Creek Flow Verification
Approximate DA~15 square miles

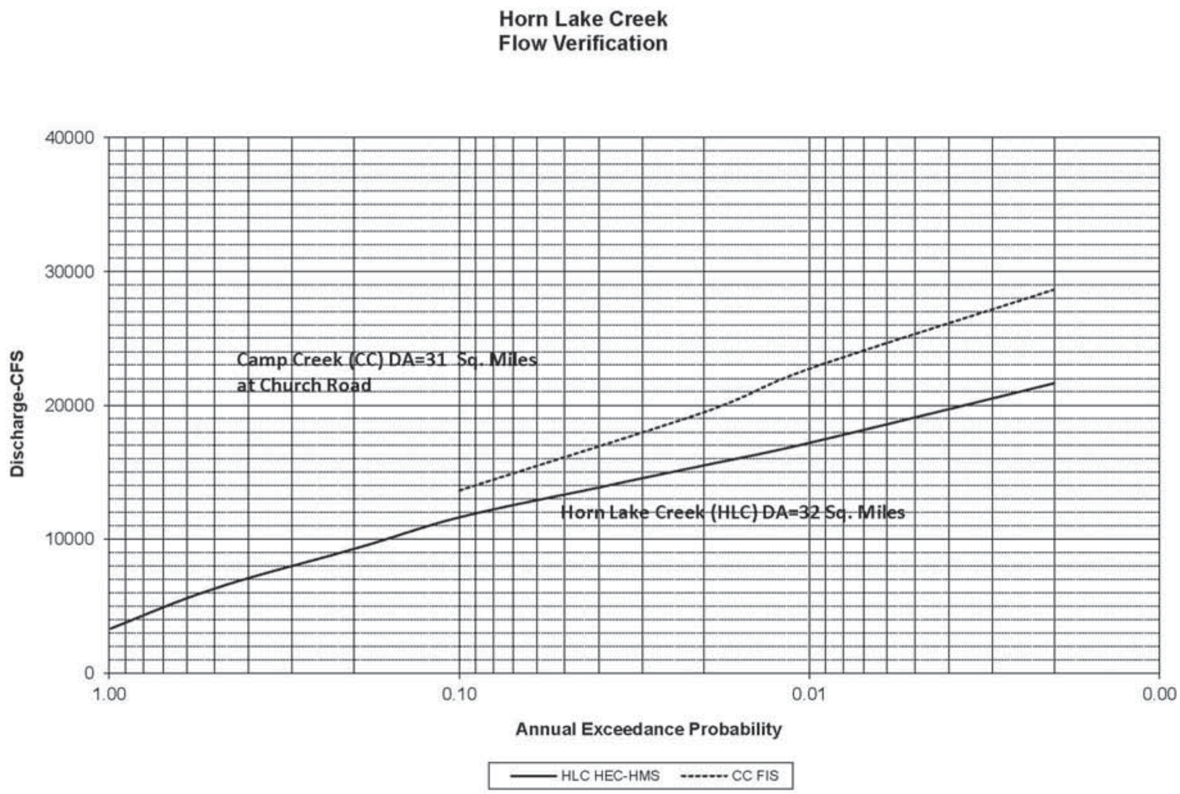

Figure 17. Horn Lake Creek Flow Verification
Approximate DA~31 square miles

7.0 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 any steam gages and flow calibration/verification was a challenge. With the lack of steam data, uncertainty parameters are based on a simplistic procedure which uses an estimated period of record. Guidance for using this procedure are 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.

Both the peak flows and the flow hydrographs used for hydraulic assessment were developed from hypothetical rainfall created by the NOAA publication Atlas 14. Based on guidance in EM 1110-2-1619, an Equivalent Record length ranging from 10 and 30 years should be used when flows were derived using an uncalibrated rainfall-runoff model (HEC-HMS). EM 110-2-1619 guidance is shown below. A period of record of 25 years was adopted for the study.

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.

Figure 18. EM 1110-2-1619, Table 4.5 Equivalent Record Length Guidelines

Shorter periods of record produce a larger uncertainty in the stage information and an estimate of 25 year would produce results based an upper uncertainty level and considered acceptable for the economic analysis. According to the FDA guidance, an equivalent record of this length corresponds to a standard deviation of 0.5 feet.

The standard deviation of 0.5 feet needed confirmation. EM 1619, Table 5-2, entitled "Minimum Standard Deviation of Error in Stage" states 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) and based on EM 1619 guidance, a value of 0.5 seems reasonable.

The previous studies used a standard deviation of 0.5 feet. This estimate is based on guidance from a HEC document entitled "Accuracy of Computed Water Surface Elevations" dated 1986. The guidance provides regression equations that use flood depths, stream slopes and the confidence in the Manning's equation estimate. These parameters were extracted from 1D HEC-RAS results to estimate stage uncertainty. The equations are shown below.

$$Emean = 0.076D^{0.60}\, S^{0.11}(5Nr+Sn)^{0.65}$$
$$Emax = 2.6E_{50}^{0.8}$$

where:
Emean= mean reach absolute profile error in feet
Emax = 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.

The equation for Emean was applied at select HEC-RAS River Stations that coincided with key economic evaluation locations. Coordination was conducted with the Economic PDT member to ensure data was compatible with the economic evaluation.

8.0 Hydraulic Modeling and Methodology

8.1 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 19 SHOWS HORN LAKE CREEK, COW PEN CREEK, ROCKY CREEK AND LATERAL D CURRENTLY MODELED IN A HEC-RAS 1D STEADY FLOW ENVIRONMENT.

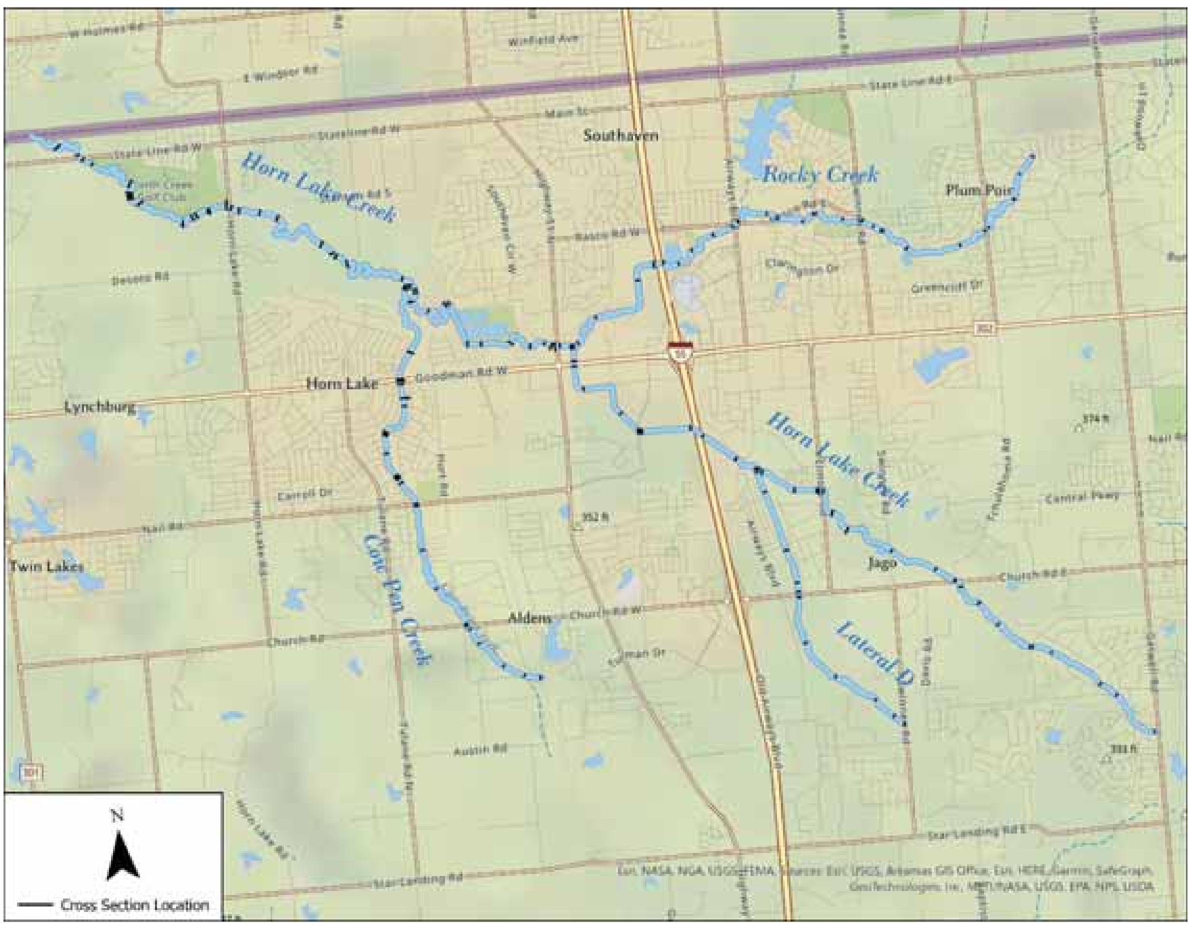

Figure 19. HEC-RAS 1D Modeled Streams

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 IN AN EFFORT TO EXPEDITE THE STUDY.

8.1.1 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.

8.1.2 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.

8.1.3 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.

8.1.4 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.

8.2 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 20 shows the main streams of investigated for flood risk management and reduction measures.

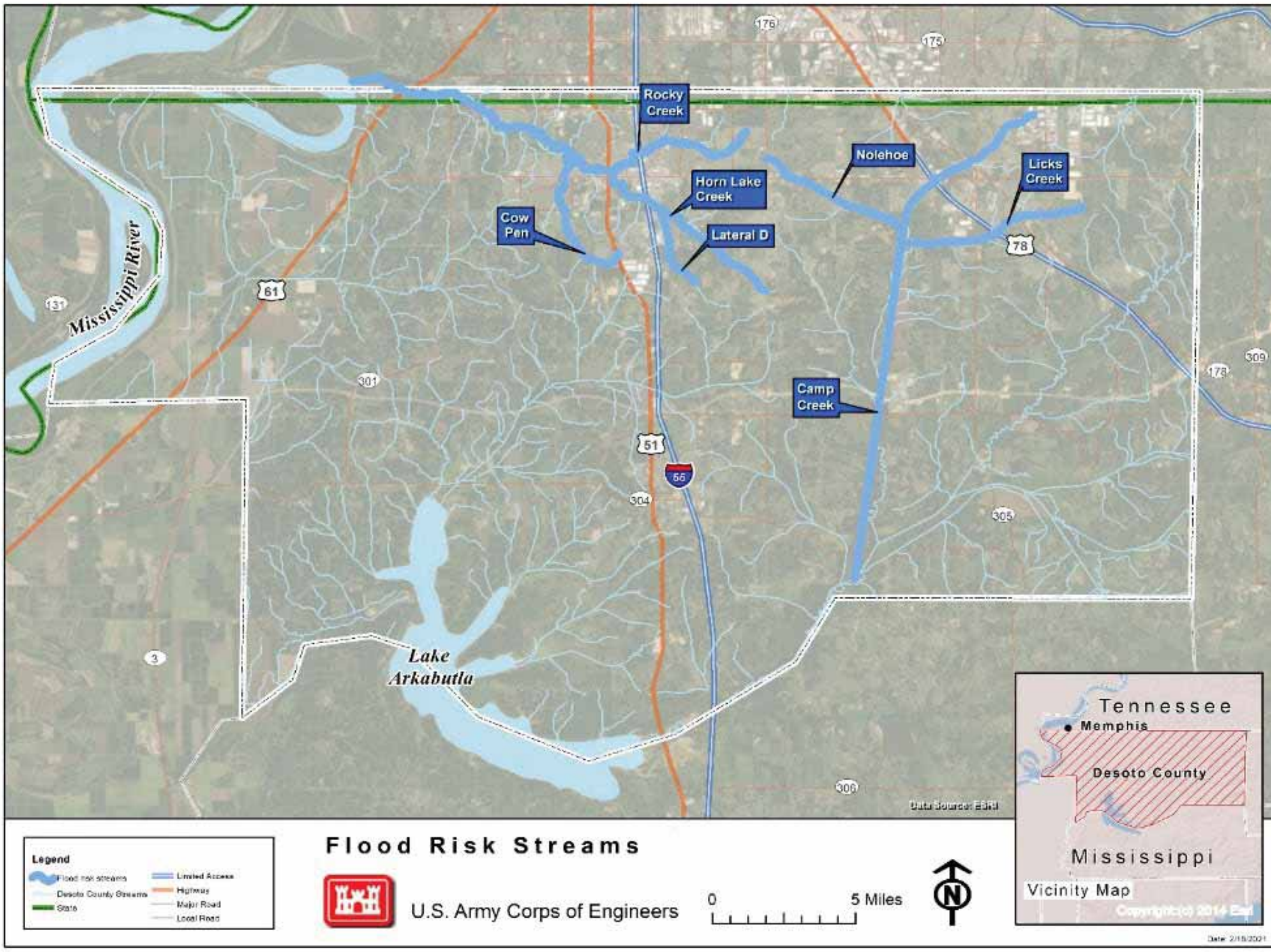

Figure 20. Coldwater River Basin Flood Risk Management Streams of Interest

8.3 Existing Conditions

8.3.1 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 were evaluation developed for the same probabilistic events. The following figures show resulting HEC-RAS water surface profile plots.

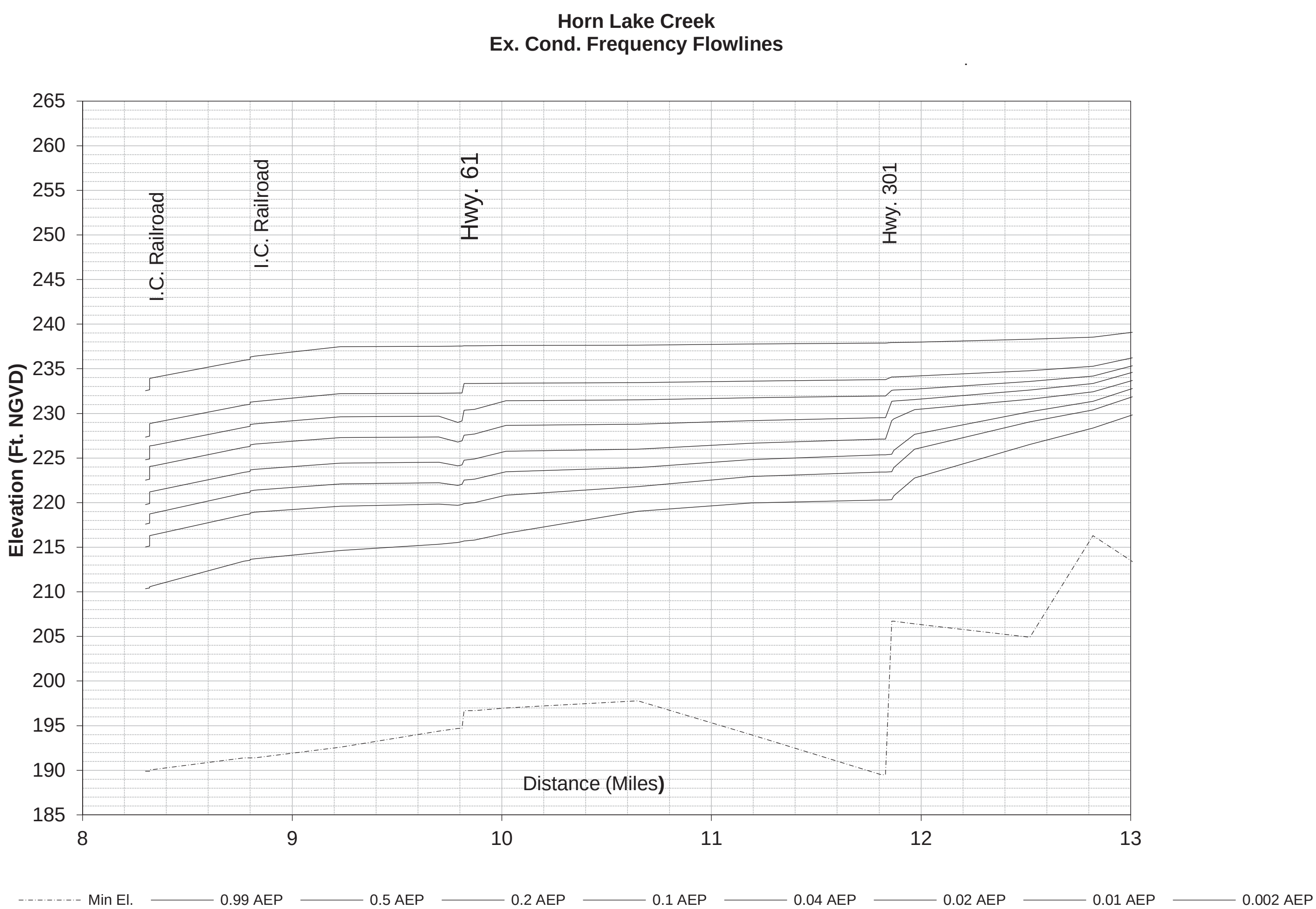

Figure 21. Horn Lake Creek Frequency Flowlines

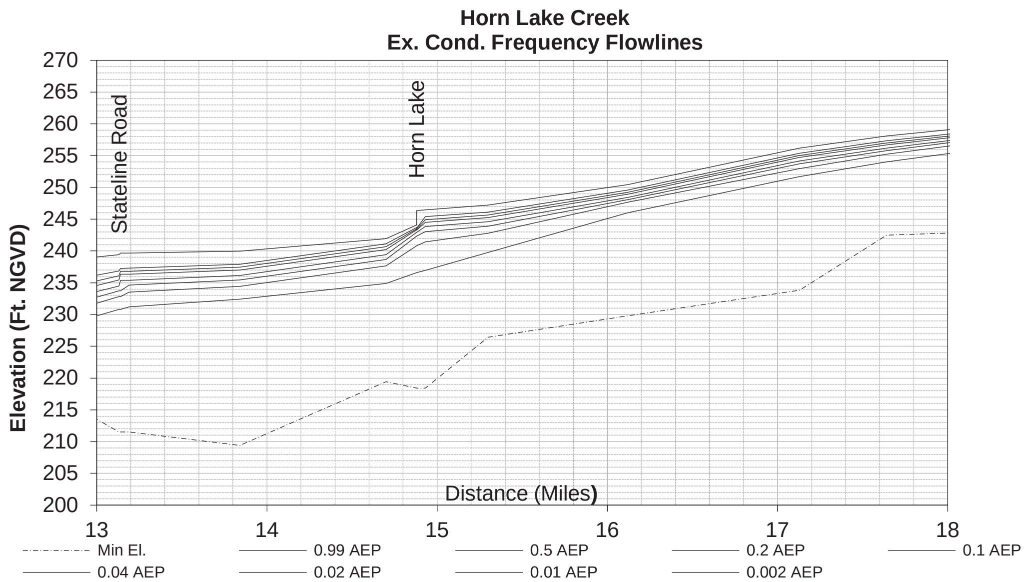

Figure 22. Horn Lake Creek Frequency Flowlines

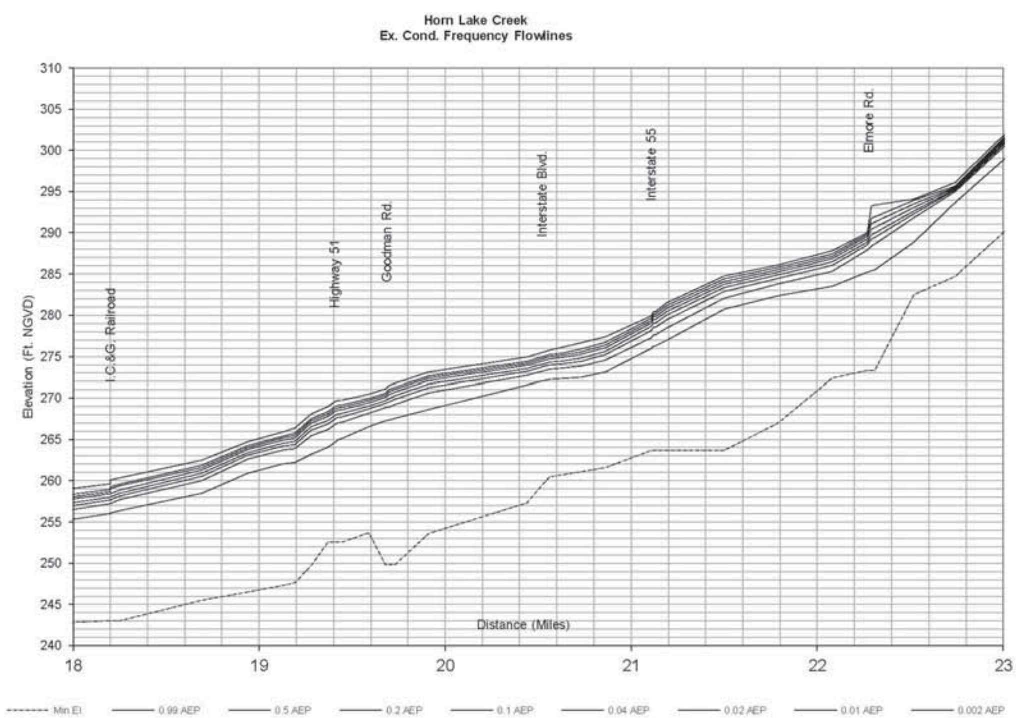

Figure 23. Horn Lake Creek Frequency Flowlines

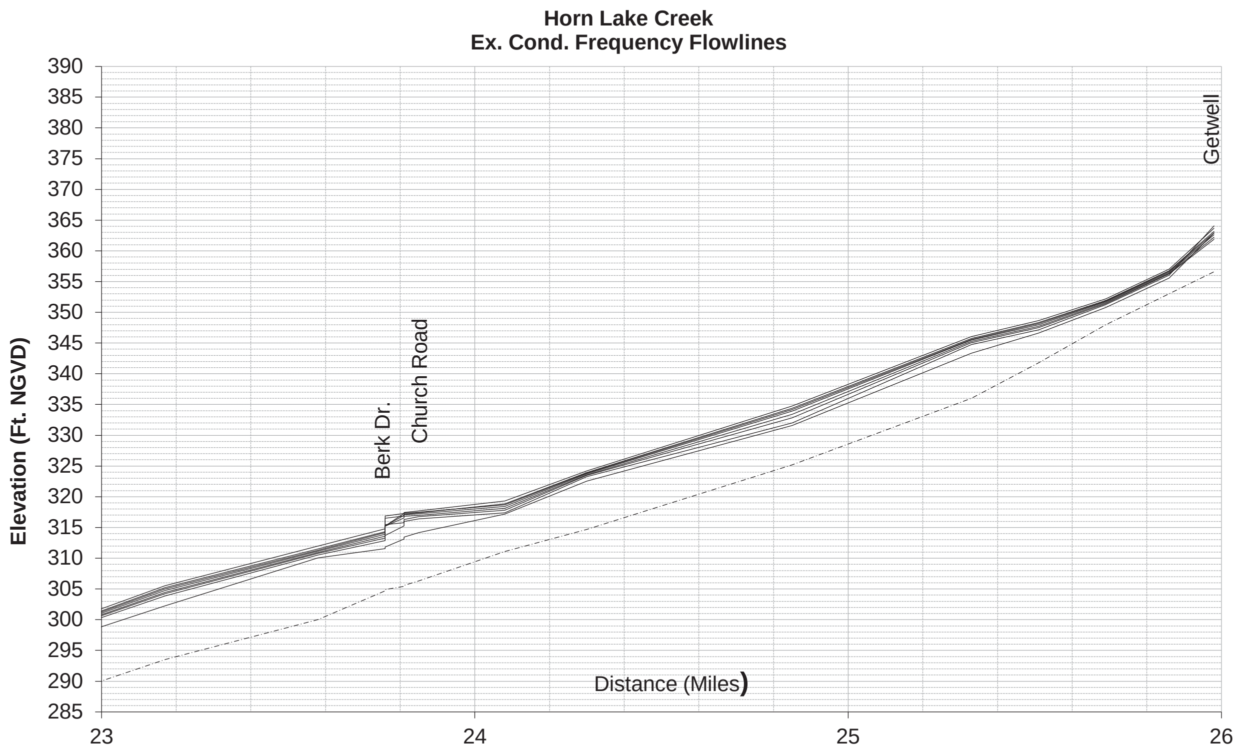

Figure 24. Horn Lake Creek Frequency Flowlines

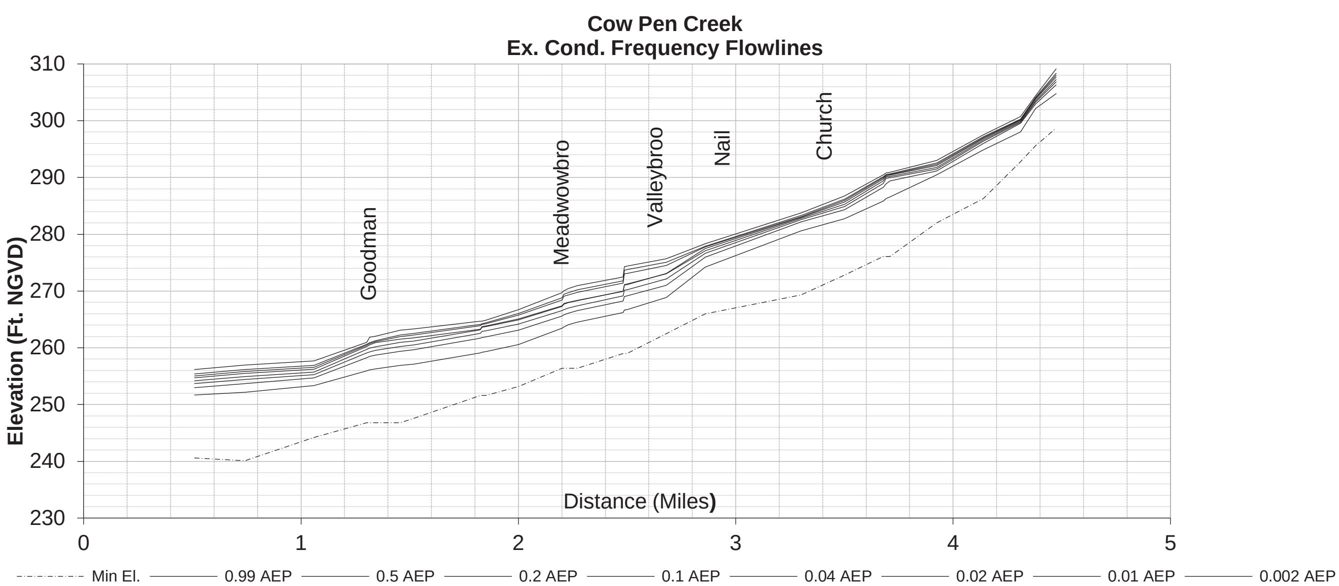

Figure 25. Cow Pen Creek Frequency Flowlines

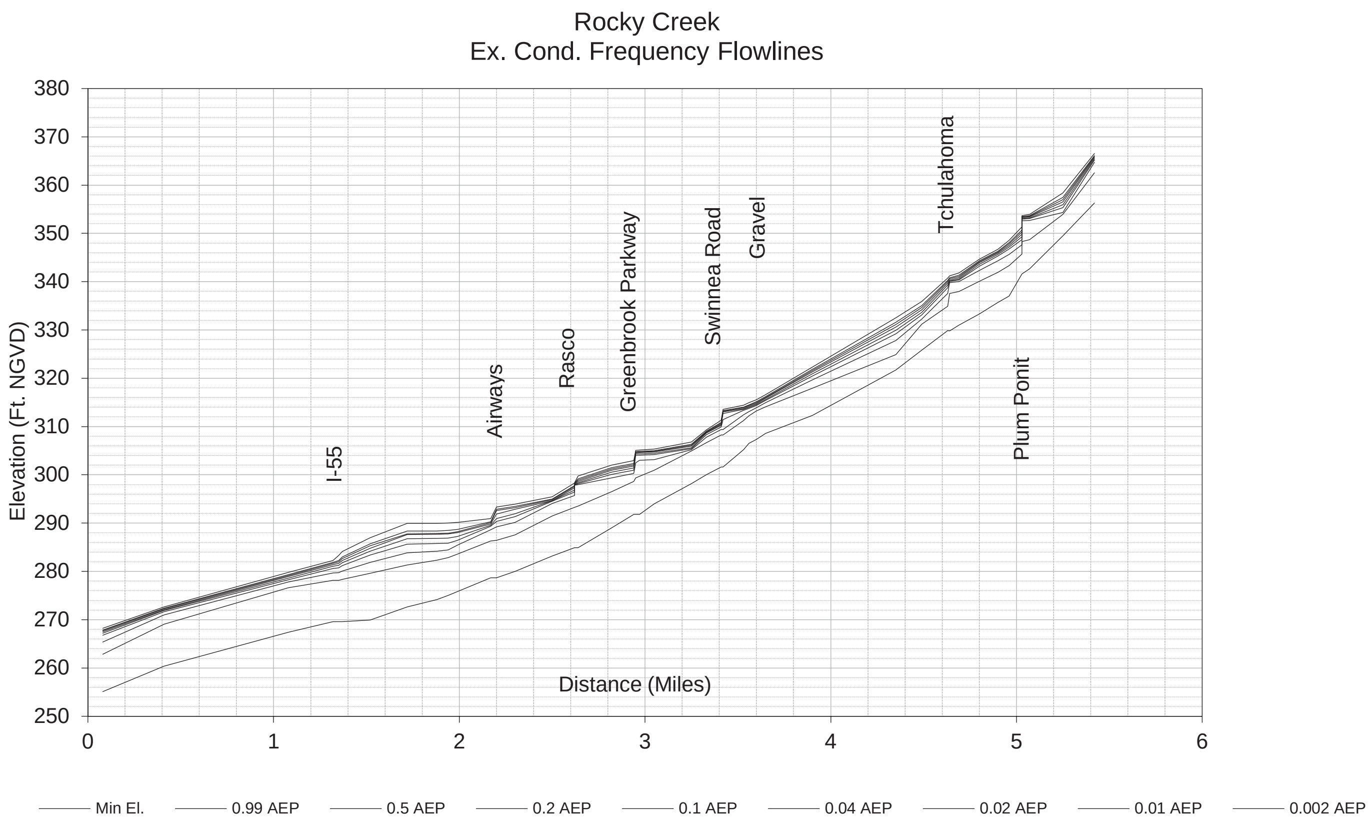

Figure 26. Rocky Creek Frequency Flowlines

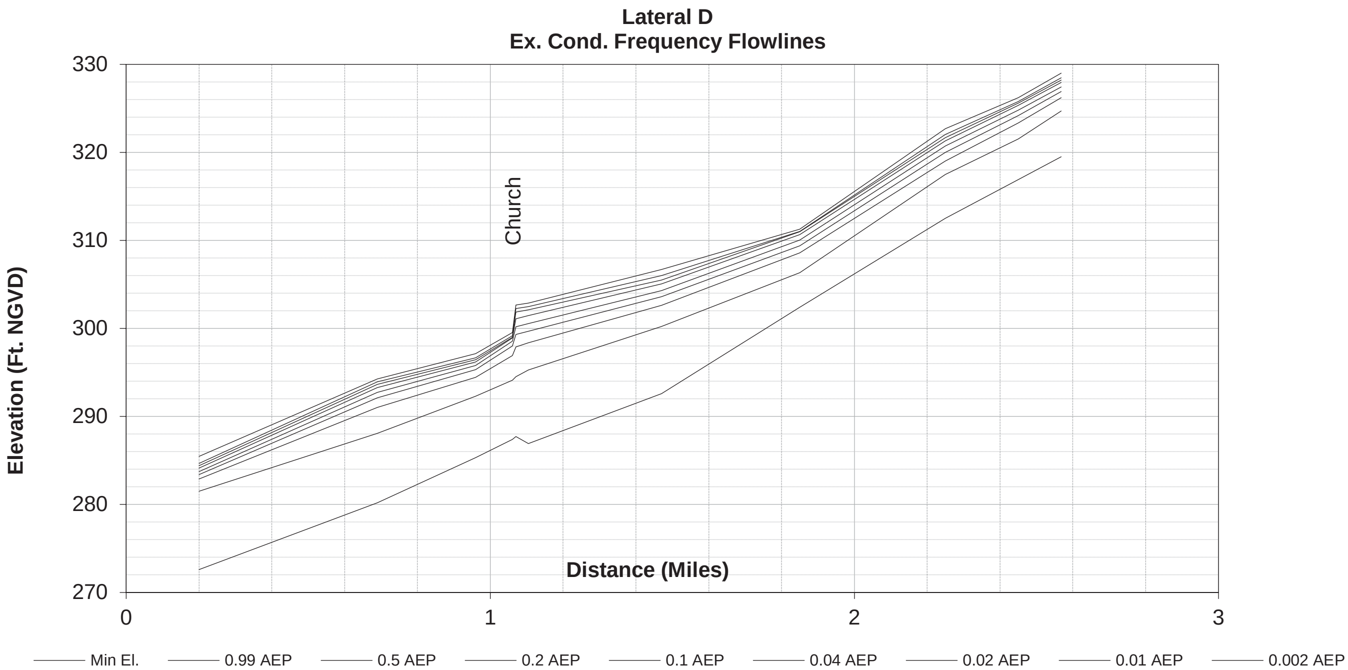

Figure 27. Lateral D Frequency Flowlines

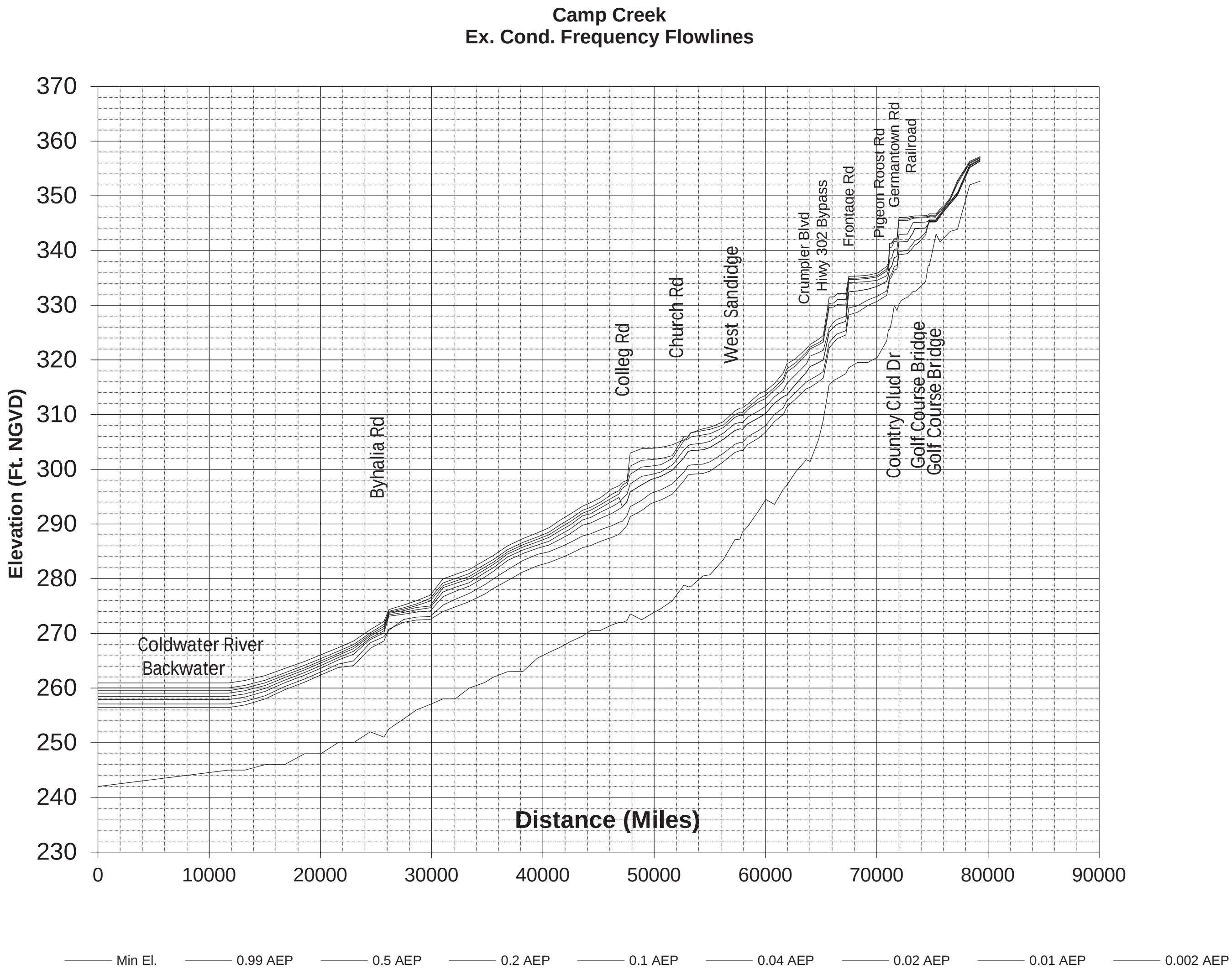

Figure 28. Camp Creek Frequency Flowlines

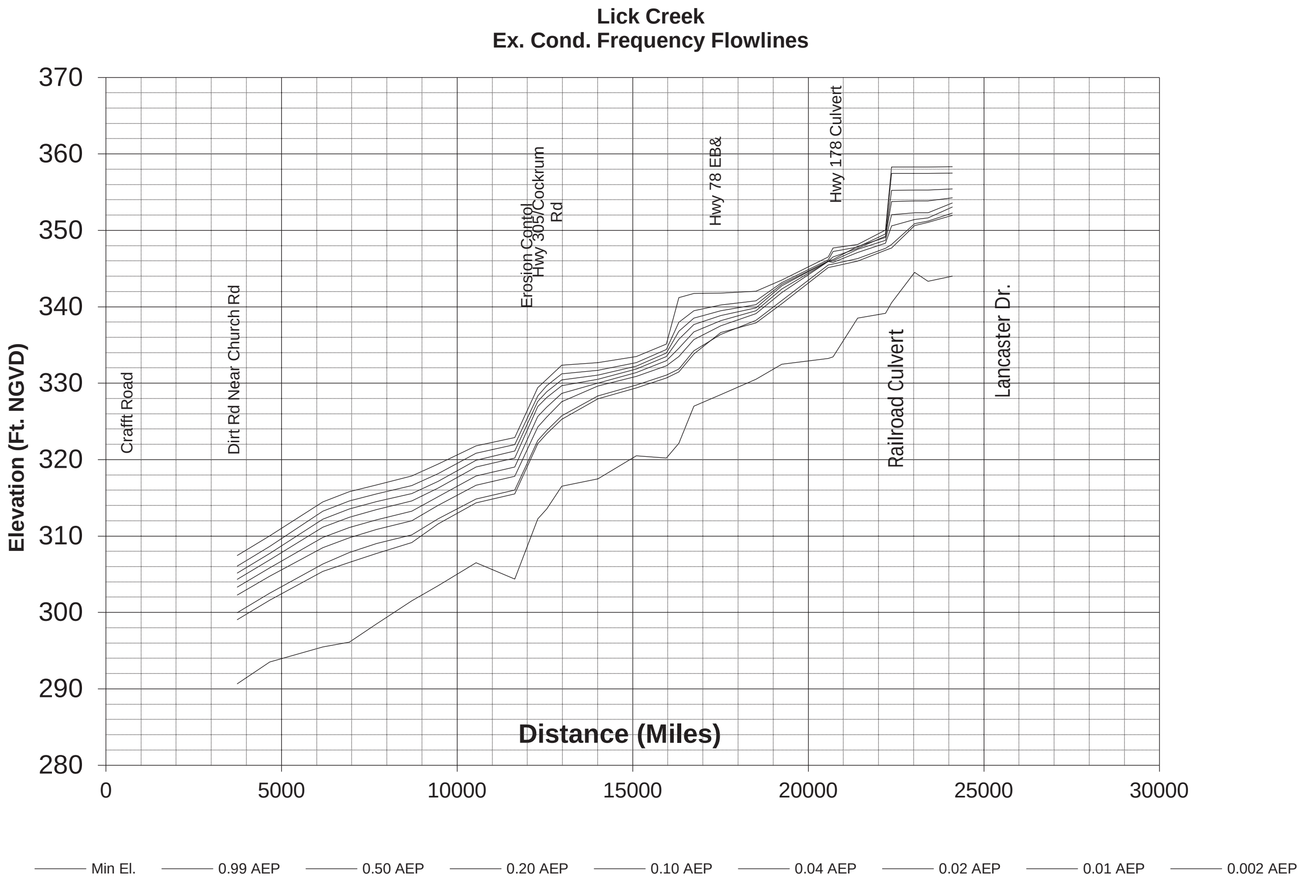

Figure 29. Lick Creek Frequency Flowlines

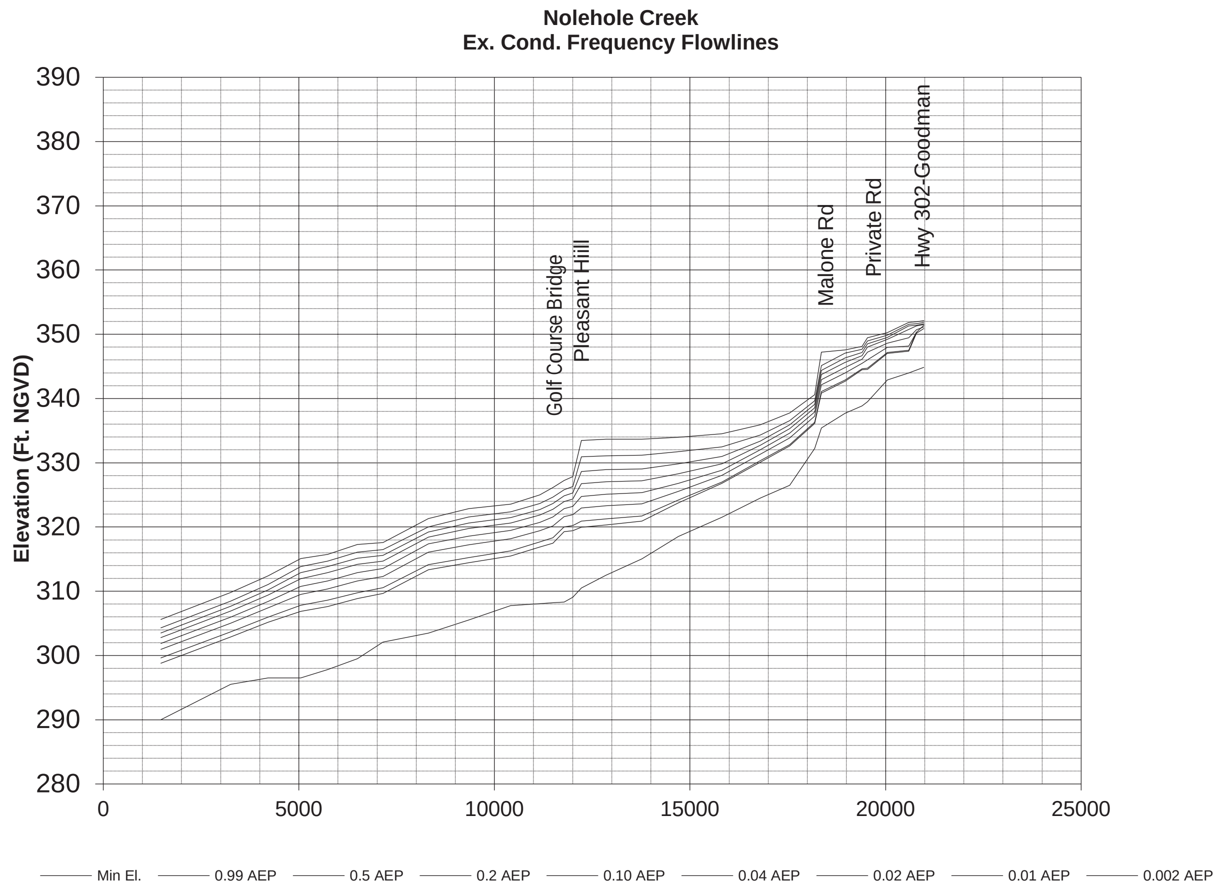

Figure 30. Nolehoe Creek Frequency Flowlines

9.0 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 4. It should be noted, the study included non-structural features and are also included in the table 4.

Table 4. Initial Array of Alternatives

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

Alt ID	Description	Measures Included
NS-50yr	0.02 AEP Nonstructural Aggregation	Elevating Residential and Flood proofing Commercial Structures
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 31

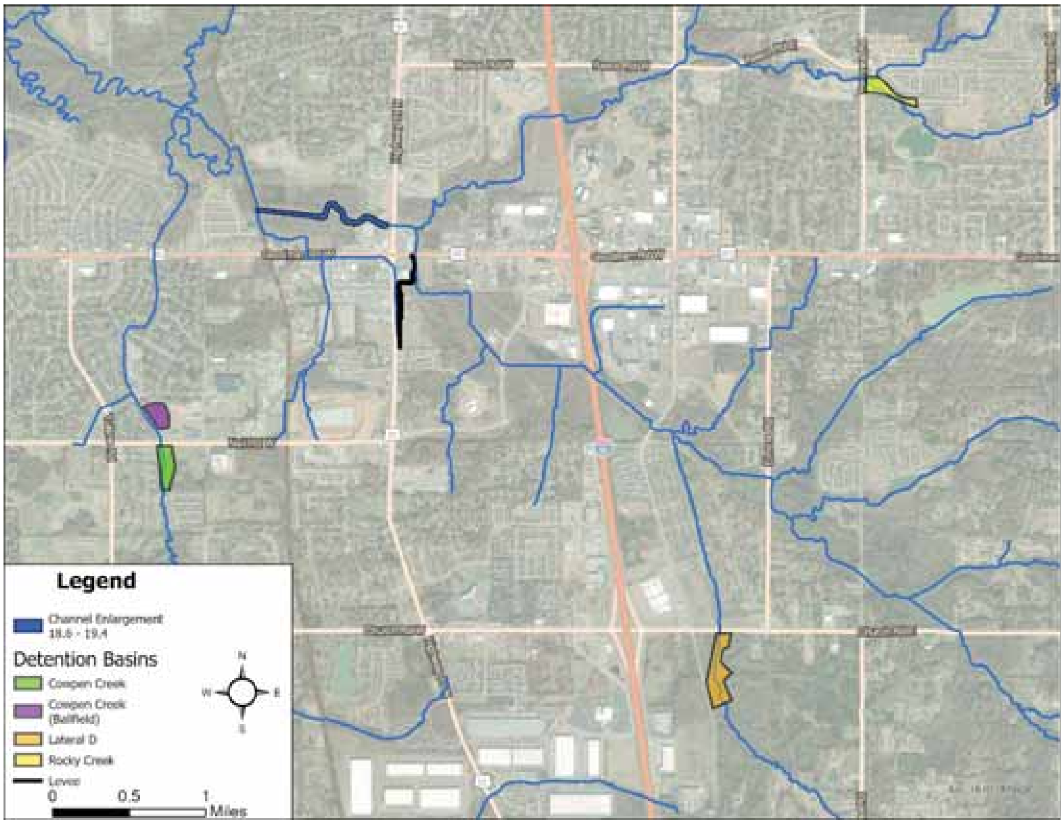

Figure 31. Flood Risk Management Alternative Sites

9.1 Horn Lake Creek Channel Enlargement

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. The downstream impacts of the channel enlargement are presented section 9.1.2.

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 3H to 1V 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 32 shows a plan view of the enlargement reach.

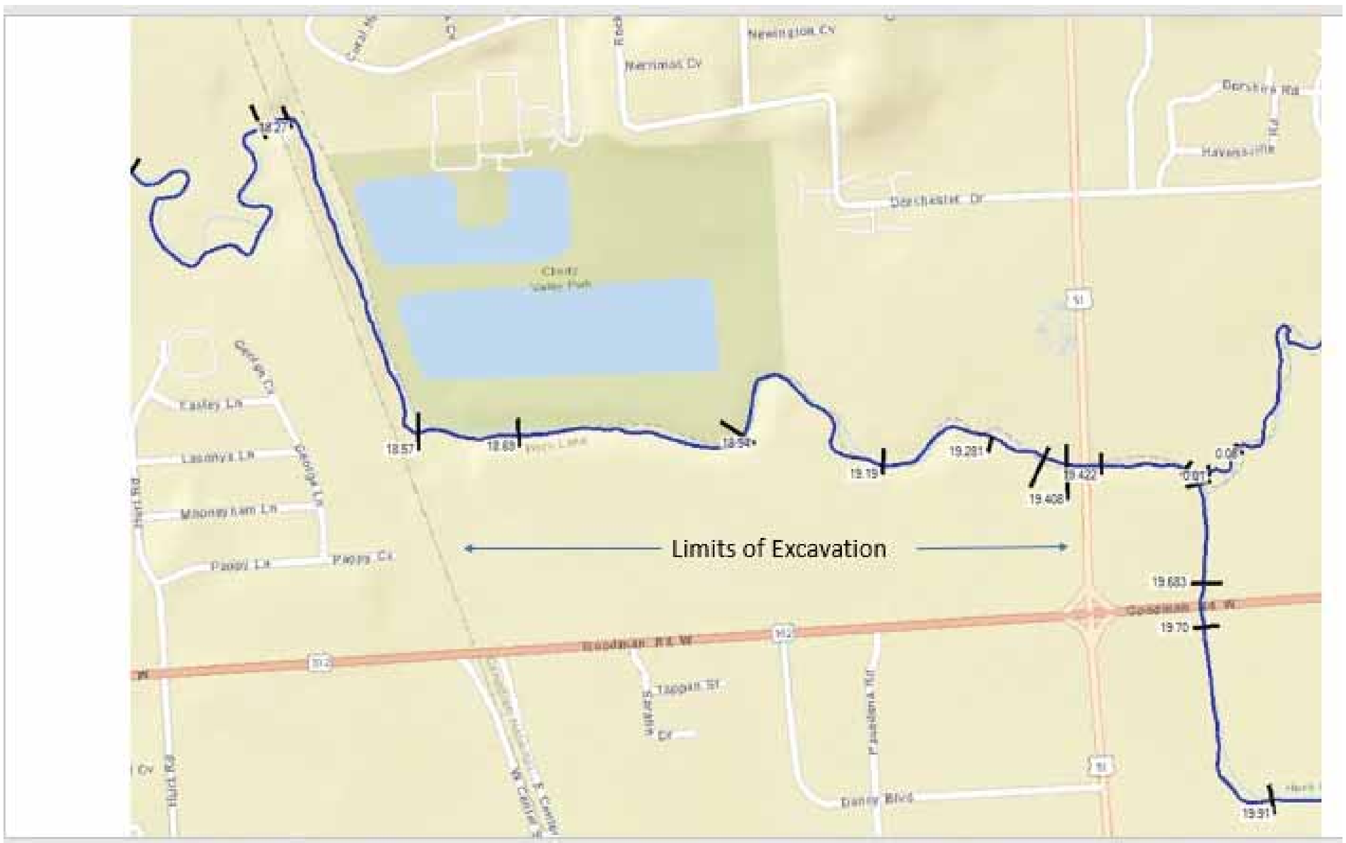

Figure 32. Horn Lake Creek Channel Enlargement

9.1.1 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 33 to 35.

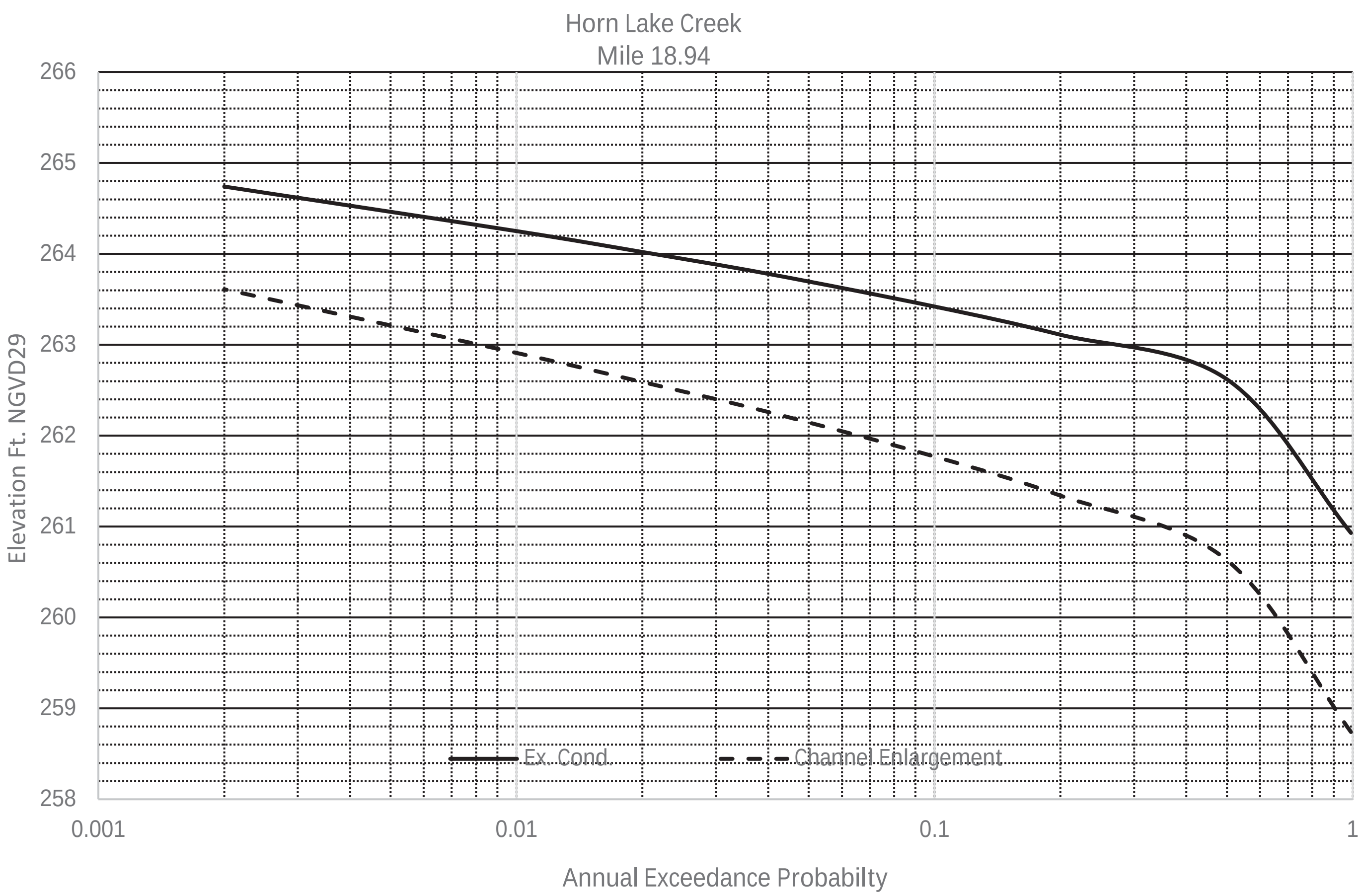

Figure 33- Water Surface Elev. Vs. Probability Curve

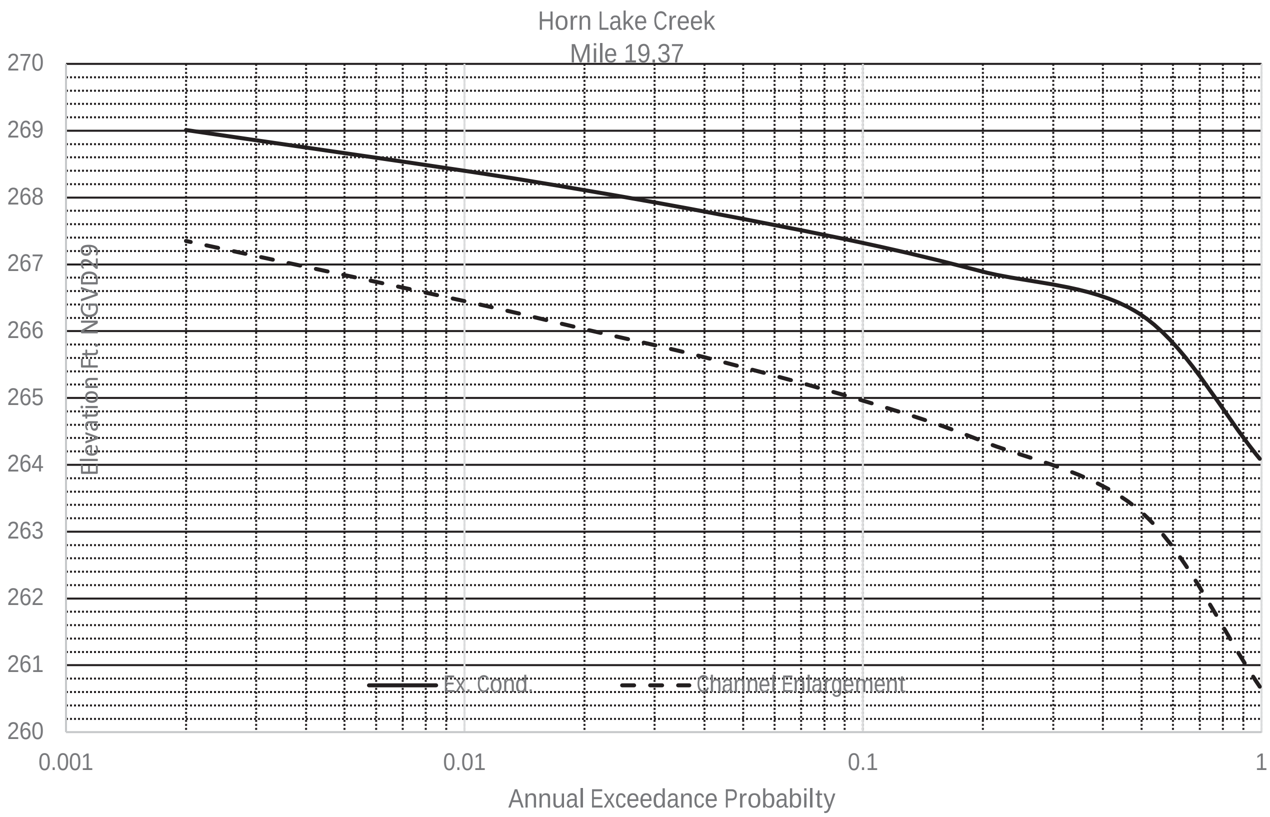

Figure 34- Water Surface Elev. Vs. Probability Curve

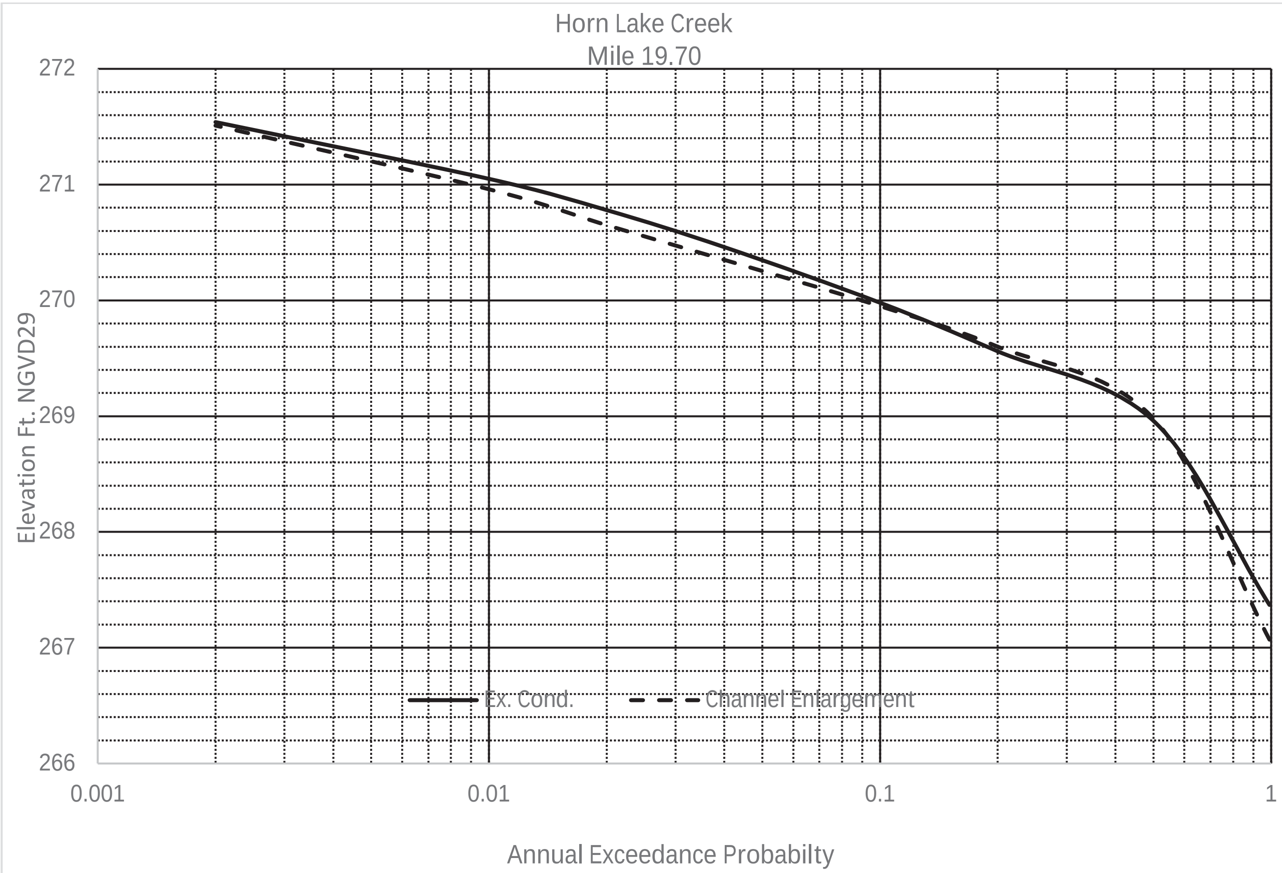

Figure 35-Water Surface Elev. Vs. Probability Curve

9.1.2 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 36. The TSP is composed of the subject channel enlargement combined with a detention basin on Lateral D at Church Road. It should be noted the 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 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.

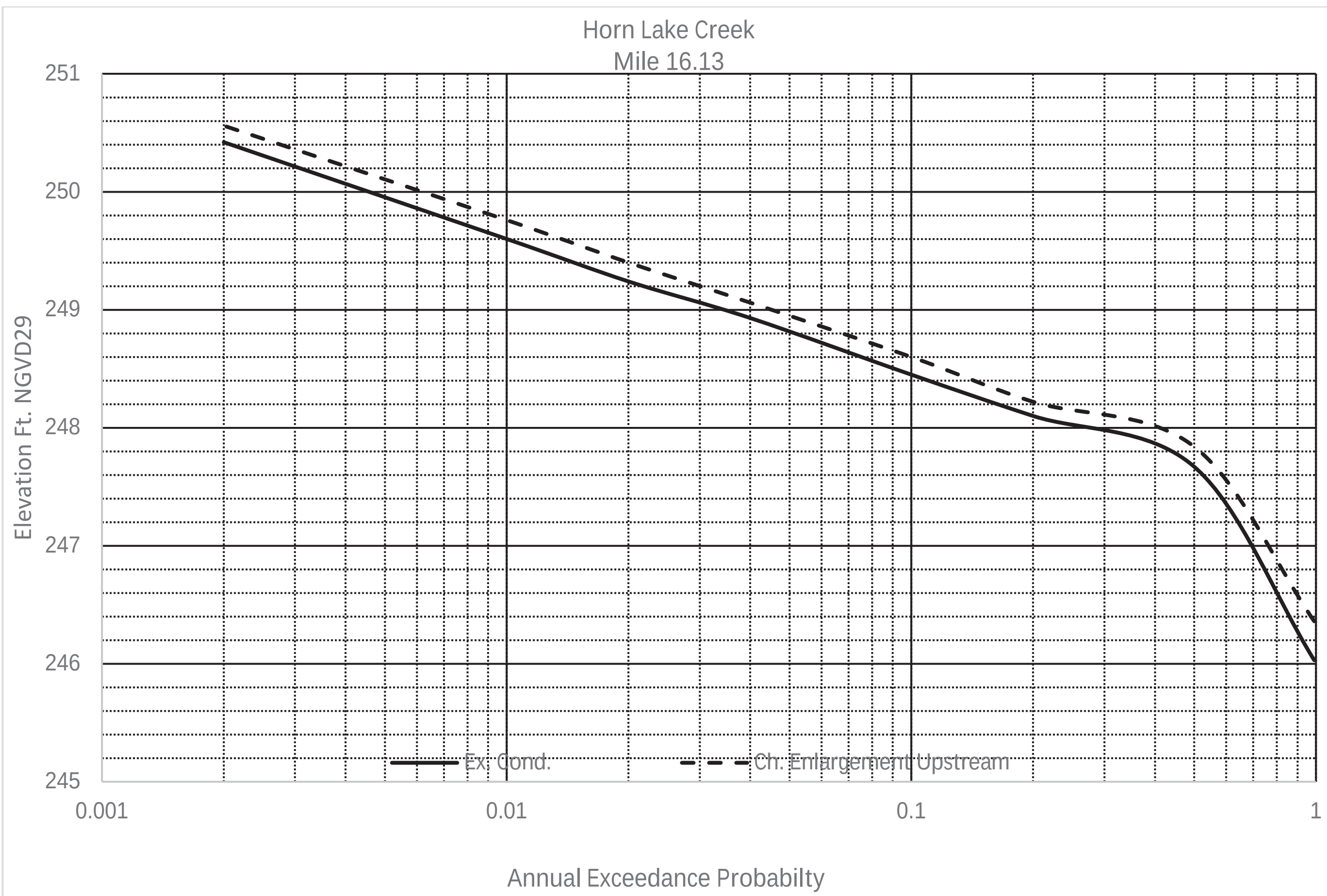

Figure 36-Water Surface Elev. Vs. Probability Curve

9.1.3 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 the National Ecosystem Restoration Appendix XX 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.

10.0 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:

a. Inflow design flood (IDF) computed using the Probable Maximum Flood.
b. Inflow unit hydrograph peaked 25 to 50 percent.
c. Runoff ratio should be 90 percent or higher.

d. 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.
e. Regulating outlets assumed to be inoperable.
f. 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.

11.0 Cow Pen Creek Basin Detention Analysis

11.01 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 3H to 1V. The locations of detention basins are shown in Figure 37.

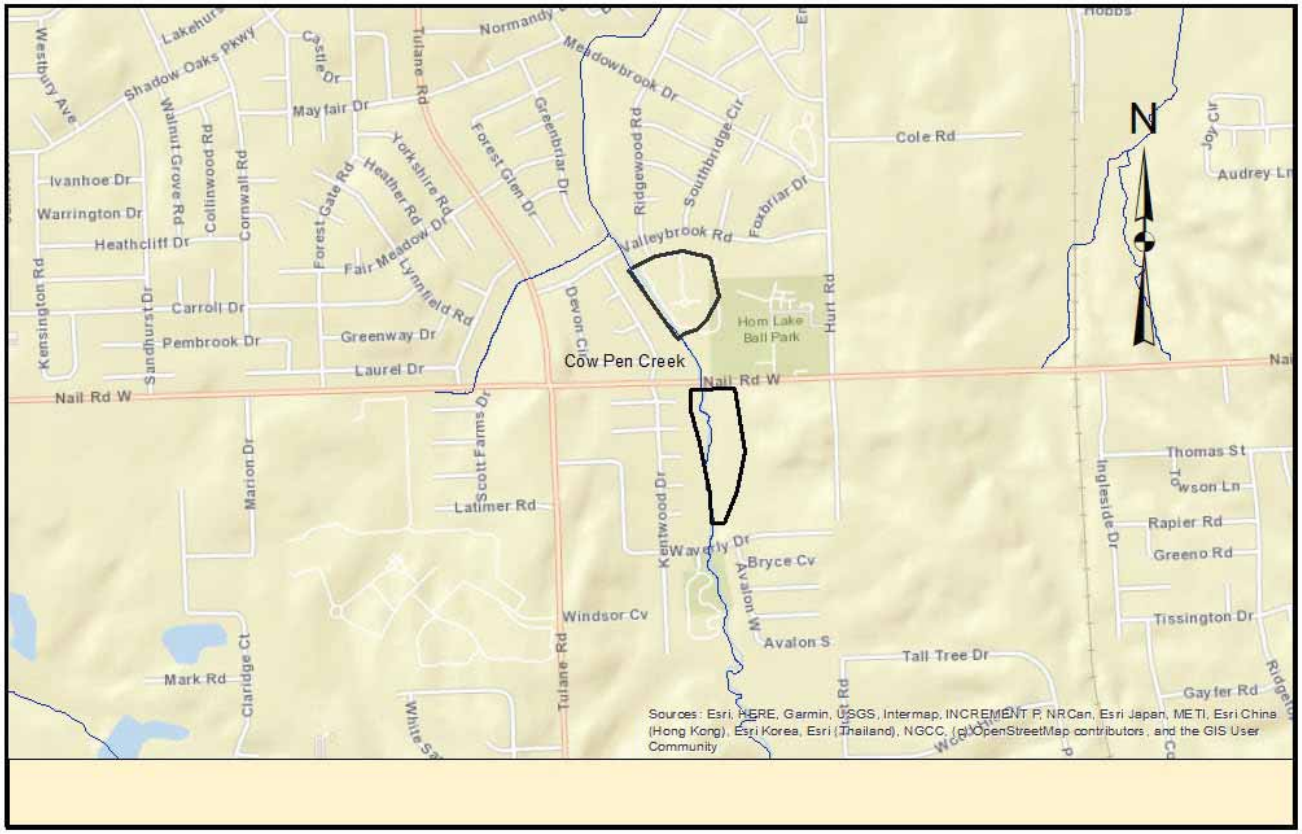

Figure 37- Cow Pen Creek Detention Basins

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.

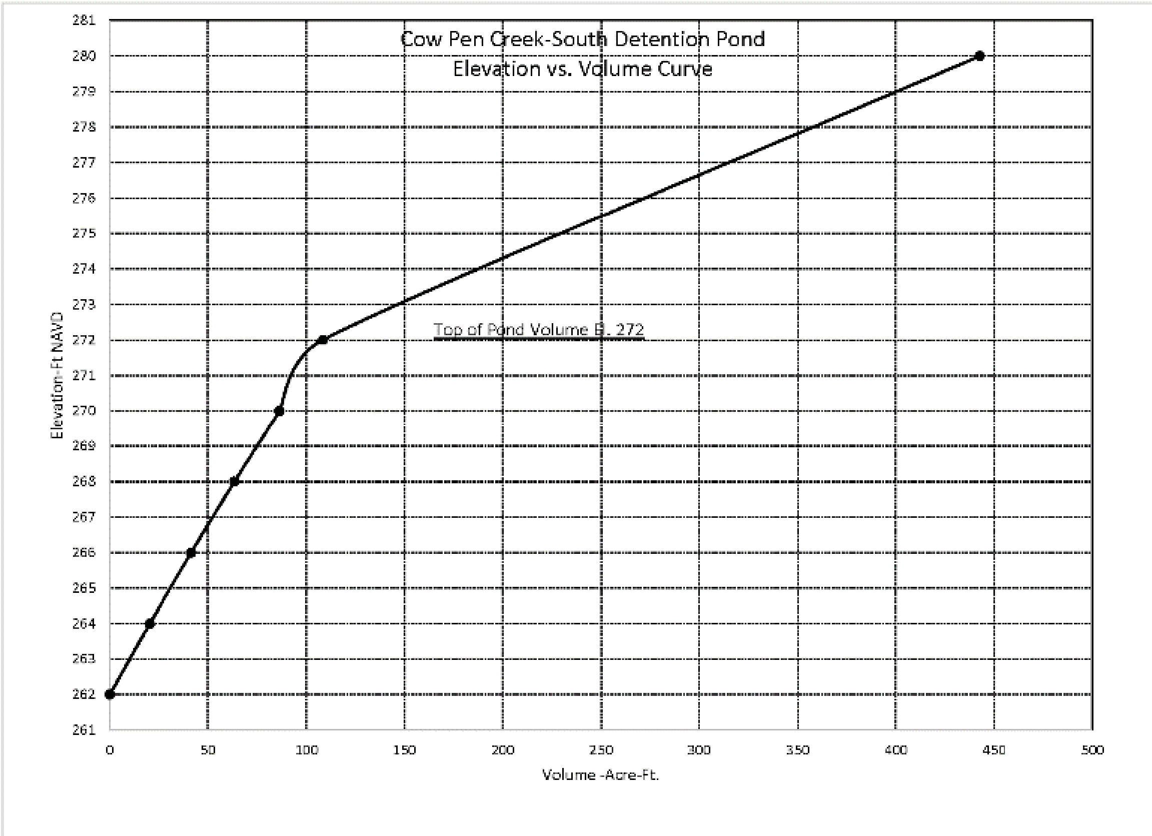

Figure 38- Cow Pen Creek South-Volume Curve

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.

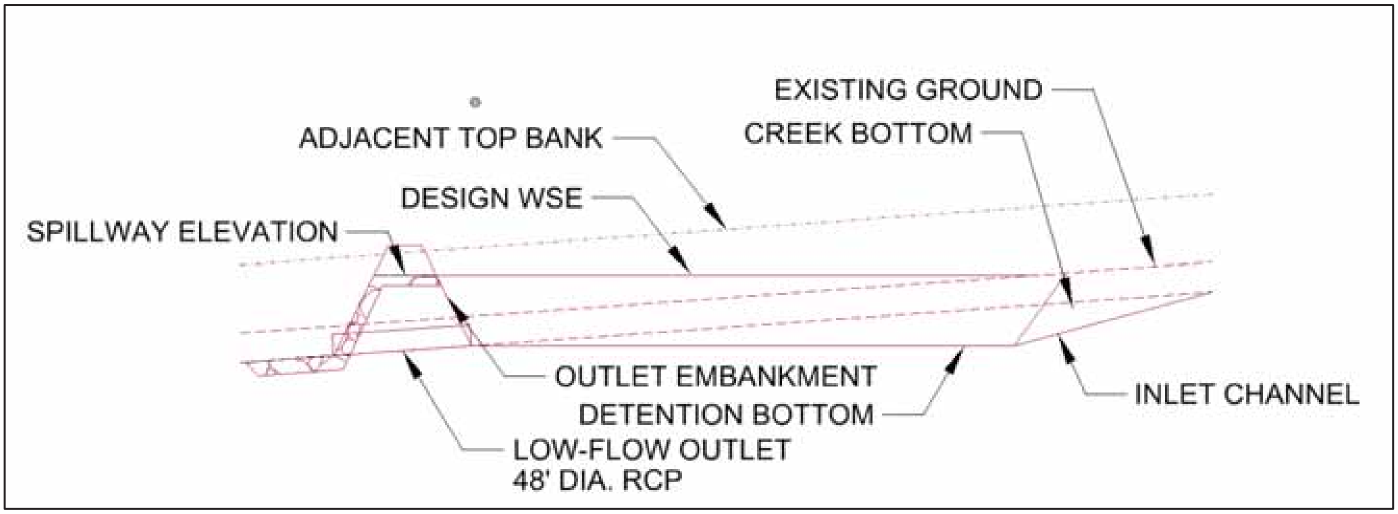

Figure 39- Typical Detention Basin

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 0.002 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.

11.02 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.

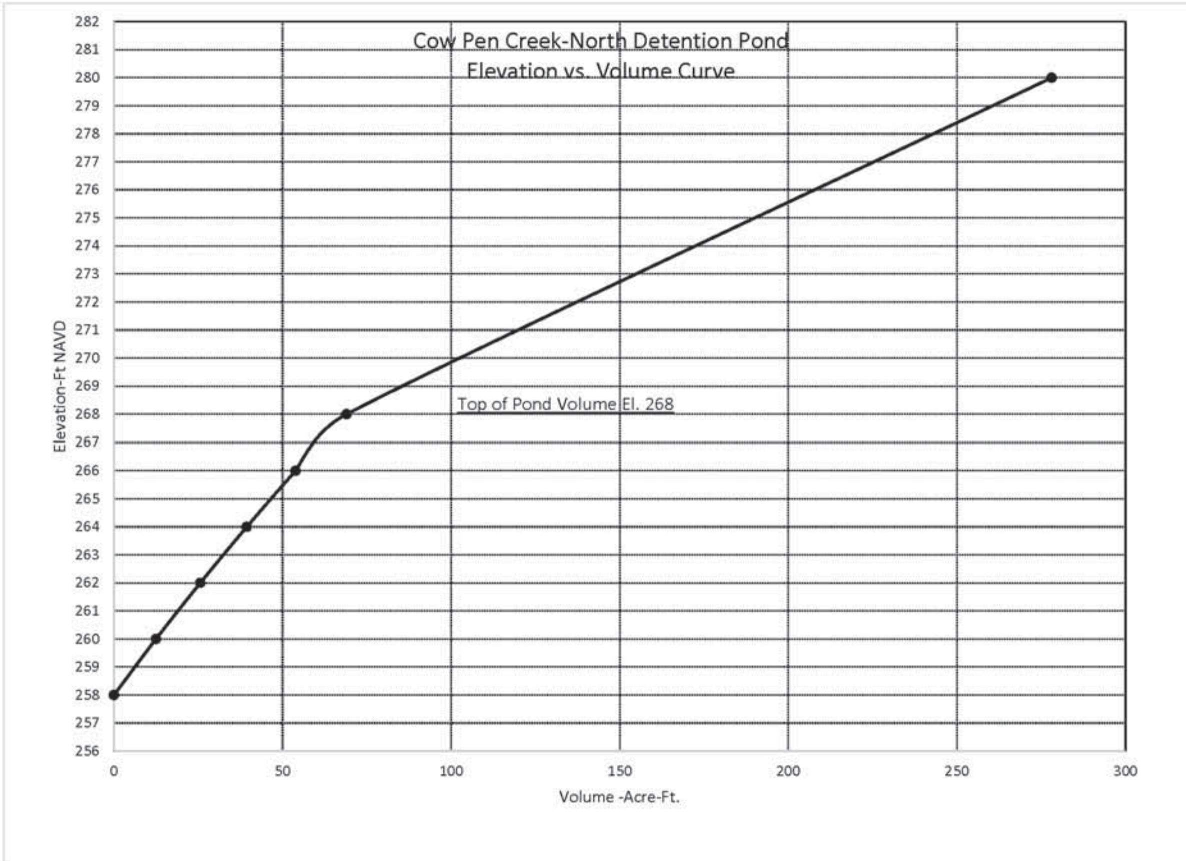

Figure 40- Cow Pen Creek North-Volume Curve

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 0.002 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.

11.03 Cow Pen Creek Frequency vs. Elevation Curves

The reduction in flows are 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.

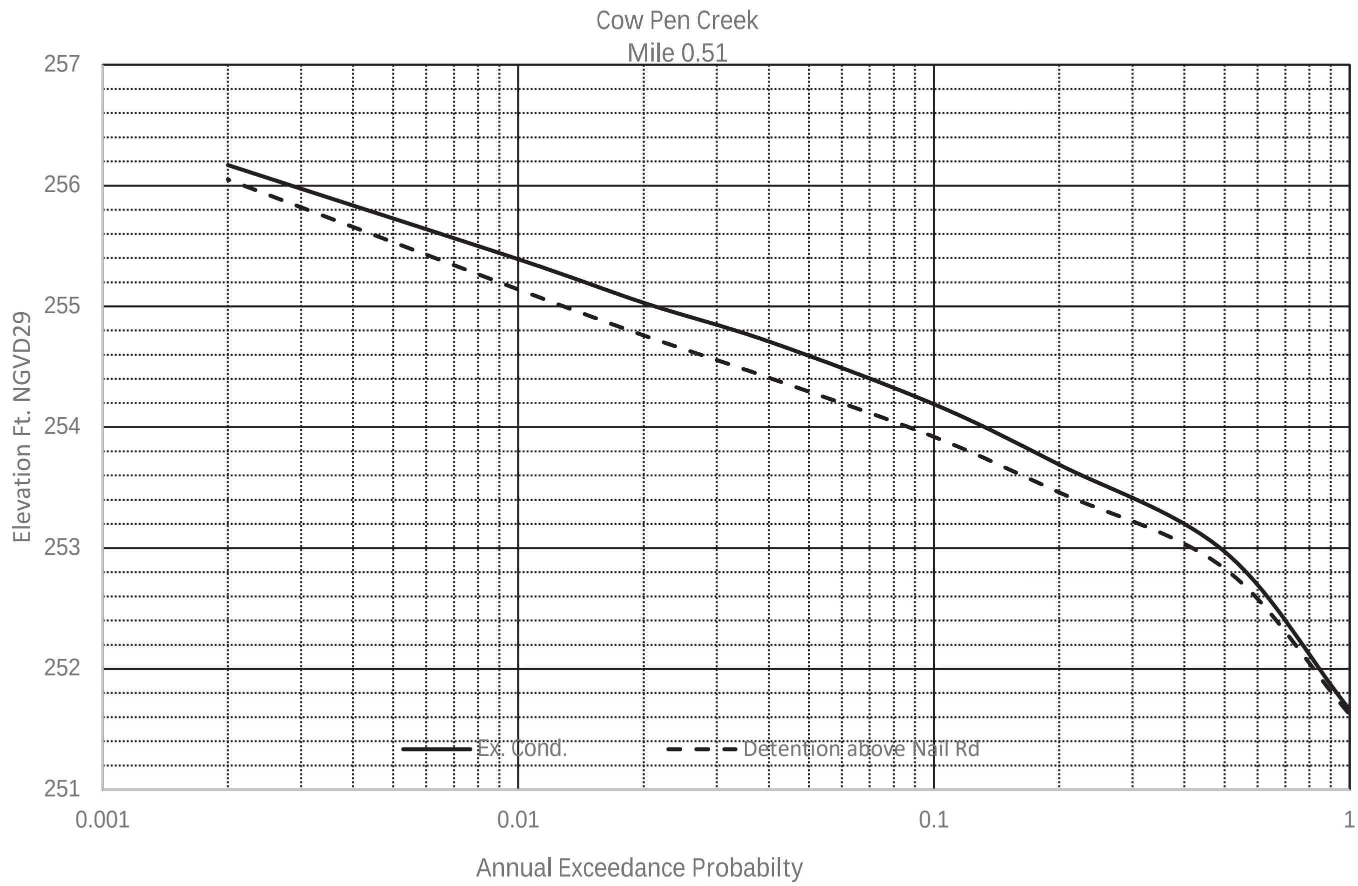

Figure 41-Water Surface Elev. Vs. Probability Curve

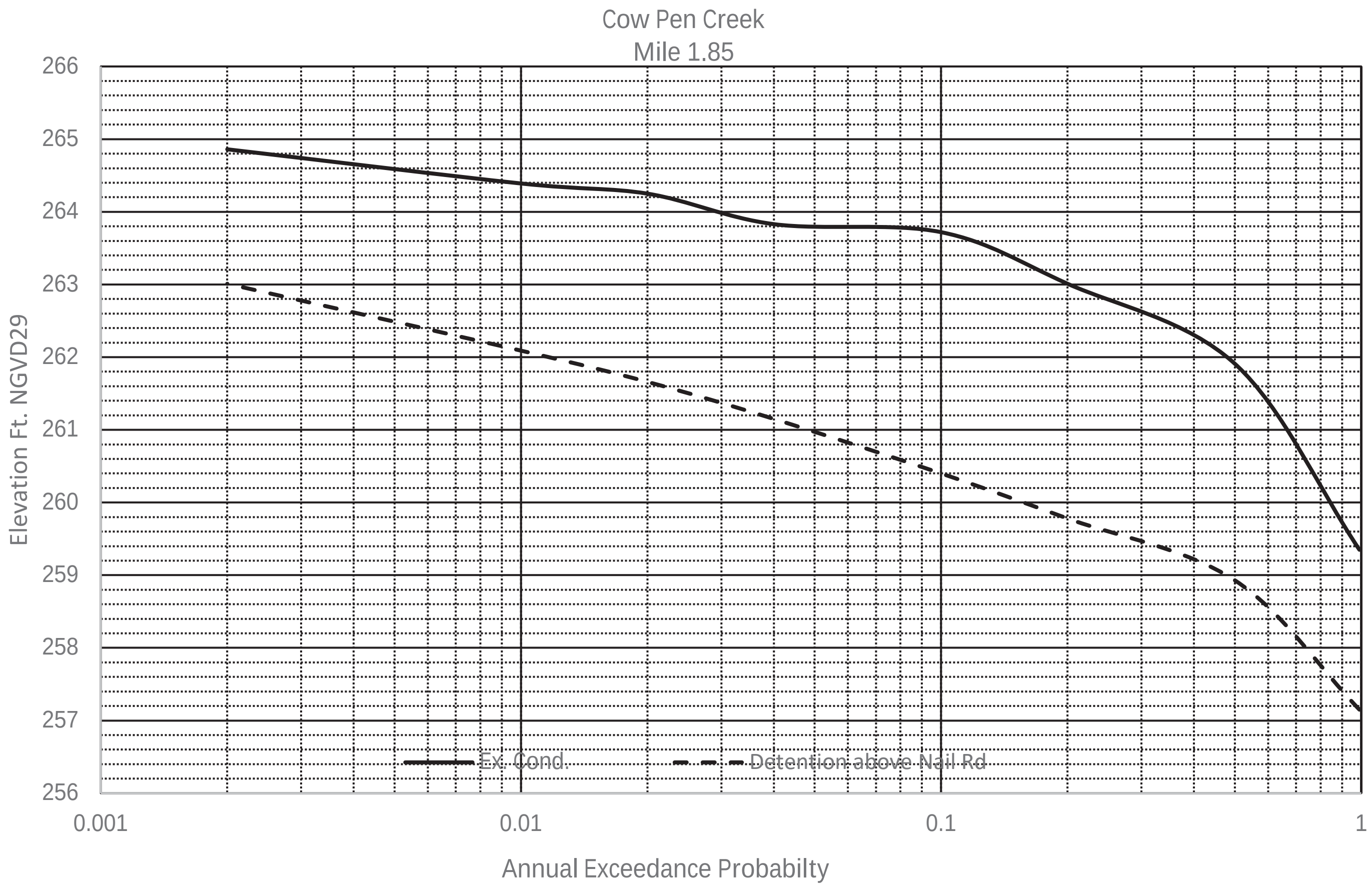

Figure 42-Water Surface Elev. Vs. Probability Curve

12 Rocky Creek Basin Detention Analysis

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. The pool bottom area is six acres. All slopes back up to grade shall be 3H to 1V. The site is shown in Figure 43.

Figure 43- 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 at the 0.50 AEP event. The maximum storage of the detention pond is 72 acre-et. The total storage curve used in HEC-HMS is shown in Figure 44.

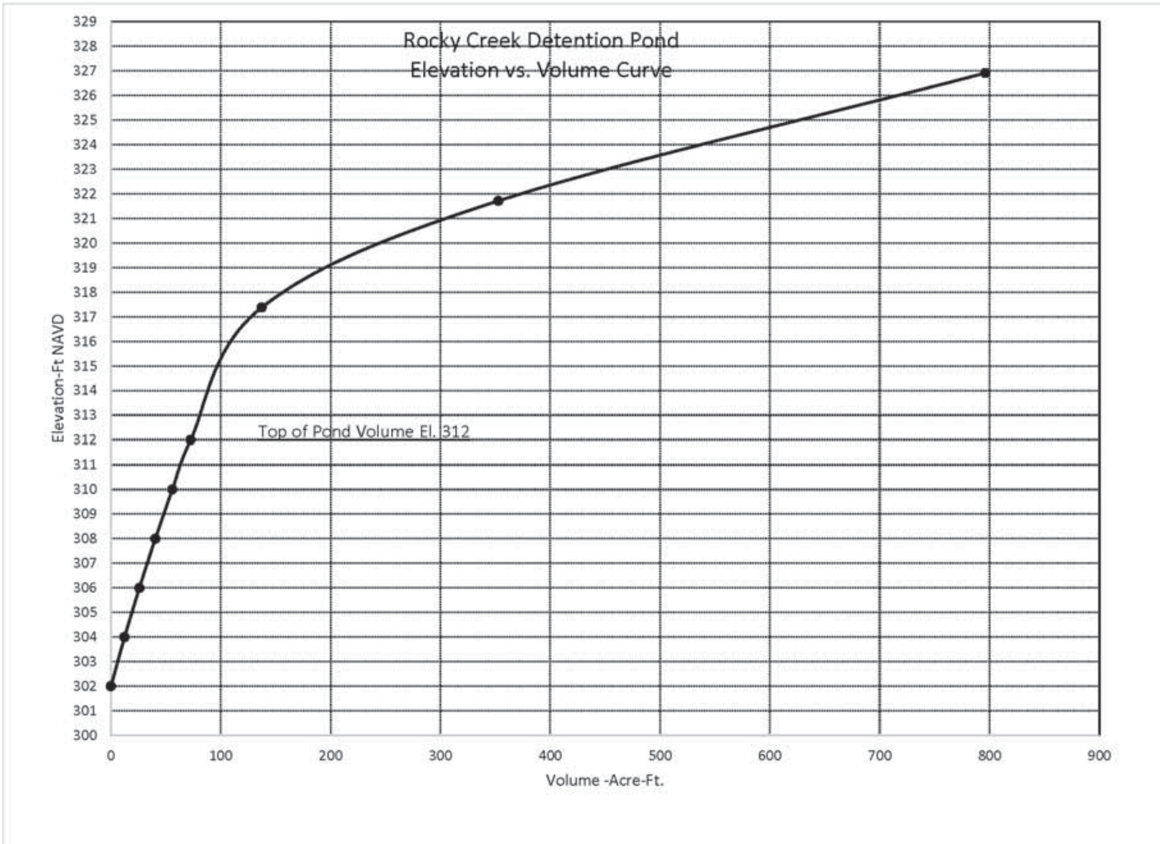

Figure 44- Rocky Creek-Volume Curve

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.

13 Lateral D Detention Basin

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, bottom area of 16 acres, and shall be sloped back up to grade at 3H to 1V. The site is shown in Figure 45.

Figure 45- 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, at the 0.50 AEP event. The maximum storage of 177 acre-feet requires approx. 350,000 CY of excavation. Figure 46 shows the total volume curve.

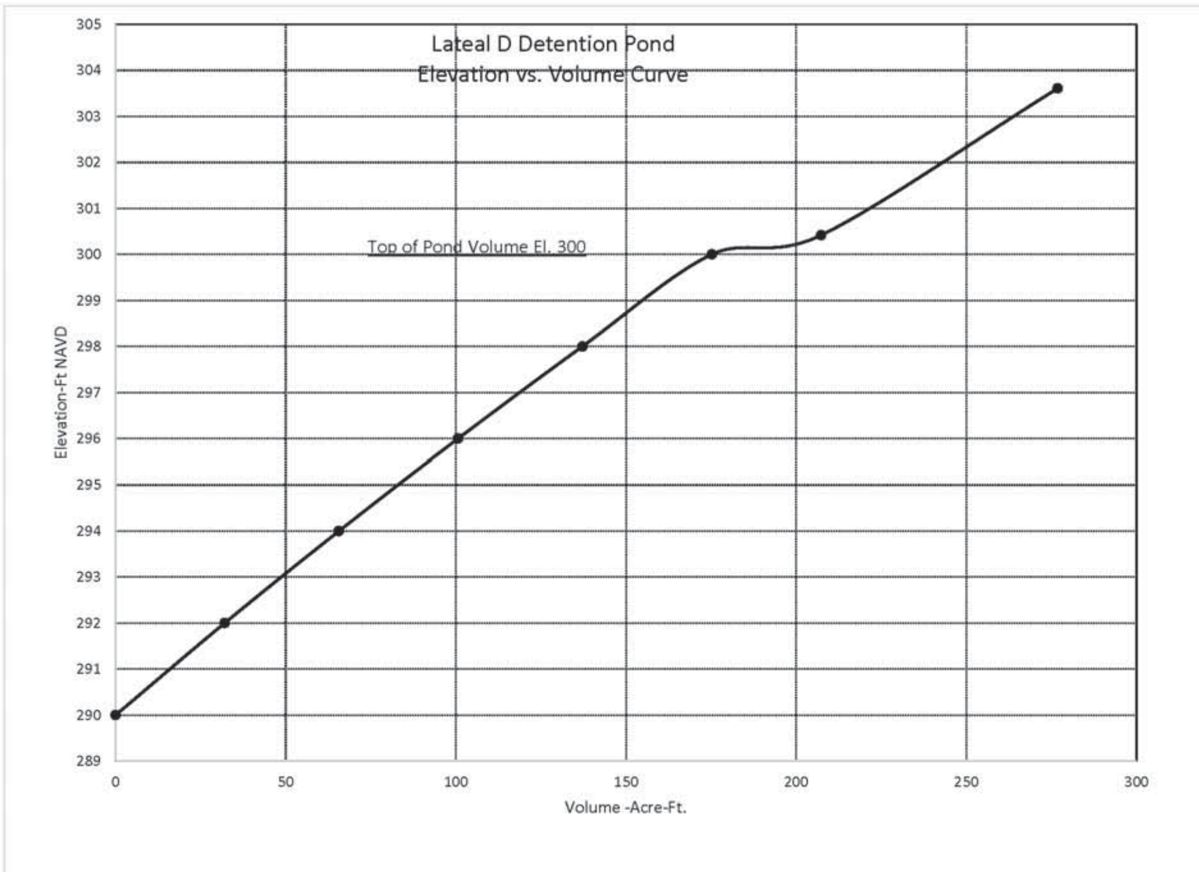

Figure 46- Lateral D-Volume Curve

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.

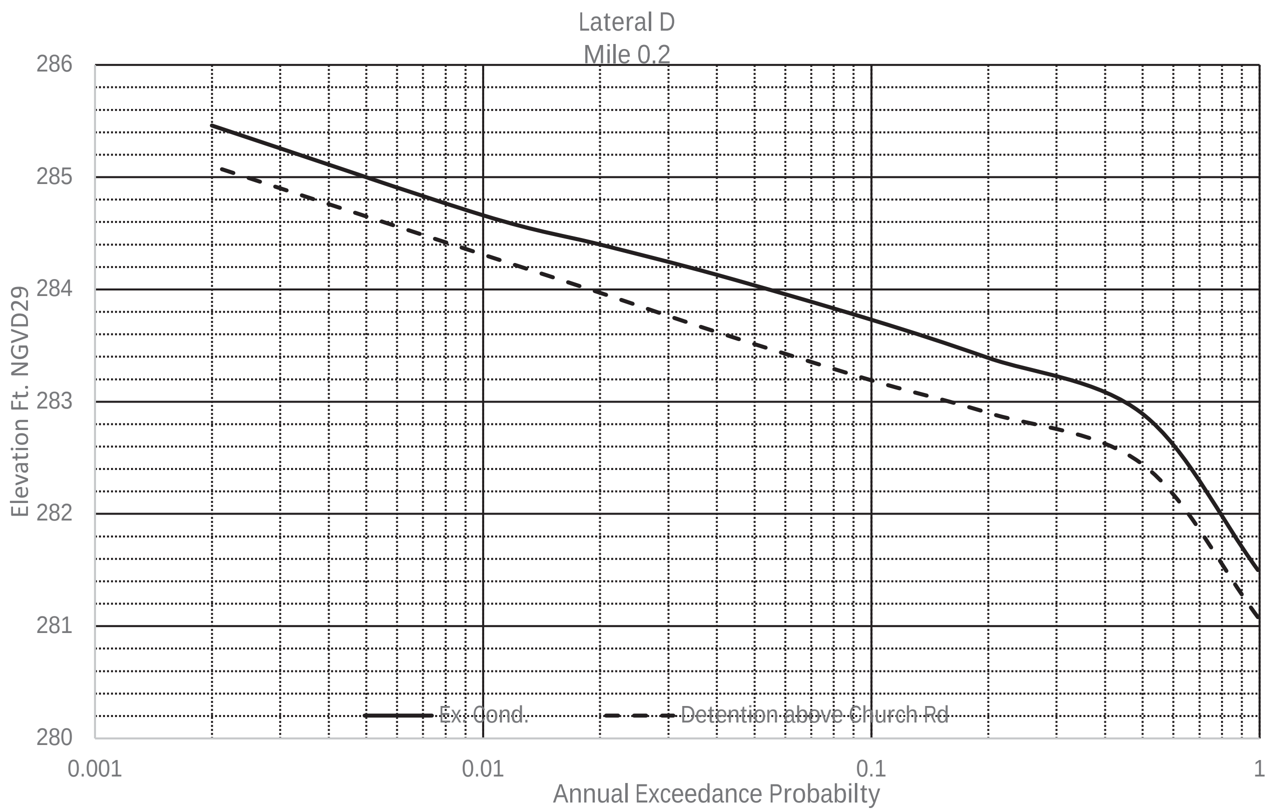

Figure 47-Water Surface Elev. Vs. Probability Curve

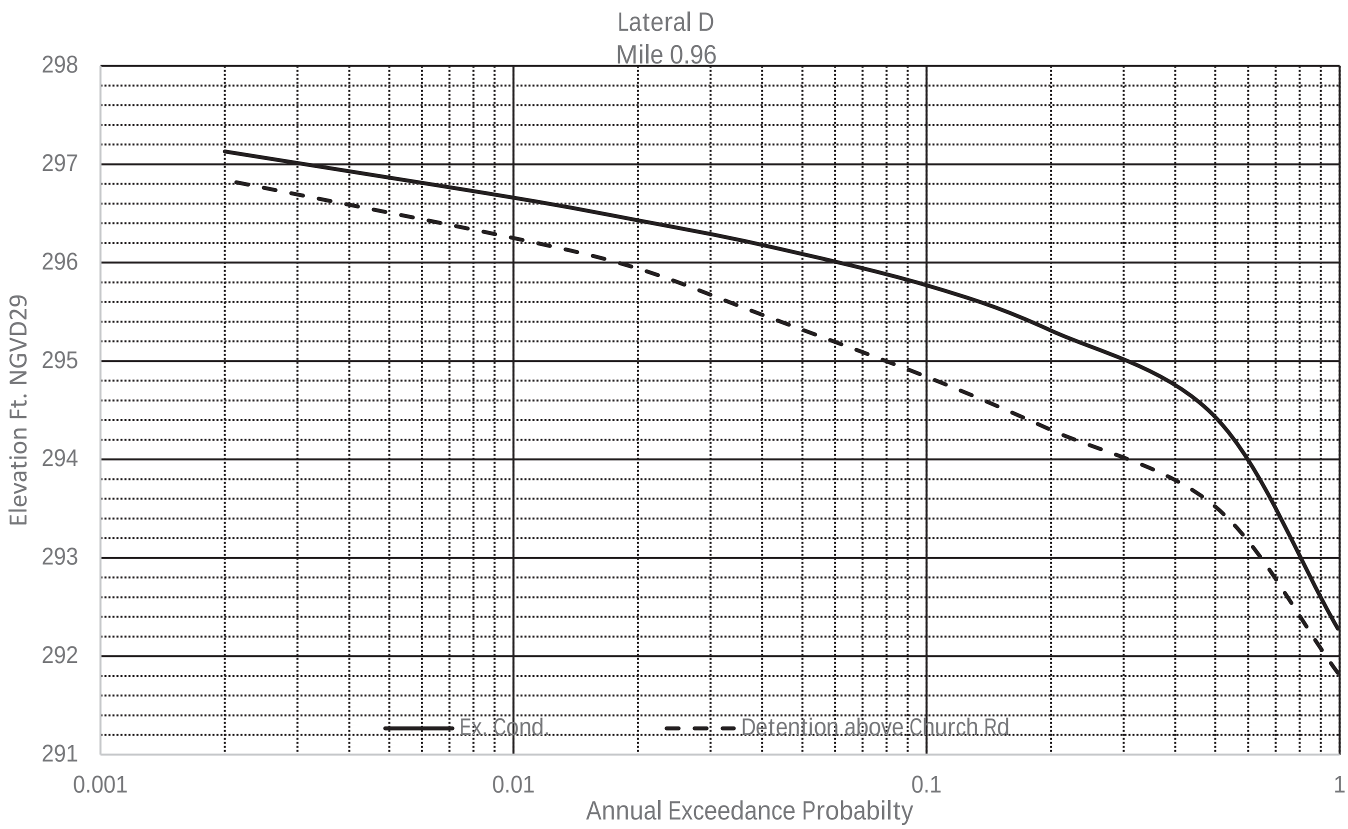

Figure 48- Water Surface Elev. Vs. Probability Curve

14 Frequency vs. Elevation Curves Tentatively Selected Plan (Original TSP)

The 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 TSP is not the National Economic Development (NED) Plan. The non-federal sponsor has 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 50 to 52.

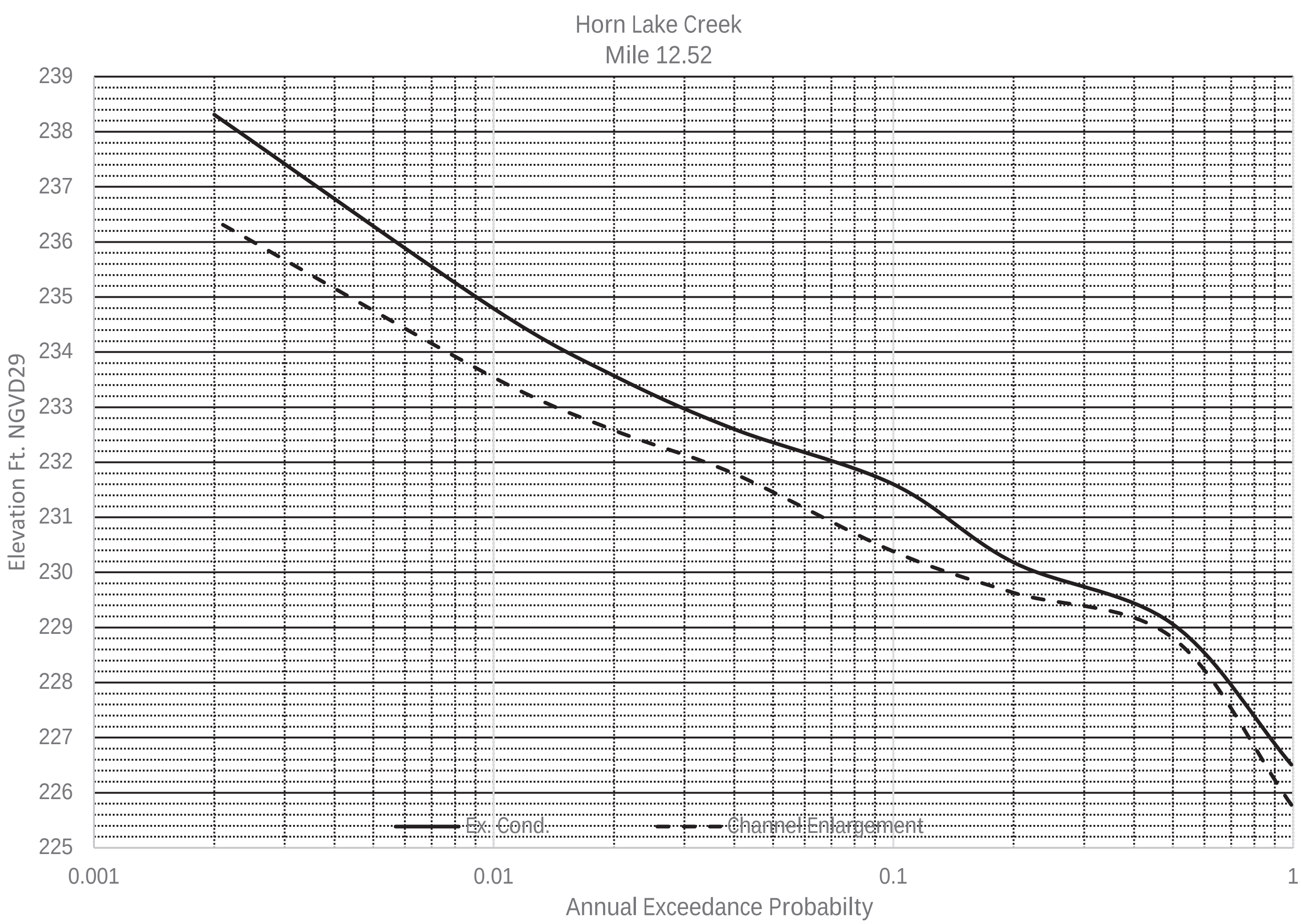

Figure 50. Water Surface Elev. Vs. Probability Curve

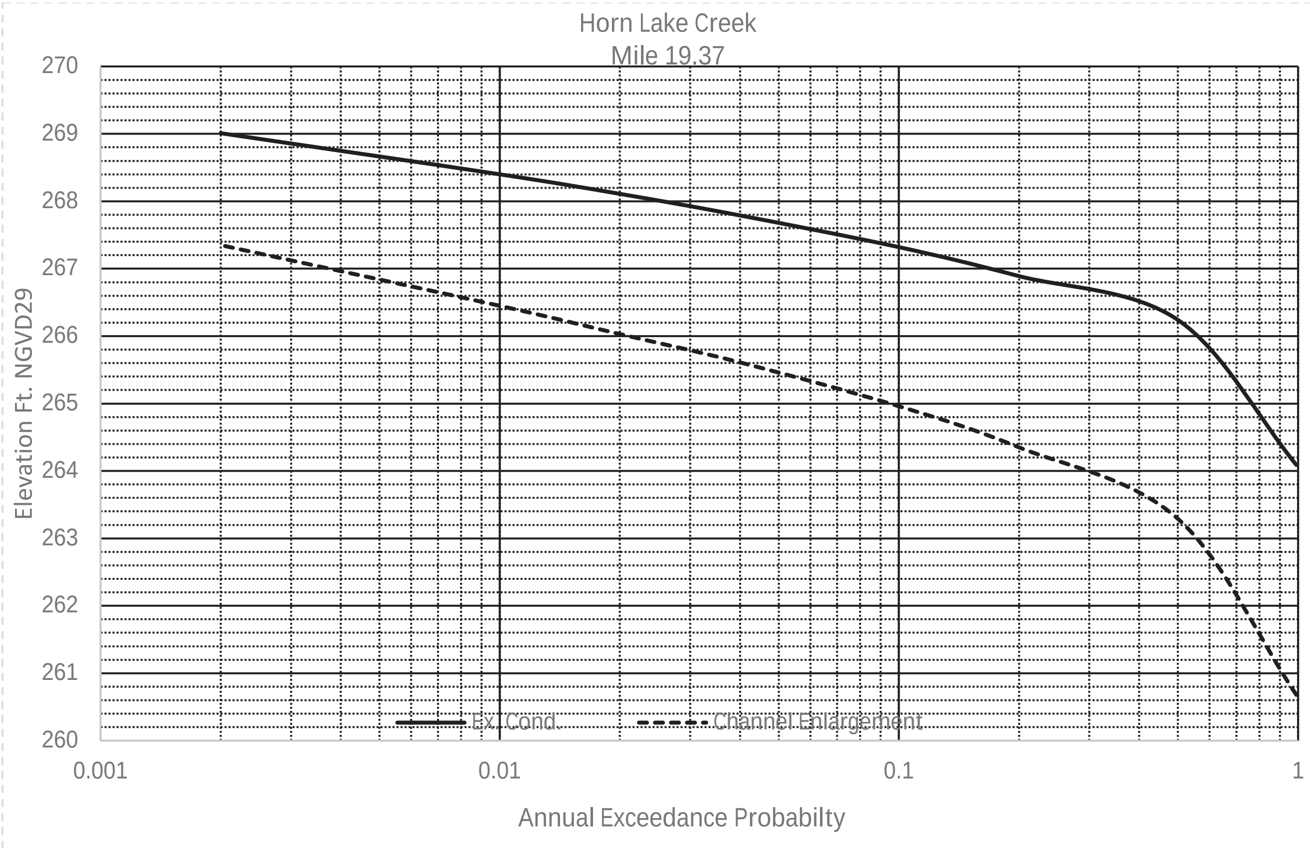

Figure 51. Water Surface Elev. Vs. Probability Curve

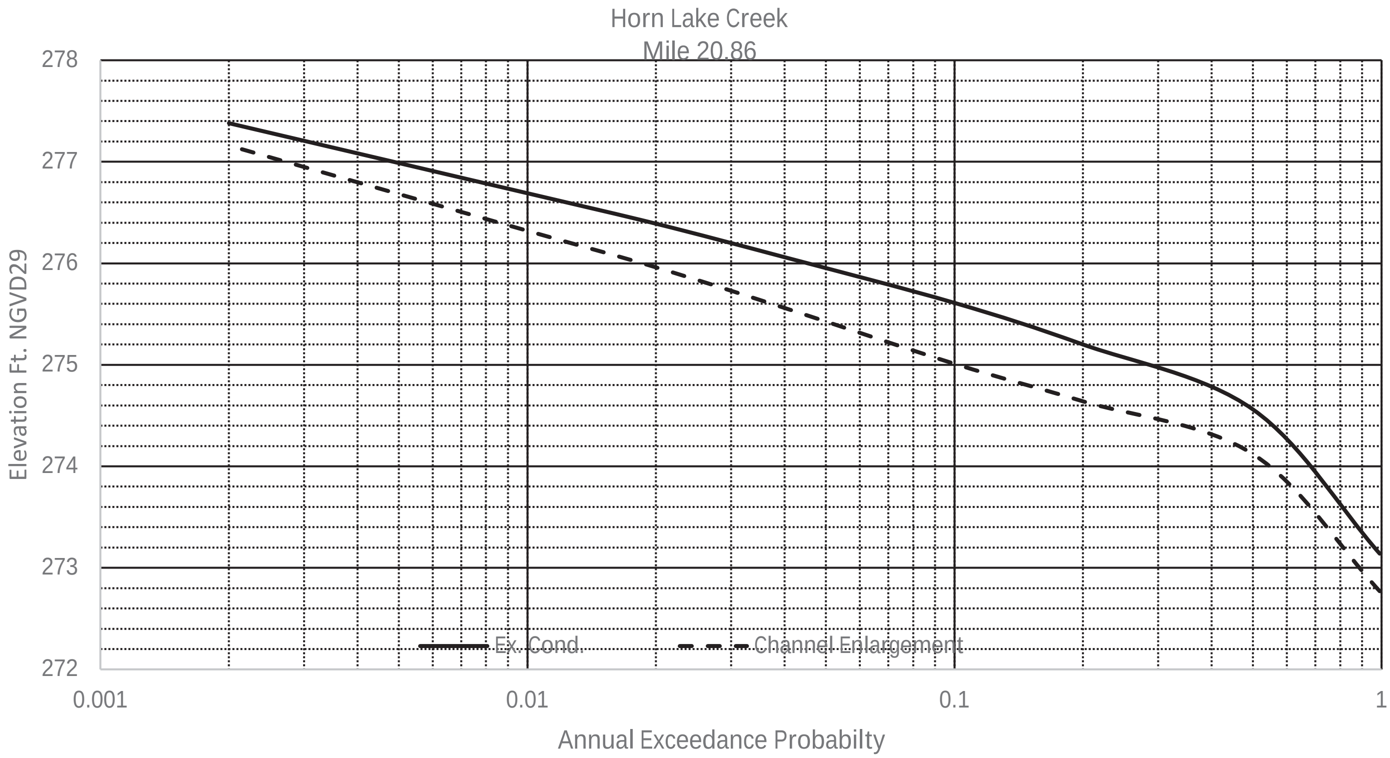

Figure 52. Water Surface Elev. Vs. Probability Curve

15 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 major damage area.

Other study requirements were identified that promoted the used 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 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.

15.1 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 53-59 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.

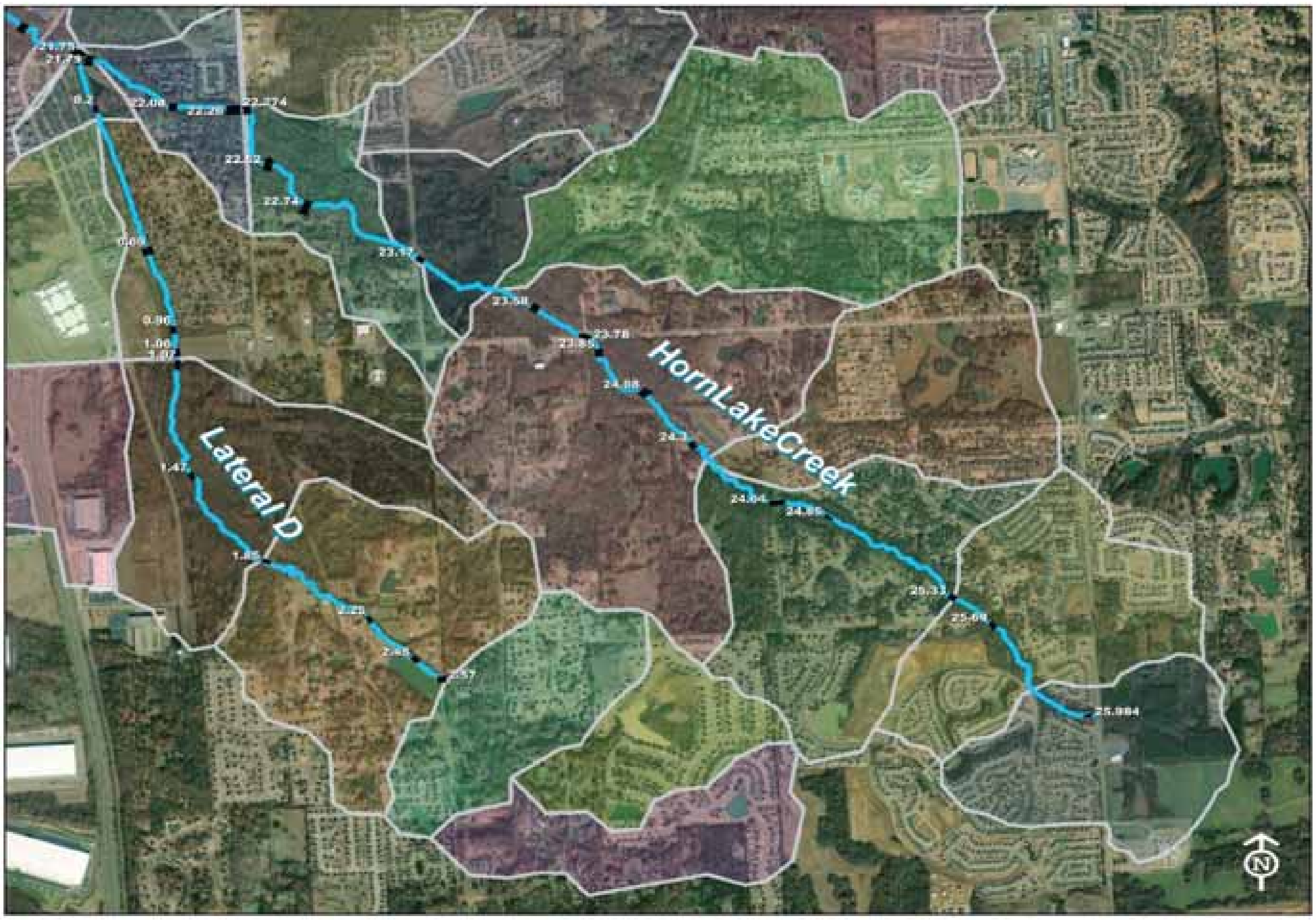

Figure 53. Horn Lake Creek (Right) and Lateral D (Left)
UPSTREAM BOUNDARY LOCATIONS

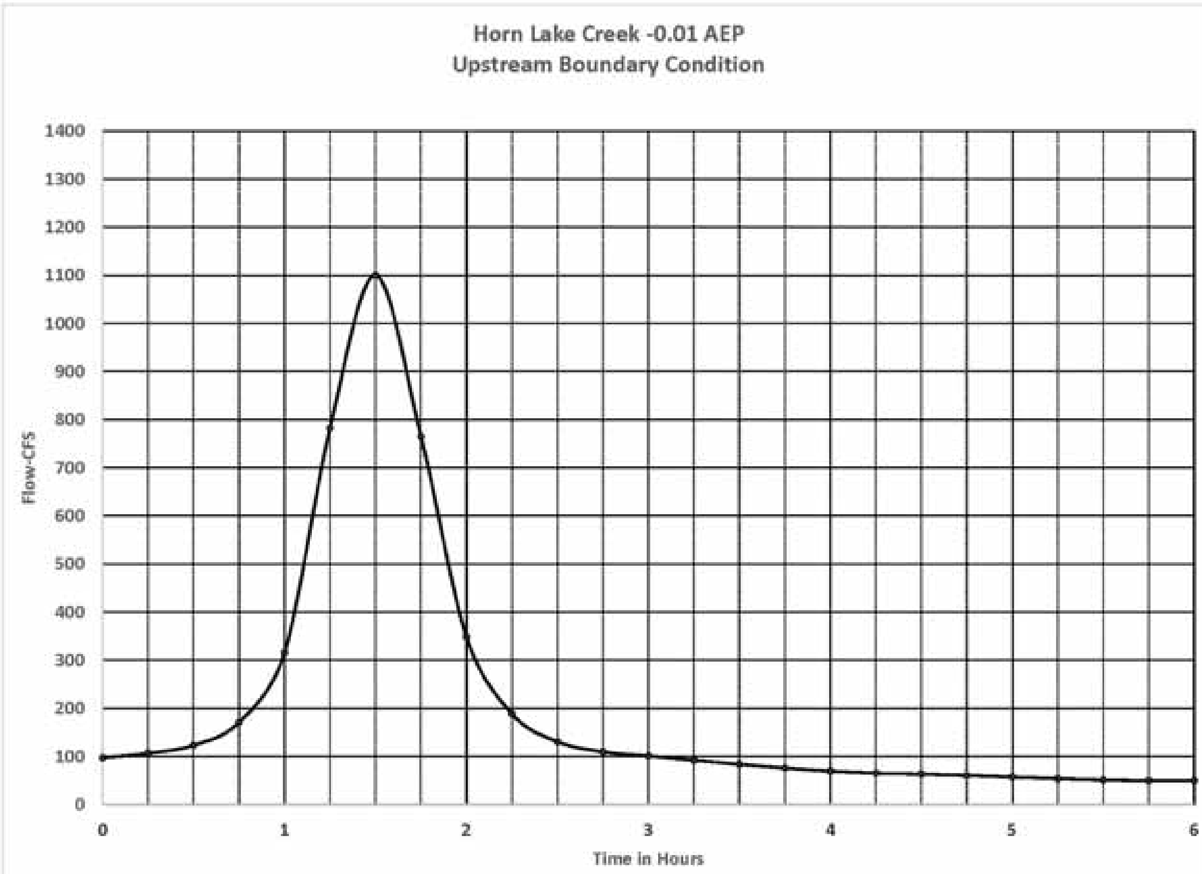

Figure 54. Horn Lake Creek and Lateral D 0.1 AEP (100 Year) Inflow Hydrographs

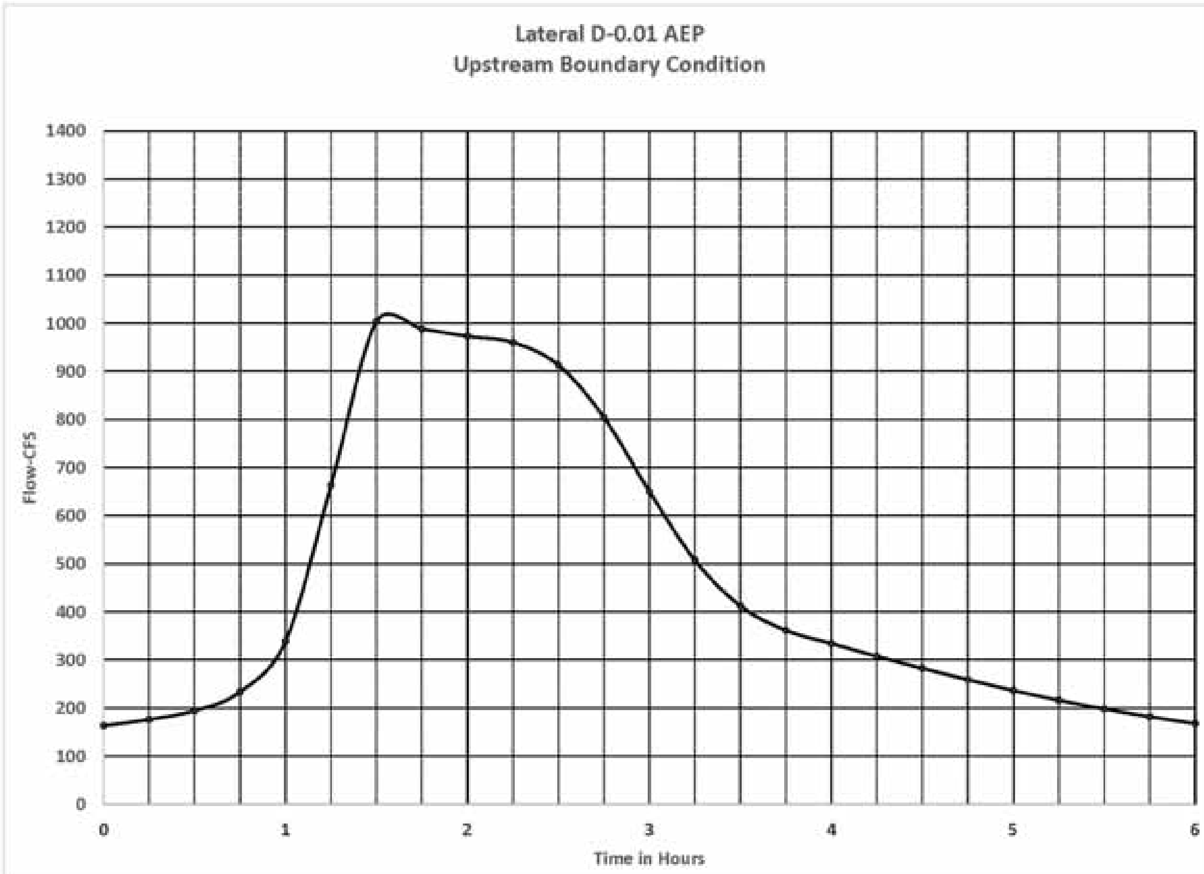

Figure 55. Horn Lake Creek and Lateral D 0.1 AEP (100 Year) Inflow Hydrographs

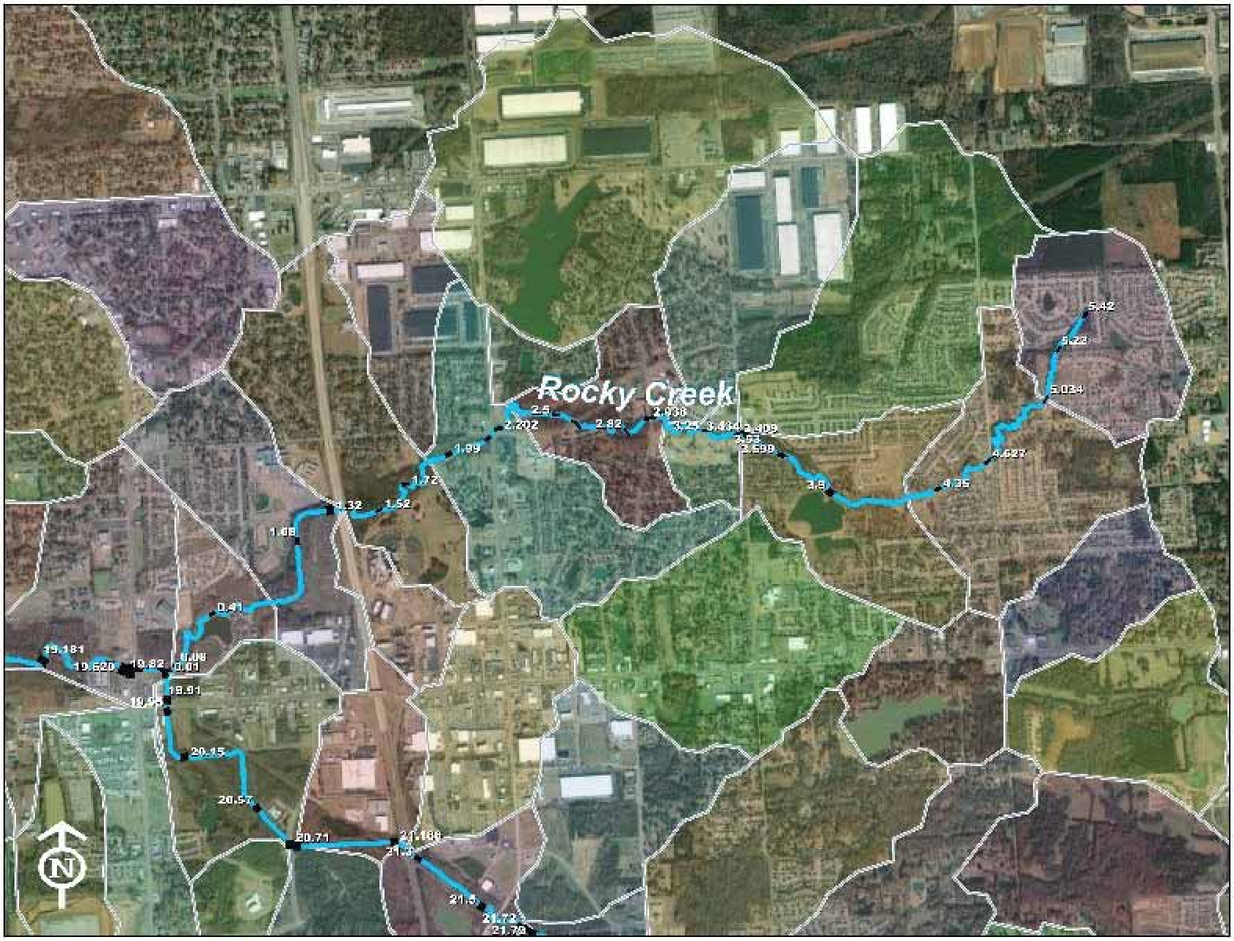

Figure 56. Rocky Creek Upstream Boundary Location and 0.1 AEP (100 Year) Inflow Hydrograph

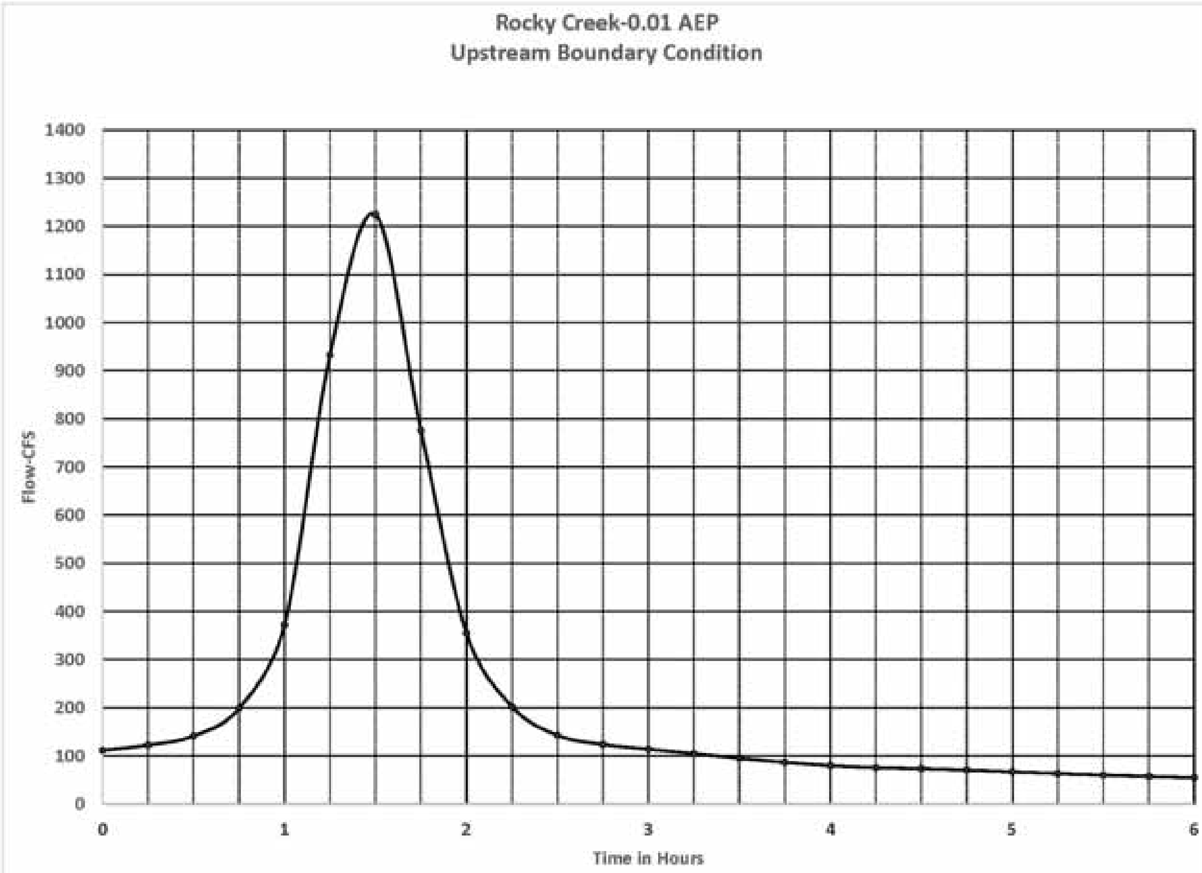

Figure 57. Rocky Creek Upstream Boundary Location and 0.1 AEP (100 Year) Inflow Hydrograph

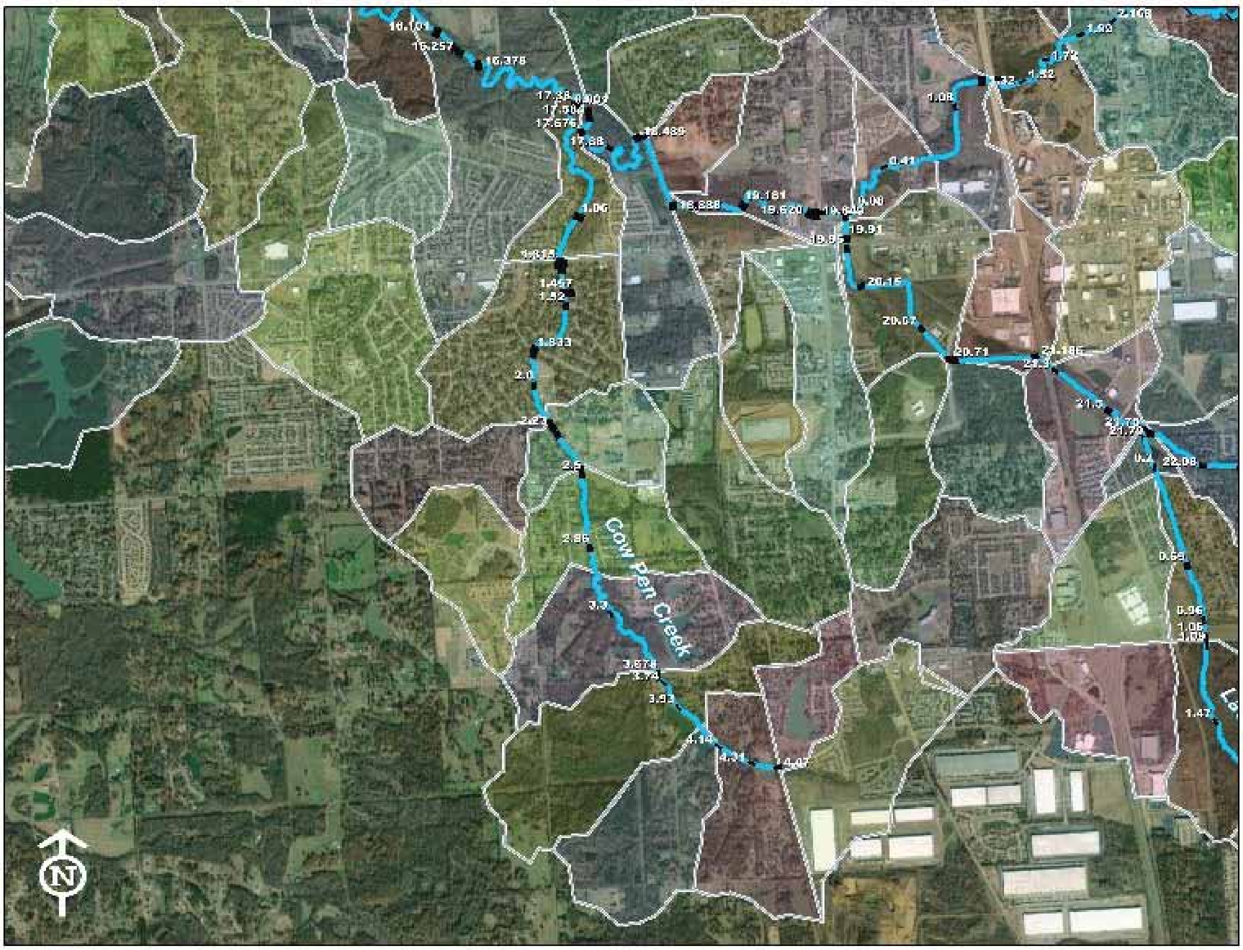

Figure 58. Cow Pen Creek Upstream Boundary Location and 0.1 AEP (100 Year) Inflow Hydrograph

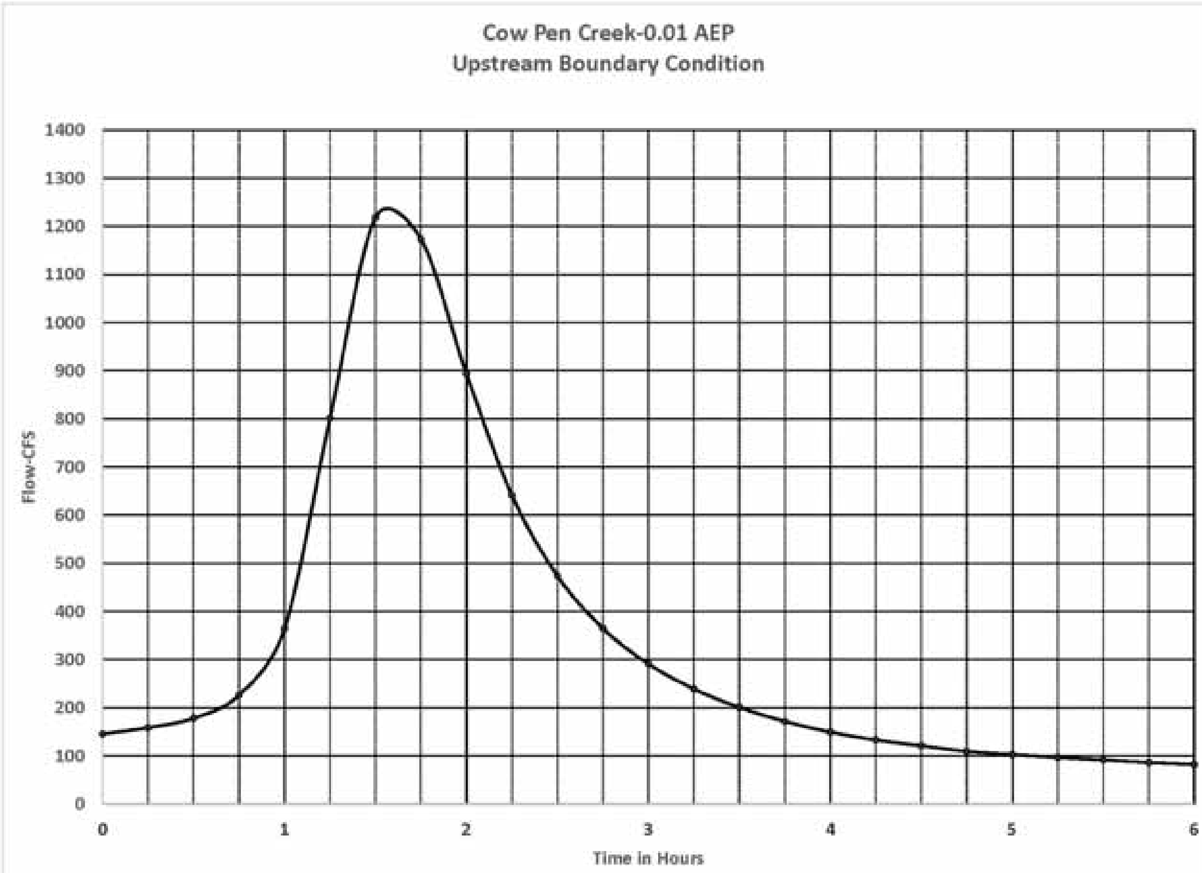

Figure 59. Cow Pen Creek Upstream Boundary Location and 0.1 AEP (100 Year) Inflow

15.1.1 Hydrograph 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.

16 Existing Conditions-HEC-RAS 1D/2D

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.

OTHER STUDY REQUIREMENTS WERE IDENTIFIED THAT PROMOTED THE USED 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 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 VERY WELL.

THE 2002 CHANNEL SECTIONS FOR THE 1D REACHES 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 ABOVE, BRIDGE DATA WAS IMPORTED FROM THE 2005 HEC-RAS 1D STEADY FLOW MODEL. SEVERAL MINOR ADJUSTMENTS ARE 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 16.3. WITH AN ELEVATED INTEREST IN FLOW CONDITIONS AT THE RAILROAD, MORE ACCURATE MODELING OF THE PILE CONFIGURATION, ALIGNMENT IMPACTS, AND OVERFLOW AREAS WERE NECESSARY.

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 60 SHOWS THE GENERAL LAYOUT OF THE 1D/2D GEOMETRIC MODEL.

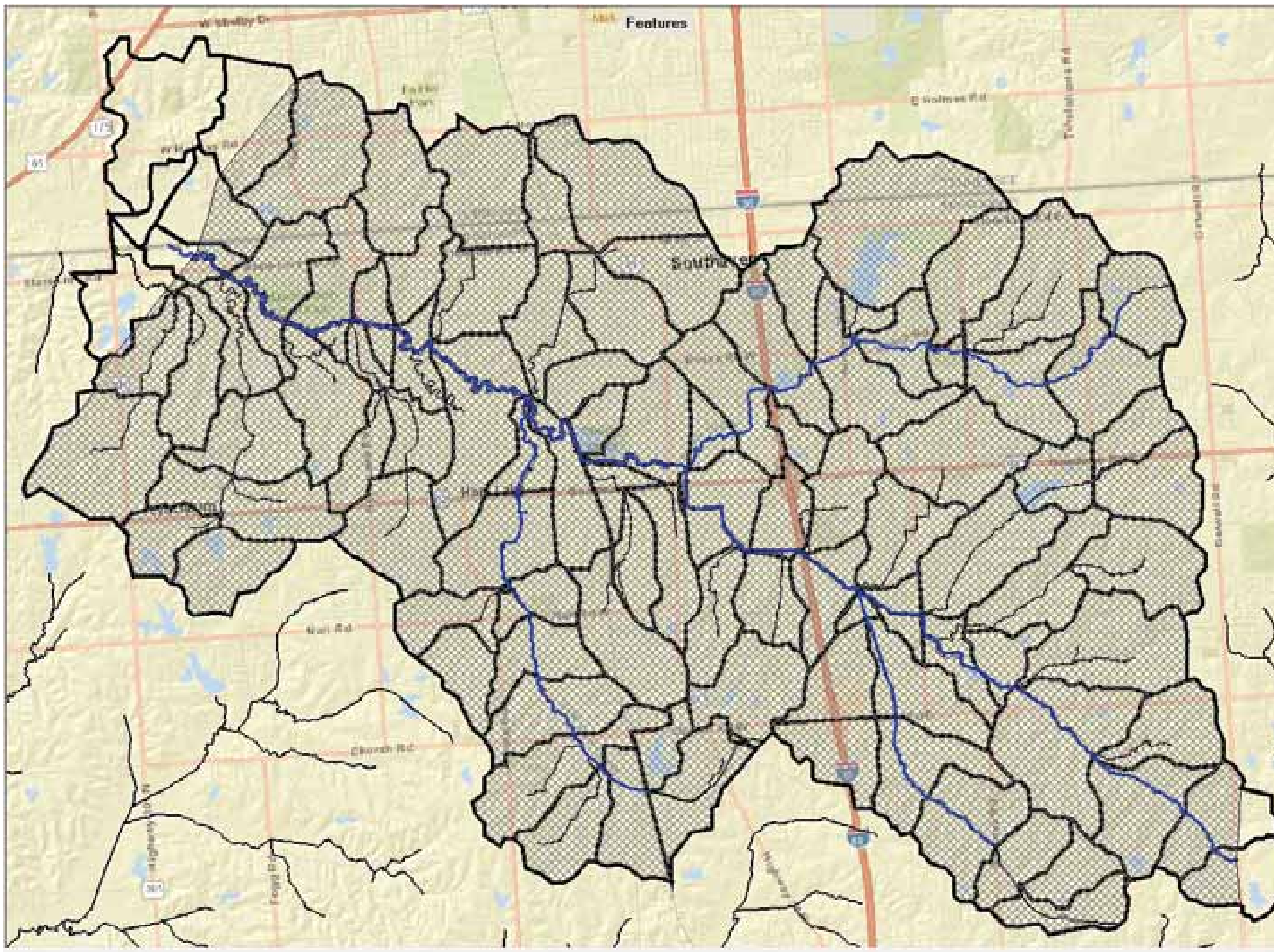

Figure 60. HEC-RAS 1D and 2D Geometric Characteristics

16.1Terrain 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.

16.2 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 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

Table 5. Manning's Roughness Summary

16.3 Bridge and Roadway Crossings

THE DECISION TO CONSTRUCT A 1D/2D MODEL AS OPPOSED TO A COMPLETE 2D MODEL WAS BIASED BY THE 1D HEC-RAS 1D CAPABILITIES IN RELATIONSHIP TO BRIDGE ANALYSIS. SINCE THE CURRENT 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 OPTION 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 5.07, 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.

17 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. THE FOLLOWING FIGURES 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.

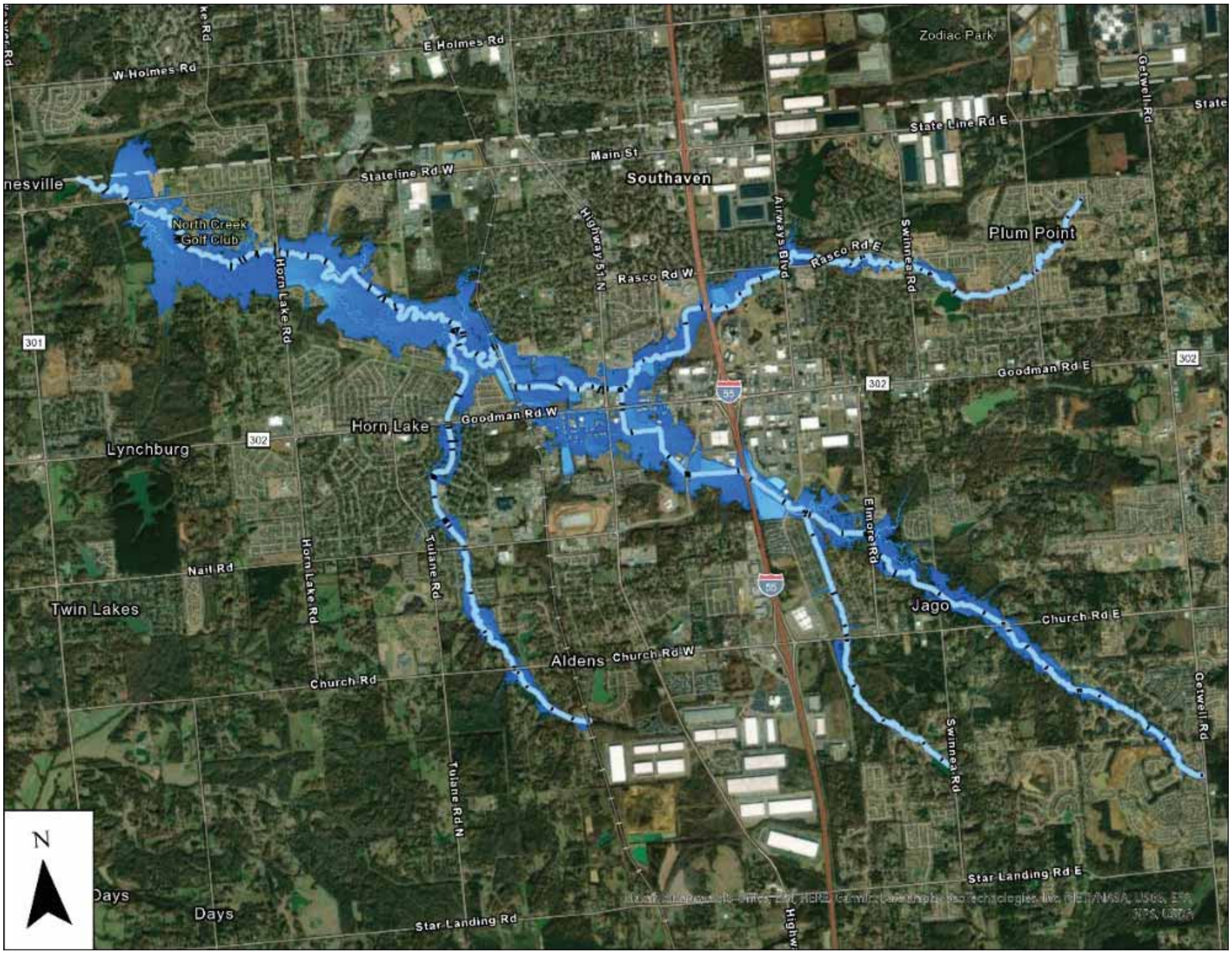

Figure 61. 0.10 AEP Inundation Map-Horn Lake Creek above Mississippi-Tennessee Stateline

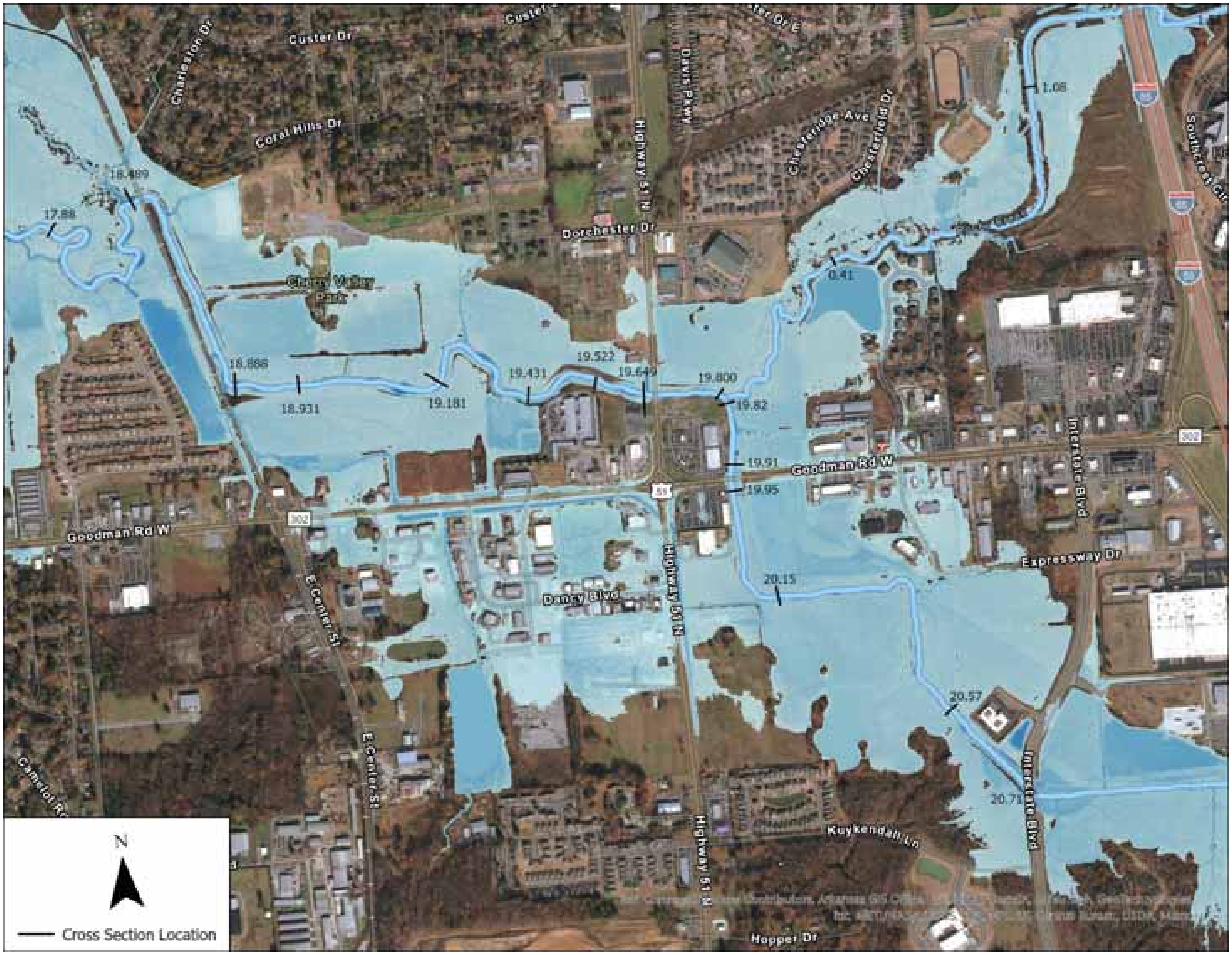

Figure 62. 0.10 AEP Inundation Map-Bullfrog Corner

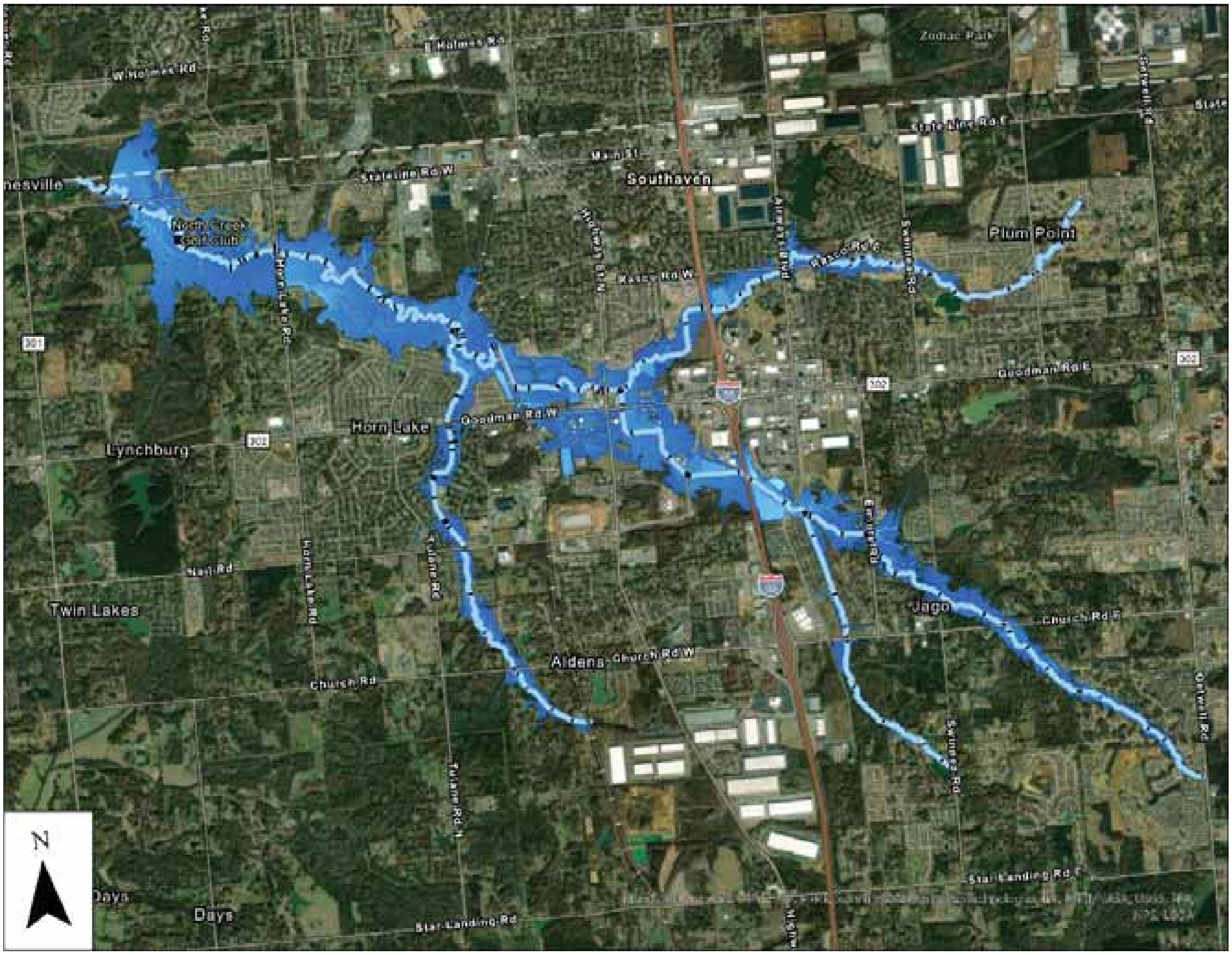

Figure 63. 0.02 AEP Inundation Map-Horn Lake Creek above Mississippi-Tennessee Stateline

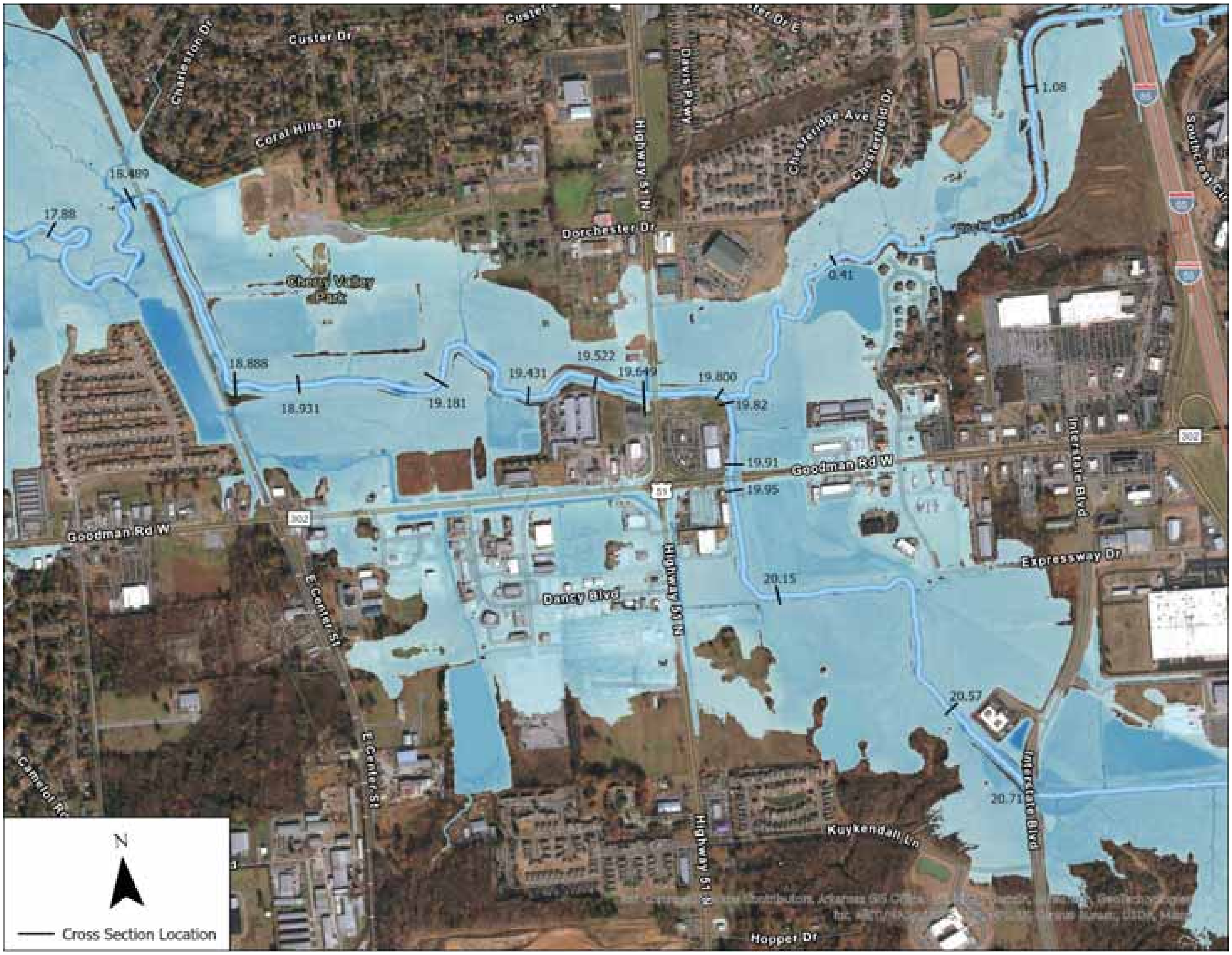

Figure 64. 0.02 AEP Inundation Map-Bullfrog Corner

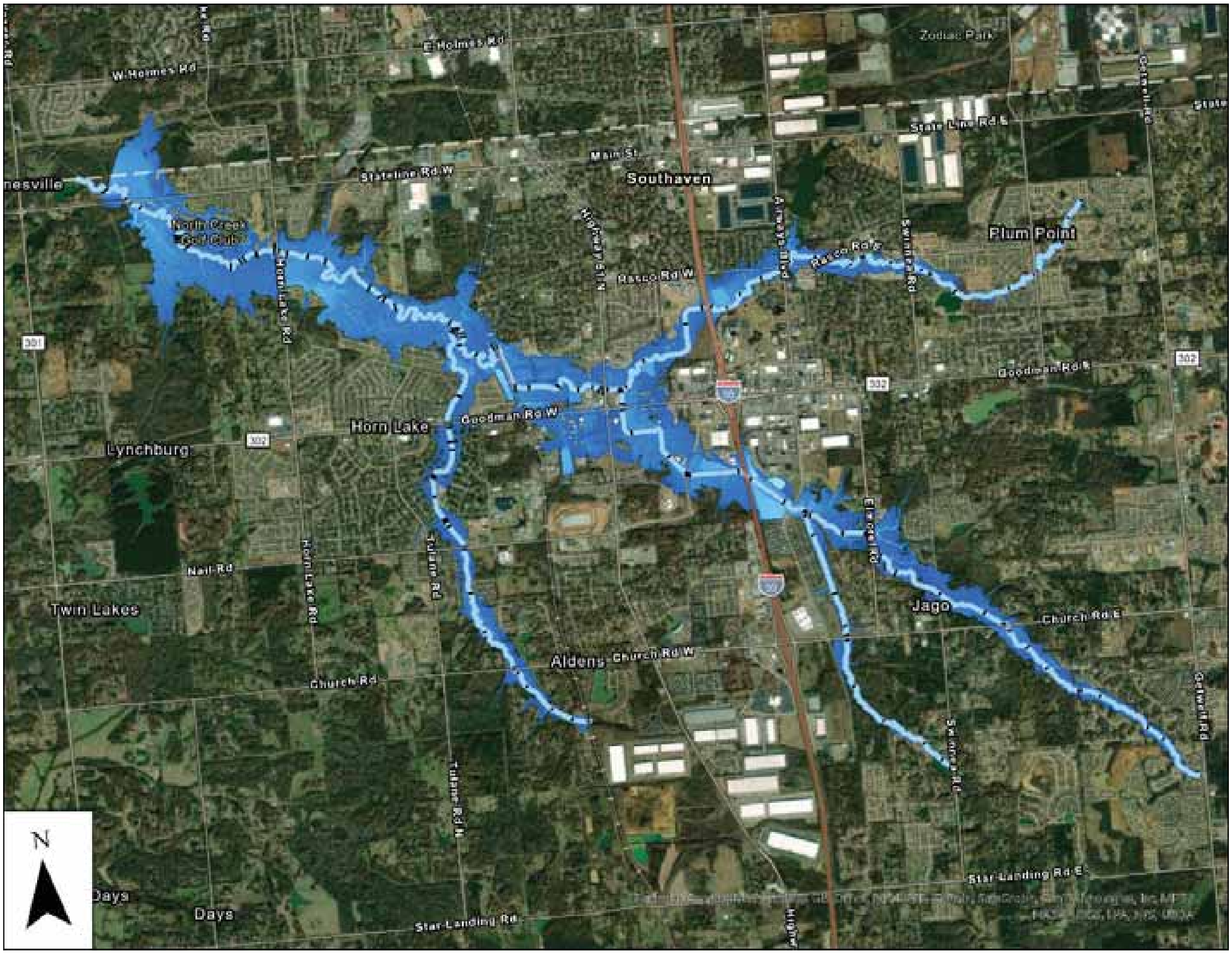

Figure 65. 0.01 AEP Inundation Map-Horn Lake Creek above Mississippi-Tennessee Stateline

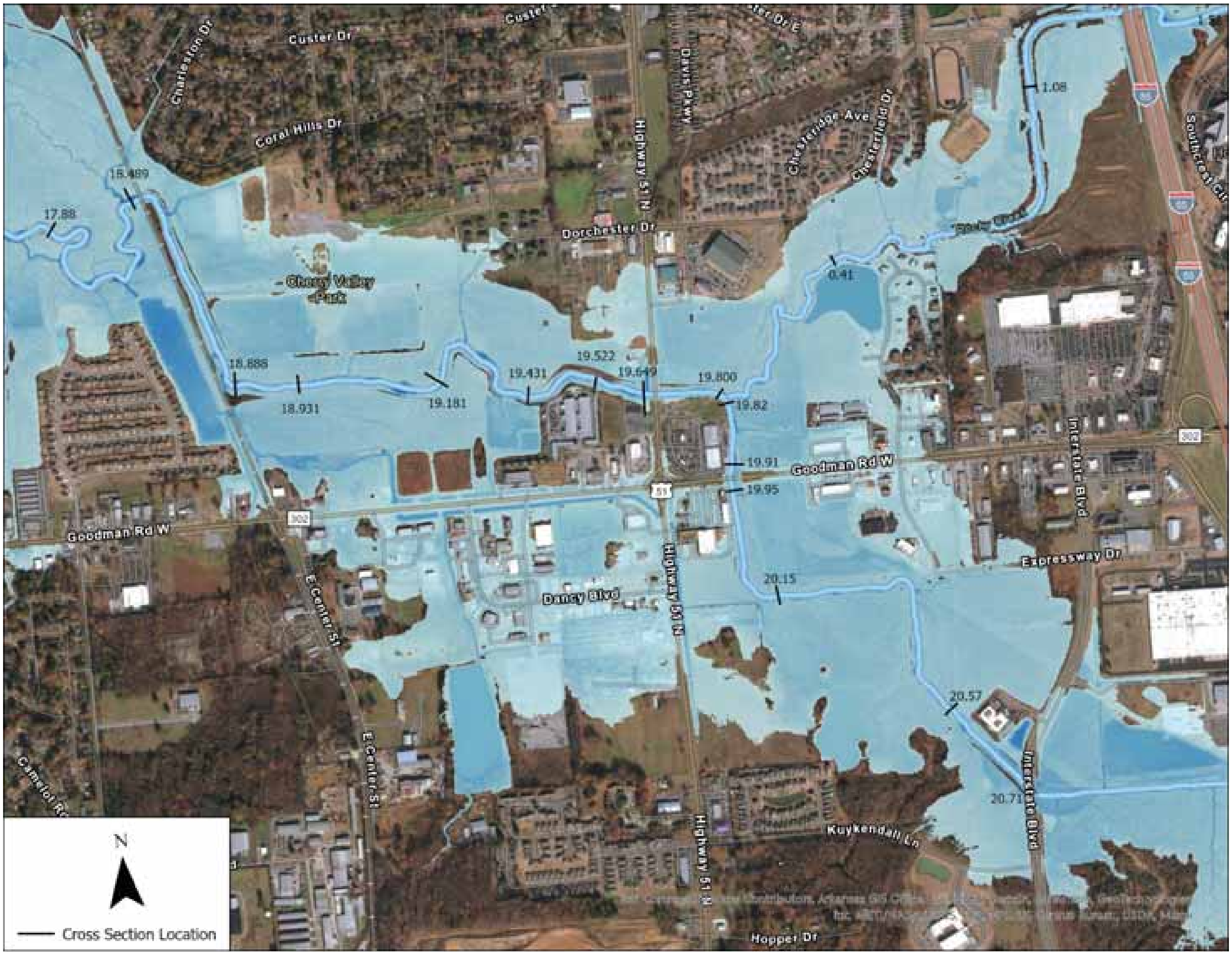

Figure 66. 0.01 AEP Inundation Map-Bullfrog Corner

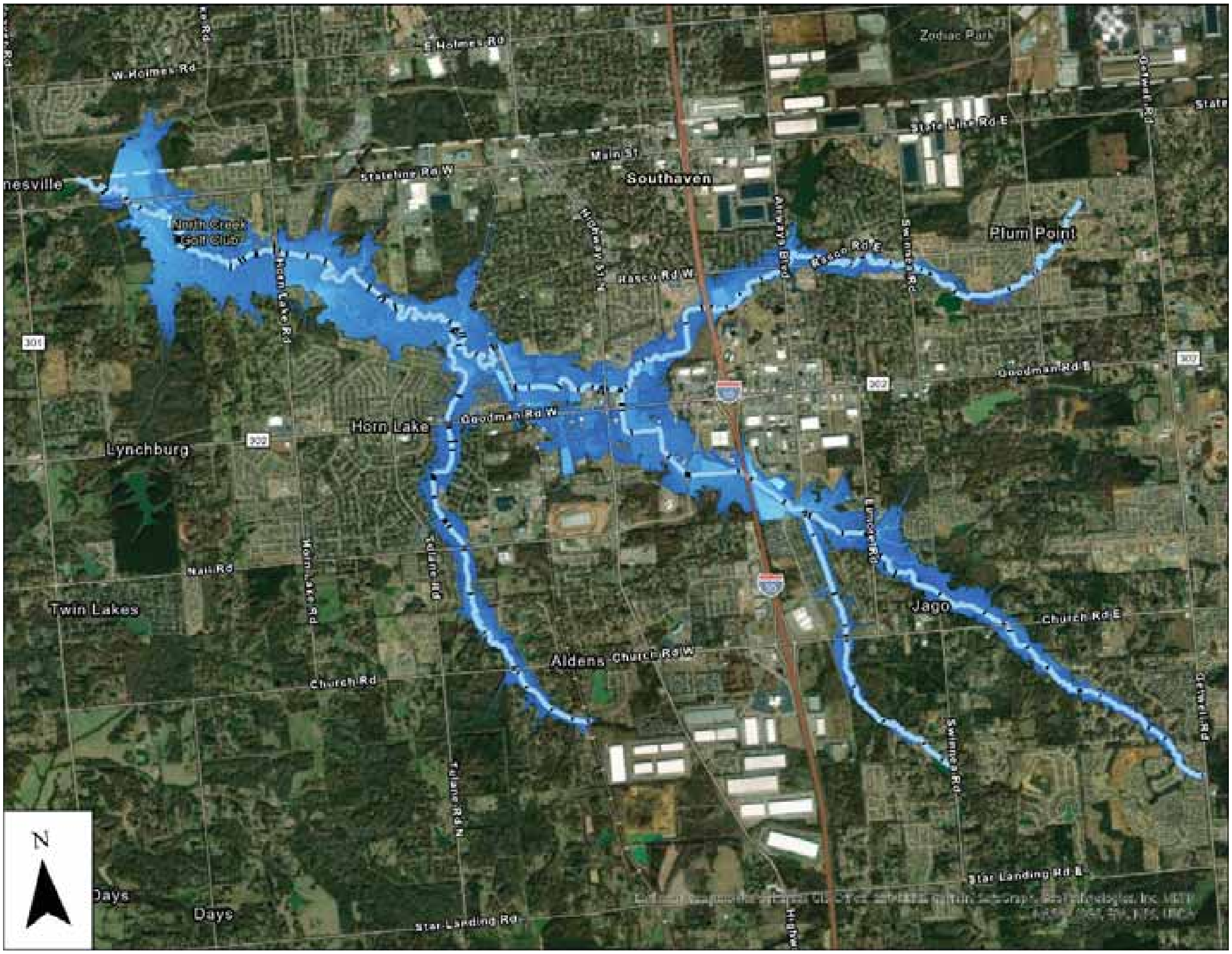

Figure 67. 0.002 AEP Inundation Map-Horn Lake Creek above Mississippi-Tennessee Stateline

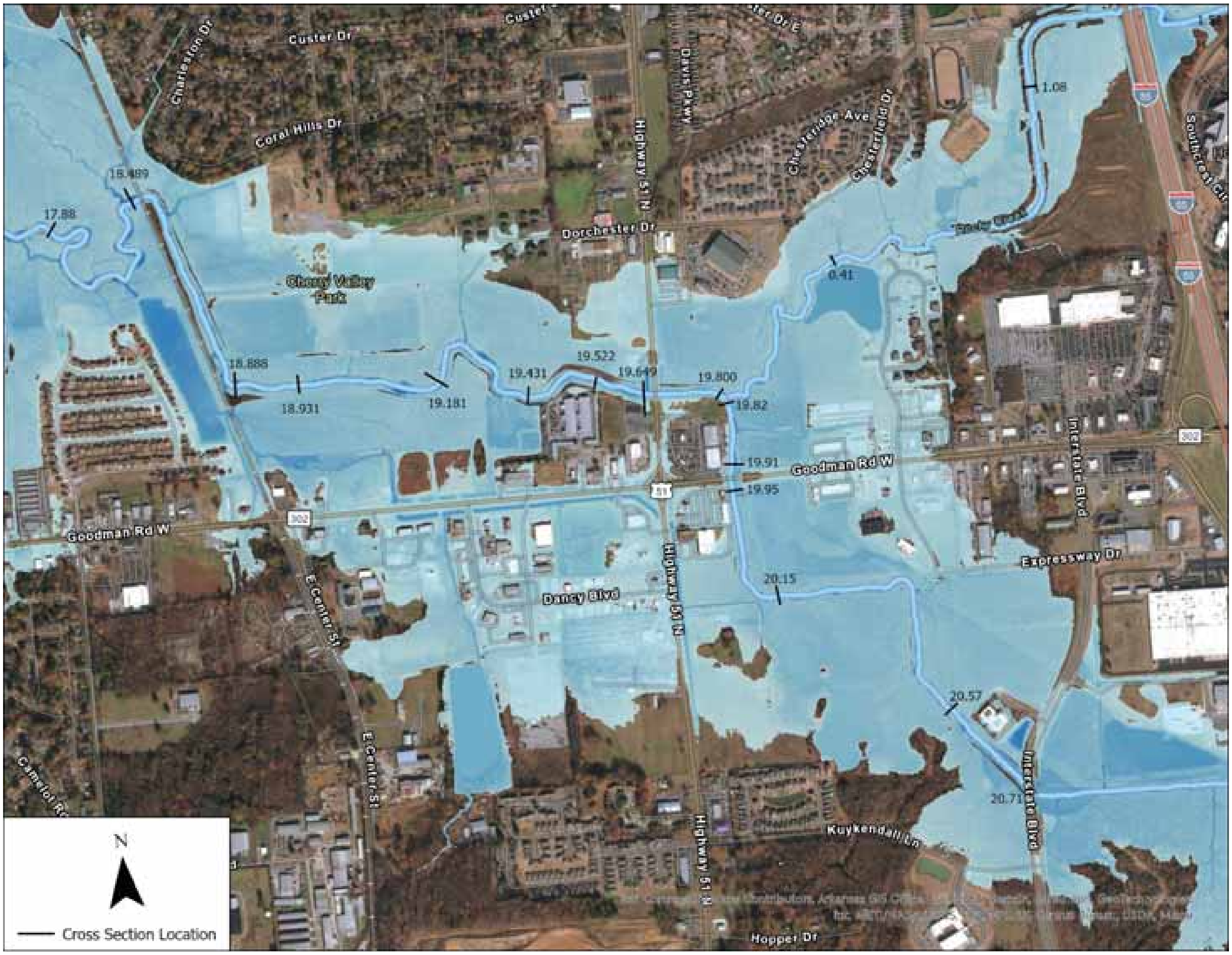

Figure 68. 0.002 AEP Inundation Depth Map-Bullfrog Corner

18 HEC-RAS 1D/2D Analysis-Alternatives

The Horn Lake Creek HEC-RAS 1D/2D model was modified to assess the channel enlargement measure. 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 69 shows the 100-foot 2D cells and overbank topography as developed in the terrain data.

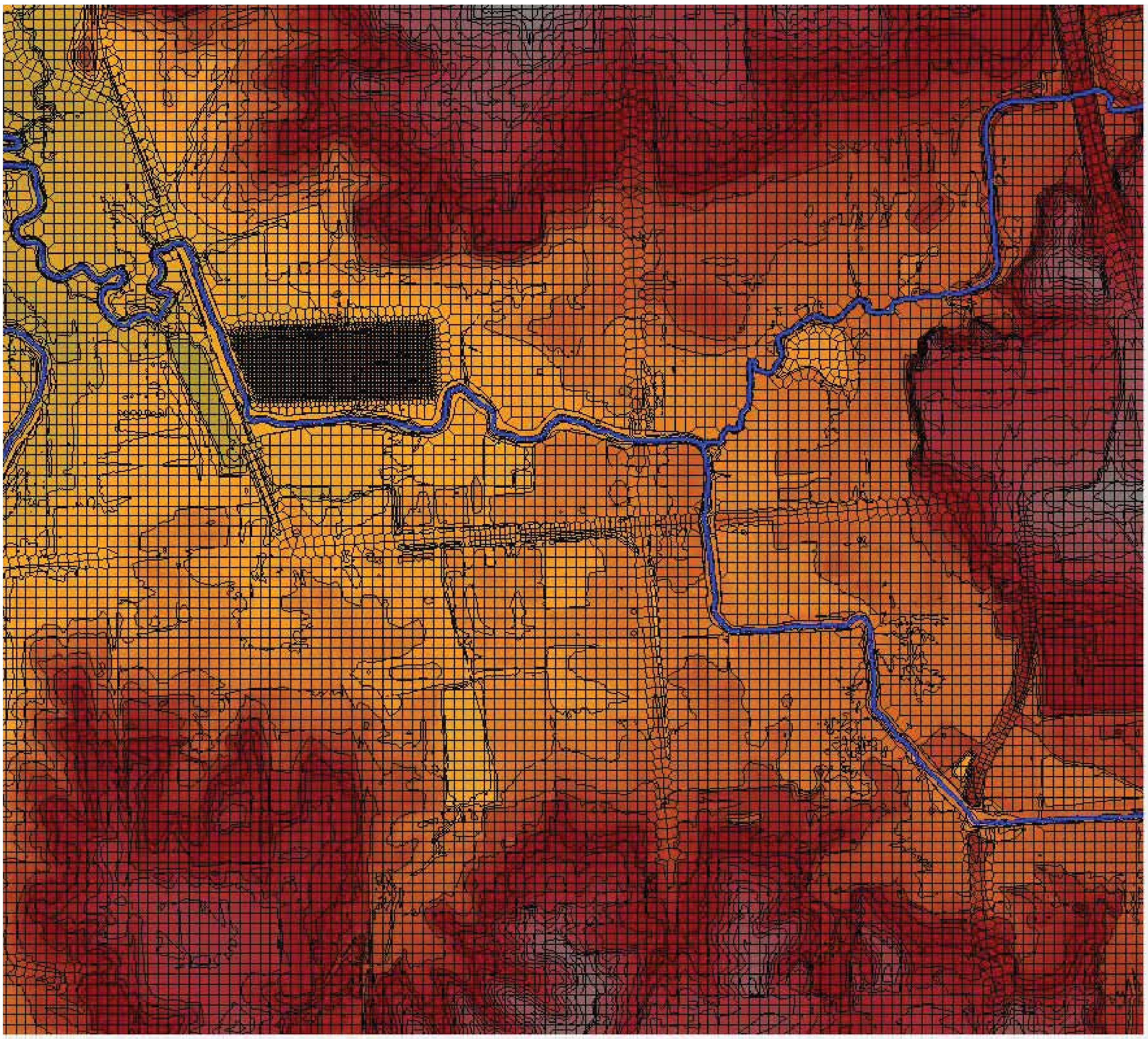

Figure 69. Horn Lake Creek 1D/2D Channel Enlargement Area

18.1 Cow Pen Creek Detention Ponds at Nail Road

The Terrain file was altered to simulate detention on the both the North and the South Detention ponds. This was accomplished using ArcMap and storage volumes used in the 1D analysis. Figure 70 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. 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 concrete diameter 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.

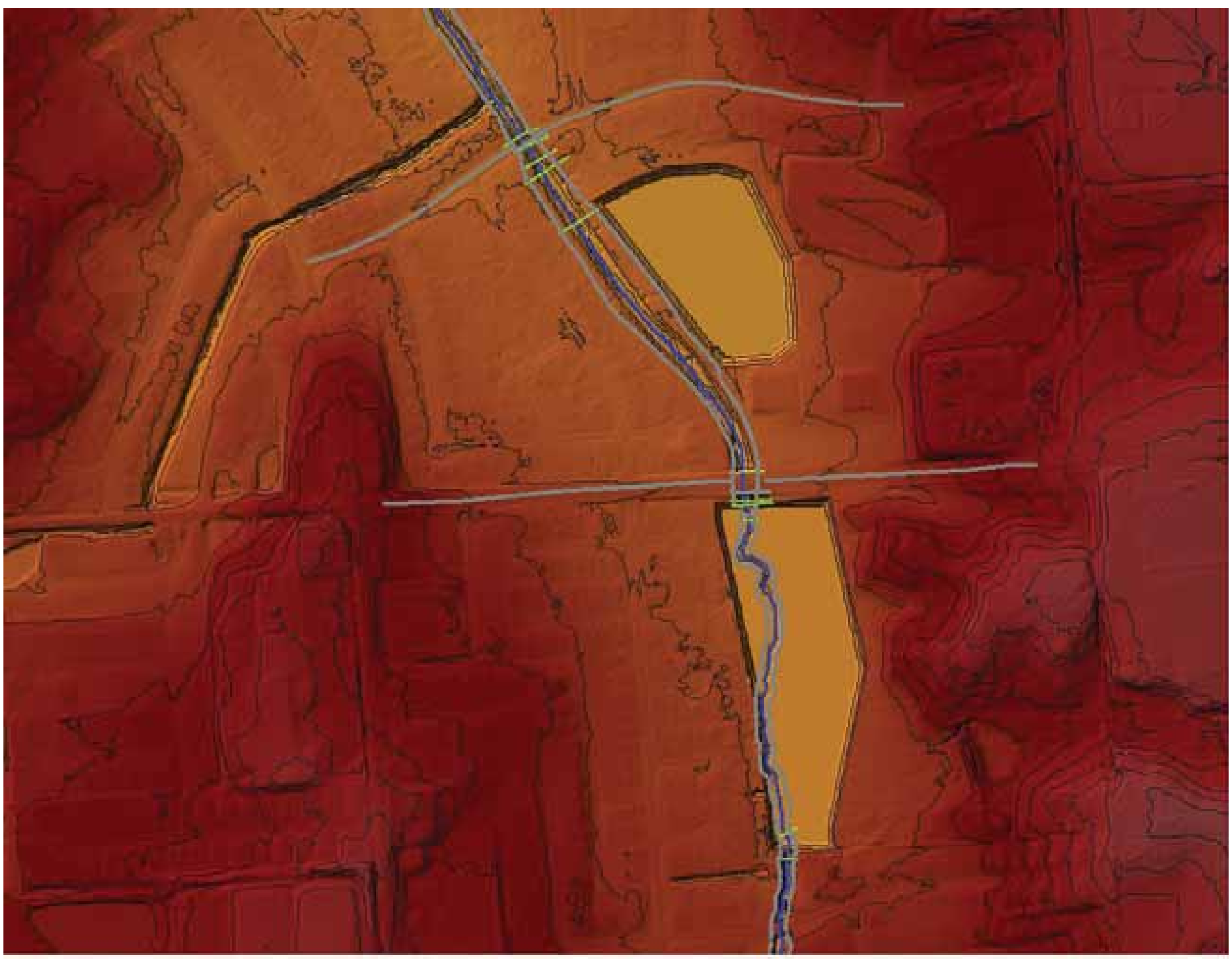

Figure 70. Cow Pen Creek Detention Pond Terrain Modification

18.2 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 71 shows the results of the terrain modifications.

Figure 71. 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.

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18.3 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 72.

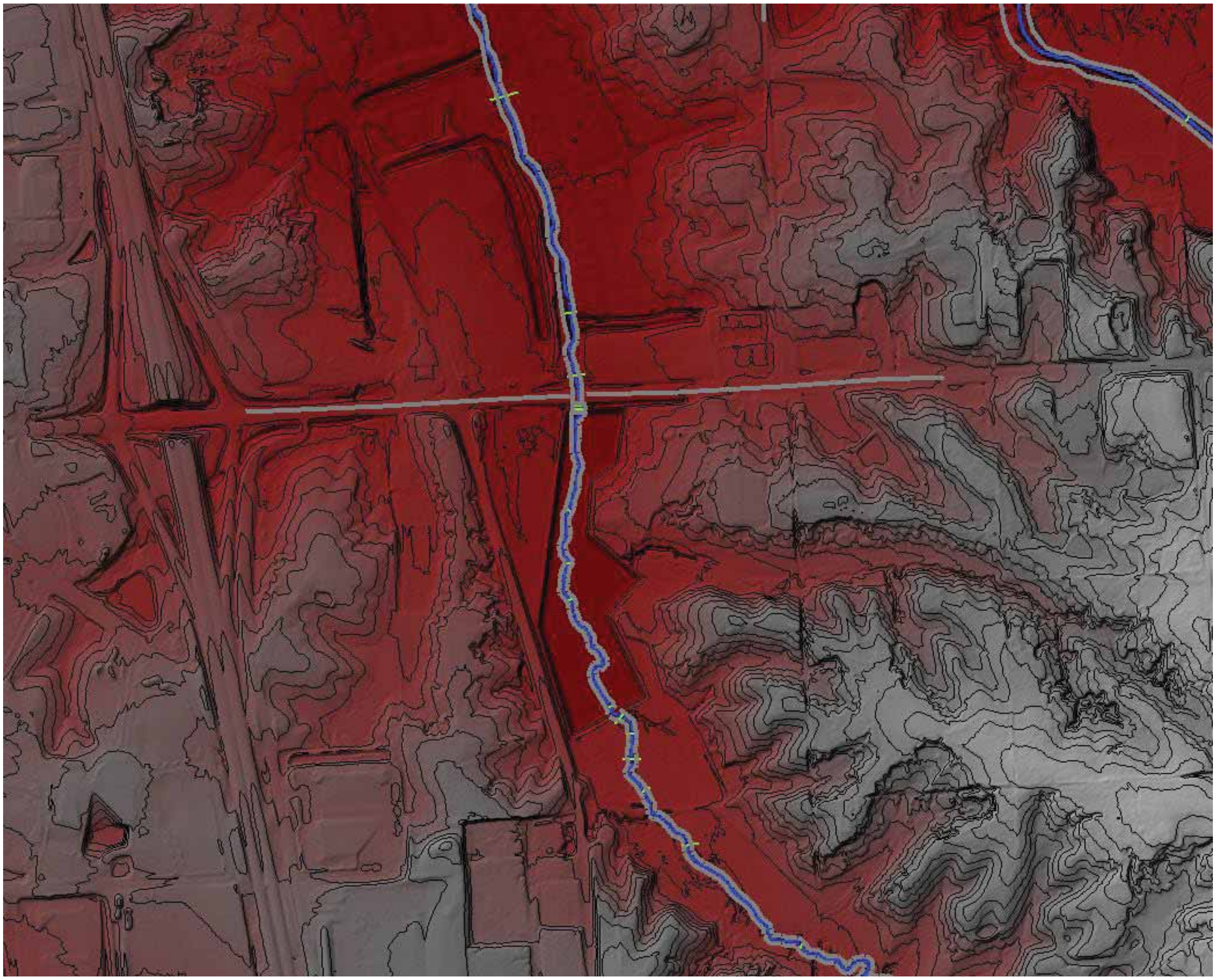

Figure 72. Lateral D Detention Pond Terrain Modification

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.

18.4 Horn Lake Creek Levee Alternative

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 foot 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 southwester quadrant of “Bullfrog Corner”. Construction of the combination levee/floodwall will reduce the frequency and magnitude of inundation in this flood prone area. The location of the levee is shown below.

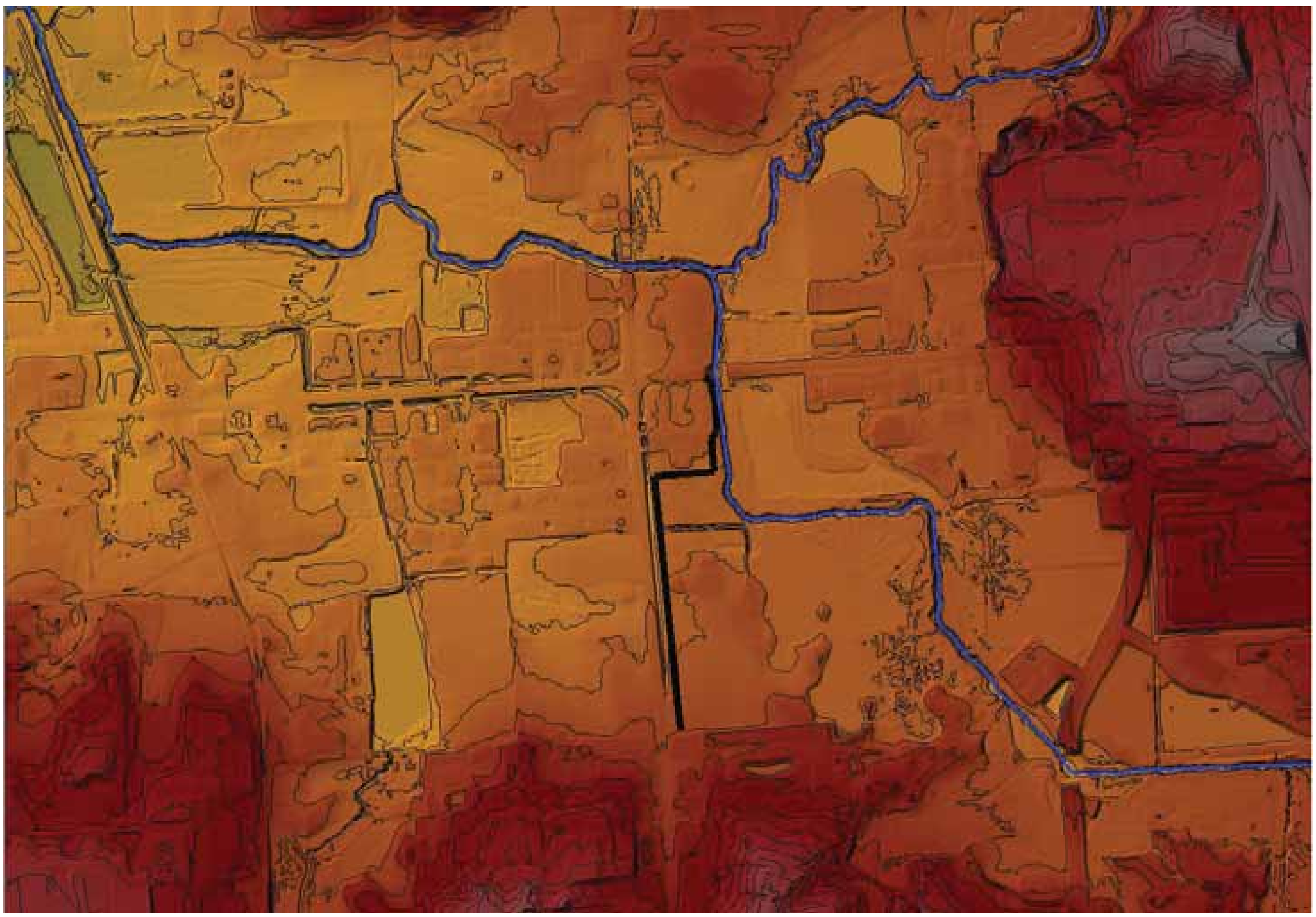

Figure 73. Levee Approximate Location

Cross sections of the floodwall and levee are shown below in Figures 74 and 75.

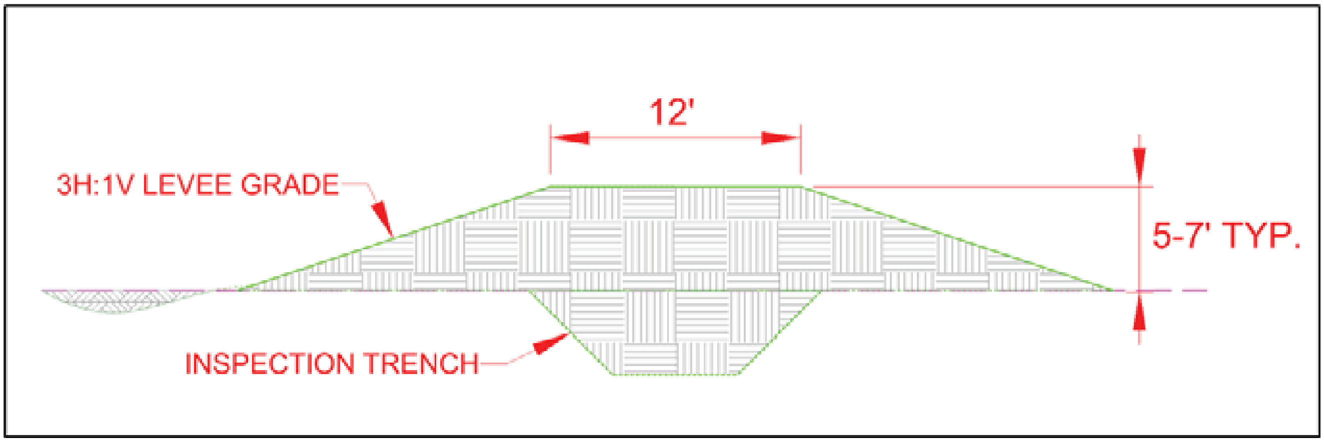

Figure 74. Levee Cross Section

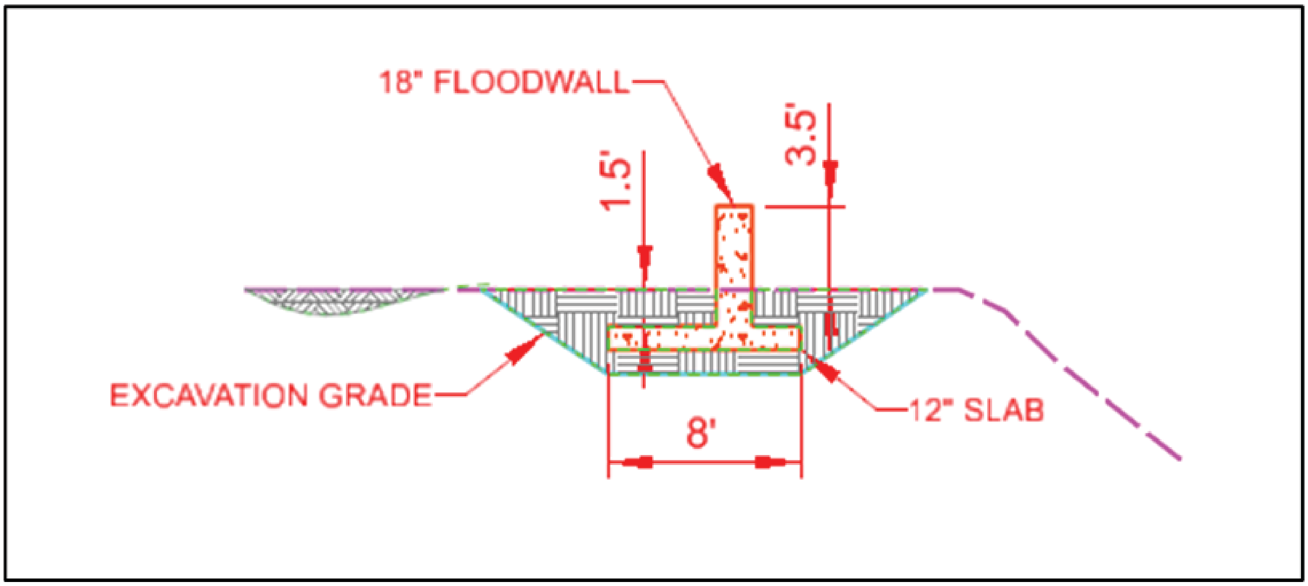

Figure 75. Floodwall Cross Section

19 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 the levee/floodwall. The final recommended plan (TSP) consisted of the 3,000 linear foot levee and floodwall feature combined with a nonstructural aggregation to address residual flooding.

Since the channel enlargement and detention storage are no longer incorporated in the TSP, 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.

Figure 76. 0.10 AEP Levee Alternative Inundation Depth Map-Bullfrog Corner

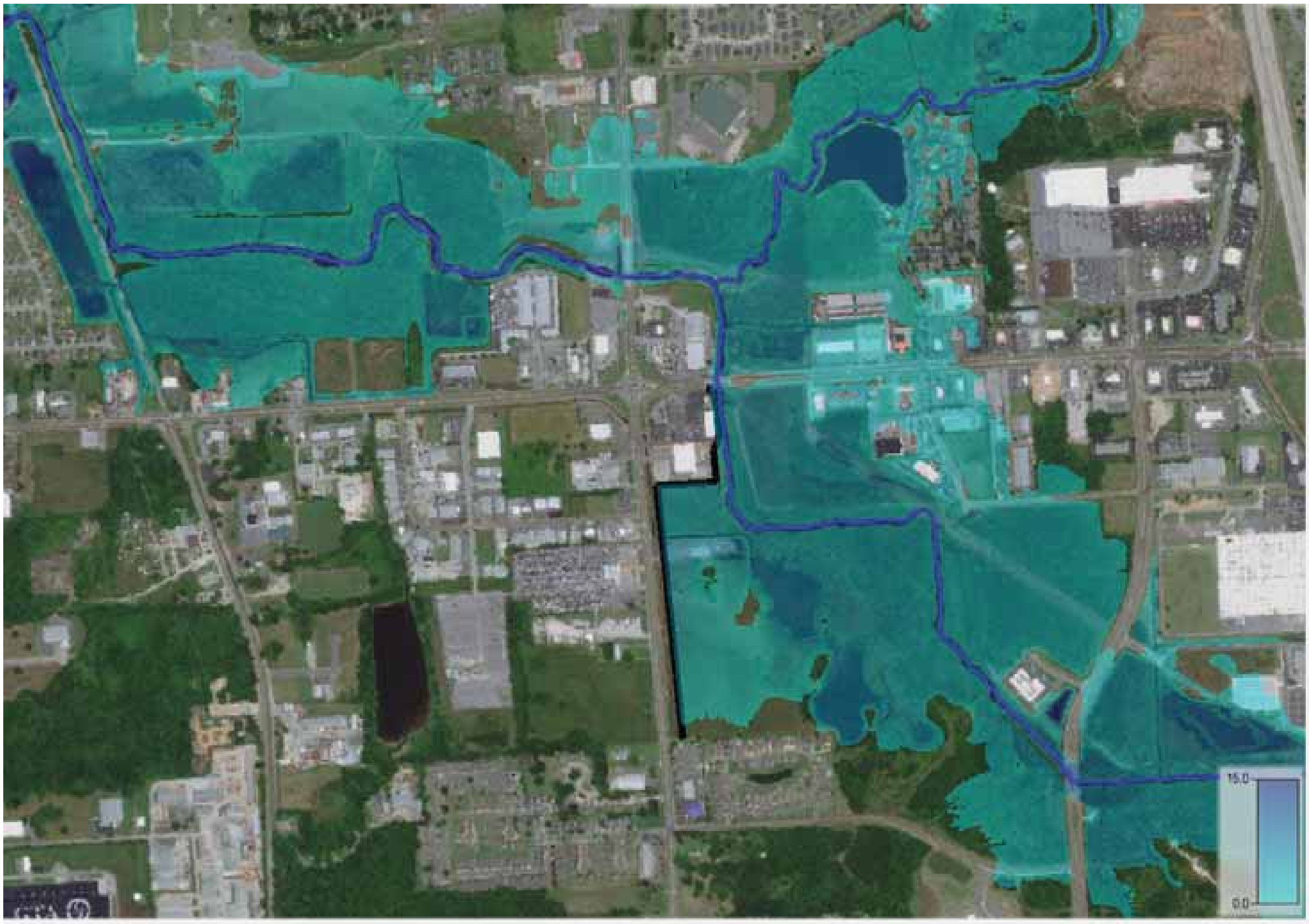

Figure 77. 0.02 AEP Levee Alternative Inundation Depth Map-Bullfrog Corner

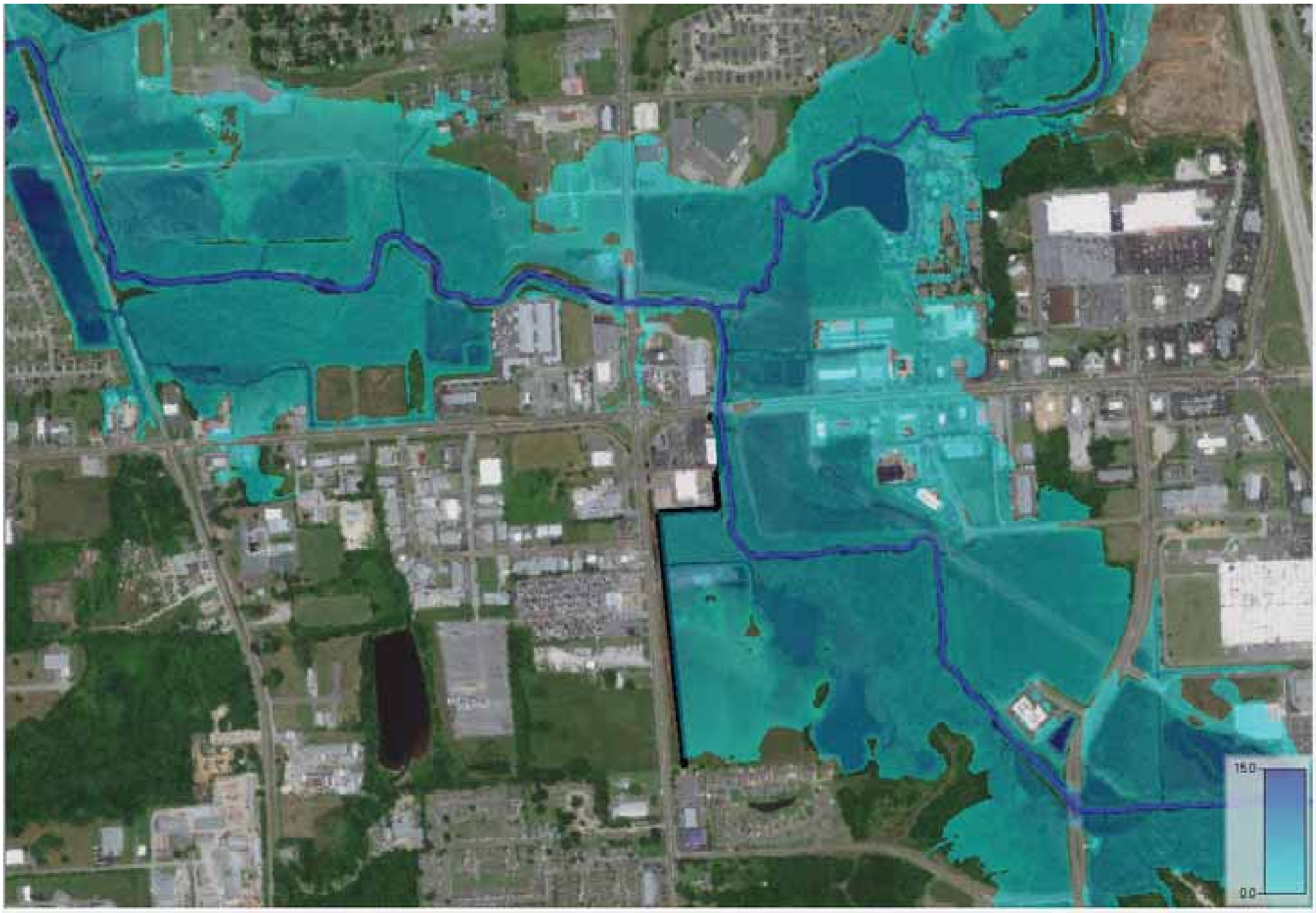

Figure 78. 0.01 AEP Levee Alternative Inundation Depth Map-Bullfrog Corner

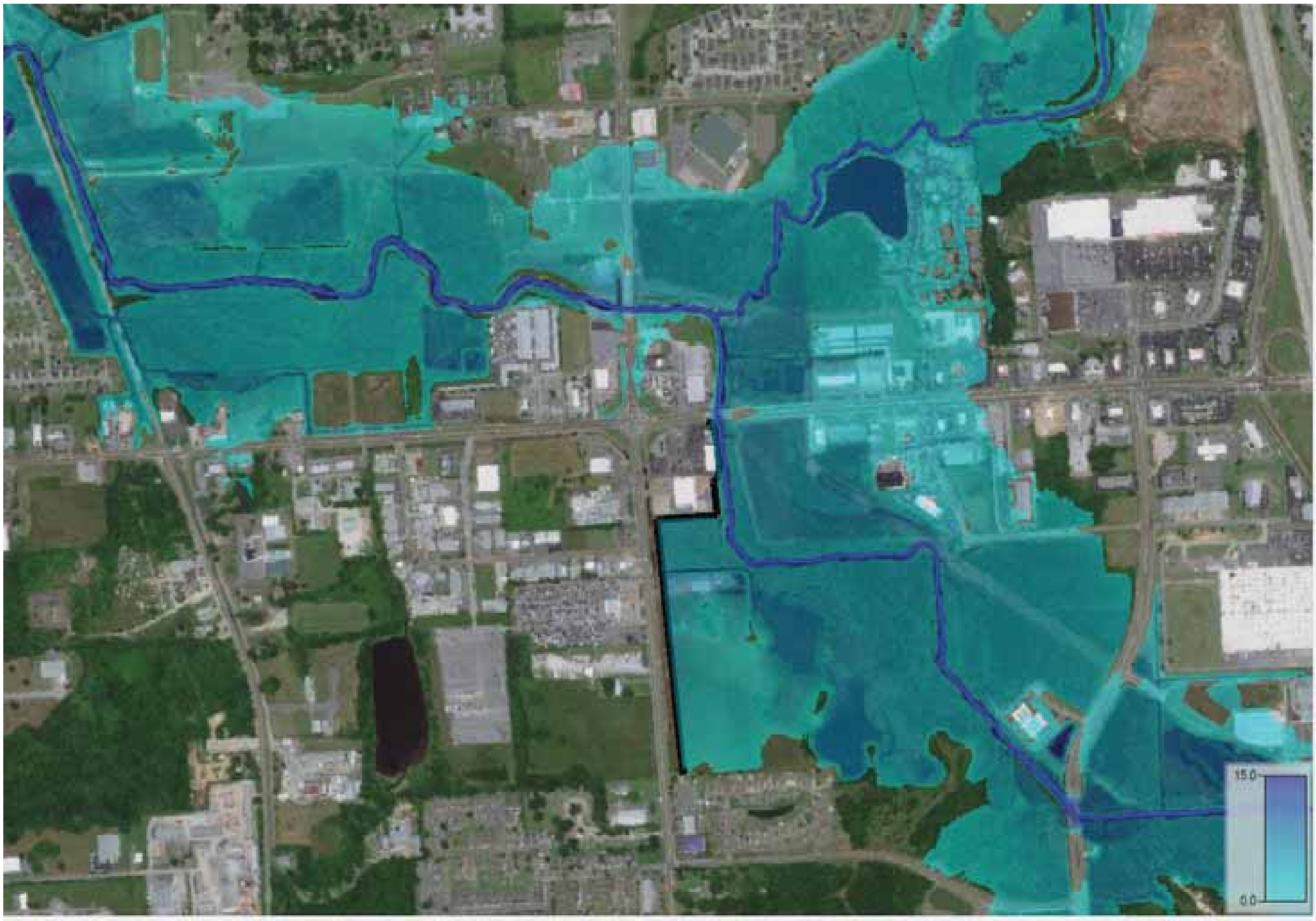

Figure 79. 0.002 AEP Levee Alternative Inundation Depth Map-Bullfrog Corner

20 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 82 grade control structures (GCS) coupled with 393 acres of riparian restoration on eleven streams (Camp, Cane, Horn Lake, Hurricane, Johnson, Lick, Mussacuna, Nolehoe, Nonconnah, Red Banks, and Short Fork Creeks) as shown below.

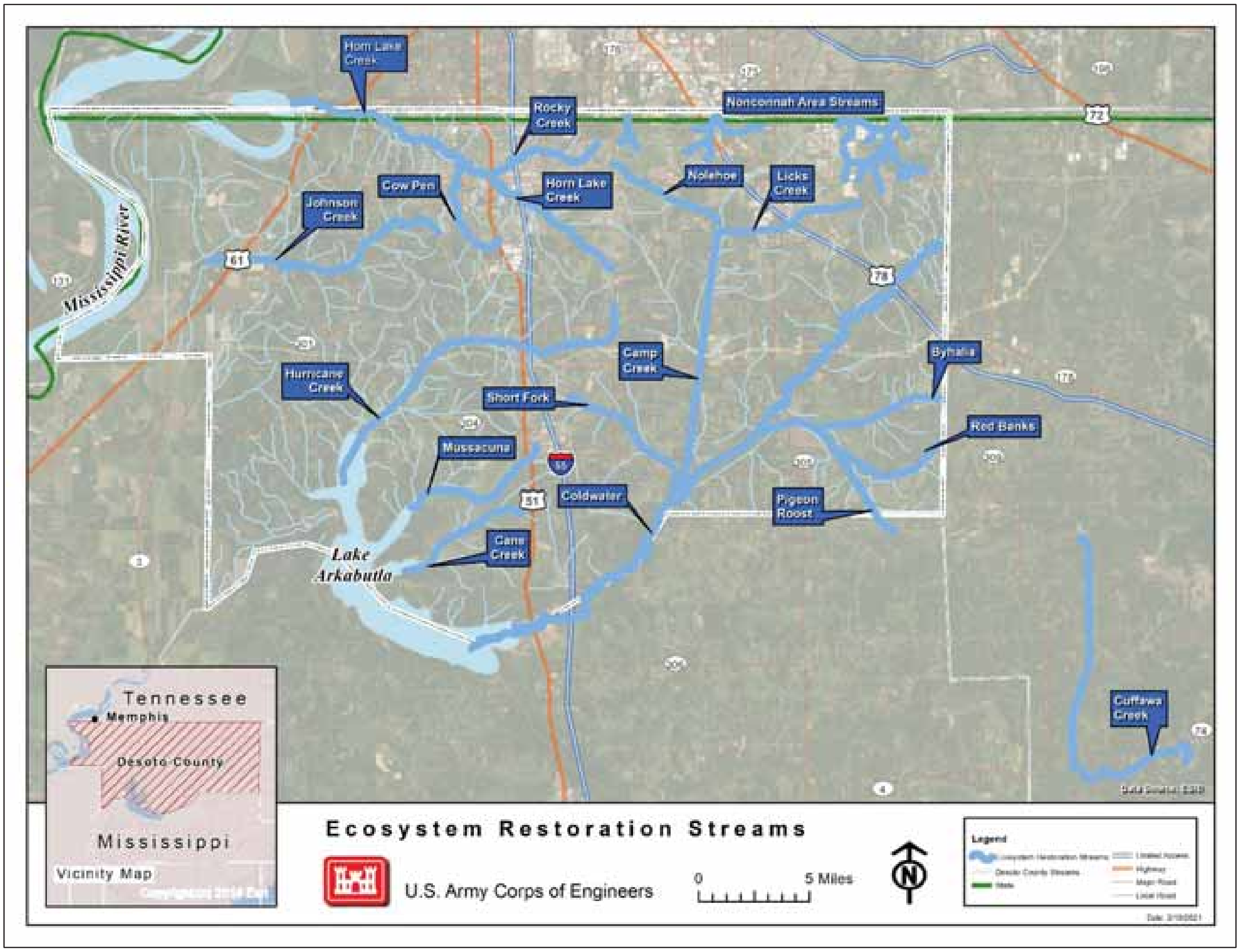

Figure 80-Ecosystem Restoration Streams

20.1 Horn Lake Creek Grade Control Structure Location Determination

Of the proposed 82 structures, 14 are located on Horn Lake Creek. 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 79 below.

Figure 81. 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 a directly dependent on the continued functionality of the existing grade control structures along the channel.

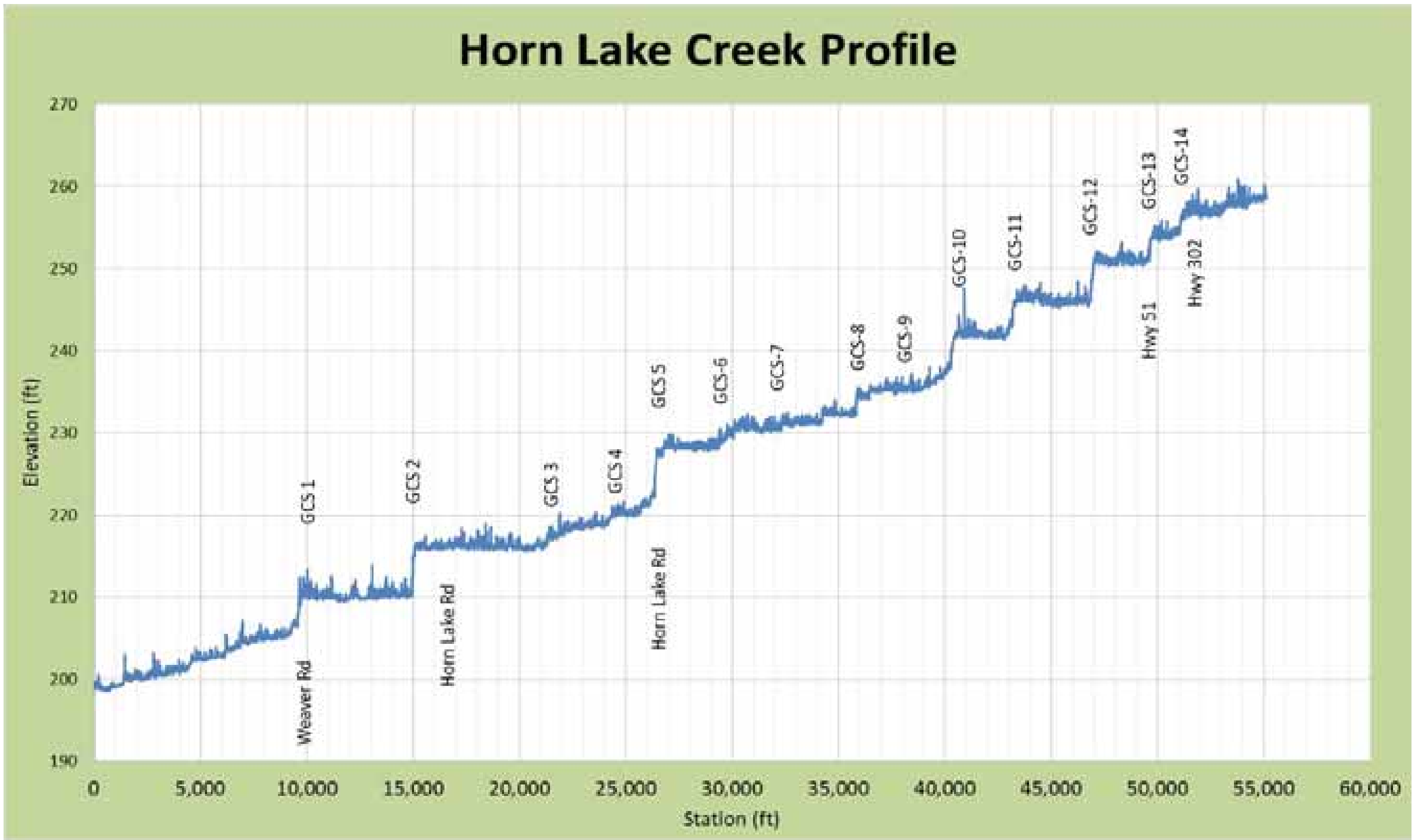

Figure 82. Profile View of Horn Lake Creek GCSs

As shown in Figure 82, 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.

20.2 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).

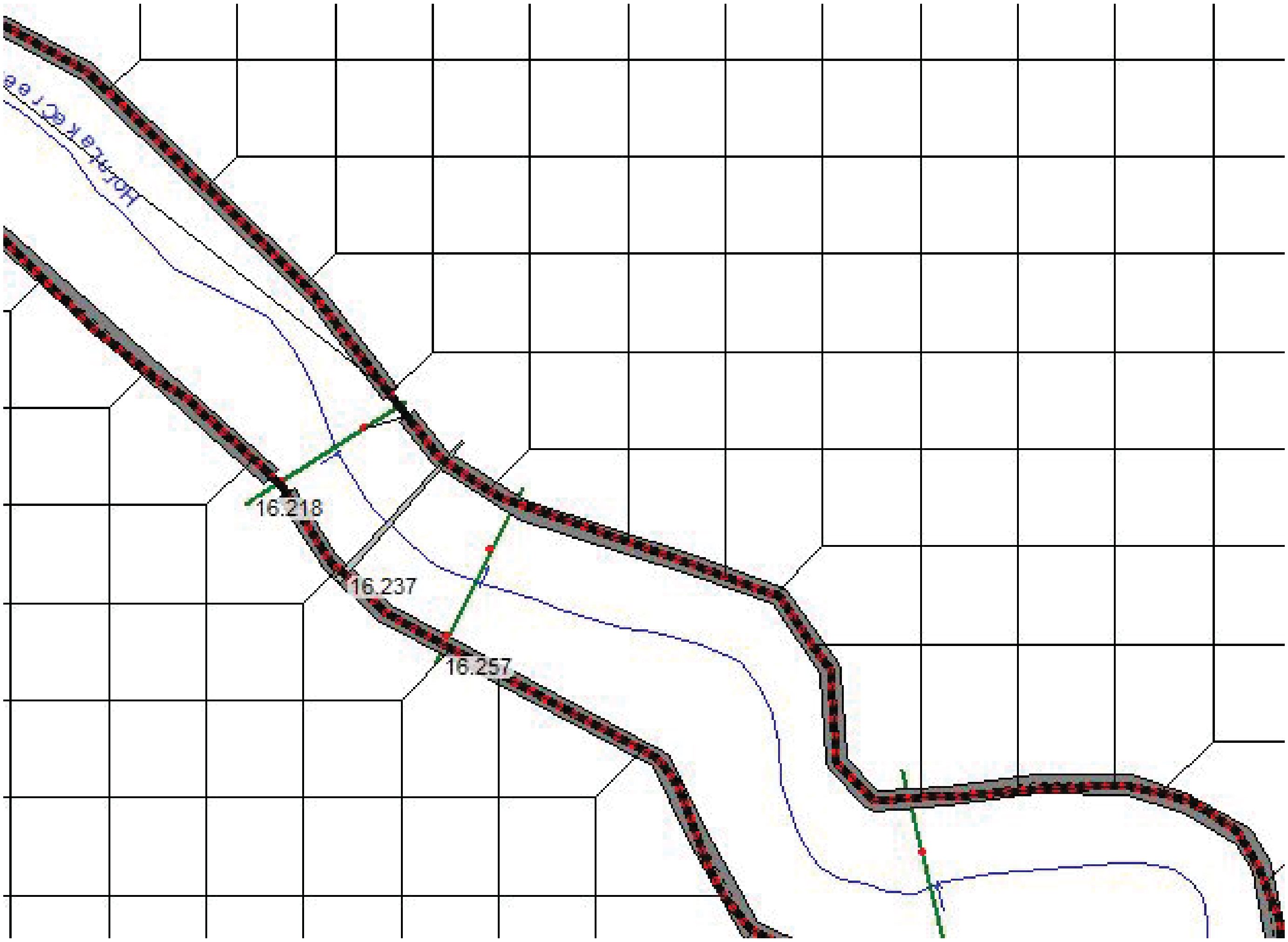

Figure 83. Existing (Green) vs. GCS (Pink) Inundation Comparison Above GCS 6, 7, and 9

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.

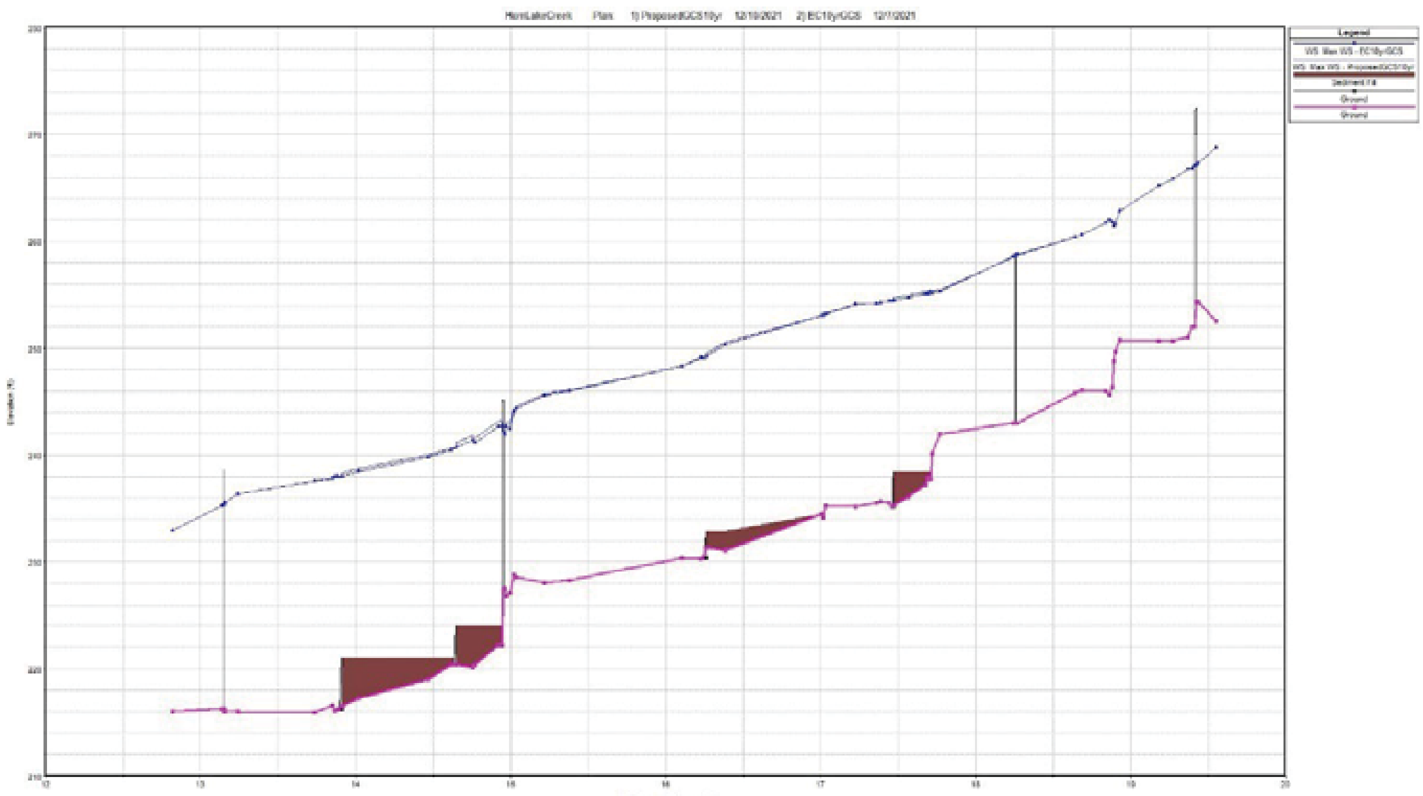

Figure 84. HEC-RAS Inline Structure Locations w/Simulated Sediment Deposition

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 85 to 87. 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.

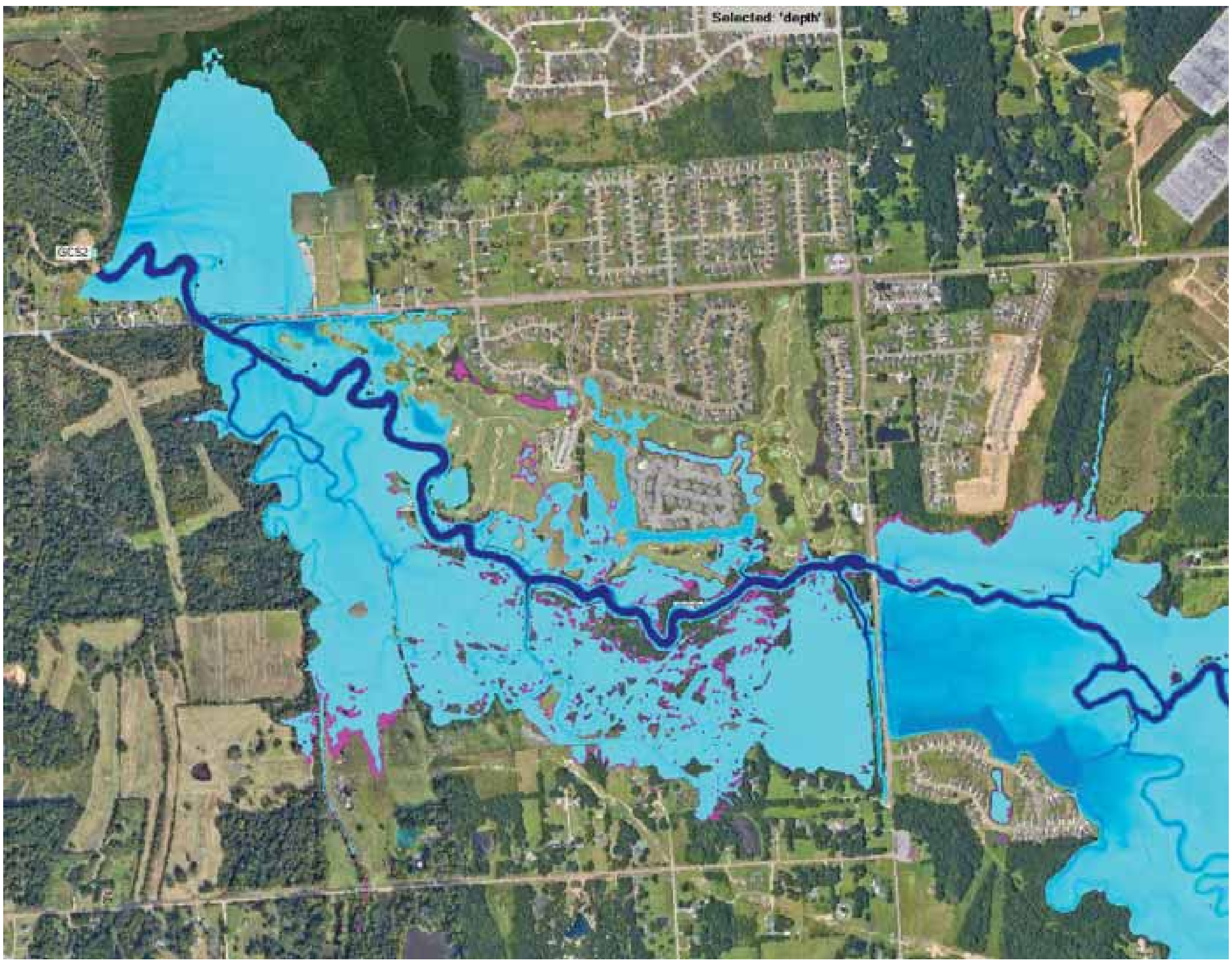

Figure 85. 0.50 AEP Existing (Blue) vs. GCS (Pink) Inundation Comparison Above GCS 3 and 4

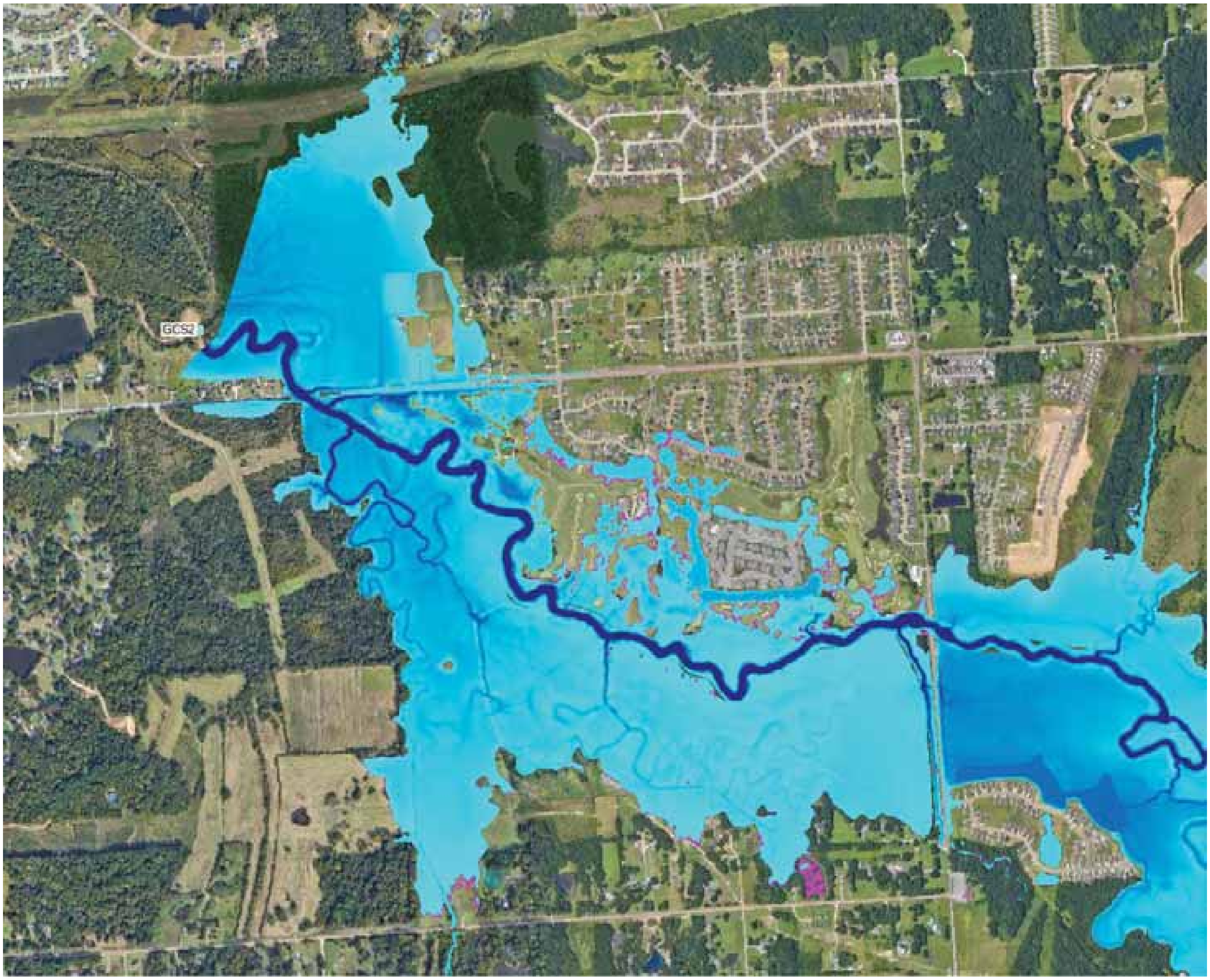

Figure 86. 0.10 AEP Existing (Blue) vs. GCS (Pink) Inundation Comparison Above GCS 3 and 4

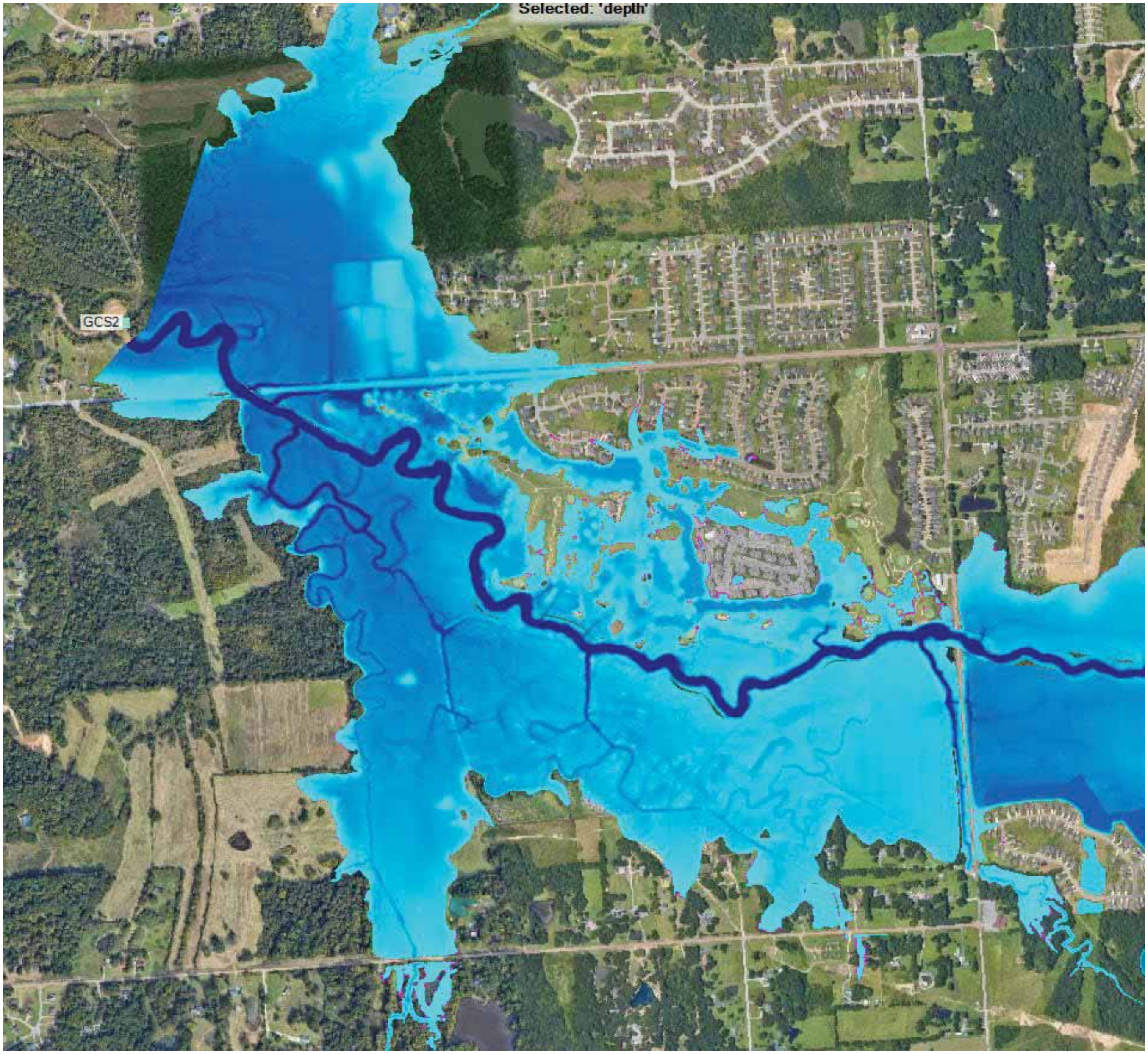

Figure 87. 0.01 AEP Existing (Blue) vs. GCS (Pink) Inundation Comparison Above GCS 3 and 4

20.3 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.

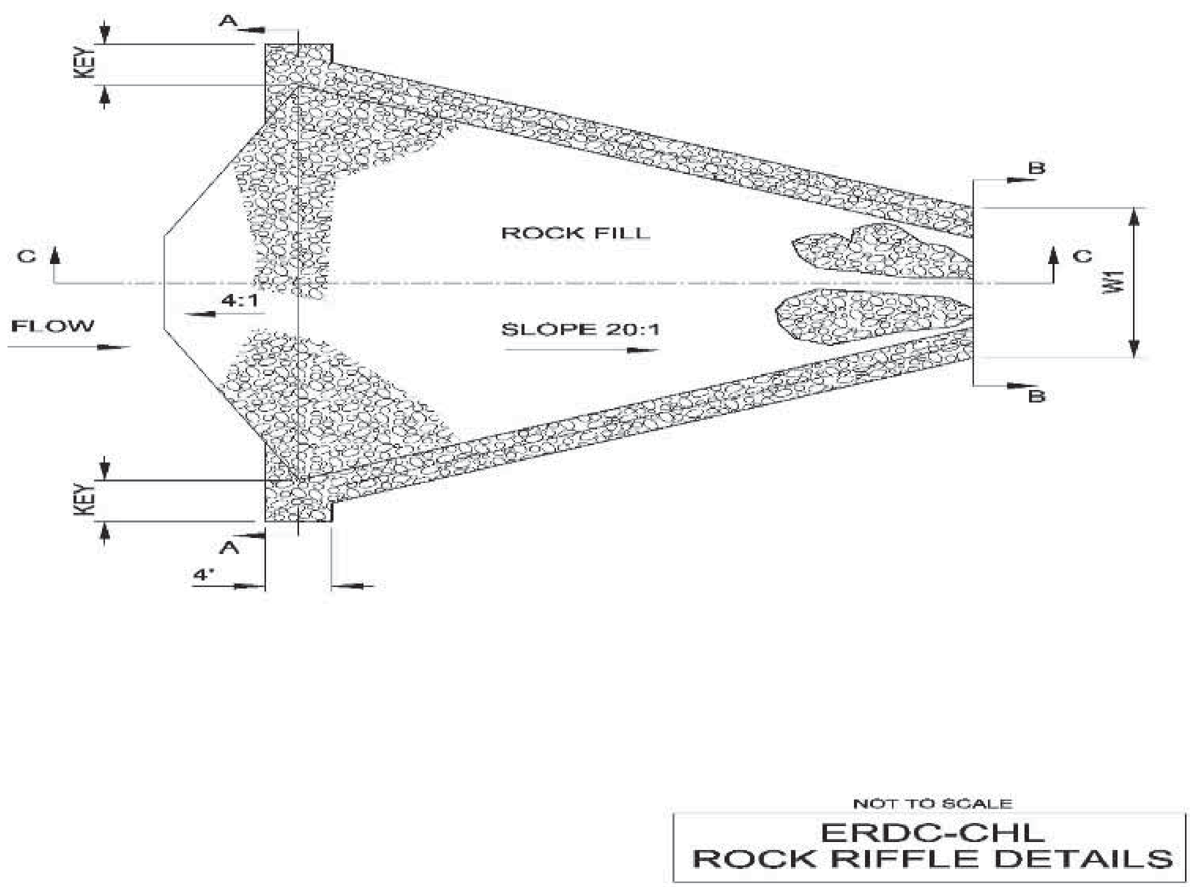

Figure 88. Plan View- ERDC Typical Rock Riffle Structure

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 would face upstream, with smaller 200-pound stone protecting the downstream side. Side slope armoring and keys would 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 Isbach Equation, R650 is stable for velocities less than 9.1 feet per second. Upper bank paving will consist of R200 gradation. Approximate riprap quantities are based on the configurations shown in Figures 89 and 90.

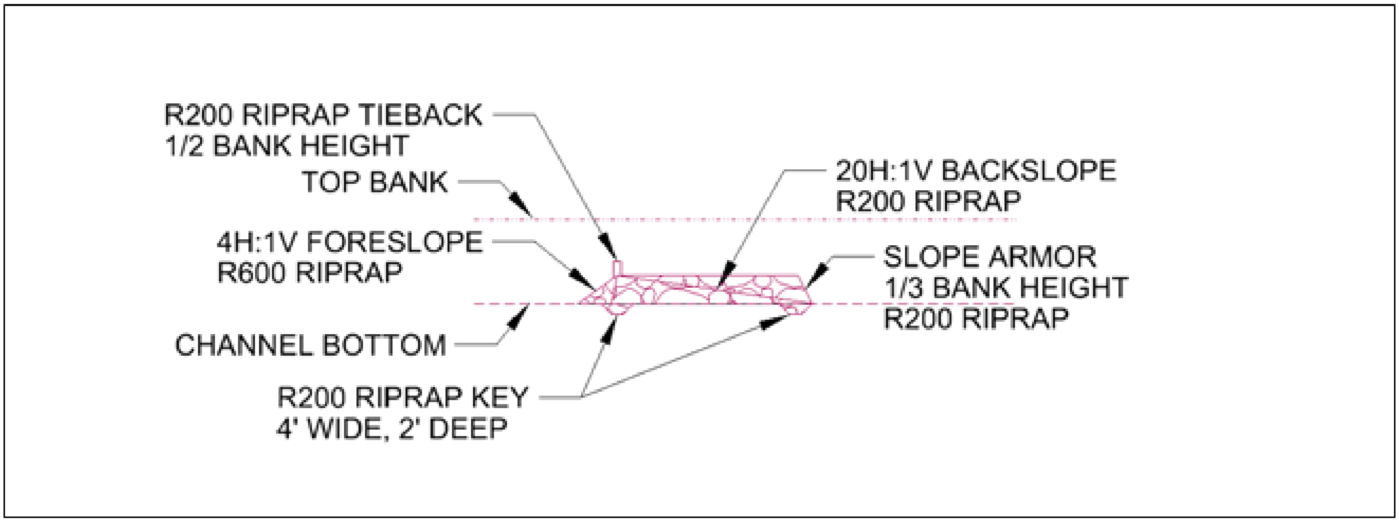

Figure 89. Conceptual Profile View-Grade Control Structure

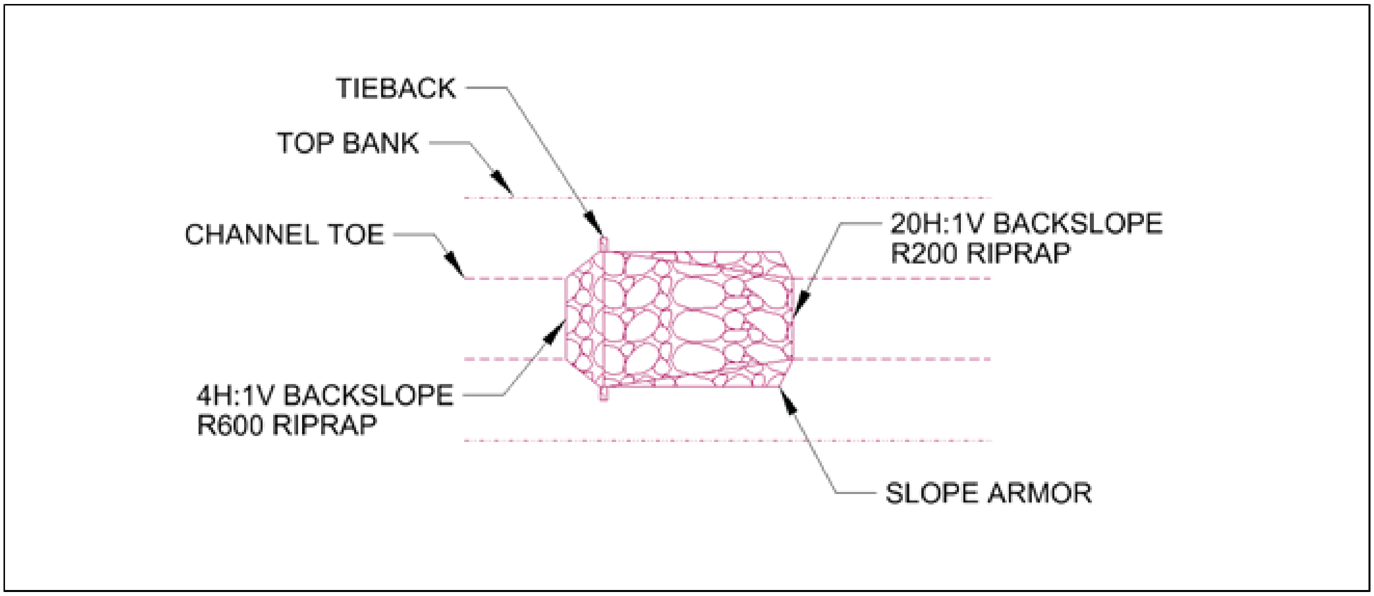

Figure 90. Conceptual Plan View-Grade Control Structure

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 4.

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

Table 5. Grade Control Structure Riprap Details