Interagency Flood Risk Management (InFRM)

Watershed Hydrology Assessment for the Neches River Basin

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InFRM
INTERAGENCY FLOOD RISK MANAGEMENT

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The InFRM Team

As flooding remains the leading cause of natural-disaster loss across the United States, the Interagency Flood Risk Management (InFRM) team brings together federal agencies with mission areas in water resources, hazard mitigation, and emergency management to leverage their unique skillsets, resources, and expertise to reduce long term flood risk throughout the region. The Federal Emergency Management Agency (FEMA) Region VI began sponsorship of the InFRM team in 2014 to better align Federal resources across the States of Texas, Oklahoma, New Mexico, Louisiana, and Arkansas. The InFRM team is comprised of FEMA, the U.S. Army Corps of Engineers (USACE), the US Geological Survey (USGS), and the National Weather Service (NWS), which serves under the National Oceanic and Atmospheric Administration (NOAA). One of the first initiatives undertaken by the InFRM team was performing Watershed Hydrology Assessments for large river basins in the region.

The Federal Emergency Management Agency (FEMA) funded the Watershed Hydrology Assessments to leverage the technical expertise, available data, and scientific methodologies for hydrologic assessment through the InFRM team. This partnership allows FEMA to draw from the local knowledge, historic data and field staff of its partner agencies and develop forward leaning hydrologic assessments at a river basin level. These studies provide outcomes based on all available hydrologic approaches and provide suggestions for areas where the current flood hazard information may require update. FEMA will leverage these outcomes to assess the current flood hazard inventory, communicate areas of change with community technical and decision makers, and identify/prioritize future updates for Flood Insurance Rate Maps (FIRMs).

The U.S. Army Corps of Engineers (USACE) has participated in the development of the Watershed Hydrology Assessments as a study manager and member of the InFRM team. USACE served in an advisory role in this study where USACE's expertise in the areas of hydraulics, hydrology, water management, and reservoir operations was required. USACE's primary scientific contributions to the study have been in rainfall runoff watershed modeling and reservoir analyses. The reservoir analyses in this study are based on USACE's firsthand reservoir operations experience and the latest scientific techniques from USACE's Dam Safety program.

The U.S. Geological Survey (USGS) Texas Water Science Center has participated in the development of this study as an adviser and member of the InFRM team. USGS served in an advisory role for this study where USGS' expertise in stream gaging, modeling, and statistics was requested. USGS's primary scientific contribution to the study has been statistical support for flood flow frequency analysis. This flood flow frequency analysis included USGS firsthand stream gaging expertise as well as advanced statistical science.

NOAA National Weather Service (NWS) has participated in the development of this study as an adviser and member of the InFRM team. NOAA NWS served in an advisory role of this study where expertise in NOAA NWS' area of practice in water, weather and climate was requested. NOAA's primary scientific contribution to the study has been the NOAA Atlas 14 precipitation frequency estimates study for Texas. This precipitation-frequency atlas was jointly developed by participants from the InFRM team and published by NOAA. NOAA Atlas 14 is intended as the U.S. Government source of precipitation frequency estimates and associated information for the United States and U.S. affiliated territories.

More information on the InFRM team and its current initiatives can be found on the InFRM website at www.InFRM.us.

EXECUTIVE SUMMARY

The Federal Emergency Management Agency (FEMA) administers the National Flood Insurance Program (NFIP), which was created in 1968 to guide new development (and construction) away from flood hazard areas and to help transfer the costs of flood damages to the property owners through the payment of flood insurance premiums. The standard that is generally used by FEMA in regulating development and in publishing flood insurance rate maps is the 1% annual chance (100-yr) flood. The 100-yr flood is defined as a flood which has a 1% chance of happening in any year. The factor that has the greatest influence on the depth and width of the 100-yr flood zone is the expected 1% annual chance (100-yr) flow value.

This report summarizes new analyses that were completed as part of a study to estimate the 1% annual chance (100-yr) flow, along with other frequency flows, for various stream reaches throughout the Neches River Basin in Texas. This study was conducted for FEMA Region VI by an Interagency Flood Risk Management (InFRM) team. The InFRM team is a partnership of federal agencies that includes subject matter experts (SME) from FEMA, the U.S. Army Corps of Engineers (USACE), the U.S. Geological Survey (USGS), and the National Weather Service (NWS). In addition to the federal partners of the InFRM team, regional stakeholders such as the Lower Neches Valley Authority (LNVA), the Angelina & Neches River Authority (ANRA), and the Texas Water Development Board (TWDB) also participated in the progress updates and review processes for this study. This study represents a significant step forward towards increasing resiliency against flood hazards in the Neches River basin.

The InFRM team used several hydrologic methods, including statistical hydrology, rainfall-runoff modeling, period of record simulations, and reservoir analyses, to estimate the 1% annual chance (100-yr) flow and then compared those results to one another. The purpose of the study is to produce 100-yr flow values that are consistent and defendable across the basin.

The InFRM team used up-to-date statistical analysis along with state-of-the-art rainfall-runoff watershed modeling and reservoir analyses to estimate the 1% annual chance (100-yr) flow values throughout the Neches River Basin. In the statistical analysis, the gage records were updated through the year 2018 or 2020 to include all recent major flood events. However, since statistical estimates inherently change with each additional year of data, their results were compared to the results of a detailed watershed model, which is less likely to change over time. For example, the rainfall depths used in the runoff watershed model came from the NOAA Atlas 14 precipitation atlas published in 2018 and most of these depths changed less than 10% from the rainfall depth published in the NWS Technical Paper 40 (TP-40) in 1961, almost 60 years ago. More significant changes in the rainfall values were observed in the extreme southern portion of the basin near the Gulf of Mexico, but the rainfall depths upstream of the B.A. Steinhagen Lake have been much more stable over time than the statistical estimates of flood frequency, specifically for the 1% annual chance event.

Rainfall-runoff watershed modeling is used to simulate the physical processes that occur during storm events including how water moves across the land surface and through the streams and rivers. A watershed model was built for the Neches River Basin with input parameters that represented the physical characteristics of the watershed. After building the model, the InFRM team calibrated the model to verify that it was accurately simulating the response of the watershed to a range of observed flood events, including large events similar to a 1% annual chance (100-yr) flood. A total of eight recent storm events spanning from 2006 to 2019 were used to fine tune the model.

For the eight storm events used to fine tune the model, the availability of National Weather Service (NWS) hourly rainfall radar data allowed for more detailed calibration of the watershed model than would have been possible during earlier modeling efforts. The final watershed model accurately simulated the response of the Neches watershed, as it reproduced the timing, shape, and magnitudes of the observed floods very well. An example plot of the modeled flow versus the recorded flow is shown below in Figure ES.1, but many more examples are available in Appendix B.

The model calibration and verification process undertaken during this study substantially exceeds the standard of a typical FEMA floodplain study. Because these rainfall-runoff models have been calibrated to observed watershed responses to storm events, there is more assurance that these models, when paired with best available precipitation frequency information, provide the best available representation of flood risk.

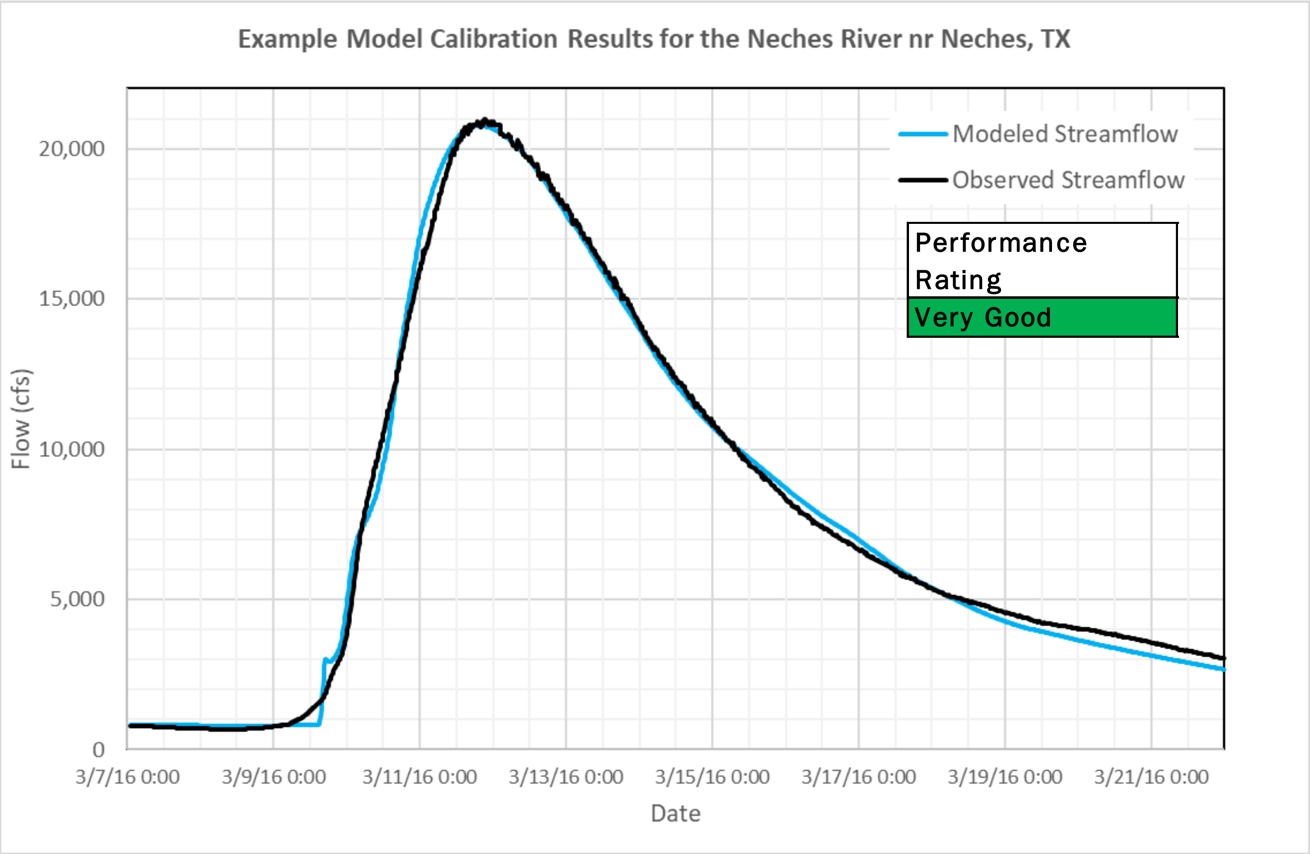


Figure ES.1: Example of Watershed Model Results versus Recorded Flow at the Streamgage

The 1% annual chance (100-yr) flow values were then calculated by applying a 100-yr storm to the watershed model. Rainfall estimates for the 100-yr storm are considered more reliable than statistical estimates for the 100-year flow due to the larger number of rainfall stations and the longer periods of time during which rainfall measurements have been made. The accuracy of those rainfall frequency estimates was further advanced by the release of NOAA Atlas 14 for Texas in 2018 (NOAA, 2018). NOAA Atlas 14 is the U.S. Government source of precipitation frequency estimates and is the most accurate, up-to-date, and comprehensive study of rainfall depths in Texas. The regional approach used in NOAA Atlas 14 incorporated at least 1,000 cumulative years of daily data into each location's rainfall estimate, yielding better estimates of rare rainfall depths such as the 100-yr storm. These new rainfall depths from NOAA Atlas 14 were applied to the calibrated watershed model for the Neches River basin.

After completing the model runs, the watershed model results were compared to previous studies and to the results of other hydrologic methods. Where there were significant differences, investigations were made into the drivers of those differences. Extensive comparisons were made between the watershed model results, the USGS gage record results, the flood of record, and previously published flow values, which can be found in Chapter 11 of this report. The expected impacts of reservoir operations for Sam Rayburn Reservoir and B.A. Steinhagen Lake were also analyzed in detail for this study, and the frequency dam releases and pool elevations that resulted from the reservoir analyses were recommended for the reaches immediately upstream and downstream of the dams.

The final recommendations for the Neches Watershed Hydrology Assessment were formulated through a rigorous process which required technical feedback and collaboration between all of the InFRM subject matter experts. This process included the following steps: (1) comparing the results of the various hydrologic methods to one another, (2) performing an investigation into the reasons for the differences in results at each location in the watershed, (3) selecting of the draft recommended methods, (4) performing internal and external technical reviews of the hydrologic analyses and the draft recommendations, and finally, (5) finalizing the study recommendations. After completing this process, the flows that were recommended for adoption by the InFRM team came from a combination of watershed model results using NOAA Atlas 14 uniform rainfall, elliptical storms, and reservoir analysis techniques.

Previously published frequency discharges from effective FEMA Flood Insurance Studies (FIS) and Base Level Engineering (BLE) data in the Neches River Basin differ from the recommended flow frequency results of this study in many locations. The new flow frequency results are higher than the previously published results in some areas, while they are lower in other areas. Figures ES.2 and ES.3 compare the recommended 1% annual chance (100-yr) results from this Watershed Hydrology Assessment with previously published flows at some key locations throughout the basin. For most areas of the upper Neches and Angelina River watersheds, the recommended results of this study are either similar to or slightly higher than the previously published 100-yr flows from the Base Level Engineering (BLE) data that was release in 2020, as shown in Figure ES.2. Similarly, the statistical analyses of the gage records are generally consistent with the recommended results from this study, but may be slightly higher or lower than the recommended results at a given location, depending on whether or not large storms have occurred in that particular watershed during the observed gage record. The upper Neches and Angelina River watersheds generally have not been studied in detail; therefore, there are no effective FEMA FIS flows available for comparison in these areas.

For the lower Neches River and its tributaries, there is more variation between the recommended results of this study and the previously published 100-yr flows from the effective FEMA FIS and the 2020 BLE data, as shown in Figure ES.3. Therefore, the changes in these flow frequency estimates can primarily be attributed to a combination of factors including (1) additional gage record length, (2) improved calibration of the rainfall runoff model, and (3) increased rainfall depths near the Gulf coast. First, the new flow frequency results from this study differ from the effective flood insurance values because there have been new floods in the gage record, which caused some of the current statistical estimates to be very different than they were when the previous FEMA FIS flow frequency estimates were developed. While the effective FEMA FIS maps in the lower Neches basin were updated between 2002 and 2011, the hydrology behind those flood insurance maps has not been updated since the 1980s or early 1990s. In addition, the current study found that the extreme magnitude of Hurricane Harvey caused many of the current statistical 100-yr estimates to be overestimated in the lower portions of the Neches watershed. Second, the rainfall-runoff watershed model underwent extensive calibration to accurately simulate the response of the watershed to a range of recent observed flood events, including large events similar to a 1% annual chance (100-yr) flood and even more extreme events like Hurricane Harvey. The frequency flow results of the calibrated rainfall-runoff watershed model exposed that some of the FIS flows calculated in the past using statistical hydrology or uncalibrated rainfall-runoff modeling did not accurately reflect the response of the watershed to a 1% annual chance (100-yr) storm event. Finally, NOAA Atlas 14 revealed that previous estimates of the 100-yr rainfall depths near Gulf coast had been underestimated by up to 3 inches for the 24-hour duration and up to 6 inches for the 4-day duration. This additional rainfall led to higher peak flows on portions of Pine Island Bayou and the lower Neches River when compared to the effective FIS flows, as shown in Figure ES.3.

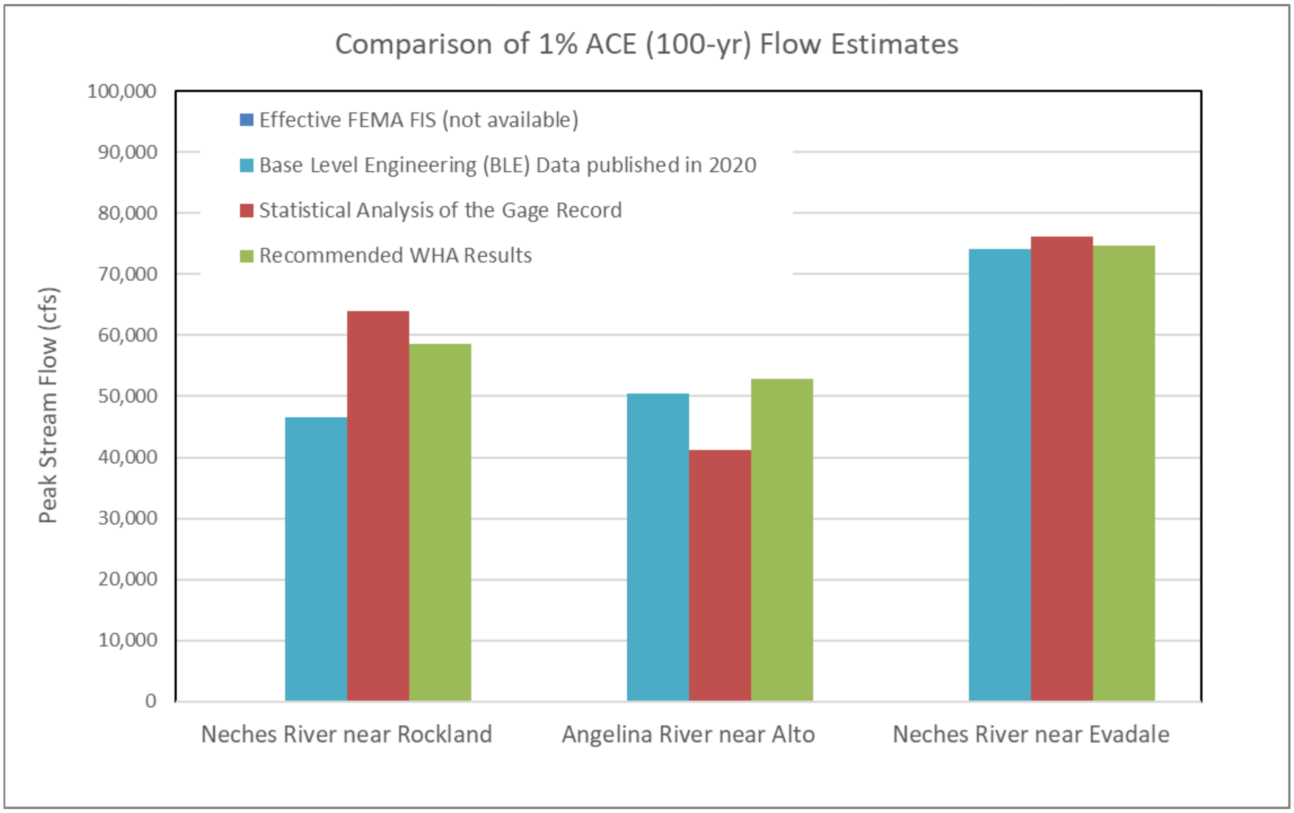


Figure ES.2: Comparison of 1% Annual Chance (100-yr) Flow Results on the Upper Neches and Angelina Rivers

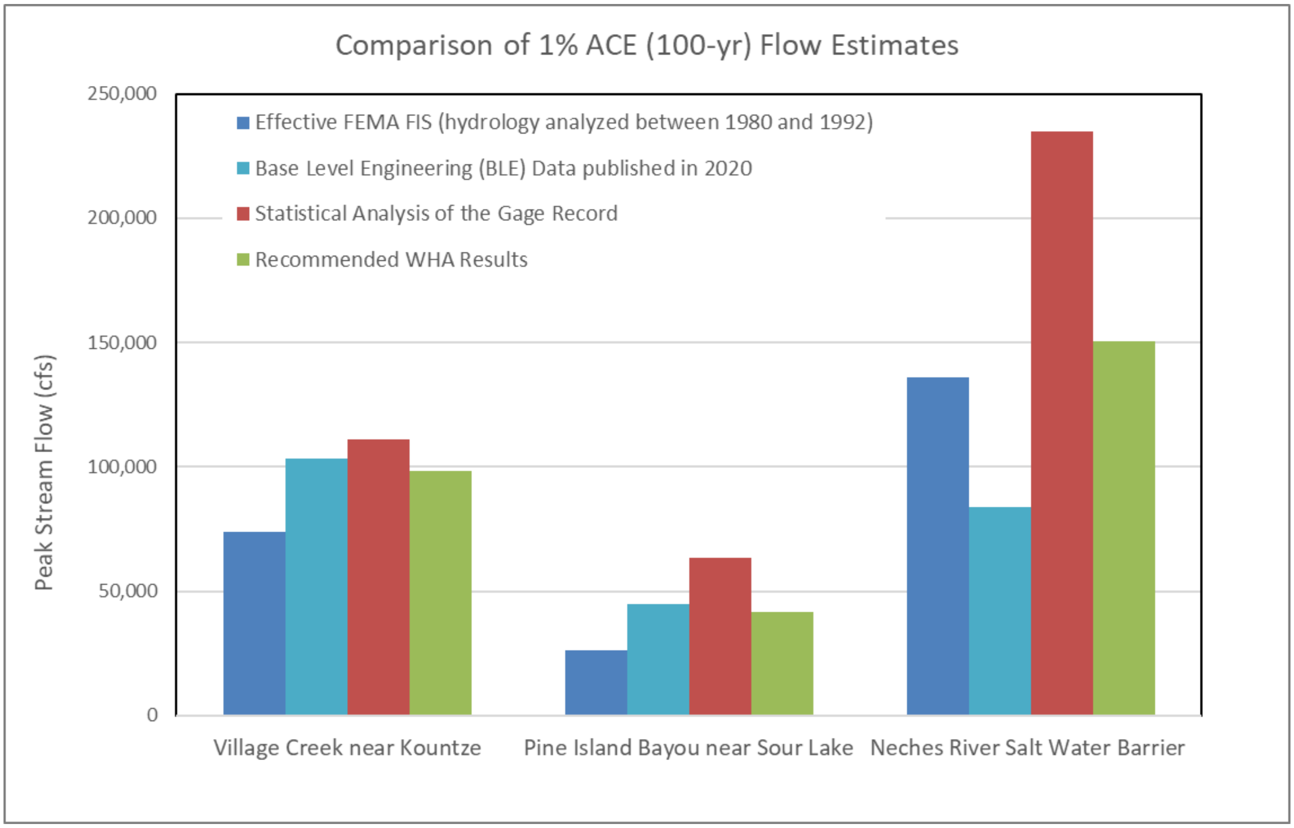


Figure ES.3: Comparison of 1% Annual Chance (100-yr) Flow Results on the Lower Neches Watershed

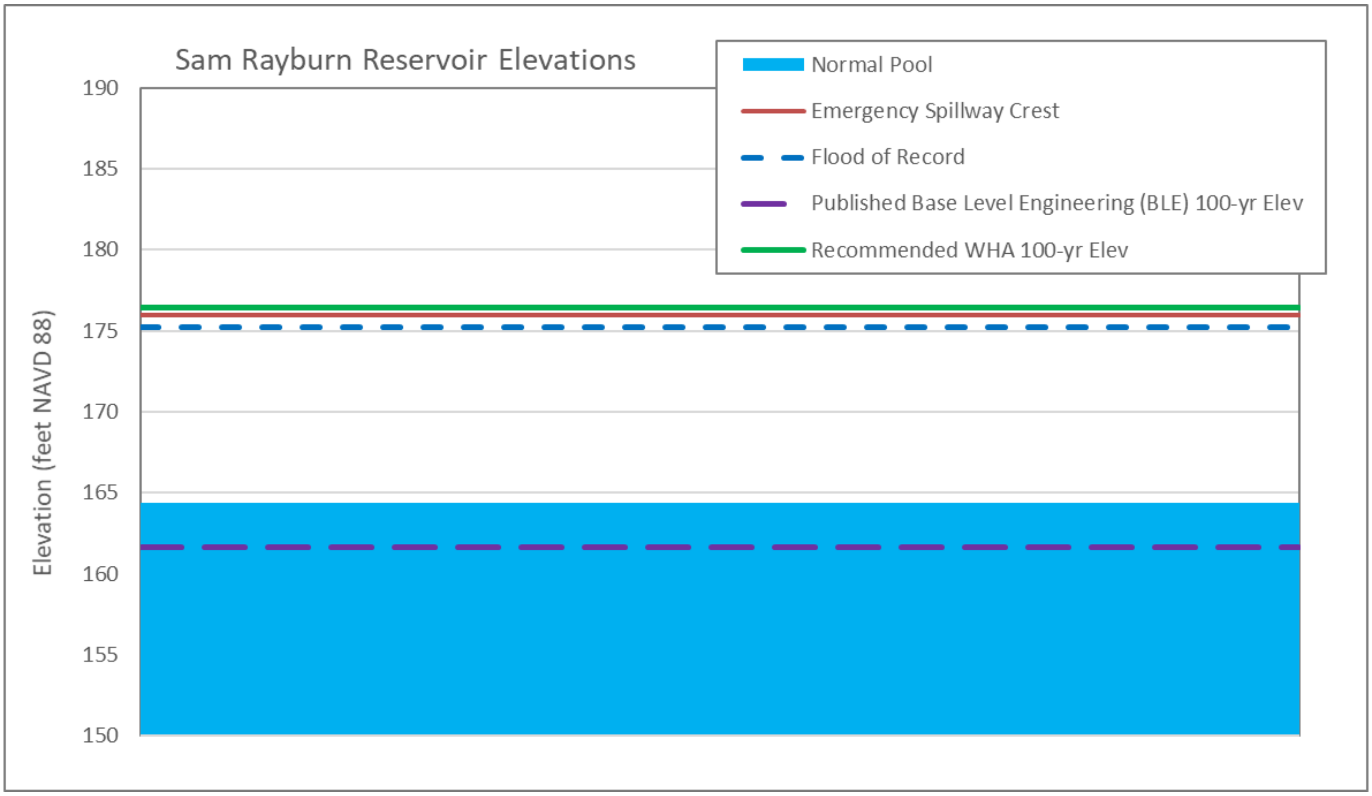


Figure ES.4: Comparison of 100-yr Pool Elevations for Sam Rayburn Reservoir

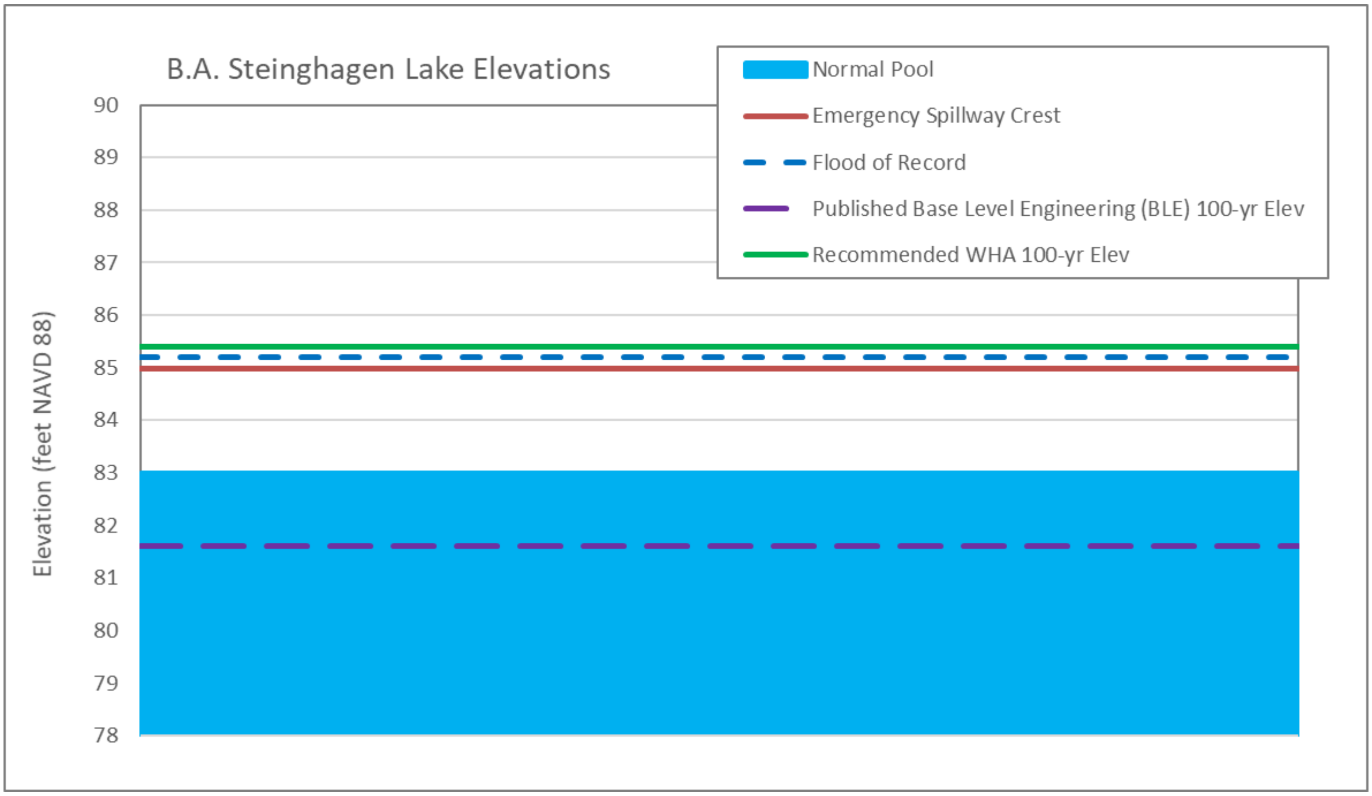


Figure ES.5: Comparison of 100-yr Pool Elevations for B.A. Steinhagen Lake

Figures ES.4 and ES.5 compare the recommended 100-yr pool elevation results from this study with the previously published water surface elevations at the two USACE reservoirs in the Neches River Basin: Sam Rayburn Reservoir and B.A. Steinhagen Lake. These figures show that the recommended 100-yr pool elevations from this study are generally just above the emergency spillway crest and the flood of record for each respective reservoir. These figures also show that the recommended results of this study are much higher than the previously published 100-yr elevations from the Base Level Engineering (BLE) data, which are actually below the reservoirs' normal pool elevations. The recommended results of this study came from detailed reservoir analyses that utilized stochastic techniques to account for the operations of the dam, its inflow frequency and volume, and the possible range of its starting pool elevations. On the other hand, these facts and effects of the reservoirs were not accounted for in the BLE data. Additionally, there are no effective pool elevations available from FEMA FIS for comparison due to the fact that the areas near these two reservoirs have not been included in any detailed FEMA mapping studies.

Given the severe loss of life and property that has occurred during recent floods within the State of Texas, it is imperative that future updates to the published flood insurance rate maps for the Neches River Basin accurately reflect the known levels of flood risk in the basin. The recommended results from this study should be considered the best available estimate of flood risk for the larger streams in the Neches River basin, based on a range of hydrologic methods performed by an expert team of engineers and scientists from multiple federal agencies. For smaller tributaries in the Neches basin, the recommended results provide a good starting point which could be further refined by adding additional subbasins to the watershed model and by using methodologies that are consistent with this study.

As a result of the level of investment, analyses, and collaboration that went into this Watershed Hydrology Assessment, the flood risk estimates contained in this report are recommended as the basis for future NFIP updates or other flood risk studies within the Neches River basin. These results can also be used to plan new infrastructure and safely locate new neighborhoods and other urban development. These federally developed modeling results form a consistent understanding of hydrology across the Neches watershed, which is a key requirement outlined in FEMA's General Hydrologic Considerations Guidance. Furthermore, the models and data used to produce these flood risk estimates are available upon request, at no charge, to communities, local stakeholders, and architecture engineering firms. Requests for the models should be sent to the InFRM team through the InFRM website at www.InFRM.us.

While the results from this study should be considered the best available estimates of flood risk for many areas of the Neches River basin, significant uncertainty still remains, as it does in any hydrologic study. Because of this uncertainty and because of the potential impacts these estimates can have on life and property, the InFRM team strongly recommends and supports local communities that implement higher standards, such as additional freeboard requirements, floodplain management practices based on standards greater than the 1% annual chance flood, and/or "no valley storage loss" criteria.

1 Study Background and Purpose

1.1 THE NATIONAL FLOOD INSURANCE PROGRAM

The National Flood Insurance Program (NFIP) was created in 1968 to guide new development (and construction) away from flood hazard areas and to help transfer the costs of flood damages to the property owners through the payment of flood insurance premiums. The NFIP program is administered by FEMA within the Department of Homeland Security. The NFIP is charged with determination of the 1% and 0.2% annual chance flood risk and with mapping that flood risk on the Flood Insurance Rate Maps (FIRMs). FEMA Region 6 has an inventory of hundreds of thousands of river miles across Texas, Louisiana, Arkansas, Oklahoma, and New Mexico that are in need of flood risk mapping updates or validation. The current flood hazard inventory is available for viewing on FEMA's National Flood Hazard Layer (NFHL) Viewer at https://msc.fema.gov/nfhl.

FEMA's inventory is focused on determining the extent and areas that are vulnerable to flooding during the 1% annual chance (1 in 100 chance of occurrence each calendar year) and 0.2% chance (1 in 500 chance of occurrence each calendar year). Flood hazards are assessed along natural drainage elements such as rivers, streams, and creeks. The program focuses on comprehensive and broad analysis to define, determine and communicate flooding potential.

The Flood Insurance Rate Maps (FIRMs) published by FEMA define the area where flood insurance purchase is mandatory. The mandatory purchase area includes insurable structures within the defined 1% annual chance floodplain with federally backed mortgages. However, the engineering modeling and the flood extents produced and released on FIRMs do not describe the full potential for flooding, as the FIRMs focus on natural streams, creeks and rivers that traverse the watershed and generally do not determine flood hazards related to highly urbanized flooding problems from man-made drainage systems such as sewers and pipe networks.

The standard that is generally used by FEMA in publishing Flood Insurance Rate Maps (FIRMs) for the NFIP is the 1% annual exceedance probability (AEP) flood, also known as the 100-year flood. The 1% AEP, or 100-year flood is defined as a 1 in 100 chance of occurrence each calendar year. The chance of a 100-year flood occurring during the life of a 30-year mortgage or over the life of a structure is much more probable than its name suggests, as shown in Figure 1.1. These statistics underline the need to minimize uncertainty in flood frequency estimates.

Engineering modeling prepared by Federal, State, local, academic and private industry utilize standard engineering practices to determine:

- **Hydrologic Conditions in a Study Area.** In a hydrologic analysis, ground slope, land use, soil types and climatic factors are analyzed to determine how much flood water is expected to collect on the landscape. This flood volume is entered into hydraulic engineering models.
- **Hydraulic Conditions.** Hydraulic engineering efforts generalize stream and channel geometries utilizing ground elevation information to define the areas available to convey flood volumes. These analyses describe stream cross-sections that are analyzed to determine how high the water will rise in the stream channel and/or if it will expand into the natural floodplain areas adjacent to these stream channels. The output of these analysis is a series of calculated water surface elevations.
- **Flood Extent.** The water surface elevations determined by the hydraulic analysis are then reviewed against ground elevation information to define the areas which are prone to flooding during the analyzed event.

Figure 1.1: Probabilities of the 100-yr Flood

1.2 THE CHALLENGE AND IMPORTANCE OF HYDROLOGY

In standard engineering practice, the factor that has the greatest influence on the depth and width of the 100-year floodplain is the 1% annual chance (100-yr) flow estimate. As a result, hydrology remains the single largest source of uncertainty in the estimation of flood risk. The challenge of hydrology is that there are many different commonly used and accepted methods for estimating the 1% annual chance flow, and every method will result in a different answer. In Texas, where the climate can cause dramatic shifts between drought and flood cycles, the variation in flood risk estimation can be quite extreme. The challenge of climactic and hydrologic variation points to the need for a more thorough approach to hydrology using multiple scientific methods.

In addition to the natural variation described above, urbanization and reservoir regulation provide additional challenges to hydrology and the estimation of flood risk. For basins which include major reservoirs, such as the Neches River basin, first-hand knowledge of reservoir operations and additional analysis is needed for accurate flood risk estimation. For basins experiencing major population growth and urban development, land use change must also be considered in the analysis.

1.3 PURPOSES OF THE WATERSHED HYDROLOGY ASSESSMENT

The InFRM Watershed Hydrology Assessment for the Neches River Basin summarizes new analyses that were completed as part of a study to estimate the 1% annual chance (100-yr) flow, along with other frequency flows, for various stream reaches across the river basin. This study also produces greatly refined meteorologic and hydrologic tools, analysis and data, including verification studies that ensure that the tools accurately reflect the basin's response to intense rainfall events. The tools, analyses and data produced in this study can be leveraged by local communities to manage their growth and development and to better estimate the risk of flooding associated with constructing infrastructure and urban development in the vicinity of significant streams and rivers.

This study was conducted for FEMA Region 6 by the InFRM team. The InFRM team includes subject matter experts (SME) from USACE, the USGS, and the NWS. The Watershed Hydrology Assessment employed a thorough approach to the hydrology of the Neches River basin. The multi-layered analysis used in this assessment applied a range of hydrologic methods, including rainfall runoff modeling, statistical hydrology, period-of-record simulations, and reservoir analyses, and then compared the results of those methods to one another. This type of multi-layered analysis helped to reduce the uncertainty in the 1% annual chance (100-yr) flow estimates by ensuring that all possible variables affecting flood risk in the basin have been examined. The analysis also accounts for the impacts of non-stationary factors, such as reservoir regulation and climate variation, which helps to tell the story of how the 1% annual chance flow estimate has changed over time.

The purpose of this study is to produce 1% annual chance and other frequency flows that are consistent and defendable across the Neches River basin based on analyses from multiple methods. The end product of this hydrology assessment will include a hydrology report for use as a reference to evaluate against existing studies and to support new local studies. The results of the watershed hydrology assessment will provide FEMA suggested 1% and 0.2% peak flow rates along the major rivers and tributaries and will inform future updates to Flood Insurance Rate Maps (FIRMs). These analyses will allow Federal, State and Local entities to leverage these basin wide results in a variety of ways.

FEMA will leverage the outcomes from this study to assess the current flood hazard inventory, communicate areas of change with community technical staff and decision makers, and identify/prioritize future updates for FIRMs. This watershed hydrology assessment also provides the recommended hydrologic methods and results needed for use on local studies, which may add the detail necessary to develop frequency flows at a smaller scale. The watershed assessment gives a consistent avenue of updating the hydrology for large, complex river systems, such as the Neches River basin, much of which is either mapped with methods or has not had its hydrology updated in decades.

This report summarizes all of the hydrologic analyses that were completed to estimate frequency peak stream flows for significant stream reaches throughout the Neches River Basin. The results of all hydrologic analyses and the recommended frequency discharges are summarized herein. Additional technical detail is also available in the appendices to this report.

1.4 STUDY TEAM MEMBERS

The following table lists the primary InFRM team members who participated in the development of the InFRM Watershed Hydrology Assessment for the Neches River Basin. Helena Mosser, a hydraulic engineer from USACE Fort Worth District, served as the team lead for this study. In addition to those listed, the InFRM team would also like to acknowledge the many others who served supervisory and support roles during this effort.

Table 1.1: Study Team Members

	Name	Agency	Office
1	Allen Avance, P.E.	USACE	RMC
2	Simeon Benson, P.E.	USACE	Fort Worth
3	Kristine Blickenstaff, P.E.	USGS	Fort Worth
4	Jerry Cotter, P.E.	USACE	Fort Worth
5	Landon Erickson, P.E.	USACE	Fort Worth
6	Heitem Ghanuni, P.E.	USACE	Fort Worth
7	Timothy Helms	USACE	Fort Worth
8	Bret Higginbotham, P.E.	USACE	Fort Worth

	Name	Agency	Office
9	Diane Howe	FEMA	Region 6
10	Kris Lander, P.E.	NWS	WGRFC
11	Craig Loftin, P.E.	USACE	Fort Worth
12	Helena Mosser, P.E.	USACE	Fort Worth
13	Stephen Pilney	USACE	Fort Worth
14	Max Strickler, CFM	USACE	Fort Worth
15	Jon Thomas	USGS	Fort Worth
16	Larry Voice	FEMA	Region 6
17	Sam Wallace	USGS	Fort Worth
18	Kara Watson	USGS	Fort Worth
19	Josh Willis	USACE	HEC

1.5 TECHNICAL REVIEW PROCESS

The InFRM Hydrology Assessments undergo a rigorous review process. Numerous peer reviews are performed by InFRM team members throughout the study. Each model, analysis, and technical product is peer reviewed as it is developed by an InFRM Subject Matter Expert (SME). Any technical issues that are discovered during the review process are thoroughly discussed and resolved, often with input from multiple team members. This same review process is also applied to the process of comparing the results from different methods. Any significant differences in the results are thoroughly investigated and discussed with multiple team members, which sometimes leads to changes in the assumptions of the analyses. After completing all the comparisons and investigations, the draft results are shared with the rest of the InFRM team, and input is solicited from multiple subject matter experts. The draft study recommendations are then documented in the draft report, which is sent out for peer review.

Representatives from the following entities were invited to participate as peer reviewers of the InFRM Watershed Hydrology Assessment of the Neches River basin: the Lower Neches Valley Authority (LNVA), the Angelina & Neches River Authority (ANRA), the Texas Water Development Board (TWDB), the Texas Department of Transportation (TxDOT), the General Land Office (GLO) of Texas, and the InFRM Academic Council. The InFRM Academic Council is comprised of a select group of professors from local universities with unique skillsets and regional expertise in water resources and hydrology. Their involvement provides an independent and unbiased review of the InFRM team's methods and results. Collaboration with the InFRM Academic Council also helps the InFRM team to stay abreast with the latest advances in hydrologic science and technology. The primary InFRM Academic Council reviewers for the Neches Watershed Hydrology Assessment include Dr. Nick Fang from the University of Texas at Arlington and Dr. Hatim Sharif from the University of Texas at San Antonio. The peer review comments that were received for this study and the responses from the InFRM team have been documented in Appendix H.

2 Neches River Basin

The Neches River basin was selected for study by FEMA based upon their NFIP mapping needs and the availability of existing models and LiDAR data. USACE already had sufficiently detailed modeling products available as a starting point for the Neches Watershed Hydrology Assessment from USACE's Corps Water Management System (CWMS) Implementation program. CWMS is the automated decision support tool developed by the Hydrologic Engineering Center (HEC) for USACE Water Managers. In 2013, USACE began a national implementation effort to have all watersheds containing USACE managed flood control systems (dams, levees, etc.) fully modeled within CWMS. The models that were developed for the national CWMS implementation included basin-wide models for surface water hydrology in HEC-HMS, reservoir operations in HEC-ResSim, river hydraulics in HEC-RAS, and economic flood damages in HEC-FIA. For the Neches River basin, CWMS implementation modeling was completed in 2015, and representatives of FEMA Region 6 attended the CWMS handoff meeting at the USACE Fort Worth District office. In addition, FEMA had new LiDAR data flown for the entire Neches River basin in 2016, and FEMA also had future floodplain mapping activities scheduled in the basin.

2.1 WATERSHED AND RIVER SYSTEM DESCRIPTION

The Neches River begins in Van Zandt County approximately 60 miles southeast of Dallas, Texas. It flows in a southeasterly direction for approximately 416 miles to empty into Sabine Lake, 20 miles southeast of Beaumont, Texas. The Neches River and its principal tributary, the Angelina River, rise in a region of rolling hills and flow through an area of moderately to extremely hilly relief to the vicinity of Jasper and Woodville, where the rolling terrain abruptly changes to the flat coastal prairie. The watershed of the Neches River has a total drainage area of 10,129 square miles. The main river system has two principal branches above the junction with the Angelina - the Neches River, with a length of about 290 miles, and the Angelina River, with a length of about 205 miles. The bed slope of the Neches River in the vicinity of Town Bluff Dam, which is located just downstream of the confluence of the Angelina River with the Neches River, is about 0.7 feet per mile. The Angelina River runs roughly parallel to the Neches River and enters it at mile 126.1. Above their confluence the Neches River has a drainage area of 3,819 square miles, and the Angelina River has a drainage area of 3,574 square miles. The drainage area between the confluence of the two rivers and the mouth is approximately 2,736 square miles. See Figure 2.1 for a location map of the Neches River basin.

The Angelina River is formed by the junction of Shawnee and Barnhart Creeks in southwestern Rusk County near Henderson, Texas. From there it flows in a general southeasterly direction to its confluence with the Neches River on the left bank of the Neches River at mile 126.1, near Jasper, Texas, the streambed elevation at the mouth being about 60 ft msl. Sam Rayburn Reservoir is located on the Angelina River near mile 25.2. The average slope of the Angelina River streambed is less than 0.5 foot per mile in the pine flats below Sam Rayburn Reservoir. The Angelina River has four main tributaries above the Sam Rayburn Dam. Striker Creek, a left bank tributary, enters at mile 178.0 and has a length of 33 miles. Mud Creek enters at mile 168.2 and has a length of 67 miles. Attoyac Bayou enters at mile 53.7 and has a length of 119 miles. Ayish Bayou enters just above the dam at mile 25.7 and has a length of 70 miles.

The Angelina River Watershed is located within the West Gulf Coast Plains Section of the Coastal Plains Physiographic Province. Its headwaters are in a region of sharply rolling timbered hills and flows through an area of moderately to extremely hilly relief to just above Sam Rayburn Reservoir where the terrain becomes more gently rolling but is heavily forested; thence to near the mouth where it enters the Texas Pine Flats in which the timber is sparser and there is little topographic relief.

Sam Rayburn Dam and Reservoir are located on the Angelina River at river mile 25.2 above its confluence with the Neches River and about 10 miles northwest of Jasper, Texas. The total drainage area above Sam Rayburn dam is 3,449 square miles, and deliberate impoundment began in March of 1965. Sam Rayburn Reservoir is owned and operated by the USACE Fort Worth District. It is a multi-purpose project, which includes 1.1 million acre-feet of storage for flood control. It is operated in conjunction with Town Bluff Dam to provide flood control to the Angelina and Neches River Basin system by controlling flows below Town Bluff Dam to not exceed 20,000 cfs, when possible. Sam Rayburn also provides water supply to the Lower Neches Valley Authority (LNVA) and the Beaumont and Lufkin municipal areas and electric generation to the regional power grid.

Town Bluff Dam and B.A. Steinhagen Lake are located on the Neches River at river mile 113.7 about 12.4 miles below the mouth of the Angelina River and approximately 0.5 miles north of Town Bluff, Texas. The lake straddles Jasper and Tyler Counties. The total drainage area above Town Bluff Dam is 7,573 square miles, which includes the entire Angelina drainage basin and 4,017 square miles of the drainage basin of the Neches River. Deliberate impoundment began in April of 1951. Town Bluff Dam and B.A. Steinhagen Lake are a multi-purpose project used for flood control, water supply, hydropower, navigation, fish and wildlife, and recreation. The purposes of Town Bluff Dam and B.A. Steinhagen Lake are to assist Sam Rayburn Reservoir in providing flood control to the Angelina and Neches River Basin system in Southeast Texas, re-regulate flows from Sam Rayburn Dam's hydropower generation, supply water to the Lower Neches Valley Authority (LNVA) and the Beaumont area, and produce a clean source of electric generation. However, unlike Sam Rayburn's large amount of flood control storage, Town Bluff Dam only includes 57,700 acre-feet of flood storage between its top of normal pool and its uncontrolled spillway crest.

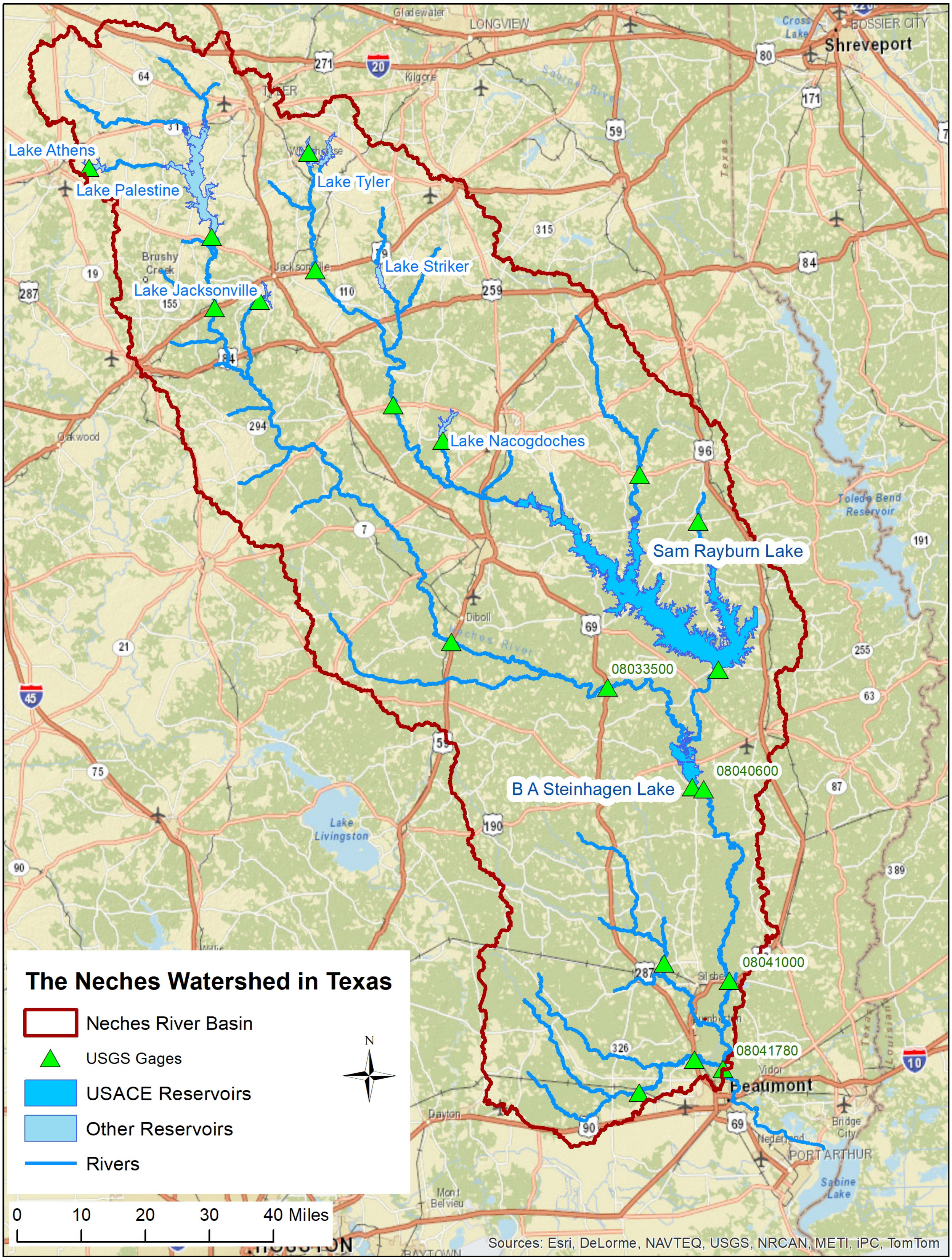


Figure 2.1: Neches River Basin Location

2.2 CLIMATE

The climate over the entire Neches River watershed is generally mild, and temperatures are moderate. Freezing temperatures and snowfall are rare in the coastal section of the basin but occur occasionally in the northern part of the watershed. The mean annual temperature is about 67 degrees Fahrenheit. January, the coldest month, has an average minimum daily temperature of about 38 degrees; August, the warmest month, has an average minimum daily temperature of about 93 degrees. Temperatures in the watershed have ranged from maximum of 112 degrees recorded at San Augustine to a minimum of minus 8 degrees recorded at Tyler. The prevailing winds over the watershed are from the south. However, during winter months, the influence of high-pressure system moving from the northwest causes the wind to shift from the north. Average annual precipitation over the Neches River Basin varies from 48 inches in the northernmost headwaters of the basin to 60 inches at the downstream end of the basin where the Neches River enters Sabine Lake, based on climatological data from 1981 - 2010 (TWDB, 2012). While the climate of the Neches River basin is generally mild, like most of Texas, it is also subject to a variety of extreme weather events, including hurricanes, tornadoes, droughts, heat waves, cold waves, and intense precipitation (NCEI, 2017).

2.3 MAJOR FLOODS IN THE NECHES RIVER BASIN

The Neches River Watershed is subject to three general types of flood-producing rainfall: thunderstorms, frontal rainfall, and tropical cyclones. Generally, the highest precipitation accumulations for the daily through monthly durations have occurred during tropical cyclones. However, there are some instances of heavy precipitation resulting from local thunderstorms. Because of the slow rate of runoff and the small conveyance capacity of the natural channel, it is not unusual for floods to prevail above the channel bank stage for several months. The period of most frequent flooding is during Hurricane Season (June to November). However, floods may occur at any time during the year.

The Neches River basin has a history of flooding that spans back to 1884, when the highest known flood stages were recorded on the Neches River at Diboll, Rockland, Town Bluff and Evadale, Texas. The following sections summarize information on some of the major floods in the Neches basin, including the May 1884, August 1915, May 1944, October 2006, and August 2017 floods on the Neches River and its tributaries. Other major floods at significant stream gages in the Neches River basin are listed in Table 2.1.

2.3.1 The Flood of May 1884

The flood of May 1884 produced the highest known stages on the Neches River near Diboll, near Rockland, at Evadale, and on the Angelina River near Lufkin. The stages for the flood of May 1884 have been established from identified flood marks. A comparison of the May 1884 stages with the stages of the other major floods indicates that the flood of May 1884 was probably the greatest flood that has occurred along the Neches River from the city of Reese to at least Evadale, Texas. The peak discharge for this flood was estimated to be 110,000 cfs near Diboll, 62,000 cfs near Rockland, 125,000 cfs at Evadale, and 130,000 cfs near Lufkin on the Angelina River.

2.3.2 The Flood of August 1915

This flood resulted from the storm of 16-21 August 1915. The storm was centered at San Augustine, where the total depth of rainfall was 19.8 inches over a 4-day period. The flood of August 1915 produced the highest known stages on Village Creek near Kountze until Hurricane Harvey and produced near-maximum stages in the Neches River below the mouth of the Angelina River. Flood marks indicate that the August 1915 flood stage at Evadale was about 1.70 feet lower than the May 1884 flood and had an estimated peak discharge of 102,000 cfs. The 1915 flood also resulted in the highest stage of record at the Neches River at Beaumont prior to Hurricane Harvey with a peak stage of 14.0 feet.

2.3.3 The Flood of May 1944

From 29 April to 5 May 1944, heavy rains occurred over the Neches River Basin, with 16.00, 15.91, and 12.00 inches of rain reported at Pollok, Jackson Hill, and Flint, respectively. The flood of May 1944 was the second highest flood of record near Rockland and the third highest at Evadale on the Neches River. The peak discharges of this flood at Rockland and Evadale were 49,800 and 92,100 cfs, respectively.

2.3.4 The Flood of October 2006

October 2006 was a significant flood event on the lower Neches basin. In October 2006, 15" to 18" of rainfall fell over a 3-day period, and 24-hour totals of 8" to 10" were recorded by the NWS radar data. This was the second highest flood event of record at 3 out of 5 stream gages in the lower Neches basin below Town Bluff Dam, including a peak discharge of 95,800 cfs at Beaumont.

2.3.5 The Flood of August 2017 - Hurricane Harvey

Hurricane Harvey (Aug-Sep 2017) was the flood of record for 4 out of 5 stream gages in the lower portion of the Neches basin below Town Bluff Dam. The NWS recorded 26" to 40" of rainfall over the 5-day period of Aug 26-30, 2017 in the lower portions of the Neches River basin with higher amounts towards the coast. At the Saltwater Barrier on the Neches River near Beaumont, TX, the peak flood stage from Hurricane Harvey exceeded the previous flood of record by more than 10 feet and had an estimated peak discharge of 232,000 cfs.

Table 2.1: Major Floods in the Neches River Basin

Date of Flood	Observed Peak Flow (cfs)			
	Neches River nr Rockland, TX	Neches River nr Town Bluff, TX	Neches River at Evadale, TX	Neches River Saltwater Barrier at Beaumont, TX
	USGS 08033500	USGS 08040600	USGS 08041000	USGS 08041780
	3,633 sq mi	7,569 sq mi	7,895 sq mi	9,859 sq mi
May 1884	62,000	120,000	125,000	-
Aug 1915	-	-	102,000	-
Apr 1922	45,200	-	71,500	-
Jun 1929	34,200	-	83,800	-
Feb 1932	33,600	-	73,400	-
May 1935	39,700	-	64,100	-
Nov 1940	25,900	-	59,600	-
May 1944	49,800	-	92,100	-
May 1953	34,400	90,900	80,300	-
May 1957	29,700	62,000	55,300	-
May 1969	34,000	30,500	31,300	-
Jul 1989	42,000	49,200	47,900	-
Oct 1994	42,300	35,400	41,400	-
Oct 2006	18,600	26,900	33,800	95,800
May 2015	28,700	28,000	31,200	52,700
May-Jun 2016	29,600	29,300	39,700	76,100
Aug-Sep 2017	37,900	91,000	71,800	232,000
May 2021	23,000	52,000	41,600	78,300

2.4 PREVIOUS STUDIES AND CURRENTLY EFFECTIVE FEMA FLOWS

The large majority of the Neches River basin is currently mapped with approximate “Zone A” designations on the FEMA Flood Insurance Rate Maps (FIRMs), meaning that the hydrology for these portions of the basin has never been studied in detail. However, data and models from several existing hydrologic and hydraulic studies were available at the time of this study. Some of these studies used approximate methods, while others used detailed methods for limited portions of the basin. Table 2.2 below summarizes the most notable existing studies, models, and hydrologic information that were previously performed in the Neches River basin. From this table, one can see that most of the frequency flow estimates in the basin that were calculated with detailed methods have not been updated since the 1980s or early 1990s, including the hydrology behind the effective FEMA Flood Insurance Studies for Jefferson and Hardin Counties.

Table 2.2: Previous Hydrologic Studies in the Neches River Basin

Study Name	River Extents	Frequency Flows	Hydrologic Methods	Description
Base Level Engineering (BLE) Analysis, 2019	Neches River Basin	Yes	Regression equations, Statistical hydrology	Approximate 1D HEC-RAS models for the entire Neches River basin with approximate hydrology
Sam Rayburn Dam and Reservoir Water Control Manual, 2018	Angelina River	No	Rainfall-runoff modeling	Original spillway design flood hydrology from the 1947 Project report.
Town Bluff Dam and B.A. Steinhagen Lake Water Control Manual, 2016	Neches and Angelina Rivers	No	Rainfall-runoff modeling	Original spillway design flood hydrology from the 1947 Project report.
Neches CWMS Implementation Forecast Models, 2015	Neches River Basin	No	Rainfall-runoff modeling	USACE reservoir forecast models and calibrated rainfall runoff models developed for the entire Neches River Basin.
Jefferson County Preliminary Flood Insurance Study (FIS), 2011	Neches River at Beaumont, Pine Island Bayou	Yes	Rainfall-runoff modeling	HEC-1 rainfall runoff modeling from a 1985 USACE Feasibility study.
Hardin County Flood Insurance Study (FIS), 2010	Pine Island Bayou, Village Creek,	Yes	Statistical hydrology, and Rainfall-runoff modeling	HEC-1 Rainfall runoff modeling and gage statistical analyses were last updated in 1992.
Jefferson County Flood Insurance Study (FIS), 2002	Neches River at Beaumont, Pine Island Bayou	Yes	Rainfall-runoff modeling	1980 Rainfall runoff modeling from original 1982 FIS.
Sam Rayburn Dam and Reservoir-Dam Safety Assurance Study-Spillway Modification and Freeboard Restoration, 1992	Angelina River	No	Rainfall-runoff modeling	Probable Maximum Flood based on HMR51 and HMR52, used for modification of the spillway.
Design Memorandum No. 15 on McGee Bend Reservoir, Hydrology (Revised), 1958	Angelina River	No	Rainfall-runoff modeling	Spillway design flood based on the maximum storms observed in the area as of 1944.
USACE Definite Project Report for Dam “B”, Rockland and Dam “A” Reservoirs, Neches and Angelina Rivers, Volume 3, 1947	Neches and Angelina Rivers	No	Rainfall-runoff modeling	Spillway design flood based on the maximum storms observed in the area as of 1947.

2.5 THE EFFECTS OF FUTURE CONDITIONS

Future conditions can impact the hydrology of a given watershed due to changes in both land use and climate. For the Neches River Basin, which does not contain a major metropolitan area, future land use conditions are not expected to change substantially for the foreseeable future. Therefore, future land use change is not expected to cause significant changes to the hydrology of the Neches River Basin.

Future climate change, on the other hand, is expected to increase the intensity and frequency of storms in Texas and in the Neches River basin. According to NOAA's National Centers for Environmental Information (NCEI), mean annual temperatures in Texas have increased by approximately 1 degree Fahrenheit since the first half of the 20th century, and additional warming is expected by the end of the 21st century. Higher temperatures will increase soil moisture loss during dry spells, increasing the intensity of naturally occurring droughts (NCEI, 2017).

Over the past 50 years, significant flooding and rainfall events have followed drought for approximately one-third of the drought-affected periods in this region. Understanding this rapid swing from extreme drought to flood is an important and ongoing area of research, and climate change is likely to exacerbate the extremes of both drought and flood in Texas (Kloesel, 2018).

Increases in intense rainfall and extreme precipitation events are also likely under future conditions. According to the Fourth National Climate Assessment, the frequency and intensity of heavy precipitation in Texas are anticipated to continue to increase. The expected increase of precipitation intensity implies fewer soaking rains and more time to dry out between events. In addition, rare events such as 1% AEP (100-year) floods are likely to become more common (Kloesel, 2018). Differing climate projections indicate a 10% to 20% increase in 20% AEP (5-year) rainfall intensity for the State of Texas (Kunkel, 2020). In addition, research is showing that the rarer events, such as the 1% AEP, will tend to have larger increases in rainfall intensity than the more frequent events, such as the 20% AEP, regardless of duration (Kunkel, 2020).

Some of the most extreme flood events that have occurred in the Neches River Basin were due to hurricanes and tropical storms. As the climate warms, hurricane rainfall rates, storm surge height due to sea level rise, and the intensity of the strongest hurricanes are also projected to increase (NCEI, 2017).

While most climate scientists agree on general increasing trends in the intensity of hurricanes and other extreme rainfall events due to a warming climate, additional research is needed to quantify the effects of these changes on flood frequency and severity. The InFRM team is currently waiting on additional guidance from the climatological scientific community in order to quantify the effects of future climate change on the hydrology of Texas and the Neches River basin. A quantitative assessment of future climate conditions may be added as an addendum to this report when the appropriate science is available to support it.

3 Methodology

Assessing flood potential within complex river basins requires considerable expertise and experience. The methodology that was used for this watershed hydrology assessment was a multi-layered analysis that calculated frequency flows in the Neches River Basin through several different methods and compared their results to one another before making final flow recommendations. The purpose of this analysis was to produce a set of frequency flows that are consistent and defendable across the basin.

The current study builds upon the information that was available from previous hydrology studies by combining detailed data from different models, updating land use data, calibrating the models to multiple recent flood events, and updating statistical analyses to include the most recent flood events.

The multi-layered analysis for the current study of the basin consists of four main components: (1) statistical analysis of the stream gages, (2) rainfall-runoff watershed modeling in the Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS), (3) extended period-of-record modeling in RiverWare, and (4) reservoir analyses. Details on the methodology of each analysis are included in their respective report chapters and appendices.

After completing all of these different types of analyses, the final recommendations for the InFRM Watershed Hydrology Assessment were then formulated through a rigorous process which required technical feedback and collaboration between all of the InFRM subject matter experts. This process included the following steps at a minimum: (1) comparing the results of the various hydrologic methods to one another, (2) performing an investigation into the reasons for the differences in results at each location in the watershed, (3) selecting the draft recommended methods, (4) performing internal and external technical reviews of the hydrologic analyses and the draft recommendations, and finally, (5) finalizing the study recommendations. The comparisons of results are included in Chapter 11, and additional details on the process of selecting draft recommendations and finalizing the results can be found in Chapter 12.

4 Data Sources

This chapter provides a general summary of the data that was collected, reviewed, or utilized in the InFRM Watershed Hydrology Assessment of the Neches River Basin, including geospatial and climatic information, field observations and previous reports. A more complete list of the data sources used in each type of analysis is included in their respective appendices.

4.1 SPATIAL TOOLS AND REFERENCE

ArcGIS version 10.2.2 (developed by ESRI), together with HEC-GeoHMS version 10.2 were used to process and analyze the data necessary for hydrologic modeling and to generate the sub-basin boundaries. The geographic projection parameters used for this study are listed below:

- Horizontal Datum: North American Datum 1983 (NAD83)
- Projection: USA Contiguous Albers Equal Area Conic USGS version
- Vertical Datum: North American Vertical Datum, 1988 (NAVD 88)
- Linear Units: U.S. feet

4.2 DIGITAL ELEVATION MODEL (DEM)

As part of USACE's Corp Water Management System (CWMS) implementation for the Neches River basin, 30-meter DEMs were collected from the seamless USGS National Elevation Dataset (NED, accessed January 2013) for the study watershed from the http://seamless.usgs.gov website. The elevations of the NED are in meters. The vertical elevation units were converted from meters to feet, and the datasets were projected into the standard map projection used in this study. The watershed and subbasin delineations for the Neches HEC-HMS model were performed using the 30-meter NED data.

In addition, high resolution 2016 LiDAR data was available for the entire basin. The LiDAR data was used in this study to develop new 1D and 2D HEC-RAS routing data for the Neches HEC-HMS model and to perform a 2D rain-on-grid analysis for the upper Angelina River watershed. This LiDAR data was collected in the form of high resolution, 1-meter LiDAR digital elevation model (DEM) tiles which were downloaded directly from the U.S. Geological Survey (USGS) 3DEP LidarExplorer website (USGS, 2018). Specific LiDAR projects that were used for this include the TX FEMA R6 Neches Basin LiDAR 2016 D16. The 1-meter DEM data led to cumbersome file output sizes when attempting to mosaic together the entire 10,000 square miles of the Neches River basin. Therefore, the data was resampled into a 10-foot cell size before being mosaiced together into a single DEM for the Neches River basin. The projected coordinate system used for the final DEM was USA Contiguous Albers Equal Area Conic USGS version in feet.

4.3 VECTOR AND RASTER GEOSPATIAL DATA

The mapping team member utilized web mapping services and downloaded the USGS hydrologic unit boundaries, USGS stream gages, USGS medium resolution National Hydrography Dataset (NHD), National Inventory of Dams (NID) data, National Levee Database (NLD) levee centerlines as well as general base map layers. Additional vector data were obtained from the ESRI database and used in figures prepared for the final report. Raster Data includes the National Land Cover Database (NLCD) 2011 and 2016 land cover layers and percent imperviousness layers from the https://seamless.usgs.gov website, accessed October 2016 and August 2018.

4.4 AERIAL IMAGES

The Neches CWMS implementation team utilized current high-resolution imagery from the National Aerial Imagery Program (NAIP) with a horizontal accuracy based upon National Map Accuracy Standards (NMAS), with 1"=200' scale (1-foot imagery) accuracy of +/- 5.0-feet and the 1"=100' scale (0.5-foot imagery) accuracy of +/- 2.5-feet. Digital photos were used to verify watershed boundaries as well as delineate centerlines and other geographic features. In addition, Google Earth and Bing Maps were also used to locate important geographic features.

4.5 SOIL DATA

Gridded Soil Survey Geographic (SSURGO) datasets were obtained from the NRCS soil survey website during the Neches CWMS implementation (NRCS, 2014). These datasets were used to estimate initial and constant loss rates for the frequency storm events in HEC-HMS and to calculate initial estimates of the Snyder's lag time. The lag times were modified during calibration.

4.6 PRECIPITATION DATA

4.6.1 Radar Data for Observed Storms

Historic precipitation data for observed storm events were collected from the NWS gridded precipitation data files. NEXRAD Stage IV grids were used for the basin. The NEXRAD Stage IV grids are stored in a binary file format called XMRG. The historical XMRG data were processed into hourly precipitation grids in HEC-DSS format using HEC-METVUE. This data was acquired from the NWS West Gulf River Forecasting Center (WGRFC). The radar rainfall data has the spatial resolution of approximately a 4 km x 4 km grid, and the rainfall depths are calibrated by the NWS to on-the-ground observations at rainfall gages.

4.6.2 NOAA Atlas 14 Frequency Point Rainfall Depths

Frequency point rainfall depths of various durations and recurrence intervals were collected from NOAA Atlas 14. NOAA Atlas 14 contains precipitation frequency estimates for the United States along with their associated lower and upper 90% confidence bounds. The Atlas is divided into volumes based on geographic sections of the country. NOAA Atlas 14 is intended as the U.S. Government source of precipitation frequency estimates. NOAA Atlas 14 Volume 11, which covers the state of Texas, was recently published in September of 2018 (NOAA, 2018). The new rainfall depths that were published in NOAA Atlas 14 (NA14) were applied to the HEC-HMS model for this study, as they are the most up-to-date precipitation frequency estimates in Texas. NOAA Atlas 14 point rainfall depths from the annual maximum series for various durations and recurrence intervals were collected from the NA14 Precipitation Frequency Data Server (PFDS) for the centroid of each HEC-HMS subbasin (NOAA, 2020).

4.7 STREAM FLOW AND STAGE DATA

The USGS stream flow and reservoir pool elevation gages located in the basin are listed in Table 4.1. Table 4.1 also indicates whether the gage record was used in this study's statistical analysis or in the calibration of the HEC-HMS model. For these gage sites, annual peak flow data and 15-minute stream flow and stage data was collected from the USGS National Water Information System (NWIS) database (USGS, 2018).

Table 4.1: USGS Stream Flow and Reservoir Pool Elevation Gages in the Neches River Basin

	USGS ID	Gage Name	Drainage Area (sq mi)	Data Type	Used in HEC-HMS Model Calibration	Used for Statistical Analysis
1	08031290	Lake Athens nr Athens, TX	22	Pool Elevation	Yes	
2	08031400	Lake Palestine nr Frankston, TX	838	Pool Elevation	Yes	
3	08032000	Neches River nr Neches, TX	1,146	Stream Flow	Yes	Yes
4	08032200	Lake Jacksonville nr Jacksonville, TX	40	Pool Elevation	Yes	
5	08032500	Neches River nr Alto, TX*	1,943	Stream Flow		Yes
6	08033000	Neches River nr Diboll, TX	2,726	Stream Flow	Yes	Yes
7	08033500	Neches River nr Rockland, TX	3,633	Stream Flow	Yes	Yes
8	08034000	Lake Tyler nr Whitehouse, TX	113	Pool Elevation	Yes	
9	08034500	Mud Creek nr Jacksonville, TX	377	Stream Flow	Yes	Yes
10	08036500	Angelina River nr Alto, TX	1,286	Stream Flow	Yes	Yes
11	08036700	Lake Nacogdoches nr Nacogdoches, TX	89	Pool Elevation	Yes	
12	08037000	Angelina River nr Lufkin, TX*	1,622	Stream Flow		Yes
13	08038000	Attoyac Bayou nr Chireno, TX	503	Stream Flow	Yes	Yes
14	08039100	Ayish Bayou nr San Augustine, TX	89	Stream Flow	Yes	Yes
15	08039300	Sam Rayburn Reservoir nr Jasper, TX	3,452	Pool Elevation	Yes	Yes
16	08040000	B.A. Steinhagen Lake at Town Bluff, TX	7,569	Pool Elevation	Yes	Yes
17	08040600	Neches River nr Town Bluff, TX	7,574	Stream Flow		Yes
18	08041000	Neches River at Evadale, TX	7,895	Stream Flow	Yes	Yes
19	08041500	Village Creek nr Kountze, TX	861	Stream Flow	Yes	Yes
20	08041700	Pine Island Bayou nr Sour Lake, TX	398	Stream Flow	Yes	Yes
21	08041749	Pine Island Bayou above BI Pump Plant, Beaumont, TX	698	Stream Flow	Yes	Yes
22	08041780	Neches River at the Saltwater Barrier at Beaumont, TX	9,859	Stream Flow	Yes	Yes

* Former USGS Gage, currently inactive.

4.8 RESERVOIR PHYSICAL DATA

According to the National Inventory of Dams (NID), approximately 300 dams exist within Neches River basin, most of which are NRCS structures or other small dams (USACE, 2016). Of these, eight dams were selected to be modeled in detail in the HEC-HMS rainfall-runoff model. These dams were selected to be modeled in detail due to their sizable flood storage and their noticeable influence on discharges in the major rivers downstream. Table 4.2 summarizes the reservoir data obtained for these dams and their corresponding data sources.

The eight modeled reservoirs include the two USACE reservoirs, Sam Rayburn and B.A. Steinhagen. For these reservoirs, the elevation-storage tables, spillway rating curves, and outlet structure rating curves were all obtained from the USACE Fort Worth District, and the dams were modeled as reservoir elements in HEC-HMS.

The remaining 300 smaller dams were scattered throughout the rural areas of the basin, especially in the headwaters of the Neches and Angelina watersheds. These dams were not modeled in detail but were accounted for in the model through adjustments to the subbasins' initial losses and peaking coefficients. Data for these dams was obtained from the National Inventory of Dams (USACE, 2016).

Table 4.2: Reservoir Data and Sources for Dams Modeled in Detail

Reservoir Name	Drainage Area (sq mi)	Normal Storage (ac-ft)	Data	Source(s)
Lake Athens	21.6	29,475	Elevation-Storage, Elevation-Discharge rating	Texas Water Development Board, Dam Design Documentation
Lake Palestine	838.1	367,312	Elevation-Storage, Elevation-Discharge rating	Texas Water Development Board, Upper Neches River Municipal Water Authority
Lake Jacksonville	39.6	25,732	Elevation-Storage, Elevation-Discharge rating	Texas Water Development Board, 1978 Texas Dam Inspection Report
Lake Striker	182.0	22,865	Elevation-Storage, Elevation-Discharge rating	Texas Water Development Board, 2009 Standard Operating Procedure for Striker Floodgate Operations
Lake Tyler	113.3	77,378	Elevation-Storage, Elevation-Discharge rating	Texas Water Development Board, 1978 Texas Dam Inspection Reports
Lake Nacogdoches	89.0	39,523	Elevation-Storage, Elevation-Discharge rating	Texas Water Development Board, Texas Commission on Environmental Quality
Sam Rayburn Reservoir	3451.8	2,862,335	Elevation-Storage capacity, Spillway and Outlet Structures	USACE - Fort Worth District
B.A. Steinhagen Lake (Town Bluff Dam)	7569.3	66,973	Elevation-Storage capacity, Spillway and Outlet Structures	USACE - Fort Worth District

4.9 SOFTWARE

The following table provides a summary of the significant computer software programs and versions that were used in in this study for the hydrologic analyses of the Neches River basin.

Table 4.3: Summary of Software Used in the Watershed Hydrology Assessment

Program	Version	Capability	Developer
ArcGIS	10.2.2	Geographical Information System	ESRI
HEC-DSSVue	2.0.1	Plot, tabulate, edit and manipulate data in HEC-DSS format	HEC
HEC-GeoHMS	10.2	Watershed delineation and generating HEC-HMS input	HEC
HEC-METVUE	3.0	Processing and viewing precipitation data	HEC
HEC-HMS	4.3 and 4.4.1	Rainfall-Runoff Simulation	HEC
HEC-RAS	5.0.7	1D and 2D Hydraulic Routing	HEC
HEC-SSP	2.1.00.137	Statistical Software Package	HEC
RiverWare	7.4	River and Reservoir Simulation	CADSWES
RMC-RFA	1.0.0	Reservoir Frequency Analysis	RMC
PeakFQ	7.1	Statistical Analysis of Gage Records for Flood Frequency	USGS

5 Statistical Hydrology

Statistical analysis of the observational record from U.S. Geological Survey (USGS) streamgaging stations and other historical information provides an informative means of estimating flood flow frequency. Flood flow frequency is defined by values or quantiles of discharge for selected annual exceedance probabilities (AEPs) (England and others, 2018). The annual peak discharge data as part of systematic operation of a streamgaging station provides the foundation for a detailed analysis of peak discharge, but additional historical information pertaining to peak discharges also can be used. An annual peak discharge is defined as the maximum instantaneous discharge for a streamgaging station for a given water year, and annual peak discharge data for USGS streamgaging stations can be acquired through the USGS National Water Information System (NWIS) database (USGS, 2018). The statistical analyses are based on water-year increments. A water year is the 12-month period from October 1 of a given year through September 30 of the following year designated by the calendar year in which it ends.

For the statistical hydrology portion of the multi-layered analysis, InFRM team members from the USGS analyzed annual peak discharge records for the 15 USGS streamgaging stations (gages) shown on Figure 5.1. Information on the period of record data for those USGS gages are listed in Table 5.1.

This chapter provides a general summary of the data, analyses and results of the statistical analyses of the stream gage records that were completed for the InFRM Watershed Hydrology Assessment of the Neches River Basin. Additional details on the statistical analyses are available in Appendix A: Statistical Hydrology.

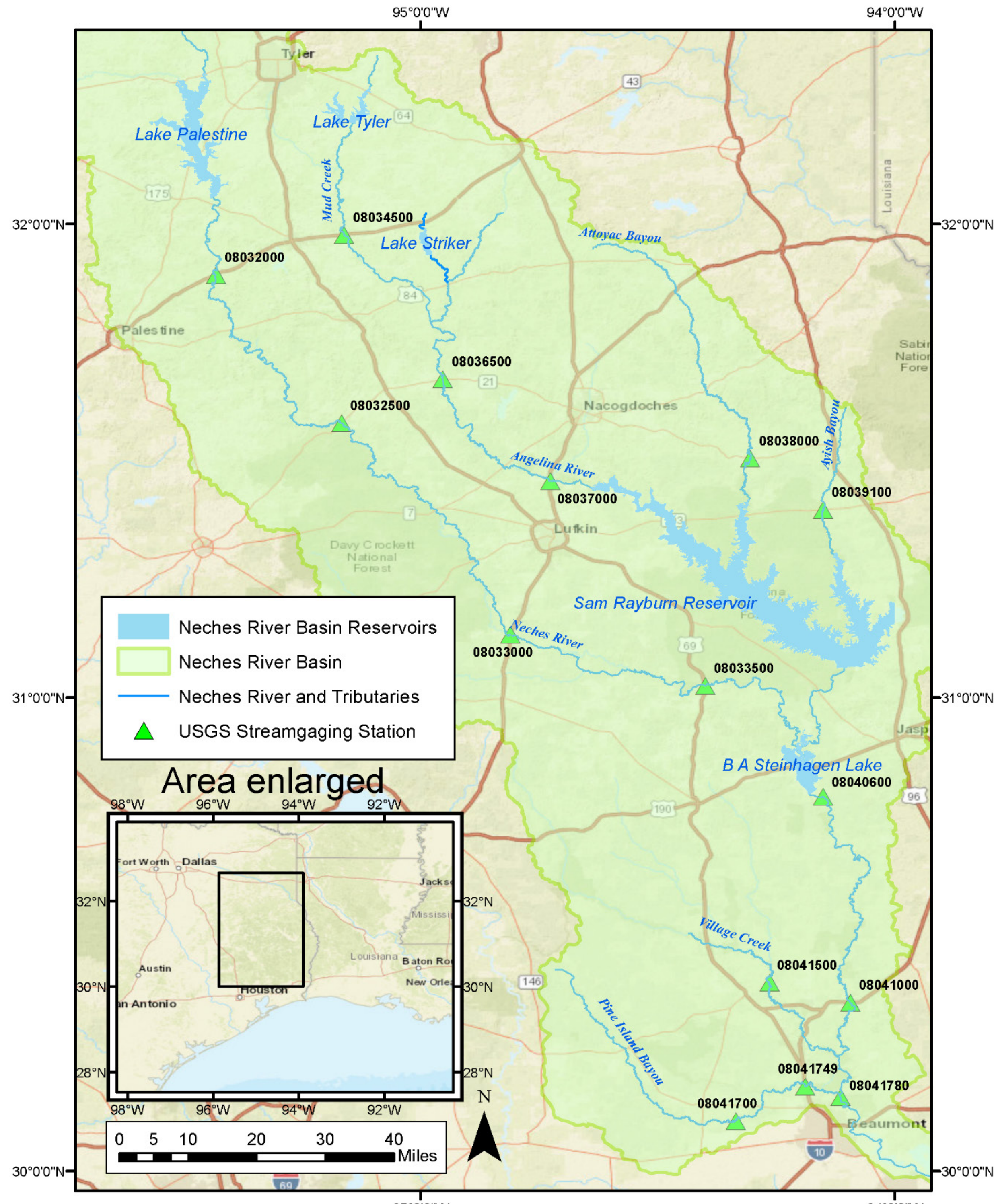


Figure 5.1: Map of U.S. Geological Survey (USGS) Streamgaging Stations included in the Statistical Analysis

Table 5.1: Summary of the Fifteen U.S. Geological Survey Streamgaging Stations in the Neches River Basin Study Area, Texas with Ancillary Information Concerning Statistical Analyses

Station number	Streamgage name or reservoir station name	Latitude	Longitude	Period of analyzed annual peak streamflows	Contri-buting drainage area	Main channel slope (Asquith and Slade, 1997)	Mean annual rainfall (PRISM 1981–2010)	Regional skew (*If applicable*; Judd and others, 1996)	Kendall's Tau of analyzed annual peak streamflows	Kendall's Tau *p*-value of analyzed annual peak streamflows
					(mi^2)	(--)	(in.)	(--)	(--)	(--)
08032000	Neches River near Neches, Tex.	31.8924	95.4308	1962–2017(R)	1,145	0.0006	46.2	--	0.038	0.687
08032500	Neches River near Alto, Tex.	31.5795	95.1660	1884–1978(U/R)	1,945	0.0004	47.5	--	-0.165	0.161
08033000	Neches River near Diboll, Tex.	31.1330	94.8099	1884–2017(U/R)	2,724	0.0004	51.9	--	0.054	0.482
08033500	Neches River near Rockland, Tex.	31.0250	94.3994	1884–2017(U/R)	3,636	0.0003	54.0	--	0.030	0.644
08034500	Mud Creek near Jacksonville, Tex.	31.9766	95.1608	1939–2017(U/R)	376	0.0010	47.1	--	-0.115	0.208
08034500	Mud Creek near Jacksonville, Tex. (*Alternative Analysis*)	31.9766	95.1608	1939–2017(U/R)	376	0.0010	47.1	--	-0.086	0.262
08036500	Angelina River near Alto, Tex.	31.6718	94.9527	1943–2017(U/R)	1,276	--	48.0	--	0.056	0.532
08037000	Angelina River near Lufkin, Tex.	31.4574	94.7263	1924–1979(U/R)	1,600	0.0005	50.1	--	-0.125	0.197
08038000	Attoyac Bayou near Chireno, Tex.	31.5044	94.3044	1902–2017(U)	503	0.0008	54.6	--	0.012	0.879
08039100	Ayish Bayou near San Augustine, Tex.	31.3963	94.1510	1958–2017(U)	89	0.0019	53.8	--	0.072	0.418
08040600	Neches River near Town Bluff, Tex.	30.7910	94.1510	1965–2017(R)	7,574	--	56.3	--	0.163	0.087
08040600	Neches River near Town Bluff, Tex. *(Alternative Analysis)*	30.7910	94.1510	1965–2017(U/R)	7,574	--	56.3	0.089	0.203	0.028
08041000	Neches River at Evadale, Tex.	30.3558	94.0932	1965–2017(R)	7,951	0.0003	57.7	0.300	0.157	0.098
08041000	Neches River at Evadale, Tex. (*Alternative Analysis*)	30.3558	94.0932	1965–2020(U/R)	7,951	0.0003	57.7	0.300	0.196	0.033
08041500	Village Creek near Kountze, Tex.	30.3980	94.2635	1924–2017(U)	860	0.0009	58.4	--	0.058	0.447
08041500	Village Creek near Kountze, Tex. *(Alternative Analysis)*	30.3980	94.2635	1924–2020(U)	860	0.0009	58.4	0.225	0.055	0.461
08041700	Pine Island Bayou near Sour Lake, Tex.	30.1061	94.3346	1968–2017(U)	336	0.0004	58.3	--	-0.006	0.960
08041749	Pine Island Bayou above BI Pump Plant, Beaumont, Tex.	30.1788	94.1888	2005–2017(U)	633	--	59.4	0.299	0.154	0.502
08041780	Neches River Saltwater Barrier at Beaumont, Tex.	30.1569	94.1144	2004–2017(R)	9,789	--	59.9	0.300	0.205	0.360

[mi^2, square miles; --, dimensionless or not applicable; in., inches; PRISM, data product of the Northwest Alliance for Computational Science and Engineering (2015, accessed on July 30, 2018 at *http://www.prism.oregonstate.edu/explorer/*); (U), unregulated annual peak streamflow; (R), regulated annual peak streamflow]

5.1 STATISTICAL METHODS

The statistical methods in this chapter describe the fitting of a log-Pearson type III probability distribution (LPIII) to the annual peak discharge data for the Neches River Basin. The general purpose of fitting a probability distribution is to provide an objective mechanism to extrapolate to hazard levels (as represented by AEPs and equivalently expressed as annual recurrence interval or recurrence interval measured in years) beyond those represented by the sample size of annual peak discharge data for a given streamgaging station. The LPIII distribution was fit to the logarithms (base-10) of the annual peak discharge data. The USGS-PeakFQ software version 7.1 (Veilleux and others, 2013; USGS, 2014) provides the foundation for the results of the flood flow frequency estimates that are specified by average annual recurrence intervals computed and extracted from software output at 2-, 5-, 10-, 25-, 50-, 100-, 200-, and 500-year recurrence intervals or respective AEPs of 0.500, 0.200, 0.100, 0.040, 0.020, 0.010, 0.005, and 0.002 along with the accompanying 95-percent confidence limits.

A complementary statistical technique used for data evaluation included the Kendall's Tau (correlation) test. The Kendall's Tau test (Hollander and Wolfe, 1973; Helsel and Hirsch, 2002) was used through the USGS-PeakFQ software to detect for the presence of trends in the annual peak discharge data. Kendall's Tau test is a popular statistic for quantifying the presence of monotonic changes in the central tendency of discharge data in time. The p-values of the Kendall Tau results are listed in Table 5.1, and only two of the gages showed a significant trend in annual peak discharge (p-value of 0.10 or less). These values are discussed further in the next section with the flood flow frequency results for each streamgage and in Appendix A.

Flood flow frequency analyses were made for the streamgaging stations using the annual peak data from the USGS NWIS database (USGS, 2018) with historical information when available and data augmentation when required. The Interagency Advisory Committee on Water Data (IACWD, 1982) describes the Bulletin 17B method (B17B) to conduct the frequency analysis (USGS, 2014), but the statistical frequency analysis performed for the streamgages in the Neches River Basin uses the updated guidelines from the Bulletin 17C (England and others, 2018). In particular, the usTae of the expected moments algorithm (EMA) was used for this study (England and others, 2018; USGS, 2014).

EMA enables sophisticated interpretations of the historical record intended to enhance the estimates of peak discharge, especially for the rare frequency events such as the 100-year discharge (AEP of 0.010). When available, inclusion of historical record interpretations can have the net effect of lowering (decreasing) flood flow frequency estimates for the largest of discharges because the largest documented events are assigned lower empirical probabilities. EMA also permits inclusion of nonstandard information such as data censoring. For example, an annual peak might be known to be lower than a specified discharge threshold. EMA can also accommodate time varying discharge thresholds based on assigning a discharge threshold as a 'highest since' within discrete blocks/intervals of time. This nonstandard information collectively can be thought of as a framework fostering record extension. Not all streamgaging stations have nonstandard information, but the use of EMA is preferred because confidence limits and associated standard errors of sampling for the flood quantiles are mathematically correct.

Two especially important options of the USGS-PeakFQ software are the choice of low-outlier threshold and regional skew coefficient (also known as the generalized skew coefficient) and whether to incorporate such skew in the analyses in a weighting between the regional skew and that computed using the site-specific data. Low outliers within a time series of peak discharge, such as annual peak discharges that in reality were likely not storm flows or highly localized storm flow, often need removal from the analysis using a form of conditional probability adjustment. To this end, the Multiple Grubbs-Beck low-outlier threshold (MGBT) was used. For streamgaging station-specific reasons, the analyst can manually specify a low-outlier threshold. Low-outlier threshold values for each streamgaging station are identified in Section 5.2.

Although the ultimate decision for specifying a low-outlier threshold to remove potentially influential low floods is based on engineering judgement, Bulletin 17C provides some general guidelines for choosing an appropriate threshold (England et al., 2018). For each flood frequency analysis, the computed curve is evaluated for its fit to the data. If the data appear to have a clear inflection point or shift in the ordered peaks that the MGBT did not identify, then the low outlier may be adjusted. Furthermore, if there are any low-flow peaks that are clearly non-flood years that were not caught by the MGBT because they did not influence the fitting of the frequency curve, then those peaks may be removed anyways because they are not considered a part of the record of floods at the location. Additionally, a brief sensitivity analysis is performed at all sites to determine the effects of the MGBT choice of low-outlier threshold on the flood frequency curve. Can the low-outlier threshold be adjusted to improve the station skew? Can the low-outlier threshold be adjusted to bring the estimates more in-line with upstream and downstream gages as one would expect at the analyzed location? These factors and more are considered for the MGBT estimate for each and every gaged location analyzed.

Skew is an expression of the curvature or shape of the LPIII distribution intended to mimic that of the data (Asquith, 2011a, 2011b). The importance of a regional skew is stressed in IACWD (1982) to mitigate for high sampling variance by using typical streamgaging station record lengths. A substantial motivation for a regional skew is to compensate for inefficient estimation of the product moment skew for highly variable and skewed data such as annual peak discharge. The generalized skew coefficient is a built-in feature of the USGS-PeakFQ software but can be overridden by the user. Because of age as well as study objectives for the present (2018) study, the maps of regional skew for Texas in IACWD (1982) or Judd and others (1996) are of uncertain applicability for this study. The former reference represents a highly generalized estimate of skew dating from about the late 1970s, the later reference represents a substantially more recent, but still dated, estimate of regional skew for Texas. However, because two streamgages with short periods of record are present in this study (USGS station 08041749, Pine Island Bayou above BI Pump Plant, Beaumont, Tex., and USGS station 08041780, Neches River Saltwater Barrier, at Beaumont, Tex.) it was decided to weight the PeakFQ computed skew with the regional skew values from Judd and others (1996) of the nearest streamgages with similar characteristics. In order to use the regional skew values, the weighted-skew option in USGS-PeakFQ software was required in conjunction with manual entry of skew information (USGS, 2014). Additionally, the Neches River at Evadale analysis was weighted by a regional skew value because the PeakFQ software was unable to produce an acceptable fit based on the station skew value. The Judd and others (1996) regional skew values used are listed in Table 5.1. The remaining 13 streamgages had a period of record deemed long enough to use the station skew computed by PeakFQ.

As with the MGBT and low-outlier threshold, some brief sensitivity analyses were performed at sites where station skew deviates considerably from published regional skew values, where the calculated flood frequency curve does not appear to fit the ordered peak floods well, or where the calculated flood frequency curve produced estimates that were not in line with estimates at upstream and downstream gages. Otherwise, preference was given to utilizing the station skew at each streamgage given the assumptions involved in applying a regional skew value to a unique location. Although a calculated station skew that differs greatly from the regional skew estimate is cause for further investigation, it is not necessarily justification for weighting by the regional skew value. This is because a gaged location may have watershed characteristics that differ from the greater regionalized hydrologic characteristics.

Confidence limits of flood flow frequency can be informative to decision makers. The lower and upper limits of 95-percent confidence intervals were computed for this study. Confidence intervals can be expected to encompass the true value 95 percent of the time (Good and Hardin, 2006, p. 101). The range in these numbers for the lower and upper 95-percent confidence limits increases with the more extreme events.

5.2 STREAM GAGE DATA AND STATISTICAL FLOW FREQUENCY RESULTS

This section provides a summary of available stream gage data and graphical flow frequency results for five example stream gages in the Neches River basin along with a summary of results for all gages in Table 5.2. A full description of the stream gage data and flow frequency results for all analyzed gages in the basin can be found in Appendix A.

08032000 Neches River near Neches, Tex.

The period of record at USGS streamgage station 08032000 Neches River near Neches, Tex. (hereinafter referred to as the "Neches River near Neches gage") was from 1939 through 2017. Starting with the 1962 water year, the record is flagged as influenced by regulation in the USGS NWIS database (USGS peak code 6; USGS, 2018). To maintain a homogenous record, peak discharges recorded from 1939 through 1961 prior to the completion of Palestine Lake were not used in the analysis.

The largest peak in the record for the location is the 1968 peak discharge of 26,900 cubic feet per second (cfs) at a stage of 19.46 feet (ft). The data as set up for statistical frequency analysis are shown in a log-normal plot of annual peak discharge versus water year in Figure 5.2. The flood flow frequency for the Neches River near Neches gage is shown in Figure 5.3. The figure is exported from PeakFQ (USGS, 2014), and plots annual peak discharge vs. AEP in percent. No low outliers were identified.

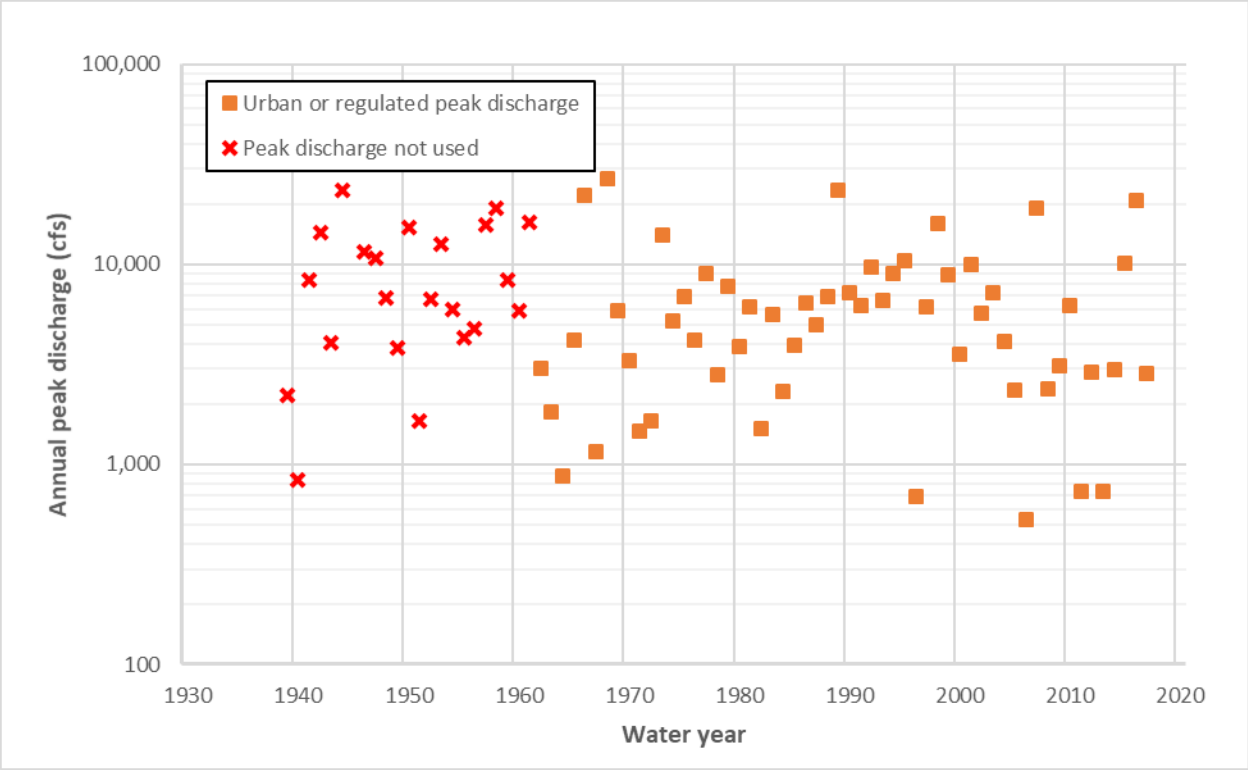


Figure 5.2: Annual Peak Discharge Data for USGS Streamgaging Station 08032000 Neches River nr Neches, Tex.

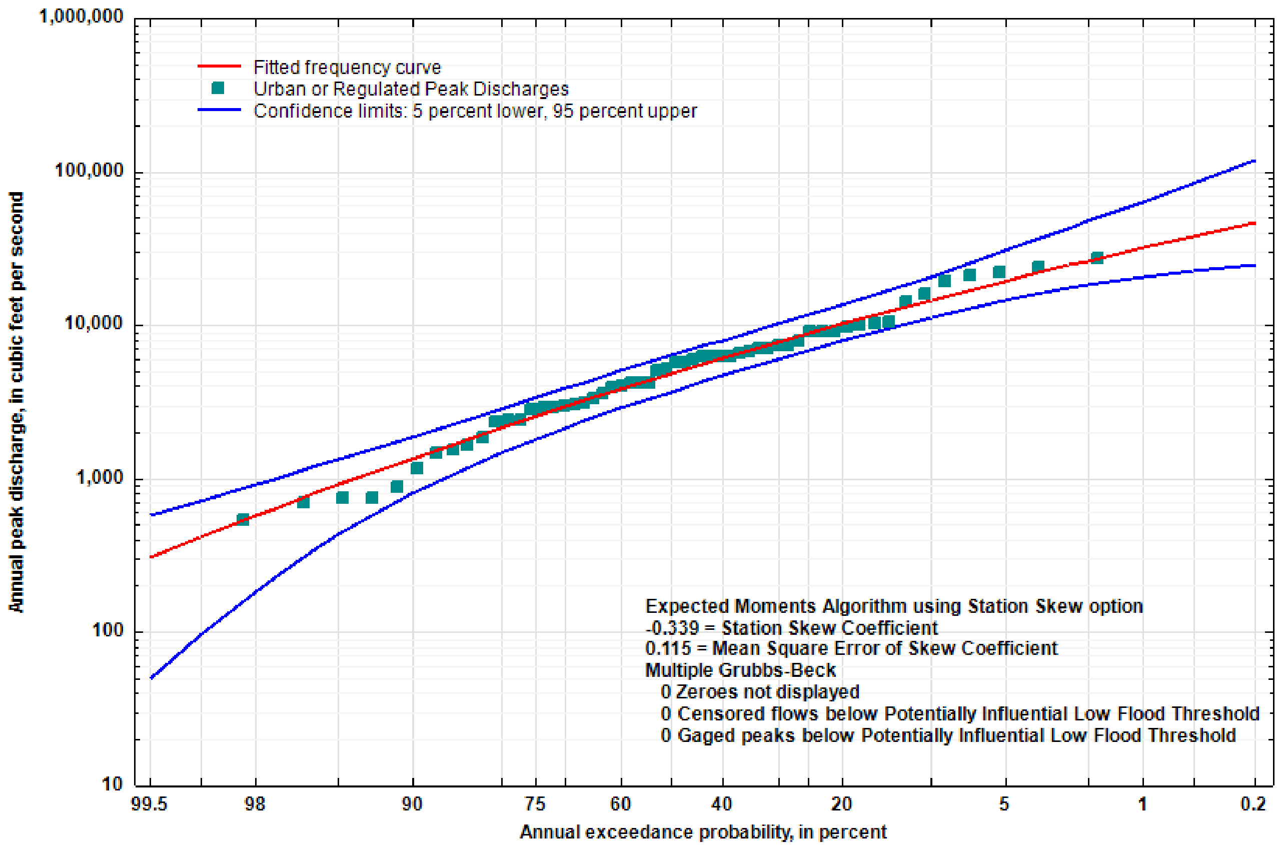


Figure 5.3: Flood Flow Frequency Curve for USGS Streamgaging Station 08032000 Neches River nr Neches, Tex.

08033500 Neches River near Rockland, Tex.

The period of record at USGS streamgaging station 08033500 Neches River near Rockland, Tex. (hereinafter referred to as the “Neches River near Rockland gage”) was from 1904 through 2017. Peak discharges beginning in 1962 are flagged as being influenced by regulation coinciding with the completion of Palestine Lake. However, Palestine Lake appears to have little to no effect as far downstream as Rockland, so the entire period of record was included in the analysis. There is approximately 2,800 square miles in contributing drainage area between Palestine Lake and the Neches River near Rockland gage. A historic peak of 62,000 cfs from 1884 was included in the analysis as well. However, the 1884 historical peak of 62,000 cfs appears to be underestimated when comparing this value to the upstream peak of 110,000 cfs near Diboll and downstream peak of 120,000 cfs near Town Bluff. It is unlikely that the 1884 flood dropped nearly 50,000 cfs in the short distance between the Diboll and Rockland gage locations. On the other hand, it is also possible that the peak at Diboll is overestimated. Upstream from the Diboll gage, the Alto gage records a historical peak of 50,000 cfs. Therefore, the 1884 historical peak near Rockland was replaced with an interval of 60,000 cfs to 110,000 cfs to incorporate the uncertainty of the historical peak value into the EMA analysis. A perception threshold of 80,000 cfs was set for the missing record from 1885 to 1903.

Peak values are missing in 1912 and 1913, and a perception threshold of 40,000 cfs was used to reflect the relative maximum peak discharges in the nearby historic peak flood event, following Bulletin 17C guidelines for perception thresholds (England and others, 2018). It is assumed that because the historic peak of this magnitude was recorded in that time span, any other events in that period of missing record would be less than that value.

The largest peak in the record for the location is the 1944 peak discharge of 49,800 cfs at a stage of 31.84 ft. The data as set up for statistical frequency analysis are shown in a log-normal plot of annual peak discharge vs. water year in Figure 5.4. The flood flow frequency for the Neches River near Rockland gage is shown in Figure 5.5. The figure is exported from PeakFQ (U.S. Geological Survey, 2014), and plots annual peak discharge vs. annual exceedance probability in percent. The low-outlier threshold was set by PeakFQ at 2,520 cfs.

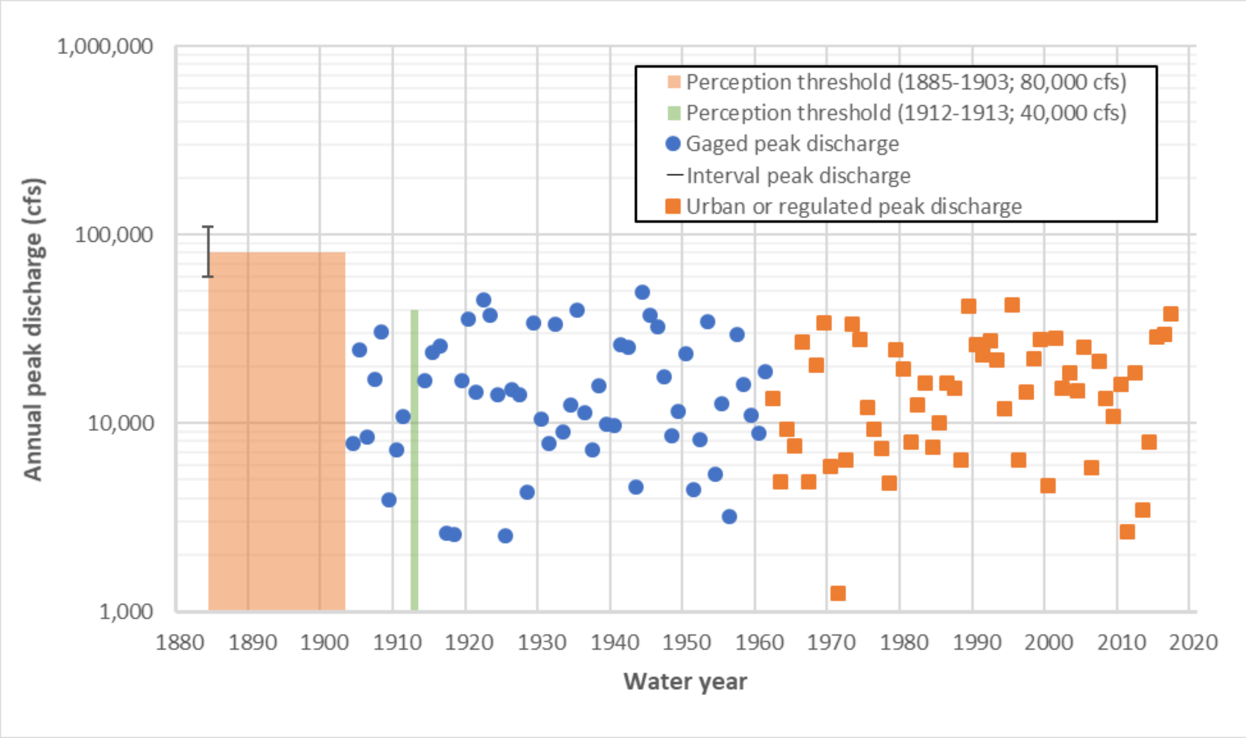


Figure 5.4: Annual Peak Discharge Data for USGS Station 08033500 Neches River nr Rockland, Tex.

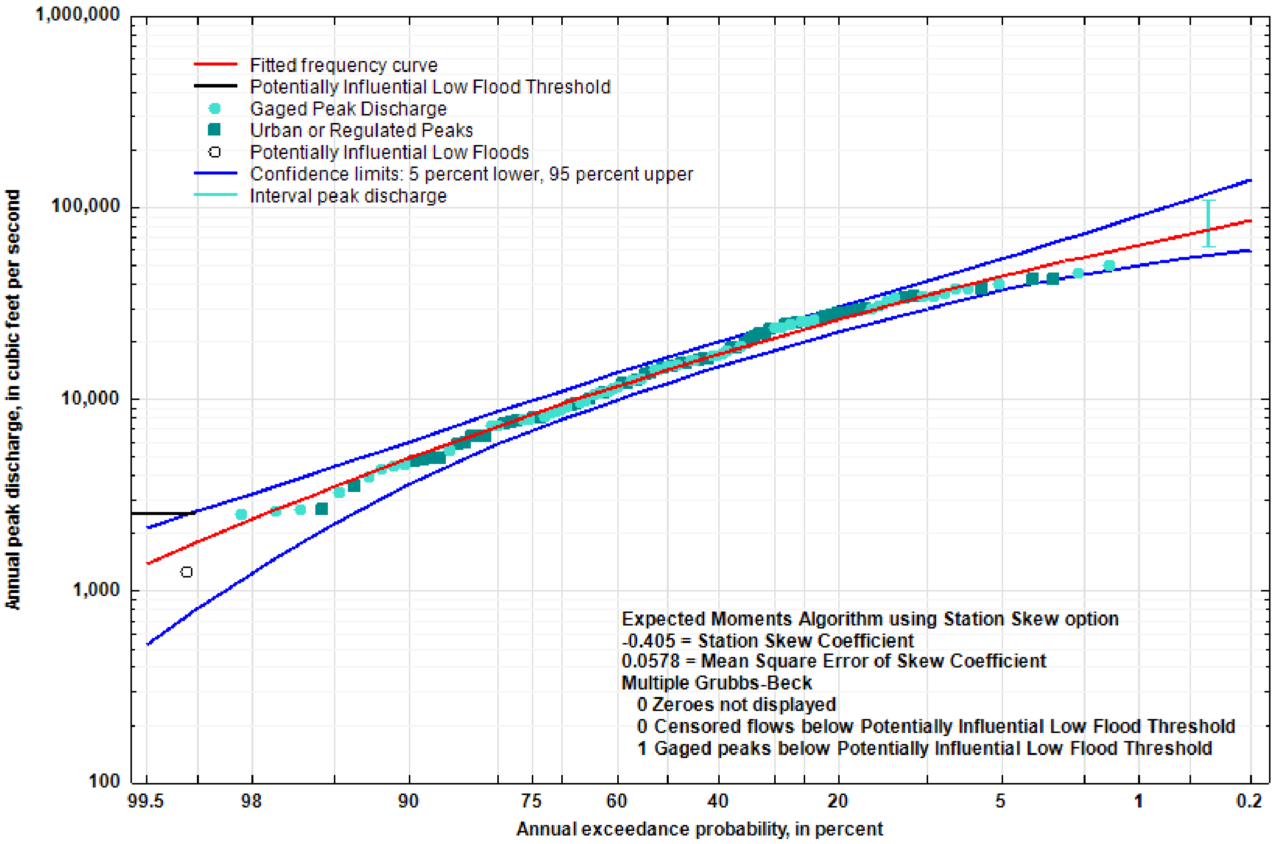


Figure 5.5: Flood Flow Frequency Curve for USGS Station 08033500 Neches River nr Rockland, Tex.

08036500 Angelina River near Alto, Tex.

For this analysis, the period of record at the Angelina River near Alto gage was from 1943 through 2017 (USGS, 2018). The gaged record is missing during 1944–1958, and using the EMA algorithm capabilities, a perception threshold of 20,000 cfs was set for the period of missing record. All peak discharges are coded as influenced by regulation since the completion of Striker Lake in 1957. However, the 1943 peak and the perception threshold were included in the analysis as well with the assumption that an event greater than this magnitude would have been recorded at this location.

The largest peak in the record for the location is the 1989 peak discharge of 42,500 cfs at a stage of 23.20 ft. The data as set up for statistical frequency analysis are shown in a log-normal plot of annual peak discharge vs. water year in Figure 5.6. The flood flow frequency for the Angelina River near Alto gage is shown in Figure 5.7. The figure is exported from PeakFQ (USGS, 2014), and plots annual peak discharge vs. AEP in percent. A low-outlier threshold was manually set at 4,900 cfs because the PeakFQ software failed to account for the apparent shift in peak discharge at that point.

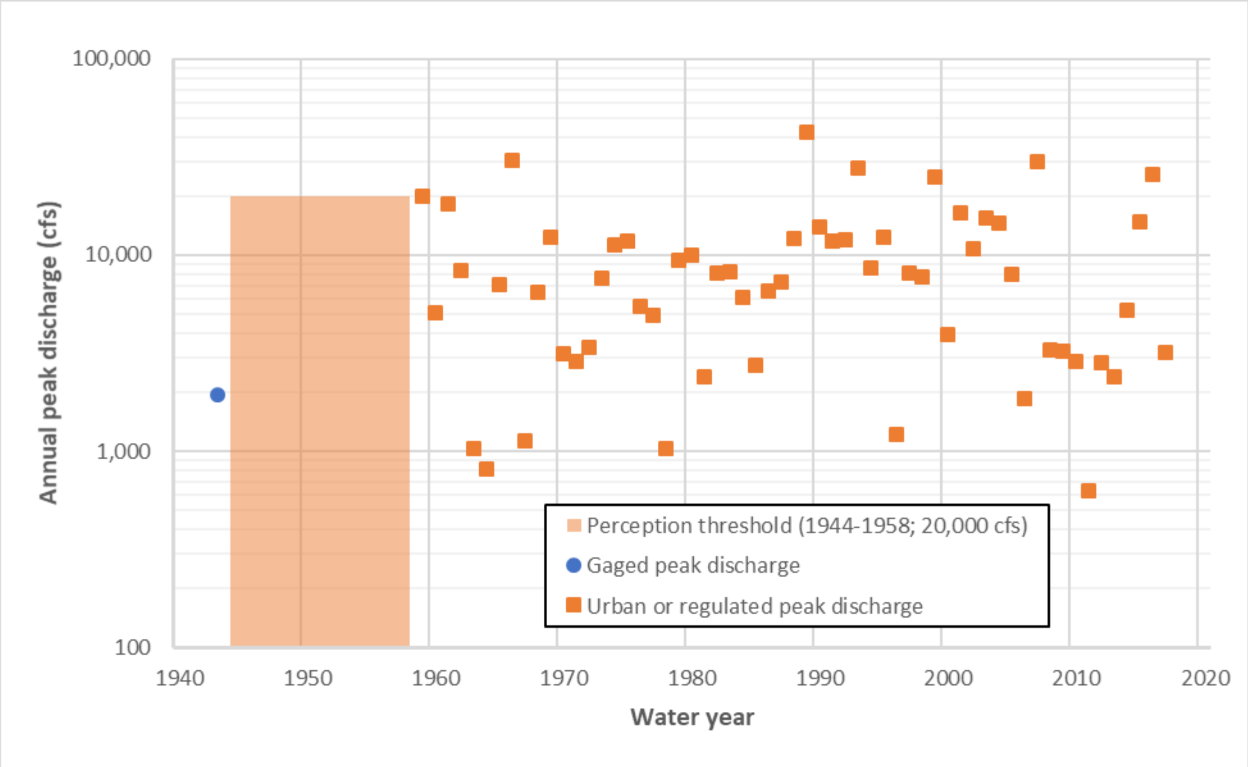


Figure 5.6: Annual Peak Discharge Data for USGS Station 08036500 Angelina River near Alto, Tex.

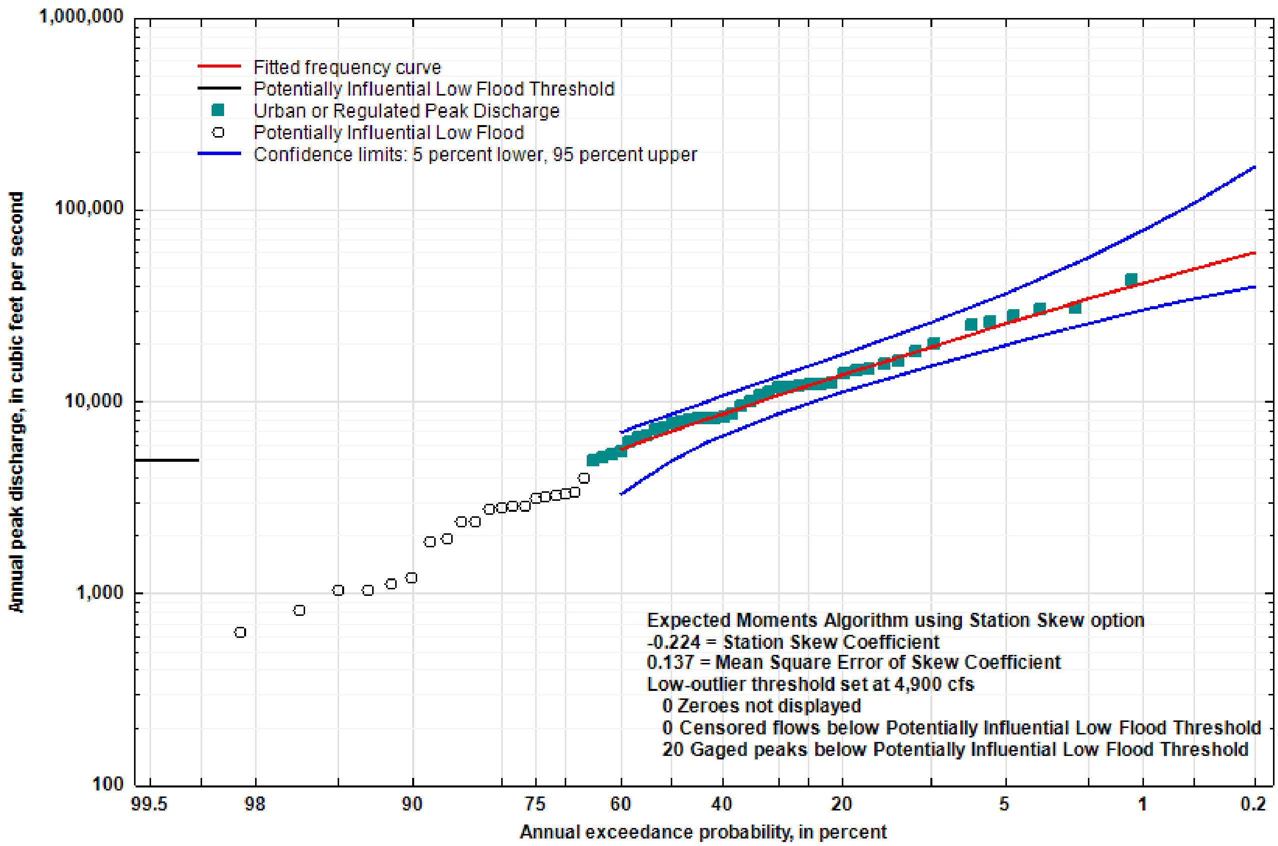


Figure 5.7: Flood Flow Frequency Curve for USGS Station 08036500 Angelina River near Alto, Tex.

08040600 Neches River near Town Bluff, Tex.

The period of record at the Neches River near Town Bluff gage was from 1951 through 2017 (USGS, 2018). A historical peak discharge record from 1884 and the peak discharge record during 1951–64 were removed from the analysis because the construction of Sam Rayburn Reservoir as a flood control reservoir, starting service in 1965, had a noticeable effect on annual peak discharges at the gage. To maintain a homogenous record, peak discharges recorded during 1951–64 prior to the completion of Sam Rayburn Reservoir were not used in the analysis. Although it may seem likely that B.A. Steinhagen Lake would have a noticeable effect on peak discharges at the Neches River near Town Bluff gage, the reservoir is not a flood control reservoir and only has a minimal effect on the peak flow measured at this gage despite the proximity of the gage to the reservoir (the Neches River near Town Bluff gage is approximately 2 mi. downstream of the dam at B.A. Steinhagen Lake).

The Kendall's Tau and *p*-value for the Neches River near Town Bluff gage (Table 5.1) indicate a statistically significant positive trend in the record with the threshold defined for this report (p-value < 0.10). Other than a few peak discharges below 10,000 cfs, the annual peak discharges after approximately 1990 exhibit a tighter grouping centered around 20,000 to 30,000 cfs, which may account for the positive trend seen in the Kendall's Tau test.

The largest peak in the record for the location is the 2017 peak discharge of 91,000 cfs at a stage of 80.70 ft. The 2017 peak discharge was a result of Hurricane Harvey. This slightly surpassed the second greatest peak on record, which occurred in 1953 with a peak discharge of 90,900 cfs. The data as set up for statistical frequency analysis are shown in a log-normal plot of annual peak discharge vs. water year in Figure 5.8. The flood flow frequency for the Neches River near Town Bluff gage is shown in Figure 5.9. The figure is exported from PeakFQ (U.S. Geological Survey, 2014), and plots annual peak discharge vs. AEP in percent. A low-outlier threshold was manually set at 8,900 cfs because the PeakFQ software failed to account for the apparent shift in peak discharge at that point.

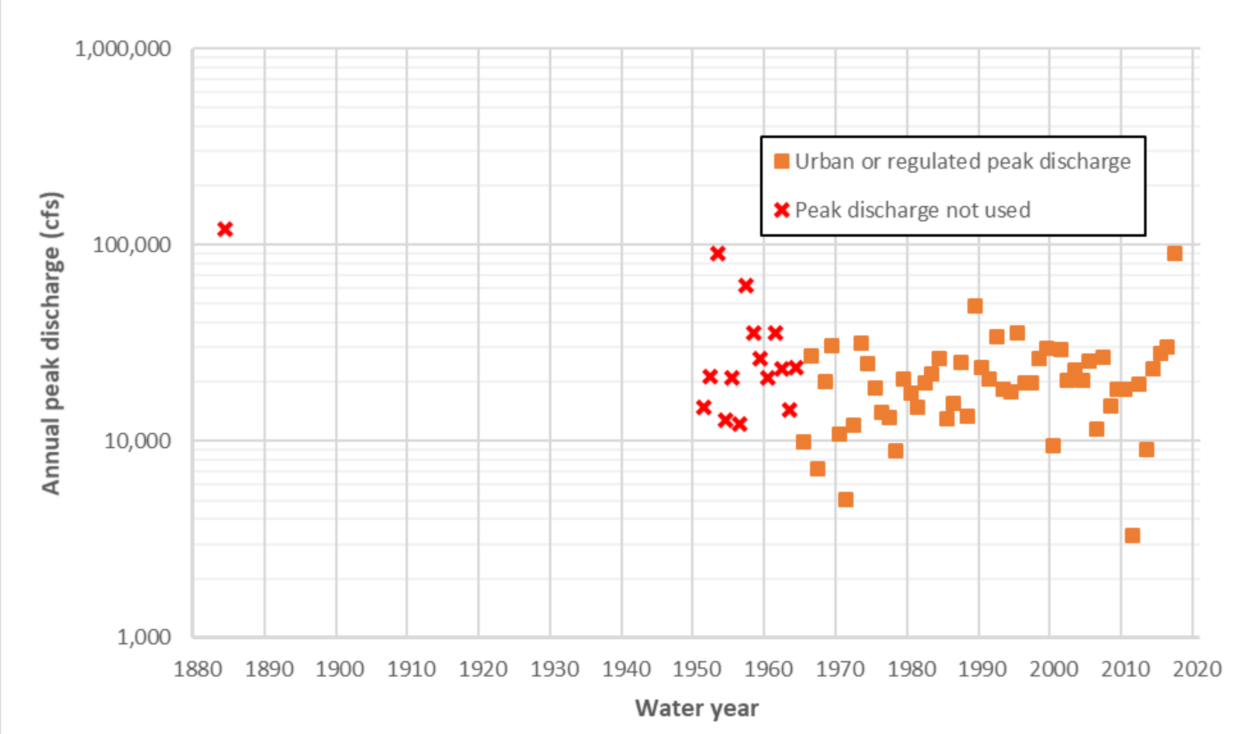


Figure 5.8: Annual Peak Discharge Data for USGS Station 08040600 Neches River near Town Bluff, Tex.

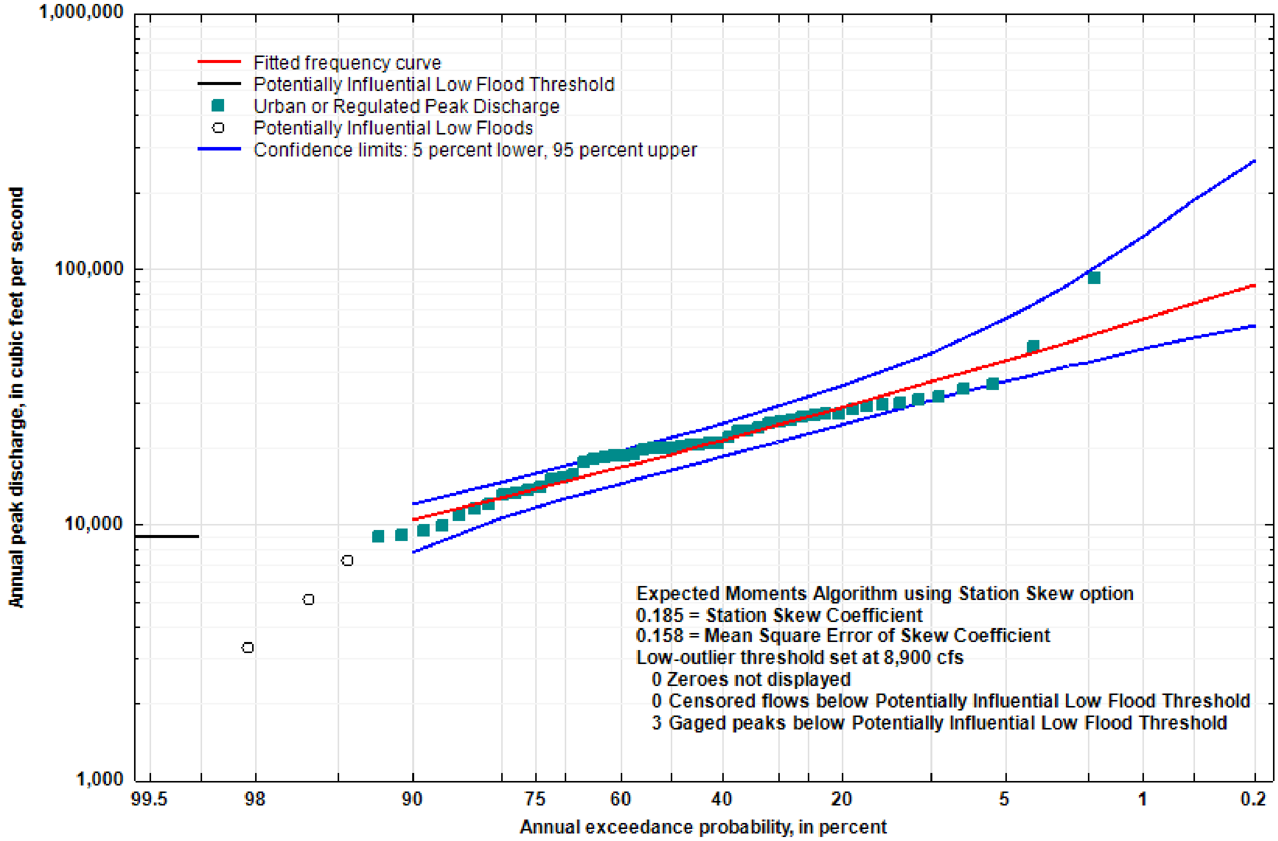


Figure 5.9: Flood Flow Frequency Curve for USGS Station 08040600 Neches River near Town Bluff, Tex.

08040600 Neches River near Town Bluff, Tex. (alternative analysis)

An alternative analysis is presented here for the Neches River near Town Bluff gage that includes the 1884 historical peak flood with a perception threshold of 80,000 cfs for the period between 1884 and 1965. The period of record for the alternative analysis is now from 1884 through 2020. The 1884 historic peak flood was included in this alternative analysis after it was determined that the flood primarily originated on the unregulated portion of the Neches River. In fact, the historic 1884 peak discharge at Diboll of 110,000 cfs was very similar to the historic peaks near Town Bluff and Evadale of 120,000 and 125,000 cfs, respectively. Therefore, the 1884 flood would not have been impacted significantly by the addition of Sam Rayburn Reservoir.

The flood flow frequency for the Neches River near Town Bluff gage is shown in Figure 5.10. The figure is exported from PeakFQ (U.S. Geological Survey, 2014), and plots annual peak discharge vs. AEP in percent. A low-outlier threshold was manually set at 8,900 cfs because the PeakFQ software failed to account for the apparent shift in peak discharge at that point.

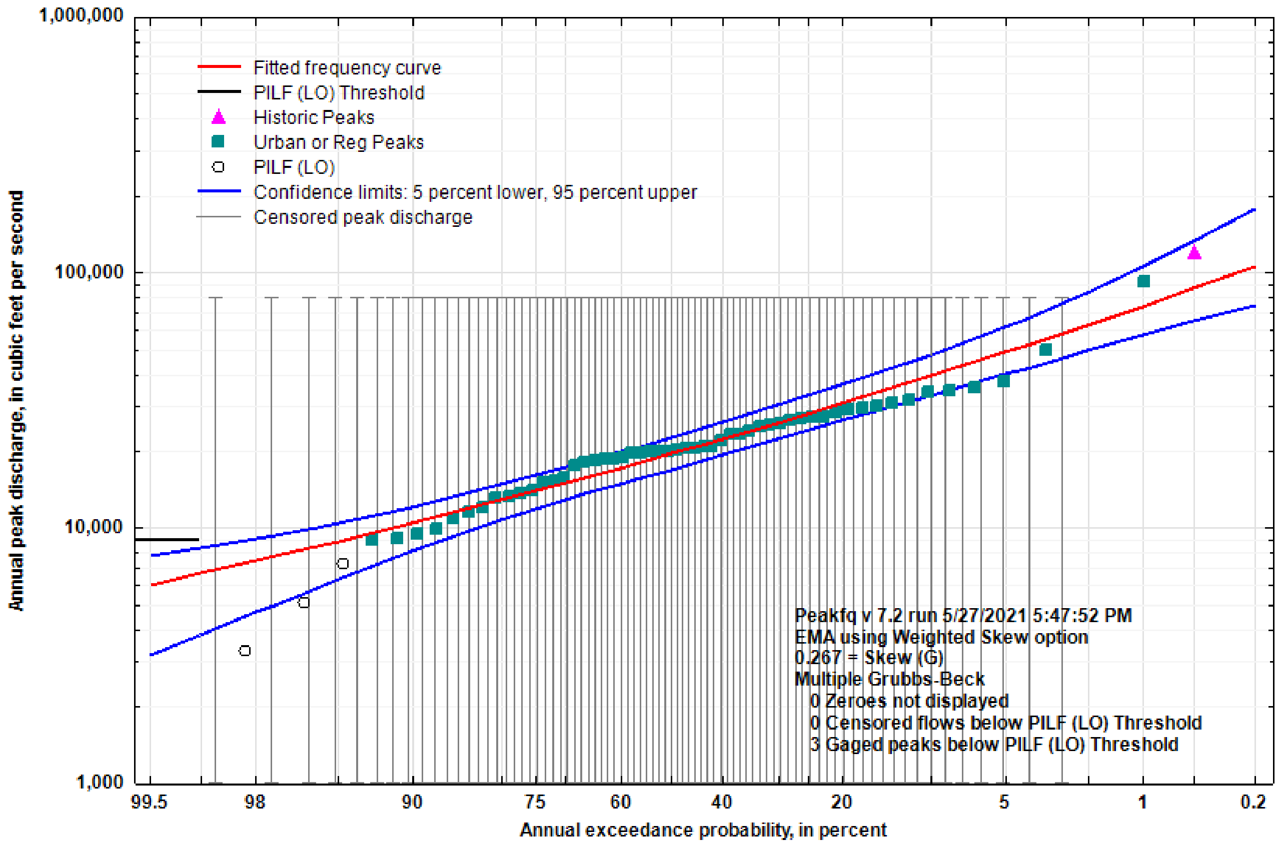


Figure 5.10: Flood Flow Frequency Curve for U.S. Geological Survey Streamgaging Station 08040600 Neches River near Town Bluff, Tex. (alternative analysis).

08041500 Village Creek near Kountze, Tex.

The period of record at USGS station 08041500 Village Creek near Kountze, Tex. (hereinafter referred to as the "Village Creek near Kountze gage") was from 1924 through 2017 (USGS, 2018). No peak discharges were coded as being influenced by regulation, so the entire period of record was used for the analysis. The gaged record is missing for the date range of 1928–1939, and using the EMA algorithm capabilities, a perception threshold of 70,000 cfs was set for the period of missing record with the assumption that an event greater than this magnitude would have been recorded at this location.

The largest peak in the record for the Village Creek near Kountze gage is the 2017 peak discharge of 182,000 cfs at a stage of 35.96 ft. The 2017 peak discharge is a result of Hurricane Harvey, and the peak discharge is nearly three times greater than the second greatest event of 1941 with the peak discharge of 67,200 cfs. The data as set up for statistical frequency analysis are shown in a log-normal plot of annual peak discharge vs. water year in Figure 5.11. The flood flow frequency for the Village Creek near Kountze gage is shown in Figure 5.12. The figure is exported from PeakFQ (USGS, 2014), and plots annual peak discharge vs. AEP in percent. The low-outlier threshold was set by PeakFQ at 2,260 cfs.

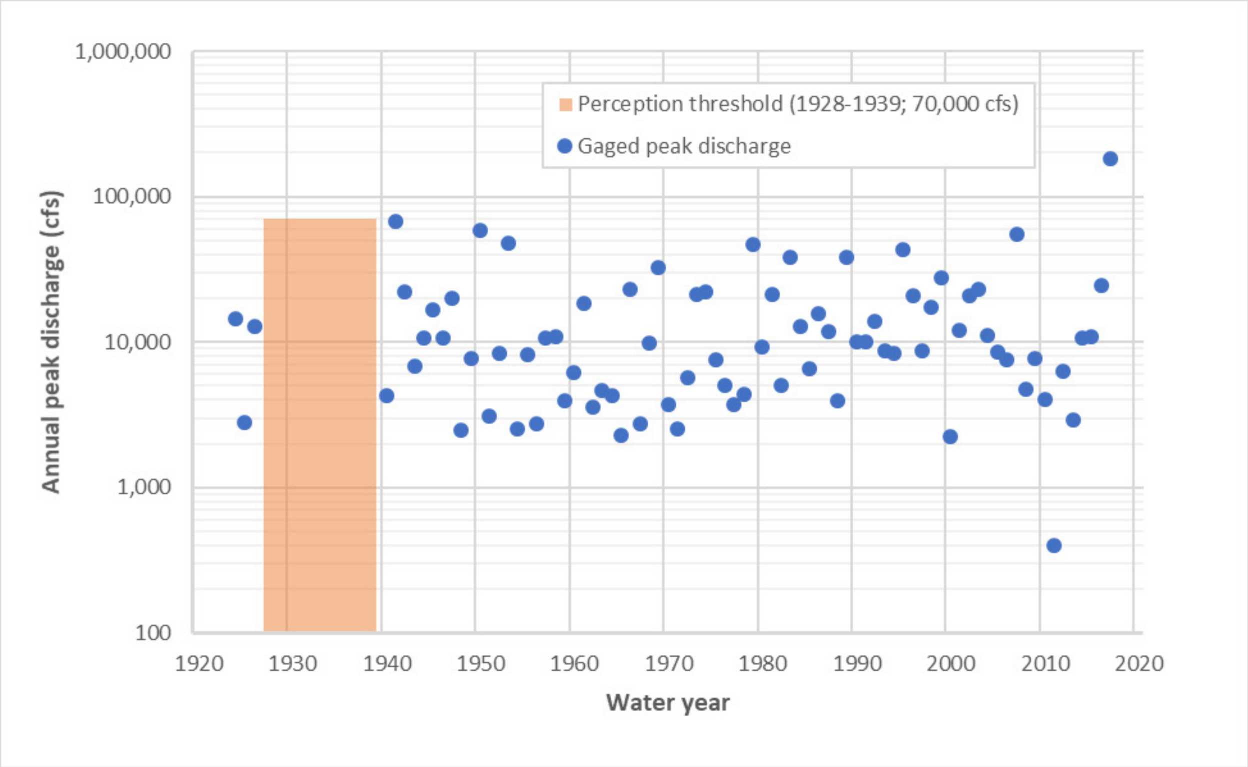


Figure 5.11: Annual Peak Discharge Data for USGS Station 08041500 Village Creek near Kountze, Tex.

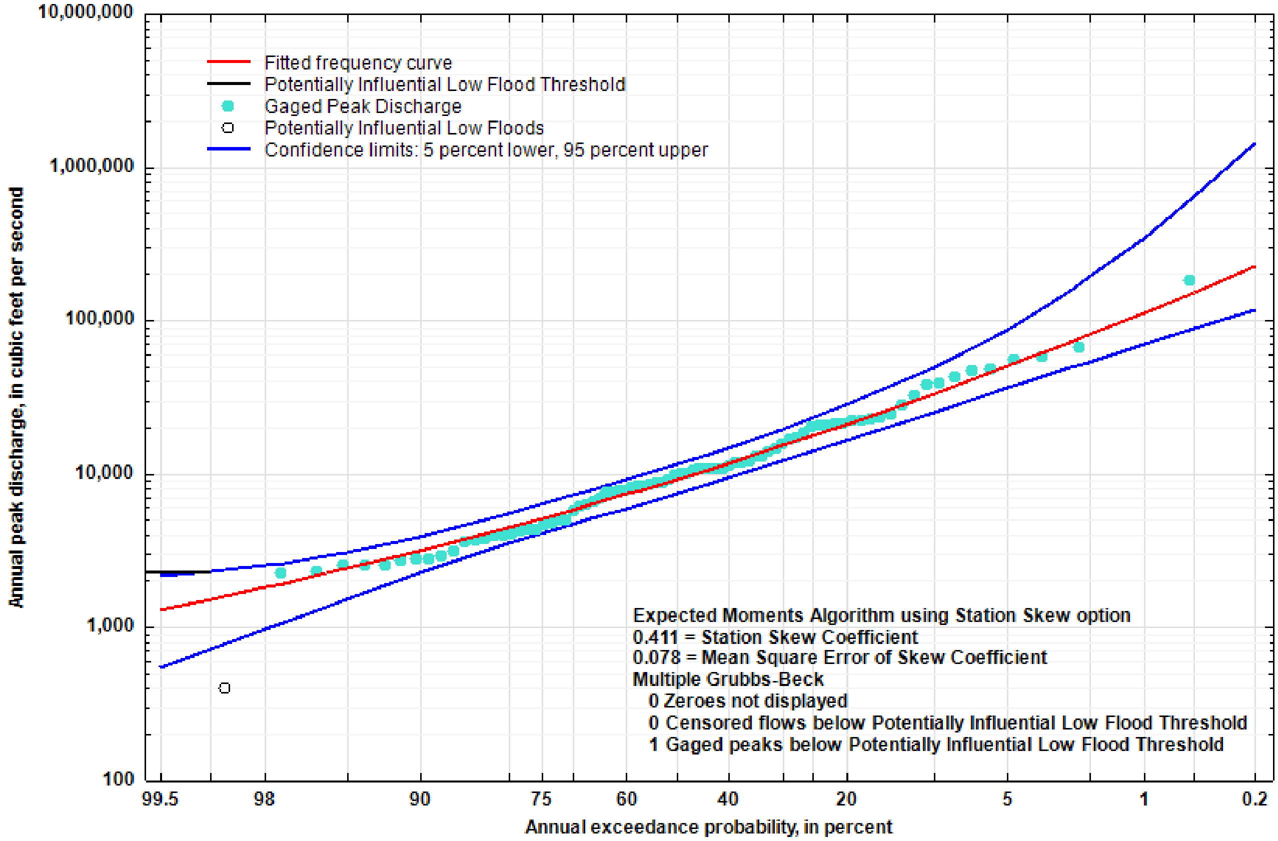


Figure 5.12: Flood Flow Frequency Curve for USGS Station 08041500 Village Creek near Kountze, Tex.

08041500 Village Creek near Kountze, Tex. (alternative analysis)

An alternative analysis is presented here for the Village Creek near Kountze gage that considers the uncertainty associated with the Hurricane Harvey peak flood event occurring in 2017. Instead of using a discrete value for the 2017 peak discharge, an interval peak of 100,000 to 175,000 cfs was instead used in order to better incorporate the uncertainty found in modeling this event. While the USGS peak discharge estimate was 182,000 cfs for Hurricane Harvey, there was a high degree of uncertainty in this estimate. The peak observed gage height during Hurricane Harvey exceeded any previous observed event at this location by over 8 feet and was well beyond the existing rating curve for the gage. An alternate estimate of the Hurricane Harvey peak discharge on Village Creek was made by the InFRM team using an existing hydraulic model from the Neches Base Level Engineering (BLE) data (FEMA, 2019), which is consistent with the USGS indirect discharge estimation methods. By applying a range of low to high roughness n-values in the hydraulic model, an interval estimate of 100,000 to 175,000 cfs was determined as the range of peak discharges that corresponded to the recorded peak gage height during Hurricane Harvey. The flood flow frequency for the Village Creek near Kountze gage is shown in Figure 5.13. The figure is exported from PeakFQ (USGS, 2014), and plots annual peak discharge vs. AEP in percent. The low-outlier threshold was set by PeakFQ at 2,260 cfs.

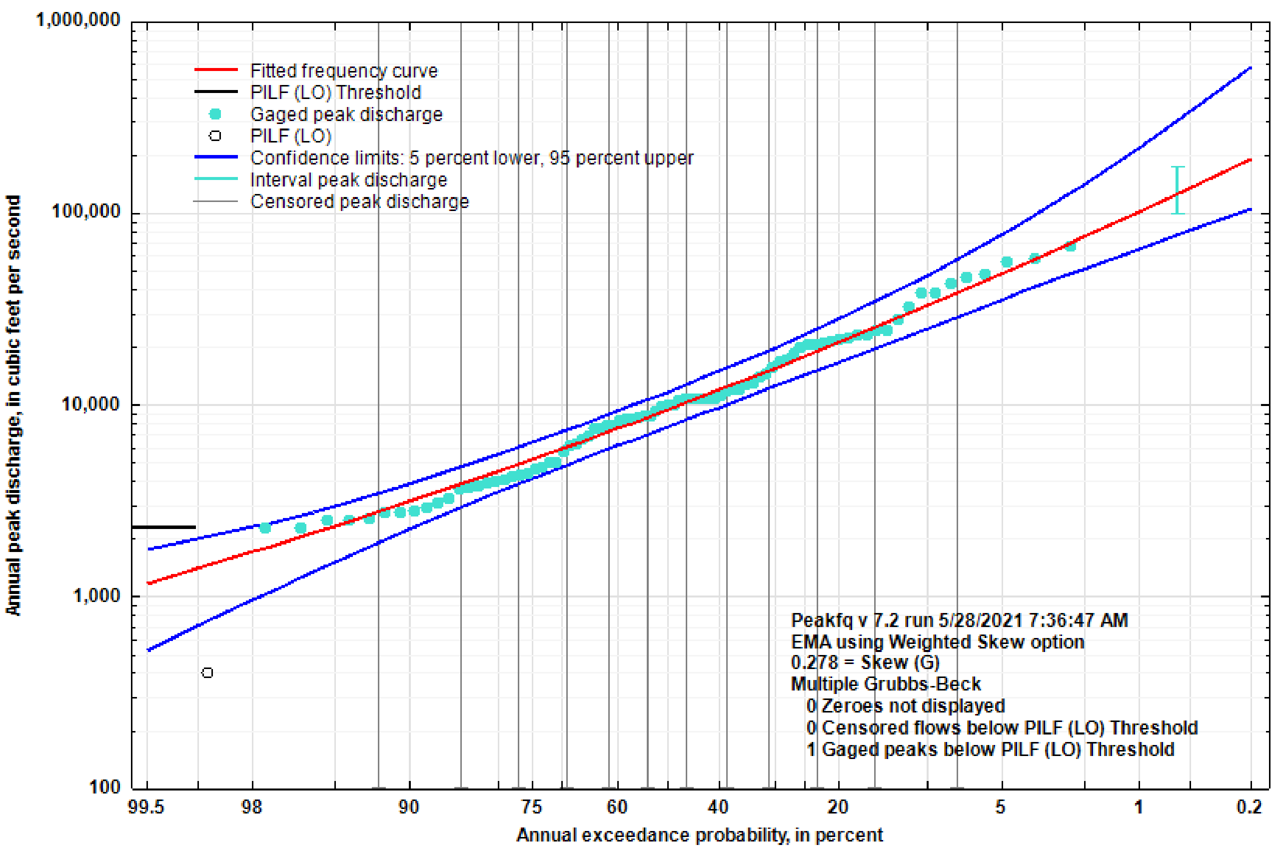


Figure 5.13: Flood Flow Frequency Curve for U.S. Geological Survey Streamgaging Station 08041500 Village Creek near Kountze, Tex. (alternative analysis)

Table 5.2: Statistically Estimated Annual Flood Flow Frequency Results and Confidence Intervals for the Fifteen U.S. Geological Survey Streamgaging Stations in the Neches River Basin, Texas Based on USGS-PeakFQ Software

[cfs, cubic feet per second; %, percent; CI, confidence limit; Note, table contents derived from EXP file (file extension name) of USGS-PeakFQ software output (USGS, 2014). The estimates are of primary interest and are accentuated using a bold typeface.]

Station number and name	Flood flow frequency by corresponding average return period (recurrence interval) in years							
	2 year (cfs)	5 year (cfs)	10 year (cfs)	25 year (cfs)	50 year (cfs)	100 year (cfs)	200 year (cfs)	500 year (cfs)
08032000 Neches River near Neches, Tex.								
Lower 95%-CI	3,660	7,810	11,100	15,500	18,300	20,700	22,600	24,800
Estimate	**4,810**	**10,100**	**14,500**	**20,900**	**26,100**	**31,600**	**37,500**	**45,800**
Upper 95%-CI	6,300	13,400	20,600	33,900	46,900	63,300	83,700	119,000
08032500 Neches River near Alto, Tex.								
Lower 95%-CI	4,630	10,700	15,700	23,100	29,300	35,900	42,800	52,100
Estimate	**6,800**	**15,100**	**22,500**	**34,000**	**44,100**	**55,600**	**68,400**	**87,600**
Upper 95%-CI	9,560	21,200	31,900	51,000	72,100	103,000	150,000	253,000
08033000 Neches River near Diboll, Tex.								
Lower 95%-CI	8,440	17,100	23,500	32,000	38,100	43,800	49,000	55,100
Estimate	**10,400**	**20,700**	**28,700**	**39,800**	**48,700**	**57,900**	**67,300**	**80,300**
Upper 95%-CI	12,600	25,100	35,500	52,200	68,200	87,900	112,000	153,000
08033500 Neches River near Rockland, Tex.								
Lower 95%-CI	12,100	22,400	29,700	38,800	44,800	50,100	54,600	59,800
Estimate	**14,100**	**25,900**	**34,600**	**46,200**	**55,000**	**63,900**	**72,900**	**84,900**
Upper 95%-CI	16,500	30,200	41,200	58,100	73,000	89,700	109,000	138,000
08034500 Mud Creek near Jacksonville, Tex.								
Lower 95%-CI	2,620	5,770	8,430	12,100	14,800	17,300	19,500	22,200
Estimate	**3,420**	**7,390**	**10,800**	**16,000**	**20,500**	**25,400**	**30,800**	**38,600**
Upper 95%-CI	4,390	9,460	14,600	24,500	34,800	47,900	64,900	94,900
08034500 Mud Creek near Jacksonville, Tex. (*Alternative Analysis*)								
Lower 95%-CI	2,940	6,310	9,080	12,800	15,500	18,000	20,200	23,000
Estimate	**3,690**	**7,870**	**11,500**	**16,800**	**21,300**	**26,300**	**31,700**	**39,400**
Upper 95%-CI	4,630	10,000	15,400	25,300	35,200	47,700	63,500	90,700
08036500 Angelina River near Alto, Tex.								
Lower 95%-CI	4,900	11,100	15,300	21,000	25,400	29,800	34,100	39,600
Estimate	**6,960**	**13,800**	**19,300**	**27,400**	**34,000**	**41,200**	**49,000**	**60,000**
Upper 95%-CI	8,550	17,500	25,700	40,400	56,100	77,500	107,000	167,000
08037000 Angelina River near Lufkin, Tex.								
Lower 95%-CI	5,840	12,100	17,000	23,500	27,900	31,700	35,000	38,700
Estimate	**7,690**	**15,700**	**22,100**	**31,200**	**38,600**	**46,300**	**54,300**	**65,500**
Upper 95%-CI	10,100	20,500	30,800	50,500	70,700	96,300	129,000	188,000
08038000 Attoyac Bayou near Chireno, Tex.								
Lower 95%-CI	3,130	12,000	17,100	23,200	27,300	30,900	33,800	36,900
Estimate	**6,500**	**15,100**	**21,400**	**29,200**	**34,600**	**39,500**	**43,900**	**49,100**
Upper 95%-CI	8,200	19,000	28,400	45,700	63,300	82,300	97,100	116,000

Table 5.2 (continued): Statistically Estimated Annual Flood Flow Frequency Results and Confidence Intervals for the Fifteen U.S. Geological Survey Streamgaging stations in the Neches River Basin, Texas Based on USGS-PeakFQ Software

Station number and name	Flood flow frequency by corresponding average return period (recurrence interval) in years							
	2 year (cfs)	5 year (cfs)	10 year (cfs)	25 year (cfs)	50 year (cfs)	100 year (cfs)	200 year (cfs)	500 year (cfs)
08039100 Ayish Bayou near San Augustine, Tex.								
Lower 95%-CI	3,030	6,480	9,260	13,000	15,600	18,000	20,200	22,700
Estimate	**3,940**	**8,350**	**12,000**	**17,400**	**21,900**	**26,700**	**31,800**	**39,100**
Upper 95%-CI	5,110	11,000	16,900	28,400	40,600	56,600	77,500	116,000
08040600 Neches River near Town Bluff, Tex.								
Lower 95%-CI	16,300	24,600	30,500	37,800	43,200	48,400	53,500	60,100
Estimate	**18,900**	**28,700**	**36,100**	**46,300**	**54,600**	**63,500**	**73,000**	**86,700**
Upper 95%-CI	21,900	34,600	46,500	69,900	96,400	134,000	185,000	267,000
08040600 Neches River near Town Bluff, Tex. (*Alternative Analysis*)								
Lower 95%-CI	16,900	26,100	32,900	42,000	49,100	56,500	64,100	74,500
Estimate	**19,500**	**30,600**	**39,200**	**51,700**	**62,100**	**73,500**	**86,100**	**105,000**
Upper 95%-CI	22,600	36,200	47,500	65,700	83,200	105,000	132,000	177,000
08041000 Neches River at Evadale, Tex.								
Lower 95%-CI	17,100	26,500	32,800	40,800	46,600	52,100	57,500	64,300
Estimate	**19,900**	**31,000**	**39,000**	**49,900**	**58,400**	**67,300**	**76,600**	**89,700**
Upper 95%-CI	23,200	37,400	49,300	68,300	86,100	107,000	133,000	175,000
08041000 Neches River at Evadale, Tex. (*Alternative Analysis*)								
Lower 95%-CI	17,600	28,100	35,400	45,100	52,400	59,700	67,100	76,800
Estimate	**20,600**	**33,000**	**42,200**	**55,000**	**65,300**	**76,200**	**87,800**	**104,000**
Upper 95%-CI	24,100	39,100	51,000	68,900	85,300	105,000	129,000	168,000
08041500 Village Creek near Kountze, Tex.								
Lower 95%-CI	7,340	16,300	25,200	39,700	53,200	68,900	87,100	115,000
Estimate	**9,190**	**20,900**	**33,200**	**56,200**	**80,100**	**111,000**	**152,000**	**225,000**
Upper 95%-CI	11,500	27,700	48,600	104,000	189,000	345,000	635,000	1,430,000
08041500 Village Creek near Kountze, Tex. (*Alternative Analysis*)								
Lower 95%-CI	7,660	16,700	25,100	38,700	50,900	65,000	81,000	105,000
Estimate	**9,410**	**21,000**	**32,800**	**53,700**	**74,700**	**101,000**	**134,000**	**191,000**
Upper 95%-CI	11,700	27,700	46,900	88,500	140,000	217,000	334,000	580,000
08041700 Pine Island Bayou near Sour Lake, Tex.								
Lower 95%-CI	3,140	6,950	11,000	17,800	24,500	32,600	42,500	58,800
Estimate	**4,190**	**9,600**	**15,800**	**28,500**	**42,800**	**63,200**	**91,700**	**147,000**
Upper 95%-CI	5,580	14,700	30,100	89,900	232,000	501,000	1,090,000	3,050,000

Table 5.2 (continued): Statistically Estimated Annual Flood Flow Frequency Results and Confidence Intervals for the Fifteen U.S. Geological Survey Streamgaging stations in the Neches River Basin, Texas Based on USGS-PeakFQ Software

Station number and name	Flood flow frequency by corresponding average return period (recurrence interval) in years							
	2 year (cfs)	5 year (cfs)	10 year (cfs)	25 year (cfs)	50 year (cfs)	100 year (cfs)	200 year (cfs)	500 year (cfs)
08041749 Pine Island Bayou above BI Pump Plant, Beaumont, Tex.								
Lower 95%-CI	5,000	10,700	16,000	24,500	32,200	41,200	51,600	67,500
Estimate	**7,860**	**17,700**	**28,100**	**47,400**	**67,500**	**93,900**	**128,200**	**189,100**
Upper 95%-CI	13,500	39,600	81,300	200,000	386,000	733,000	1,380,000	3,140,000
08041780 Neches River Saltwater Barrier at Beaumont, Tex.								
Lower 95%-CI	27,300	47,400	63,500	87,100	107,000	129,000	153,000	188,000
Estimate	**37,800**	**68,200**	**95,800**	**141,000**	**184,000**	**235,000**	**298,000**	**399,000**
Upper 95%-CI	55,300	121,000	206,000	406,000	671,000	1,100,000	1,810,000	3,440,000

5.3 CHANGES TO FLOOD FLOW FREQUENCY ESTIMATES OVER TIME

Statistically based flood flow frequency estimates are dependent on the observational data and historical information that is available at the time of analysis. Changes to flood flow frequency estimates over time were analyzed for nine gages in the Neches River basin, as shown in Appendix A. The same five gages that were shown as examples in Section 5.2 have also been included in this section as examples of the changes to flood flow frequency estimates over time: Neches River near Neches, Neches River near Rockland, Angelina River near Alto, Neches River near Town Bluff, and Village Creek near Kountze. Collectively, these are shown in Figures 5.14–5.18. The annual recurrence intervals of interest here are 2, 10, 100, and 500 years, which correspond to AEPs of 0.500, 0.100, 0.010, and 0.002, respectively.

While each of these figures is discussed individually in downstream order, some general remarks are necessary. Each of these examples is intended to illustrate that there is a progression in statistical estimates over time. Peak discharges outside the period of record are not shown. For example, the 1884 peak at Neches River near Rockland is 62,000 cfs but not shown in Figure 5.15 because the streamgage record begins in 1904. Because the data used to plot the values of the 2, 10, 100, and 500-year discharge estimates in a given year are dependent on all data before that year, it is anticipated to see more variation in the line for a given recurrence interval than the line shown in the extreme right of the plot. This occurs because the total sample size as a measure of information content of flood flows increases at a proportionally smaller rate. For example, one more year of data for a sample of 10 years represents a 10-percent increase in information, whereas one more year of data for a sample of 50 years is only a 2-percent increase in information. In other words, as the record length increases given other factors remaining relatively constant (land use for example), the curves should vary year to year to a lesser degree for the simple reason that proportionally less information is included with each successive year.

The USGS-PeakFQ software when setup for data processing by EMA does not readily facilitate computations such as those required for similar graphics. The computations involved were based on fitting the LPIII to the L-moments (Asquith, 2011a, 2011b) of the data points shown from a given year backwards in time. The computations included a minimum of 10 years. As a result, the actual starting year varies amongst the figures. The results of USGS-PeakFQ as listed in Table 5.2 provide the ordinates for 2017 (right-most side of the figures), and logarithmic-derived offsets between the L-moment-based LPIII fit in 2017 were used to adjust the curves in prior years for each of the four recurrence intervals.

08032000 Neches River near Neches, Tex.

The relative effects of record length and magnitudes of substantial floods for the Neches River near Neches gage are shown in Figure 5.14. In general, all the 500, 100, and 10-year events trend downwards over time, whereas the 2-year event increases through about water year 2000 because of a lack of low-flow peak discharges (less than 5,000 cfs) in the 1990s. Contrary to the general downward trend, exceptional events in 1989, 2007, and 2016 cause marked increases in the 100 and 500-year events. However, after about the year 2000, the 100 and 500-year events appear to approach an asymptote of approximately 30,000 and 40,000 cfs, respectively. Exceptional events after this date appear to only result in a slight alteration of the trend, indicating that the frequency curve for the Neches River near Neches appears to be a relatively stable estimate.

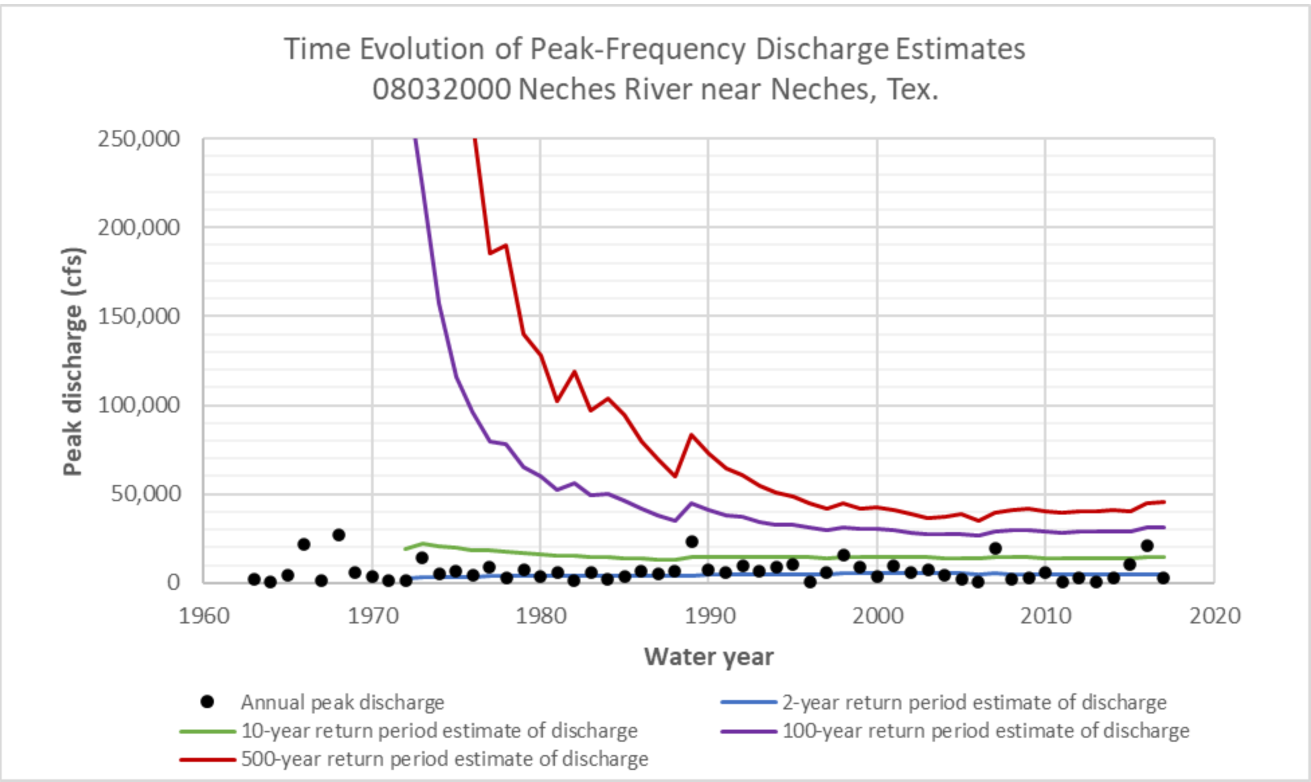


Figure 5.14: Statistical Frequency Flow Estimates versus Time for U.S. Geological Survey Streamgaging Station 08032000 Neches River near Neches, Tex.

08033500 Neches River near Rockland, Tex.

The relative effects of record length and magnitudes of substantial floods for the Neches River near Rockland gage are shown in Figure 5.15. There are a series of large events in the first half of the period of record, leading to a gradual increase in the 100 and 500-year return periods during that time. The 100 and 500-year estimates then begin to decrease around 1990 as the period of record increases and subsequent peak events do not exceed the record peak set back in 1944. The 2 and 10-year return period estimates remain somewhat constant through the period of record at approximately 15,000 and 35,000 cfs, respectively. The 2017 peak related to Hurricane Harvey results in a slight increase in discharge estimate across all return periods, but its effect on the overall trend of the estimates is difficult to discern because it is at the end of the analyzed record.

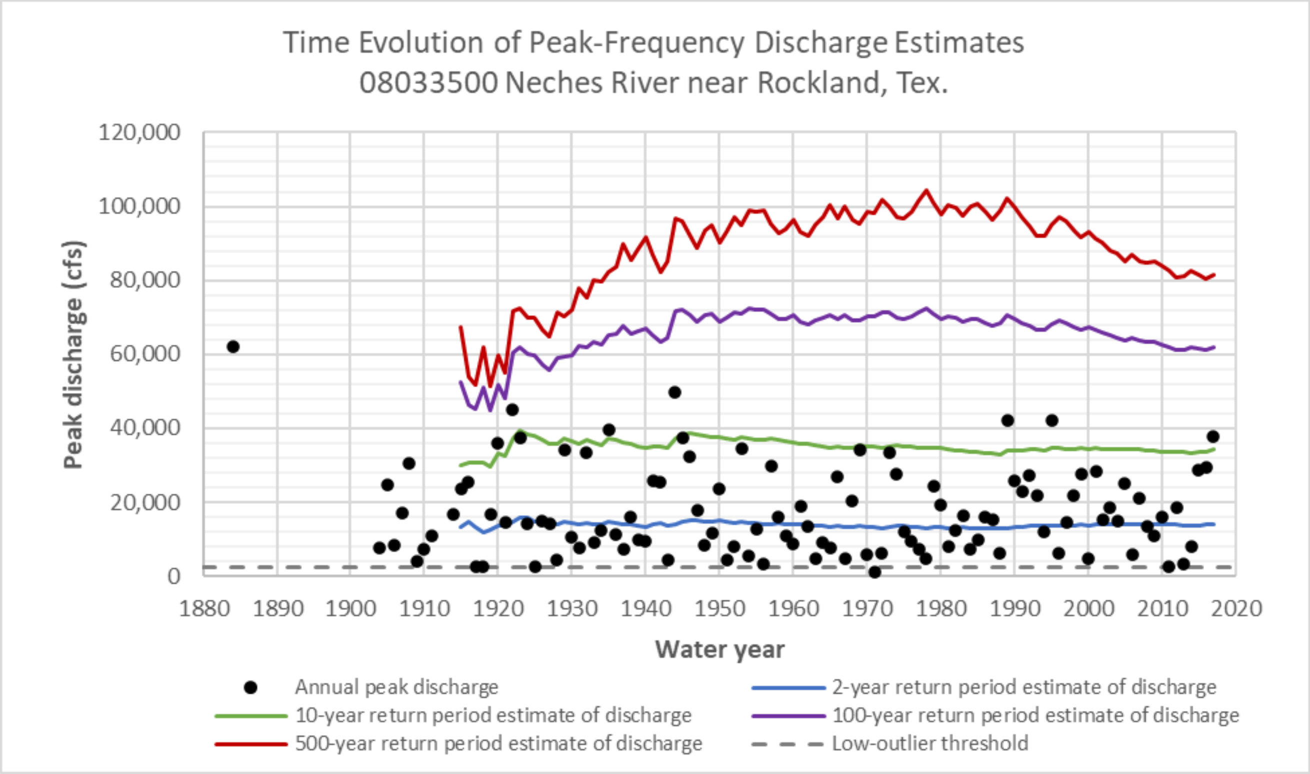


Figure 5.15: Statistical Frequency Flow Estimates versus Time for U.S. Geological Survey Streamgaging Station 08033500 Neches River near Rockland, Tex.

08036500 Angelina River near Alto, Tex.

The relative effects of record length and magnitudes of substantial floods for the Angelina River near Alto gage can be seen in Figure 5.16. In general, the trend is downward with increasing period of record until the greatest peak event occurs in 1989. After this event, there are several other large peak discharges discordant with the earlier period of record that cause jumps in the greater return period estimates. Because of this change in the peak flow record beginning in 1989, it is not clear whether a stable estimate is reached, although the 100-year estimate currently appears to reach a temporary asymptote of approximately 41,000 cfs.

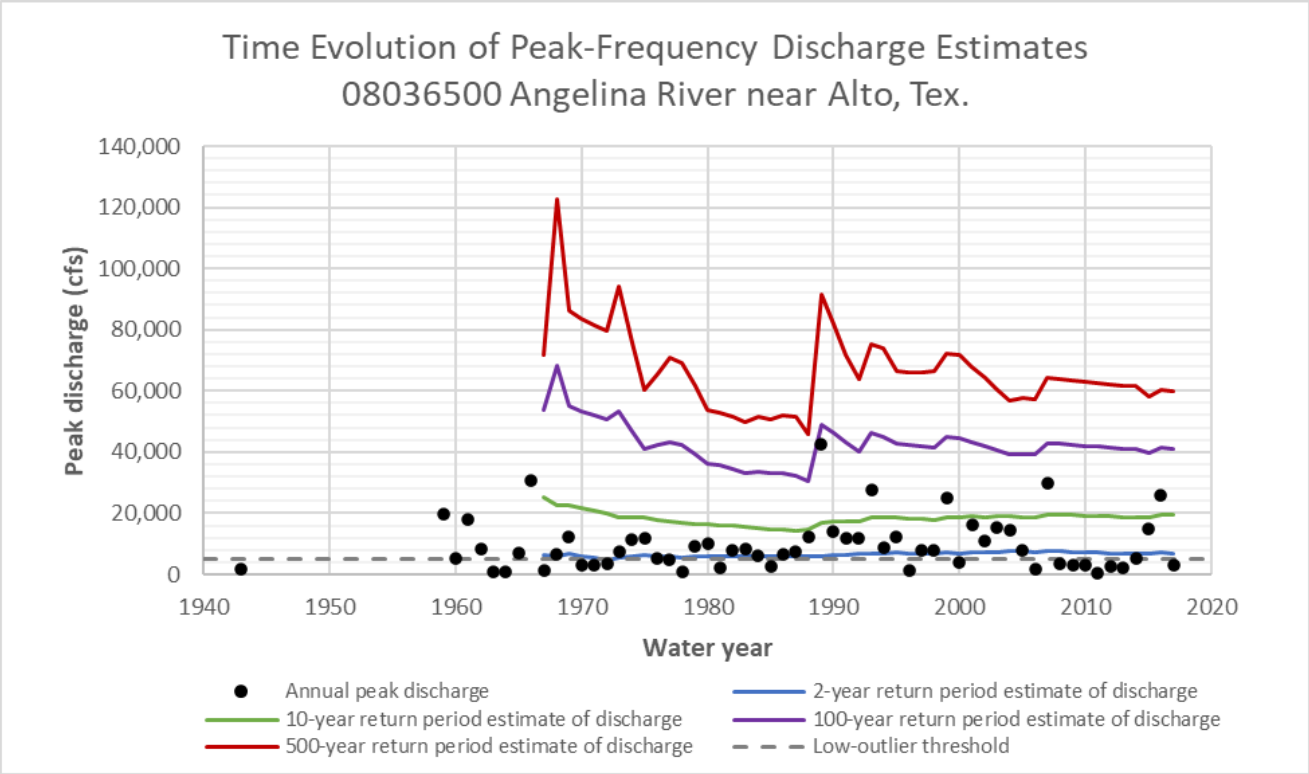


Figure 5.16: Statistical Frequency Flow Estimates versus Time for U.S. Geological Survey Streamgaging Station 08036500 Angelina River near Alto, Tex.

08040600 Neches River near Town Bluff, Tex.

The relative effects of record length and magnitudes of substantial floods for the Neches River near Town Bluff gage can be seen in Figure 5.17. Two events create a noticeable increase in the frequency curve estimates. These events are the 1989 and 2017 peak events at 49,200 and 91,000 cfs, respectively. The 2017 peak related to Hurricane Harvey resulted in a noticeable increase in discharge estimate across all return periods, but it is unclear whether this increase will be temporary or continuous since it is at the end of the analyzed record.

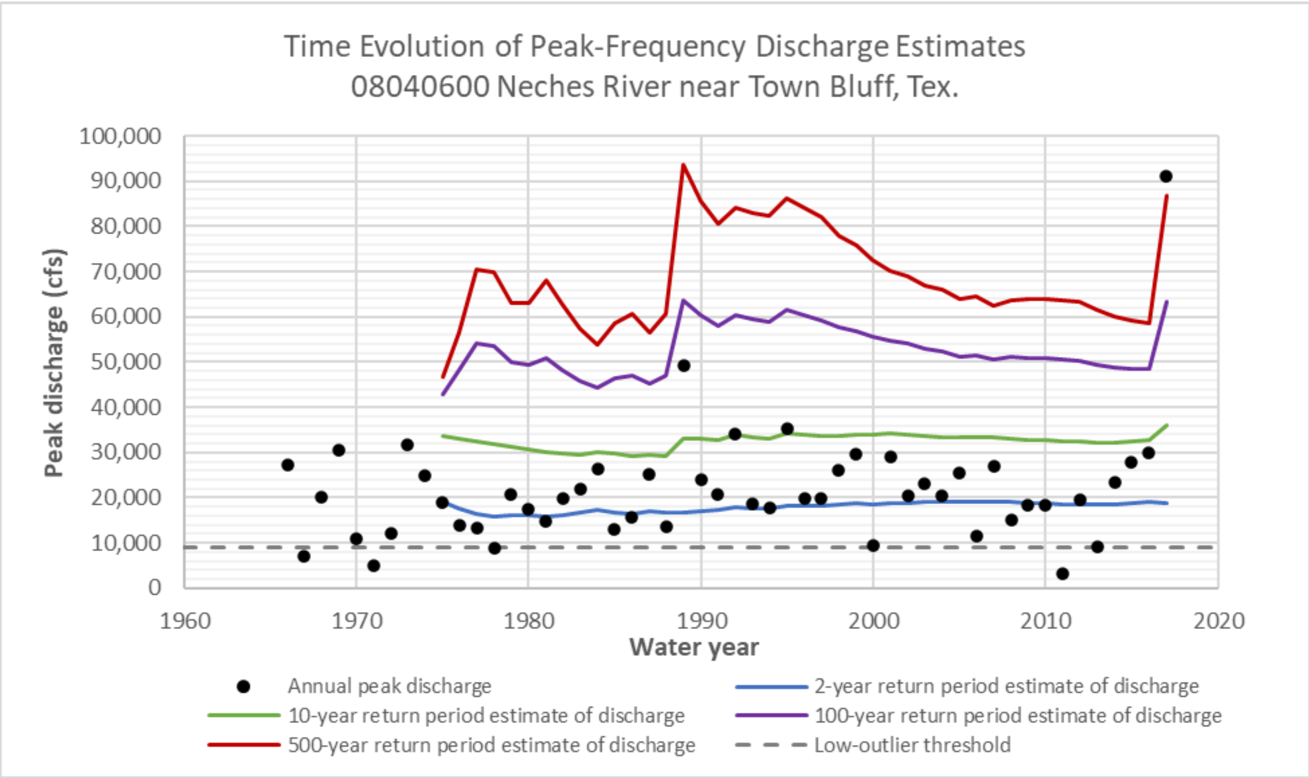


Figure 5.17: Statistical Frequency Flow Estimates versus Time for U.S. Geological Survey Streamgaging Station 08040600 Neches River near Town Bluff, Tex.

08041500 Village Creek near Kountze, Tex.

The relative effects of record length and magnitudes of substantial flood effects for Village Creek near Kountze gage can be seen in Figure 5.18. The discharge estimates for the Village Creek near Kountze gage are more erratic when compared to the other analyses in this section. Large changes in the estimates are observed through 1990. After an increase associated with the fourth greatest peak event on record in 2007, the 100-year return estimate appears to reach a steady estimate of approximately 88,000 cfs. However, the 2017 peak event associated with Hurricane Harvey is nearly three times greater than the next greatest peak event, which causes a pronounced increase in the 100 and 500-year return period estimates.

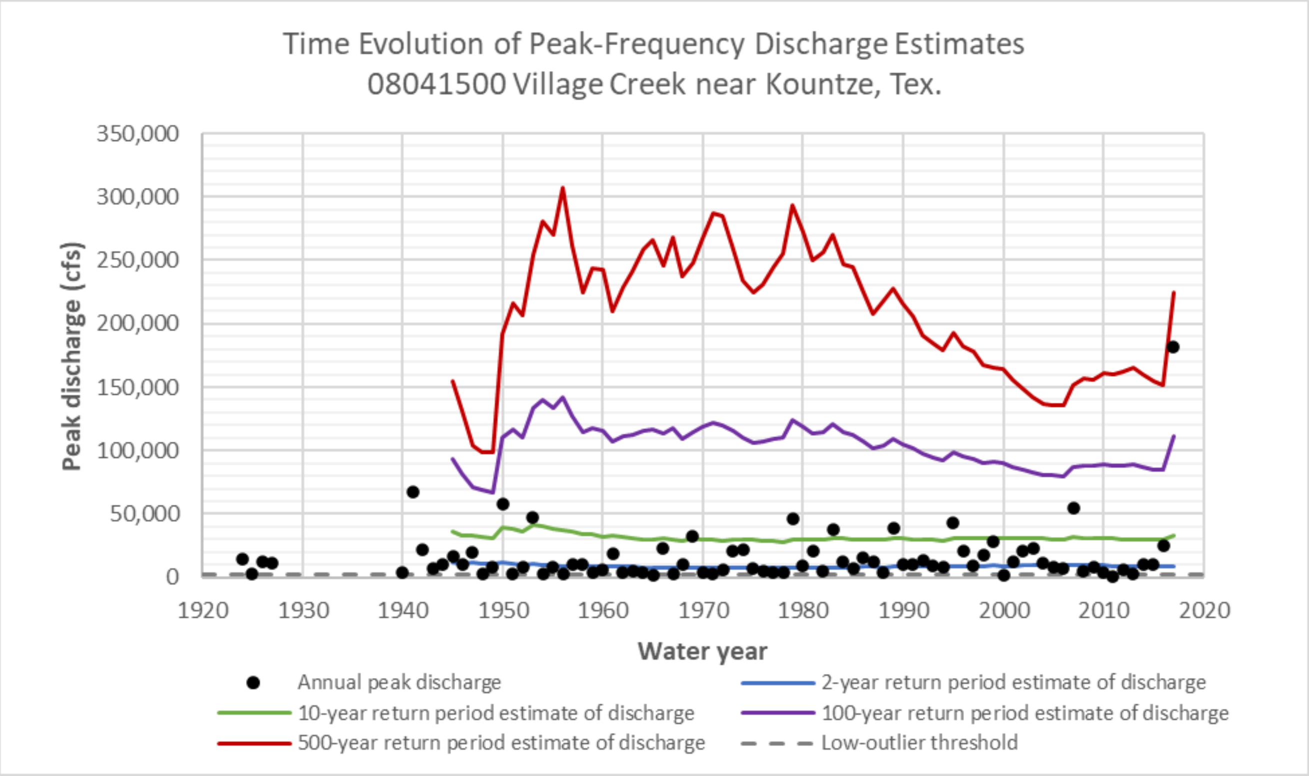


Figure 5.18: Statistical Frequency Flow Estimates versus Time for U.S. Geological Survey Streamgaging Station 08041500 Village Creek near Kountze, Tex.

6 Rainfall-Runoff Modeling in HEC-HMS

Rainfall-runoff watershed modeling is used to simulate the physical processes that occur during storm events that move water across the land surface and through the streams and rivers. While the statistical analyses of the gage records from the previous chapter are a valuable means of estimating the magnitude of flood frequency flows at the gages, watershed rainfall-runoff modeling is often used to estimate the rare frequency events whose return periods exceed the gaged period of record as well as to account for non-stationary watershed conditions such as urban development, reservoir storage and regulation, and climate variability. Rainfall-runoff modeling also provides a means of estimating flood frequency flows at other locations throughout the watershed that do not coincide with a stream flow gage.

In this phase of the multi-layered hydrologic analysis, a rainfall-runoff model was developed for the Neches River Basin with input parameters that represented the physical characteristics of the watershed. The rainfall-runoff model for the basin was completed using the basin-wide Hydrologic Engineering Center – Hydrologic Modeling System (HEC-HMS) model developed for USACE's 2015 Neches River Basin Corps Water Management System (CWMS) Implementation as a starting point (USACE, 2015). This model was further refined by adding additional detailed data, updating the land use, and calibrating the model to multiple recent flood events. Through calibration, the updated HEC-HMS model was verified to accurately reproduce the response of the watershed to multiple recent observed storm events, including those similar in magnitude to a 1% annual chance (100-yr) storm. Finally, frequency storms were built using the depth area analysis in HEC-HMS and the latest published frequency rainfall depths from National Oceanic and Atmospheric Administration (NOAA) Atlas 14 (NOAA, 2018). These frequency storms were run through the calibrated model, yielding consistent estimates of the 1% annual chance (100-yr) and other frequency peak flows at various locations throughout the basin.

This chapter provides a general summary of the model development, calibration and results of the HEC-HMS rainfall runoff modeling that was completed for the InFRM Watershed Hydrology Assessment of the Neches River Basin, but additional details on the development and application of the HEC-HMS model are available in Appendix B: HEC-HMS Model Development and Uniform Rainfall Frequency Results. In addition to the uniform rainfall frequency storm results presented in this chapter, the InFRM team also developed elliptical frequency storms for stream reaches with drainage areas greater than 400 square miles in the Neches River Basin. The results from the elliptical frequency storms in HEC-HMS are presented in Chapter 7 of this report and in Appendix C: Elliptical Frequency Storms in HEC-HMS.

6.1 EXISTING HEC-HMS MODELS

The existing HEC-HMS model from the Neches CWMS Implementation was used as the starting point for the current study. The CWMS model contained 46 subbasins in the Neches River Basin above the Saltwater Barrier near Beaumont, Texas and totaled approximately 9,859 square miles. The subbasins were delineated using the HEC-GeoHMS program and utilized 30-meter National Elevation Dataset (NED) terrain data. The Neches CWMS HEC-HMS model used the following methods:

- Losses – Deficit and Constant
- Transform – ModClark
- Baseflow – Recession
- Routing – Modified Puls & Muskingum
- Computation Interval – 60 minutes

A map of the Neches CWMS subbasins is shown in Figure 6.1. More information on the CWMS model development is given in the final CWMS implementation report for the Neches River Basin (USACE, 2015).

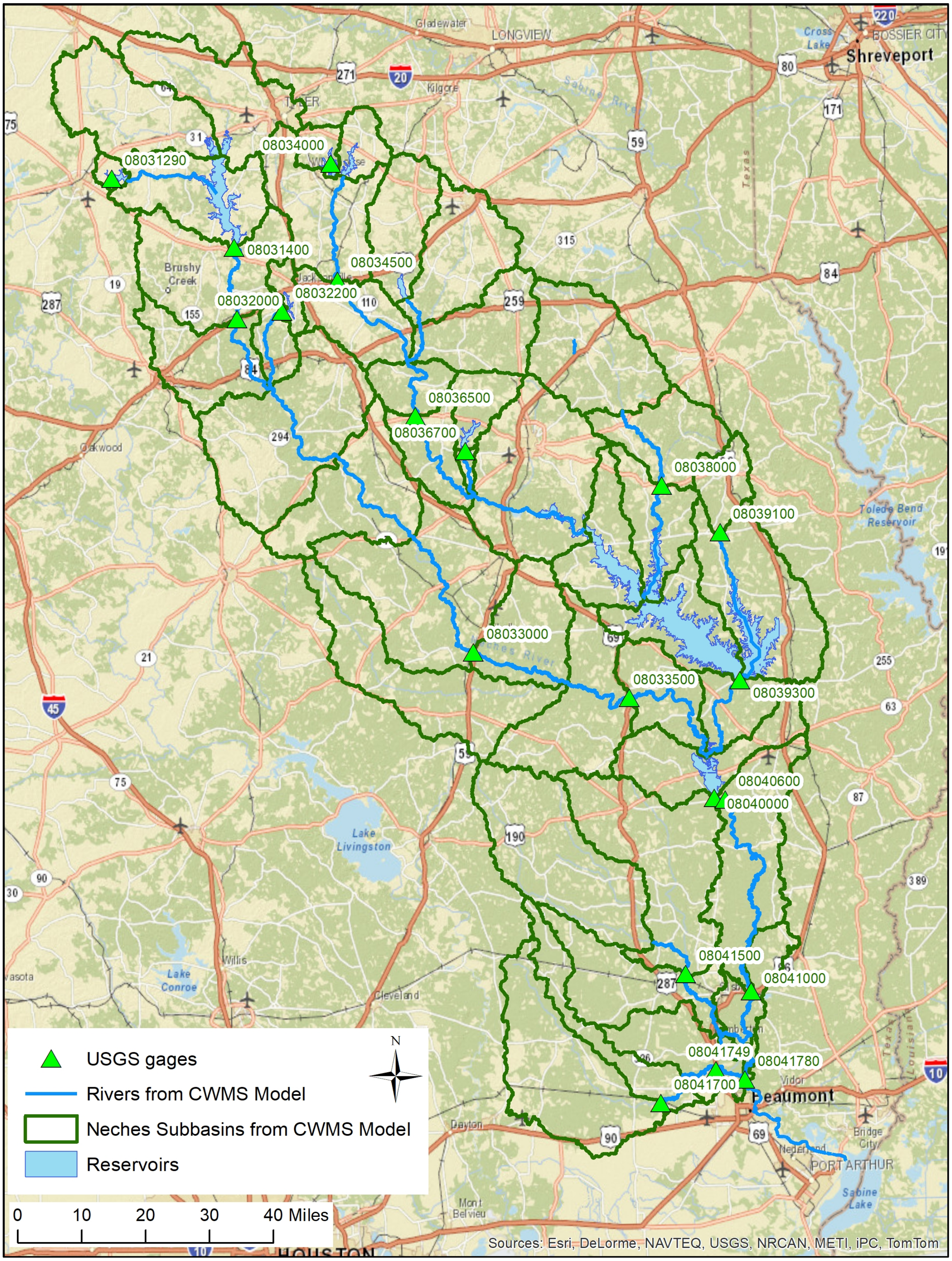


Figure 6.1: Existing CWMS Subbasins for the Neches River Basin

6.2 UPDATES TO THE HEC-HMS MODEL

To better define the hydrology of the Neches River Basin, additional subbasin breaks were added to the original CWMS delineation in order to have better definition of the flow change locations. The number of subbasins in the basin was increased from 46 to 93, with break points primarily at major tributaries, major roads and stream gages. Figure 6.2 shows the final HEC-HMS subbasin delineation for the InFRM Watershed Hydrology Assessment for the Neches River basin. The subbasin sizes in the final HEC-HMS model varied from 10 to 300 square miles, with a median subbasin size of 89 square miles.

After breaking out the additional subbasins, detailed routing data was added to the HEC-HMS model for the associated new river reaches. New detailed Modified Puls routing data was developed throughout the basin using the recent 2017 LiDAR elevation data acquired by FEMA for the Neches River basin. The Modified-Puls routing method calculates the change in flow through the reach based on the volume of floodplain storage through that reach. The new detailed Modified Puls routing data was used to replace the existing Muskingum routing data and the previous Modified Puls routing data, which had been developed from 10-meter NED data.

Finally, after adding all of the above detailed data, the loss method was updated from deficit constant to initial and constant, since the focus of this study is on single storm events, whereas the original CWMS model was used for multi-storm event real-time forecasting. Both of these methods have been found to adequately capture the range of observed losses experienced in Texas from extreme drought to 100% saturated soil conditions and are also simple to adjust for real-time forecasting purposes.

The computation interval of the model was also decreased from 60 to 15 minutes to show more refinement of the hydrographs on the smaller tributaries. The final Neches HEC-HMS model was run in HEC-HMS version 4.3 and used the following methods:

- Losses – Initial and Constant
- Transform – ModClark
- Baseflow – Recession
- Routing – Modified Puls from LiDAR
- Computation Interval – 15 minutes

The Neches HEC-HMS model also includes eight significant reservoirs, which were modeled as reservoir elements in HEC-HMS. These reservoirs are Lake Athens, Lake Palestine, Lake Jacksonville, Lake Striker, Lake Tyler, Lake Nacogdoches, Sam Rayburn Reservoir, and B.A. Steinhagen Lake. While the National Inventory of Dams (NID) shows that approximately 300 dams exist within Neches River basin (USACE, 2016), these eight reservoirs were selected to be modeled in detail due to their sizable flood storage and their noticeable influence on discharges in the major rivers downstream.

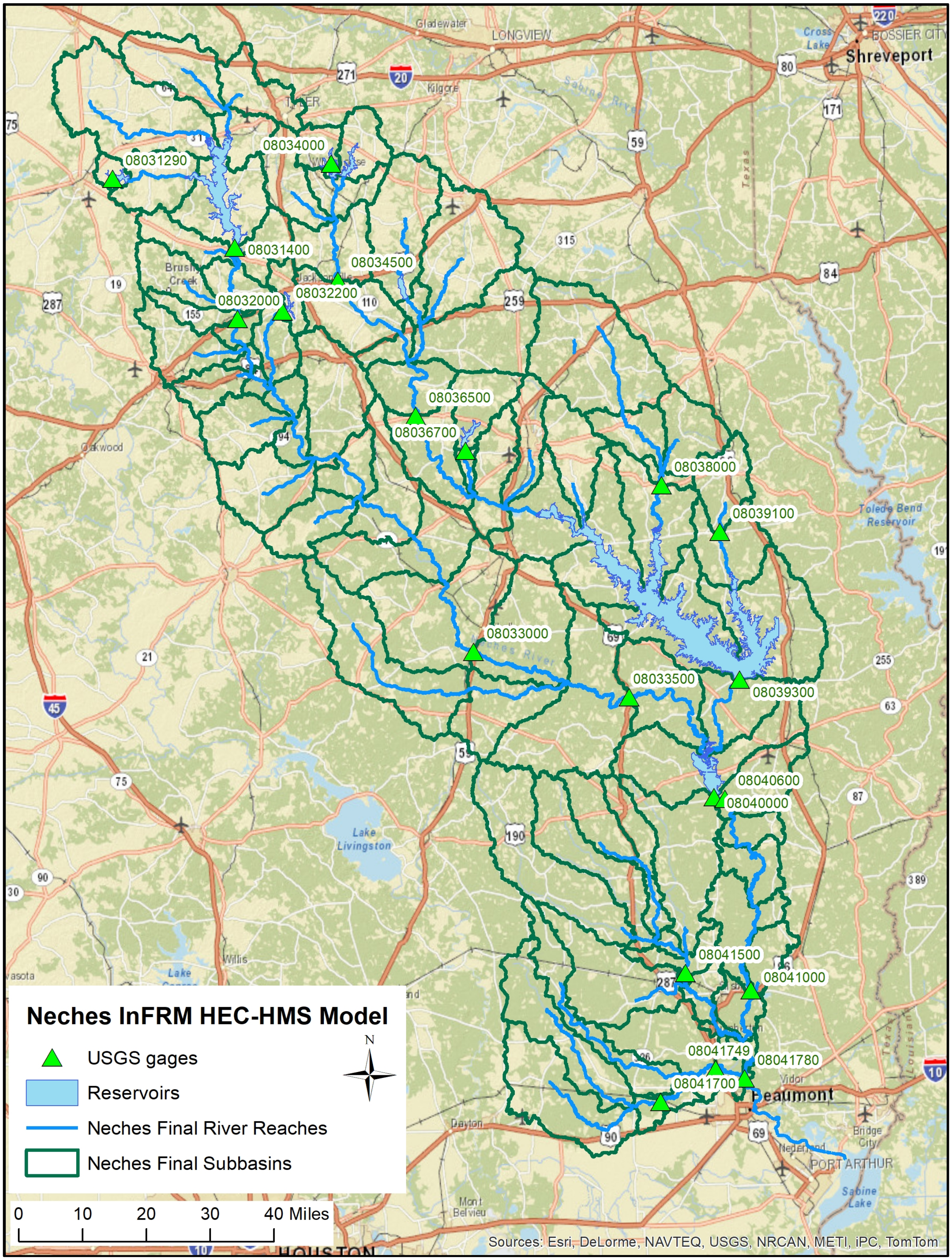


Figure 6.2: Final InFRM HEC-HMS Subbasins for the Neches River Basin

6.3 HEC-HMS MODEL INITIAL PARAMETERS

6.3.1 Subbasin and Routing Initial Parameters

The Neches River HEC-HMS model contains 93 subbasins totaling about 9,859 square miles above the Saltwater Barrier on the Neches River. The subbasins were delineated using the HEC-GeoHMS program and utilized 30-meter NED terrain data. The Neches River HEC-HMS model used initial and constant losses, ModClark transform parameters, recession baseflows, and Modified Puls routing. The sources of the initial estimates for these parameters are described below.

- **Initial Loss and Constant Loss Rate** – For calibration, the initial and constant losses were initially set to zero and then increased according to the antecedent conditions during each event. The calibrated initial and constant losses are inherently influenced by the soil moisture conditions occurring at that particular point in time and therefore varied for each observed event. More information on the initial and constant loss adjustments is found in Section 6.4.2. For the frequency storms, the initial and constant loss rates were calculated based on Natural Resources Conservation Service (NRCS) soil type, according to the Fort Worth District Loss Rate equations, which vary the loss rates by frequency. More information on the losses for the frequency events is given in Section 6.5.

- **Percent Impervious** – The percent impervious values were developed based on the 2011 National Land Cover Database (NLCD) percent developed impervious dataset, which was the dataset that was available at be beginning of this study. Later, this data was compared to the 2016 NLCD dataset and very little difference was observed in the Neches study area.

- **ModClark Transform Parameters** – Regional equations for estimating unit hydrograph parameters were available for the Snyder's unit hydrograph method. Therefore, transform parameters were initially estimated with the Snyder's regional equations and were then converted to gridded ModClark parameters for input into the HEC-HMS model. The process for developing and converting these transform parameters is described below.

Transform parameters were initially developed from regional equations for the Snyder's unit hydrograph method based on watershed characteristics such as length of slope that were extracted from HEC-GeoHMS. From this data, two regional equations were used to develop initial estimates of lag time for the Snyder unit hydrographs.

The first regression equation was developed during the Sam Rayburn and B.A. Steinhagen Dam Assurance Study that was performed prior to development of the HEC-1 Forecast model in 1998. This equation results in relatively long, slow lag times that are characteristic of some portions of the Neches River Basin, and it was used to calculate initial lag times below B.A. Steinhagen Reservoir.

The following regional equation was used to calculate subbasin lag times below B.A. Steinhagen Lake:

$$T_p = 2.9\left(\left(\frac{L * L_{ca}}{S_{st}}\right)^{0.5}\right)^{0.38}$$

where:

T_p = Snyder's lag time (hours)

L = longest flow path within the subbasin (miles)

L_{ca} = distance along the stream from the subbasin centroid to outlet (miles)

S_{st} = stream slope over reach between 10% and 85% of L (feet per mile)

The Snyder's peaking coefficients were set to a value of 0.625 for all subbasins below B.A. Steinhagen Lake, based on the 1998 HEC-1 Neches forecast model.

The second regional equation that was used to develop initial estimates of lag time for the Snyder unit hydrograph was from the U.S. Army Corps of Engineers (USACE) Fort Worth District urban studies (Nelson, 1979) (Rodman, 1977) (USACE, 1989). This equation estimates subbasin lag time based on the length and slope of the watershed, the percent urban values taken from land cover data, and the percent sand values estimated from the NRCS soil data. This equation results in quicker lag times that are more characteristic of the headwater portions of the Neches River basin, and it was used to calculate initial lag times for subbasins above B.A. Steinhagen Lake.

The following regional equation was used to calculate subbasin lag times above B.A. Steinhagen Lake:

$$\log(Tp) = .383\log(L*Lca/(Sst \wedge .5))+(Sand*(\log 1.81-\log .92)+\log .92)-(BW*Urban./100)$$

where: Tp = Snyder's lag time (hours)

L = longest flow path within the subbasin (miles)

Lca = distance along the stream from the subbasin centroid to outlet (miles)

Sst = stream slope over reach between 10% and 85% of L (feet per mile)

Sand = percentage of sand factor as related to the permeability of the soils

(0% Sand = low permeability, 100% Sand = high permeability)

BW = log(tp) bandwidth between 0% and 100% urbanization = 0.266 (log hours)

Urban. = percentage urbanization factor

The Snyder's peaking coefficients were set to a value of 0.5 for all subbasins above B.A. Steinhagen Lake, based on the recommendations of Design Memorandum No. 15 on Sam Rayburn Reservoir, Angelina River, Texas, Hydrology (Revised), (USACE, 1958).

After developing the Snyder's unit hydrograph parameters, these values were added to the HEC-HMS model and the program was run. The HEC-HMS program internally converted Snyder's unit hydrograph to Clark's unit hydrograph parameters, and the message bar indicated the resulting Clark's T_c and R values that were calculated internally to produce the unit hydrograph. The transform method was then switched to ModClark, and the T_c and R values which were calculated within the HEC-HMS program were subsequently used.

- **Baseflow Parameters** – Initial baseflow parameters were taken from the existing USACE Neches CWMS HEC-HMS model.

- **Routing Parameters (Modified Puls)** – Storage-discharge curves for the Modified Puls routing were extracted from new hydraulic routing models in HEC-RAS, which were developed from the 2017 LiDAR data. Initial subreach values were estimated based on the reach length and an average travel time through the reach.

The initial subbasin and routing parameters that were entered into the HEC-HMS model can be seen in Tables B.1 through B.4 of Appendix B. Some of these parameters were adjusted during calibration.

6.3.2 Initial Reservoir Data

According to the National Inventory of Dams (NID), approximately 300 dams exist within Neches River basin, most of which are NRCS structures or other small dams (USACE, 2016). Of these, reservoir elements were used in the HEC-HMS rainfall-runoff model for eight reservoirs in the Neches basin. These dams were selected to be modeled in detail due to their sizable flood storage and their noticeable influence on discharges in the major rivers downstream. Table 6.1 summarizes the reservoir data obtained for these dams and their corresponding data sources, and Figure 6.3 illustrates their locations within the basin.

The eight modeled reservoirs include two USACE reservoirs, Sam Rayburn and B.A. Steinhagen. For these reservoirs, the elevation-storage tables, spillway rating curves, and outlet structure rating curves were all obtained from the USACE Fort Worth District, and the dams were modeled as reservoir elements in HEC-HMS.

The remaining 300 smaller dams were scattered throughout the rural areas of the basin, especially in the headwaters of the Neches and Angelina watersheds. These dams were not modeled in detail but were accounted for in the model through adjustments to the subbasins' initial losses and peaking coefficients. Data for these dams was obtained from the National Inventory of Dams (USACE, 2016).

Table 6.1: Reservoir Data and Sources for Dams Modeled in HEC-HMS

Reservoir Name	Data	Source(s)
Lake Athens	Elevation-Storage, Elevation-Discharge rating	Texas Water Development Board, Dam Design Documentation
Lake Palestine	Elevation-Storage, Elevation-Discharge rating	Texas Water Development Board, Upper Neches River Municipal Water Authority
Lake Jacksonville	Elevation-Storage, Elevation-Discharge rating	Texas Water Development Board, 1978 Texas Dam Inspection Report
Lake Striker	Elevation-Storage, Elevation-Discharge rating	Texas Water Development Board, 2009 Standard Operating Procedure for Striker Floodgate Operations
Lake Tyler	Elevation-Storage, Elevation-Discharge rating	Texas Water Development Board, 1978 Texas Dam Inspection Reports
Lake Nacogdoches	Elevation-Storage, Elevation-Discharge rating	Texas Water Development Board, Texas Commission on Environmental Quality
Sam Rayburn	Elevation-Storage capacity, Spillway and Outlet Structures	USACE - Fort Worth District
B.A. Steinhagen (Town Bluff Dam)	Elevation-Storage capacity, Spillway and Outlet Structures	USACE - Fort Worth District

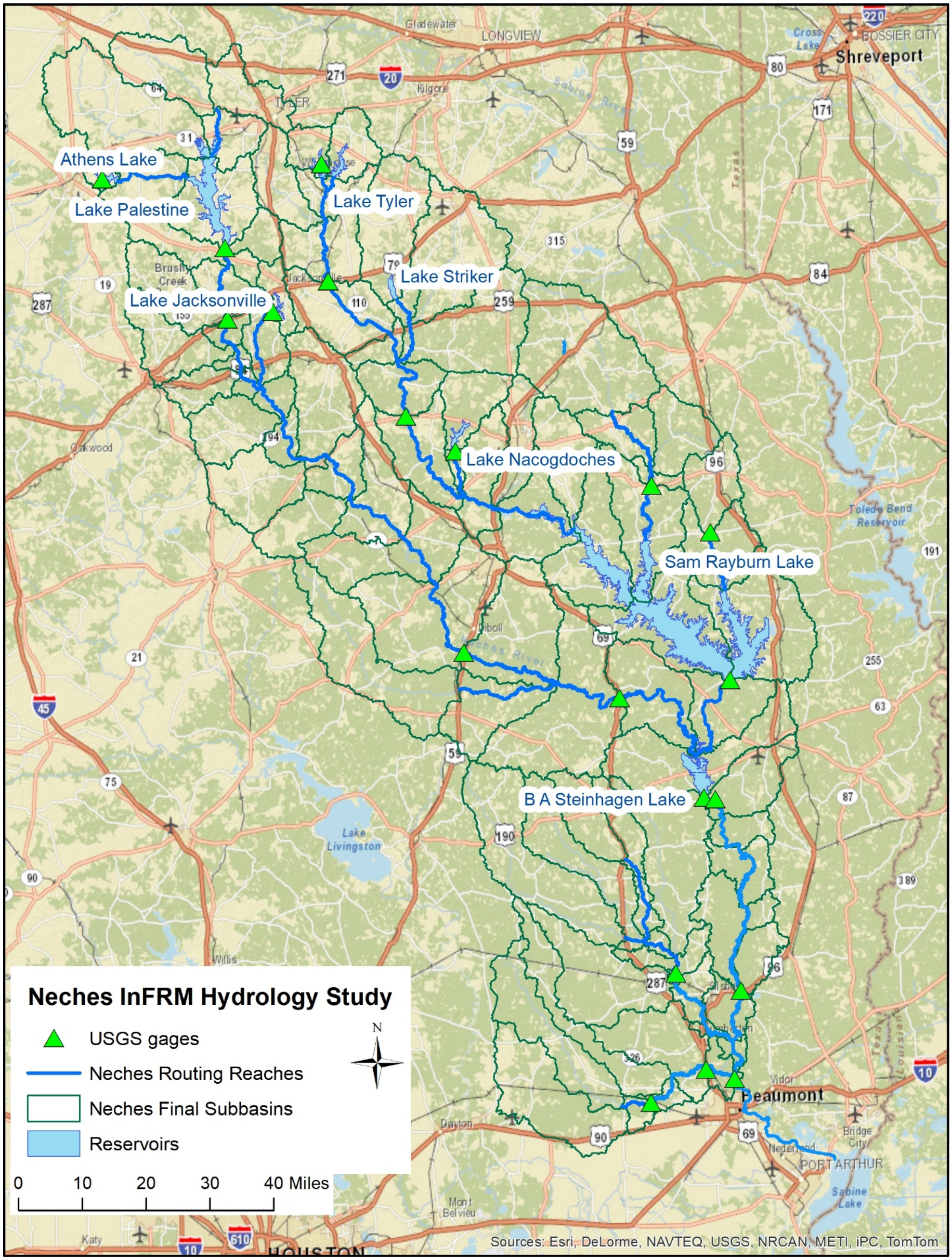


Figure 6.3: Locations of Reservoirs Modeled in HEC-HMS

6.4 HEC-HMS MODEL CALIBRATION

After building the more detailed HEC-HMS model with its initial parameters, the model was calibrated to ensure that it would accurately simulate the response of the watershed to a range of observed flood events, including large events similar to a 1% annual chance (100-yr) flood. The goal of calibration is to accurately simulate the response of the watershed to a given storm by reproducing the timing, shape, and magnitudes of the observed flows at the stream gages. A total of eight recent storm events were used throughout different parts of the watershed to calibrate the model. For these storms, the National Weather Service (NWS) hourly rainfall radar data allowed the team to fine tune the rainfall runoff model through detailed calibration. This radar rainfall data is a gridded product with a spatial resolution of approximately 4 km x 4 km cell sizes, and the rainfall depths are calibrated by the NWS to on-the-ground observations at rainfall gages. Prior to the late 1990s, the NWS radar data was not available for use during earlier modeling efforts. The model calibration and verification process undertaken during this study exceeds the standards of a typical FEMA floodplain study.

6.4.1 Calibration Storms

Table 6.2 lists the storms that were used to calibrate each portion of the watershed, and Figures 6.4 through 6.11 illustrate the total depth of rain for the major calibration storms and how that rain was distributed spatially throughout the Neches River watershed. These plots were extracted from the HEC-MetVue meteorological program for visualizing and processing rainfall data. These storms were selected as the largest available storms during the time that NWS radar data was available.

October 2006 was a significant flood event on the lower Neches basin. In October 2006, 15” to 18” of rainfall fell over a 3-day period, and 24-hour totals of 8” to 10” were recorded by the NWS radar data. This was the second highest flood event of record at 3 out of 5 stream gages in the lower Neches River basin below Town Bluff Dam.

May-June 2015 was a period of heavy rainfall that impacted all portions of the Neches River basin. Rainfall totals ranged from 10” to 20” across the Neches River basin over a three week period.

Hurricane Harvey (Aug-Sep 2017) was the flood of record for 4 out of 5 stream gages in the lower portion of the Neches basin below Town Bluff Dam. The NWS recorded 26” to 40” of rainfall over the 5-day period of Aug 26-30, 2017 in the lower portions of the Neches River basin with higher amounts towards the coast.

Tropical Storm Imelda (Sep 2019) was a significant storm event in the Pine Island Bayou watershed. The NWS recorded 13” to 27” of rainfall in those portions of the Neches watershed over a 2-day period.

These and other storms listed in Table 6.2 were used to calibrate the Neches River rainfall runoff model. Since the rain fell on different parts of the basin from one historic storm event to another, the calibration of each storm was focused on those areas of the basin that received the greatest and most intense rainfall. Calibration was also only performed when the USGS stream gages were recording and experienced a significant peak flow for that event. Table 6.3 shows which storms were calibrated for each USGS stream and reservoir gage location.

Table 6.2: Storm Events Used for Model Calibration

Historic Storm Event	Portion of the Neches River Basin that was Calibrated		
	Neches River above Town Bluff Dam	Angelina River above Town Bluff Dam	Neches River below Town Bluff Dam
Oct 2006			Yes
Jun-Jul-2007	Yes	Yes	
May-Jun 2015	Yes	Yes	Yes
Mar-2016	Yes	Yes	Yes
Apr-May-2016	Yes	Yes	
May-Jun-2016	Yes	Yes	Yes
Aug-Sep 2017	Yes	Yes	Yes
Sep 2019			Yes

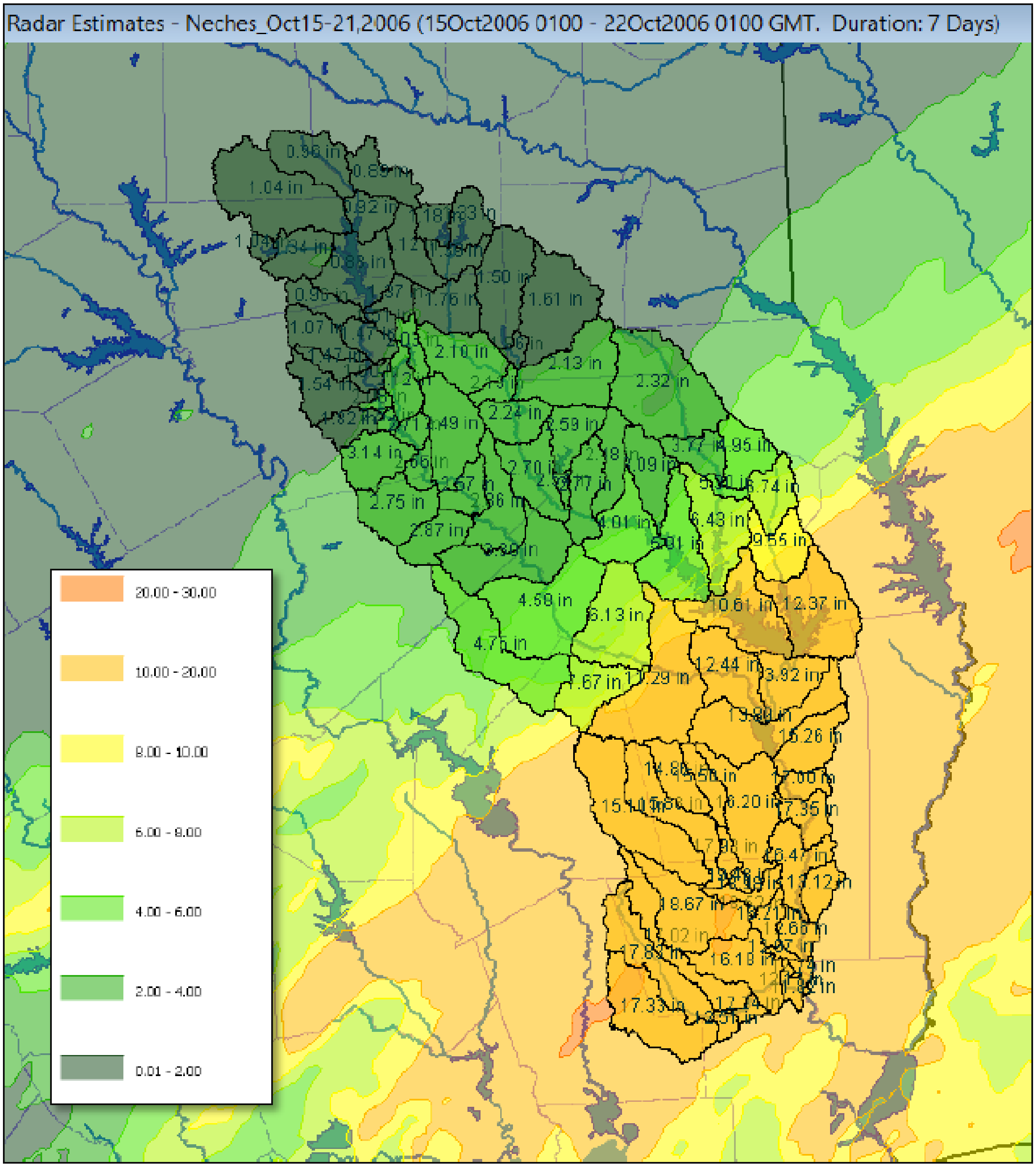


Figure 6.4: Total Rainfall Depths (inches) for the October 2006 Calibration Storm

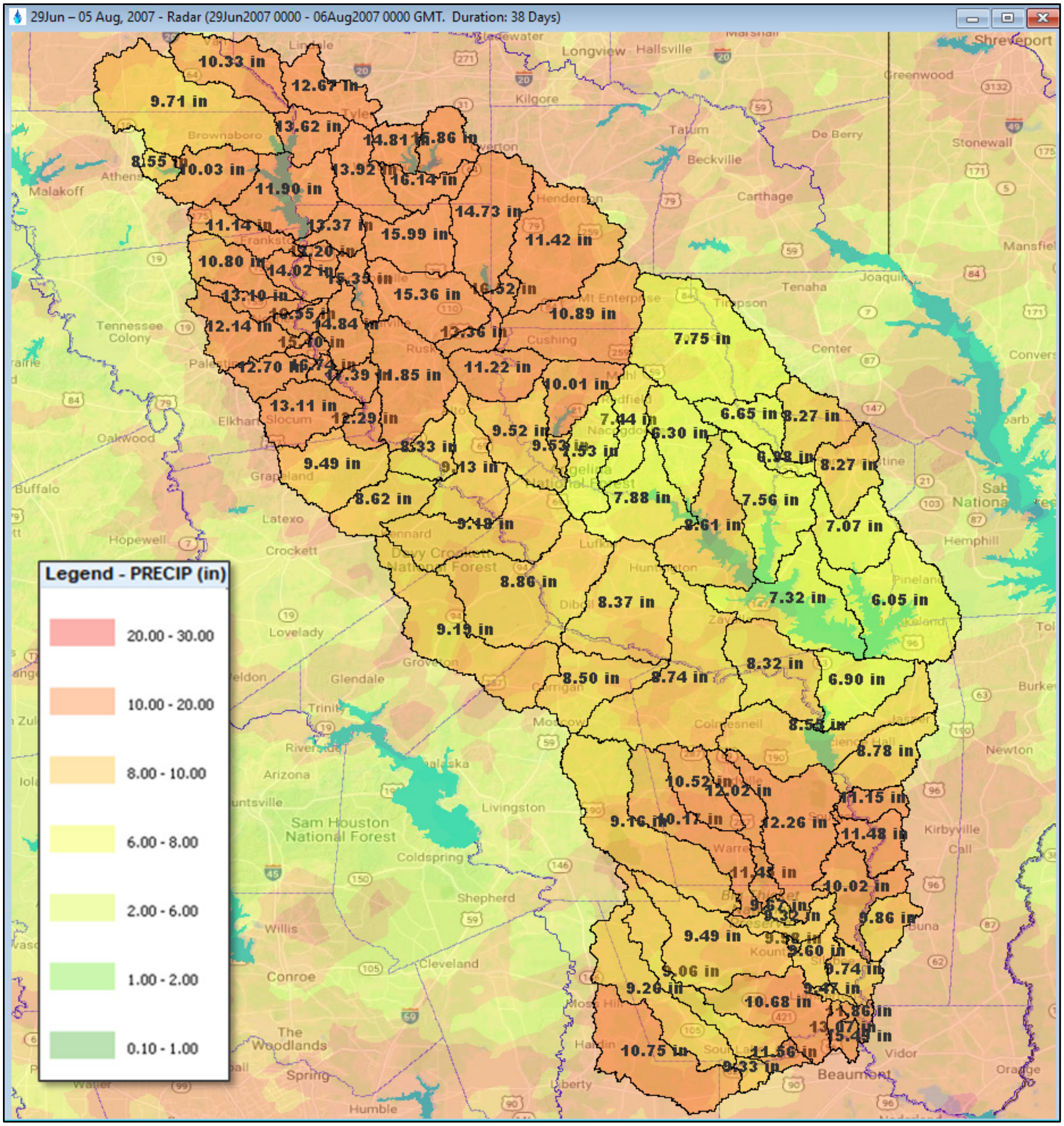


Figure 6.5: Total Rainfall Depths (inches) for the Jun-Jul 2007 Calibration Storm

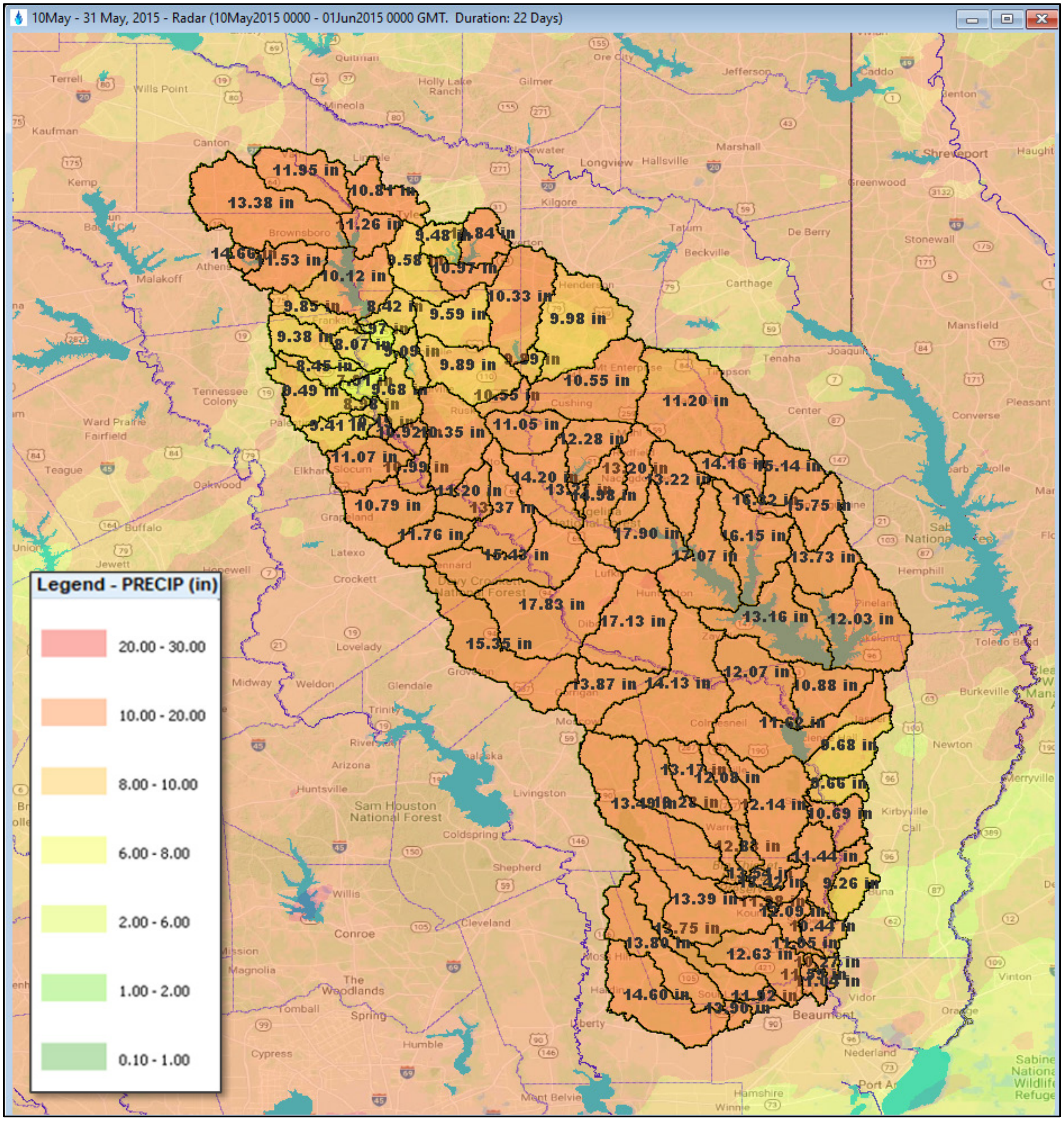


Figure 6.6: Total Rainfall Depths (inches) for the May-Jun 2015 Calibration Storm

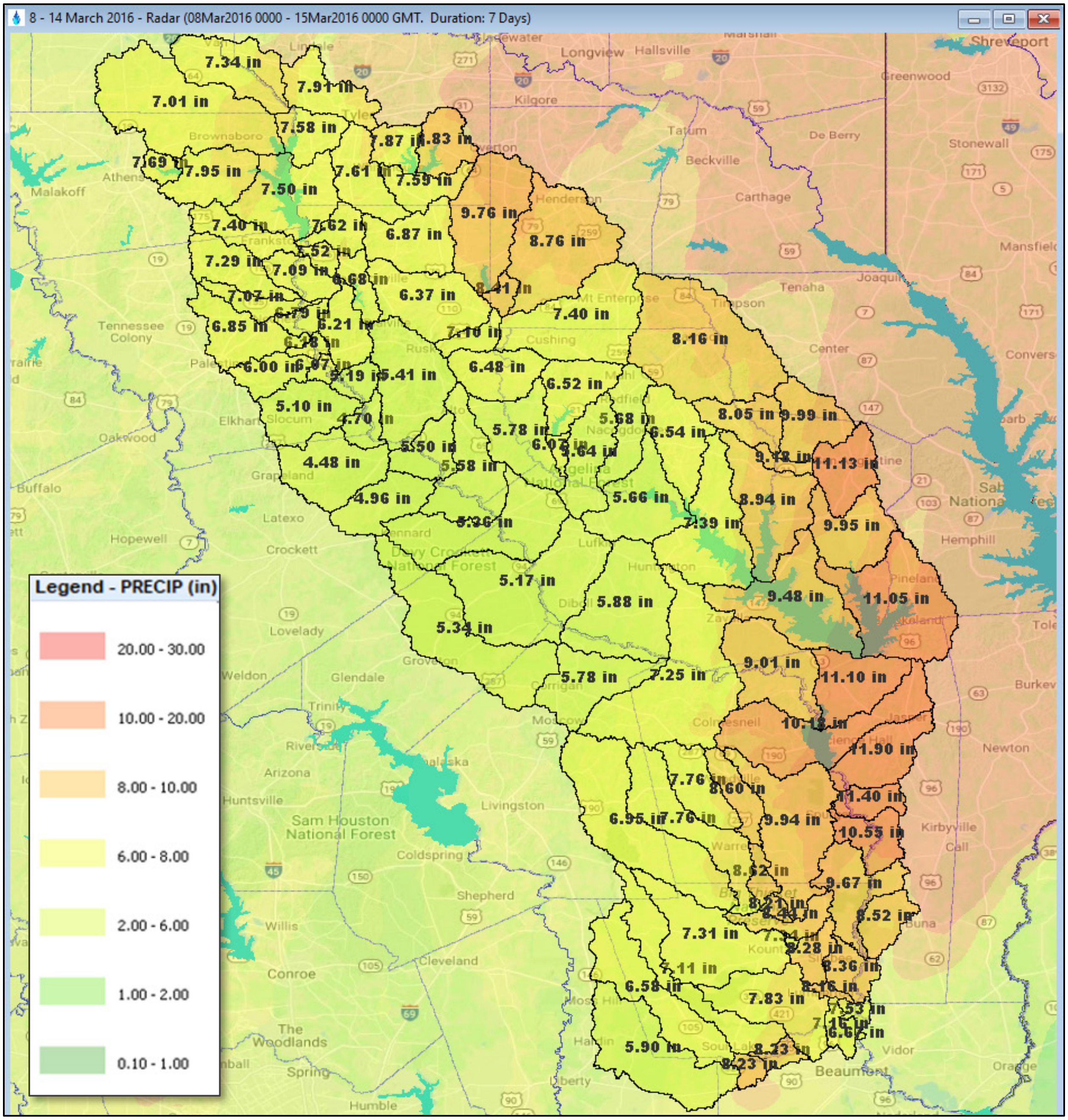


Figure 6.7: Total Rainfall Depths (inches) for the Mar 2016 Calibration Storm

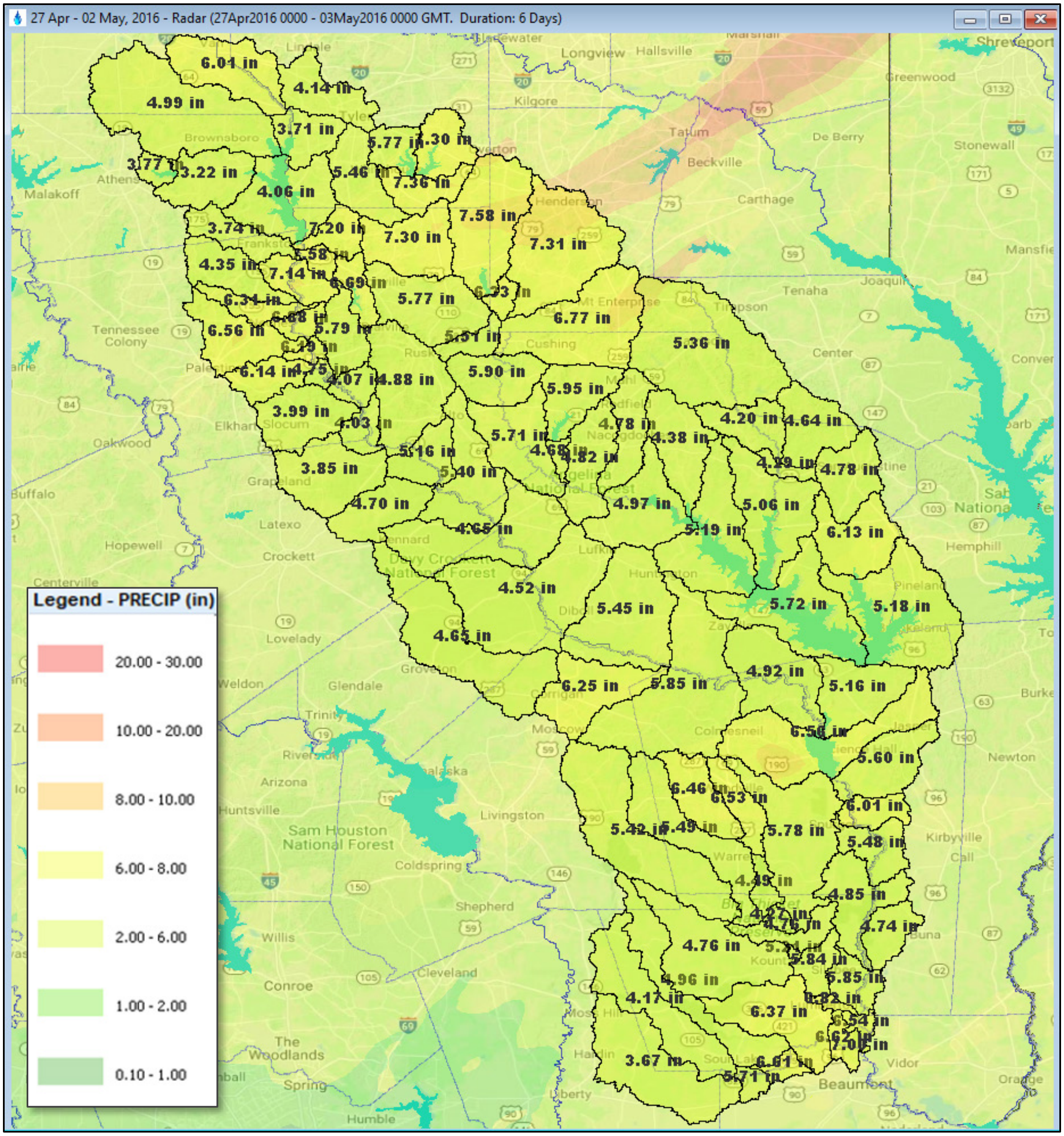


Figure 6.8: Total Rainfall Depths (inches) for the Apr-May 2016 Calibration Storm

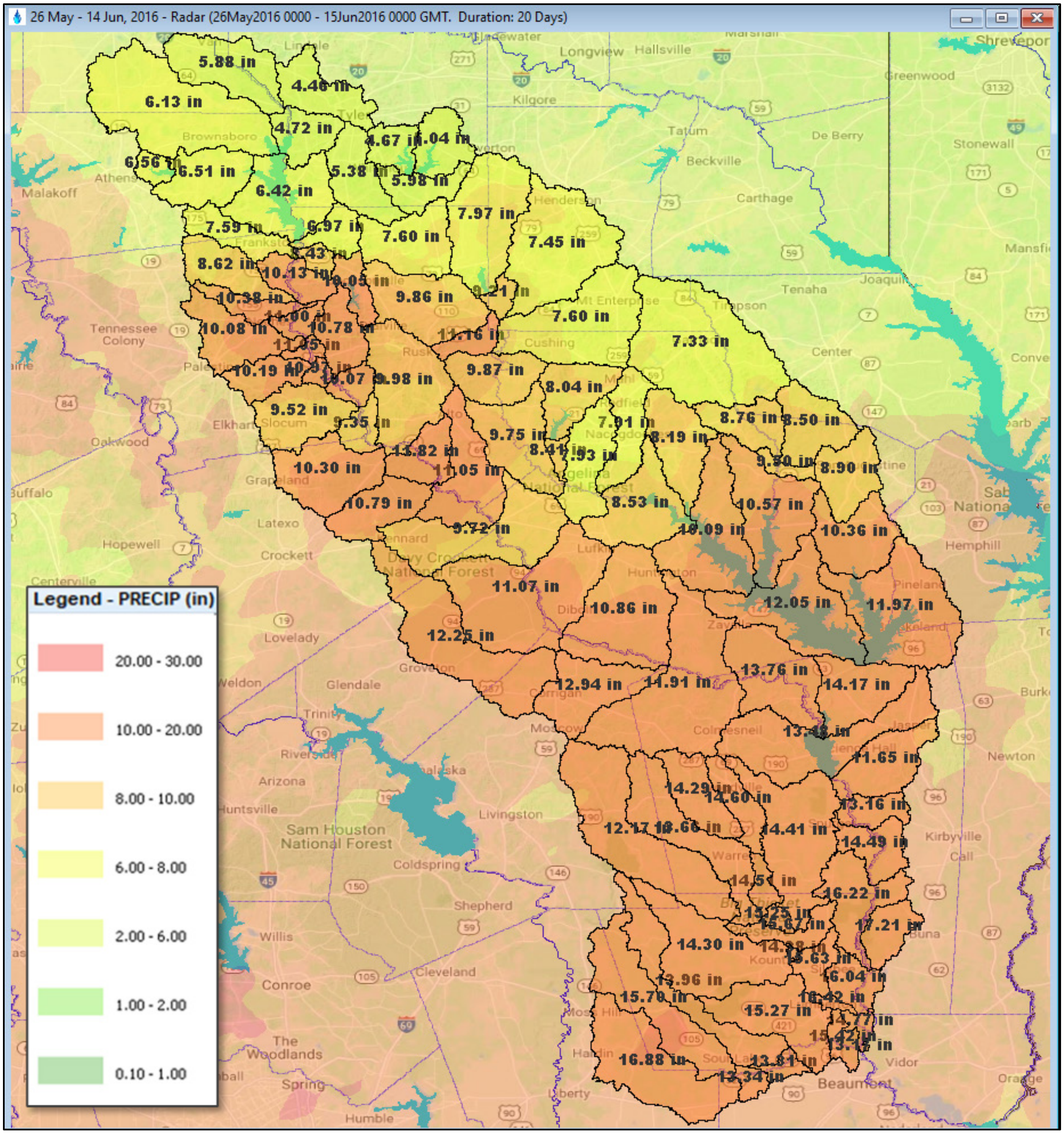


Figure 6.9: Total Rainfall Depths (inches) for the May-Jun 2016 Calibration Storm

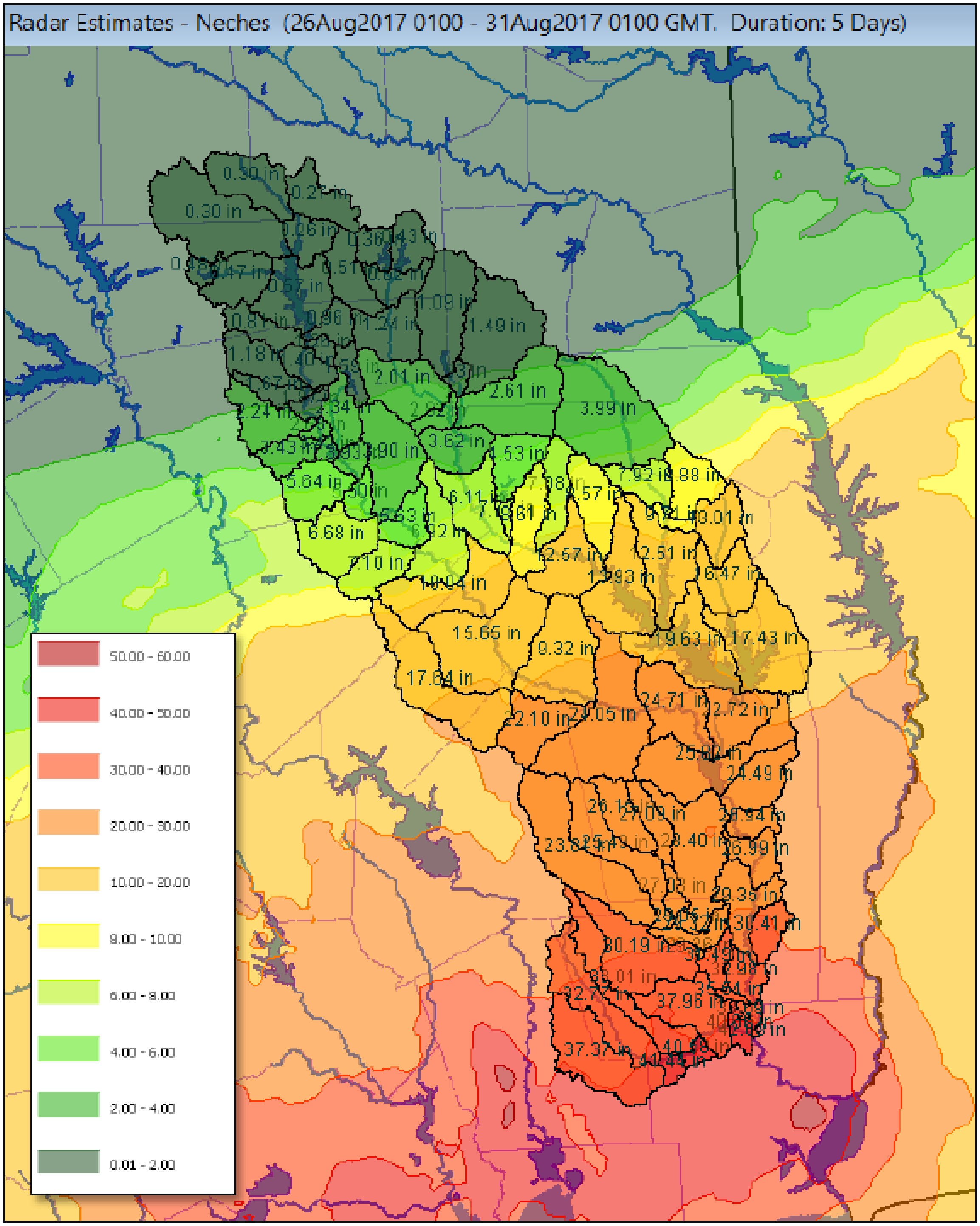


Figure 6.10: Total Rainfall Depths (inches) for Hurricane Harvey, Aug-Sep 2017, Calibration Storm

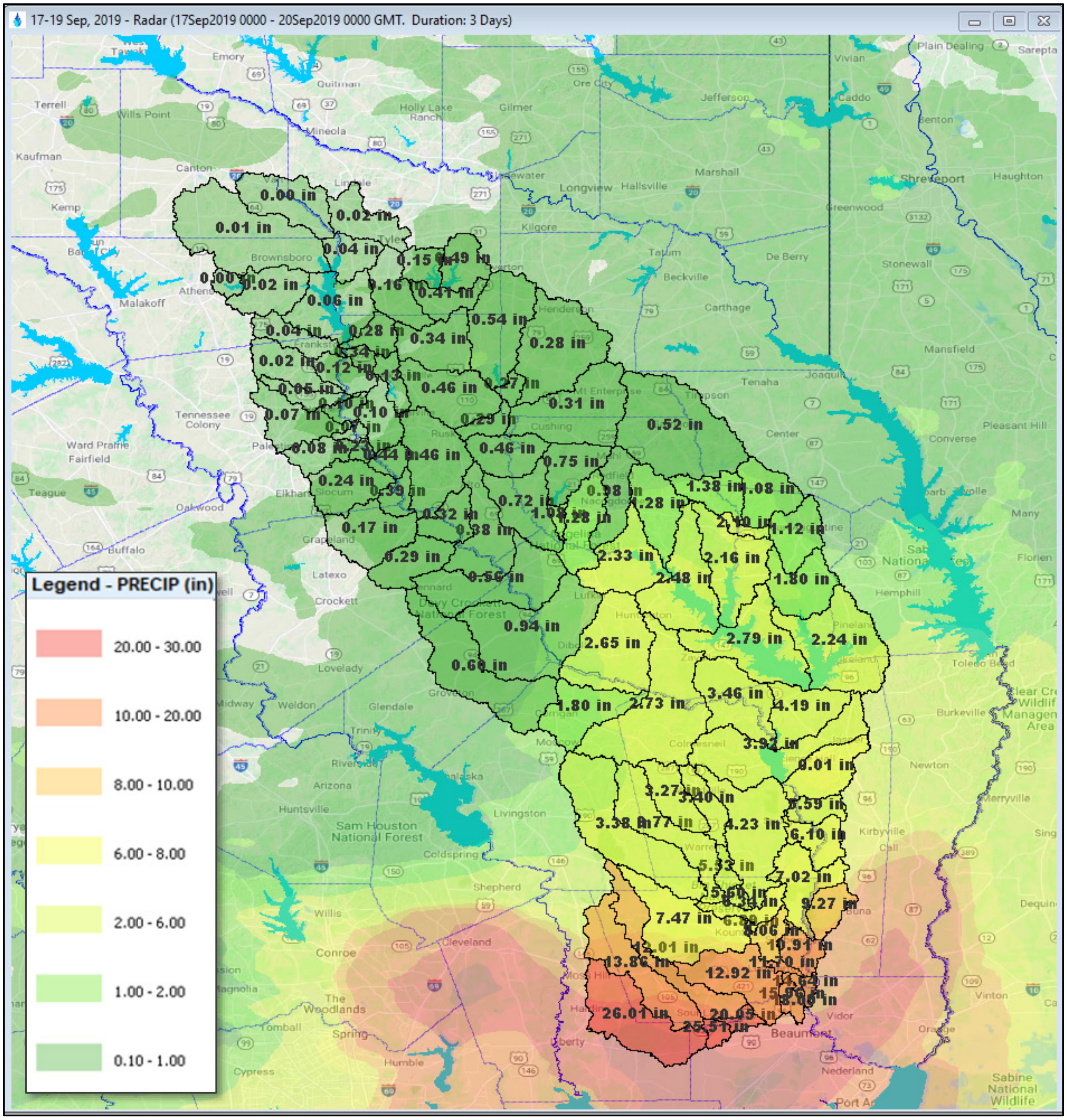


Figure 6.11: Total Rainfall Depths (inches) for Tropical Storm Imelda, Sep 2019, Calibration Storm

Table 6.3: Calibrated Storm Events for Specific Gage Locations

USGS Gage Location	Peak Observed Stream Flow (cfs) or Reservoir Elevation (ft NAVD 88)							
	Oct 2006	Jun-Jul 2007	May-Jun 2015	Mar 2016	Apr-May 2016	May-Jun 2016	Aug-Sep 2017	Sep 2019
Lake Athens	-	441.4	441.5	441.9	440.8	-	-	-
Lake Palestine	-	349.0	347.8	349.7	347.0	-	-	-
Neches River nr Neches, TX USGS Gage 08032000 at US-79 bridge	-	19,200	10,200	21,000	10,700	-	-	-
Lake Jacksonville	-	424.8	422.8	422.9	424.4	424.6	-	-
Neches River nr Diboll, USGS gage 08033000 at US-59 bridge	-	-	24,800	17,300	14,100	21,000	-	-
Neches River near Rockland, USGS gage 08033500 at US-69 bridge	-	20,700	28,700	20,800	16,000	29,600	37,900	-
Lake Tyler	-	378.1	376.3	377.2	378.1	-	-	-
Mud Creek near Jacksonville, USGS gage 08034500 at US-79 bridge	-	18,600	3,420	11,900	16,700	-	-	-
Angelina River near Alto, USGS gage 08036500 at TX-21 bridge	-	30,000	8,600	25,800	23,900	-	-	-
Lake Nacogdoches	-	280.2	281.1	282.5	282	-	-	-
Attoyac Bayou nr Chireno, USGS gage 08038000 at TX-21 bridge	-	1,190	8,290	17,700	5,550	-	-	-
Ayish Bayou near San Augustine, USGS gage 08039100	-	1,590	4,040	11,100	1,280	5,380	15,700	-
Sam Rayburn Reservoir	-	166.9	173.8	170.8	169.0	169.8	167.3	-
Town Bluff Dam	-	82.8	83.7	83.5	82.8	83.8	84.0	-
Neches River nr Town Bluff, USGS Gage 08040600	26,900	-	28,000	30,000	-	29,000	67,900	-

	Peak Observed Stream Flow (cfs) or Reservoir Elevation (ft NAVD 88)							
USGS Gage Location	**Oct 2006**	**Jun-Jul 2007**	**May-Jun 2015**	**Mar 2016**	**Apr-May 2016**	**May-Jun 2016**	**Aug-Sep 2017**	**Sep 2019**
Neches River at Evadale, USGS Gage 08041000	33,800	-	31,200	38,600	-	39,700	71,800	9,330
Village Creek near Kountze, USGS gage 08041500 at FM 418 bridge	55,200	-	7,930	24,600	-	17,500	182,000	2,610
Pine Island Bayou near Sour Lake, USGS gage 08041700	9,920	-	3,750	4,390	-	9,800	47,800	33,400
Pine Island Bayou above BI Pump Plant, USGS Gage 08041749	18,300	-	8,170	9,570	-	18,400	71,500	29,500
Neches River at the Saltwater Barrier, USGS Gage 08041780	90,800	-	48,000	63,100	-	74,500	232,000	46,500

6.4.2 Calibration Methodology

Following the initial parameter estimates, calibration simulations were made using observed hourly Next-Generation Radar (NEXRAD) Stage IV gridded precipitation data obtained from the West Gulf River Forecast Center (WGRFC). For each storm event, the model's calculated flow hydrographs were compared to the observed USGS stream flow data at the gages. The model's parameters were then adjusted to improve the match between the simulated and observed hydrographs for the observed events. Calibration was performed for the 8 storm events previously listed in Table 6.2. Subbasin parameters that were adjusted during calibration included the subbasins' initial and constant loss rates, ModClark time of concentration and storage coefficients, and baseflow parameters. For the routing reaches, the Modified Puls number of subreaches were adjusted as needed.

Calibration was generally performed from upstream to downstream, with all subbasins upstream of a specific gage receiving uniform adjustments, unless specific rainfall or observed flow patterns necessitated adjusting subbasin parameters on an individual basis. Generally, subbasin parameters were adjusted in a consistent order: first baseflow parameters, then loss rates, and then time of concentration and storage coefficients. Routing subreaches were the last to be adjusted. The methods of adjustment for each parameter are summarized in Table 6.4.

To the extent possible, effort was made to calibrate the model's results to the volume, timing, peak magnitude, and shape of the observed flow hydrograph. However, imperfections in the observed rainfall data and streamflow data did not always allow for a perfect match. For example, the gridded NEXRAD rainfall data from the National Weather Service was only available on an hourly basis. This meant that intense bursts of rain that occurred in 15-min or 30-min timespans might not be adequately represented in the hourly rainfall data. It also meant that even though the model was being run on a 15-min time step, the timing of the hydrographs could only be calibrated to the nearest hour. Likewise, the observed flow values at the gages are calculated indirectly from the observed stage and a limited number of flow measurements. While abundant flow measurements were usually available in the low flow range, the number and quality of USGS flow measurements were often very limited in the high flow range, leading to uncertainty in some of the observed flow hydrographs. In cases where all aspects of the observed flow hydrograph could not be matched simultaneously, priority was given to matching the peak flow magnitude first, followed by the peak timing, which are the aspects of model calibration that are most relevant to the final frequency flow estimation.

Table 6.4: HEC-HMS Calibration Approach

Parameter	Calibration Approach
Baseflow Parameters	First, the baseflow parameters were adjusted to match the observed flow rates at the start and end of each model simulation period. The initial discharges for the subbasins upstream of a certain gage were adjusted uniformly up or down to match the initial observed discharge at that gage. Similarly, the recession constant was adjusted to match the slope of the recession limb of the observed hydrograph, and the ratio to peak was adjusted to match the observed discharge at the end of the calibration event. All baseflow parameters were adjusted uniformly for all subbasins upstream of a given gage.
Initial Loss (in)	After adjusting the baseflow parameters, the initial and constant losses were adjusted to calibrate the total volume of the flood hydrograph. The initial loss was adjusted according to the antecedent soil moisture conditions at the beginning of each observed storm event. The initial loss was increased or decreased until the timing and volume of the initial runoff generally matched the observed arrival of the flow hydrograph at the nearest downstream gage. All subbasins that were upstream of each gage were generally adjusted uniformly, unless specific rainfall and observed flow patterns necessitated adjusting the subbasin initial losses on an individual basis.
Constant Loss Rate (in/hr)	After adjusting the baseflow and initial loss parameters, the constant losses were adjusted to calibrate the total volume of the flood hydrograph. The subbasins' constant loss rates were increased or decreased until the volume and magnitude of the simulated hydrographs generally matched the observed volume of the flow hydrograph at the nearest downstream gage. The combination of the adjusted baseflow and loss rate parameters led to the total calibrated volume of runoff at the gage.
ModClark Time of Concentration (hours)	After adjusting the loss rates, the ModClark Time of Concentrations (T_c) were the next parameters to be adjusted upstream of an individual gage. The ModClark Time of Concentrations were adjusted to match the timing of the observed peak flow at the gage. Normally, all of the subbasin T_c's upstream of an individual gage were adjusted uniformly and proportionally to their initial values, unless the magnitude or shape of the observed hydrograph necessitated making individual adjustments. Efforts were also made to ensure that the adjusted T_c's still fell within a reasonable range, using the equivalent Snyder's lag times from the Fort Worth District regional lag time equations as a guide.
ModClark Storage Coefficient	ModClark Storage Coefficients (R) were adjusted to match the general shape of the observed flow hydrograph as lower storage coefficients produce steeper, narrower flood hydrographs, and higher storage coefficients produce flatter, wider flood hydrographs. An attempt was made to use the same ratio of R to T_c for all subbasins with similar watershed characteristics. For example, steep, hillier subbasins were given a lower R to Tc ratio, whereas flatter subbasins or subbasins with many NRCS dams were given a higher R to Tc ratio. Efforts were also made to ensure that the adjusted ratio of R to T_c storage coefficients fell within a reasonable range of 0.3 to 3.0. Whenever possible, the ratio of R to T_c was adjusted once and then kept consistent between subsequent events.
Modified Puls Routing Subreaches	The number of subreaches in the Modified Puls routing reaches were the final parameters to be adjusted when necessary. Calibration of routing parameters focused on storms that fell near the upstream end of the watershed and were routed downstream with little intervening subbasin flow. Adjustments to the number of subreaches in a given routing reach were made in order to match the amount of attenuation in the peak flow that occurred from the upstream end of a reach to the downstream gage. In a very few cases, where an adjustment to the subreaches was not sufficient to match the observed downstream hydrograph, a factor was also applied to the reach's storage volume in the storage-discharge curve.

In addition to the calibration procedures described above, the watershed above the Angelina River near Alto USGS gage was a location that received additional investigation following the preliminary calibration results. The investigation included a unit hydrograph peaking study performed to improve the accuracy of flood frequency estimates. There were two primary reasons for this investigation. The first reason is that the historic calibration events available for HEC-HMS model calibration were very limited and much smaller in rainfall magnitude than those used to administer the National Flood Insurance Program (NFIP) program such as the 1% Annual Exceedance Probability (AEP) (100-yr) rainfall event. The calibration events had 24-hour rainfall totals between 3 - 6 inches, while the 1% AEP NOAA Atlas 14 rainfall depth for this area is 11 inches for a 24-hour storm. It is well documented in literature that more intense storm events have a more rapid and severe runoff response than smaller less intense events (Snyder; Minshall; USACE, 1991). This introduced some concern that the calibrated HEC-HMS parameters would not sufficiently represent physical watershed response to a much more intense storm event, such as the 1% AEP event. The second reason for the investigation was to confirm and/or update the storage volumes in the routing reaches, especially near the confluence of Mud Creek with the Angelina River, where there is suspected to be some comingling of the floodplains.

The primary tool used for this investigation of the upper Angelina River watershed was HEC-RAS version 5.0.7, which includes the ability to apply excess-precipitation onto a 2-dimensional mesh and simulate the excess-runoff being routed through the system with the unsteady 2D equations in RAS. The results of the detailed 2D HEC-RAS investigation were used to update the ModClark unit hydrograph parameters and the Modified Puls storage discharge curves in the watershed above the Angelina River nr Alto USGS gage. Additional information about the 2D HEC-RAS analysis performed in the upper Angelina River watershed can be found in Chapter 10 of this report and in *Appendix F: 2D HEC-RAS Analysis of the Angelina River and Mud Creek.*

6.4.3 **Calibrated Parameters**

The resulting calibrated subbasin and routing reach parameters that were adjusted for each storm event can be seen in Tables B.9 through B.17 of Appendix B.

6.4.4 **Calibration Results**

The final calibration results showed that the HEC-HMS model was able to accurately simulate the response of the watershed, as it reproduced the volume, timing, shape, and peak magnitudes of most observed floods very well. Some resulting hydrograph comparisons can be seen in the following figures of this section. The figures show the HEC-HMS computed versus the USGS observed flow hydrographs at each stream gage location. For each reservoir, the figures show the HEC-HMS computed pool elevation versus the USGS observed pool elevation. Calibration figures are only shown for the locations where the USGS stream gages were recording for that event and where the magnitude of the flow was significant enough to warrant calibration. For the sake of brevity, only a few calibration plots have been included as examples in this section of the report. The resulting hydrograph comparisons for all of the calibrations performed for this study have been included in Appendix B.

In addition to graphical comparisons of simulated to observed flow hydrographs, statistical tests were also employed in evaluating model performance. The statistical metrics used to evaluate the HEC-HMS model performance included the Nash-Sutcliffe Efficiency (NSE), the Root Mean Square Error – Observed Standard Deviation Ratio (RSR), and the Percent Bias (PBIAS). For the purposes of this study, the performance metrics were evaluated using the performance ratings shown in Table 6.5. These performance ratings are consistent with standard practices in watershed modeling (Moriasi, 2007) (Moriasi, 2012). In cases where each metric had a different performance rating, the overall performance rating for that calibration was assigned as the lowest of the three ratings, which is the strictest method of assigning performance ratings.

Table 6.5: HEC-HMS Model Calibration Evaluation Metrics

Performance Rating	NSE	RSR	PBIAS
Very Good	0.80 ≤ NSE < 1.00	0 ≤ RSR ≤ 0.50	0 ≤ PBIAS ≤ ±5
Good	0.70 ≤ NSE < 0.80	0.50 < RSR ≤ 0.60	±5 < PBIAS < ±10
Satisfactory	0.50 ≤ NSE < 0.70	0.60 < RSR ≤ 0.70	±10 ≤ PBIAS ≤ ±25
Unsatisfactory	NSE < 0.50	RSR > 0.70	PBIAS > ±25

Table 6.6 contains a summary of the model performance ratings for all the HEC-HMS calibrations performed for this study. The statistical metrics used to assign these performance ratings are shown on the figures for each individual calibration.

As shown in Table 6.6, over 87% of the all of the HEC-HMS model calibrations were rated as Good or Very Good. These ratings indicate that the HEC-HMS model performed very well in all three metrics when compared to observed data. For the other 13% of calibrations, there were missing data or problems with the observed data that resulted in a lower performance rating for several of the calibrations. B.A. Steinhagen Lake was particularly difficult to calibrate, due to a number of factors, including uncertainty in its outflows and its very small flood storage volume. These factors meant that a relatively small error in the assumed outflow from the dam can result in relatively large errors in its modeled pool elevation. However, since during large flood events, B.A. Steinhagen Lake operates with outflows essentially equal to its inflows, these calibrations will not impact the final frequency flow results. More discussion on B.A. Steinhagen Lake is included with its calibration plots in Appendix B.

For the sake of brevity, only a few calibration plots have been included as examples in this section of the report. The resulting hydrograph comparisons for all of the calibrations performed for this study have been included in Appendix B.

There are two types of figures which are shown in this section of the report: streamflow gages and reservoirs. In the streamflow gage figures, the solid blue line represents the total modeled streamflow at the gage, while the black line represents the observed streamflow that was recorded by the gage. The other dotted blue lines on these figures represent the runoff from individual model components (i.e., a single subbasin or routing reach), and they should be ignored as they are not relevant to the gage comparison. In the reservoir figures, the observed pool elevation at the reservoir gage is compared to the modeled pool elevation in the top half of the figure. The other lines on this plot shows reservoir storage, inflow, and outflow, but they are not relevant to the comparison with the observed pool elevations and can be ignored.

Table 6.6: Summary of HEC-HMS Model Calibration Performance Ratings

USGS Gage Location	Oct 2006	Jun-Jul 2007	May-Jun 2015	Mar 2016	Apr-May 2016	May-Jun 2016	Aug-Sep 2017	Sep 2019
Lake Athens	-	Very Good	Very Good	Very Good	Satisfactory	-	-	-
Lake Palestine	-	Very Good	Very Good	Very Good	Very Good	-	-	-
Neches River nr Neches, TX USGS Gage 08032000 at US-79 bridge	-	Very Good	Very Good	Very Good	Very Good	-	-	-
Lake Jacksonville	-	Very Good	Very Good	Very Good	Very Good	Very Good	-	-
Neches River nr Diboll, USGS gage 08033000 at US-59 bridge	-	-	Very Good	Very Good	Very Good	Very Good	-	-
Neches River near Rockland, USGS gage 08033500 at US-69 bridge	-	Very Good	Very Good	Very Good	Very Good	Very Good	Very Good	-
Lake Tyler	-	Very Good	Very Good	Very Good	Very Good	-	-	-
Mud Creek near Jacksonville, USGS gage 08034500 at US-79 bridge	-	Very Good	Very Good	Very Good	Good	-	-	-
Angelina River near Alto, USGS gage 08036500 at TX-21 bridge	-	Very Good	Very Good	Very Good	Very Good	-	-	-
Lake Nacogdoches	-	Very Good	Very Good	Very Good	Satisfactory*	-	-	-
Attoyac Bayou nr Chireno, USGS gage 08038000 at TX-21 bridge	-	Unsatisfactory*	-	Good	Good	-	-	-
Ayish Bayou near San Augustine, USGS gage 08039100 at TX-103	-	Good	Satisfactory*	Good	Satisfactory*	Very Good	Very Good	-
Sam Rayburn Reservoir	-	Very Good	Very Good	Very Good	Very Good	Very Good	Very Good	-
B.A. Steinhagen Lake**	-	Unsatisfactory	Unsatisfactory	Unsatisfactory	Unsatisfactory	Unsatisfactory	Unsatisfactory	-
Neches River at Evadale, USGS Gage 08041000	Very Good	-	Very Good	Very Good	-	Very Good	Good	Good
Village Creek near Kountze, USGS gage 08041500 at FM 418 bridge	Very Good	-	Very Good	Very Good	-	Very Good	Very Good	Very Good
Pine Island Bayou near Sour Lake, USGS gage 08041700 at Old Beaumont Rd bridge	Good	-	Very Good	Good	-	Very Good	Very Good	Very Good
Pine Island Bayou above BI Pump Plant, USGS Gage 08041749	Good	-	Good	Very Good	-	Very Good	Good	Unsatisfactory*
Neches River at the Salt Water Barrier, USGS Gage 08041780	Good	-	Very Good	Very Good	-	Very Good	Very Good	Very Good

* Lower rating is due to missing data or a problem with the observed data.

** B.A. Steinhagen calibrations are negatively affected by uncertainty in its outflows and a very small storage volume.

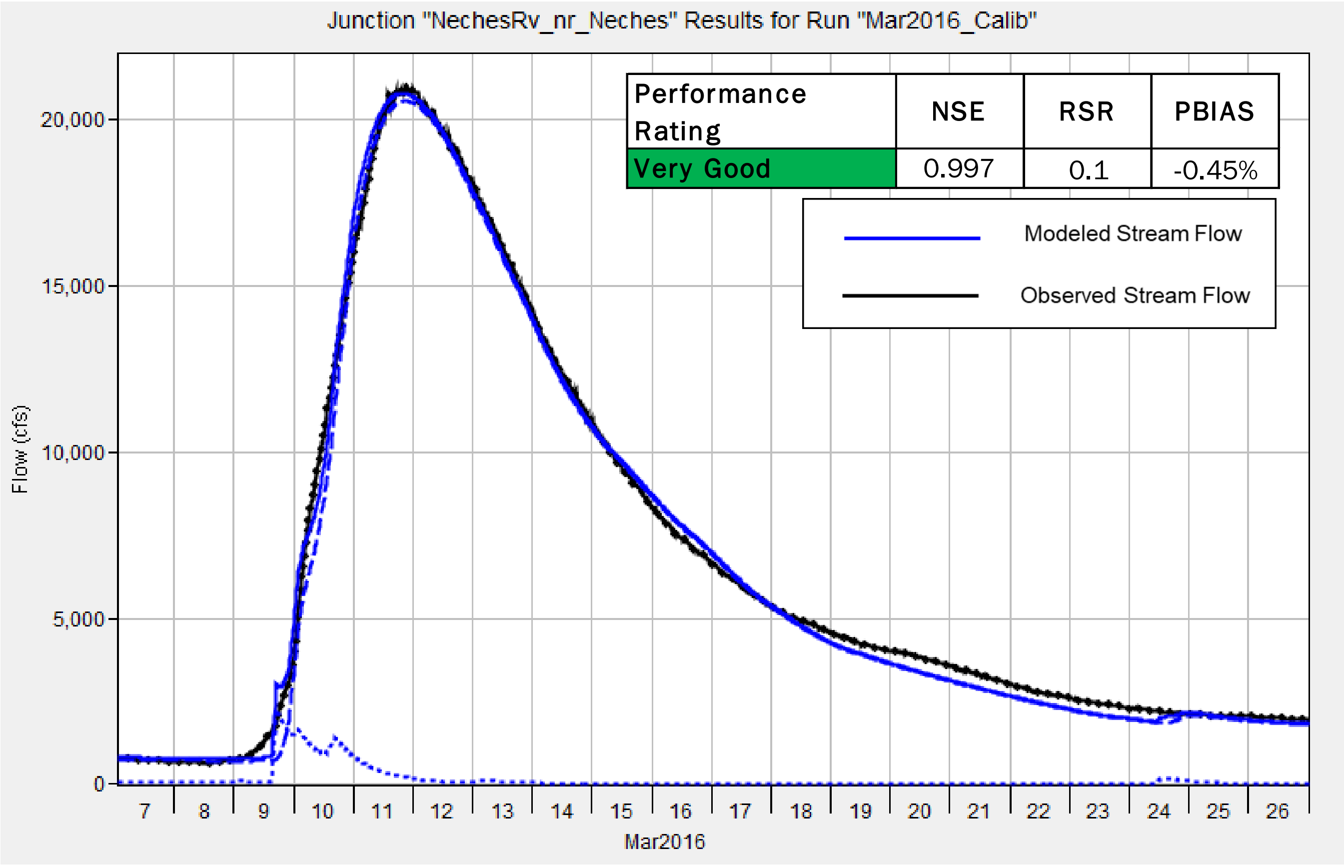


Figure 6.12: March 2016 Calibration Results for the Neches River nr Neches, TX USGS Gage

The Neches River near Neches is the upper most USGS stream gage on the Neches River. It has a total drainage area of 1,145 square miles, but only 300 square miles of that is below Lake Palestine. The modeled flow versus the observed flow at the gage had "Very Good" performance ratings in all four calibrations, as shown in Table 6.6. The largest calibration event at this location was March 2016, and it had a near perfect match between the modeled and observed river flow, with a Nash-Sutcliffe Efficiency (NSE) of 0.997, as shown in Figure 6.12.

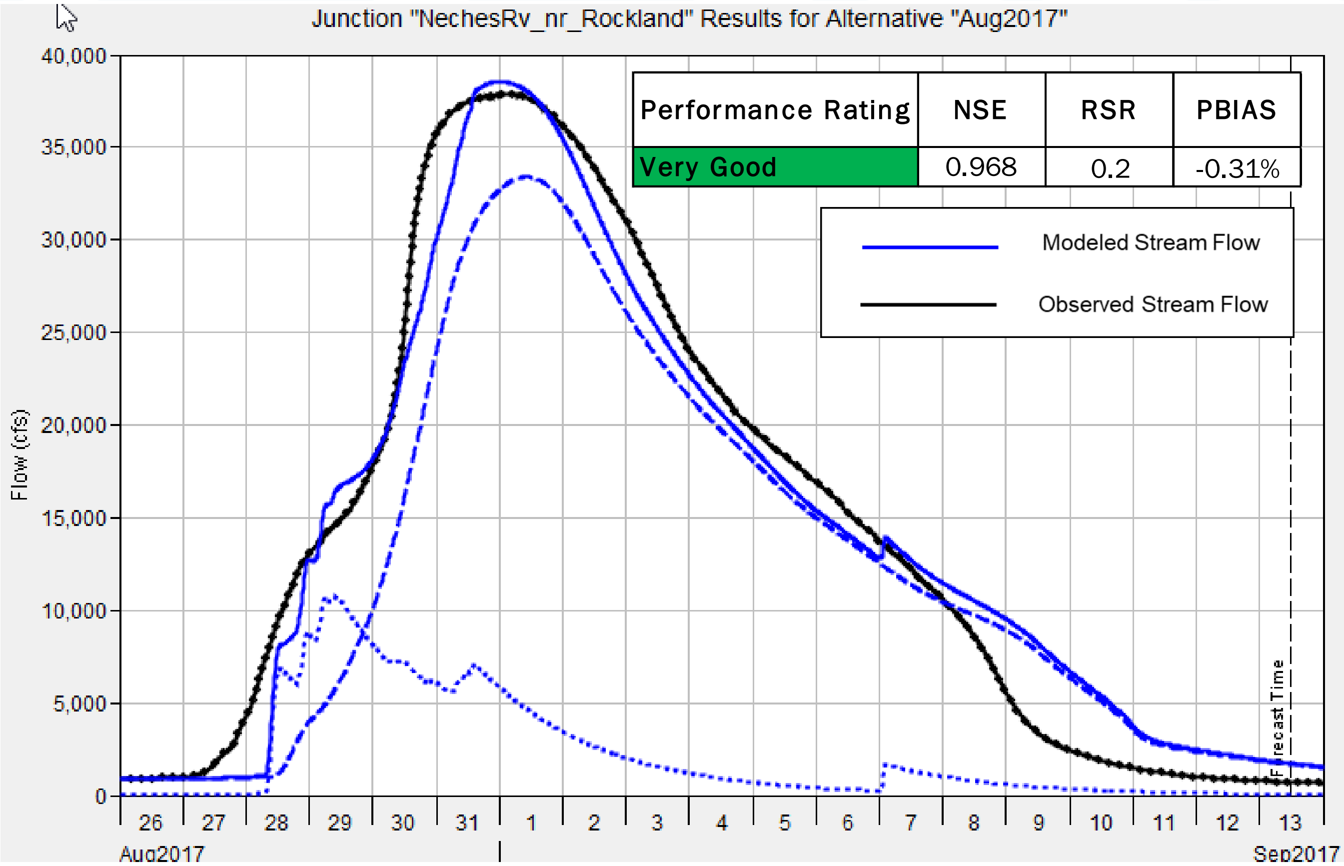
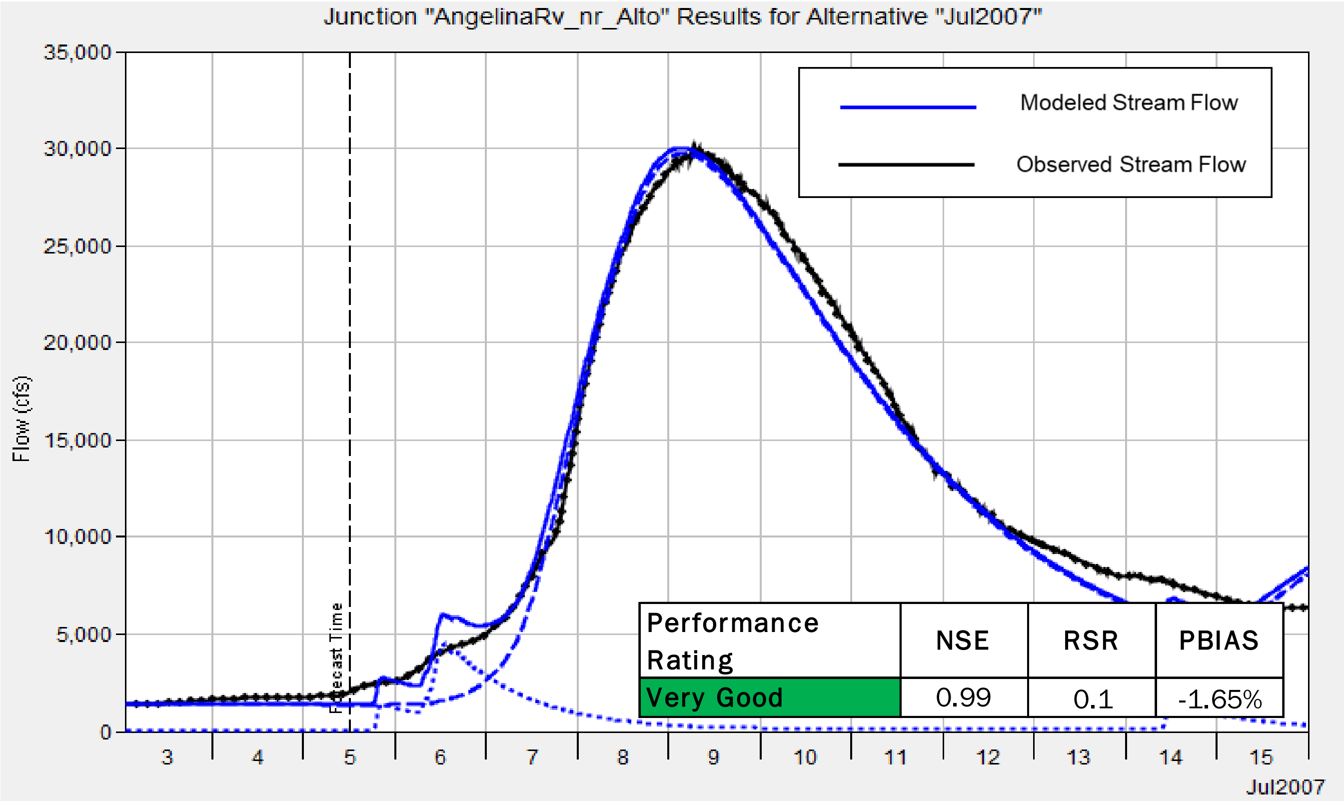


Figure 6.13: Aug 2017 Calibration Results for the Neches River at Rockland, TX USGS Gage

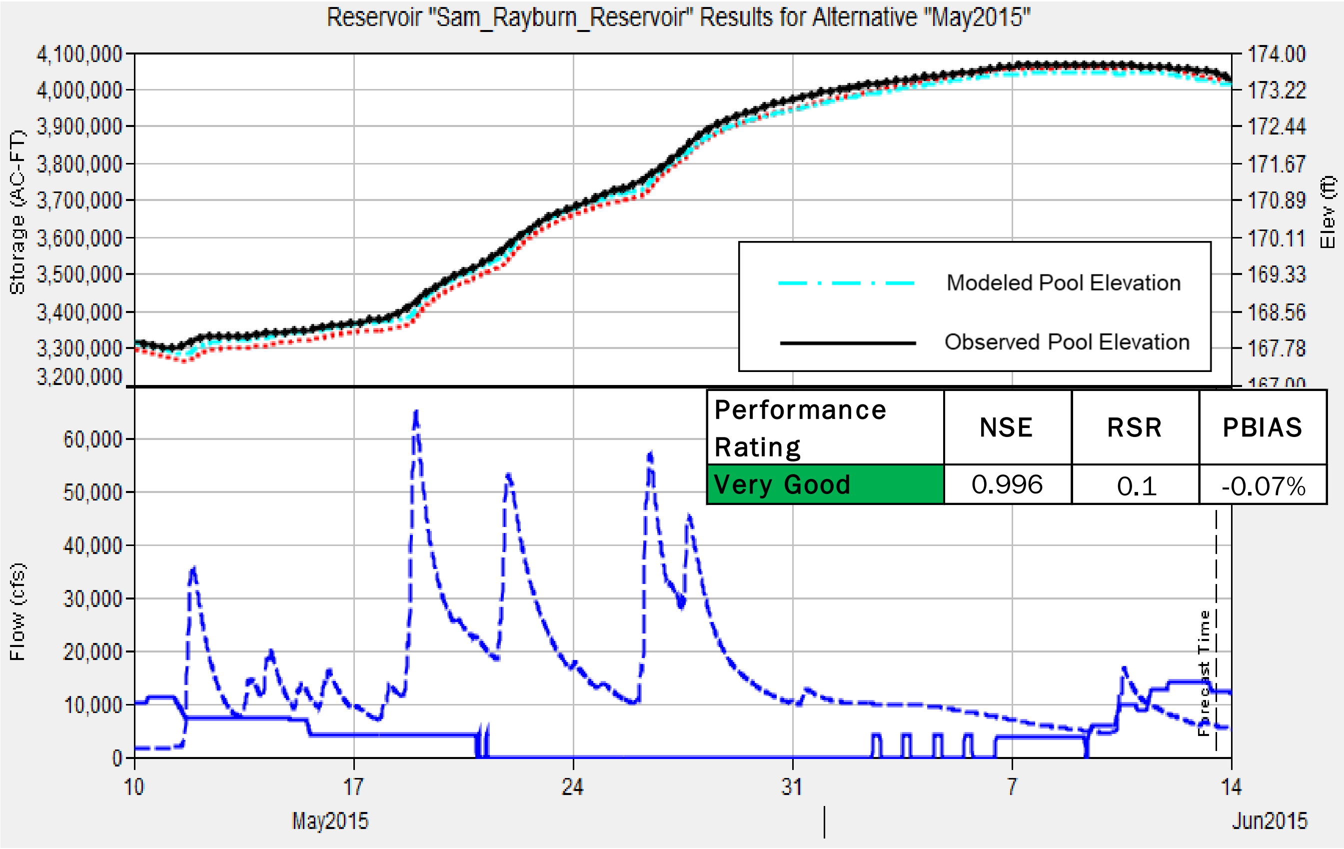
The Neches River near Rockland, Texas is a USGS stream gage with about 3,600 square miles of drainage area. Piney Creek is the largest tributary that enters the Neches River between Diboll and Rockland, but it encompasses only 10% of the drainage area near Rockland. Most of the Neches River flows near Rockland tend to be routed from the further upstream reaches of the Neches River. The modeled flow versus the observed flow at the gage had “Very Good” performance ratings in all five calibrations, as shown in Table 6.6. The largest calibration event at this location was Hurricane Harvey with a peak flow of almost 38,000 cfs, as shown in Figure 6.13.



Performance Rating	NSE	RSR	PBIAS
Very Good	0.99	0.1	-1.65%

Figure 6.14: July 2007 Calibration Results for the Angelina River nr Alto, TX USGS Gage

The Angelina River near Alto, Texas is a USGS stream gage with about 1,286 square miles of drainage area, which is located about 9 miles south of the confluence of the Angelina River with Mud Creek. The modeled flow versus the observed flow at the gage had "Very Good" performance ratings in all four of the calibration events. Three of the four calibrations had nearly perfect Nash-Sutcliffe Efficiencies (NSE) of 0.98 or 0.99, including July 2007, which was the largest calibration event at this location.



Performance Rating	NSE	RSR	PBIAS
Very Good	0.996	0.1	-0.07%

Figure 6.15: May 2015 Calibration Results for the Sam Rayburn Reservoir USGS Gage

Sam Rayburn Reservoir is a USACE reservoir on the Angelina River. It has a normal surface area of over 112,000 acres and a drainage area of about 3,450 square miles. The reservoir has a very large flood storage capacity of over 1.5 million acre-feet between the top of conservation pool and the spillway crest. The dam has hydropower turbines, a gated outlet works, and a labyrinth uncontrolled spillway. The spillway at Sam Rayburn has never been engaged (as of December 2021), but the observed pool elevations have come within about one foot of the spillway crest on more than one occasion. The modeled pool elevation versus the observed pool elevation had “Very Good” performance ratings in all six of the calibration events, as shown in Table 6.6. All of the events had nearly perfect Nash-Sutcliffe Efficiencies (NSE) of 0.98 or 0.99, which indicate an excellent match between the modeled and observed results. The calibration event with the highest pool elevation was May 2015 with a peak pool elevation of 173.8 feet, which is 0.8 feet above the top of the flood control pool, and the match between the modeled and observed pool elevation was nearly perfect, as shown in Figure 6.15.

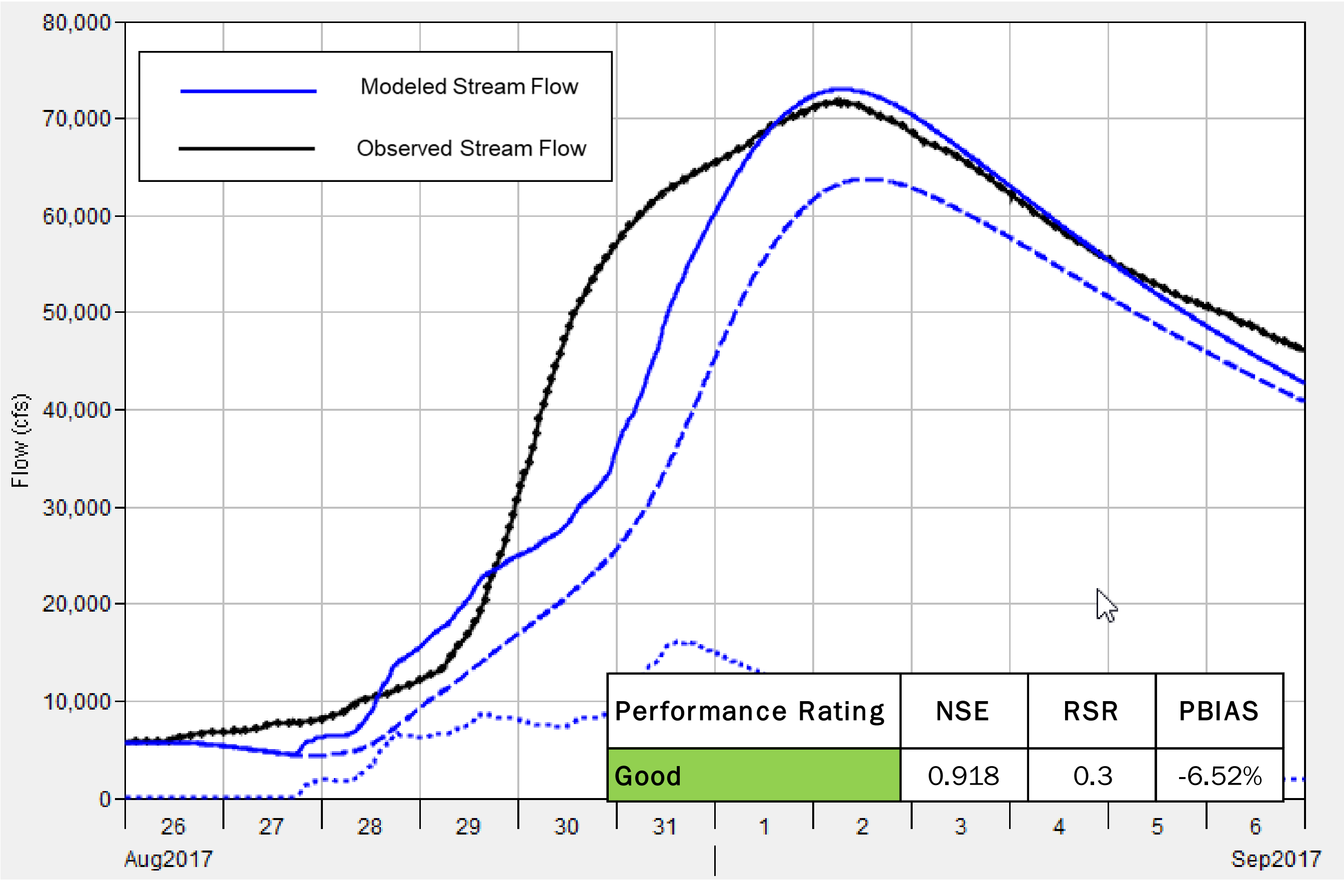


Figure 6.16 Aug 2017 Calibration Results for the Neches River at Evadale, TX USGS Gage

The Neches River at Evadale, Texas is a USGS stream gage with almost 7,900 square miles of drainage area, which is located on the Neches River about 30 miles south of Town Bluff Dam and B.A. Steinhagen Lake. The modeled flow versus the observed flow at the gage had “Good” or “Very Good” performance ratings for all six calibration events, as shown in Table 6.6. The largest calibration event at this location was Hurricane Harvey, which had a peak observed flow of 71,800 cfs, and that event had a very strong Nash-Sutcliffe Efficiency (NSE) of 0.91, as shown in Figure 6.16. The main deviation between the observed and modeled streamflow was on the rising limb of this event, which may have been caused by backwater from downstream tributaries and local runoff. One 55 square mile tributary enters the Neches River just a half mile downstream of the gage. Precipitation on the lower Neches River also increased substantially the further one moved downstream, as shown in Figure 6.10, which may have caused more backwater effects for this event than are normally observed at this location.

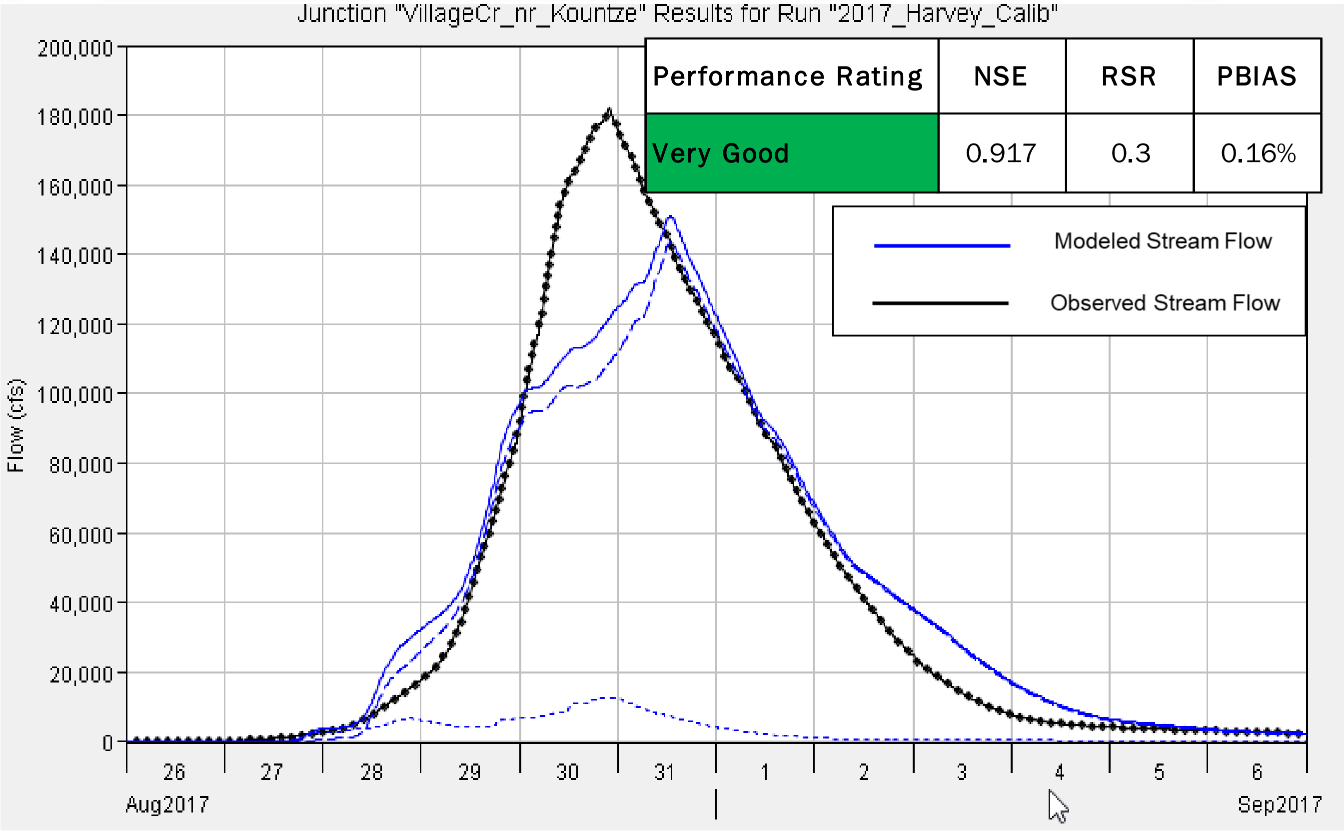


Figure 6.17: Aug 2017 Calibration Results for the Village Creek nr Kountze, TX USGS Gage

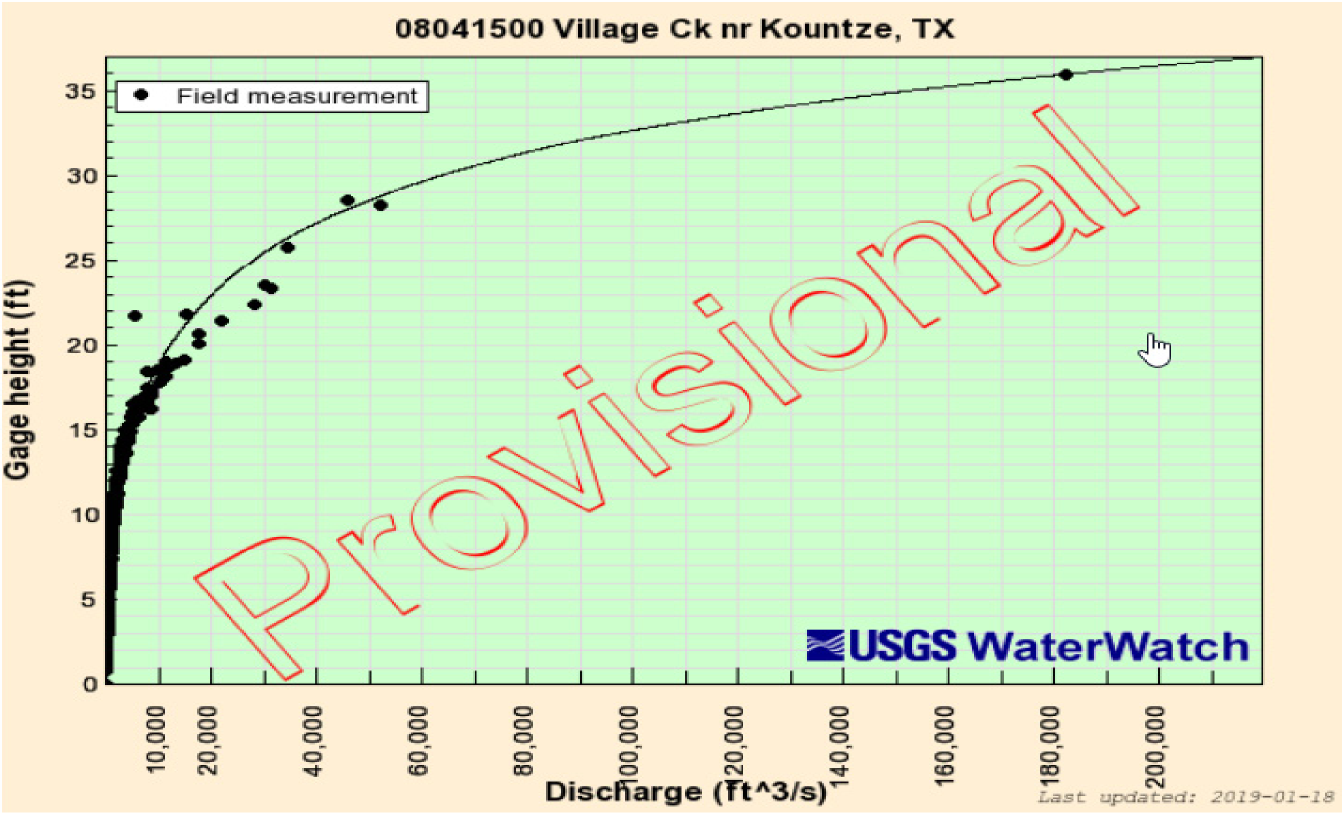
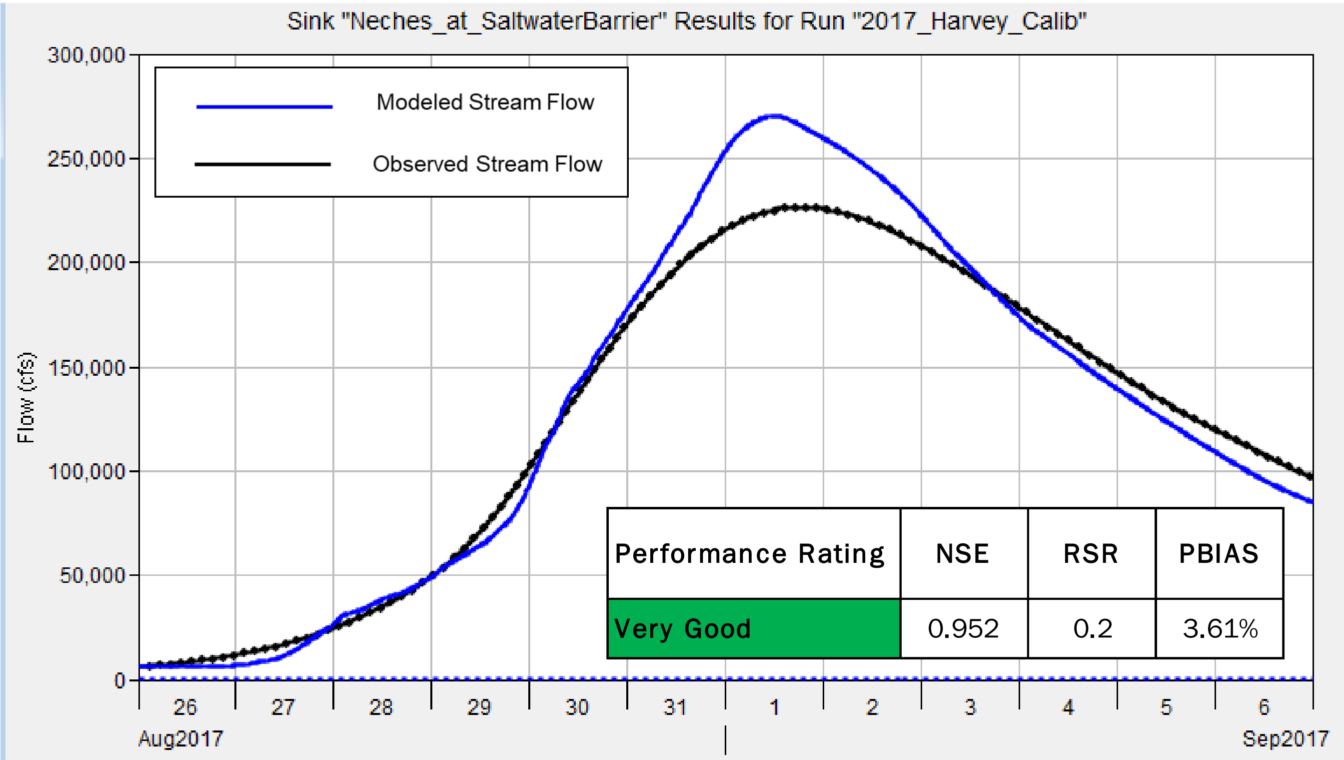


Figure 6.18: USGS Rating Curve and Field Measurements for Village Creek nr Kountze, TX USGS Gage

Village Creek near Kountze, Texas is a USGS stream gage with about 860 square miles of drainage area, which is located on a tributary to the Neches River. The modeled flow versus the observed flow at the gage had "Very Good" performance ratings for all six calibration events, as shown in Table 6.6. The largest calibration event at this location was Hurricane Harvey, which had a peak observed flow of 182,000 cfs, as shown in Figure 6.17. However, there is a large amount of uncertainty in the actual peak flow for this event. The peak observed gage height during Hurricane Harvey exceeded any previous observed event at that location by over 8 feet and was well beyond the existing rating curve for the gage. The USGS estimated the peak flow for Hurricane Harvey indirectly after the fact, and they assigned an uncertainty of +/- 20% to that flow value. Figure 6.18 shows the current USGS rating curve for this gage along with the available flow measurements that have been made at that location. From this figure, one can see that there are no available flow measurements for discharges much over 50,000 cfs, except for Hurricane Harvey; therefore, there is a great deal of uncertainty in the rating curve for this location for discharges above 50,000 cfs. The HEC-HMS model results for that event showed a peak flow of approximately 150,000 cfs, which is within the uncertainty range of the USGS peak flow value.

The Neches River at the Saltwater Barrier is the most downstream USGS stream gage in the study area, with a drainage area of over 9,800 square miles. The modeled flow versus the observed flow at the gage had "Good" or "Very Good" performance ratings for all six calibration events, as shown in Table 6.6. The largest calibration event and the flood of record at this location was Hurricane Harvey, which had a peak USGS observed flow of 232,000 cfs and a "Very Good" model performance rating, as shown in Figure 6.19. Hurricane Harvey exceeded the previous record stage and the existing rating curve by over 10 feet at this location; therefore, the USGS estimated that there is at least +/- 20% uncertainty in the actual peak discharge for that event. Due to this uncertainty, the HEC-HMS model calibration for that event did not attempt to match the observed peak discharge magnitude, but rather the timing of the peak flow and stage at that location.



Performance Rating	NSE	RSR	PBIAS
Very Good	0.952	0.2	3.61%

Figure 6.19: Aug 2017 Calibration Results for the Neches River at the Saltwater Barrier USGS Gage

6.5 FINAL MODEL PARAMETERS

After the initial parameter estimates were made and the calibration process was completed, the final parameters were established. The final ModClark time of concentrations and storage coefficients were developed by taking a weighted average of the time of concentrations and storage coefficients from the calibration events. The peak discharge from the subbasin for that event was used to weight the calibrated lag times. This method has the effect of granting a higher weight to the time of concentrations that were calibrated from larger, more intense storms, and it ignores the storms that generated no runoff from a particular subbasin. The final ModClark time of concentrations and storage coefficients can be found in Table B.20 of Appendix B.

The final baseflow parameters were selected based on the results of the calibration runs. Specifically, the initial flows per square miles were selected based on typical flow rates observed on each reach of the river prior to a storm event, and the recession constant and ratio to peak were selected based on the slope and shape of the receding limb of the hydrograph at the downstream gages. The final baseflow parameters can also be found in Table B.20 of Appendix B.

The Modified Puls storage discharge relationships were calculated from 1D and 2D HEC-RAS models, and the final number of subreaches were selected based on calibration to the observed attenuation of the flood hydrograph in between stream gages. Once again, the final subreach values were calculated from a weighted average based on the peak magnitude of the flow through the reach for a given storm event. The final routing subreach values can be found in Table B.21 of Appendix B. A few routing reaches used lag routing parameters. The final lag times can be found in Table B.22 of Appendix B.

In observed storm events, the initial and constant losses vary from storm to storm according to the antecedent moisture conditions of the soil. Therefore, no final set of loss rates was selected based on the calibration events. Instead, the losses for the frequency storms were developed using the USACE Fort Worth District Method for determining losses based on soil type (percent sand) (Rodman, 1977). This method produces a different set of loss rates for each frequency event, based on the soil type in each subbasin. The method assumes that the antecedent moisture conditions become wetter and the losses decrease as the rarity of the flood event increases, which is consistent with other research (McEnroe, 2003). In general, the 50% AEP loss rates correspond to an “average” or “normal” antecedent soil moisture condition, and the 0.2% AEP loss rates correspond to a “wet” soil moisture condition. Table 6.7 summarizes the range of default loss rates of the Fort Worth District method by frequency and soil type. A geospatial grid of percent sand for the State of Texas developed by the USACE Fort Worth District from the SSURGO data was used to spatially calculate the percent sand for each subbasin. That percent sand value was then used to interpolate between the 0% and 100% sand loss rate values in Table 6.7 to assign the default initial and constant loss rates to each subbasin.

After calculation of the default frequency loss rates, an additional initial loss was added to the default initial losses based on the presence of NRCS flood control structures in the watershed that have not been modeled in detail. Data from the National Inventory of Dams (NID) was used for this adjustment (USACE, 2016). In this case, the percent of each subbasin area that was controlled by NRCS structures was multiplied by the inches of runoff that can typically be stored between the riser and spillway of the NRCS structures in that basin (typically up to 4 inches of runoff). For the frequent storm events (50% to 4% AEP), the initial loss due to the NRCS structures was decreased in proportion to the total depth of rain for that event.

Table 6.7: Default Frequency Loss Rates by Soil Type for the USACE Fort Worth District Method

Annual Exceedance Probability (AEP) %	Initial Abstraction (inches) for Soil with 0% Sand	Infiltration Rate (inches per hour) for Soil with 0% Sand	Initial Abstraction (inches) for Soil with 100% Sand	Infiltration Rate (inches per hour) for Soil with 100% Sand
50%	1.50	0.20	2.10	0.26
20%	1.30	0.16	1.80	0.21
10%	1.12	0.14	1.50	0.18
4%	0.95	0.12	1.30	0.15
2%	0.84	0.10	1.10	0.13
1%	0.75	0.07	0.90	0.10
0.5%	0.61	0.06	0.73	0.09
0.2%	0.50	0.05	0.60	0.08

After running the uniform rain frequency storms in HEC-HMS with the calculated frequency loss rates, a comparison was made between the preliminary HEC-HMS results and the statistical flow frequency curves from the USGS gage records. A final adjustment was then made to the initial and constant losses for the 50% through 10% AEP storms in order to have a better correlation with the statistical frequency curves estimated from the USGS gage records. This step was performed because of the increased confidence level in the gage records' statistical frequency curves for the 50% through 10% AEP range. The 4% losses were also adjusted when needed to create a smoother transition between the 2% and 10% AEP flow values. Loss rates for events with an AEP at or below 2% were not adjusted based on the statistical frequency curves because stream gage records in Texas are not long enough and there is too much variability in the rare AEP statistical flow estimates over time (see the change over time plots in Appendix A) to justify adjusting the rare AEP loss rates. Generally, a stream gage record that is 3 to 4 times the length of the return period being estimated is needed before the statistical results can be considered reliable enough for this type of adjustment (Faber, 2018). For the 1% AEP event, this would require a stream gage record of 300 to 400 years in length, which is not available anywhere in Texas.

The final loss rates that were used for each uniform rainfall frequency storm event can be found in Tables B.24 and B.25 of Appendix B. These adopted loss rates for the frequency events fell well within the band of observed losses from the calibration storms, as shown in Figures 6.20 and 6.21. Based on the range of observed initial and constant losses from the calibration storms, the adopted losses for the frequency storms could be characterized to represent "average" to "wet" conditions (the "average" moisture conditions being applied to the 50% AEP storm, and "wet" moisture conditions being applied to the 0.2% AEP storm), which are appropriate assumptions for modeling hypothetical flood events. However, none of the adopted frequency losses are at the extreme wet or extreme dry ends of the range of calibrated losses.

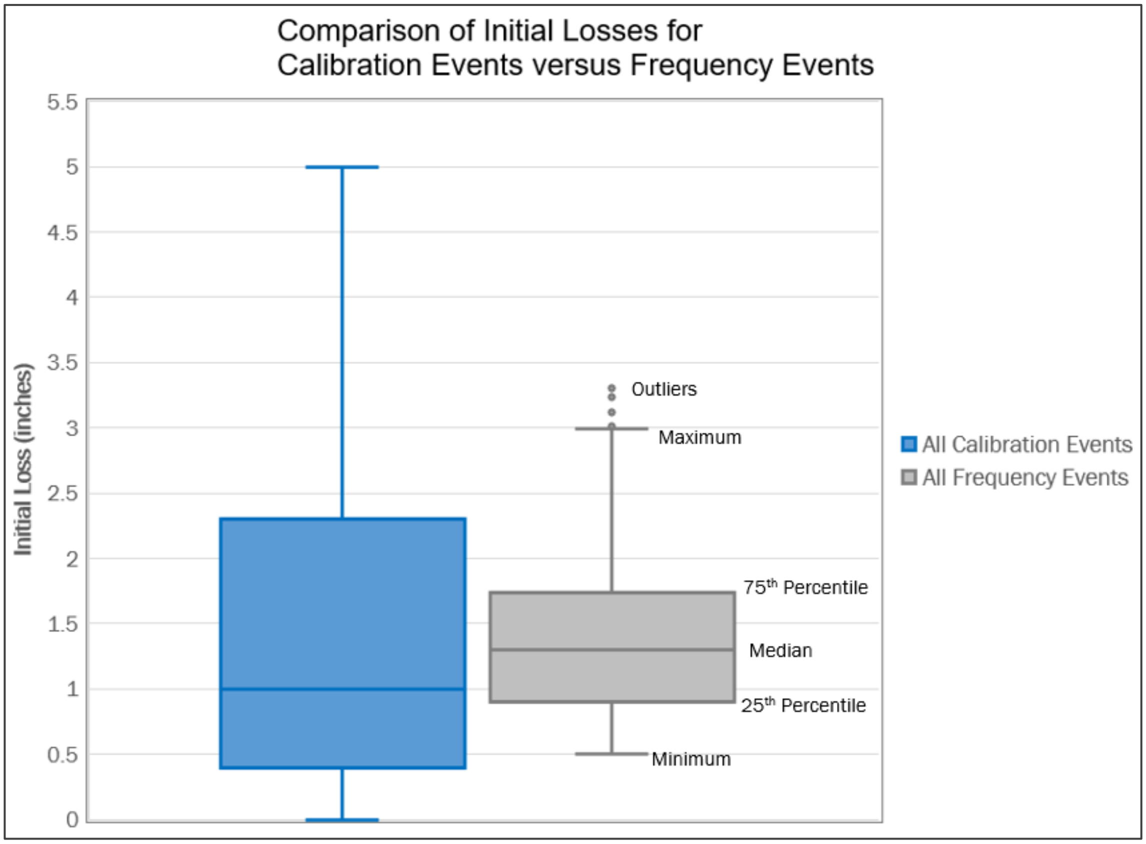


Figure 6.20: Comparison of Adopted versus Calibrated Initial Losses (inches)

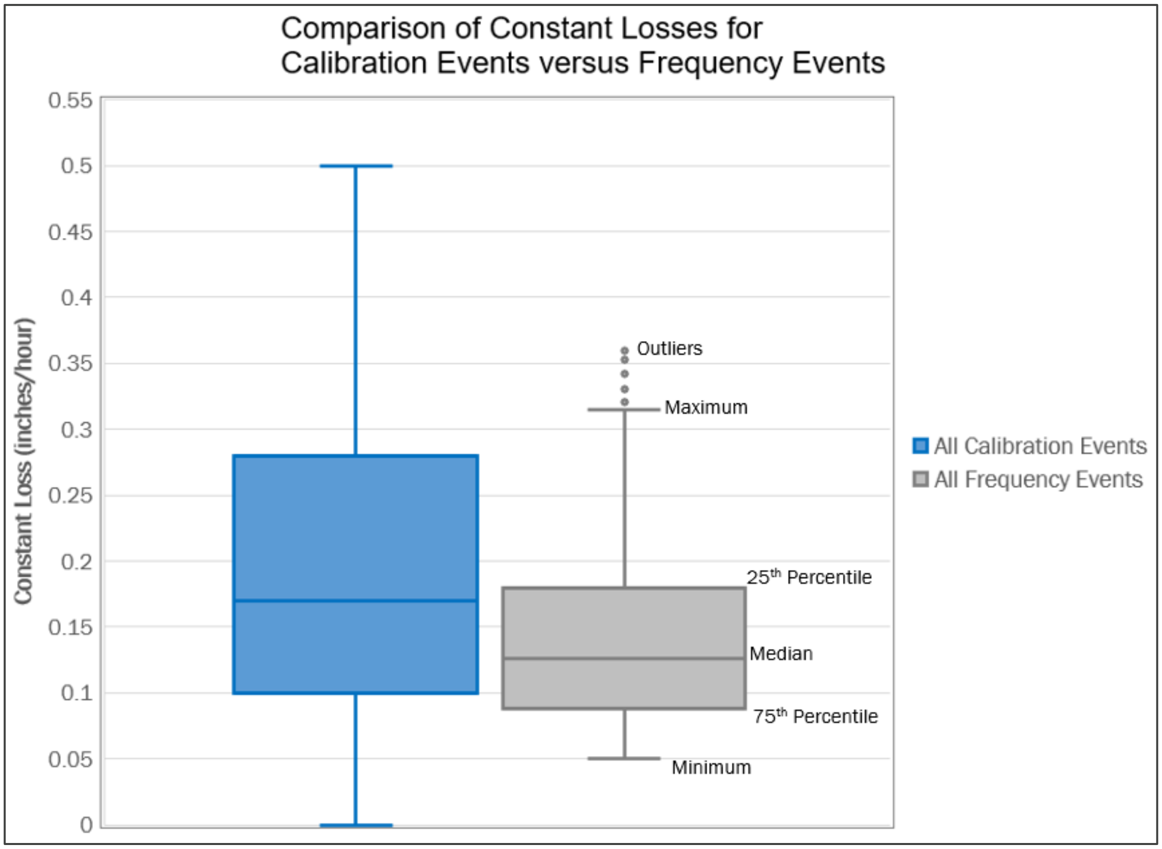


Figure 6.21: Comparison of Adopted versus Calibrated Constant Losses (inches per hour)

6.6 UNIFORM RAINFALL FREQUENCY STORMS

The frequency flow values were then calculated in HEC-HMS by applying frequency rainfall depths to the final watershed basin models through a series of depth-area analyses. This rainfall pattern is referred to as the uniform rainfall method because the assigned point rainfall depths for each subbasin are reduced uniformly over the entire watershed based on the published depth-area reduction factors from Figure 15 of the National Weather Service TP-40 publication (Herschfield, 1961). A depth area analysis was set up for every junction and node of interest within the HEC-HMS model in order to apply the appropriate depth-area reduction for each drainage area of interest.

Due to the longer travel times and slower watershed response of the Neches River basin, a 4-day duration frequency storm with a 67% intensity position and a 15-minute intensity duration was adopted in the HEC-HMS model. Sensitivity tests were also run for durations ranging from 24-hours to 10-days and for intensity positions ranging from 25% to 75%, but in most cases, the peak flow results were not particularly sensitive to these settings (generally within +/- 5%). The adopted 4-day duration is also consistent with large storms which have been observed in the basin, such as Hurricane Harvey. Additional information on the sensitivity tests results is provided in Appendix B.

6.6.1 Point Rainfall Depths for the Uniform Frequency Storms

NOAA Atlas 14 contains precipitation frequency estimates for the United States along with their associated lower and upper 90% confidence bounds. The Atlas is divided into volumes based on geographic sections of the country. NOAA Atlas 14 is intended as the U.S. Government source of precipitation frequency estimates. NOAA Atlas 14 Volume 11, which covers the state of Texas, was recently published in September of 2018 (NOAA, 2018). The new rainfall depths that were published in NOAA Atlas 14 (NA14) were applied to the HEC-HMS model for this study, as they are the most up-to-date precipitation frequency estimates in Texas.

Figures 6.22 to 6.24 illustrate the NA14 1% Annual Exceedance Probability (AEP) point rainfall depths for the 6, 24, and 96-hour durations across the Neches River basin. These figures show that the largest rainfall depths were consistently shown in the very downstream portion of the basin nearest to the coast. Geographically, it makes sense that this area would receive the most rainfall because areas near the coast tend to receive more rainfall than inland areas due to their proximity to the large source of moisture at the Gulf of Mexico.

NOAA Atlas 14 point rainfall depths from the annual maximum series for various durations and recurrence intervals were collected from the NA14 Precipitation Frequency Data Server (PFDS) for the centroid of each subbasin (NOAA, 2020). This method resulted in 93 separate point rainfall tables being applied in the Neches River basin, one for each subbasin. The appropriate point rainfall depth table was assigned to each subbasin within the HEC-HMS frequency storm editor. It should be noted that precipitation frequency estimates from NOAA Atlas 14 are point estimates and are not directly applicable to larger areas. The conversion from a point to an areal estimate was accomplished for the uniform rainfall method by using the depth area analyses in HEC-HMS with the default TP-40 area reduction curves.

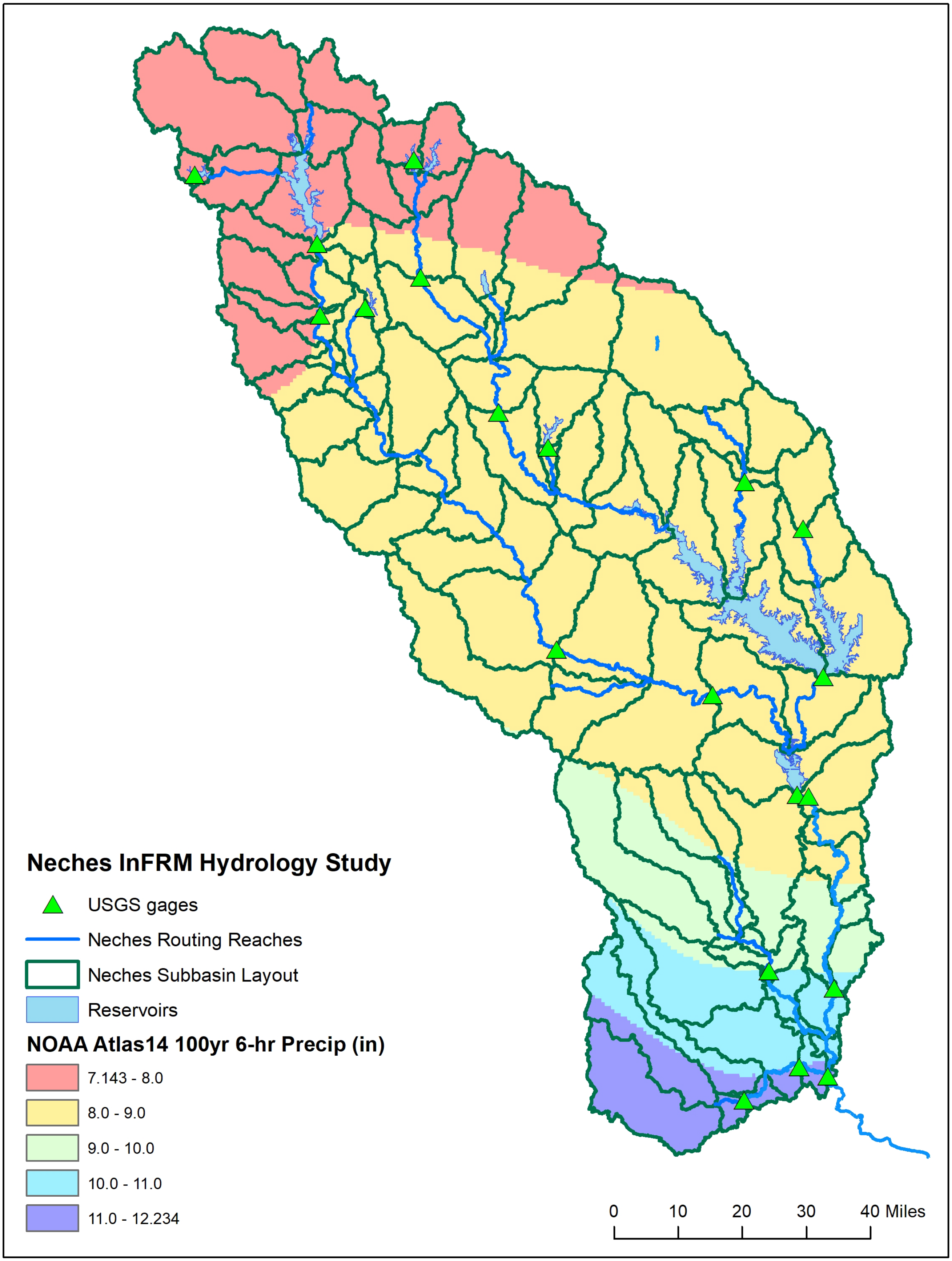


Figure 6.22: 1% AEP, 6-hour Rainfall Depths for the Neches River Basin from NOAA Atlas 14

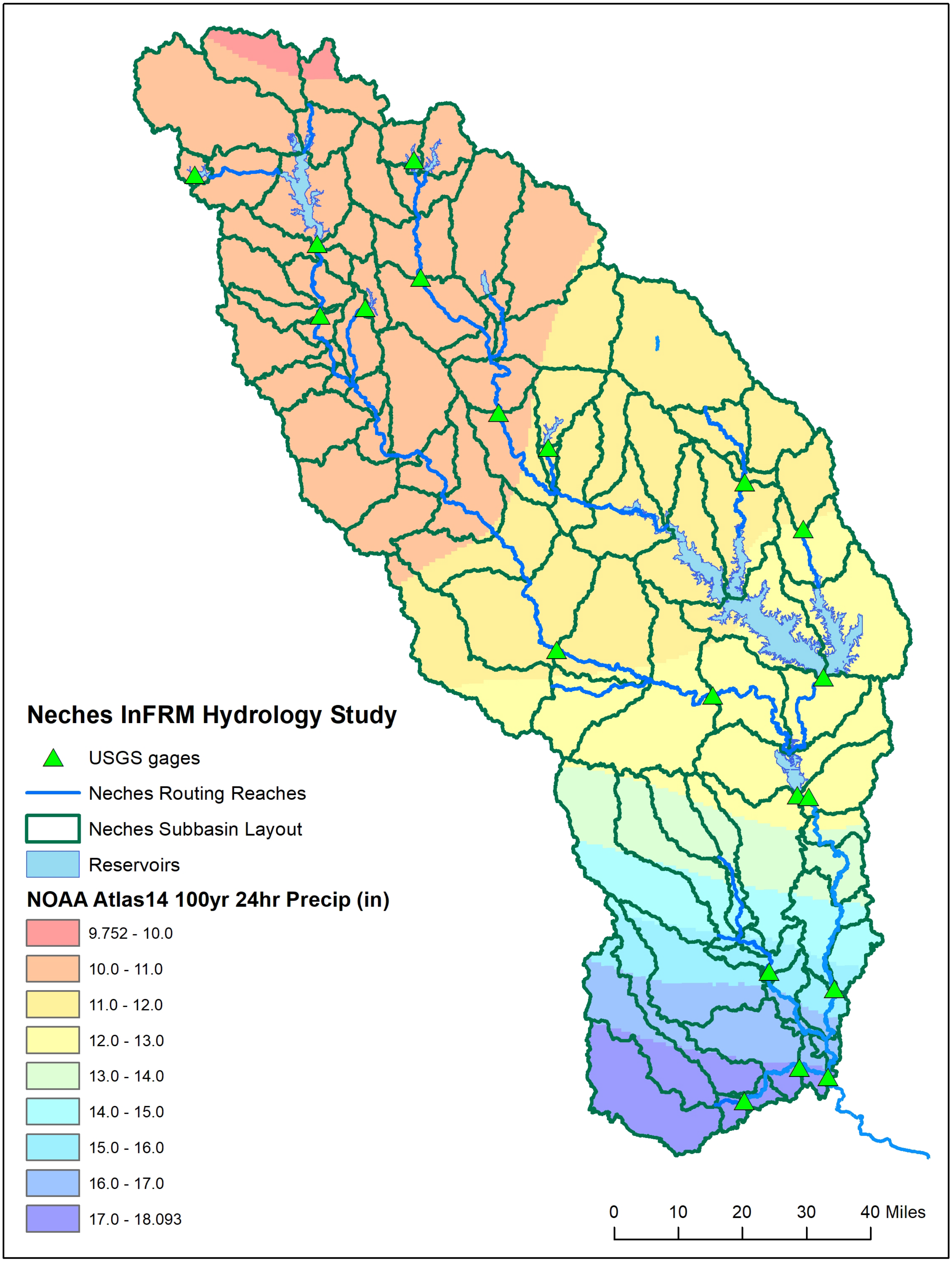


Figure 6.23: 1% AEP, 24-hour Rainfall Depths for the Neches River Basin from NOAA Atlas 14

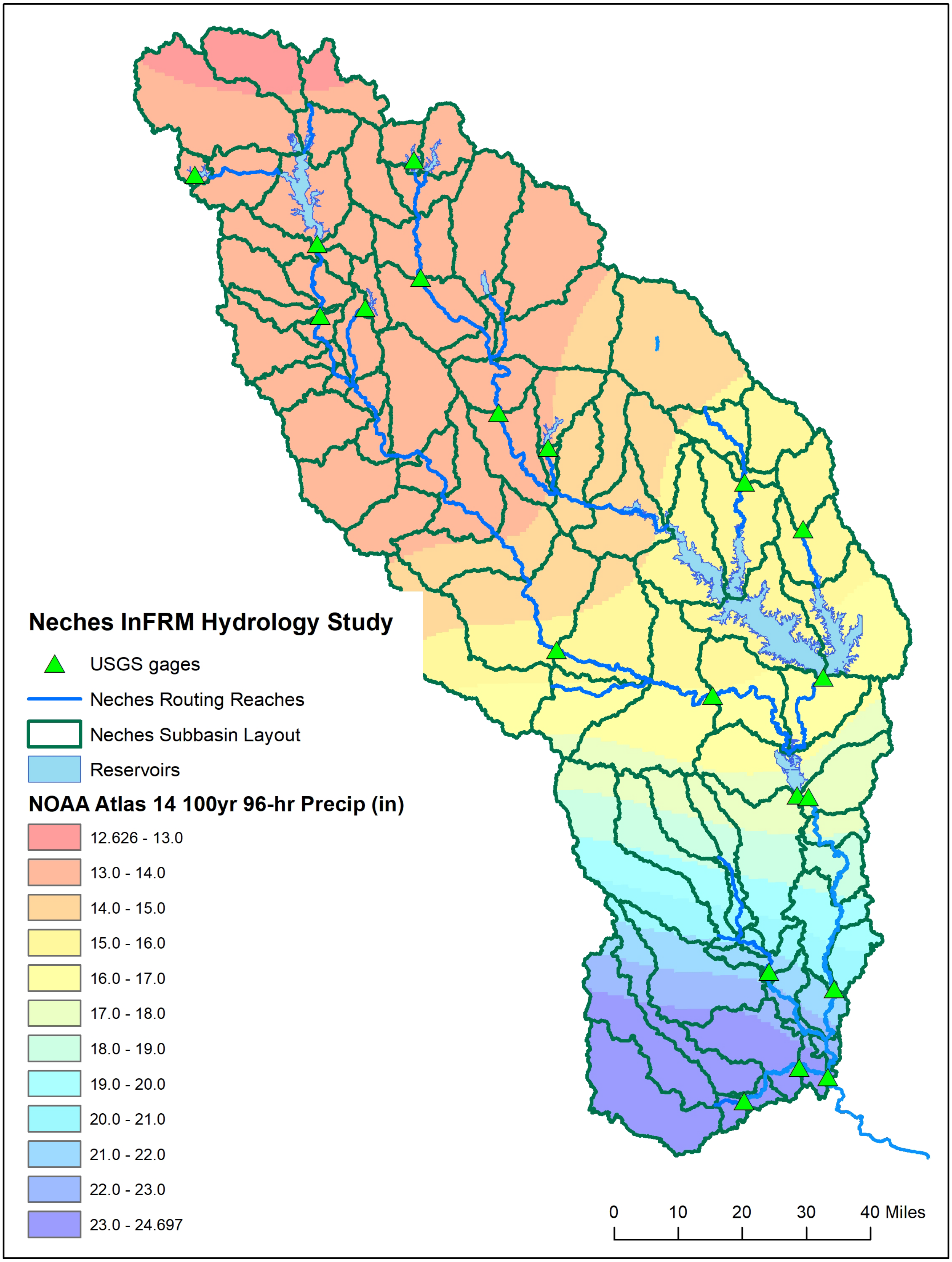


Figure 6.24: 1% AEP, 96-hour Rainfall Depths for the Neches River Basin from NOAA Atlas 14

6.6.2 Frequency Storm Results – Uniform Rainfall Method

The final frequency results for the uniform rainfall method were then computed in HEC-HMS by applying the NOAA Atlas 14 frequency rainfall depths to the final watershed model through a series of depth-area analyses of the applied frequency storms. This rainfall pattern is referred to as the uniform rainfall method because the assigned point rainfall depths for each subbasin are reduced uniformly over the entire watershed.

The final uniform rain HEC-HMS frequency flow results for significant locations throughout the watershed model can be seen in Table 6.8. In this table, the highlighted rows indicate calibrated gage locations. The final uniform rain HEC-HMS frequency pool elevation results are summarized in Table 6.9. These results were then compared to the elliptical shaped storm results from HEC-HMS along with other methods from this study, as shown in Chapter 11 of the main report.

In some cases, one may observe in Table 6.8 that the simulated peak discharge decreases in the downstream direction. It is not an uncommon phenomenon to see decreasing frequency peak discharges for some river reaches as flood waters spread out into the floodplain and the hydrograph becomes dampened as it moves downstream. This can be due to a combination of peak attenuation due to river routing as well as the difference in timing between the peak of the main stem river versus the runoff from the local tributaries and subbasins.

Table 6.8: Summary of Peak Discharges (cfs) from the HEC-HMS Uniform Rainfall Frequency Storms

Location Description	HEC-HMS Element Name	Drainage Area	50%	20%	10%	4%	2%	1%	0.50%	0.20%
		sq mi	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	200-YR	500-YR
Neches River above Prairie Creek	Neches_S021	127.0	4,800	9,300	12,600	17,300	21,400	26,400	31,200	37,800
Prairie Creek above Neches River	Neches_S022	89.8	6,000	10,400	13,700	18,300	22,200	26,800	31,400	37,900
Neches River below Prairie Creek	Neches+PrairieCk	216.7	8,800	16,500	22,300	31,000	38,400	47,500	56,100	68,300
Neches River above Kickapoo Creek	Neches_abv_KickapooCr	281.0	7,800	15,700	22,500	33,000	42,000	54,200	65,500	81,400
Kickapoo Creek above Neches River	Neches_S010	289.6	5,600	10,800	15,000	21,400	26,900	33,900	40,500	50,200
Neches River below Kickapoo Creek	Neches_blw_KickapooCr	570.6	12,100	22,300	30,200	42,000	52,300	65,900	78,800	97,400
Lake Athens Inflow	Lk_Athens_S010	21.6	2,200	3,700	4,900	6,500	7,900	9,500	11,100	13,500
Flat Creek below Lake Athens	FlatCk_blw_LkAthens	21.6	300	500	800	1,200	1,200	1,200	1,300	2,000
Flat Creek above Lake Palestine	FlatCk_abv_LkPalestine	118.6	4,600	8,600	11,700	16,300	20,200	25,100	29,900	36,800
Lake Palestine Inflow	Lk_Palestine_Inflow	838.1	17,800	31,800	44,100	65,200	83,800	108,100	131,400	164,600
Neches River below Lake Palestine	Neches_blw_LkPalestine	838.1	3,600	7,900	11,300	18,600	25,100	34,300	43,600	57,600
Neches River at US-175	Neches_at_US-175	882.5	3,600	8,000	11,400	18,700	25,300	34,600	43,900	58,000
Neches River above Caddo Creek	Neches_abv_CaddoCr	901.9	3,700	8,000	11,500	18,800	25,400	34,700	44,000	58,200
Caddo Creek	CaddoCr_S010	64.8	2,400	5,200	7,500	10,800	13,400	16,600	19,800	24,400
Neches River below Caddo Creek	Neches_blw_CaddoCr	966.7	4,100	8,700	12,700	19,400	26,300	36,000	45,900	60,800
Neches River above Brushy Creek	Neches_abv_BrushyCr	1020.4	3,800	8,200	11,800	19,300	26,200	36,000	45,900	61,000
Brushy Creek above Neches River	BrushyCr_S010	84.0	3,300	6,600	9,200	13,300	16,500	20,400	24,300	30,000
Neches River below Brushy Creek	Neches_blw_BrushyCr	1104.4	5,300	10,300	14,700	21,900	27,500	37,200	48,300	65,500
Neches River nr Neches, TX USGS Gage 08032000 at US-79 bridge	NechesRv_nr_Neches	1145.8	4,700	10,300	15,700	25,000	32,600	42,200	53,000	72,200
Neches River above Hurricane Creek	Neches_abv_HurricaneCr	1171.2	3,800	8,300	12,000	19,700	26,800	37,500	48,800	66,600
Hurricane Creek above Neches River	HurricaneCr_S010	103.8	4,100	8,700	12,000	16,800	20,800	25,600	30,400	37,400
Neches River below Hurricane Creek	Neches_blw_HurricaneCr	1275.0	5,500	11,000	15,200	23,400	32,100	45,200	58,900	80,700
Neches River above Stills Creek	Neches_abv_StillsCr	1289.5	3,900	8,400	13,100	23,000	31,600	44,400	58,000	79,400
Stills Creek above Neches River	StillsCr_S010	56.0	3,300	6,700	9,400	13,000	15,900	19,400	22,900	27,900
Neches River below Stills Creek	Neches_blw_StillsCr	1345.5	4,400	8,500	13,500	23,800	32,800	46,200	60,400	83,100

Location Description	HEC-HMS Element Name	Drainage Area	50%	20%	10%	4%	2%	1%	0.50%	0.20%
		sq mi	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	200-YR	500-YR
Neches River above Tails Creek	Neches_abv_TailsCr	1358.7	3,900	8,500	13,400	23,300	32,000	45,800	60,300	82,800
Lake Jacksonville Inflow	Lk_Jacksonville_S010	39.6	6,600	11,100	14,400	19,200	23,000	27,700	32,400	39,200
Tails Creek below Lake Jacksonville	TailsCr_blw_LkJacksonville	39.6	1,200	1,200	1,300	1,300	1,300	1,500	3,400	6,900
Tails Creek above Neches River	TailsCr_abv_Neches	107.0	3,000	6,000	8,300	11,700	14,500	17,900	21,400	26,500
Neches River below Tails Creek	Neches_blw_TailsCr	1465.8	5,200	10,600	15,500	25,800	35,000	50,100	66,900	94,300
Neches River above Ioni Creek	Neches_abv_IoniCr	1497.3	4,300	9,800	15,000	25,200	34,200	48,900	65,400	92,300
Ioni Creek above Neches River	IoniCr_S010	104.3	4,900	10,200	14,100	19,900	24,400	30,100	35,700	44,000
Neches River below Ioni Creek	Neches_blw_IoniCr	1601.6	6,900	13,500	18,600	26,200	35,500	51,200	69,200	98,500
Neches River above San Pedro Creek	Neches_abv_SanPedroCr	1637.6	4,300	9,700	15,100	25,500	34,800	50,000	68,000	97,000
San Pedro Creek	SanPedroCr_S010	134.9	5,200	11,000	15,400	21,800	27,000	33,500	39,900	49,100
Neches River below San Pedro Creek	Neches_blw_SanPedroCr	1772.6	7,700	15,000	20,700	29,300	36,400	52,600	72,600	104,600
Neches River at TX-21 Bridge, former USGS gage near Alto 08032500	Neches_at_TX-21	1943.4	6,800	15,200	23,700	38,100	52,600	69,200	87,700	119,400
Neches River above Hickory Creek	Neches_abv_HickoryCr	2008.3	6,100	13,300	20,700	33,600	45,700	62,700	83,600	119,400
Hickory Creek above Neches River	HickoryCr_S010	90.5	4,400	9,200	13,300	17,500	21,000	25,900	31,100	37,600
Neches River below Hickory Creek	Neches_blw_HickoryCr	2098.8	8,200	16,300	23,900	35,500	48,000	66,100	89,000	123,800
Neches River at TX-7 bridge near Pollok, TX	Neches_at_TX-7	2236.5	7,200	13,900	22,200	37,200	49,100	68,700	91,900	133,900
Neches River at TX-94 bridge near Apple Springs, TX	Neches_at_TX-94	2433.3	11,200	19,900	27,900	36,200	47,100	67,100	91,500	134,600
Neches River nr Diboll, USGS gage 08033000 at US-59 bridge	NechesRv_nr_Diboll	2726.2	10,100	21,100	29,400	40,100	49,600	71,700	97,100	149,400
Neches River above Piney Creek	Neches_abv_PineyCr	2941.0	10,500	14,700	21,800	34,800	47,100	67,500	91,400	129,000
Piney Creek above Neches River	PineyCr_S010	247.7	4,800	9,500	14,600	19,300	23,400	29,800	36,200	45,300
Piney Creek at US-59 bridge near Corrigan, TX	PineyCr_at_US-59	247.7	4,800	9,500	14,600	19,300	23,400	29,800	36,200	45,300
Piney Creek above Neches River	PineyCr_abv_Neches	374.4	3,700	7,200	12,400	17,600	22,100	31,300	40,100	52,900
Neches River below Piney Creek	Neches_blw_PineyCr	3315.4	14,200	21,100	28,300	40,800	53,100	76,800	103,500	147,500

Location Description	HEC-HMS Element Name	Drainage Area	50%	20%	10%	4%	2%	1%	0.50%	0.20%
		sq mi	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	200-YR	500-YR
Neches River near Rockland, USGS gage 08033500 at US-69 bridge	NechesRv_nr_Rockland	3633.1	14,000	25,900	34,800	45,600	54,000	77,700	104,300	147,000
Neches River above the Angelina River	Neches_abv_Angelina	3791.1	15,200	17,800	27,000	40,800	52,900	76,000	101,900	142,800
Inflow to Lake Striker	StrikerCr_S011	182.0	3,800	6,900	9,600	13,800	17,400	22,100	26,700	33,500
Striker Creek below Lake Striker	StrikerCr_blw_LkStriker	182.0	3,100	6,000	8,600	12,600	16,000	20,100	23,800	28,600
Striker Creek above Angelina River	StrikerCr_abv_Angelina	201.8	2,300	4,300	6,300	10,000	13,400	18,000	22,400	28,500
Angelina River above Striker Creek	Angelina_S010	224.4	4,200	6,700	9,000	11,600	15,100	19,500	23,700	29,700
Angelina River below Striker Creek	Angelina_blw_StrikerCr	426.3	5,600	9,800	13,700	19,900	26,900	36,000	44,000	56,100
Angelina River above Mud Creek	Angelina_abv_MudCr	638.6	5,800	10,300	14,600	20,700	28,500	39,300	49,700	64,900
Mud Creek above West Mud Creek	MudCr_abv_WestMudCr	172.0	2,600	5,200	7,100	9,700	12,700	18,200	23,800	32,200
West Mud Creek above Mud Creek	West_MudCr_S010	92.5	3,200	6,200	8,300	11,200	14,300	17,700	21,100	26,100
Mud Creek below West Mud Creek	MudCr_blw_WestMudCr	264.5	4,900	10,000	13,900	19,400	25,700	33,200	41,200	54,400
Mud Creek near Jacksonville, USGS gage 08034500 at US-79 bridge	MudCr_nr_Jacksonville	377.4	3,800	7,600	11,300	18,100	26,300	37,300	48,700	66,000
Mud Creek at US-84 bridge, near Reklaw, TX	MudCr_at_US-84	523.3	5,200	8,300	11,100	15,500	23,400	34,400	46,400	64,900
Mud Creek above the Angelina River	MudCr_abv_Angelina	556.3	2,700	5,500	9,100	15,200	22,900	33,900	45,700	64,200
Angelina River below Mud Creek	Angelina_blw_MudCr	1194.8	7,900	15,300	23,200	35,400	51,200	73,200	95,300	128,200
Angelina River near Alto, USGS gage 08036500 at TX-21 bridge	AngelinaRv_nr_Alto	1286.4	6,700	13,500	20,400	31,300	46,000	66,700	87,900	120,000
Angelina River above Bayou Loco	Angelina_abv_BayouLoco	1415.8	6,400	13,100	19,600	30,000	44,400	64,300	86,500	120,200
Inflow to Lake Nacogdoches	Lk_Nacogdoches_S010	89.0	5,900	12,000	15,000	18,700	22,300	27,600	33,200	40,300
Bayou Loco below Lake Nacogdoches	BayouLoco_blw_LkNacogdoches	89.0	1,000	2,400	2,800	2,900	3,400	6,200	9,400	13,600
Bayou Loco above Angelina River	BayouLoco_abv_Angelina	102.5	1,000	2,300	2,800	3,300	3,900	6,100	9,500	14,000
Angelina River above Bayou Loco	Angelina_blw_BayouLoco	1518.3	6,700	14,000	20,800	31,800	47,200	67,400	90,000	124,900
Angelina River at Hwy 59 near Lufkin USGS gage, above Bayou La Nana	Angelina_abv_BayouLaNana	1621.5	7,700	14,800	20,700	31,500	46,500	66,200	88,700	122,400
Bayou La Nana above the Angelina River	Bayou_La_Nana_S010	83.3	4,000	7,000	9,200	12,700	15,700	19,500	23,200	28,400

Location Description	HEC-HMS Element Name	Drainage Area	50%	20%	10%	4%	2%	1%	0.50%	0.20%
		sq mi	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	200-YR	500-YR
Angelina River below Bayou La Nana	Angelina_blw_BayouLaNana	1704.9	9,500	19,100	23,600	31,700	46,700	66,600	89,200	123,400
Angelina River above Bayou Carrizo	Angelina_abv_BayouCarrizo	1842.3	7,600	13,600	20,100	30,500	44,800	64,200	85,300	120,800
Bayou Carrizo above Angelina River	Bayou_Carrizo_S010	110.2	5,800	9,800	12,900	17,800	22,100	27,600	32,800	40,200
Angelina River below Bayou Carrizo	Angelina_blw_BayouCarrizo	1952.4	12,300	20,100	26,700	36,900	45,700	64,700	85,900	121,600
Attoyac Bayou below West Creek	AttoyacRv_blw_WestCr	314.2	12,000	20,800	26,600	35,200	40,100	51,000	61,200	75,600
Attoyac Bayou above Big Iron Ore Creek	Attoyac_abv_BigIronOreCr	388.1	6,000	13,200	18,400	26,900	31,300	44,900	56,600	73,500
Big Iron Ore Creek above Attoyac Bayou	BigIronOreCr_S010	97.2	6,000	9,800	12,400	16,200	18,400	23,000	27,400	33,600
Attoyac Bayou below Big Iron Ore Creek	Attoyac_blw_BigIronOreCr	485.2	7,400	14,900	20,900	30,900	36,200	52,300	66,400	87,100
Attoyac Bayou nr Chireno, USGS gage 08038000 at TX-21 bridge	Attoyac_Bayou_nr_Chireno	503.1	6,500	14,300	20,000	29,400	34,500	49,700	63,300	84,800
Attoyac Bayou above Angelina River	Attoyac_abv_Angelina	670.7	8,100	13,800	19,600	29,500	35,300	51,900	67,400	90,400
Angelina River below Attoyac Bayou	Angelina_blw_Attoyac	2808.0	30,100	48,500	64,500	89,700	111,100	138,600	165,100	203,500
Ayish Bayou near San Augustine, USGS gage 08039100 at TX-103 bridge	Ayish_Bayou_nr_San_Augustine	88.6	4,100	8,500	12,100	16,900	20,500	25,500	30,200	37,100
Ayish Bayou above Sam Rayburn Reservoir	Ayish_Bayou_abv_SamRayburn	202.1	9,000	14,300	19,000	26,100	32,200	39,900	47,300	58,100
Total Inflow to Sam Rayburn Dam	SamRayburn_Inflow	3451.8	61,400	96,800	129,200	180,700	226,000	283,900	340,800	420,200
Angelina River below Sam Rayburn	Angelina_blw_SamRayburn	3451.8	8,400	14,000	14,000	14,000	15,700	17,000	18,600	43,900
Angelina River above the Neches River (with release from Sam Rayburn)	Angelina_abv_Neches	3566.9	8,600	14,300	14,900	16,300	19,500	24,500	29,300	44,300
Angelina River above the Neches River (with Sam Rayburn's gates shut)	Angelina_S070	115.1	4,400	6,700	9,100	12,000	16,600	20,800	24,900	30,800
Total Inflow to Town Bluff Dam	TownBluff_Inflow	7569.3	27,600	36,700	47,300	59,900	81,200	100,800	121,400	174,400
Neches River below Town Bluff Dam, USGS gage 08040600	NechesRv_nr_TownBluff	7569.3	20,000	32,200	41,900	55,900	69,200	93,400	119,500	169,600
Neches River below Big Creek	Neches_blw_BigCr	7673.6	21,800	32,400	42,100	56,100	71,000	93,600	119,800	170,100
Neches River below Mill Creek at FM 1013 bridge	Neches_at_FM1013	7716.9	21,700	32,400	42,100	56,100	70,100	93,600	119,700	170,000

Location Description	HEC-HMS Element Name	Drainage Area	50%	20%	10%	4%	2%	1%	0.50%	0.20%
		sq mi	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	200-YR	500-YR
Neches River below Black Branch	Neches_blw_BlackBranch	7784.9	21,400	32,400	42,100	56,200	69,300	93,600	119,700	170,000
Neches River at Evadale	Neches_at_Evadale	7894.7	21,200	32,500	42,200	56,200	69,300	93,500	119,300	170,200
Neches River below Evadale	Neches_blw_Evadale	7950.3	21,300	32,600	42,300	56,400	69,500	93,800	119,600	170,600
Neches River above Village Creek	Neches_abv_VillageCr	8001.7	21,200	32,700	42,300	56,300	69,400	93,600	119,200	169,900
Village Creek at US-69 bridge	VillageCr_at_US-69	265.0	4,700	10,900	17,000	25,300	30,600	38,600	47,800	61,100
Village Creek above Turkey Creek	VillageCr_abv_TurkeyCr	423.1	5,600	13,500	21,100	35,600	43,100	56,400	71,700	93,000
Big Cypress Creek at US-69	BigCypressCr_at_US-69	85.1	3,700	8,000	11,500	16,100	19,000	23,000	27,500	34,000
Turkey Creek at FM 1943	TurkeyCr_at_FM1943	141.4	3,600	9,300	15,700	23,900	28,800	35,500	43,000	53,600
Turkey Creek above Village Creek	TurkeyCr_abv_VillageCr	166.3	2,800	6,900	11,700	19,400	24,300	31,700	40,400	53,000
Village Creek below Turkey Creek	VillageCr_blw_TurkeyCr	589.4	8,100	19,000	29,300	47,100	56,800	73,300	93,500	121,600
Village Creek above Beech Creek	VillageCr_abv_BeechCr	601.7	7,300	16,600	26,200	43,800	56,100	73,000	93,000	121,000
Beech Creek above Village Creek	BeechCr_S010	213.0	5,200	10,800	16,300	23,300	27,600	33,800	41,100	51,700
Village Creek below Beech Creek	VillageCr_blw_BeechCr	814.7	9,600	21,300	33,900	57,600	74,900	98,400	125,800	163,900
Village Creek near Kountze, USGS gage 08041500 at FM 418 bridge	VillageCr_nr_Kountze	861.1	9,400	21,100	33,000	54,100	71,000	98,100	126,600	165,600
Village Creek above Cypress Creek	VillageCr_abv_CypressCr	864.4	8,400	20,300	32,900	53,200	69,000	90,300	123,200	163,400
Cypress Creek above Village Creek	CypressCr_S010	199.7	1,500	4,900	7,000	10,700	14,300	19,400	24,500	32,400
Village Creek below Cypress Creek	VillageCr_blw_CypressCr	1064.0	9,300	23,400	37,400	61,000	80,500	107,500	146,500	194,800
Village Creek at US-96 bridge near Lumberton, TX	VillageCr_at_US-96	1104.4	7,900	20,700	33,200	56,200	71,800	95,300	121,600	172,700
Village Creek above Neches River	VillageCr_abv_Neches	1113.9	7,800	20,500	32,900	55,900	70,700	93,800	118,500	172,300
Neches River below Village Creek	Neches_blw_VillageCr	9115.6	27,800	44,800	64,500	93,700	123,400	166,600	209,300	266,300
Neches River above Pine Island Bayou	Neches_abv_PineIsBayou	9132.5	27,500	43,400	62,000	88,100	114,400	157,300	202,200	262,700
Willow Creek above Pine Island Bayou	WillowCr_S010	206.1	4,400	8,800	12,600	18,400	23,300	30,400	38,400	50,600
Pine Island Bayou above Willow Creek	PineIsBayou_S011	171.8	2,300	4,400	6,500	9,600	12,300	16,300	20,700	27,500
Pine Island Bayou below Willow Creek	PineIsBayou_blw_WillowCr	377.9	5,400	10,800	15,900	23,700	30,700	41,400	53,000	70,600

Location Description	HEC-HMS Element Name	Drainage Area	50%	20%	10%	4%	2%	1%	0.50%	0.20%
		sq mi	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	200-YR	500-YR
Pine Island Bayou near Sour Lake, USGS gage 08041700 at Old Beaumont Rd bridge	PineIsBayou_nr_SourLake	397.7	4,400	9,400	14,700	23,100	30,500	41,800	53,600	72,200
Pine Island Bayou above Little Pine Island Bayou	PineIsBayou_abv_LittlPIBayou	417.3	3,900	8,200	13,400	21,900	29,400	41,200	53,200	71,900
Little Pine Island Bayou above Pine Island Bayou	Little_PIBayou_S010	134.8	2,100	3,700	5,000	6,800	8,400	11,100	14,000	18,600
Pine Island Bayou below Little Pine Island Bayou	PineIsBayou_blw_LittlPIBayou	552.2	5,400	11,300	17,800	28,300	37,600	52,100	67,000	90,200
Pine Island Bayou above BI Pump Plant	PineIsBayou_abv_BIPumpPlant	697.7	6,200	13,000	20,300	32,600	43,800	61,200	79,200	107,600
Pine Island Bayou above the Neches River	PineIsBayou_abv_Neches	726.2	6,300	13,200	20,700	33,400	45,000	62,900	81,500	110,900
Neches River below Pine Island Bayou	Neches_blw_PineIsBayou	9858.6	33,300	55,000	77,500	112,900	146,800	203,900	265,400	359,900
Neches River at the Saltwater Barrier	Neches_at_SaltwaterBarrier	9858.7	33,300	55,000	77,500	112,900	146,800	203,900	265,400	359,900

Table 6.9: Peak Reservoir Pool Elevations (feet NAVD88) from the HEC-HMS Uniform Rainfall Frequency Storms

Reservoir Name	Drainage Area	50%	20%	10%	4%	2%	1%	0.50%	0.20%
	sq mi	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	200-YR	500-YR
Lake Athens	21.6	441.1	441.7	442.2	442.9	443.6	444.6	445.7	446.9
Lake Palestine	838.1	346.6	347.6	348.3	349.5	350.5	351.6	352.7	354.2
Lake Jacksonville	39.6	423.4	424.8	426.0	427.8	429.5	431.4	432.8	434.4
Lake Striker	182.0	293.3	293.6	293.8	294.2	294.5	295.1	295.8	297.1
Lake Tyler	113.3	376.5	377.2	377.7	378.6	379.3	380.1	381.0	382.1
Lake Nacogdoches	89.0	280.5	282.6	283.8	285.2	286.7	288.4	289.9	291.3
Sam Rayburn Reservoir	3451.8	165.7	166.7	167.9	169.9	171.7	174.2	176.5	178.5
B.A. Steinhagen Lake	7569.3	83.0	83.3	83.6	83.9	84.5	86.0	87.4	90.7

6.6.3 Uniform Rainfall Frequency Results versus Drainage Area

As a quality check, the peak flow results from the 1% AEP uniform rainfall frequency storms were plotted versus drainage area and outliers were examined, as shown in Figure 6.25. This figure shows that the analyzed junctions followed generally expected patterns of increasing peak flow with drainage area, with exceptions for the effects of large lakes and reservoirs.

As one can see from this figure, the peak inflows to large lakes, such as Sam Rayburn, tend to be high outliers. This is because the entire lake surface is treated as a single point within HEC-HMS. However, the effects of that peak inflow on pool elevation are mitigated by the large amounts of storage in the lake. Resulting pool elevations and outflows from the dam are reasonable and correctly reflect the operations of the dam.

From this figure, one can also see that Lake Palestine and Lake Sam Rayburn caused large reductions in peak flow downstream, while Town Bluff dam (B.A. Steinhagen Lake) caused a much more modest decrease in peak flow. This reflects Lake Palestine and Sam Rayburn's larger amounts of storage relative to their drainage areas, whereas the available storage in B.A. Steinhagen Lake is much smaller relative to its drainage area.

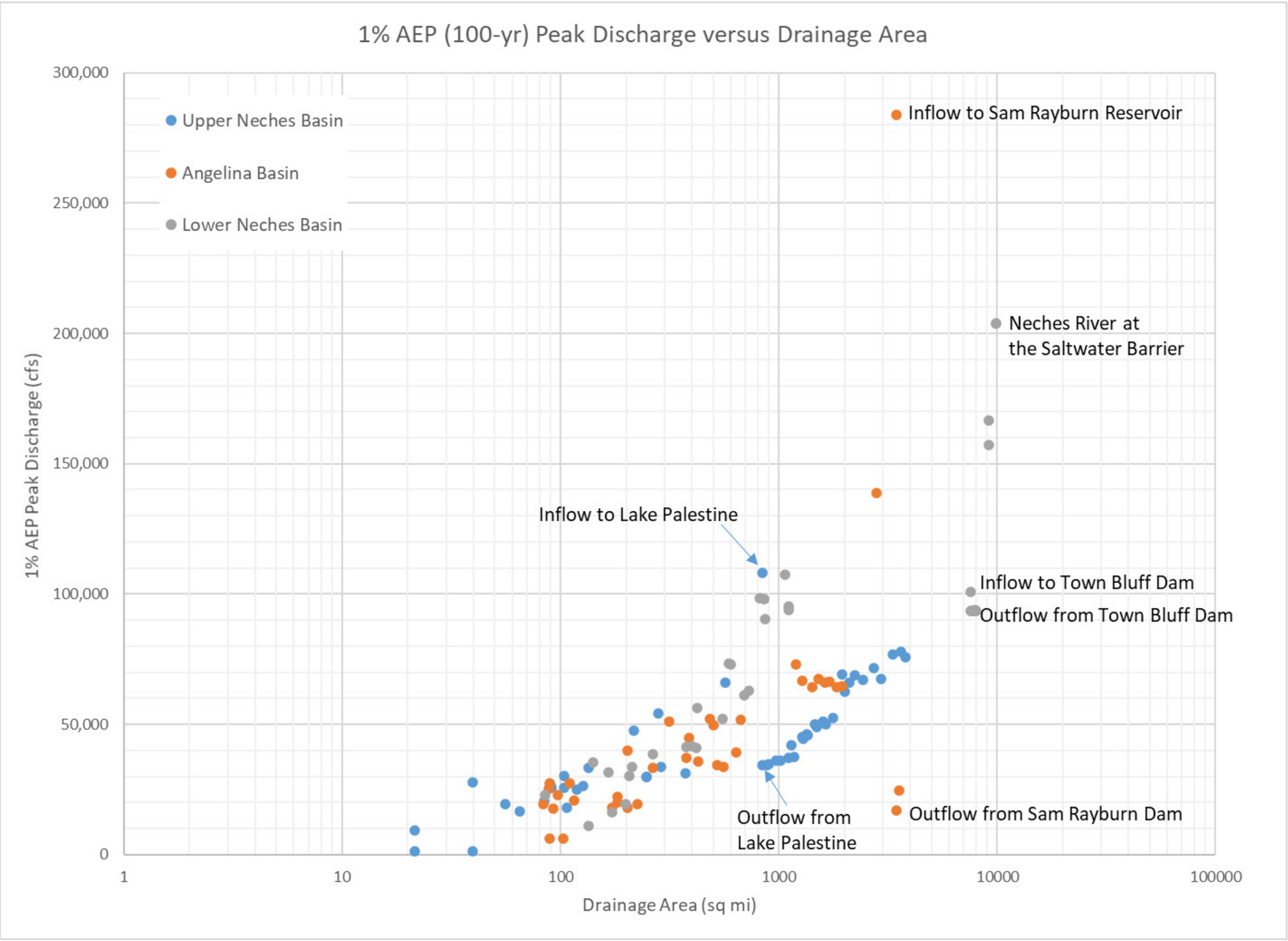


Figure 6.25: NA14 1% AEP Uniform Rain Frequency Storm Results versus Drainage Area

6.7 HEC-HMS MODEL VERIFICATION

After the adoption of the final HEC-HMS parameters and the frequency storm results, a new large flood event occurred in the Neches River Basin in May of 2021. During the second half of May 2021, a two-week period of heavy rainfall resulted in a new record pool elevation for Sam Rayburn Reservoir and the second highest outflow from Town Bluff Dam since Sam Rayburn was completed in 1965 (second only to Hurricane Harvey). As a result, the InFRM team decided to use this event as an opportunity to verify that the HEC-HMS model and its final parameters were accurately simulating the response of the watershed to a new observed flood event.

6.7.1 May 2021 Storm Event

May 2021 was a period of heavy rainfall that heavily impacted Sam Rayburn Reservoir, B.A. Steinhagen Lake, and the lower portions of the Neches River basin. Consistent rainfall throughout April and May led to wet antecedent conditions in the watershed. During the second half of May, more intense storms developed. Rainfall totals ranged from 6 – 11 inches across the Neches basin over a two-week period. Figure 6.26 shows the daily rainfall totals at Sam Rayburn Dam leading up to and during the May 2021 storm event, and Figure 6.27 illustrates the total depth of rain for this verification storm and how that rain was spatially distributed across the Neches River watershed. This plot was extracted from the HEC-MetVue meteorological program for visualizing and processing rainfall data.

Table 6.10 lists the observed peak discharges and peak reservoir elevations that resulted from the May 2021 storm event at 21 USGS observed data locations. Sam Rayburn Reservoir in particular received over 1.25 million acre-feet of inflow, which is equivalent to over 6.8 inches of basin average runoff. This event resulted in a new record pool elevation for Sam Rayburn Reservoir, with the observed pool elevation peaking at just 0.6 feet below the emergency spillway crest. This event also resulted in the second highest observed release from Town Bluff Dam since the impoundment of Sam Rayburn Reservoir in 1965.

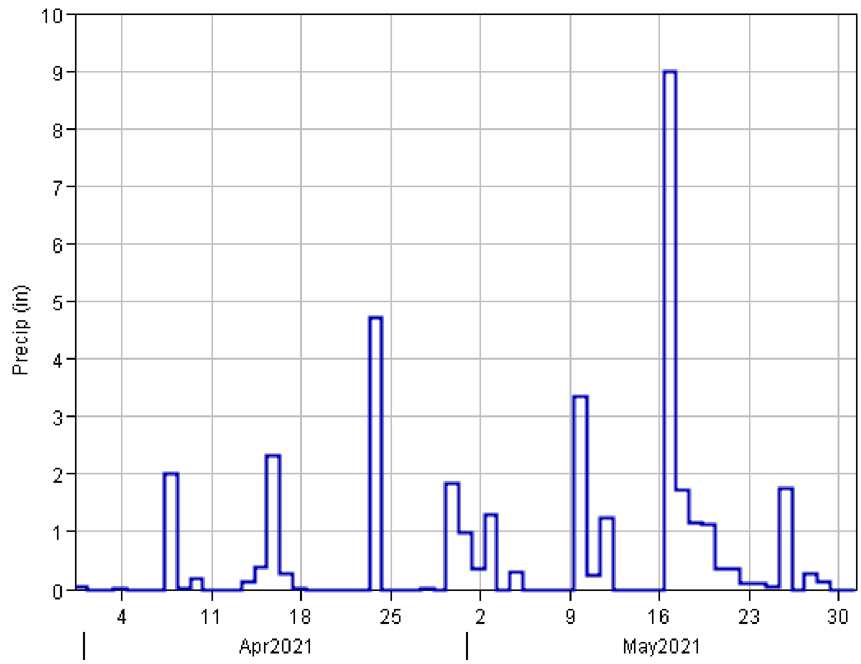


Figure 6.26: Daily Rainfall Totals (inches) at Sam Rayburn Dam for April-May 2021

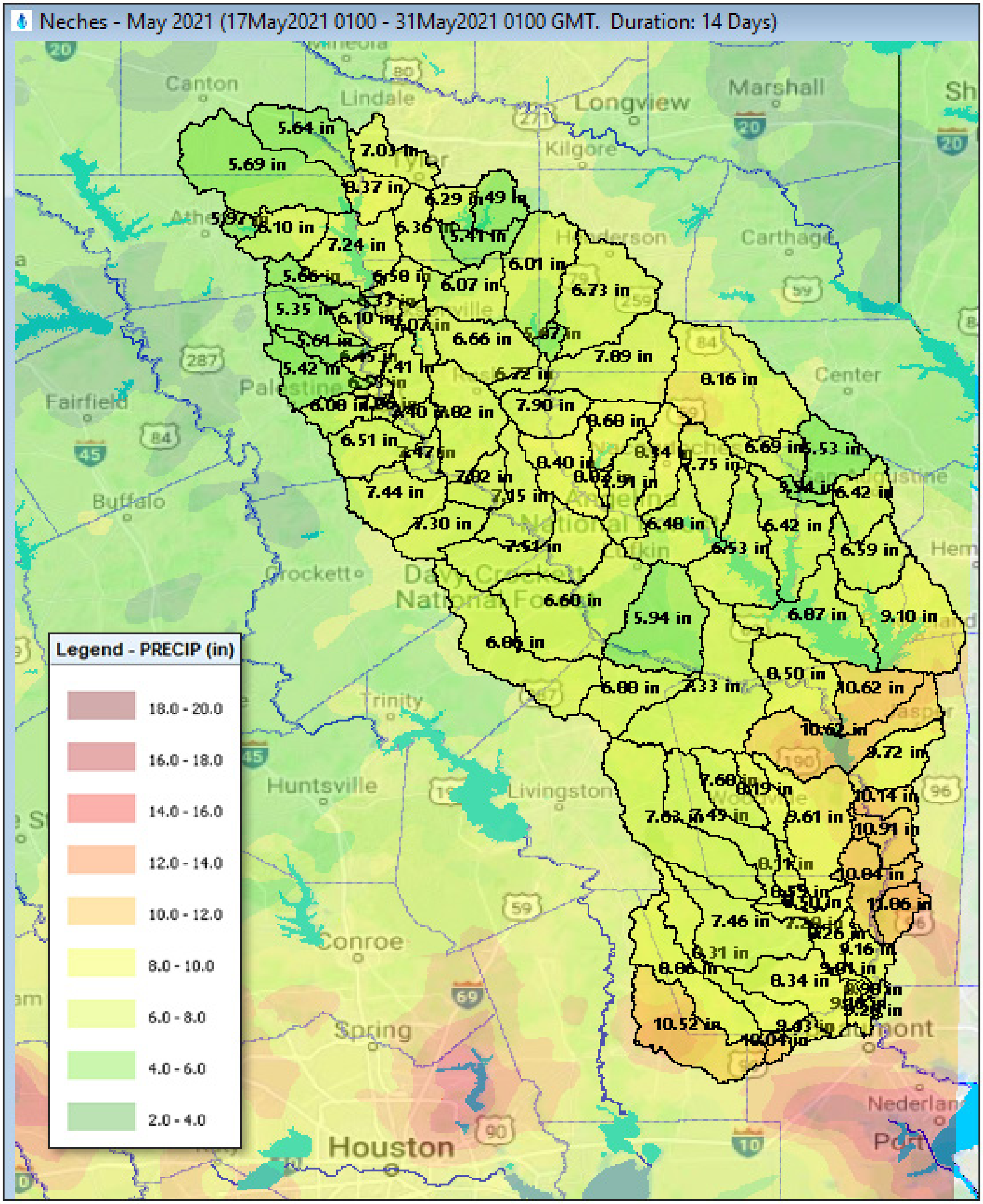


Figure 6.27: Total Rainfall Depths (inches) for the May 2021 Storm Event

Table 6.10: Observed Peak Stream Flow and Reservoir Elevations from the May 2021 Flood Event

USGS Gage Location	Peak Observed Stream Flow (cfs) or Reservoir Elevation (ft NAVD 88)	Notes
Lake Athens	441.13	
Lake Palestine	347.37	
Neches River nr Neches, TX USGS Gage 08032000 at US-79 bridge	7,150	
Lake Jacksonville	423.20	
Angelina River near Alto, USGS gage 08036500 at TX-21 bridge	11,400	
Neches River nr Diboll, USGS gage 08033000 at US-59 bridge	19,200	
Neches River near Rockland, USGS gage 08033500 at US-69 bridge	23,000	
Lake Tyler	376.36	
Mud Creek near Jacksonville, USGS gage 08034500 at US-79 bridge	3,110	
Lake Striker	293.5	
Angelina River near Alto, USGS gage 08036500 at TX-21 bridge	10,100	
Lake Nacogdoches	281.64	
Attoyac Bayou nr Chireno, USGS gage 08038000 at TX-21 bridge	5,560	
Ayish Bayou near San Augustine, USGS gage 08039100 at TX-103 bridge	2,330	
Sam Rayburn Reservoir	175.4	New Record Pool Elevation
Town Bluff Dam	84.1	
Neches River nr Town Bluff, USGS Gage 08040600	52,000	2nd Highest Release since 1965
Neches River at Evadale, USGS Gage 08041000	44,000	
Village Creek near Kountze, USGS gage 08041500 at FM 418 bridge	14,000	
Pine Island Bayou near Sour Lake, USGS gage 08041700 at Old Beaumont Rd	5,870	
Pine Island Bayou above BI Pump Plant, USGS Gage 08041749	12,500	
Neches River at the Saltwater Barrier, USGS Gage 08041780	78,300	3rd Highest Peak Flow

6.7.2 Model Verification Methodology

The goal of the verification event was to verify that the HEC-HMS model and its final parameters are accurately simulating the response of the watershed to observed flood events. For the May 2021 verification storm, the National Weather Service (NWS) hourly rainfall radar data was applied to the final HEC-HMS model as a gridded product with 4 km by 4 km cell sizes. Then, the model's calculated flow hydrographs were compared to the observed USGS stream flow hydrographs at the gages, and the model's calculated reservoir pool elevations were compared to the observed USGS reservoir elevation data.

Since the May 2021 storm event was being run for the purposes of model verification, minimal changes were made to the model's parameters in order to match the watershed's initial conditions at the beginning of that storm event. The only HEC-HMS model parameters that were adjusted for the verification event were the initial and constant losses, the initial baseflow, and the initial pool elevations for the reservoirs. The calibration events showed that these parameters varied considerably from one time period to the next based on the antecedent moisture conditions in the watershed. In other words, these parameters can be expected to change based on how wet or dry the watershed was at the beginning of a given storm event. All other HEC-HMS model parameters were left unchanged from the final model parameters described in Section 6.5. The adjusted model parameters for the May 2021 verification event can be seen in Tables B.35 and B.36 of Appendix B.

6.7.3 May 2021 Verification Results

The verification results for the May 2021 storm event showed that the HEC-HMS model was able to accurately simulate the response of the watershed, as it reproduced the observed hydrographs at most locations very well. The resulting hydrograph comparisons can be seen in the following figures of this section. The figures show the HEC-HMS computed versus the USGS observed flow hydrographs at each stream gage location. For each reservoir, the figures show the HEC-HMS computed pool elevation versus the USGS observed pool elevation.

In addition to graphical comparisons of simulated to observed flow hydrographs, statistical tests were also employed to evaluate the model's performance. The same statistical metrics and ratings were applied for the verification event as were applied for the calibration events, as described in Section 6.4.4. Table 6.11 lists a summary of the model performance ratings for all the observed data locations that were available for the May 2021 storm event. The individual statistical metrics used to assign these performance ratings are shown on the figure for each observed location.

As shown in Table 6.11, the model's performance was rated as Very Good for over 70% of the observed data locations for the May 2021 verification event, including the most significantly impacted locations of Sam Rayburn Reservoir and the Neches River near Town Bluff. These Very Good ratings indicate that the HEC-HMS model performed very well in all three statistical metrics when compared to the observed data for the validation event. Of the remaining six locations, three were rated as Satisfactory and three were rated as Unsatisfactory. The three Satisfactory ratings occurred at a few of the smaller reservoirs in the basin: Lake Jacksonville, Lake Tyler, and Lake Striker. For those three lakes, May 2021 was an insignificant event that resulted in less than one foot of rise in each of their respective pool elevations. The model was calibrated to larger flood events, which do not have the same watershed response as these smaller runoff events. The three Unsatisfactory ratings occurred at the Neches River nr Rockland, B.A. Steinhagen Lake, and Ayish Bayou. For the Neches River near Rockland, this was also an insignificant event in that it had a very flat observed flow hydrograph with only small changes in observed discharge. B.A. Steinhagen Lake had issues with all of its observed events due to its relatively small flood storage volume and the uncertainty in its outflows (see Section 6.4.4 for more information). Finally, Ayish Bayou near San Augustine had an unsatisfactory rating primarily due to the impacts of missing observed data during the low flow periods. As a whole, the May 2021 verification event confirmed that the HEC-HMS model accurately simulates the response of the watershed, and the model performed very well when compared to the observed data.

Table 6.11: Summary of HEC-HMS Model Verification Performance Ratings

USGS Gage Location	May 2021 Verification Performance Rating	NOTES:
Lake Athens	Very Good	
Lake Palestine	Very Good	
Neches River nr Neches, TX USGS Gage 08032000	Very Good	
Lake Jacksonville	Satisfactory	Insignificant: pool rise was less than 1 foot
Neches Rv nr Alto, TX, USGS Gage 08032500	Very Good	
Neches River nr Diboll, USGS gage 08033000	Very Good	
Neches River near Rockland, USGS gage 08033500	Unsatisfactory	Insignificant: very little change in observed flow during the event
Lake Tyler	Satisfactory	Insignificant: pool rise was less than 1 foot
Mud Creek near Jacksonville, USGS gage 08034500	Very Good	
Lake Striker	Satisfactory	Insignificant: pool rise was less than 1 foot
Angelina River near Alto, USGS gage 08036500	Very Good	
Lake Nacogdoches	Very Good	
Attoyac Bayou nr Chireno, USGS gage 08038000	Very Good	
Ayish Bayou near San Augustine, USGS gage 08039100	Unsatisfactory	Lower rating due to missing observed data during the low flow periods.
Sam Rayburn Reservoir	Very Good	New Record Pool: 0.6 ft below the Spillway
B.A. Steinhagen Lake (Town Bluff Dam)	Unsatisfactory	Lower rating due to small storage volume and uncertainty in dam's releases.
Neches River nr Town Bluff, USGS Gage 08040600	Very Good	2nd highest release since Sam Rayburn's completion in 1965
Neches River at Evadale, USGS Gage 08041000	Very Good	
Village Creek near Kountze, USGS gage 08041500	Very Good	
Pine Island Bayou near Sour Lake, USGS gage 08041700	Very Good	
Pine Island Bayou above BI Pump Plant, USGS Gage 08041749	Very Good	
Neches River at the Saltwater Barrier, USGS Gage 08041780	Very Good	3rd highest peak flow on record

For the sake of brevity, only a few verification plots at the most significantly impacted locations have been included as examples in this section of the report. The resulting verification comparisons for all of the available locations compared for the May 2021 event can be seen in Appendix B.

There are two types of figures which are shown in this section of the report: reservoirs and streamflow gages. In the reservoir figure for Sam Rayburn (Figure 6.28), the observed pool elevation at the reservoir gage is compared to the modeled pool elevation in the top half of the figure. The other lines on this plot shows reservoir storage, inflow, and outflow, but are not relevant to the comparison with the observed pool elevations.

In the streamflow gage figures (Figures 6.29 to 6.31), the solid blue line represents the total modeled streamflow at the gage, while the black line represents the observed streamflow that was recorded by the gage. The other dotted blue lines on these figures represent the runoff from individual model components (i.e., a single subbasin or routing reach), and they should be ignored as they are not relevant to the gage comparison.

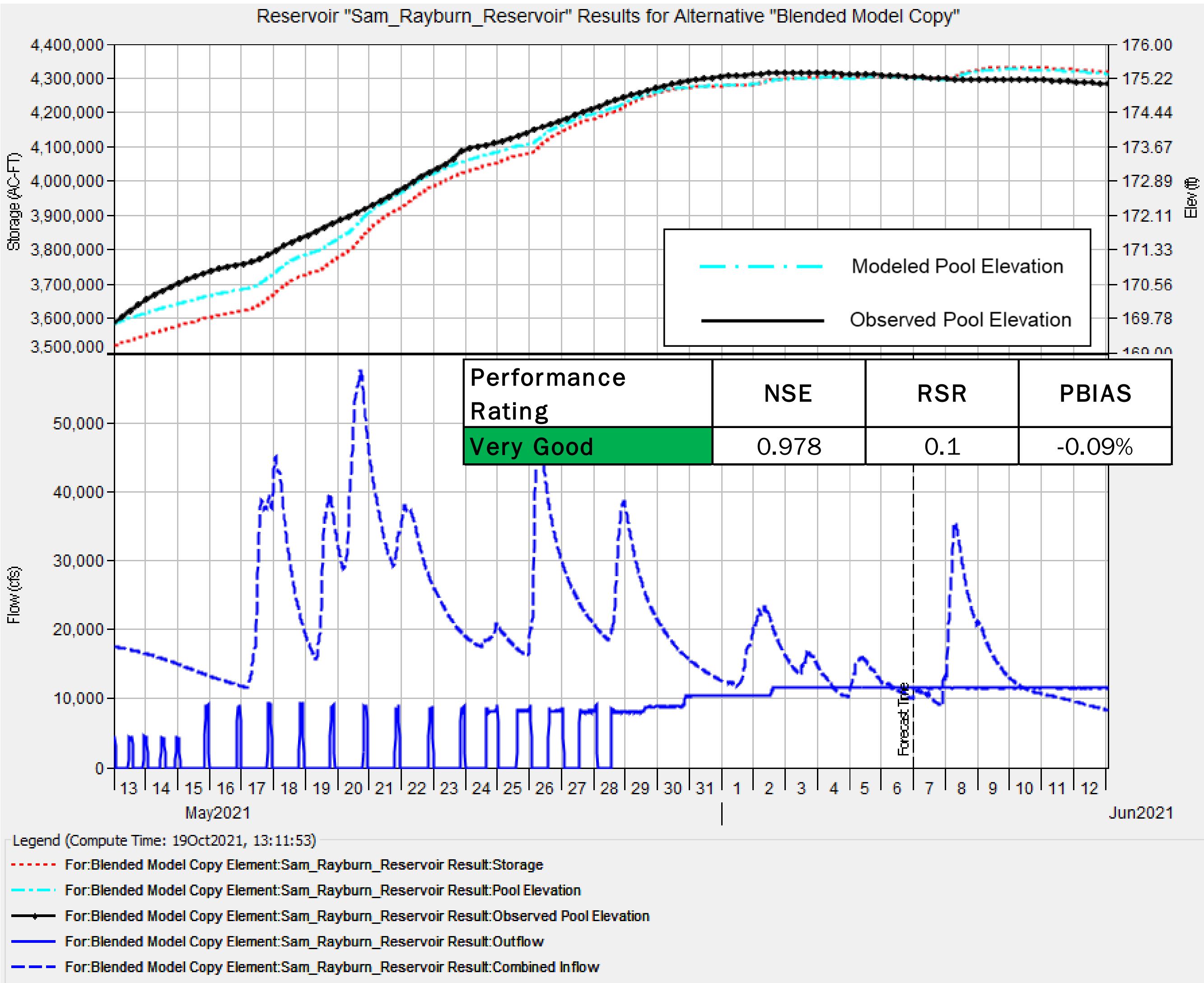


Figure 6.28: May 2021 Verification Results for Sam Rayburn Reservoir

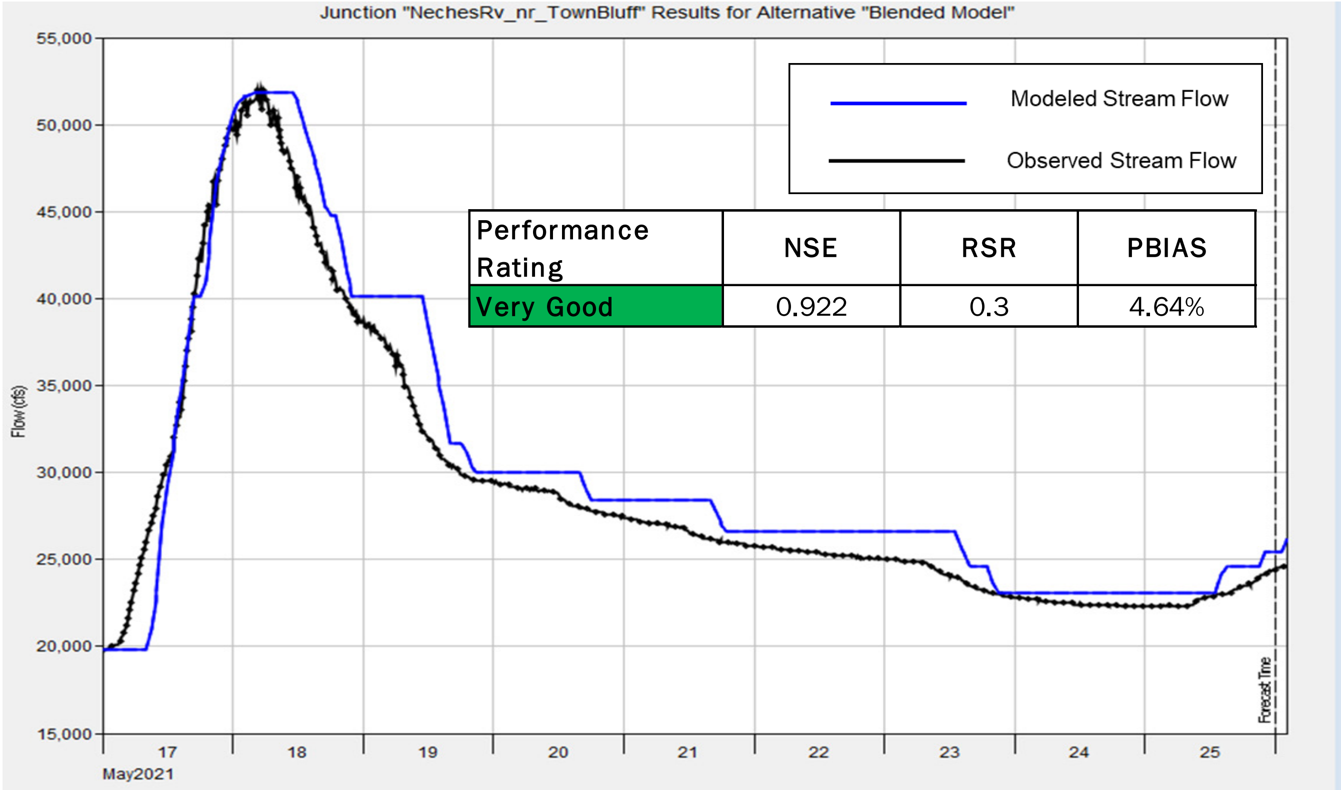


Figure 6.29: May 2021 Verification Results for Neches River nr Town Bluff

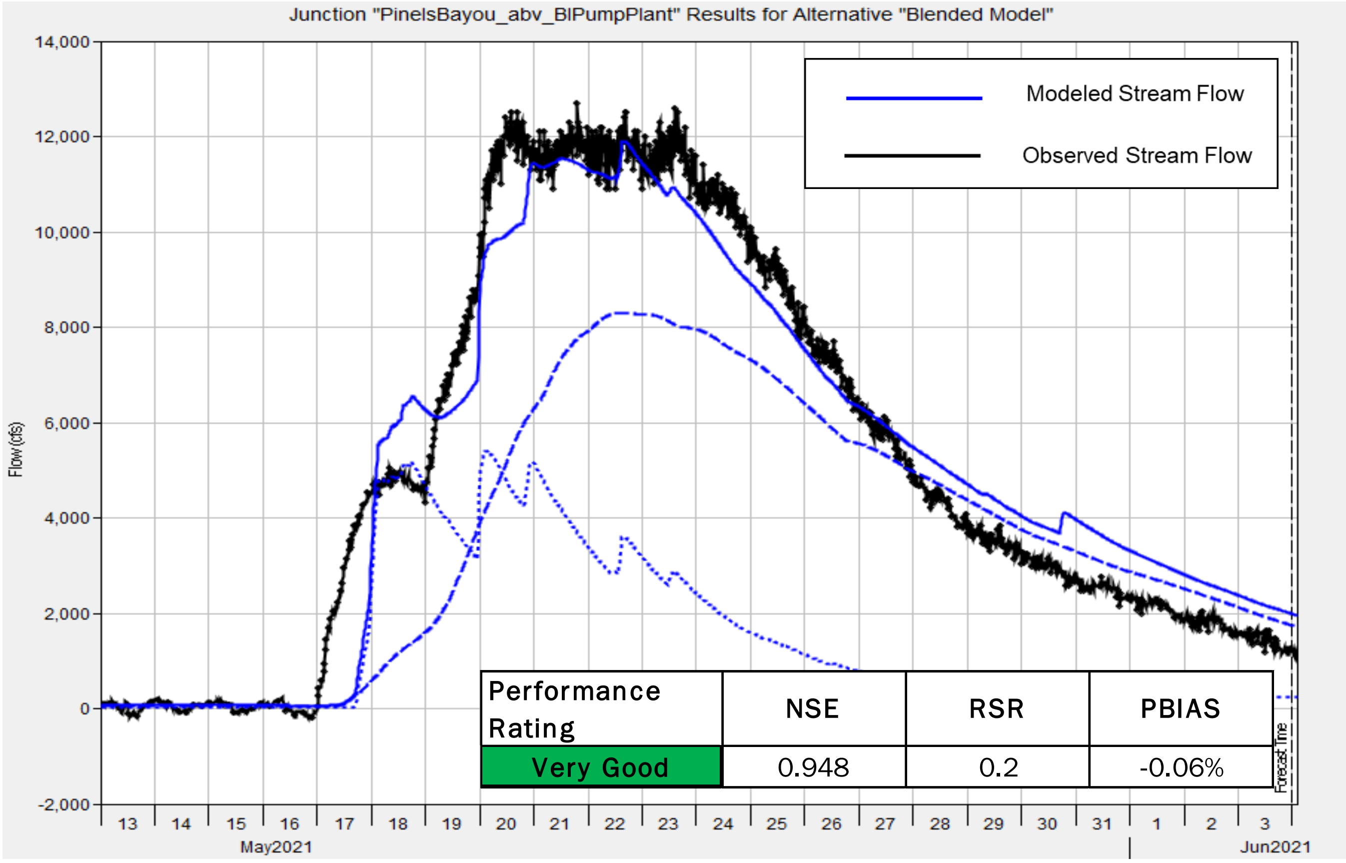


Figure 6.30: May 2021 Verification Results for Pine Island Bayou above BI Pump Plant

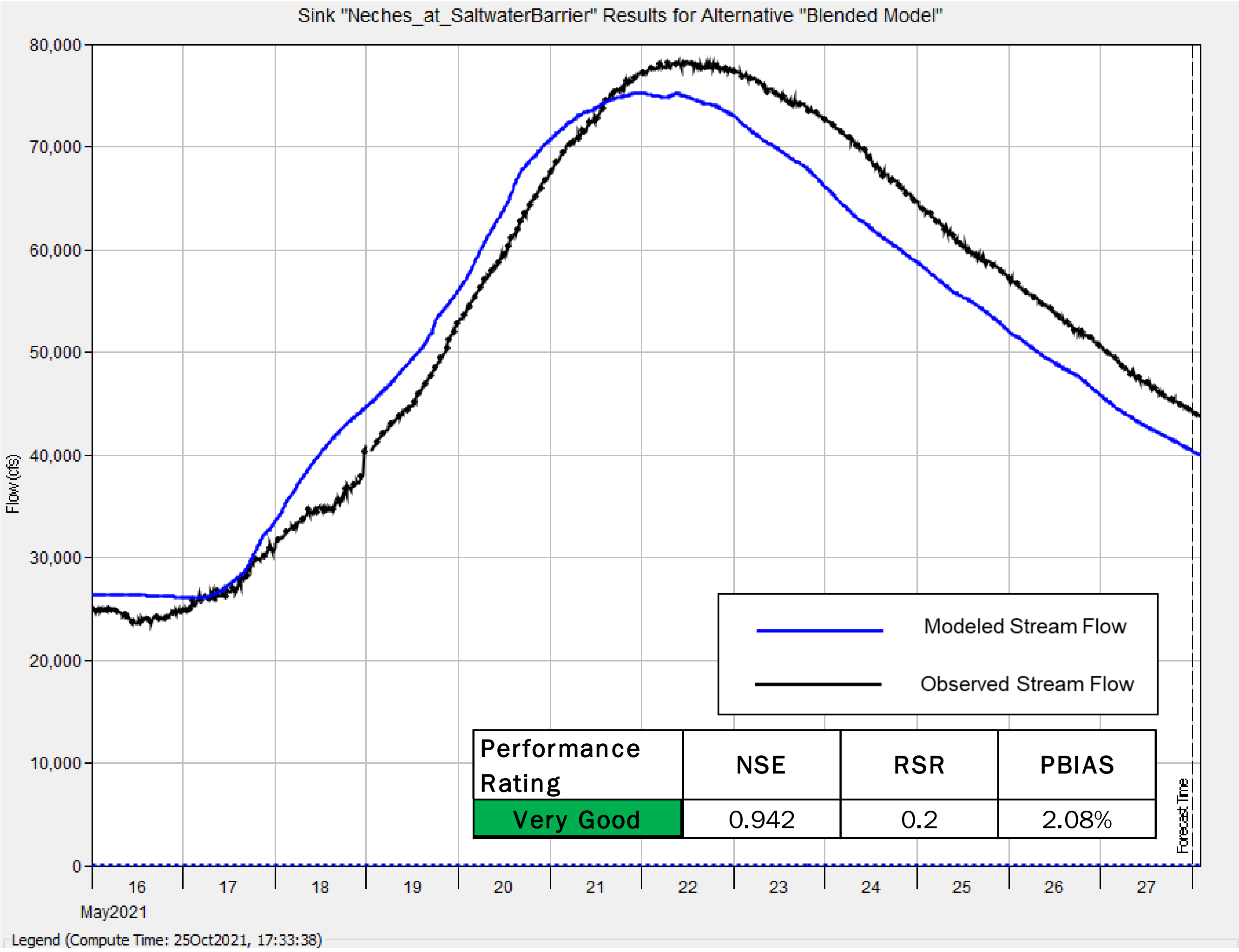


Figure 6.31: May 2021 Verification Results for Neches River at the Saltwater Barrier

7 Elliptical Frequency Storms in HEC-HMS

7.1 INTRODUCTION TO ELLIPTICAL STORMS

Observations of actual storm events show that average precipitation intensity decreases as the area of a storm increases (Meyers, 1980) (Asquith, 2000). The uniform rainfall method results (documented in the previous chapter and Appendix B) use the depth-area analysis in HEC-HMS to produce frequency peak flow estimates (Version 4.4; USACE, 2018). The depth-area analysis in HEC-HMS applies the appropriate depth-area reduction factor to the given point rainfall depths based on the drainage area at a given evaluation point, which are derived from the published depth-area reduction factors from Figure 15 of the National Weather Service TP-40 publication (Hershfield, 1961), as shown in the figure below.

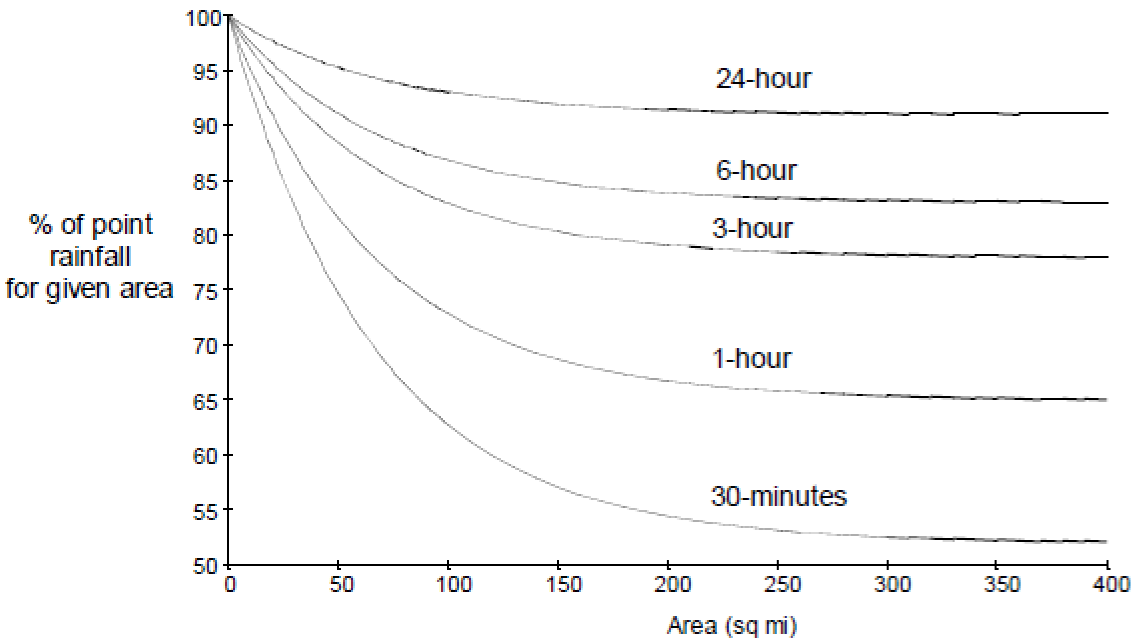


Figure 7.1: Published Depth-Area Reduction Curves from TP-40

When evaluating a stream location with a drainage area greater than 400 square miles, the HEC-HMS software issues a warning that the NWS depth-area reduction factors do not support storms beyond 400 square miles, as seen in the figure above. The program will still calculate the peak discharge, but the warning implies that the calculated volume of the storm may be overestimated for larger drainage areas.

Since the Neches hydrology study involves calculating frequency discharges for points with up to nearly 10,000 square miles of drainage area, the InFRM team developed elliptical frequency storms for gage points and junctions with drainage areas greater than 400 square miles. In these elliptical frequency storms, the same point rainfall depths and durations were applied as in the uniform rainfall method, but the spatial distribution of the rainfall varied in an elliptical shaped pattern with higher rainfall amounts in the center of the ellipse and lesser amounts towards the outer fringes.

Elliptical shaped storms have been used in a variety of hypothetical design applications, including the Probable Maximum Precipitation (PMP) storms from Hydrometeorological Report No 52 (HMR 52) (Hansen, 1982). The elliptical frequency storms constructed for this study are similar to those of HMR 52 in that concentric ellipses are used to construct the storm's spatial pattern, and the storm's location is optimized over the watershed by identifying the storm center location and the angle of its major axis that lead to a maximum peak flow at a downstream junction of interest. Figure 7.2 shows an example of an elliptical 1% annual exceedance probability (100-yr) storm that was optimized over the watershed above the Neches River nr Rockland, TX USGS gage. This particular junction has a contributing drainage area of approximately 3,600 square miles.

This chapter provides a general summary of the methods and results from the elliptical frequency storm analyses that were completed for the InFRM Watershed Hydrology Assessment of the Neches River Basin, but additional details on the development and application of the elliptical frequency storms are available in Appendix C: Elliptical Frequency Storms in HEC-HMS.

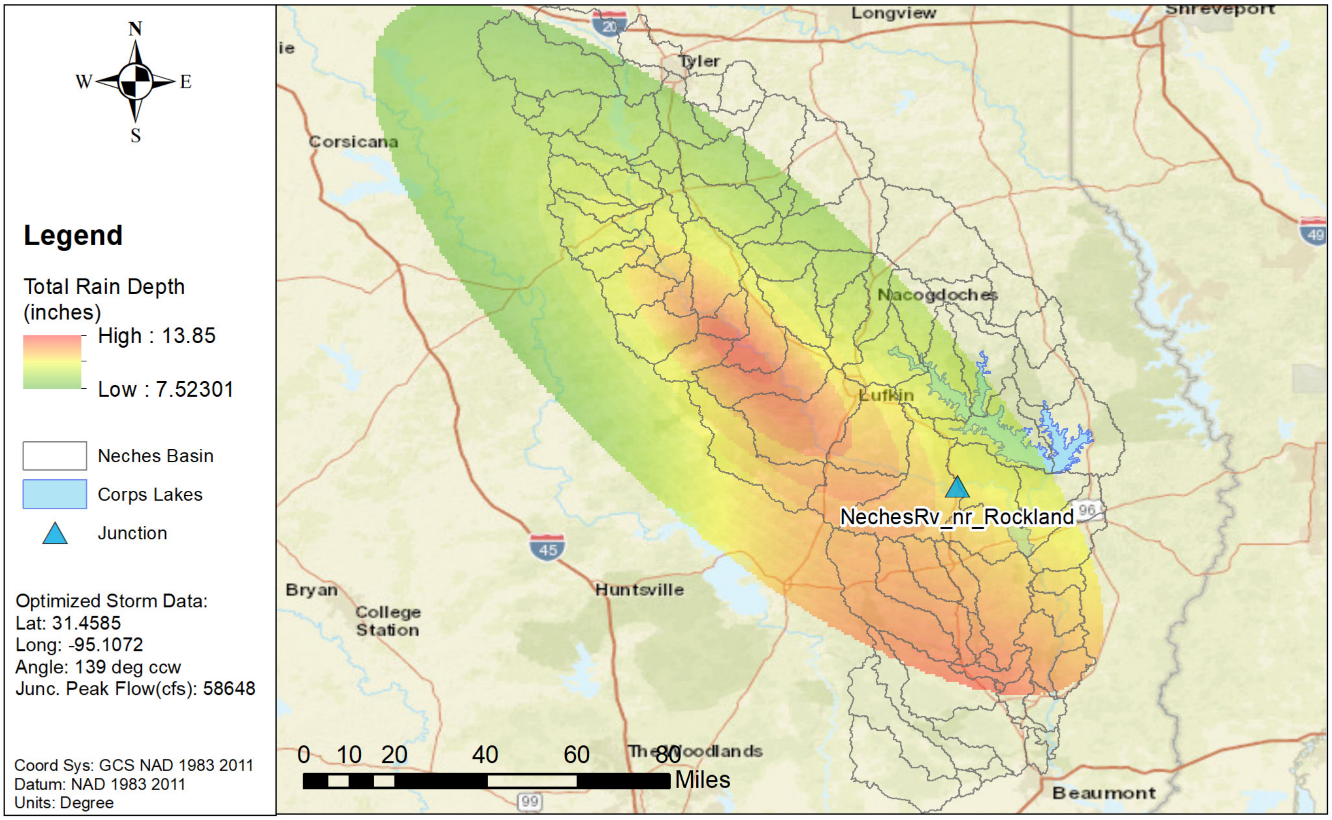


Figure 7.2: Example 1% AEP (100-yr) Elliptical Frequency Storm

7.2 ELLIPTICAL STORM PARAMETERS

The elliptical storm parameters covered below in sections 7.2.1 through 7.2.5 are applicable to the entire Neches Basin. Unique, optimized elliptical storm configurations were developed for 76 different junction elements within the Neches HEC-HMS model, 15 of which were USGS stream gage locations.

When comparing the upper reaches of the Neches Basin with the downstream portion closer to the Gulf of Mexico, the meteorology is noticeably different as demonstrated below in Figure 7.3. In the upper Neches, the NOAA Atlas 14 precipitation gradient is relatively uniform due to the similar, largely convective storm fronts that frequent the region. Moving downstream, however, the precipitation gradient in the lower half of the Neches increases drastically as it nears the Gulf. Historic rain events in the lower Neches can be the result of either convective storms or tropical storms like Hurricane Harvey. The meteorological distinction of the upper and the lower Neches was addressed in the sampling of the point precipitation depths and in the development of the depth-area-reduction curves (covered in depth in sections 7.2.3 and 7.2.4, respectively).

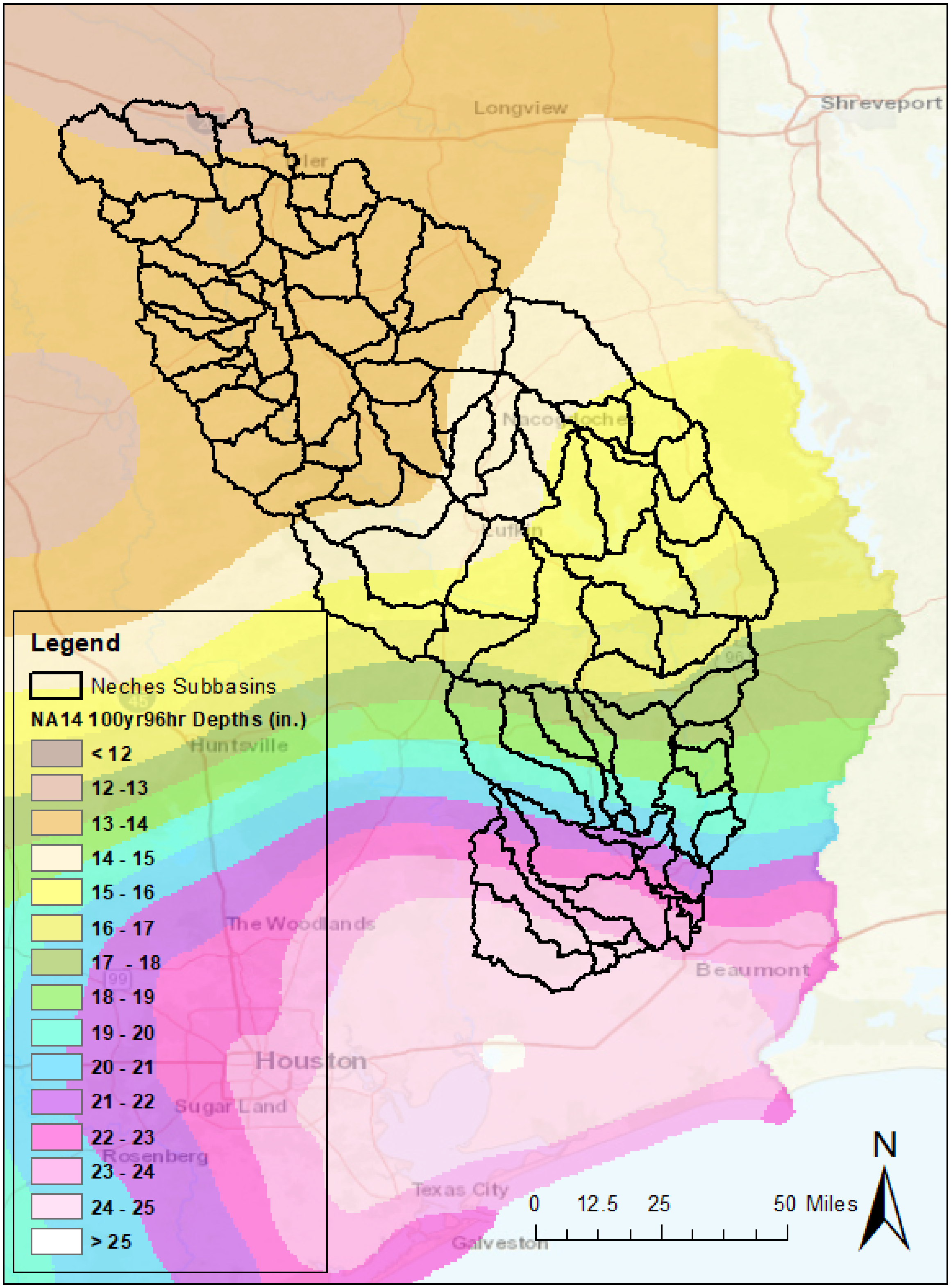


Figure 7.3: NOAA Atlas 14 100-yr 96-hr Precipitation Gradient – Neches River Basin

7.2.1 Elliptical Storm Area

This study uses a storm extent of 10,000 square miles. This is due, in part, to historical rainfall studies rarely including data beyond 10,000 square miles (USACE, 1945). However, many of the more recent, historic storm events analyzed in southeast Texas for this study did extend to 10,000 square miles and beyond in coverage. Furthermore, 10,000 square miles also coincides well with the total drainage area of the Neches River basin. While this storm extent is somewhat arbitrary, testing was done in previous InFRM studies to limit the storm extent to 3,000 square miles and the resulting peak discharges were only slightly reduced.

7.2.2 Storm Ellipse Ratio

The HMR-52 study presents the option to design a storm with a major: minor ellipse axis ratio ranging from 2:1 to 3:1. For the final results, a 2.5:1 ellipse was used, as it matched well with the general shape of the Neches basin. A 3:1 ellipse was tested in several sections within the Neches basin which led to only nominal differences in regard to optimized storm centerings, storm orientations, and resulting peak flows when compared to the results obtained from using a 2.5:1 ellipse.

7.2.3 Elliptical Storm Rainfall Depths

Elliptical storms were designed for each of the following annual exceedance probabilities (AEP): 1 in 2 years, 1 in 5 years, 1 in 10 years, 1 in 25 years, 1 in 50 years, 1 in 100 years, 1 in 200 years and 1 in 500 years. Point rainfall depths and durations were applied directly from NOAA Atlas 14 Volume 11 which contains depth duration frequency estimates of precipitation for the state of Texas (NOAA, 2018). The point precipitation values that were applied to each elliptical storm were based on the storm's optimized location, not the location of the outlet of interest. It is important to note that out of all the design storm parameters that are discussed here, peak flows were most sensitive to adjustments in the NOAA Atlas 14 point frequency depths.

For the Neches basin, since the precipitation gradient varies rapidly near the Gulf, all of the precipitation depths that fell under the 10,000 sqmi elliptical storm positioning were queried instead of just the one depth at the storm center. Then all of the queried precipitation depths were reduced based on which of the concentric, DAR ellipses they overlapped with (demonstrated in Figure 7.8). In regions where the precipitation depths vary greatly over a short distance, this method performs better since the precipitation gradient is reflected in the makeup of the elliptical storm.

7.2.4 Storm Depth Area Reduction (DAR) Factors

A depth-area-duration (DAD) table can be used to track the volume of a historic storm event, both spatially and temporally. For this design storm analysis, HEC-MetVUE software (Version 3.1; USACE, 2019) was utilized to compute a depth-area-duration table for each observed storm event using the NWS gridded hourly rainfall radar data. A depth-area-reduction (DAR) factor table can be derived from a DAD table; applying DAR factors to a storm results in a storm that has been spatially normalized to a unit depth at the storm center. Thus, the remainder of the storm proceeding outward from the storm center is a fraction of the center depth. Examples of a DAD table, DAR factor table, and DAR curve processed for a single, observed Neches storm event can be seen below in Table 7.1, Table 7.2, and Figure 7.4.

Table 7.1: Example Depth-Area-Durations from an Observed Event.

Area(sqmi)	1-hr (in.)	2-hr (in.)	3-hr (in.)	6-hr (in.)	12-hr (in.)	24-hr (in.)	48-hr (in.)	72-hr (in.)
10	3.070	4.002	4.313	10.149	16.298	18.457	27.374	27.453
20	2.819	3.793	4.074	9.934	16.015	18.215	26.878	26.930
30	2.716	3.639	4.010	9.762	15.792	18.037	26.474	26.536
40	2.632	3.578	3.947	9.637	15.566	17.895	26.174	26.253
50	2.570	3.518	3.883	9.524	15.340	17.796	25.951	26.015
60	2.508	3.457	3.843	9.414	15.157	17.704	25.751	25.822
70	2.455	3.401	3.810	9.306	14.973	17.619	25.582	25.647
80	2.417	3.361	3.777	9.201	14.786	17.558	25.424	25.486
90	2.378	3.322	3.744	9.112	14.610	17.497	25.278	25.337
100	2.340	3.282	3.711	9.025	14.441	17.436	25.140	25.197
200	2.058	3.013	3.500	8.342	12.973	17.017	23.863	23.921
300	1.904	2.838	3.351	7.826	11.993	16.695	22.835	22.903
400	1.779	2.723	3.226	7.417	11.282	16.377	22.013	22.085
500	1.707	2.617	3.117	7.083	10.744	16.054	21.275	21.351
600	1.635	2.536	3.029	6.800	10.318	15.722	20.594	20.676
700	1.563	2.466	2.941	6.548	9.970	15.406	19.996	20.082
800	1.502	2.397	2.874	6.324	9.669	15.093	19.456	19.547
900	1.467	2.338	2.817	6.123	9.412	14.787	18.961	19.061
1000	1.431	2.286	2.759	5.941	9.181	14.503	18.506	18.617

Table 7.2: Example Depth-Area-Reductions (Derived from the Storm of Table 7.1).

Area(sqmi)	1-hr DAR	2-hr DAR	3-hr DAR	6-hr DAR	12-hr DAR	24-hr DAR	48-hr DAR	72-hr DAR
10	1	1	1	1	1	1	1	1
20	0.918	0.948	0.945	0.979	0.983	0.987	0.982	0.981
30	0.885	0.909	0.930	0.962	0.969	0.977	0.967	0.967
40	0.857	0.894	0.915	0.950	0.955	0.970	0.956	0.956
50	0.837	0.879	0.900	0.938	0.941	0.964	0.948	0.948
60	0.817	0.864	0.891	0.928	0.930	0.959	0.941	0.941
70	0.800	0.850	0.883	0.917	0.919	0.955	0.935	0.934
80	0.787	0.840	0.876	0.907	0.907	0.951	0.929	0.928
90	0.775	0.830	0.868	0.898	0.896	0.948	0.923	0.923
100	0.762	0.820	0.860	0.889	0.886	0.945	0.918	0.918
200	0.670	0.753	0.811	0.822	0.796	0.922	0.872	0.871
300	0.620	0.709	0.777	0.771	0.736	0.905	0.834	0.834
400	0.579	0.681	0.748	0.731	0.692	0.887	0.804	0.804
500	0.556	0.654	0.723	0.698	0.659	0.870	0.777	0.778
600	0.532	0.634	0.702	0.670	0.633	0.852	0.752	0.753
700	0.509	0.616	0.682	0.645	0.612	0.835	0.730	0.732
800	0.489	0.599	0.666	0.623	0.593	0.818	0.711	0.712
900	0.478	0.584	0.653	0.603	0.577	0.801	0.693	0.694
1000	0.466	0.571	0.640	0.585	0.563	0.786	0.676	0.678

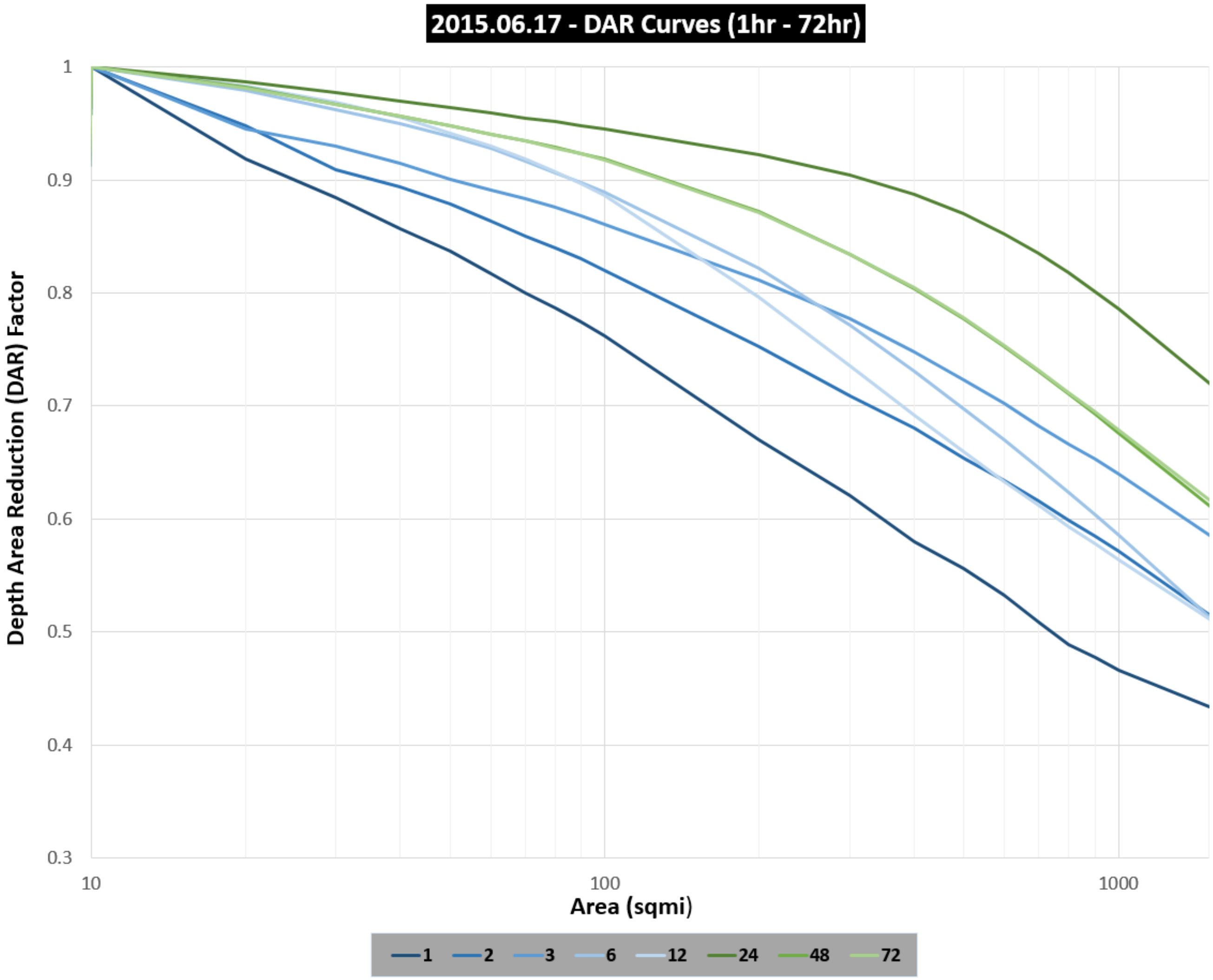


Figure 7.4: Example Depth-Area-Reduction Curves (Plotted from Table 7.2)

A storm catalog consisting of approximately 30 large, rainfall events that occurred within or in close proximity to the Neches basin were used in the DAD and DAR analyses for this study. A set of DAR curves (1 hour to 96 hour) was developed for each event. Given the meteorological differences between the upper and lower Neches, the rainfall event data were initially bifurcated into two groups for separate analysis; the separation was based on which half of the Neches the storms fell closest to. Storms that fell in the upper half of the Neches were classified as 100-year events if the maximum observed storm depths fell within the lower and upper 90% confidence bounds for the NOAA Atlas 14 100-year precipitation frequency estimates in the upper Neches. Likewise, storms in the lower Neches were similarly classified based on the confidence bounds for the 100 year precipitation frequency estimates in the lower Neches. An individual storm was allowed to be classified as a 100 year event for one duration and not another. For example, the 96 hour precipitation depth for Hurricane Harvey was much greater than the 100 year 96 hour precipitation upper confidence limits for the lower Neches basin. Therefore, Harvey was not classified as an eligible 100 year 96 hour event. However, the 1 hour precipitation depth for Harvey did fall within the 100 year 1 hour confidence bounds and was thus classified as a 100 year 1 hour event.

The 1, 24, and 96 hour DAR curves for the classified 100 year upper Neches storms were averaged and compared to the 1, 24, and 96 hour DAR curves for the 100 year lower Neches storms. Only nominal differences were observed when comparing the two averaged datasets for the 24 and 96 hour. When comparing the 1 hour average curves, the lower Neches curve was more reducing than the upper Neches curve. However, the 1 hour 100 year storm subset was also the smallest sample size and a confident conclusion regarding the data could not be made. Therefore, the upper and lower Neches curves were ultimately combined to create a singular set of DAR curves that were applied to the entire basin. After several sensitivity runs, the 75th percentile DAR curve for each duration was adopted. To ensure that the DAR curves for each duration nested nicely without any overlap, the 1hr, 24hr, and 96hr curves were used and the intermediate durations were interpolated. The final set of 75th percentile nested DAR factors used in this study can be observed in Table 7.3 and Figure 7.5.

Table 7.3: Adopted Depth-Area-Reduction Values for the Neches InFRM Watershed Hydrology Assessment

Area(sqmi)	*1-hr DAR*	2-hr DAR	3-hr DAR	6-hr DAR	12-hr DAR	*24-hr DAR*	48-hr DAR	72-hr DAR	*96-hr DAR*
10	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
20	0.970	0.972	0.974	0.976	0.978	0.980	0.980	0.980	0.979
30	0.959	0.961	0.964	0.966	0.968	0.971	0.970	0.970	0.970
40	0.946	0.949	0.952	0.954	0.957	0.960	0.961	0.962	0.963
50	0.935	0.938	0.941	0.945	0.948	0.951	0.953	0.955	0.957
60	0.923	0.927	0.931	0.935	0.938	0.942	0.946	0.949	0.952
70	0.912	0.916	0.920	0.925	0.929	0.934	0.938	0.943	0.948
80	0.901	0.906	0.911	0.916	0.921	0.926	0.932	0.938	0.944
90	0.890	0.896	0.902	0.908	0.914	0.920	0.926	0.933	0.940
100	0.878	0.886	0.893	0.901	0.908	0.915	0.922	0.929	0.936
200	0.837	0.846	0.855	0.865	0.874	0.883	0.890	0.896	0.903
300	0.798	0.811	0.823	0.835	0.848	0.860	0.868	0.876	0.883
400	0.761	0.776	0.791	0.806	0.821	0.836	0.846	0.857	0.867
500	0.738	0.753	0.768	0.783	0.798	0.813	0.826	0.838	0.851
600	0.717	0.733	0.748	0.763	0.779	0.794	0.808	0.822	0.835
700	0.700	0.715	0.730	0.746	0.761	0.777	0.792	0.808	0.823
800	0.684	0.700	0.716	0.731	0.747	0.763	0.780	0.797	0.814
900	0.669	0.687	0.704	0.722	0.739	0.757	0.773	0.790	0.806
1000	0.655	0.674	0.693	0.712	0.732	0.751	0.767	0.783	0.800
2000	0.601	0.622	0.642	0.662	0.682	0.702	0.716	0.730	0.744
3000	0.570	0.590	0.609	0.628	0.648	0.667	0.680	0.692	0.705
4000	0.542	0.563	0.584	0.604	0.625	0.646	0.655	0.663	0.672
5000	0.515	0.537	0.559	0.582	0.604	0.627	0.637	0.647	0.657
6000	0.491	0.515	0.538	0.562	0.585	0.609	0.620	0.631	0.642
7000	0.479	0.502	0.525	0.547	0.570	0.593	0.605	0.617	0.628
8000	0.468	0.490	0.513	0.535	0.558	0.580	0.593	0.605	0.617
9000	0.456	0.479	0.501	0.523	0.545	0.568	0.580	0.593	0.606
10000	0.445	0.467	0.489	0.512	0.534	0.556	0.569	0.581	0.594

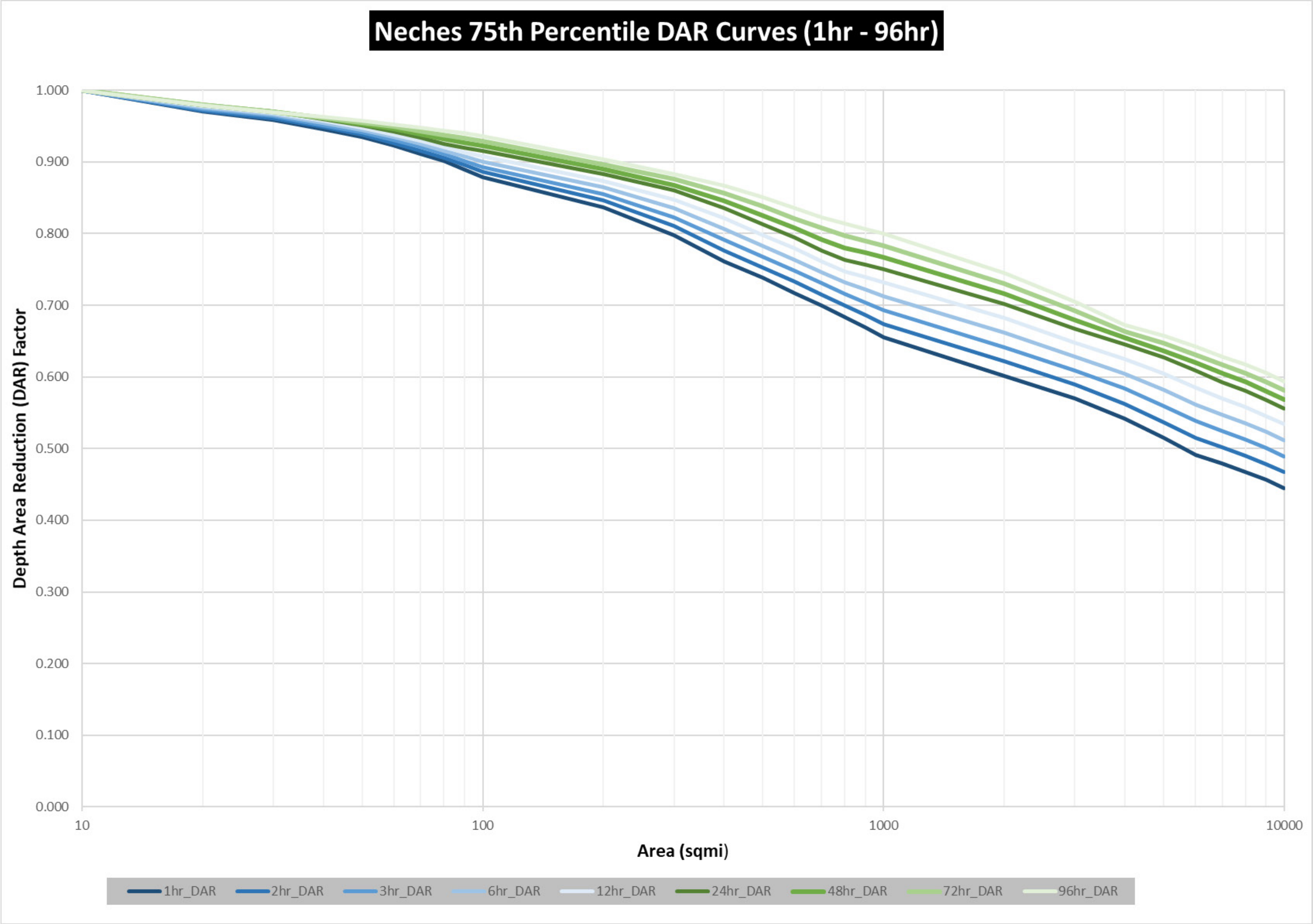


Figure 7.5: Adopted Depth-Area-Reduction Curves for the Neches InFRM Study (Plotted from Table 3)

For this study, the adopted DAR table values were combined with the adopted storm extent of 10,000 square miles (section 7.2.1) and the adopted ellipse ratio of 2.5 to 1 (section 7.2.2) to create rasterized DAR ellipses for each duration. The rasterized DAR ellipse for the 96 hour duration can be seen below in Figure 7.6. The ellipses serve as a blueprint for creating the design storms; then they are rotated, shifted and multiplied by the corresponding NOAA Atlas 14 precipitation rasters to create spatially reduced rainfall for each storm duration.

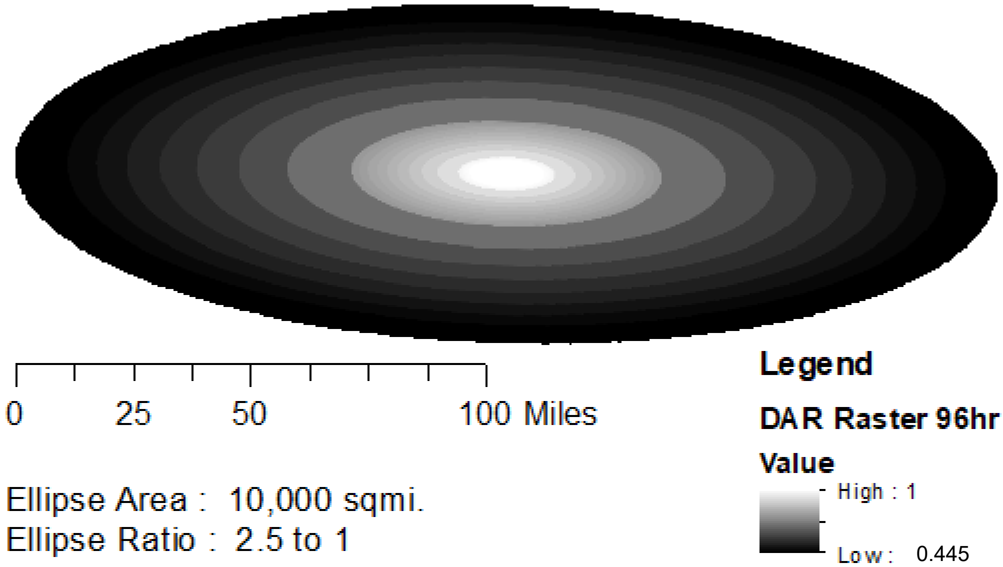


Figure 7.6: Adopted Depth-Area-Reduction Rasterized Ellipse for the 96-hr Duration

It is important to note that the same set of DAR rasters were applied for each elliptical frequency storm analysis (2 year through 500 year). Recent research has been done that compares the spatiotemporal characteristics of “fixed-area” DAR factors and “storm-centered” DAR factors (Kang et al., 2019). The "fixed-area" method is what was used in TP29 and later referenced in TP40 (shown previously in Figure 7.1). It results from an unsynchronized frequency analysis between point and areal rainfall. A second method called the "storm-centered" method typically uses radar data to develop the DAR factors. It is a synchronized method in that the point and areal rainfall data are gathered during the same event. The research by Kang et al. concluded that while DAR curves developed via the “fixed-area” method are insensitive to different frequencies, DAR curves developed via the “storm-centered” approach may very well be sensitive to different frequencies. They found that DAR curves may be more reducing for rare frequencies (i.e., the 100yr event) and less reducing for more common frequencies (i.e., the 5yr event). The InFRM Neches elliptical storm analysis discussed in this appendix used a “storm-centered” approach to develop the DAR curves, but did not collect enough storm data to build different sets of DAR curves for different frequencies. The adopted set of DAR curves were built off of 100 year type events but were applied to all frequencies, rare and common.

7.2.5 Storm Temporal Pattern / Hyetograph

Historically, storms have varying intensities and temporal distributions and many studies have been done to document storm patterns. The six storm temporal distributions that were tested for a previous InFRM study on the Guadalupe Basin are shown in Figure 7.7. The Soil Conservation Service (1986) documented different distributions for the United States. Type II is the distribution applicable to Texas; it was also included in the testing. Other distributions were also previously tested, including the alternating block Frequency Rainfall temporal distributions from HEC-HMS with the storm centroid occurring at the 25%, 33%, 50%, 67%, and 75% of the total distribution. The HEC-HMS Frequency Rainfall alternating block temporal distributions maintain the appropriate storm intensity for all durations throughout the storm. In other words, the 100 year, 1 hour rainfall depth is maintained within the 100 year, 2 hour rainfall depth and so on all the way through the 100 year, 96 hour rainfall depth. For this Neches elliptical storm analysis, temporal distributions with maximum intensities occurring at 33%, 50%, and 67% of the total distribution were tested with a negligible effect on downstream peak flows. Centrally distributed (50%) alternating block temporal distributions were adopted for the final runs.

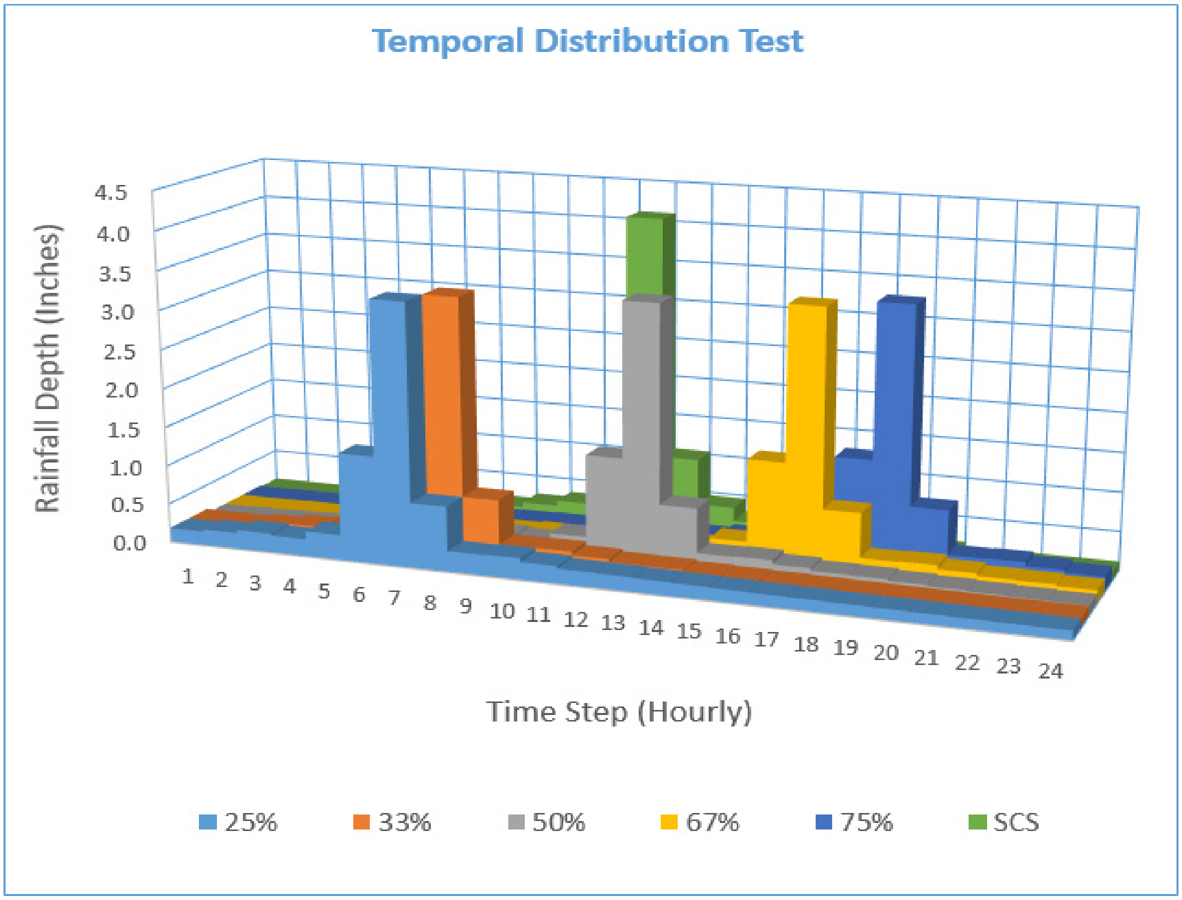


Figure 7.7: Previously Tested Storm Temporal Distributions

During the uniform rainfall analysis covered in a separate appendix, storm durations ranging from 24 to 240 hours were tested on the Neches basin. A duration of 96 hours was ultimately adopted for the uniform rainfall modeling. The 96 hour results yielded slightly higher peak flows when compared to the 24 and 48 hour results, and the difference in peak flows began to taper off for durations greater than 96 hours. Furthermore, the 96 hour duration also coincides well with the duration of several observed, historic rainfall events like Hurricane Harvey. In order to be consistent with the uniform rainfall assumptions, the 96 hour duration was also adopted for the elliptical storm modeling.

7.2.6 Geospatial Process for Building the Elliptical Storms

For this Neches InFRM Watershed Hydrology Assessment, a new geospatial method was developed for creating the rainfall hyetographs that were used as input into the Neches design storm HEC-HMS model. This new method is built on three principal sources of geospatial data: 1) NOAA Atlas 14 precipitation frequency raster data in asci format for the 1, 2, 3, 6, 12, 24, 48, 72, and 96 hour durations, 2) rasterized DAR ellipses that are built off of the adopted DAR curves for each of these durations, and 3) a HEC-HMS subbasin delineated shapefile. For each unique storm location and orientation within the Neches basin, the underlying precipitation data is queried and multiplied by the appropriate rasterized DAR ellipse to get the reduced precipitation for each duration (Figure 7.8). Then zonal statistics are calculated to determine the average reduced precipitation for each subbasin. Using the subbasin-averaged reduced precipitation for the 1, 2, 3, 6, 12, 24, 48, 72, and 96 hour durations, the alternating block method is used to build rainfall hyetographs for each of the subbasins within the design storm HEC-HMS model. The geospatial algorithm employed builds the storm from the central, maximum intensity duration outwards so that the appropriate storm intensity is maintained throughout the entire storm. For example, the 100 year 1 hour rainfall is maintained within the 100 year 2 hour rainfall and so forth all the way out to 96 hours.

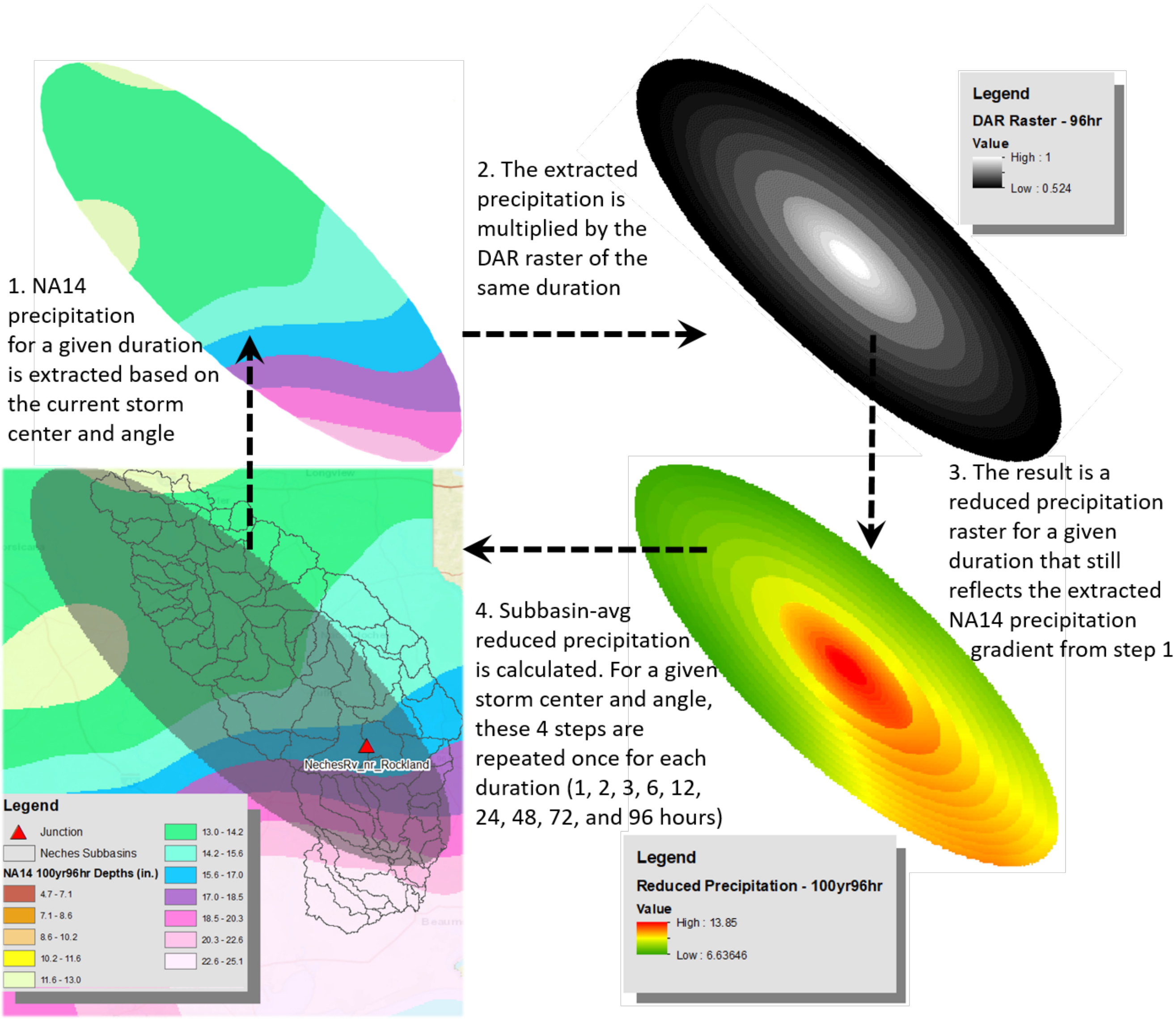


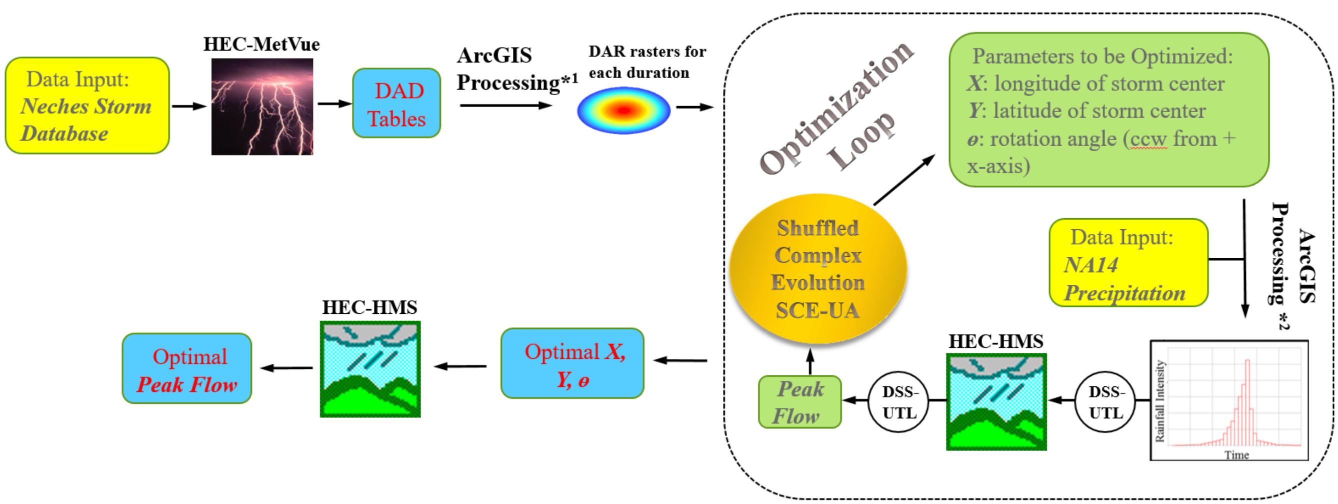
Figure 7.8: Geospatial Process for Building Elliptical Design Storms

7.3 OPTIMIZATION OF THE STORM CENTER LOCATION

For the InFRM Watershed Hydrology Assessments, a script was developed by the University of Texas at Arlington that automatically locates optimal centering locations (x and y) and rotations (ɵ) of spatially varied elliptical frequency storms for a list of receiving junctions in a HEC-HMS basin model. The script was expected to obtain the combination of the three parameters (x, y, and ɵ) that maximized either peak flow at desired junctions or reservoir pool elevations while achieving the following objectives:

- To complete the task efficiently
- To allow users to customize the scripts easily based on their needs
- To generate reasonable results that can be validated manually
- To outperform the manual grid search method in terms of precision, accuracy and efficiency
- To function normally on any machine at USACE with the available software and hardware

The ArcPy Python library, part of Esri's ArcGIS software package, was leveraged for all geospatial operations. The "Optimization Loop" section of Figure 7.9 below illustrates the schematic flow of the storm optimization script. The loop consists of two major components: 1) parameter update/optimization and 2) automatic simulation of the HEC-HMS hydrologic model. In each iteration of the optimization process, the rasterized DAR ellipses for each duration are rotated and shifted to align with the updated parameters (x, y, and ɵ) and then are applied to the corresponding NOAA Atlas 14 precipitation rasters to create spatially reduced rainfall for each storm duration. The spatially reduced depths are then allocated into each subbasin as mean areal precipitation (MAP). The subbasin MAP values for each duration are then manipulated using the alternating block method to create a complete time series (covered in section 7.2.5). The time series MAP values, i.e., the hyetographs, are stored in DSS format and transmitted to the HMS model for simulations. After each simulation, the corresponding peak flow value at a desired junction is extracted from the output DSS file. Based on the extracted peak flow value, an optimization algorithm will update the parameters (x, y and ɵ) and then optimization proceeds into the next iteration. After all optimization iterations for a junction are complete, an optimized storm center (x and y) and orientation (ɵ) that leads to a peak flow at a given junction is determined. The optimization process can then be repeated for the next junction of interest.



*1. involves creating rasterized ellipses for each NA14 duration with DAR values of 1 in the center, and decreasing values towards the outer rings.
*2. involves rotating and shifting each DAR raster, reducing the NA14 precipitation rasters, and calculating zonal statistics for each subbasin.

Figure 7.9: Schematic Flowchart for the Storm Optimization Script

Originally, the scripts were designed to automate a grid search, where all possible combinations of parameters (i.e., the 'grids') are exhaustively tested and the optimal combination of the three parameters (x, y, and ɵ) can then be obtained. Although the approach of grid search seems straightforward, it does suffer from high computational cost because the computational run time depends on the number of grids, which is further constrained by the range and the interval of each parameter. Given the need of maintaining a certain level of precision or keeping constant intervals of the parameters, the UTA team found that the grid search approach might not be appropriate for this project since the computational run time was excessively lengthy – it increases exponentially with greater drainage area (more possible x and y values).

In order to overcome this issue, the UTA team selected a global optimization (GO) algorithm entitled shuffled complex evolution (SCE) (Duan et al., 1993) - a random sampling approach. Instead of exhausting all possible grids, the random sampling approach tests the objective function around some sampled grids in an iteration while learning about the structure of the objective function for improving the sampling of grids in the next iteration. More details about GO and SCE are included in Appendix C.

7.4 ELLIPTICAL STORM LOCATIONS

The final optimized storm center locations (x, y) and rotations (ɵ) for every node of interest in the Neches watershed are listed in Appendix C. Rotation angles are measured counter-clockwise from the positive x-axis. These location and rotation parameters were determined from 100yr frequency optimizations and are assumed to be the same for other frequency events in most cases (2yr – 500yr). Sensitivity testing showed that, in general, optimized locations and orientations did not significantly change between frequency events. Once the optimum storm center location and rotation were determined for each location of interest, the elliptical frequency storms for the standard eight frequency events were constructed using the appropriate NOAA Atlas 14 point rainfall depths. See section 1.4 in Appendix C for additional information.

7.5 ELLIPTICAL FREQUENCY STORM LOSS RATES

The elliptical frequency storms were then applied to the final HEC-HMS basin model with the same frequency loss rates that were used for the uniform rainfall method which were discussed in Chapter 6 and in Appendix B. In some cases, the 2-yr through 10-yr losses were re-adjusted in order to maintain consistency with the frequent end of the statistical frequency curves at the USGS gages. This final adjustment was performed because of the increased level of confidence in the statistical frequency curve for the 2-yr through 10-yr recurrence intervals. The final 2-yr through 25-yr loss rates used for the elliptical frequency storm events are given in Appendix C. The final 50-yr through 500-yr loss rates are the same as those used for the uniform rainfall method and are also shown in Appendix C.

7.6 ELLIPTICAL FREQUENCY STORM RESULTS – PEAK FLOW

The frequency peak flow values were then calculated in HEC-HMS by applying the appropriate, optimized elliptical frequency storms for each junction of interest in the final HEC-HMS basin model. These results will later be compared to the uniform rain results from HEC-HMS along with other methods from this study.

In some cases, one may observe that the simulated peak discharge decreases in the downstream direction. It is not an uncommon phenomenon to see decreasing frequency peak discharges for some river reaches as flood waters spread out into the floodplain and the hydrograph becomes dampened as it moves downstream. This can be due to a combination of peak attenuation due to river routing as well as the difference in timing between the peak of the main stem river versus the runoff from the local tributaries and subbasins.

7.6.1 Tabular Results

The final HEC-HMS frequency flows for the locations of interest throughout the watershed model using the NOAA Atlas 14 rainfall depths can be seen below in Table 7.4.

Table 7.4: Summary of Discharges (cfs) from the HEC-HMS Elliptical Frequency Storm Method

Location Description	HEC-HMS Element Name	Drainage Area	50%	20%	10%	4%	2%	1%	0.40%	0.20%
		sq mi	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR
Neches River below Kickapoo Creek	Neches_blw_KickapooCr	570.6	10,100	22,000	30,300	41,000	51,000	62,900	75,300	92,500
Lake Palestine Inflow	Lk_Palestine_Inflow	838.1	13,500	28,100	40,600	58,500	75,300	96,100	117,400	147,800
Neches River below Lake Palestine	Neches_blw_LkPalestine	838.1	2,500	6,900	10,000	15,700	21,100	28,300	36,400	48,300
Neches River at US-175	Neches_at_US-175	882.5	2,500	7,000	10,100	15,900	21,400	28,600	36,700	48,700
Neches River above Caddo Creek	Neches_abv_CaddoCr	901.9	2,600	7,000	10,200	15,900	21,500	28,700	36,900	48,900
Neches River below Caddo Creek	Neches_blw_CaddoCr	966.7	3,500	7,600	11,500	18,100	24,400	30,900	37,400	46,900
Neches River above Brushy Creek	Neches_abv_BrushyCr	1020.4	2,600	7,000	10,300	16,100	21,900	29,400	37,900	50,400
Neches River below Brushy Creek	Neches_blw_BrushyCr	1104.4	5,300	10,400	14,800	22,900	30,200	37,300	44,600	58,500
Neches River nr Neches, TX USGS Gage 08032000 at US-79 bridge	NechesRv_nr_Neches	1145.8	4,700	10,100	15,100	25,000	34,500	43,800	53,300	67,400
Neches River above Hurricane Creek	Neches_abv_HurricaneCr	1171.2	2,700	5,900	9,400	16,900	25,100	33,900	44,300	59,700
Neches River below Hurricane Creek	Neches_blw_HurricaneCr	1275.0	4,500	9,700	13,400	20,800	30,500	41,000	53,700	72,800
Neches River above Stills Creek	Neches_abv_StillsCr	1289.5	3,200	7,100	11,200	20,200	29,700	40,200	52,600	71,300
Neches River below Stills Creek	Neches_blw_StillsCr	1345.5	3,300	7,300	11,500	20,900	30,600	41,400	54,300	73,800
Neches River above Tails Creek	Neches_abv_TailsCr	1358.7	3,100	7,100	11,200	20,000	29,100	40,600	54,000	73,400
Neches River below Tails Creek	Neches_blw_TailsCr	1465.8	3,800	8,800	13,100	22,300	31,700	44,000	58,600	81,500
Neches River above Ioni Creek	Neches_abv_IoniCr	1497.3	3,400	8,000	12,600	21,600	31,000	42,600	56,500	78,900
Neches River below Ioni Creek	Neches_blw_IoniCr	1601.6	5,000	11,300	15,800	23,200	31,900	44,000	58,700	81,900
Neches River above San Pedro Creek	Neches_abv_SanPedroCr	1637.6	3,400	7,900	12,500	21,600	31,100	42,700	57,300	80,200
Neches River below San Pedro Creek	Neches_blw_SanPedroCr	1772.6	8,400	16,300	21,800	31,200	39,400	48,200	57,400	70,800
Neches River at TX-21 Bridge, former USGS gage near Alto 08032500	Neches_at_TX-21	1943.4	6,600	14,700	22,000	36,100	50,800	66,300	82,100	107,300
Neches River above Hickory Creek	Neches_abv_HickoryCr	2008.3	5,600	12,500	18,600	30,600	42,200	56,000	73,100	99,700
Neches River below Hickory Creek	Neches_blw_HickoryCr	2098.8	6,000	13,300	19,600	32,200	44,200	58,600	77,300	105,700
Neches River at TX-7 bridge near Pollok, TX	Neches_at_TX-7	2236.5	8,000	15,000	19,200	32,800	43,600	59,600	77,600	105,400

Location Description	HEC-HMS Element Name	Drainage Area	50%	20%	10%	4%	2%	1%	0.40%	0.20%
		sq mi	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR
Neches River at TX-94 bridge near Apple Springs, TX	Neches_at_TX-94	2433.3	12,000	22,100	28,800	39,300	45,700	55,100	73,400	103,900
Neches River nr Diboll, USGS gage 08033000 at US-59 bridge	NechesRv_nr_Diboll	2726.2	10,200	20,500	27,900	40,100	45,400	57,600	76,700	107,000
Neches River above Piney Creek	Neches_abv_PineyCr	2941.0	11,600	15,700	18,700	27,900	37,800	52,300	69,000	96,500
Neches River below Piney Creek	Neches_blw_PineyCr	3315.4	15,700	22,400	27,300	35,000	41,800	58,400	77,200	106,900
Neches River near Rockland, USGS gage 08033500 at US-69 bridge	NechesRv_nr_Rockland	3633.1	13,800	25,100	33,500	45,200	50,200	58,600	76,800	106,400
Neches River above the Angelina River	Neches_abv_Angelina	3791.1	10,400	14,700	19,900	31,400	40,300	56,300	73,500	102,300
Angelina River below Striker Creek	Angelina_blw_StrikerCr	426.3	7,000	11,200	14,800	19,700	25,400	33,400	41,000	52,200
Angelina River above Mud Creek	Angelina_abv_MudCr	638.6	7,000	11,400	15,300	20,100	26,000	35,000	44,500	58,100
Mud Creek near Jacksonville, USGS gage 08034500 at US-79 bridge	MudCr_nr_Jacksonville	377.4	3,200	7,100	10,700	17,400	24,400	30,100	37,700	51,700
Mud Creek at US-84 bridge, near Reklaw, TX	MudCr_at_US-84	523.3	5,900	8,900	11,400	14,400	21,700	30,600	41,500	57,800
Mud Creek above the Angelina River	MudCr_abv_Angelina	556.3	3,200	5,300	8,100	13,900	21,200	30,000	40,700	57,000
Angelina River below Mud Creek	Angelina_blw_MudCr	1194.8	8,200	14,200	20,300	30,200	42,600	59,100	78,000	105,500
Angelina River near Alto, USGS gage 08036500 at TX-21 bridge	AngelinaRv_nr_Alto	1286.4	6,500	12,500	17,900	26,300	37,400	52,800	70,400	96,900
Angelina River above Bayou Loco	Angelina_abv_BayouLoco	1415.8	6,200	12,000	17,100	25,100	35,700	50,300	67,100	94,600
Angelina River above Bayou Loco	Angelina_blw_BayouLoco	1518.3	6,500	12,700	18,000	26,400	37,400	53,000	70,000	97,900
Angelina River at Hwy 59 near Lufkin USGS gage, above Bayou La Nana	Angelina_abv_BayouLaNana	1621.5	6,500	12,600	17,900	26,000	36,800	52,000	68,600	96,300
Angelina River below Bayou La Nana	Angelina_blw_BayouLaNana	1704.9	6,500	12,600	18,000	26,100	36,900	52,300	69,000	97,100
Angelina River above Bayou Carrizo	Angelina_abv_BayouCarrizo	1842.3	6,300	12,100	17,300	25,100	35,000	49,700	66,000	92,400
Angelina River below Bayou Carrizo	Angelina_blw_BayouCarrizo	1952.4	12,600	22,200	28,900	38,900	47,500	58,700	69,300	84,600
Attoyac Bayou below Big Iron Ore Creek	Attoyac_blw_BigIronOreCr	485.2	6,800	14,300	20,000	28,700	33,600	47,900	61,000	80,000
Attoyac Bayou nr Chireno, USGS gage 08038000 at TX-21 bridge	Attoyac_Bayou_nr_Chireno	503.1	6,300	13,700	19,100	27,300	31,900	45,400	58,100	77,300

Location Description	HEC-HMS Element Name	Drainage Area	50%	20%	10%	4%	2%	1%	0.40%	0.20%
		sq mi	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR
Attoyac Bayou above Angelina River	Attoyac_abv_Angelina	670.7	5,800	12,800	18,200	26,400	31,200	45,500	58,700	78,400
Angelina River below Attoyac Bayou	Angelina_blw_Attoyac	2808.0	26,600	47,600	62,500	84,700	104,300	129,900	154,000	188,900
Total Inflow to Sam Rayburn Dam	SamRayburn_Inflow	3451.8	51,100	88,500	116,400	159,600	197,700	248,200	297,200	365,300
Angelina River below Sam Rayburn	Angelina_blw_SamRayburn	3451.8	8,400	9,800	11,700	14,000	14,000	15,400	17,000	17,000
Angelina River above the Neches River	Angelina_abv_Neches	3566.9	9,000	11,100	13,600	16,600	21,700	26,400	30,800	37,300
Total Inflow to Town Bluff Dam	TownBluff_Inflow	7569.3	25,400	40,300	50,200	61,400	81,300	100,400	117,800	143,000
Neches River below Town Bluff Dam, USGS gage 08040600	NechesRv_nr_TownBluff	7569.3	20,000	31,100	40,300	51,600	67,200	80,700	90,700	110,000
Neches River below Big Creek	Neches_blw_BigCr	7673.6	22,300	34,800	43,600	53,800	67,800	82,800	96,000	117,700
Neches River below Mill Creek at FM 1013 bridge	Neches_at_FM1013	7716.9	22,100	32,400	40,000	50,000	64,900	81,200	95,800	117,800
Neches River below Black Branch	Neches_blw_BlackBranch	7784.9	21,400	30,800	38,200	47,900	61,500	79,300	95,200	118,600
Neches River at Evadale	Neches_at_Evadale	7894.7	21,000	29,700	36,600	46,000	57,700	74,700	90,900	115,000
Neches River below Evadale	Neches_blw_Evadale	7950.3	21,100	30,100	37,100	46,800	58,700	76,200	92,900	117,800
Neches River above Village Creek	Neches_abv_VillageCr	8001.7	20,700	29,600	36,300	45,600	56,800	73,600	90,200	115,200
Village Creek above Turkey Creek	VillageCr_abv_TurkeyCr	423.1	6,400	14,400	22,400	33,500	40,800	52,100	66,700	86,800
Village Creek below Turkey Creek	VillageCr_blw_TurkeyCr	589.4	8,400	19,400	29,800	42,900	52,500	65,900	84,000	110,300
Village Creek above Beech Creek	VillageCr_abv_BeechCr	601.7	7,600	17,000	26,800	38,100	50,500	65,200	83,700	109,800
Village Creek below Beech Creek	VillageCr_blw_BeechCr	814.7	9,500	21,100	33,800	47,900	64,400	84,000	108,600	143,300
Village Creek near Kountze, USGS gage 08041500 at FM 418 bridge	VillageCr_nr_Kountze	861.1	8,900	20,600	32,400	45,200	59,200	80,400	107,300	142,500
Village Creek above Cypress Creek	VillageCr_abv_CypressCr	864.4	8,300	20,100	32,700	45,800	58,800	77,500	103,800	141,800
Village Creek below Cypress Creek	VillageCr_blw_CypressCr	1064.0	8,300	22,900	36,500	51,700	66,000	88,900	119,400	163,800
Village Creek at US-96 bridge near Lumberton, TX	VillageCr_at_US-96	1104.4	6,900	20,200	31,800	48,100	58,800	79,000	100,600	140,100
Village Creek above Neches River	VillageCr_abv_Neches	1113.9	6,700	20,100	31,600	47,800	58,600	77,900	99,300	138,000
Neches River below Village Creek	Neches_blw_VillageCr	9115.6	25,100	42,900	59,100	78,400	97,400	128,200	163,900	213,200
Neches River above Pine Island Bayou	Neches_abv_PineIsBayou	9132.5	24,800	41,800	56,800	73,600	90,300	118,400	153,800	206,400

Location Description	HEC-HMS Element Name	Drainage Area	50%	20%	10%	4%	2%	1%	0.40%	0.20%
		sq mi	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	250-YR	500-YR
Pine Island Bayou near Sour Lake, USGS gage 08041700 at Old Beaumont Rd bridge	PineIsBayou_nr_SourLake	397.7	4,200	9,200	14,400	21,600	28,300	39,200	50,700	67,800
Pine Island Bayou above Little Pine Island Bayou	PineIsBayou_abv_LittlPIBayou	417.3	3,800	8,100	13,100	20,200	27,000	38,300	49,900	67,000
Pine Island Bayou below Little Pine Island Bayou	PineIsBayou_blw_LittlPIBayou	552.2	5,200	10,400	16,400	25,000	33,200	46,800	60,800	81,400
Pine Island Bayou above BI Pump Plant	PineIsBayou_abv_BIPumpPlant	697.7	6,100	11,400	17,600	27,600	36,600	52,900	69,200	94,700
Pine Island Bayou above the Neches River	PineIsBayou_abv_Neches	726.2	6,300	11,700	17,900	28,300	37,600	54,500	71,400	97,700
Neches River below Pine Island Bayou	Neches_blw_PineIsBayou	9858.6	29,200	50,600	68,900	91,600	112,800	150,400	197,400	269,700
Neches River at the Salt Water Barrier	Neches_at_SaltwaterBarrier	9858.7	29,200	50,500	68,900	91,500	112,900	150,400	197,600	270,000

7.6.2 Map Results

The following 'a' figures represent the 100yr 96hr heatmap results for the optimization of a handful of example junctions of interest in the Elliptical Frequency Storm HEC-HMS model. For each junction of interest, the optimization script ran 300+ times recording the junction flow rate for various storm centerings and orientations. Each of the recorded storm centerings (x,y) and resulting flow rates (z) at the junction of interest were recorded and used to create a rasterized heat map. The red shading represents storm center locations that led to relatively high flow rates at the junction whereas the green shading represents storm center locations that led to relatively low flow rates.

The following 'b' figures show the final, total storm depths and optimized storm configurations for each example junction. Note that the peak flow values recorded in the 'a' figures may differ slightly from the final peak flow values recorded in the 'b' figures and in Table 7.4 above. These differences are due to some small adjustments to the elliptical storm and HEC-HMS model parameters that occurred during the review process. The 'b' figures include the final peak flow values after peer review.

This section includes the figures for only a small sample of example junctions from the Neches River basin. The elliptical storm maps for all of the junctions that were analyzed can be found in Appendix C.

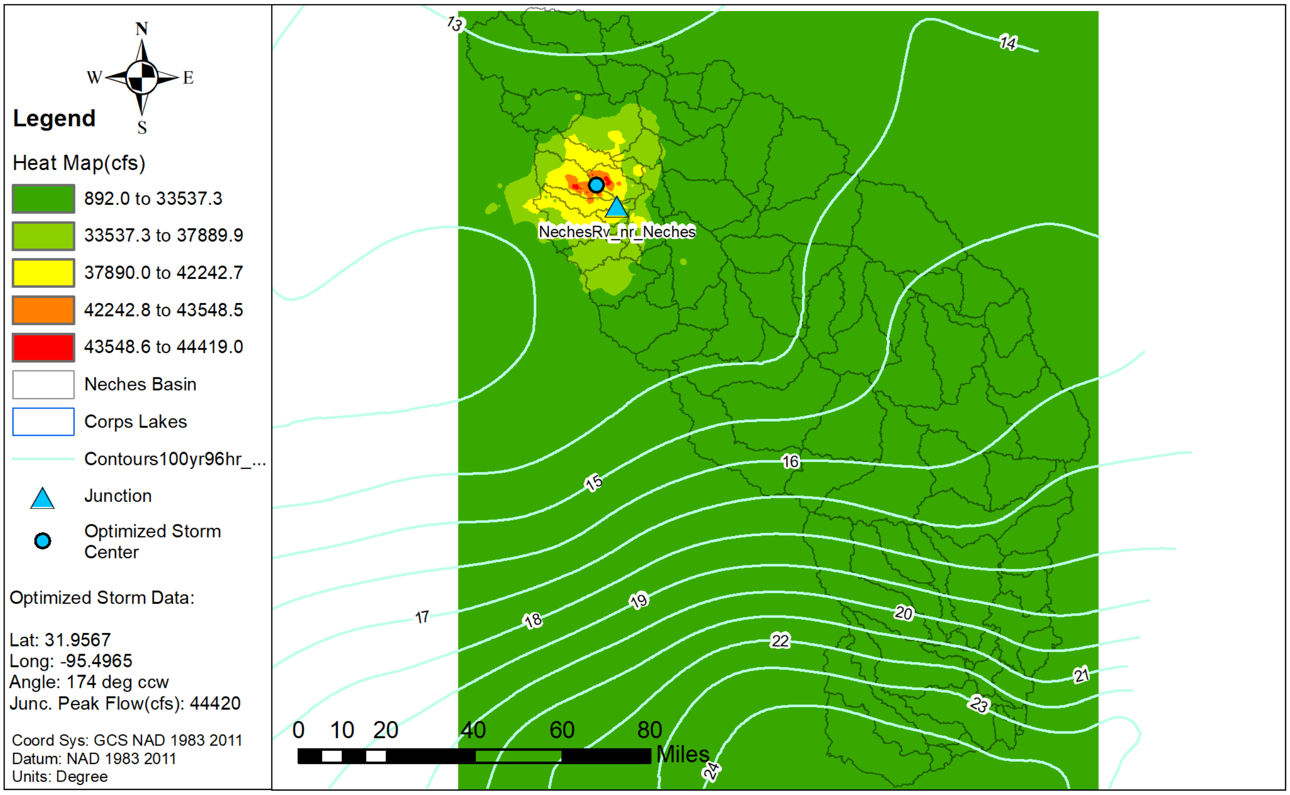


Figure 7.10a: Elliptical Storm Optimization Heat Map for NechesRv_nr_Neches

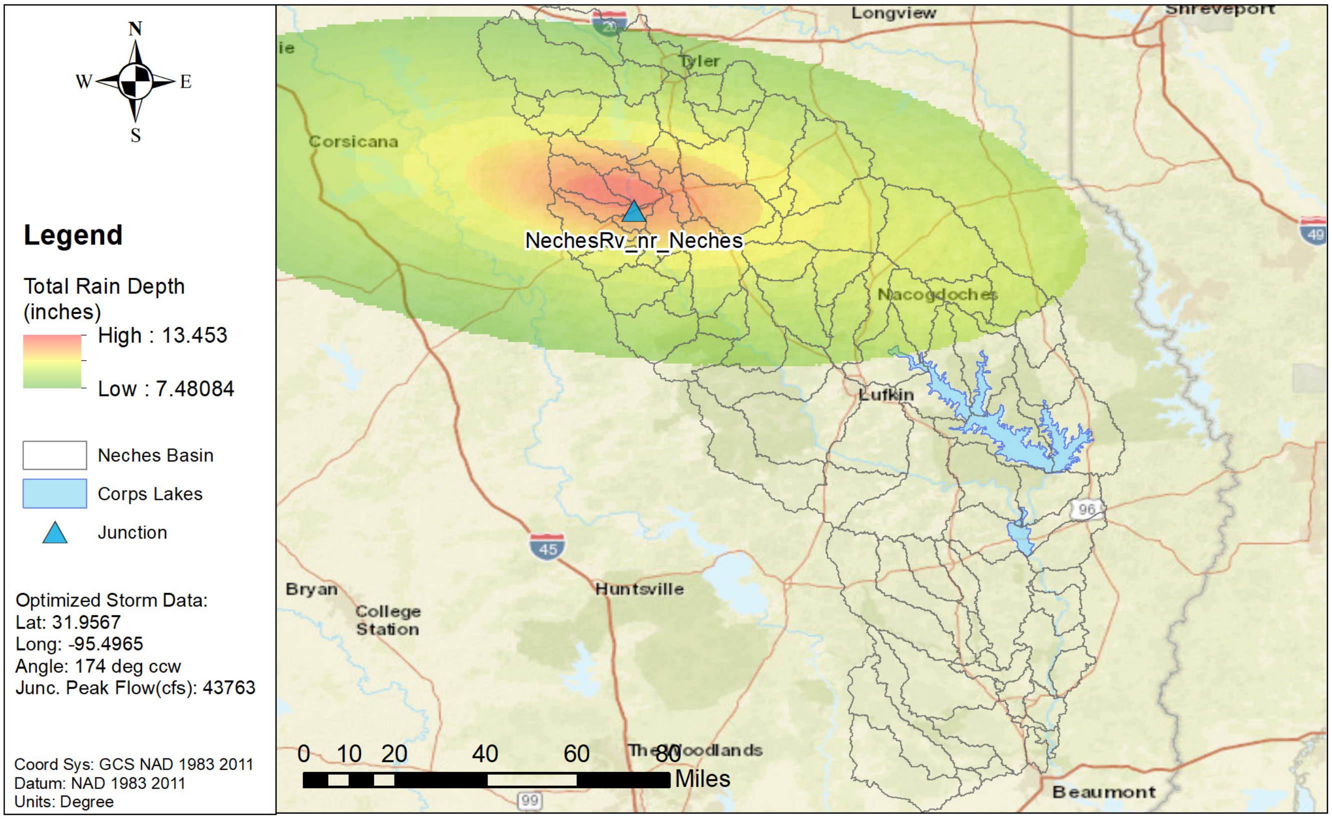


Figure 7.10b: NA14 1% AEP Elliptical Storm for NechesRv_nr_Neches

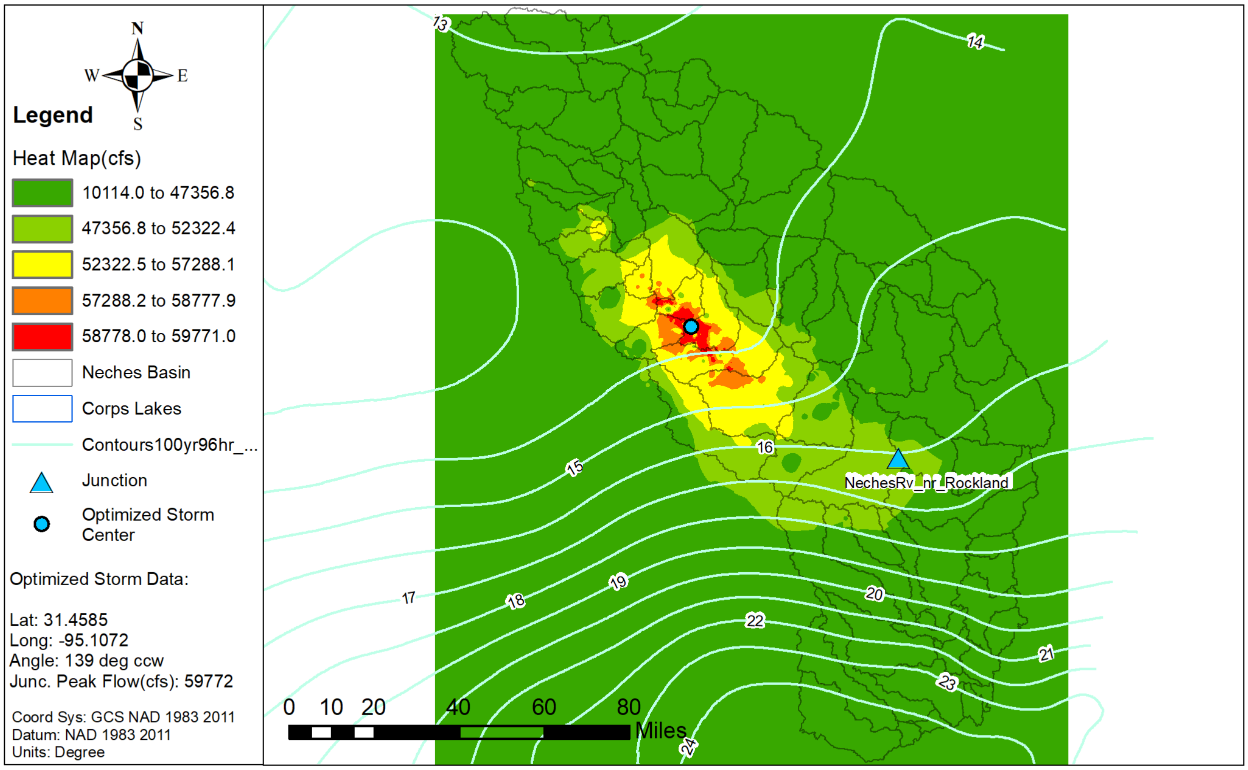


Figure 7.11a: Elliptical Storm Optimization Heat Map for NechesRv_nr_Rockland

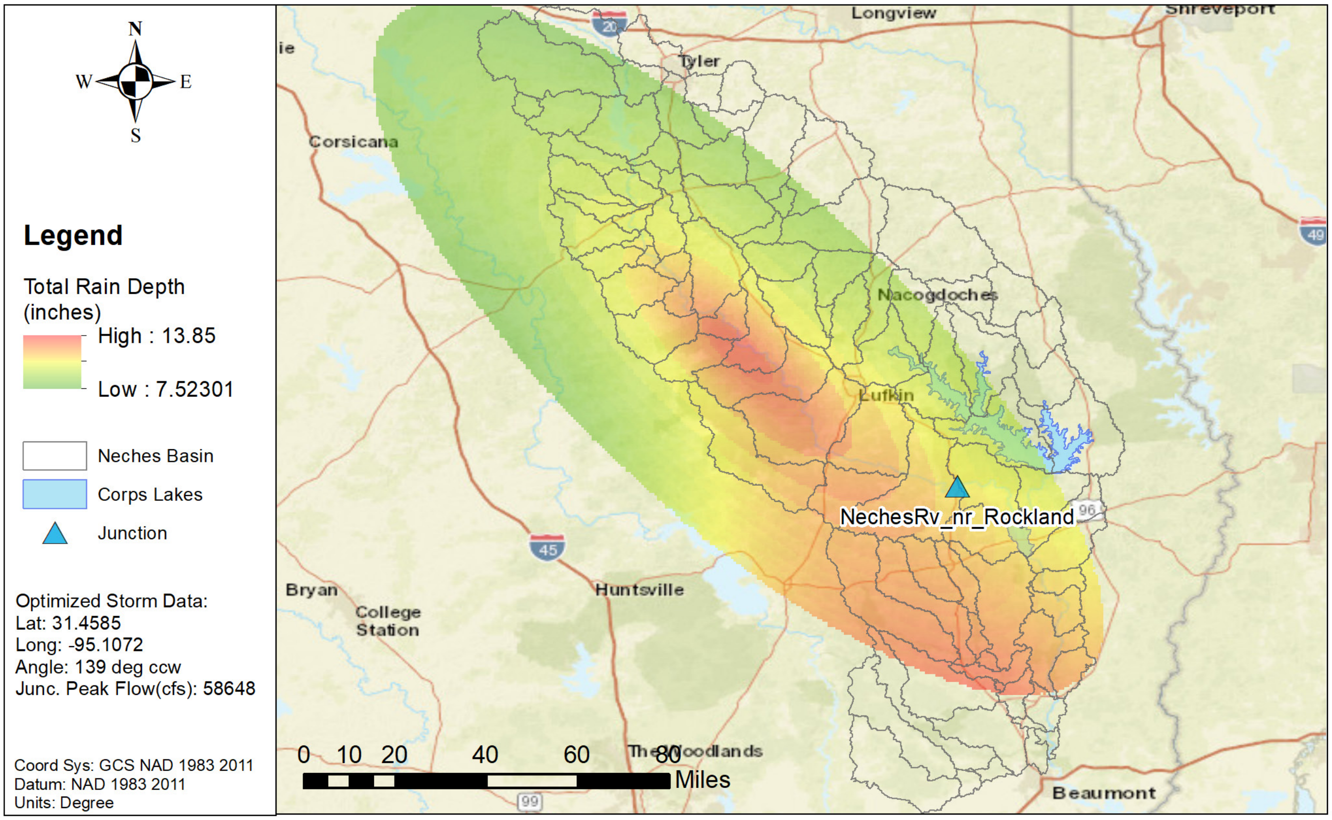


Figure 7.11b: NA14 1% AEP Elliptical Storm for NechesRv_nr_Rockland

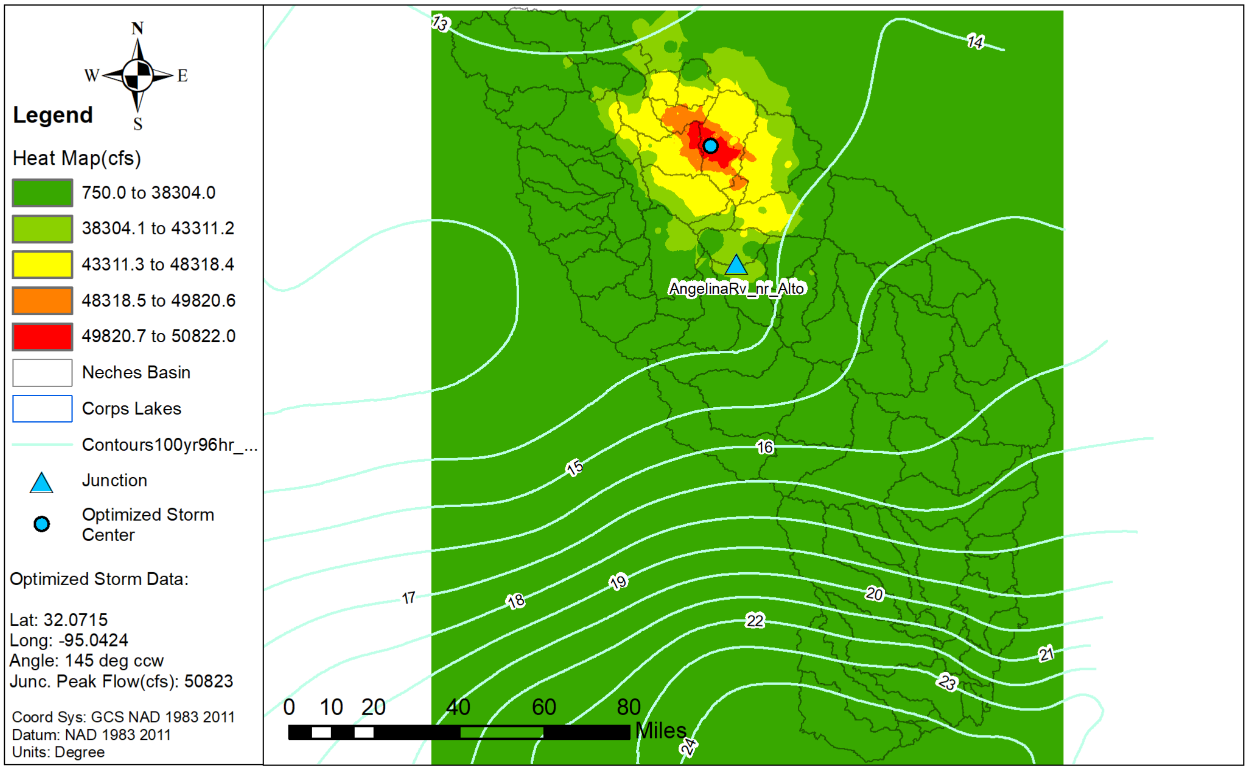


Figure 7.12a: Elliptical Storm Optimization Heat Map for AngelinaRv_nr_Alto

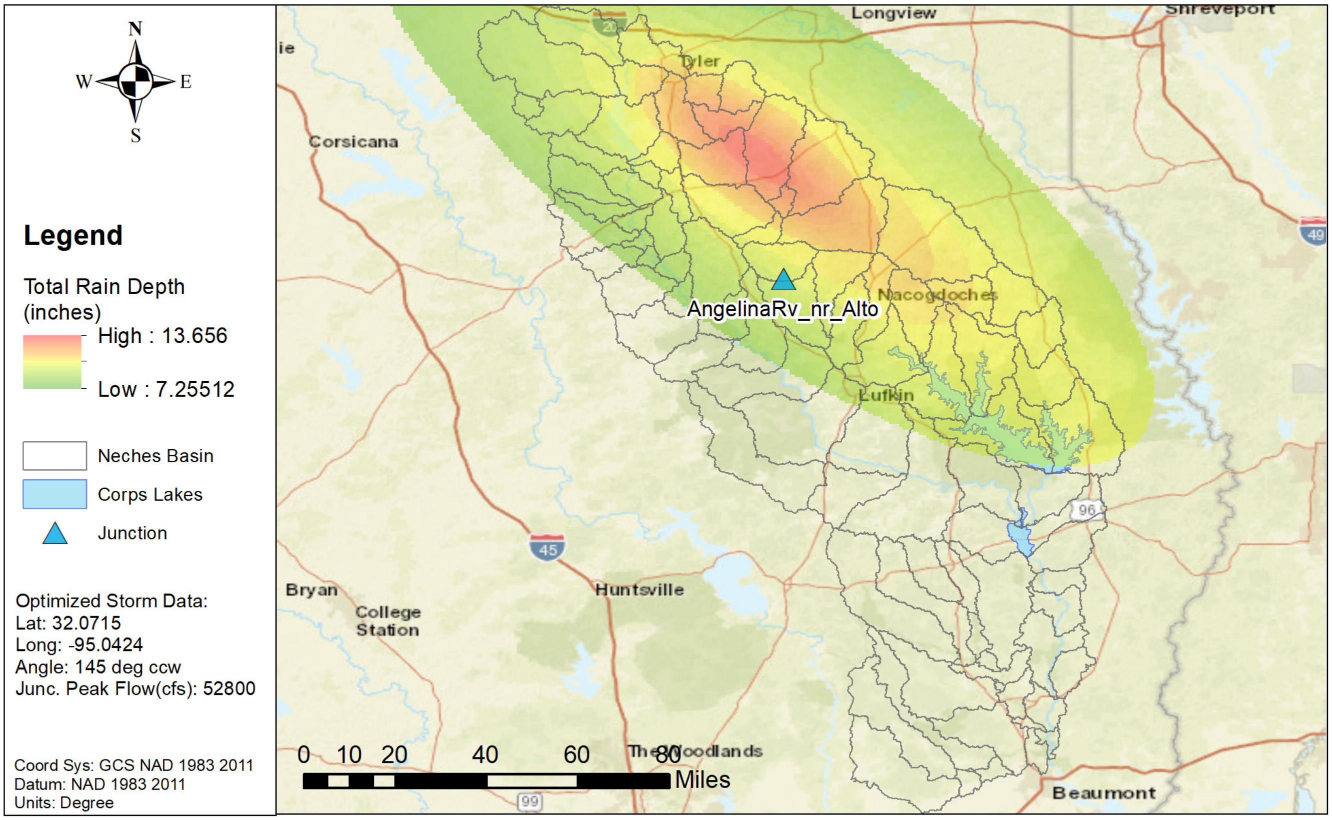


Figure 7.12b: NA14 1% AEP Elliptical Storm for AngelinaRv_nr_Alto

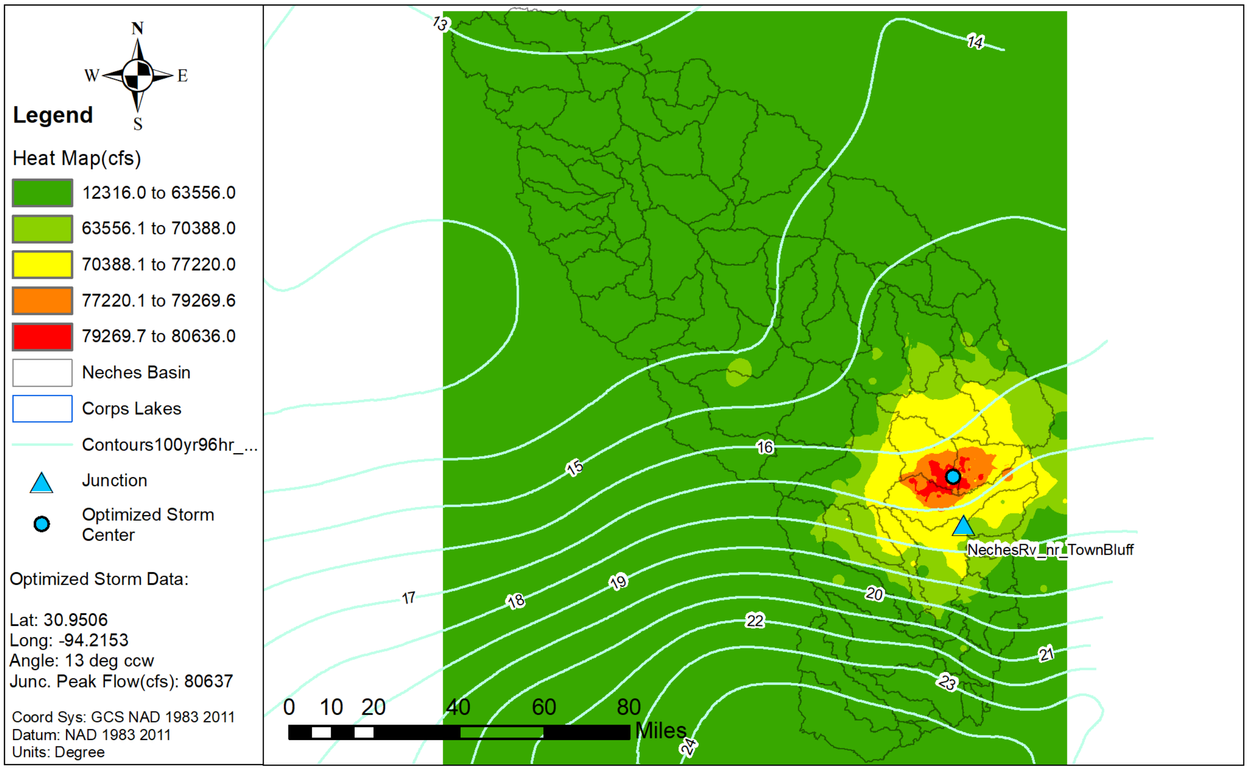


Figure 7.13a: Elliptical Storm Optimization Heat Map for NechesRv_nr_TownBluff

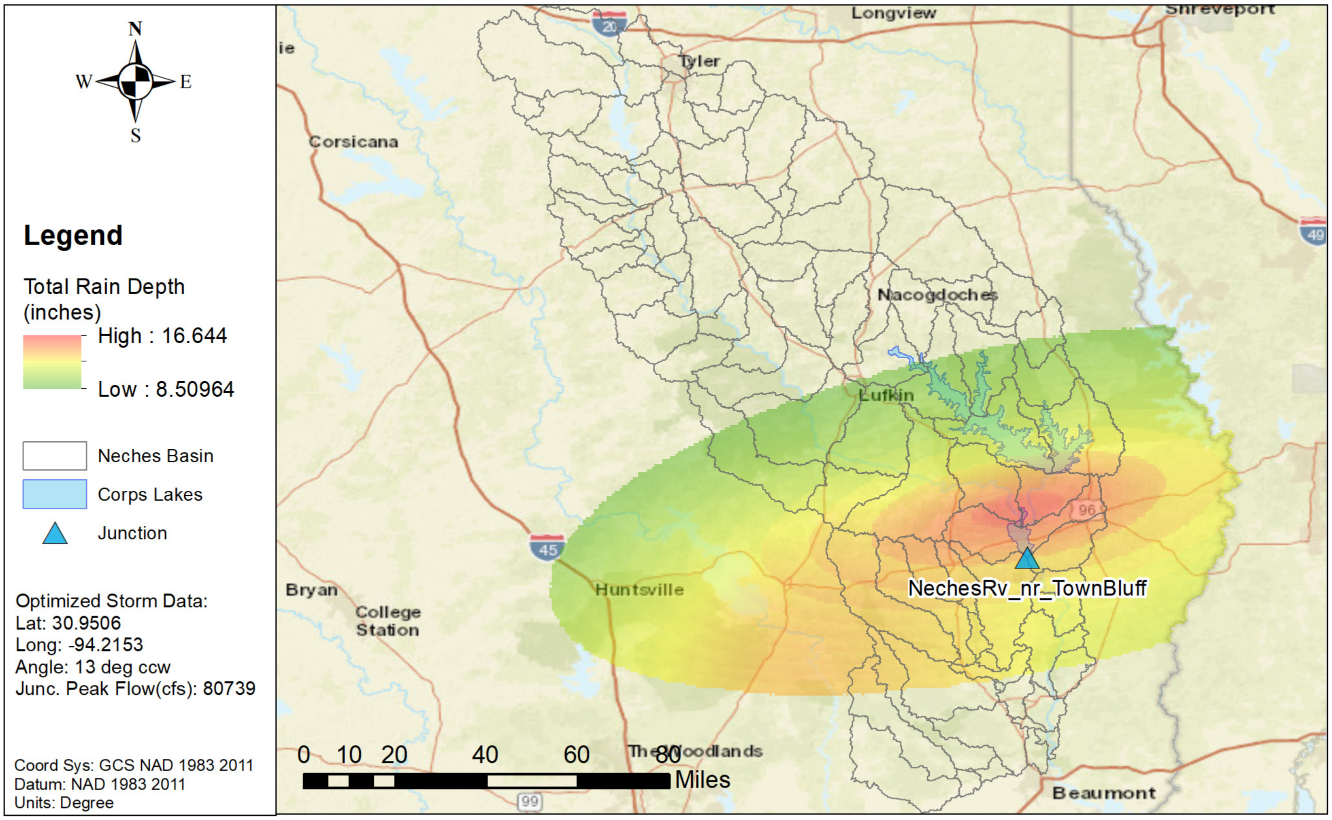


Figure 7.13b: NA14 1% AEP Elliptical Storm for NechesRv_nr_TownBluff

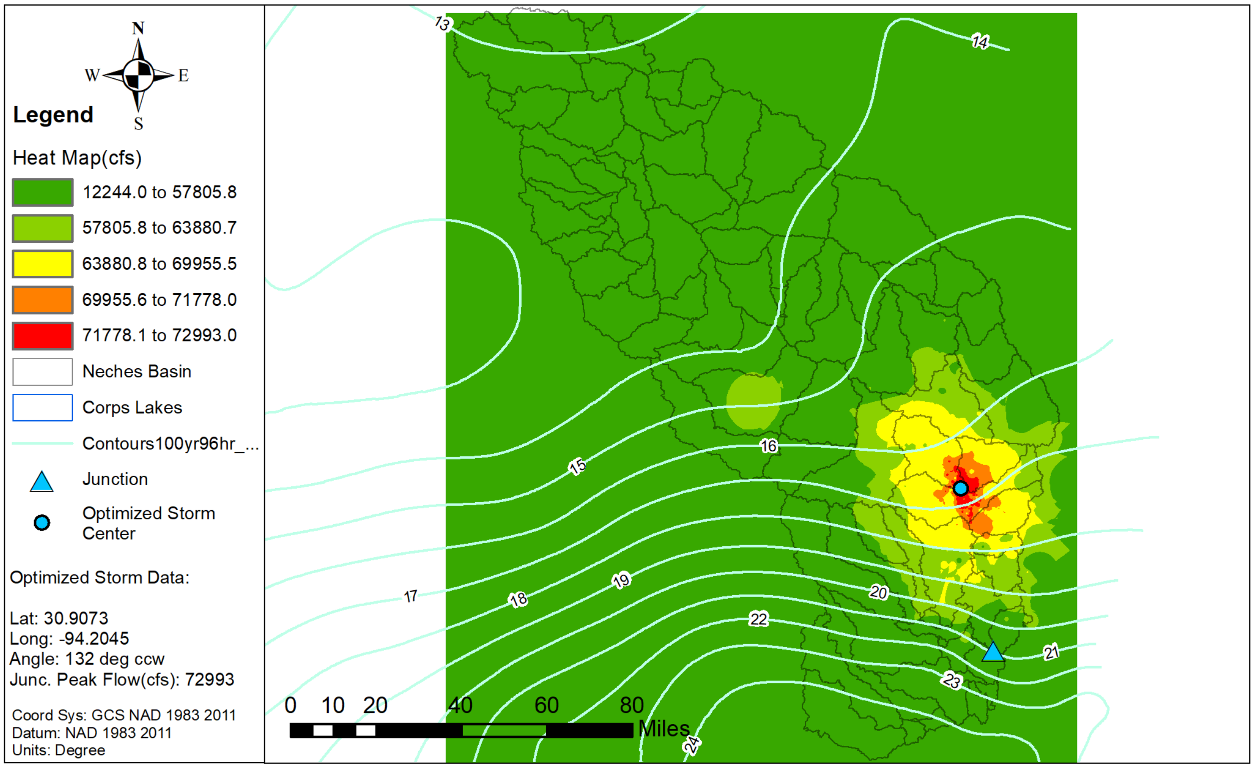


Figure 7.14a: Elliptical Storm Optimization Heat Map for Neches_at_Evadale

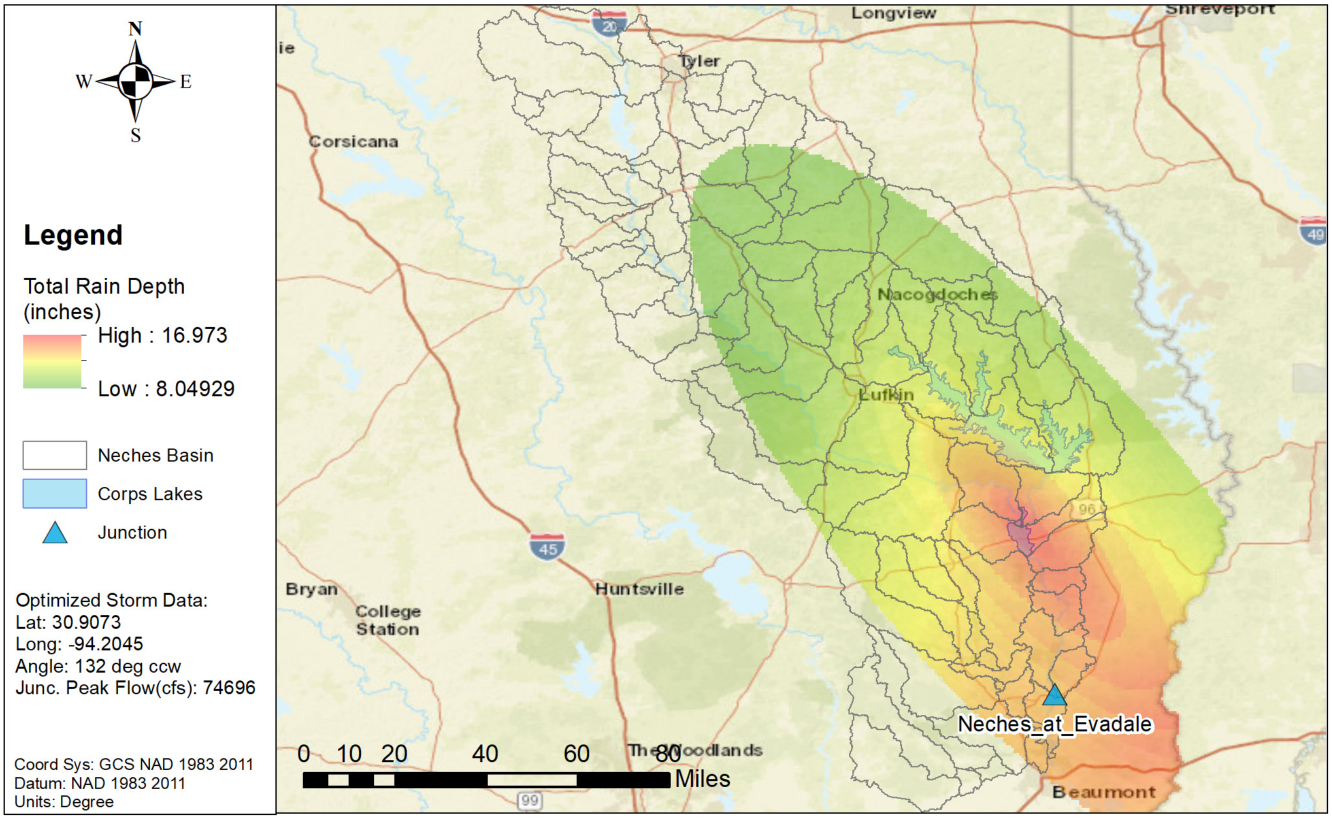


Figure 7.14b: NA14 1% AEP Elliptical Storm for Neches_at_Evadale

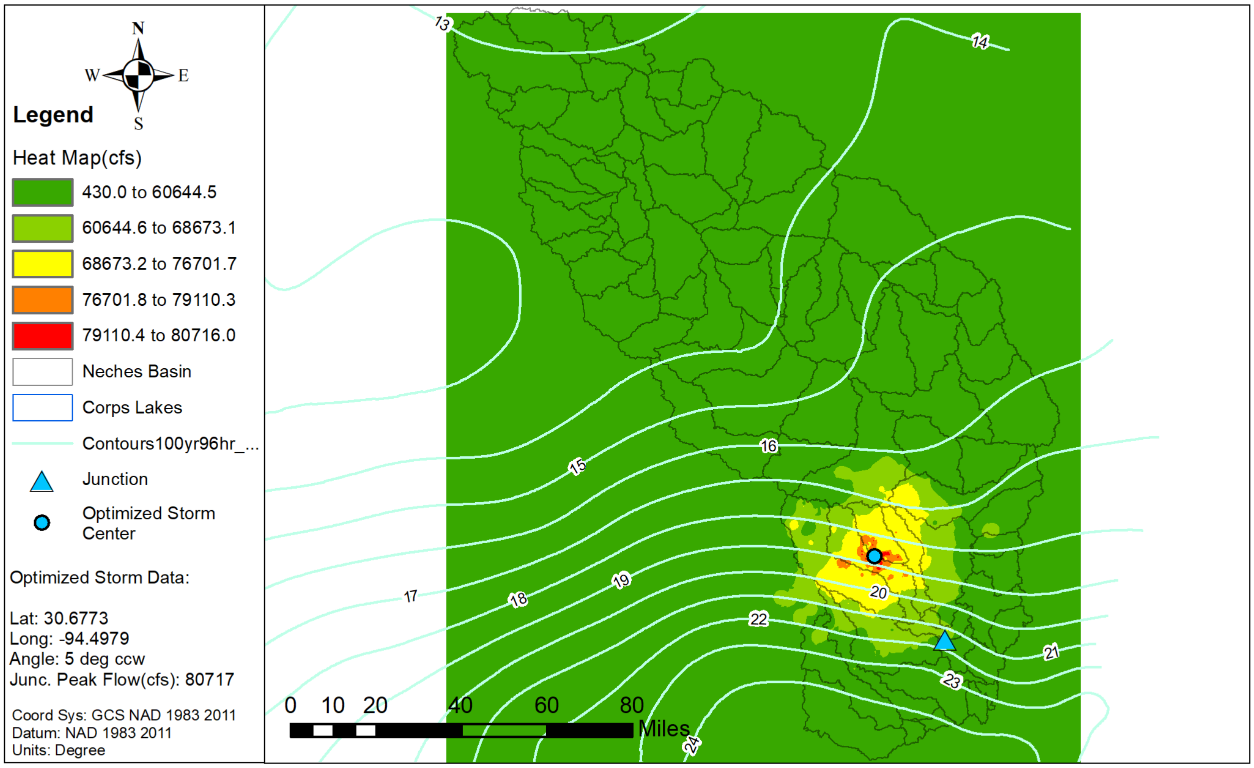


Figure 7.15a: Elliptical Storm Optimization Heat Map for VillageCr_nr_Kountze

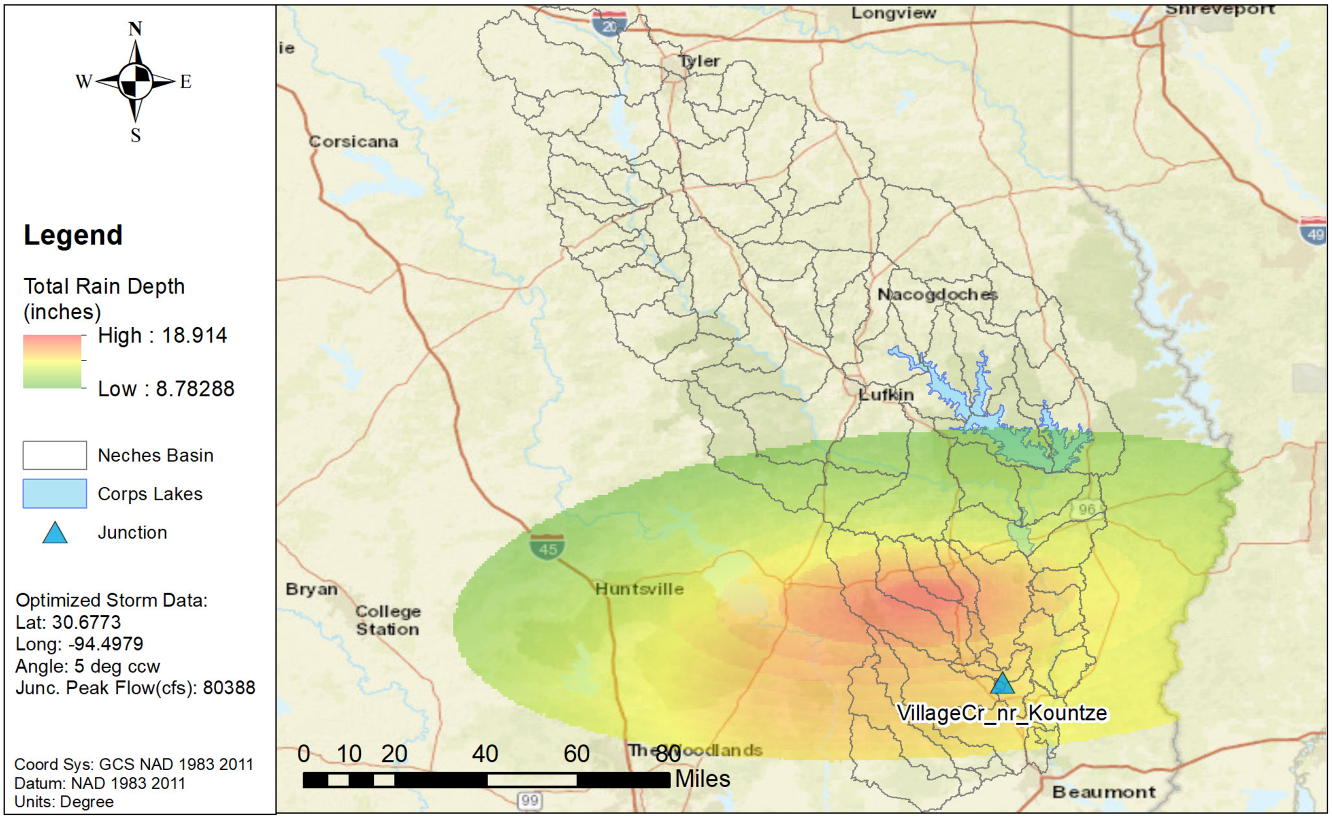


Figure 7.15b: NA14 1% AEP Elliptical Storm for VillageCr_nr_Kountze

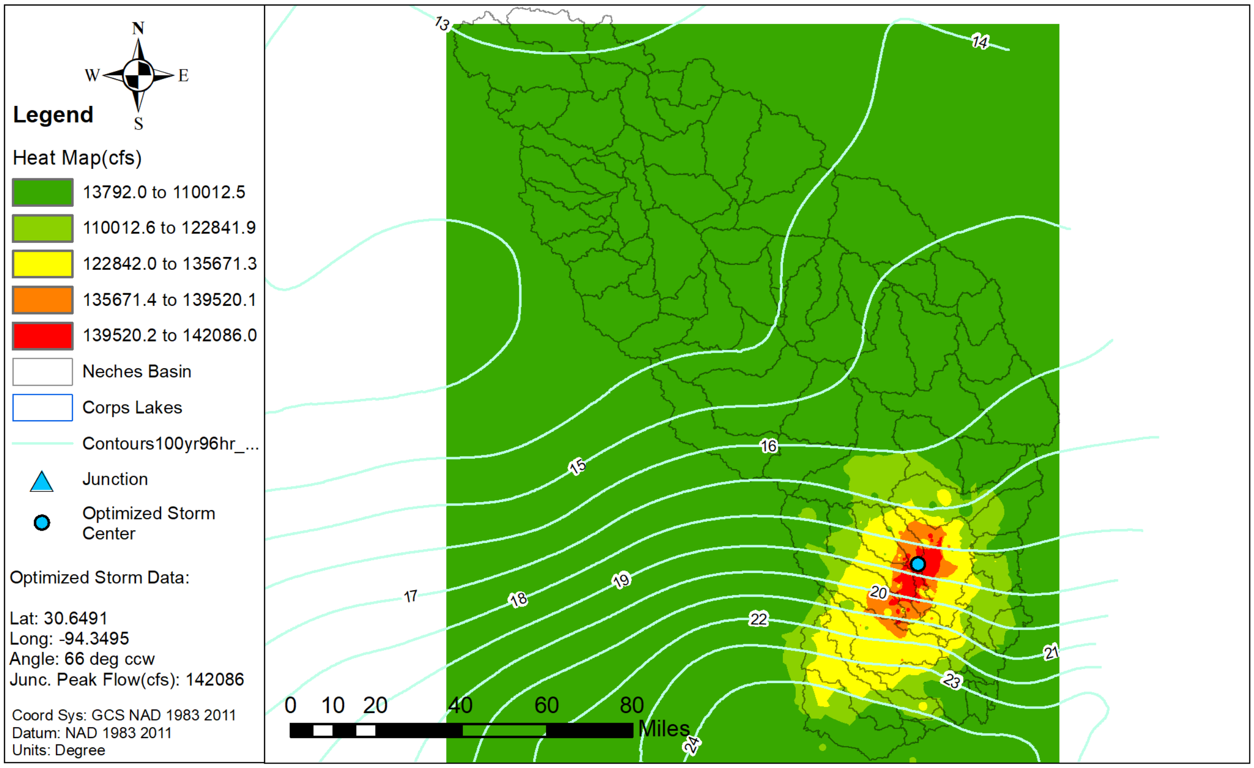


Figure 7.16a: Elliptical Storm Optimization Heat Map for Neches_at_SaltwaterBarrier

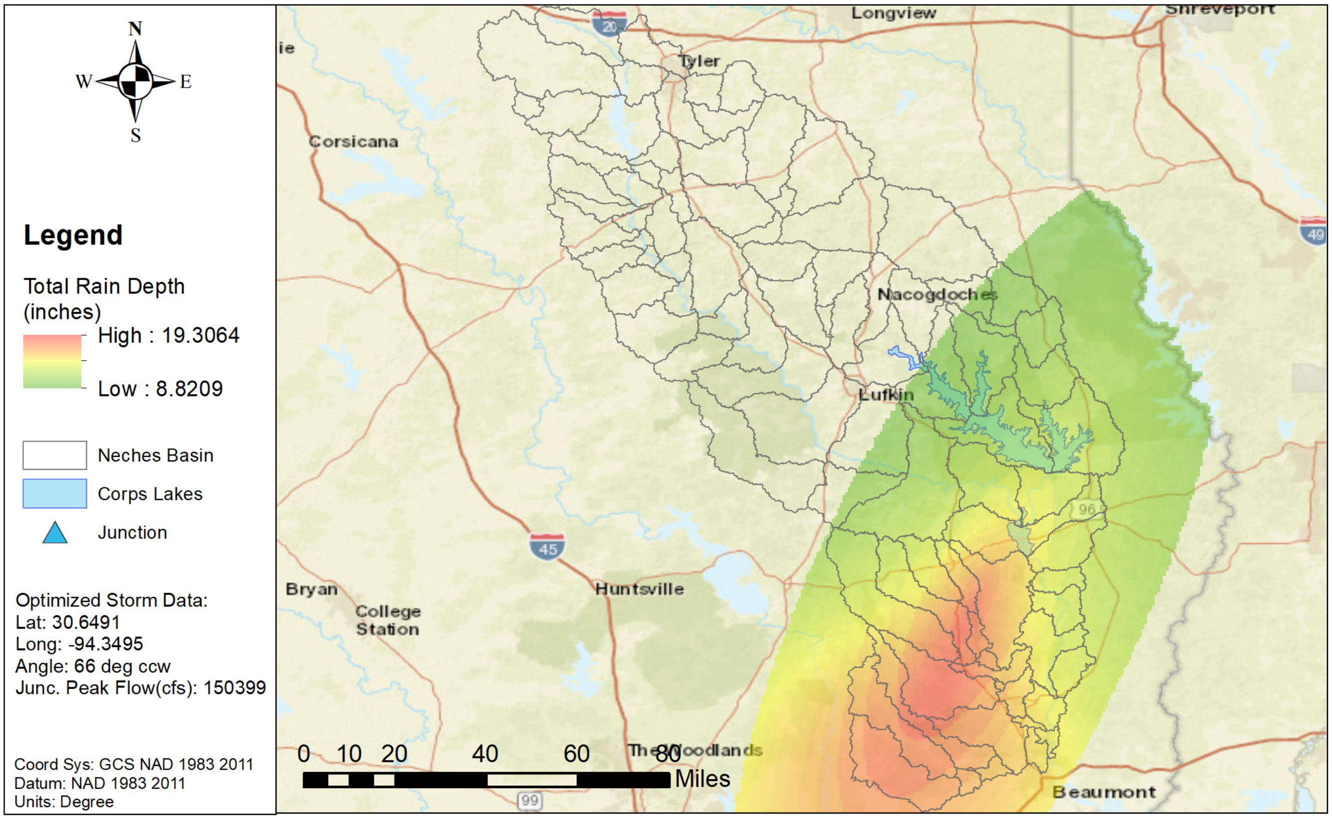


Figure 7.16b: NA14 1% AEP Elliptical Storm for Neches_at_SaltwaterBarrier

7.7 ELLIPTICAL FREQUENCY STORM RESULTS – PEAK POOL ELEVATION

In addition to analyzing the gage and junction locations within the Neches Basin, analysis was also performed on the eight reservoirs modeled in HEC-HMS. The same elliptical frequency storm methodology was applied as previously discussed, except that storm centers and angles were optimized based on peak pool elevation at a reservoir instead of peak flow at a junction. The shift to pool elevation tended to shift the position of the storm to maximize storm volume above the reservoir rather than peak inflow.

7.7.1 Tabular Results

The final HEC-HMS frequency pool elevations for the reservoirs of interest throughout the watershed can be seen below in Table 7.5.

Table 7.5: Summary of Peak Reservoir Pool Elevations (ft NAVD88) from the Elliptical Frequency Storms

HEC-HMS Element Name	Drainage Area	50%	20%	10%	4%	2%	1%	0.50%	0.20%
	(sqmi)	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	200-YR	500-YR
Lake_Athens	21.6	441.05	441.66	442.18	442.86	443.57	444.55	445.59	446.82
Lake_Palestine	838.1	346.2	347.42	348.08	349.09	349.93	350.9	351.89	353.21
Lake_Jacksonville	39.6	423.31	424.97	426.25	428.03	429.67	431.53	432.9	434.43
Lake_Striker	182.0	293.47	293.74	293.93	294.2	294.51	294.97	295.78	297.06
Lake_Tyler	113.3	376.42	377.27	377.84	378.58	379.26	380.08	380.88	381.97
Lake_Nacogdoches	89.0	281.49	282.76	283.97	285.88	286.87	288.49	289.96	291.35
Sam_Rayburn_Reservoir	3451.8	165.15	166.15	166.99	168.22	169.44	171.43	173.25	175.85
TownBluff_Dam (B.A. Steinhagen Lake)	7569.3	82.99	83.3	83.57	83.81	84.34	85.03	85.79	86.87

7.7.2 Map Results

The ‘a’ numbered figures in the following section represent the 100yr 96hr heatmap results for the optimization of each reservoir of interest in the Elliptical Frequency Storm HEC-HMS model. For each reservoir, the optimization script ran 300+ times recording the peak reservoir pool elevation for various storm centerings and orientations. Each of the recorded storm centerings (x,y) and resulting pool elevations (z) at the reservoir were recorded and used to create a rasterized heat map. The red shading represents storm center locations that led to relatively high pool elevations at the reservoir whereas the green shading represents storm center locations that led to relatively low pool elevations. The ‘b’ numbered figures in the following section show the final, total storm depths and optimized storm configurations for each reservoir.

This section includes the figures for only a small sample of example reservoirs from the Neches River basin. The elliptical storm maps for all of the reservoirs that were analyzed can be found in Appendix C.

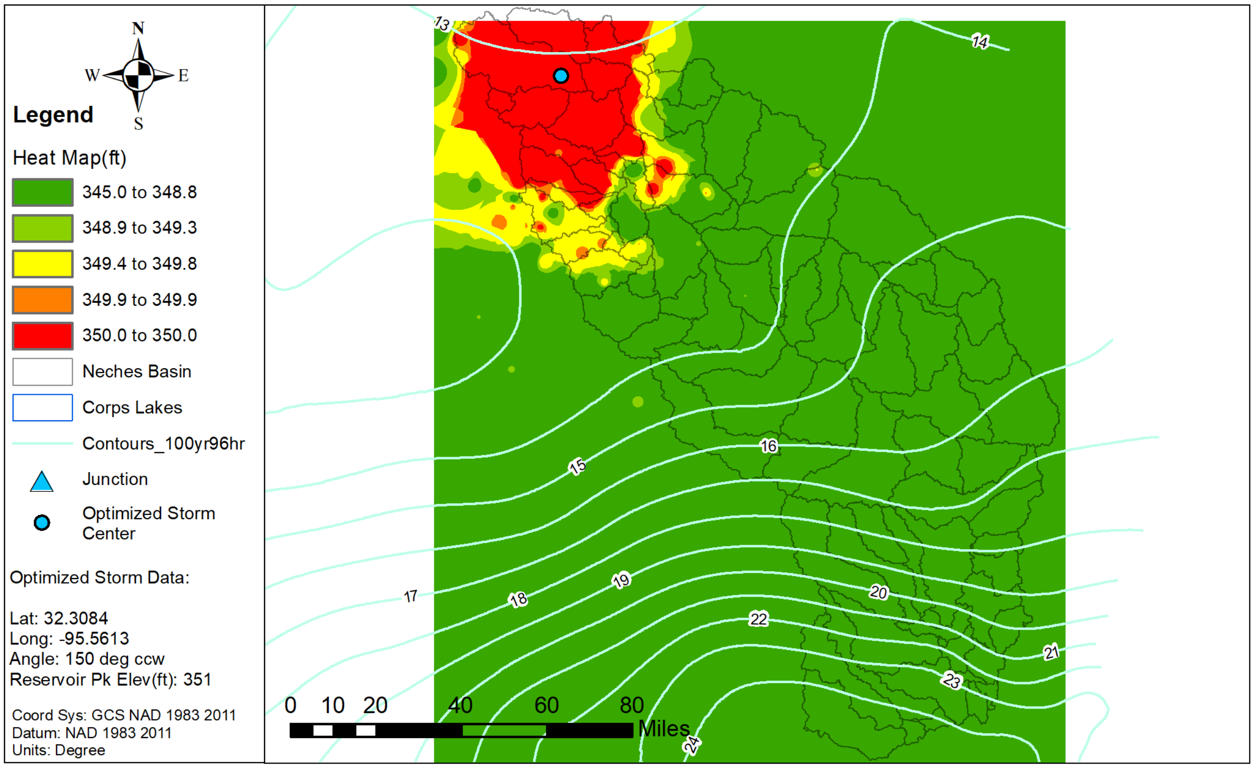


Figure 7.17a: Elliptical Storm Optimization Heat Map for Lake Palestine

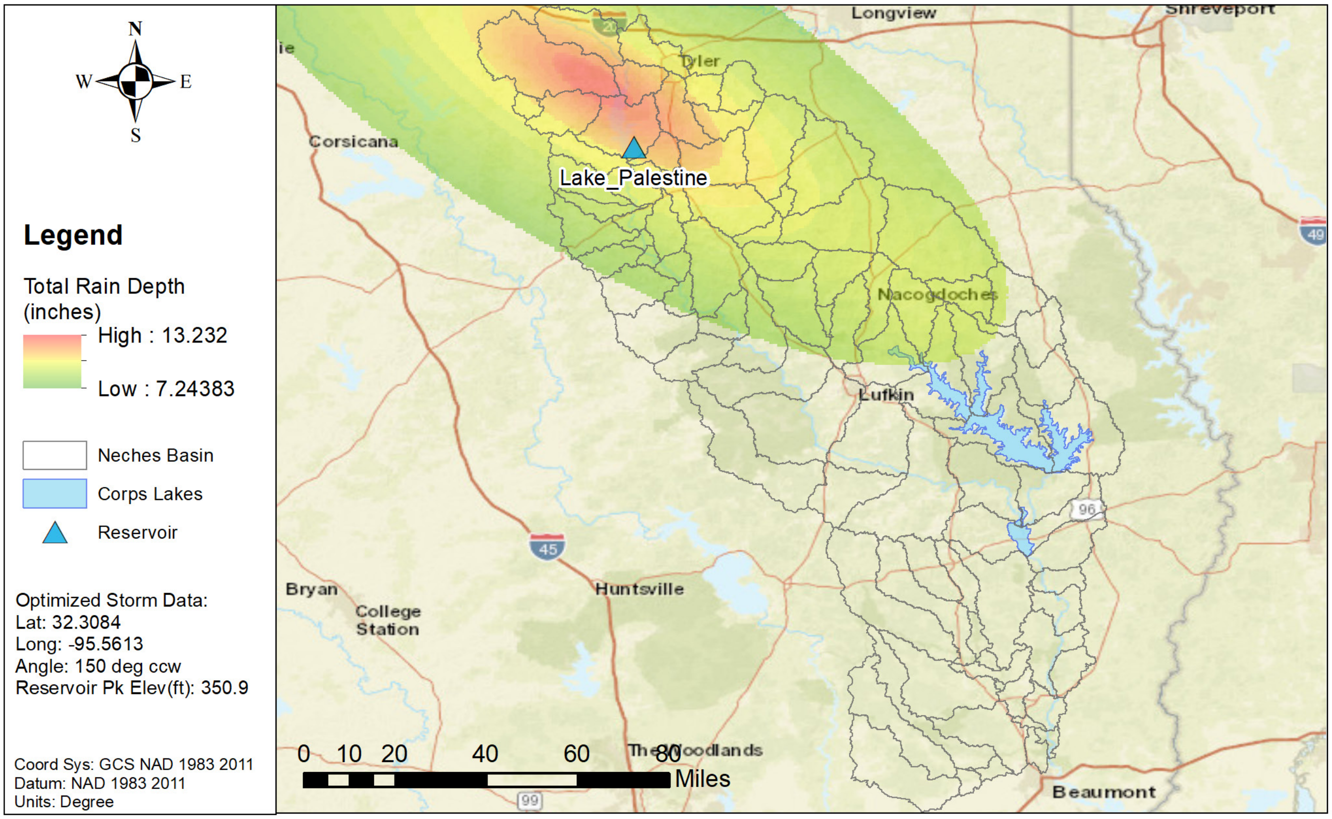


Figure 7.17b: NA14 1% AEP Elliptical Storm for Lake Palestine

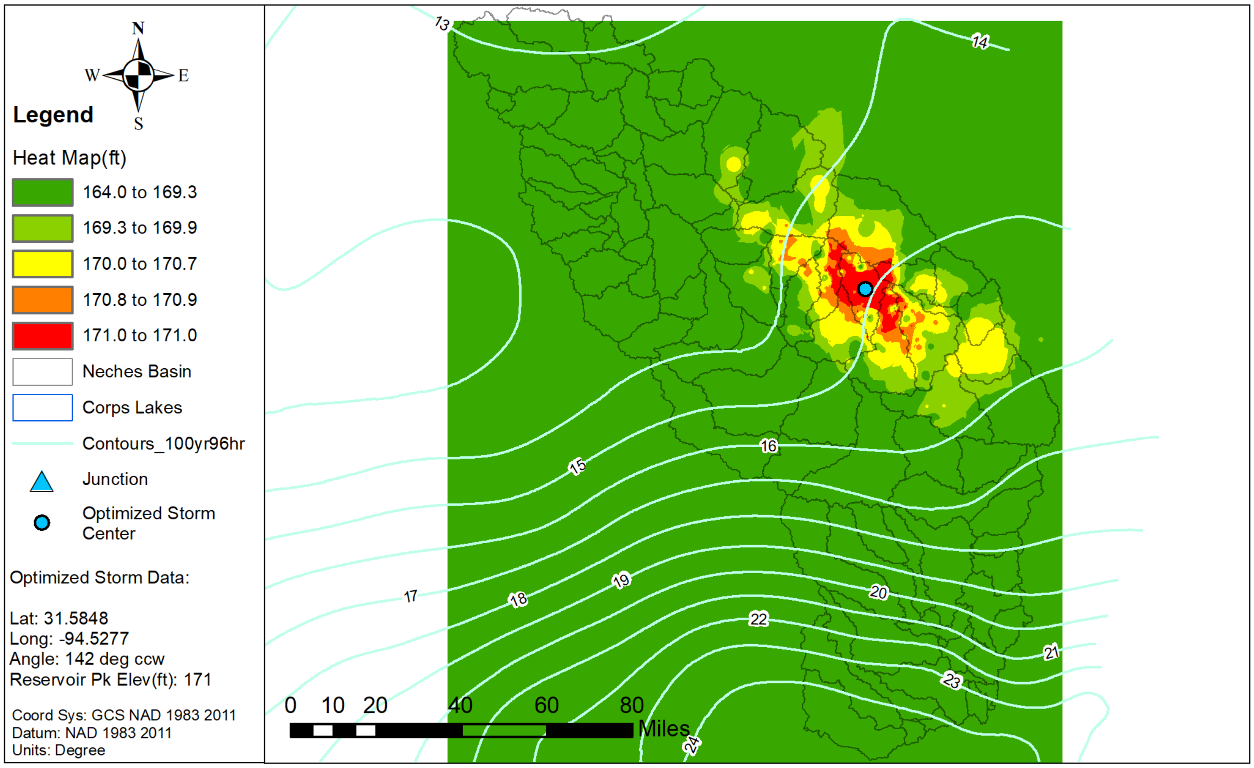


Figure 7.18a: Elliptical Storm Optimization Heat Map for Sam Rayburn Reservoir

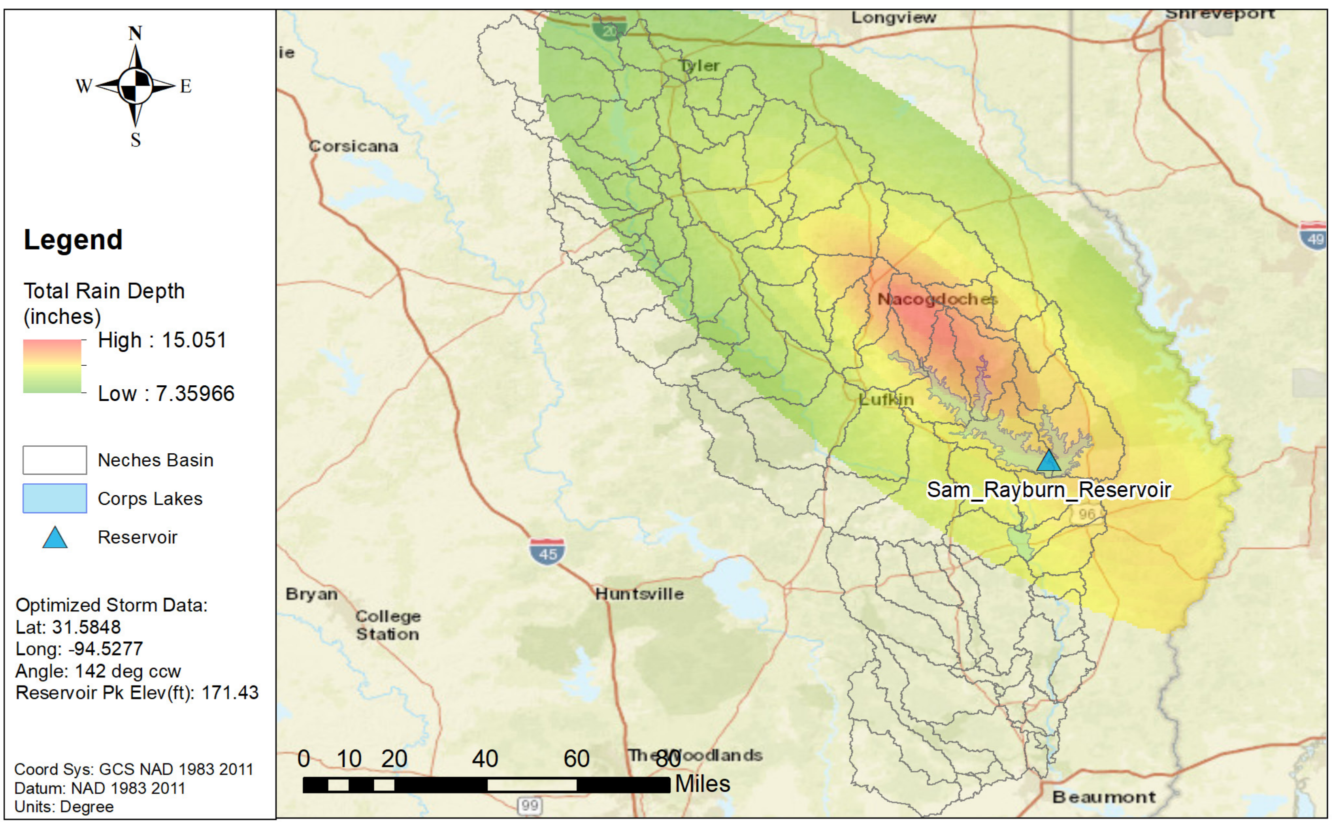


Figure 7.18b: NA14 1% AEP Elliptical Storm for Sam Rayburn Reservoir

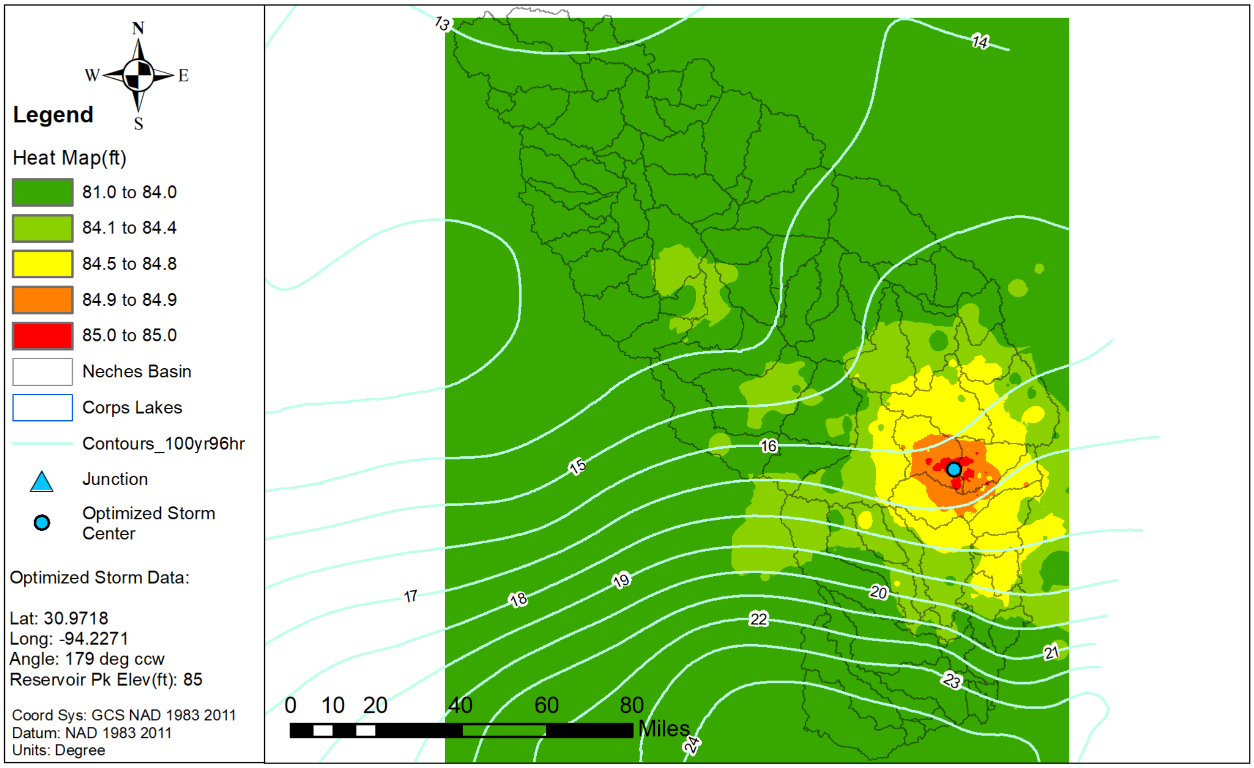


Figure 7.19a: Elliptical Storm Optimization Heat Map for B.A. Steinhagen Lake

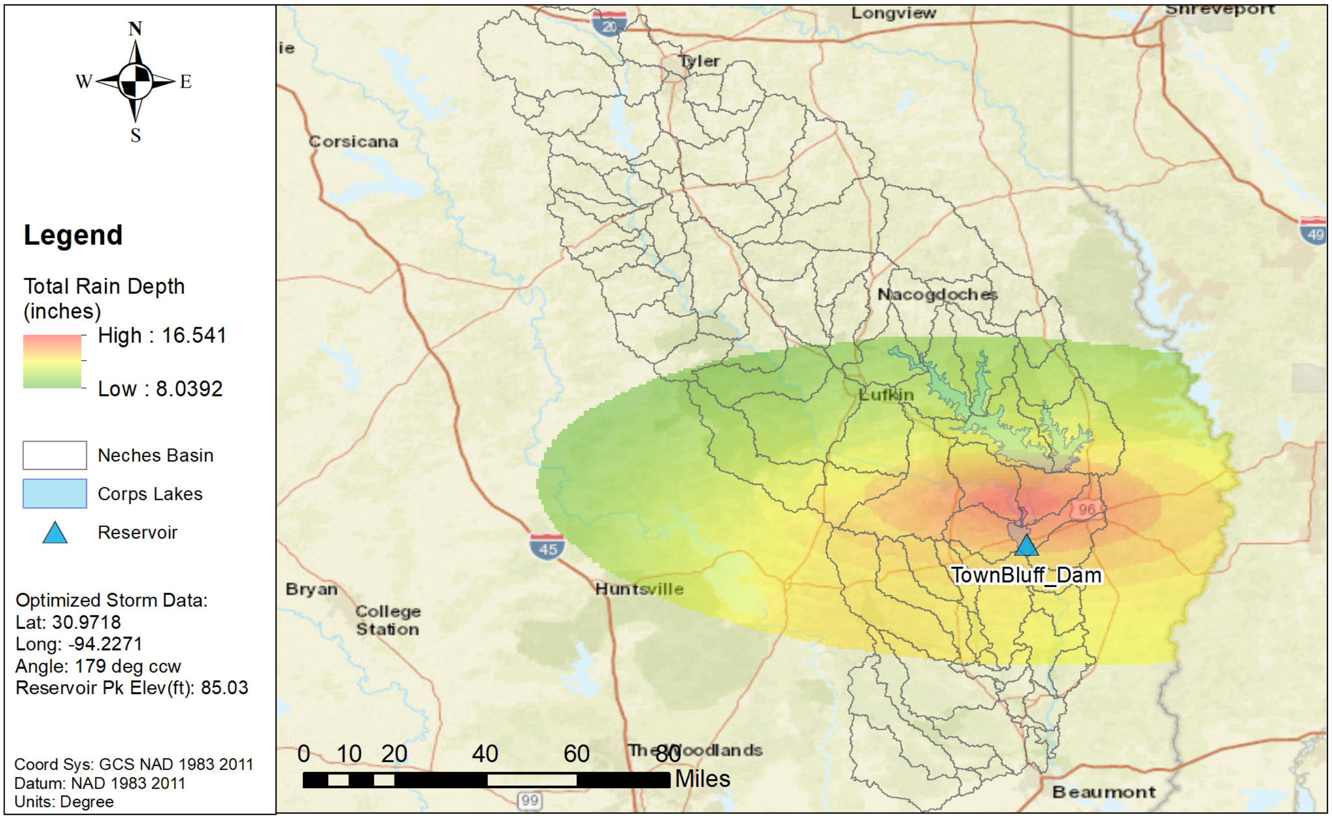


Figure 7.19b: NA14 1% AEP Elliptical Storm for B.A. Steinhagen Lake

7.8 ELLIPTICAL FREQUENCY STORM RESULTS VERSUS DRAINAGE AREA

As a quality check, the peak flow results from the 1% AEP elliptical frequency storms were plotted versus drainage area and outliers were examined, as shown in Figure 7.20. This figure shows that the analyzed junctions followed generally expected patterns of increasing peak flow with drainage area, with exceptions for the effects of large lakes.

As one can see from this figure, the peak inflows to large lakes, such as Sam Rayburn, tend to be high outliers. This is because the entire lake surface is treated as a single point within HEC-HMS. However, the effects of that peak inflow on pool elevation are mitigated by the large amounts of storage in the lake. Resulting pool elevations and outflows from the dam are reasonable and correctly reflect the operations of the dam.

From this figure, one can also see that Lake Palestine and Sam Rayburn Reservoir caused large reductions in peak flow downstream, while Town Bluff dam caused a much more modest decrease in peak flow. This reflects Lake Palestine and Sam Rayburn's larger amounts of storage relative to their drainage areas, whereas the available storage at Town Bluff Dam (B.A. Steinhagen Lake) is much smaller relative to its drainage area.

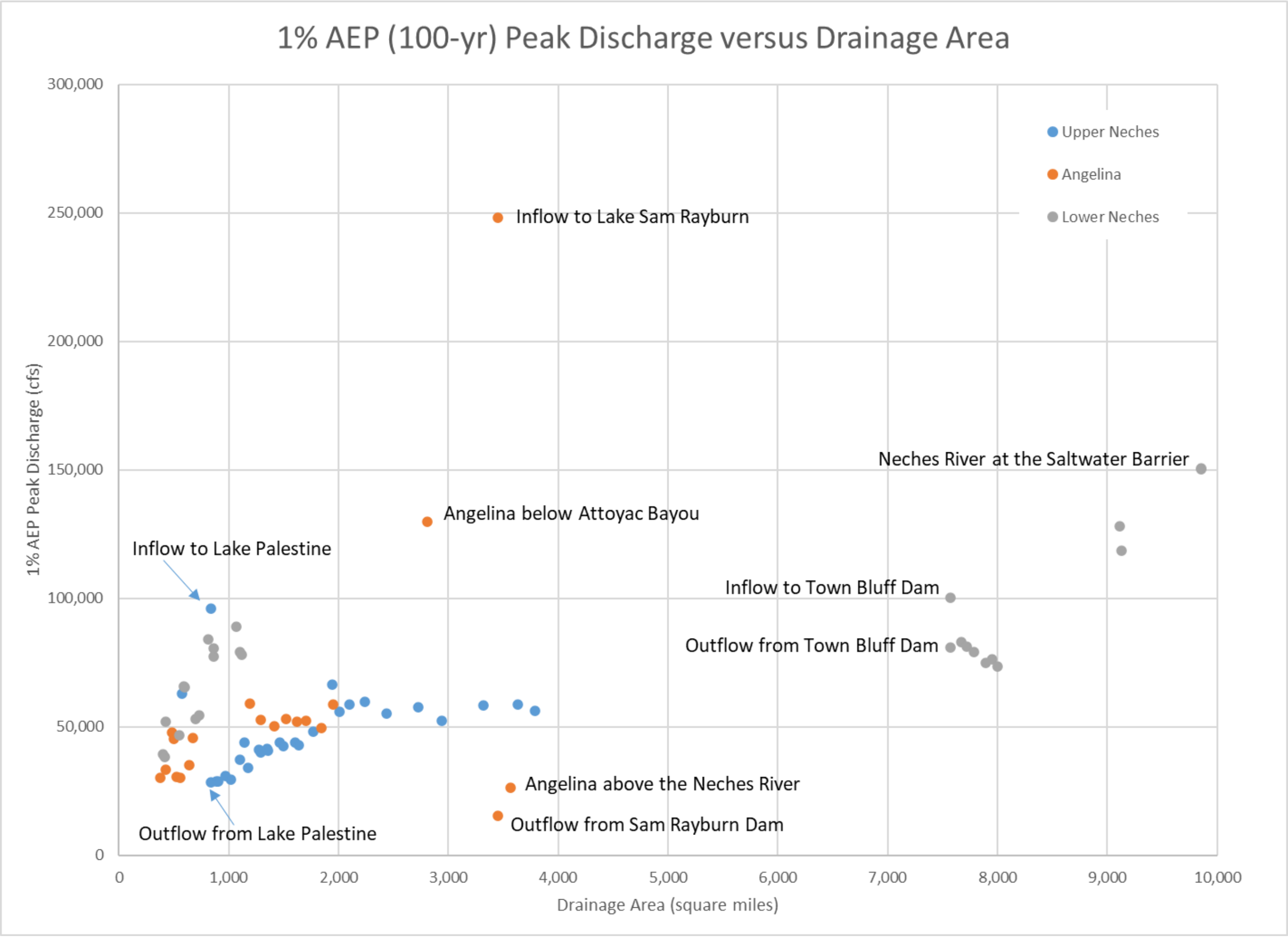


Figure 7.20: NA14 1% AEP Elliptical Storm Frequency Results versus Drainage Area

7.9 ELLIPTICAL STORM VERSUS UNIFORM RAIN FREQUENCY RESULTS

As mentioned at the beginning of this chapter, because the published depth-area reduction curves from TP-40 do not extend beyond 400 square miles, the uniform rainfall method may not always be appropriate for larger drainage areas. Therefore, elliptical frequency storms were computed in HEC-HMS as an alternate method to compare to the uniform rain frequency results for larger drainage areas.

Figure 7.21 below gives a comparison of the percent difference in the 1% annual chance (100-yr) peak flow estimate from the elliptical storms versus the uniform rainfall method. This percent difference is then plotted versus the drainage area of the point of interest. On this plot, a positive value indicates that the elliptical peak flow was higher than the uniform rain peak flow, and conversely, a negative value indicates that the elliptical peak flow was lower than the uniform rain peak flow. Figure 7.22 then plots both sets of data with peak discharge on the y-axis.

From these figures, one may observe that the difference between the two methods generally increases as drainage area increases, which is as expected. The results of the two methods stay within 10% of one another up to approximately 500 square miles. On previous InFRM watershed hydrology assessments for the Guadalupe and Trinity Rivers, the results of the two methods generally stayed within 10% of each other up to at least 1,500 square miles. For this basin, it seems that the relatively long travel times and long, narrow shapes of the upper Neches, Angelina and lower Neches watersheds tend to cause sharper drop offs in the elliptical peak flow results than were observed on the previous river basins.

Large lakes also have varying effects on the difference in peak flow in the Neches River basin and tend to cause some outliers. For example, while the Neches River near Neches, TX USGS gage has over 1,100 square miles of drainage area, the uncontrolled area below Lake Palestine is only 300 square miles. This caused the location of the elliptical storm to be optimized over the smaller uncontrolled area below the lake. As a result, the total rainfall volume being applied to that 300 square mile uncontrolled area was actually higher for the elliptical frequency storm than it was for the uniform rainfall frequency storm, which used uniformly reduced rainfall over the whole 1,100 square miles of drainage area. A similar situation is observed at the Angelina River above the Neches River, which has only 115 square miles of uncontrolled drainage area.

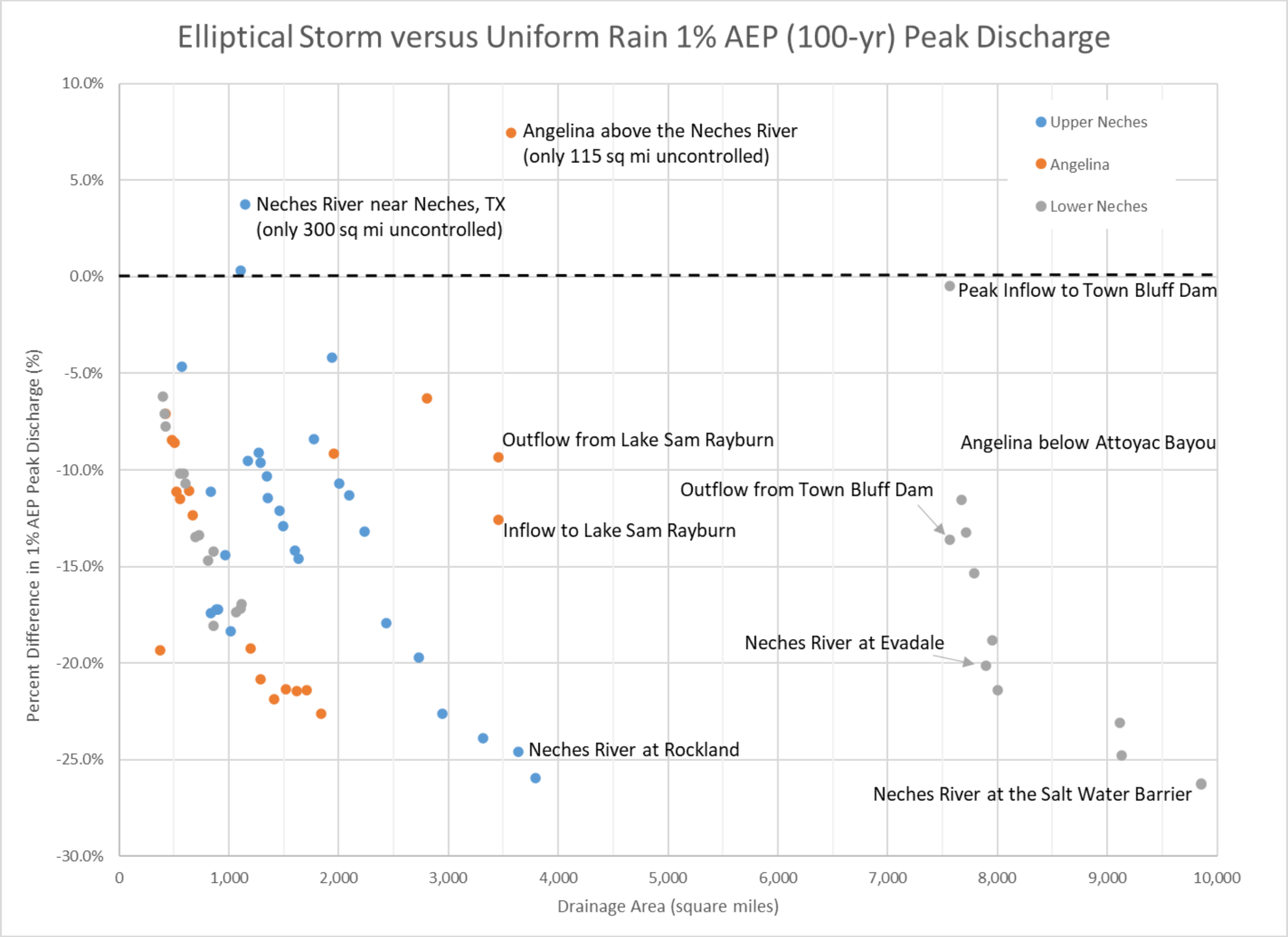


Figure 7.21: Elliptical Storm versus Uniform Rain HEC-HMS Difference in Peak Flow for the 1% ACE (100-yr)

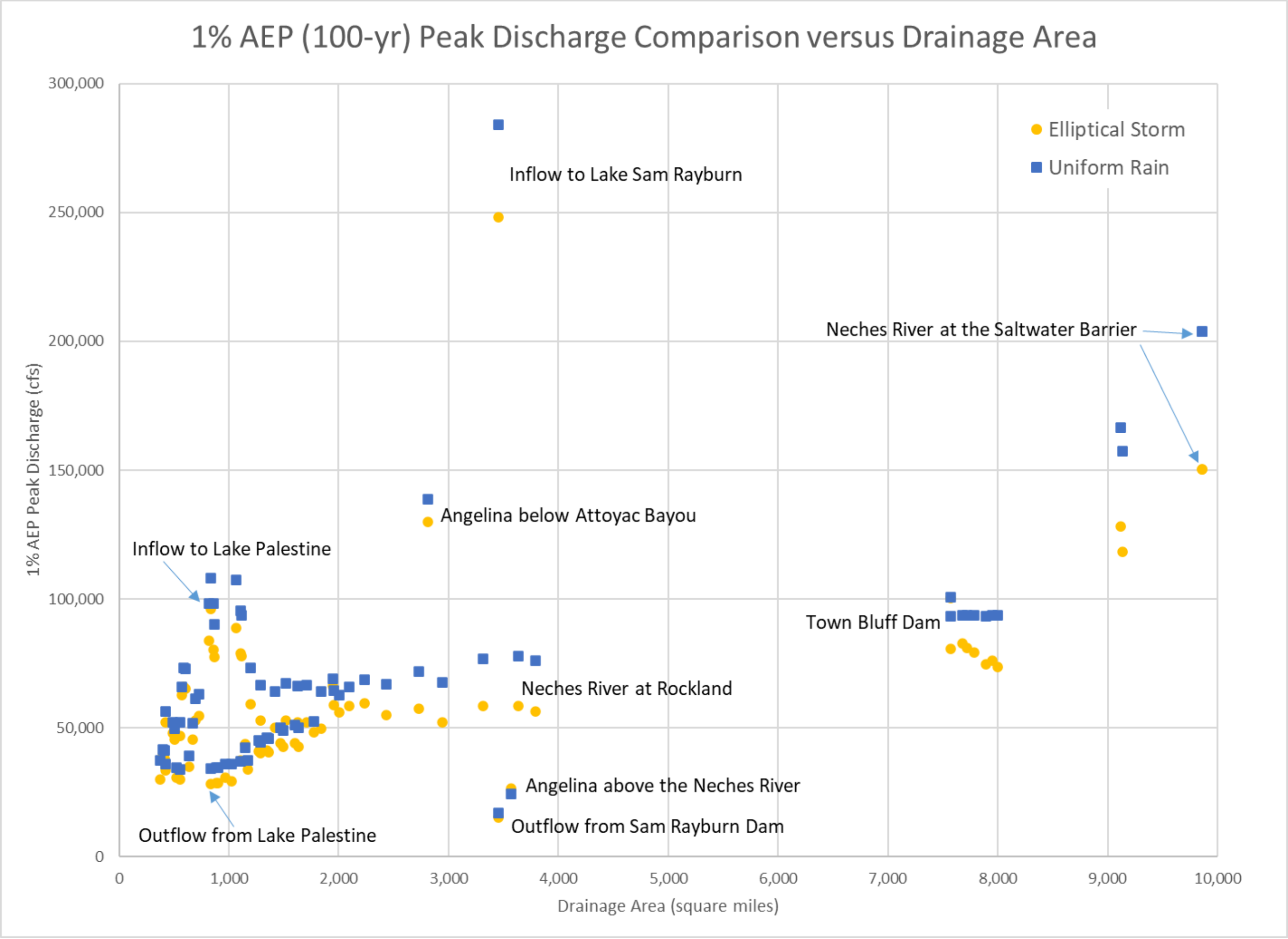


Figure 7.22: Elliptical Storm versus Uniform Rain HEC-HMS Peak Flow Results for the 1% ACE (100-yr)

8 RiverWare Analysis

For the RiverWare portion of the analysis, an existing US Army Corps of Engineers (USACE) Period of Record (POR) model in RiverWare (CADSWES, 2019) was updated for the Neches River Basin. The POR data was extended to include data through water year 2018, and additional detail was added to the model as needed. RiverWare was then used to generate a regulated POR by simulating the basin as if the reservoirs and their current rule sets had been present in the basin for the entire time period. This analysis was used to extend discharge records at various streamgaging stations within the basin from their observed records to an extended simulated record from 1929 to 2018. Statistical flood flow frequency analyses according to Bulletin 17C were then performed on the extended record. The statistical results from the RiverWare model were later compared with the results of other methods from this study.

This chapter summarizes the RiverWare portion of the hydrologic analysis that was completed for the InFRM Watershed Hydrology Assessment of the Neches River Basin. Additional details on the model and analyses are available in Appendix D: RiverWare Analyses.

8.1 INTRODUCTION TO RIVERWARE MODELING

RiverWare is a river system modeling tool developed by CADSWES (Center of Advanced Decision Support for Water and Environmental Systems) that allows the user to simulate complex reservoir operations and perform period-of-record analyses for different scenarios. For the InFRM Watershed Hydrology Assessments, RiverWare is used to generate a homogeneous regulated POR by simulating the basin as if the reservoirs and their current rule sets had been present in the basin for the entire time period. Statistical analyses can then be performed on the extended records at the gages. This report chapter summarizes the RiverWare portion of the hydrologic analysis that was completed for the InFRM Hydrology study of the Neches River Basin.

The RiverWare model described in this chapter presents development of the Neches River Basin hydrology, which mimics current operational conditions. The use of the RiverWare program allows for data extension to periods prior to dam construction. The utilization of longer streamgage record improves discharge frequency results and increases the confidence of the analysis being performed. The modeling evaluation criteria are: (1) evaluate output based on validating policies and functions, and (2) prioritize operation based on surcharge and flood control. A detailed explanation of the Neches River Basin POR hydrology will be in a later section.

Calibration results will also be shown that illustrate model performance since the Saltwater Barrier (SWB) construction was completed in 2005. The time window simulation run is for water year (WY) 2005 – WY 2018. This time window also captures the time when Hurricane Harvey occurred (late August of 2017). Each simulated water year was inspected individually to better validate the results.

After calibration, a general run for January 01, 1929 through WY 2018 was made. Historical pool elevations along with observed inflows and outflows were compared against the model simulated results. More emphasis was put on B.A. Steinhagen's operations because the dam captures two major rivers (i.e., the Angelina and the Neches Rivers). Results were inspected closely for B.A. Steinhagen's pool and releases, the simulated discharges at the Neches at Evadale gage, and the simulated discharges at the SWB at Beaumont, Texas.

8.1.1 Existing USACE Models

Two existing RiverWare models were available for the Neches River Basin at the onset of this study. The USACE Fort Worth District (SWF) Neches RiverWare model, which was based off of hydrology from the USACE Southwestern Division (SWD) legacy FORTRAN SUPER program. Additionally, a version of the Neches RiverWare model updated by Riverside Technologies Inc. (RTI International) was available, which had improved low flow capability. A functionality was developed to replicate algorithms and consolidate object methods, defined functions, and other utilities from the SUPER program to the RiverWare program, the hydrology was then generated and fed into the RiverWare improved model. The latter was used to validate operations and mimic observed data throughout the Neches River Basin. The concept of using two separate models was to generate local flows from the hydrology model that can be processed in the study model. The algorithmic based functions embedded in the hydrology model, enable the user to apply the right mass balance functions, and route flows throughout the network. The routing procedures capture lag time and peak attenuation. The parameters applied in the hydrology model are normally copied from the legacy SUPER program files. The hydrology model would also provide an accountability of producing incremental and cumulative local flows for further processing.

8.1.2 Updates to the RiverWare Model

Discharge data was updated through WY 2018. Both the hydrology and operational (study) models begin on September 30, 1928. Rule sets were written for the operational model to mimic conservation releases. As conservation releases have changed throughout the years due to differing demands, the ruleset attempted to recreate recent demands and to match approximately the last 14 years of record, from WY 2005–2018.

8.1.3 Model Description

The Neches River Basin model was developed in RiverWare for Sam Rayburn Reservoir and B.A. Steinhagen Lake operations. The upstream modeling boundary is Rockland Dam Site (Neches River near Rockland, Tex., U.S. Geological Survey (USGS) streamgaging station 08033500) on the Neches River (upstream from the confluence of the Neches and Angelina Rivers) and Lake Eastex Dam Site (Angelina River near Alto, Tex., USGS Streamgaging station 08036500), located upstream from Sam Rayburn on the Angelina River. These boundary sites are represented as RiverWare Control Point objects with imported Deterministic Incremental Local Inflow slot values. The downstream modeling boundary is the permanent Saltwater Barrier (represented as a Control Point object) downstream from the confluence of the Neches River and the Pine Island Bayou. There are additional local inflow points at Sam Rayburn, B.A. Steinhagen, Evadale, and Pine Island Bayou confluence points.

Rules in the model adapted the RiverWare USACE-SWD regulation policies for the Neches River Basin. The USACE-SWD rules solve the basin as a system and use SUPER model algorithms for flood control releases, conservation pool operations, and hydropower releases. The USACE-SWD rules also disaggregate local inflows and forecast cumulative inflows, in which the forecasted flows are used in the network algorithms.

While there is no longer a requirement to release a constant 1,700–2,500 cubic feet per second (cfs), Sam Rayburn operations still attempt to meet the Lower Neches Valley Authority (LNVA) water supply request at the Saltwater Barrier. Conservation pool releases may supplement downstream local flows on the Angelina River and flows on the Neches mainstem to satisfy LNVA water supply requests. The approximate 5-day lag between Sam Rayburn reservoir and the Saltwater Barrier; the re-regulation of Sam Rayburn releases at B.A. Steinhagen Dam; and the estimation of additional contributing flow complicates modeling water supply releases. Table 8.1 shows model element names and types. Figures 8.1a and 8.1b show the schematic of the RiverWare network for the Neches River Basin.

Table 8.1: Neches River Basin RiverWare Model Elements and Types

Element Name	Type	Element Name	Type
Lake Eastex Dam Site	Control point	B A. Steinhagen_Divert	Diversion
Lake Eastex_Alto	Reach	B A. Steinhagen Pumpage	Pump
Alto_Sam Rayburn	Reach	B A. Steinhagen Outflow	Control point
Sam Rayburn	Level power reservoir	B A. Steinhagen_Evadale	Reach
Sam Rayburn_Divert	Diversion	Evadale	Control point (Downstream control point)
Sam Rayburn Pumpage	Pump	Evadale_Village Creek Confluence	Reach
Sam Rayburn Outflow	Control point	Village Creek Confluence_Pine Island Bayou confluence	Reach
Sam Rayburn_B A. Steinhagen	Reach	Pine Island Bayou Confluence	Control point
Rockland Dam Site	Control point	Pine Island Bayou_Divert_Reach	Reach
Rockland_B A. Steinhagen	Reach	Pine Island Bayou Pumpage	Pump
Neches at Angelina	Confluence	Pine Island Bayou Confluence_Salt Water Barrier	Reach
B A. Steinhagen	Level power reservoir	Salt Water Barrier	Control point

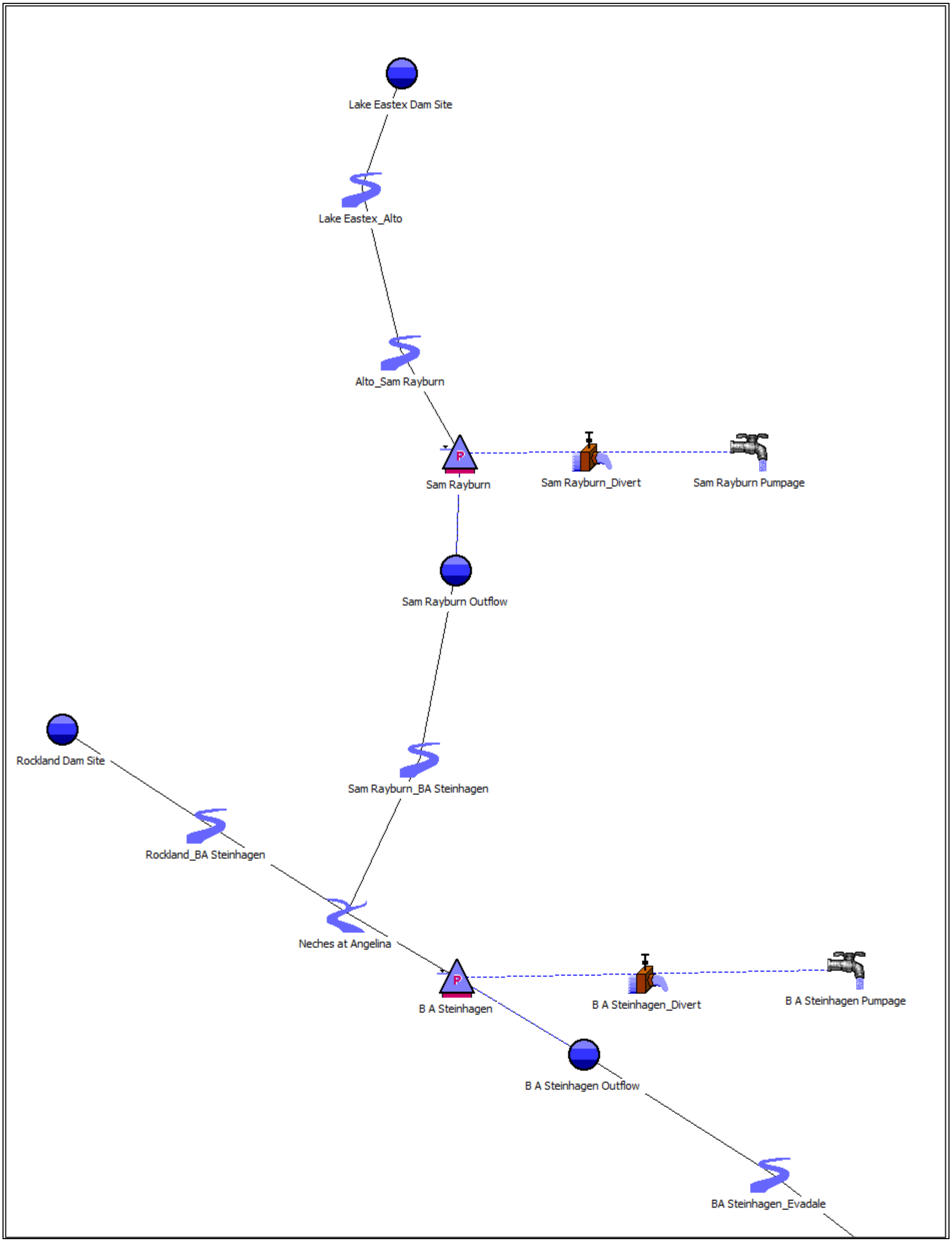


Figure 8.1a: RiverWare Neches River Basin Network above Evadale

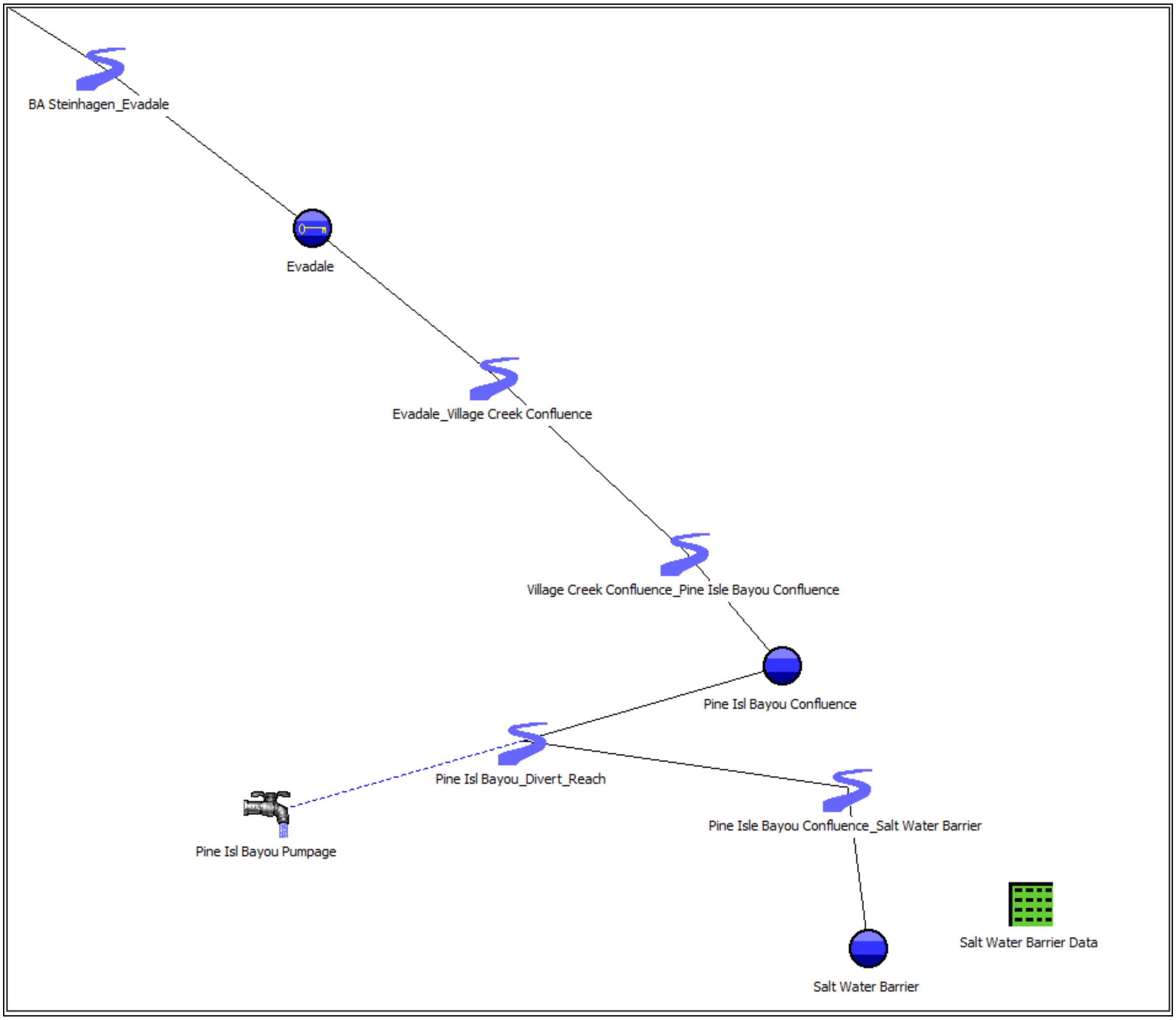


Figure 8.1b: RiverWare Neches River Basin Network below Evadale

8.2 DATA SOURCES USED IN THE RIVERWARE MODEL

The modeling efforts in the study area heavily rely upon sound hydrology. Accurate hydrologic analyses reflect more realistic runoff conditions in the watershed, which can change over time due to urbanization, population growth, agricultural demands, and climate change (i.e., drought or increased flooding due to changes in precipitation conditions). The developed hydrology was based on using the USGS streamgage data at locations of interest. Streamgaging stations with the longest POR were used as the basis for developing streamgaging stations with missing discharge records around the basin. Moreover, data consists of observed USGS discharges, which are measured by the USGS (2018), and pool elevation, adjusted inflow, gated and turbine flows, and evaporation rates, which are maintained by the USACE-Fort Worth (SWF) Water Management Section. Table 8.2 lists all gaged and ungaged data used in the RiverWare models. The locations of the USGS streamgaging stations in the Neches River Basin are shown in Figure 8.2.

Table 8.2: USGS and USACE-SWD Data Used in the RiverWare Model

Location	Data Type (Units)	Source
Sam Rayburn Reservoir	Evaporation (inch per hour)	USACE-SWD database
B.A. Steinhagen Lake	Evaporation (inch per hour)	USACE-SWD database
Neches River near Town Bluff, Tex.	Discharge (cubic feet per second)	USGS 08040600
Neches River at Evadale, Tex.	Discharge (cubic feet per second)	USGS 08041000
Village Creek near Kountze, Tex.	Discharge (cubic feet per second)	USGS 08041500
Mud Creek near Jacksonville, Tex.	Discharge (cubic feet per second)	USGS 08034500
Neches River near Rockland, Tex.	Discharge (cubic feet per second)	USGS 08033500
Pine Island Bayou above BI Pump, Beaumont, Tex.	Discharge (cubic feet per second)	USGS 08041749
Neches Rv Saltwater Barrier at Beaumont, Tex.	Discharge (cubic feet per second)	USGS 08041780
Sam Rayburn Inflow	Discharge (cubic feet per second)	USACE-SWD database
B.A. Steinhagen Inflow	Discharge (cubic feet per second)	USACE-SWD database
Sam Rayburn gated discharge	Discharge (cubic feet per second)	USACE-SWD database
Sam Rayburn turbine release	Discharge (cubic feet per second)	USACE-SWD database
Sam Rayburn Pool	Elevation (NGVD-29 feet)	USACE-SWD database
B.A. Steinhagen Pool	Elevation (NGVD-29 feet)	USACE-SWD database

Note: NGVD = National Geodetic Vertical Datum of 1929

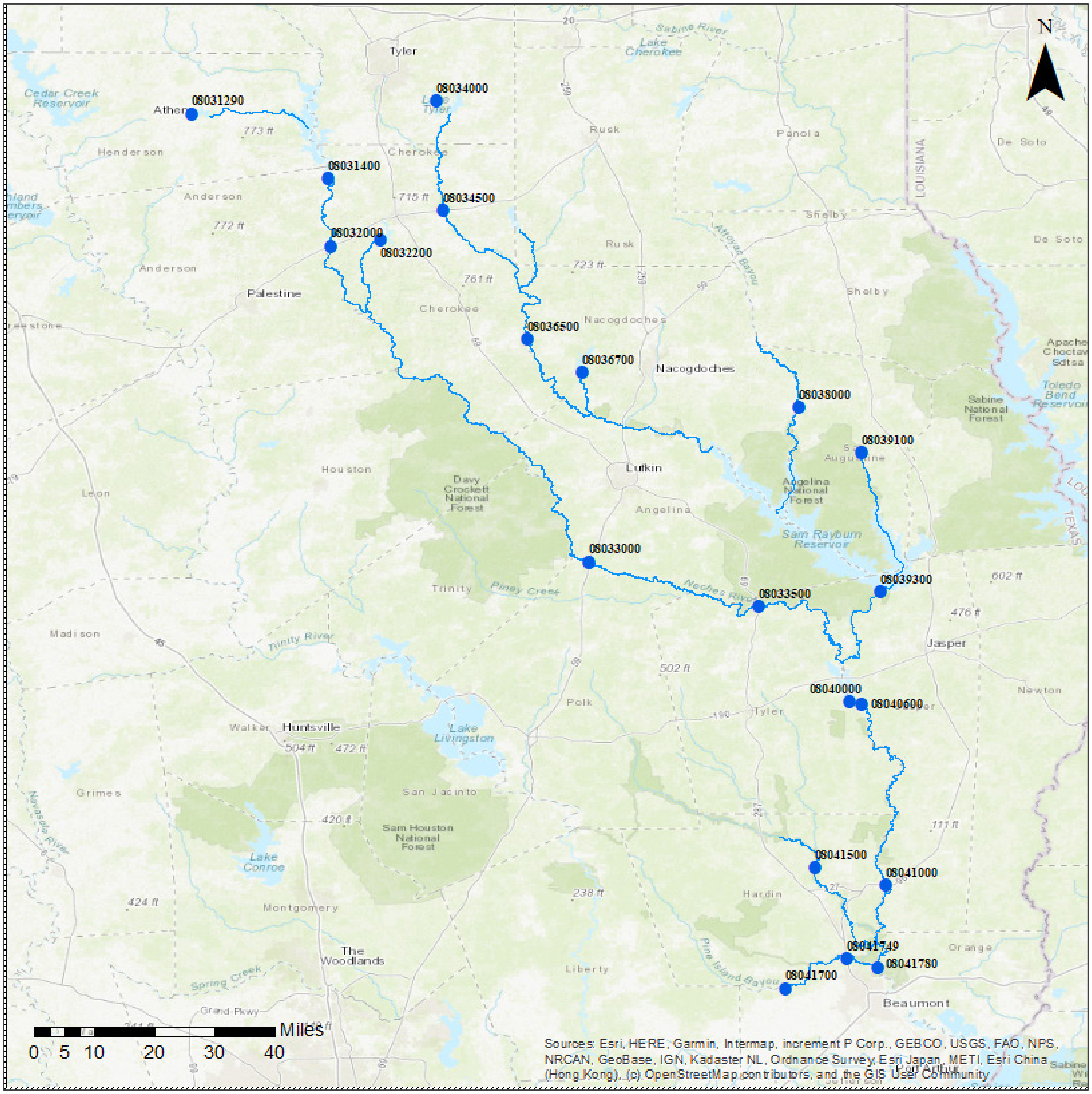


Figure 8.2: USGS Streamgage Locations in the Neches River Basin

8.3 PERIOD OF RECORD HYDROLOGY DEVELOPMENT

8.3.1 Methodology Used to Develop Period of Record Hydrology

The important methods used to develop the POR hydrology for the Neches River Basin in this chapter are the drainage-area-ratio method, reservoir inflow calculation, and reservoir inflow smoothing algorithm. This section describes the methodology used in developing the POR.

Rarely is there a POR watershed study where sufficient and consistent streamgage datasets exist. Incomplete streamgage datasets for streamgaging stations and reservoirs gages can be attributed to budget limitations and anthropogenic changes (*i.e.,* installation of reservoirs). Once filling techniques were established for each gage, a few years with missing discharges were observed. To reconcile the inconsistent dataset, the final missing discharges were generated using USGS streamgaging station 08033500, Neches River near Rockland, Tex. (USGS, 2018), applying the drainage area ratio method (Gupta, 2008). The USGS streamgage near Rockland has continuous record from 1903 through 2018 and drains about 3,636 mi^2 between Sam Rayburn Reservoir and B.A. Steinhagen Lake.

The drainage-area-ratio method provides a numerical approximation of the missing streamgage data, using streamgage datasets upstream or downstream on the same river (Equation 1).

$$Q_y = \frac{Q_x}{A_x} A_y$$

Equation 1: Drainage-Area-Ratio Method

Q_y = Discharge at ungaged site y of drainage area A_y $[L^3/T]$

Q_x = Discharge at gaged site x of drainage area A_x $[L^3/T]$

A_y = Drainage area of ungaged site y $[L^2]$

A_x = Drainage area of available gaged site x $[L^2]$

The numerous arrays of reservoir inflow calculations tolerate for thoroughness, as well as discontinuity. All reservoir inflow calculations share a priori mass balance approach. The method selection for the calculation of reservoir inflow is subjective and ultimately should be selected on a case by case basis. There is one method used to calculate reservoir inflows in this study. It is the “evaporation reservoir inflow method” (method applied to USACE datasets).

$$I = \Delta S + E + R + Q_{total}$$

Equation 2: Evaporation Reservoir Inflow Method

I = Inflow into the reservoir $[L^3/T]$

ΔS = Change in reservoir storage $[L^3/T]$

E = Evaporation from the reservoir $[L^3/T]$

R = Releases from the Reservoir $[L^3/T]$

Q_{total} = Total pumpage out of the reservoir $[L^3/T]$

The calculated reservoir inflow is subject to measurement error and numerical error. The evaporation parameter is arguably the most difficult parameter to estimate when calculating reservoir inflow. The uncertainty in measurement often leads to negative reservoir inflow values, which violates the conservation of mass theory. Reservoir release rates can also be inaccurate due to the imperfect nature of setting the gate height at the project. To resolve these inconsistencies the reservoir inflow values are numerically smoothed by scaling positive inflows and rectifying negative inflows. The smoothed inflow algorithm is applied over a monthly time period with a daily time step and preserves the volume of the monthly total (Equation 3, Equation 4, Equation 5, and Equation 6). There are additional inflow smoothing methods available, but this method is sufficient to resolve negative reservoir inflows in this case.

$$\text{Montly Total Inflow} = \sum_{i}^{i_f} I_i$$

Equation 3: Monthly Total Inflow Method

$$\text{Nonnegative Inflow} = \left\{ \begin{matrix} \text{if } I_i < 0 \\ 0 \\ \text{else} \\ I_i \end{matrix} \right\}$$

Equation 4: Nonnegative Inflow Method

$$\text{Montly Total Nonnegative Inflow} = \sum_{i}^{i_f} \text{Nonnegative Local}$$

Equation 5: Monthly Total Nonnegative Inflow Method

$$\text{Smoothed Inflow} = \left\{ \begin{matrix} \text{if Monthly Total Inflow} < 0 \text{ OR Montly Total Nonnegative Inflow} = 0 \\ \text{Nonnegative Inflow} * 0 \\ \text{else} \\ \text{Nonnegative Inflow} * \dfrac{\text{Monthly Total Inflow}}{\text{Montly Total Nonnegative Inflow}} \end{matrix} \right\}$$

Equation 6: Smoothed Inflow Method

I = Inflow into the reservoir on the i^{th} day $[L^3/T]$

$i = i^{th}$ day of the month

i_f = last day of the month

Montly Total Nonnegative Inflow = Summation of the monthly nonnegative inflows $[L^3/T]$

Montly Total Inflow = Summation of the monthly reservoir inflows $[L^3/T]$

Nonnegative Inflow = A nonnegative dataset of the reservoir inflows $[L^3/T]$

Smoothed Inflow = A smoothed dataset of the reservoir inflows $[L^3/T]$

The methods presented above along with the RiverWare modeling software have permitted the development of POR hydrology for the Neches River Basin. The following section will describe how these methods were implemented within the framework of the RiverWare modeling software and the precursor to the RiverWare modeling software.

8.3.2 Period of Record Hydrology for the Neches River Basin

The POR hydrology needed to evaluate the Neches River Basin requires the use of numerical models. RiverWare version 7.2.5 was used to analyze the hydrology and hydraulic processes of Sam Rayburn Reservoir and B.A. Steinhagen Lake and the river reaches within the Neches River Basin. The hydrology and hydraulic analysis include the use of a multiple-run and simulation-run RiverWare model. The multiple-run RiverWare model produced the POR hydrology from January 01, 1929 to September 30, 2018 for all streams and reservoirs streamgage sites. The POR hydrology is the naturalized local discharges, where major anthropogenic impacts have been removed, including effects of reservoir regulation. The simulation-run RiverWare model used the POR hydrology datasets to simulate Sam Rayburn Reservoir and B.A. Steinhagen Lake pool elevations with reservoir regulation policies incorporated for the entire POR, which will be used in the statistical frequency analysis portion of the study.

The process for developing POR hydrology, for the reservoirs and control points or streamgaging stations of interest, is to assimilate historical reservoir inflow and stream discharge datasets, then implement drainage-area-ratio methods and reservoir inflow smoothing algorithms in a multiple-run RiverWare model to numerically solve for the POR hydrology. Analyzing pool elevations and operational release over the POR requires the POR hydrology and reservoir operational policies and rule sets to be incorporated into a simulation-run RiverWare model. The reservoir operational policies and rule sets applied to reservoirs can then be compared to historical pool elevations, releases, and local inflows to verify consistency with historical datasets. Ultimately the policies and rule sets can be applied to the POR hydrology to establish synthetic pool elevation and reservoir operation before the reservoirs existed.

The current improved model developed by RTI International was used to improve simulation of water supply releases. This update is required because Sam Rayburn Reservoir conservation pool operations have deviated from the Water Control Manual since the construction of the permanent Saltwater Barrier on the Neches River in 2005. The permanent Saltwater Barrier eliminated the requirement to release a constant 1,700-2,500 cfs to prevent saltwater intrusion, allowing for operational flexibility to maximize various operational objectives at Sam Rayburn Reservoir.

8.4 WATER CONTROL PLANS FOR THE NECHES USACE RESERVOIRS

Table 8.3 lists some main operational procedures, flood control key points, and objectives of each modeled reservoir in the RiverWare model. This information can be found in chapter 7 of the respective reservoir's Water Control Manual (WCM) (USACE-SWF, 2016) (USACE-SWF, 2018).

Table 8.3: Highlights from the Water Control Manual for Sam Rayburn Reservoir and B.A. Steinhagen Lake

Purpose/Downstream Control points/Pool zones	**Sam Rayburn**	**B.A. Steinhagen**
Dam Type	Storage	Run-of-River
Purpose	flood control, water supply, and hydroelectric power, fish and wildlife, and general recreation	Regulation surges due to hydropower releases from Sam Rayburn Dam, water supply, fish and wildlife, and general recreation
Control Point Located downstream of B.A. Steinhagen	20,000 cfs at Neches River near Town Bluff (USGS 08040600), Neches River at Evadale (USGS 08041000)	20,000 cfs at Neches River near Town Bluff (USGS 08040600), Neches River at Evadale (USGS 08041000)
Pool zone Top of conservation Top of flood Surcharge Top of Spillway Crest Top of Dam	Elevation (NGVD-feet) 164.40 173.00 Above 176.00 176.00 193.60	Elevation (NGVD-feet) 83.00* N/A Above 85.00 85.00 95.00

* Top of conservation has fluctuated over the years between 81.5 and 83 feet. It is now set to 83 feet, per Hurricane Katrina regulations and for water supply requirements by the LNVA.

In RiverWare, policies and functions were written to reflect the current reservoir regulation schedule for each lake. The WCM calls for the following:

B.A. Steinhagen's pool is maintained at top of conservation (83.00-ft) with releases not to exceed 2,000 cfs. If forecast to rise pool above 83.00-ft, releases should be made not to exceed 20,000 cfs at the downstream control points, downstream of B.A. Steinhagen.

Sam Rayburn Reservoir is regulated to reduce flooding on the Angelina and Neches Rivers below the dam. Flood storage in Sam Rayburn Reservoir will be released as soon as downstream channel capacity is available. Table 8.4 lists Sam Rayburn's release schedule. Release procedures in this table were also included in the RiverWare model for simulation.

Table 8.4: Sam Rayburn Reservoir Release Schedule

Pool Elevation **(NGVD-feet)**	**Maximum Allowable Release**
164.40	No flood control release
164.4 to 165.0	4,400 day-second-feet (dsf)
165.0 to 165.5	8,800 dsf
165.5 to 173.0	Enough release not to exceed 20,000 cfs at Town Bluff or Evadale
173.0 to 176.0	Enough release not to exceed 20,000 cfs at Town Bluff or Evadale
176.0 and above	Spillway overflow should be kept to minimum

8.5 RIVERWARE OPERATIONAL MODEL APPLICATION

The RiverWare simulation model executes all flood control releases, so as to maximize flood release within the period of perfect knowledge. This period is defined as: the number of time steps for which the forecast will equal the Deterministic Incremental Local Inflow, i.e., the forecast is known with complete certainty. In real time historical operations, there are numerous and event-specific reasons as to why the reservoir was operated the way it was. Meteorological forecasts from the National Weather Service, as well as river stage forecasts issued by the West Gulf River Forecast Center could both potentially influence the rate of release from the project.

The Neches River Basin RiverWare model includes policies implemented as rules. Rule number 1 is the highest priority rule and executes last (i.e., hydropower release rule) while the rule with the highest number is the lowest priority rule and executes first (i.e., Surcharge rule). Figure 8.3 below shows the priority list of policies implemented in the model. As seen, the flood control policies execute first and this is mainly to control flooding at damage center locations located downstream.

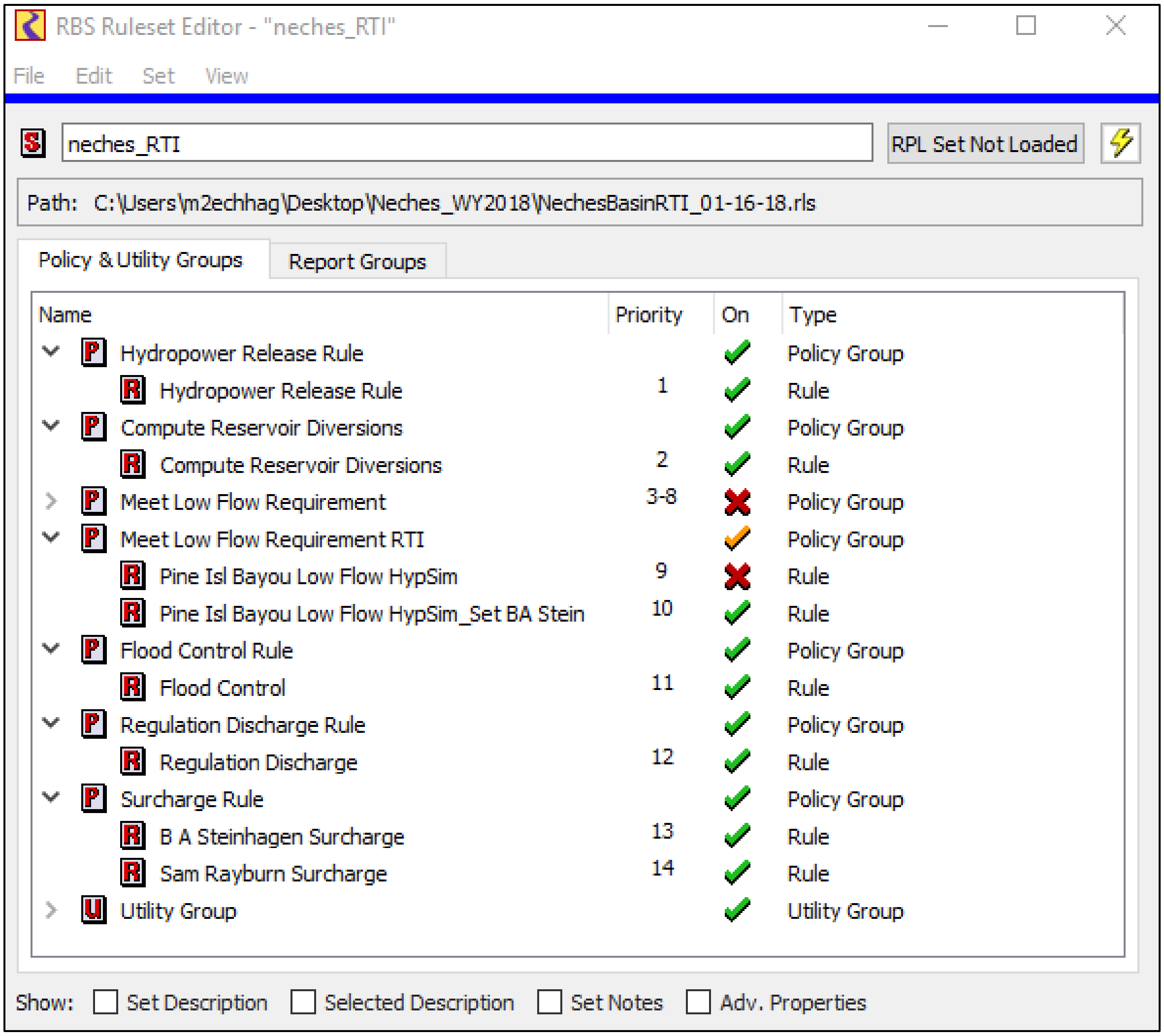


Figure 8.3: Neches River Basin Rule-based Simulation Groups

In addition to issues associated with using the built-in USACE-SWD conservation pool operations, the improved model included a custom rule to simulate water supply releases. To meet low flow requirement, USACE policy employs multiple hypothetical simulations to best estimate the necessary water supply release in an iterative manner, which estimates releases from B.A. Steinhagen Lake, as a proxy for what releases from Sam Rayburn Reservoir are needed to meet water supply demand in five days. Five days is the travel time between B.A. Steinhagen and the SWB.

8.6 MODEL CALIBRATION RESULTS AND DISCUSSION

Overall, the model displays satisfactory results between simulated and observed considering operation limitations. The rules used for simulation do not always produce matching results of the historical (observed) discharges because real-time operation is normally based on real-time forecasting, which causes release deviations from the WCM schedule. The model uses the deterministic flow with a simple forecasting technique and a set of policies. The surcharge, regulating discharge, and flood control rules execute first, while also accounting for the flow demands downstream and the travel time to the SWB.

For example, B.A. Steinhagen Lake's observed pool of WY 2009 was distinguished by steep drawdown (Figure 8.4). The pool was drawn down in August to repair a cofferdam sheet pile. This was an instance of where RiverWare results did not match observed pool levels, as observed operation was not based on flood control drawdown. The RiverWare rules regulated to a steady pool level, while a sharp drawdown was observed in real life. However, this difference in pool elevation resulted in no noticeable difference in the discharges at Evadale, where discharges remained below 20,000 cfs throughout that period, as shown in Figure 8.4.

The overall performance of the model's pool and release output can be viewed in Figure 8.5. The B.A. Steinhagen Lake RiverWare policies were written to mimic the relatively small storage that it holds and the effects of high inflows that push the pool above 83 feet and the outflow above 20,000 cfs. Once the pool gets high enough, the outflow equals the inflow. Additional examples and discussion of model calibration results are available in Appendix D.

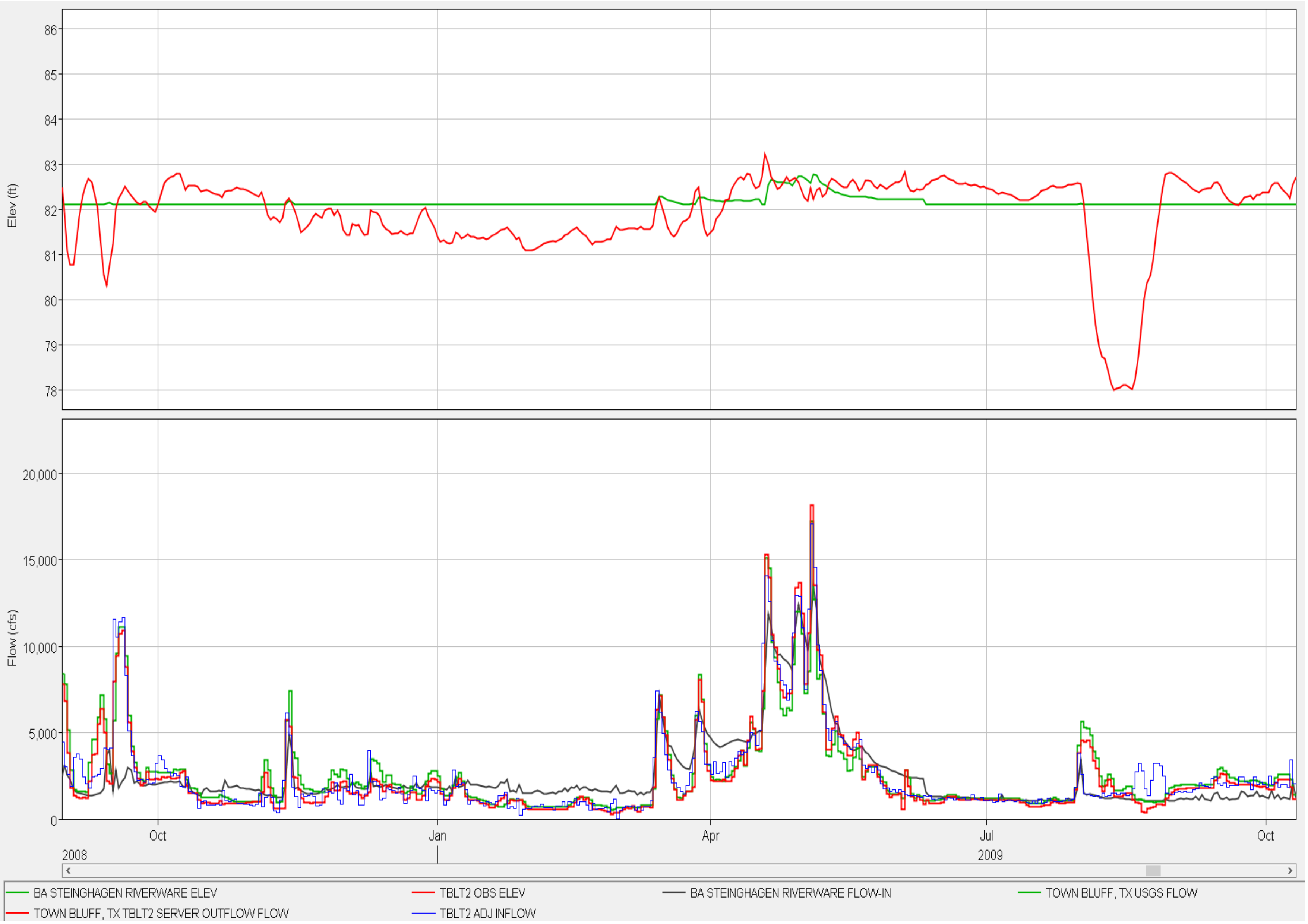


Figure 8.4: B.A. Steinhagen's Pool Operation for WY 2009

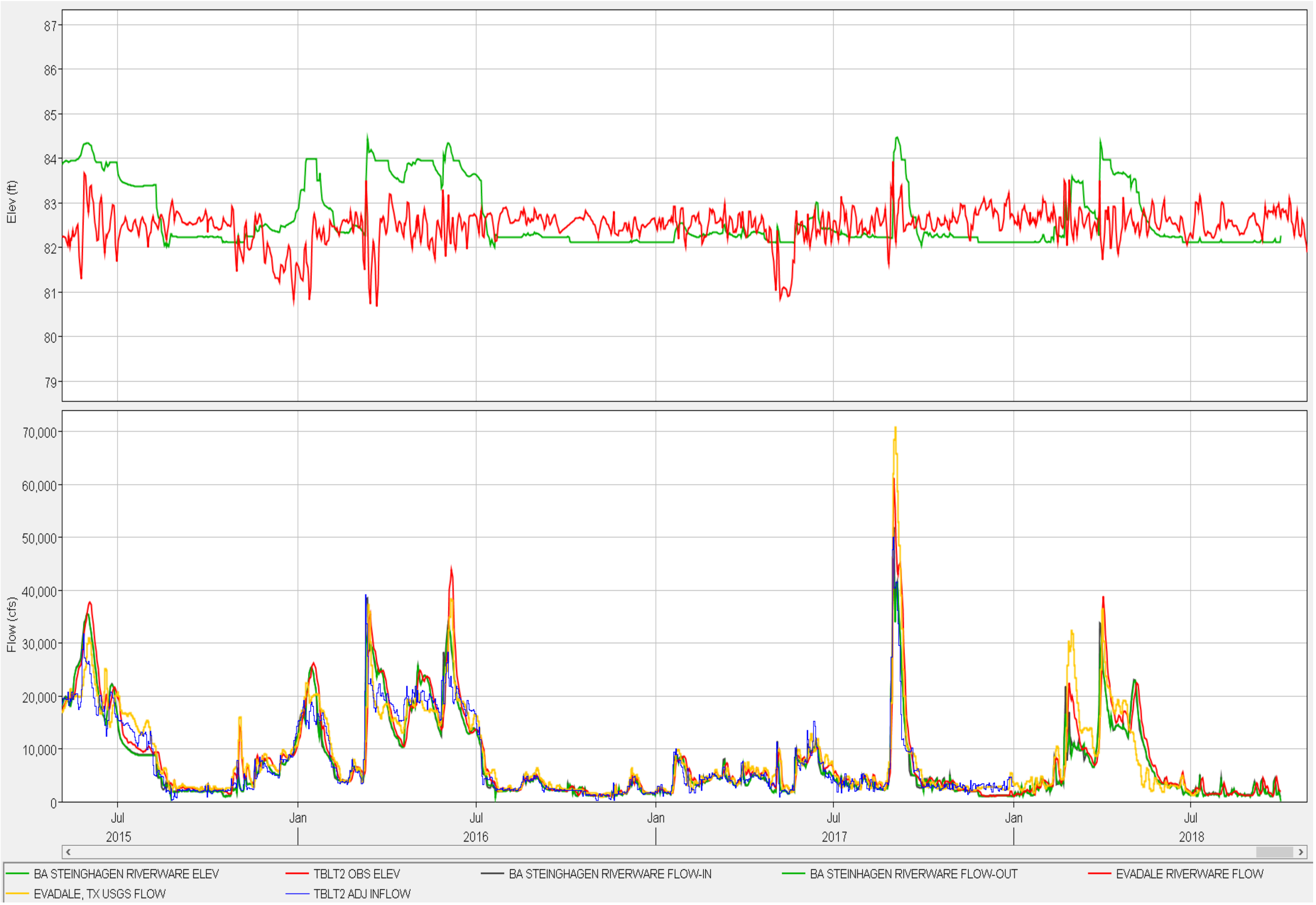


Figure 8.5: Model Simulation Results for Water Years 2015 Through 2018

8.7 SENSITIVITY ANALYSES

Additional sensitivity analyses were performed in an attempt to improve model results. The following changes showed limited improvement to pool conditions and releases downstream and therefore were not adopted into the final model. The changes were made to both reservoirs' operating conditions.

Sensitivity tests were performed by making changes to the base operating level table zones for Sam Rayburn Reservoir and B.A. Steinhagen Lake. The operating level table contains specific elevation zones. Zone 9 for example, is the top of flood control pool, and assigned 167 feet for Sam Rayburn and 83 feet for B.A. Steinhagen Lake. Zone 15 on the other hand, represents the top of spillway crest (*i.e.*, 176.0 feet for Sam Rayburn Reservoir and 85.0 feet for B.A. Steinhagen Lake). The intermittent zones create a buffer for smooth transitioning in the pool. These changes did not result in any significant improvements to the calibration results.

Moreover, changes to the level regulation table for B.A. Steinhagen's release schedule were tested to assess impacts at downstream locations. This condition table works similar to the elevation level table. These changes did not result in any significant improvements to the calibration results at the downstream locations.

Additional details on the sensitivity tests that were performed and their results can be found in Appendix D. Ultimately, the base operating level table zones for Sam Rayburn Reservoir and B.A. Steinhagen Lake were adopted for final POR simulation.

8.8 FINAL RIVERWARE MODEL PERIOD OF RECORD RESULTS

The final RiverWare adopted reservoir operations tables are shown in Tables 8.5, 8.6, and 8.7. These tables reflect actual operating conditions of the lakes. The associated simulation runs at Evadale and the SWB are shown in Figures 8.6 and 8.7. The plots match well when compared to the observed USGS streamgaging stations at the same locations. Furthermore, a snapshot of the POR simulation for B.A. Steinhagen pool, Sam Rayburn Pool, and discharges at Evadale and the SWB are shown in Figures 8.8, 8.9, and 8.10, respectively. The data in each plot was used in a tabular format as input to the flood flow frequency analyses described in the next sections.

Table 8.5: Sam Rayburn Operating Level Table

Date	**zone 1**	**zone 2**	**zone 3**	**zone 4**	**zone 5**	**zone 6**	**zone 7**	**zone 8**	**zone 9**	**zone 10**	**zone 11**	**zone 12**	**zone 13**	**zone 14**	**zone 15**	**zone 16**
	Elevation (NGVD) Feet															
1-Jan	80	90	149	152	156.7	162.3	164.4	166	167	168	170	171	172	**173**	176	190
1-Feb	80	90	149	152	160	162.3	164.4	166	167	168	170	171	172	**173**	176	190
15-Mar	80	90	149	150.5	156	162.3	164.4	166	167	168	170	171	172	**173**	176	190
21-Mar	80	90	149	150.4	156.4	162.3	164.4	166	167	168	170	171	172	**173**	176	190
22-Mar	80	90	149	150.4	156.4	162.3	164.4	166	167	168	170	171	172	**173**	176	190
1-Apr	80	90	149	150	157	162.3	164.4	166	167	168	170	171	172	**173**	176	190
1-May	80	90	149	150	159	163	164.4	166	167	168	170	171	172	**173**	176	190
15-Jn	80	90	149	150	162	163	164.4	166	167	168	170	171	172	**173**	176	190
1-Aug	80	90	149	153	159	163	164.4	166	167	168	170	171	172	**173**	176	190
2-Aug	80	90	149	153	159	163	164.4	166	167	168	170	171	172	**173**	176	190
15-Aug	80	90	149	154	158	163	164.4	166	167	168	170	171	172	**173**	176	190
1-Sep	80	90	149	153.5	157	163	164.4	166	167	168	170	171	172	**173**	176	190
1-Oct	80	90	149	152	155	162.3	164.4	166	167	168	170	171	172	**173**	176	190
15-Nov	80	90	149	150	152	162.3	164.4	166	167	168	170	171	172	**173**	176	190
31-Dec	80	90	149	152	156.7	162.3	164.4	166	167	168	170	171	172	**173**	176	190

Table 8.6: B.A. Steinhagen Operating Level Table

zone 1	zone 2	zone 3	zone 4	zone 5	zone 6	zone 7	zone 8	**zone 9**	zone 10	zone 11	zone 12	zone 13	zone 14	zone 15	zone 16
Elevation (NGVD)-Feet															
50	52	52	52	81	81.5	82	82.5	**83**	83.5	83.5	83.5	83.5	83.5	84	95

Table 8.7: B.A. Steinhagen Level Regulation Table

Zone 5	Zone 7	Zone 9	Zone 10	Zone 14	Zone 16
Discharge (cfs)					
1,500	3,000	20,000	70,000	73,200	80,000

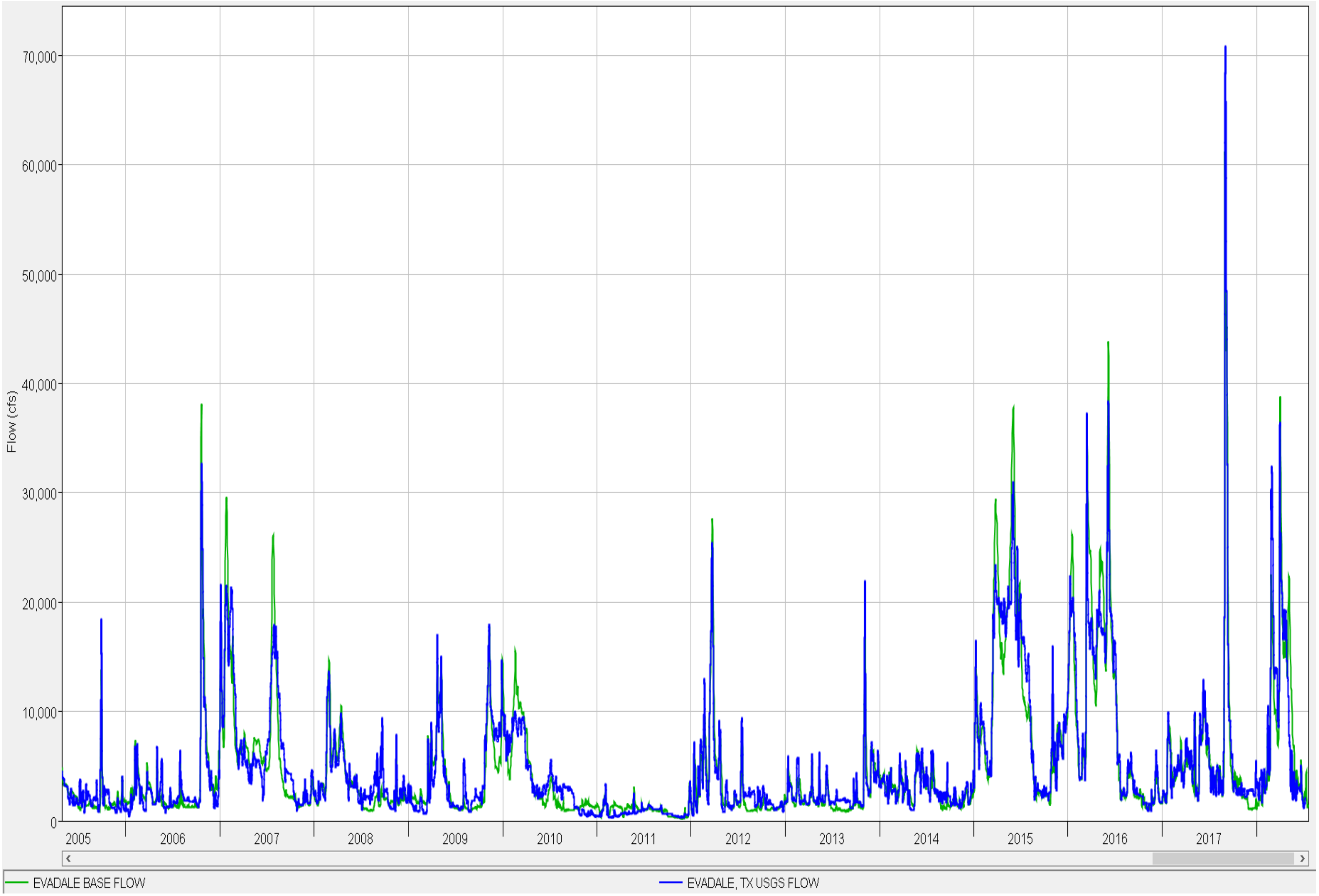


Figure 8.6: RiverWare Model Results Comparison for USGS Streamgage Station 08041000 Neches River at Evadale, Tex.

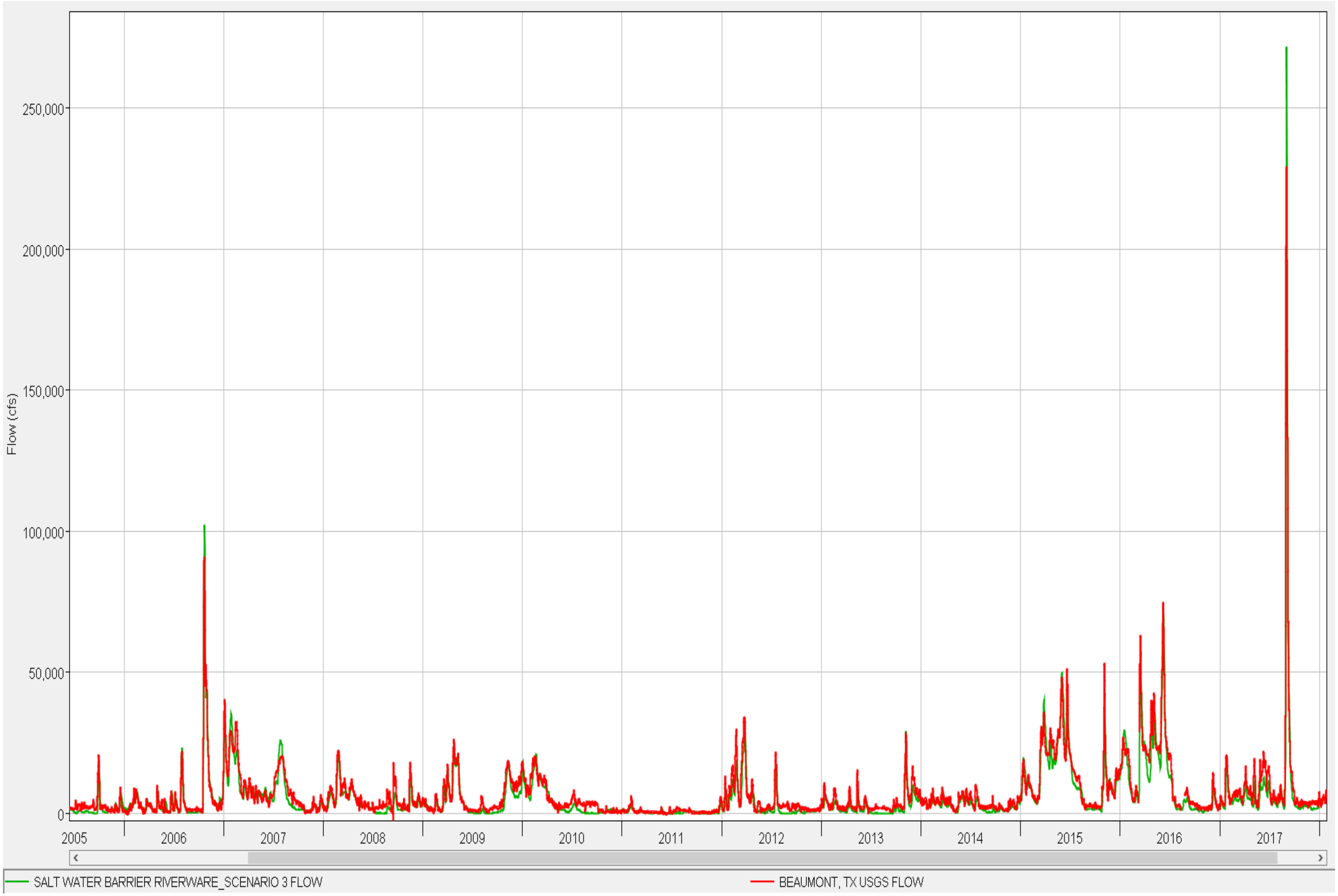


Figure 8.7: RiverWare Model Results Comparison for USGS Streamgage Station 08041780 Neches River Saltwater Barrier at Beaumont, Tex.

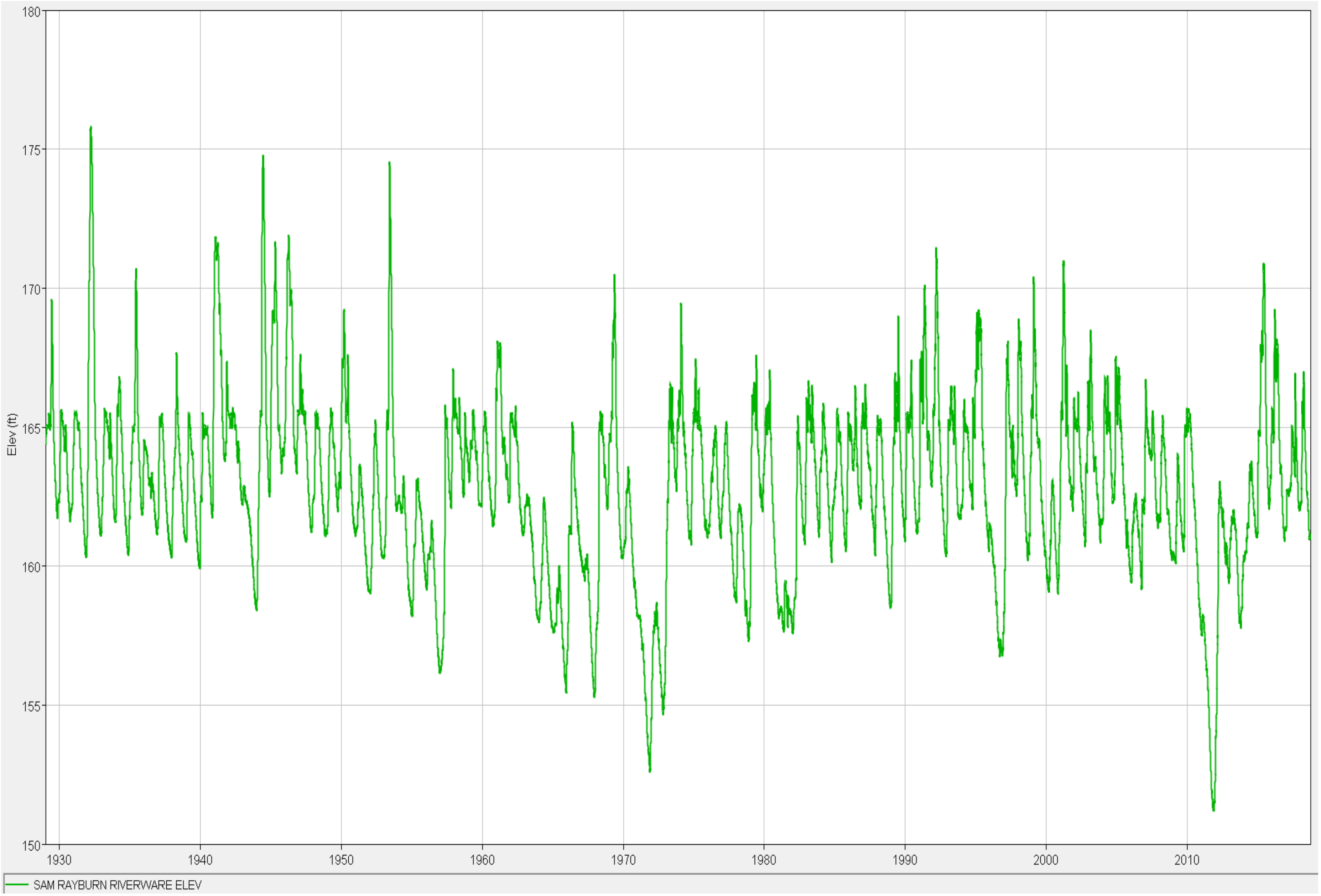


Figure 8.8: Simulated POR Results for B.A. Steinhagen's Pool Elevation

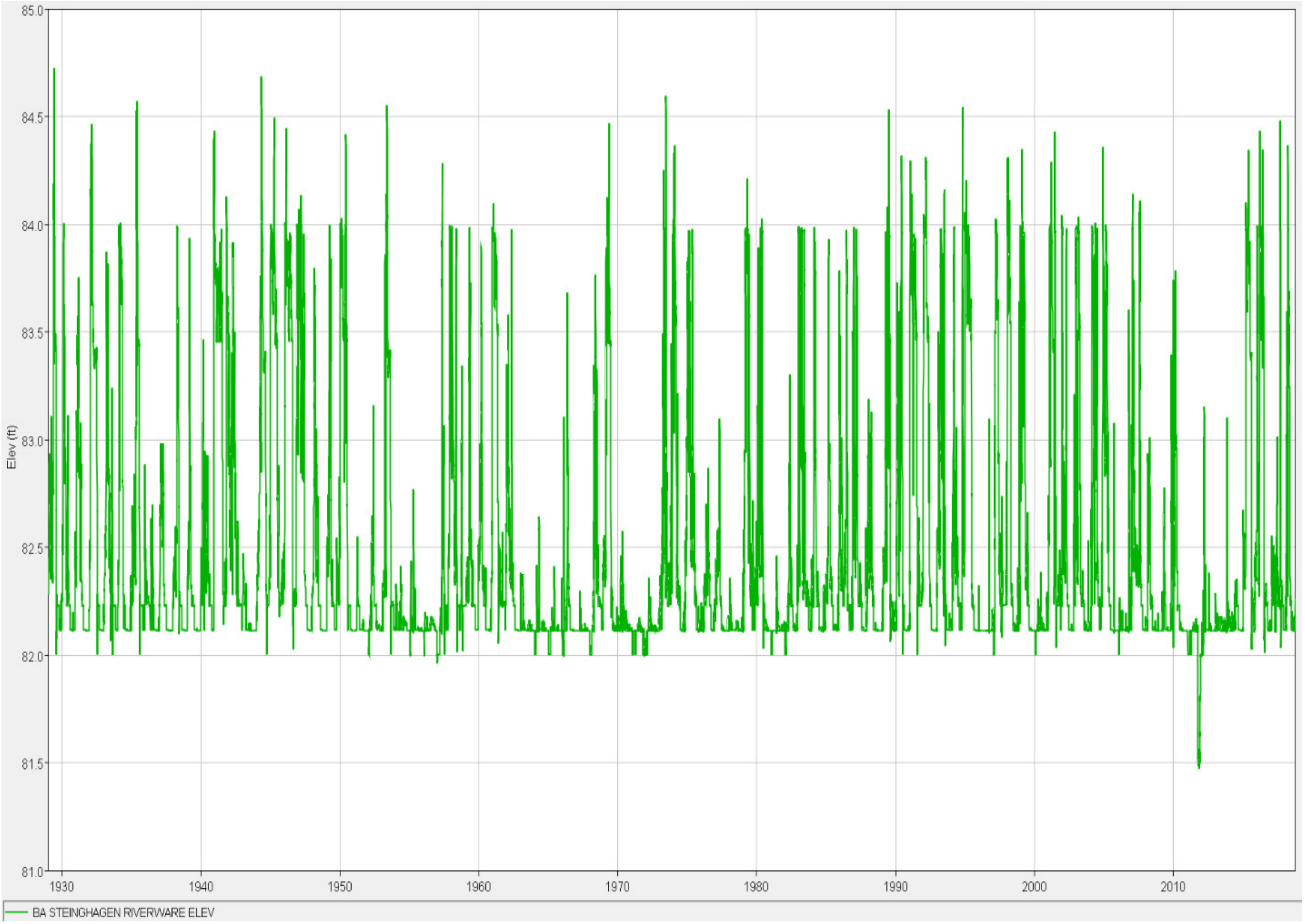


Figure 8.9: Simulated POR Results for Sam Rayburn's Pool Elevation

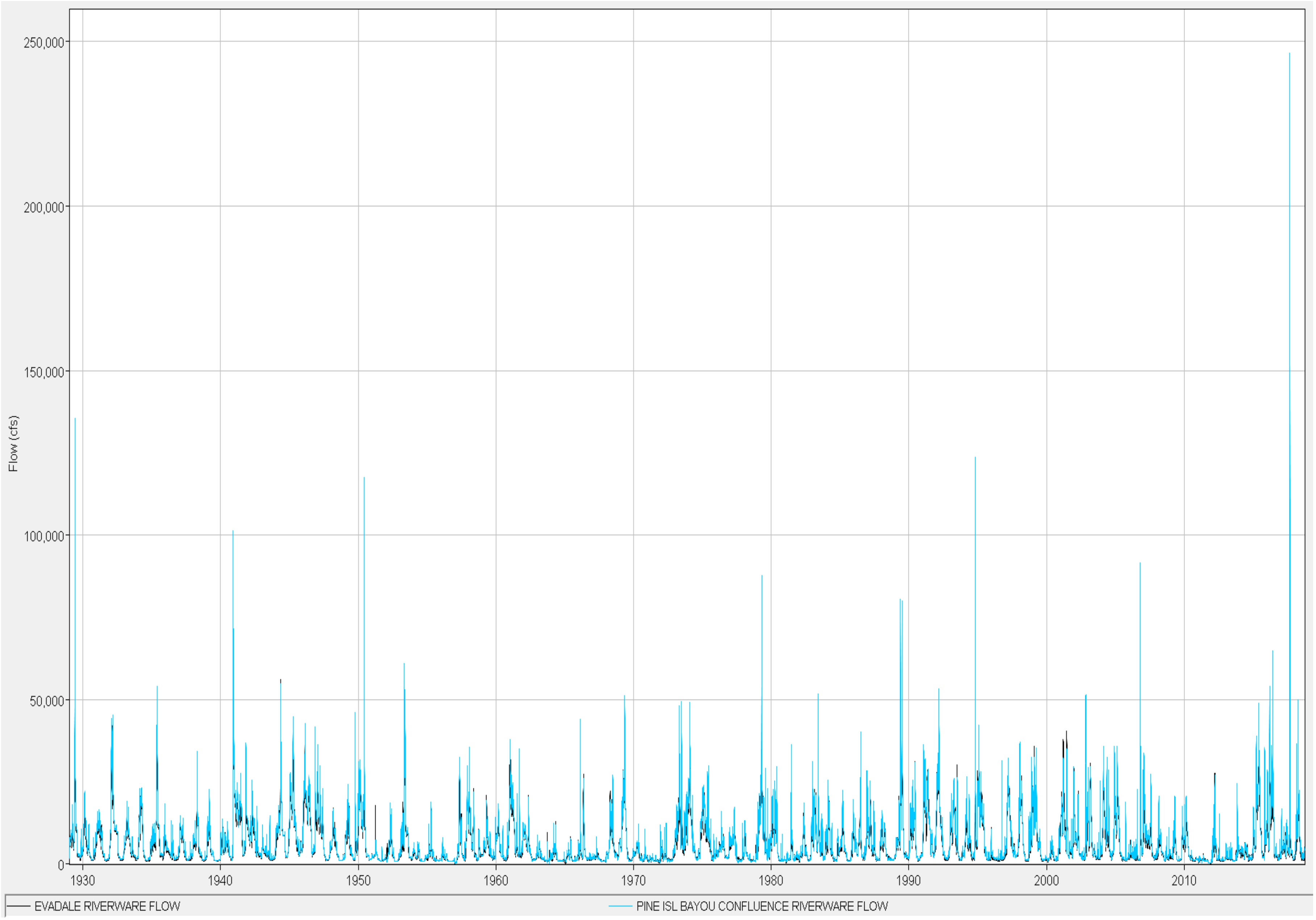


Figure 8.10: Simulated POR Results for the Neches River at Evadale and the SWB

8.9 CONVERSION OF DAILY DISCHARGES TO PEAK INSTANTANEOUS DISCHARGES

While the RiverWare model runs on a daily time step, peak instantaneous discharges are needed for flood flow frequency analysis. Therefore, a comparison of USGS observed instantaneous peak discharges and the corresponding USGS daily average discharges was made in order to convert the RiverWare daily discharges to an equivalent peak instantaneous discharge for each location of interest. A plot of instantaneous peak discharges versus USGS daily average peak discharges were made, and a regression equation was fit to each dataset. The regression equations were then applied to the daily peak discharges from RiverWare to transform them into instantaneous peak discharges. Figures 8.11 through 8.14 illustrate the corresponding relationship between datasets used to generate peaking factors to transform peak discharges.

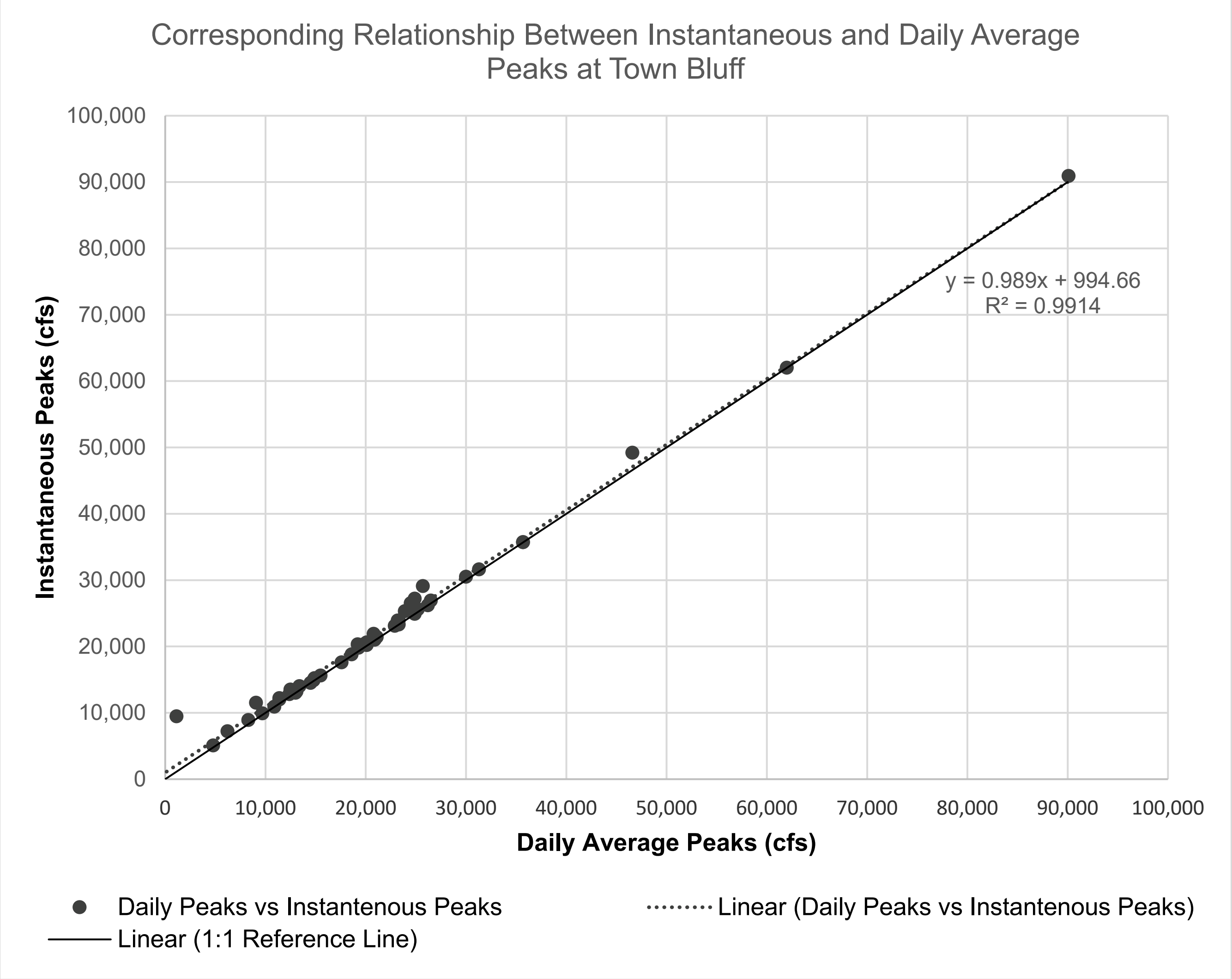


Figure 8.11: Instantaneous vs. Daily Average Peak Discharges for USGS Streamgaging Station 08040600 Neches River near Town Bluff, Tex.

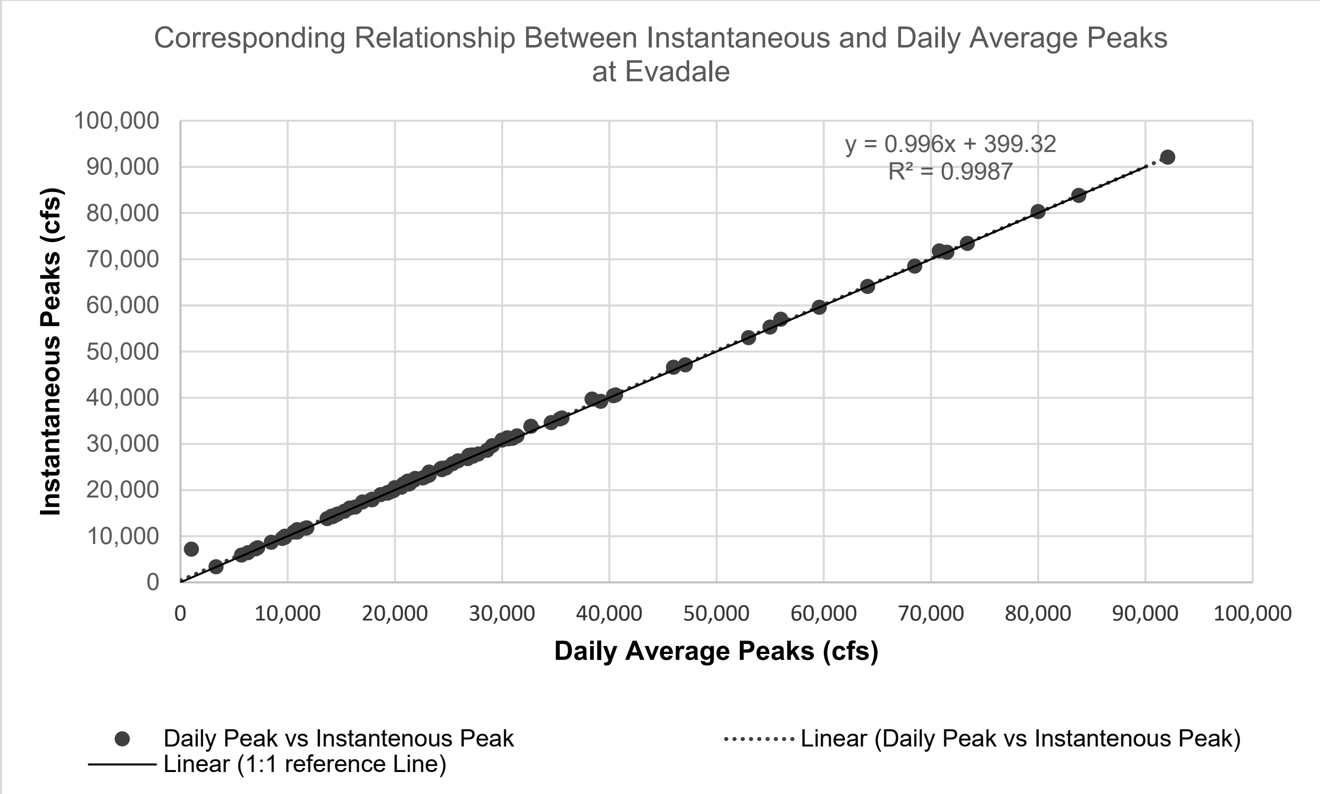


Figure 8.12: Instantaneous vs. Daily Average Peak Discharges for USGS Streamgaging Station 08041000 Neches River at Evadale, Tex.

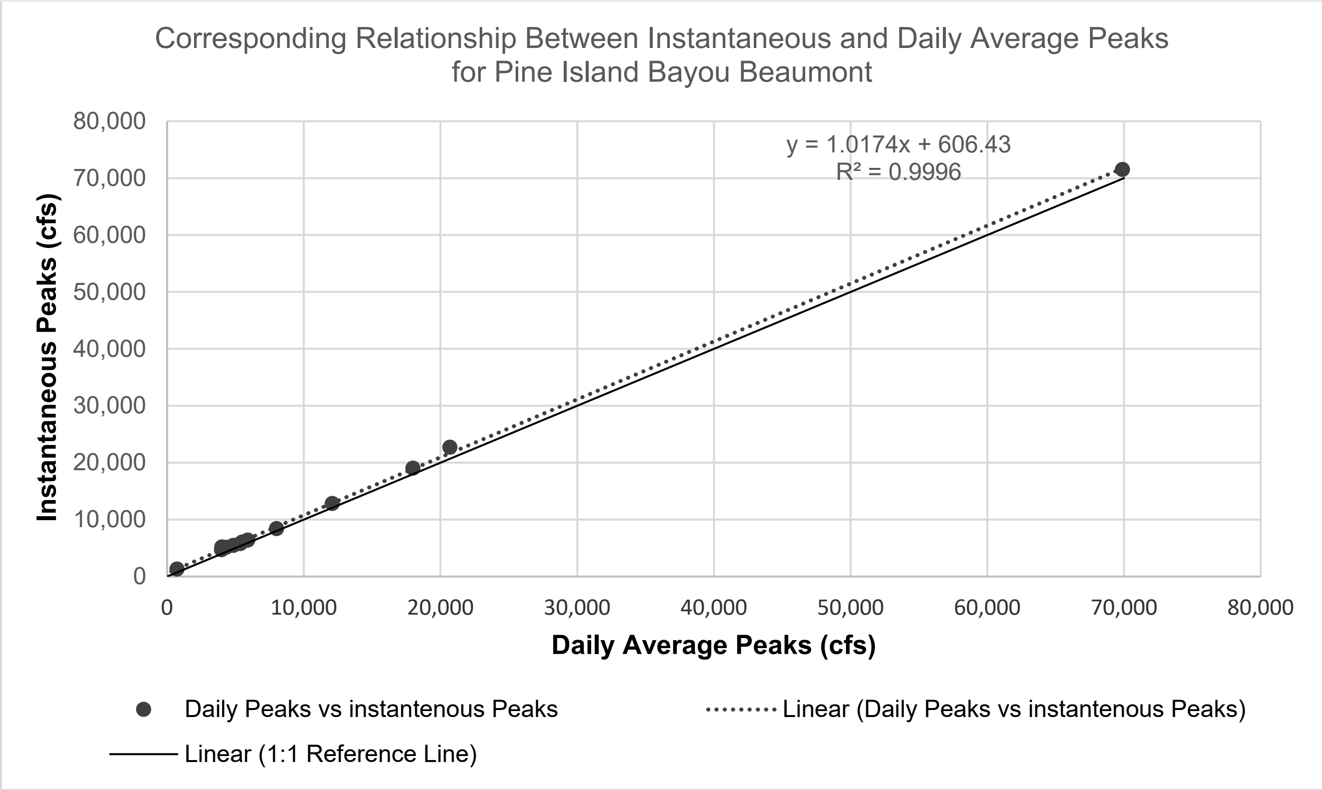


Figure 8.13: Instantaneous vs. Daily Average Peak Discharges for USGS Streamgaging Station 08041749 Pine Island Bayou above BI Pump at Beaumont, Tex.

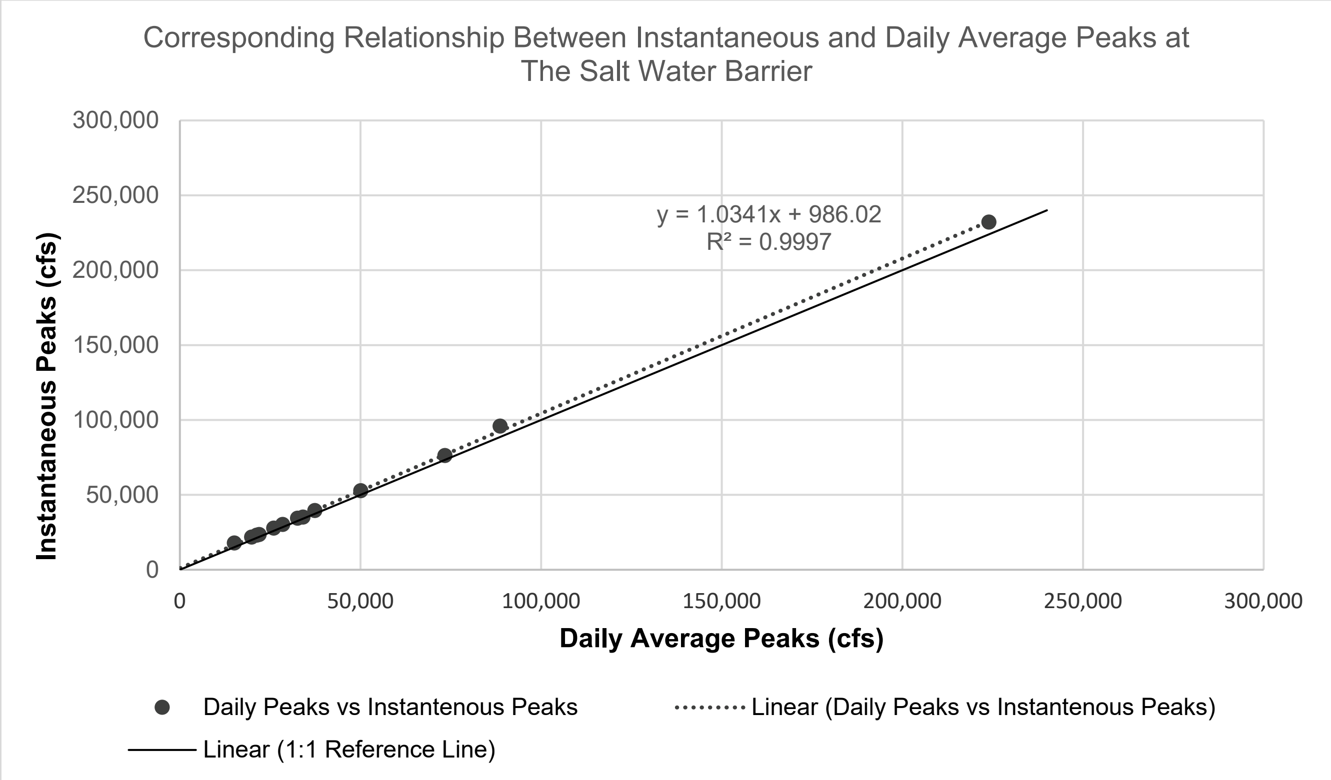


Figure 8.14: Instantaneous vs. Daily Average Peak Discharges for USGS Streamgaging Station 08041780 Neches Rv Saltwater Barrier at Beaumont, Tex.

Further analyses were made to compare the simulated peak discharges with the USGS observed annual peak discharges. A set of peak discharges extracted from the RiverWare model output was compared against the observed peak discharges of the same exact date of when the observed peak discharges occurred. This type of analysis helps increase confidence in using the extended discharge peaks used to generate discharge frequency peaks. Additional information on these comparisons can be found in Appendix D.

The finalized discharge peaks, which were used to develop the discharge frequency peaks, were a compilation of the USGS instantaneous observed peak discharges, downloaded from the USGS National Water Information System (NWIS) database (USGS, 2018) and the simulated RiverWare peak discharges. In general, RiverWare peak discharges were used instead of USGS peak discharges for the following circumstances: (1) the USGS peak discharges were missing, or (2) the period prior to completion of Sam Rayburn Reservoir.

8.10 STREAMGAGE DATA AND STATISTICAL FLOOD FLOW FREQUENCY RESULTS

For the statistical analysis of the RiverWare modeling results, the simulated instantaneous peak discharge was analyzed for three USGS streamgaging stations in the RiverWare model: 08040600 Neches River near Town Bluff, Tex., 08041000 Neches River at Evadale, Tex., and 08041780 Neches River Saltwater Barrier at Beaumont, Tex. These correspond to the RiverWare model control point elements of B.A. Steinhagen Outflow, Evadale, and Saltwater Barrier, respectively (Table 8.1). A peaking factor, described in detail in section 8.9, was applied to the RiverWare daily time-step data to convert the peak discharges to instantaneous peak discharges.

With the aim of providing the best available POR, the USGS observed peak discharge data were substituted for RiverWare simulated record when available. USGS observed peak discharge data are considered to be the most reliable of the two datasets because these data recorded actual events and are not simulated discharge. Simulated RiverWare data, however, supersedes this priority when the USGS record does not reflect the regulated watershed at the time of this analysis in 2018. Therefore, in most cases, the POR analyzed in this chapter consists of a combined record of USGS observed and RiverWare simulated peak discharge data. Henceforth, “observed record (or dataset)” refers to only the USGS observed record of peak discharge, whereas “simulated record (or dataset)” refers to the combined RiverWare and USGS peak discharge record. The details of each gage’s POR are described in each gage’s individual section below.

The flood flow frequency analysis was performed following the same methodology as is used in the analysis of the observed POR defined in Chapter 5. Bulletin 17C guidelines (England and others, 2018) were followed, although the usefulness of the expected moments algorithm (EMA) is limited in this analysis, and the sophisticated interpretation of historical peak discharges, thresholds, and so forth is not needed. This is because the combination of USGS and RiverWare peak data results in a fairly homogeneous dataset without these nonstandard forms of information. Flood flow frequency analyses were performed in the USACE Hydrologic Engineering Center’s Statistical Software Package (HEC-SSP), which is a software program designed to perform statistical analyses of hydrologic data including Bulletin17C frequency analyses (England and others, 2018; USACE, 2016). Two especially important options of the HEC-SSP software are the choice of low-outlier threshold and generalized skew and whether to incorporate such skew in the analyses in a weighting between the generalized skew and that computed using the site-specific data (USACE, 2016). Site-specific selection of skew and low-outlier thresholds are discussed in each gage’s individual writeup that follows in this section.

PeakFQ input must conform to specific data formatting requirements (Flynn and others, 2006), which means that constructing a synthetic data input file can be problematic and potentially lead to errors. USGS peak discharge data are available from the USGS NWIS database (USGS, 2018) in a format compatible with PeakFQ, but RiverWare does not provide this formatting option. Therefore, flow frequency analyses performed on RiverWare datasets were done in the USACE HEC-SSP software, which has flexible data input requirements (USACE, 2016). While the program interface might be slightly different than PeakFQ, the basic setup and methodology are the same, and when given identical input both programs will provide the same results. The final results of the simulated record flood flow frequency analyses in this chapter are summarized in Table 8.8.

08040600 Neches River near Town Bluff, Tex.

The POR used for the flood flow frequency analysis for USGS streamgaging station 08040600 Neches River near Town Bluff, Tex. (hereinafter referred to as the "Neches River near Town Bluff gage") was from 1929 through 2017 (USGS, 2018). RiverWare simulated annual peak discharge was substituted for USGS annual peak values prior to the impoundment of Sam Rayburn Reservoir in 1965. The largest peak in the peak discharge dataset, combining the RiverWare simulated peak discharge prior to 1965 and the USGS peak discharge after 1965 for the Neches River near Town Bluff gage, is the 2017 peak discharge of 91,000 cfs, which was a result of Hurricane Harvey. The data as set up for statistical frequency analysis are shown in Figure 8.15. The flood flow frequency for the Neches River near Town Bluff simulated dataset is shown in Figure 8.16. The low-outlier threshold was set by HEC-SSP at 6,342 cfs, and the skew was set to station skew (no external input). This low-outlier threshold is different than the one set for the Neches River near Town Bluff gage dataset in Appendix A because the inclusion of RiverWare data in this analysis results in a different set of ordered events (Figure 8.16) and, as a result, a different flood flow frequency analysis. This difference is applicable for all streamgaging stations in this analysis.

A comparison of the simulated flood flow frequency analysis from this chapter and observed flood flow frequency curve from Appendix A is shown in Figure 8.17. The difference between the simulated and observed flood flow frequency curves in Figure 8.17 appears to be minimal. The Town Bluff gage is approximately 30 miles downstream from Sam Rayburn Dam, and water from the Angelina River mixes with water from the Neches River in B.A. Steinhagen Lake upstream from the gage, indicating that the reservoir's regulation likely has a subdued effect on peak discharge at the Town Bluff gage. B.A. Steinhagen Lake is not a flood control reservoir and has little effect on peak discharge at the Town Bluff gage despite the gage's proximity to the reservoir.

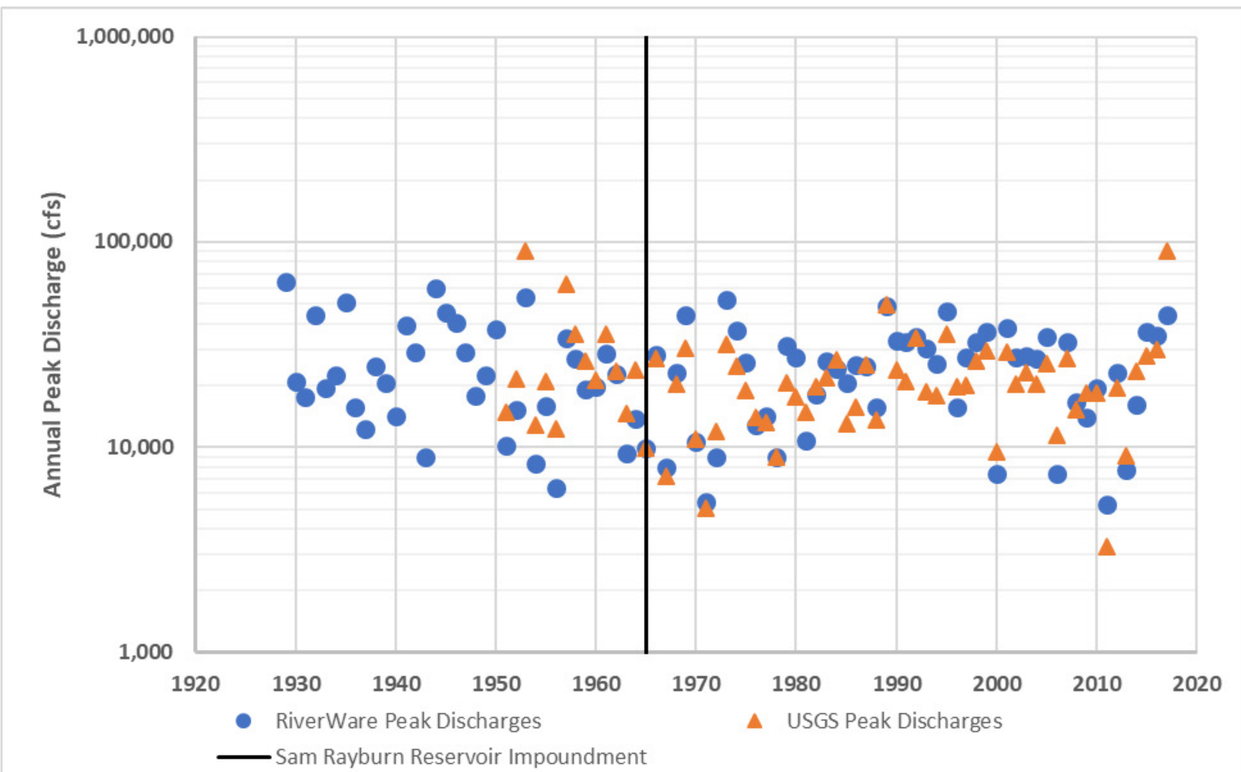
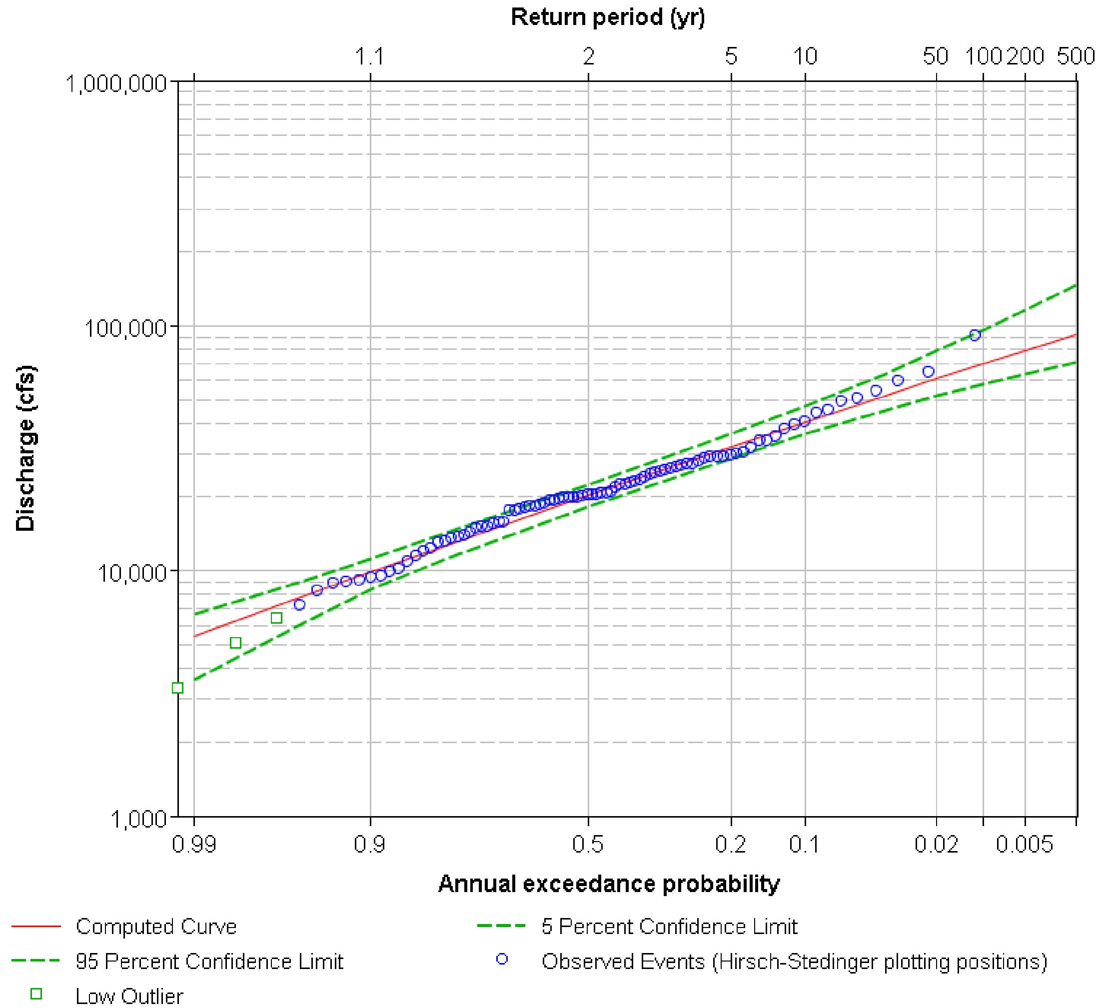


Figure 8.15: Simulated RiverWare and Observed USGS Annual Peak Discharges for USGS Streamgaging Station 08040600 Neches River near Town Bluff, Tex.



Note: This figure is a screenshot image obtained from U.S. Army Corp of Engineers Hydrologic Engineering Center's Statistical Software Package (HEC-SSP)

Observed events refer to the combined simulated record.

Hirsch-Stedinger plotting positions are described in Hirsch and Stedinger, 1987.

Figure 8.16: Simulated Flood Flow Frequency using log-Pearson Type III Distribution for USGS Streamgaging Station 08040600 Neches River near Town Bluff, Tex.

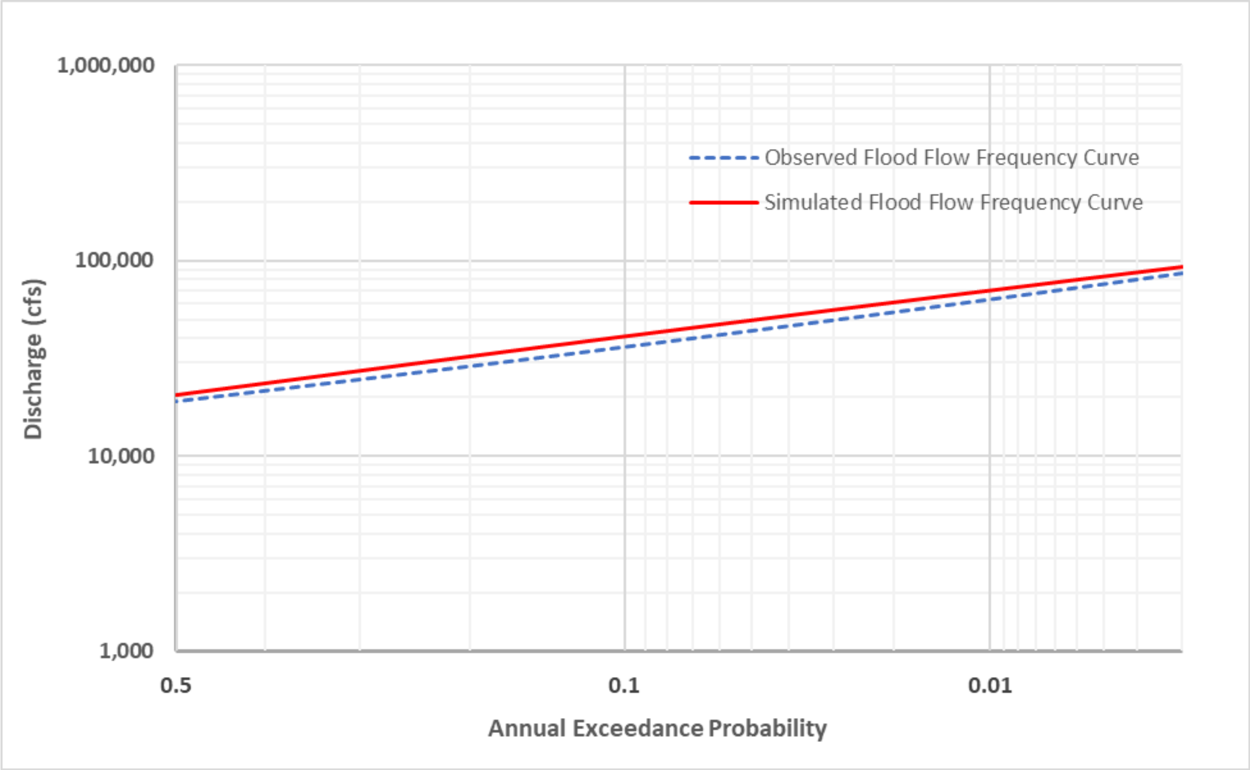


Figure 8.17: Comparison of Flood Flow Frequency Curves for the Observed and Simulated Datasets for USGS Streamgaging Station 08040600 Neches River near Town Bluff, Tex.

08041000 Neches River at Evadale, Tex.

The POR used for the flood flow frequency analysis for Neches River at Evadale gage is from 1929 through 2017 (USGS, 2018). RiverWare simulated annual peak discharge data was substituted for USGS annual peak values prior to the impoundment of Sam Rayburn Reservoir in 1965. The largest peak in the peak discharge dataset, combining the RiverWare simulated peak discharge prior to 1965 and the USGS peak discharge after 1965, for the location is the 2017 peak discharge of 71,300 cfs, which was a result of Hurricane Harvey. The data as set up for statistical frequency analysis are shown in Figure 8.18. The flood flow frequency for the Neches River at Evadale simulated dataset is shown in Figure 8.19. The low-outlier threshold was set by HEC-SSP at 9,315 cfs, and the skew was weighted by a regional skew of 0.3 (regional skew mean square error, MSE, 0.38).

A comparison of the simulated flood frequency analysis from this chapter and observed flood flow frequency curve from Appendix A is shown in Figure 8.20. The Neches River at Evadale gage is far enough downstream from Sam Rayburn Reservoir that the reservoir's regulation has a subdued effect on the location's peak discharge. B.A. Steinhagen Lake is not a flood control reservoir and has only limited effects on peak flow at the Evadale gage. Therefore, the difference between the simulated and observed flood frequency curves is minimal.

Even though the gage at Evadale is farther downstream than the gage at Town Bluff, the 500-year estimate at Town Bluff is greater (92,300 cfs simulated; 86,700 cfs observed), and the 100-year and shorter recurrence interval estimates are nearly identical (69,800 cfs simulated; 73,000 cfs observed). This is because the increase in contributing drainage area between the two gages is only 377 square miles, most of which are heavily forested bottomland, which would be expected to attenuate flows. Additionally, no major tributary inflow exists between the two gages.

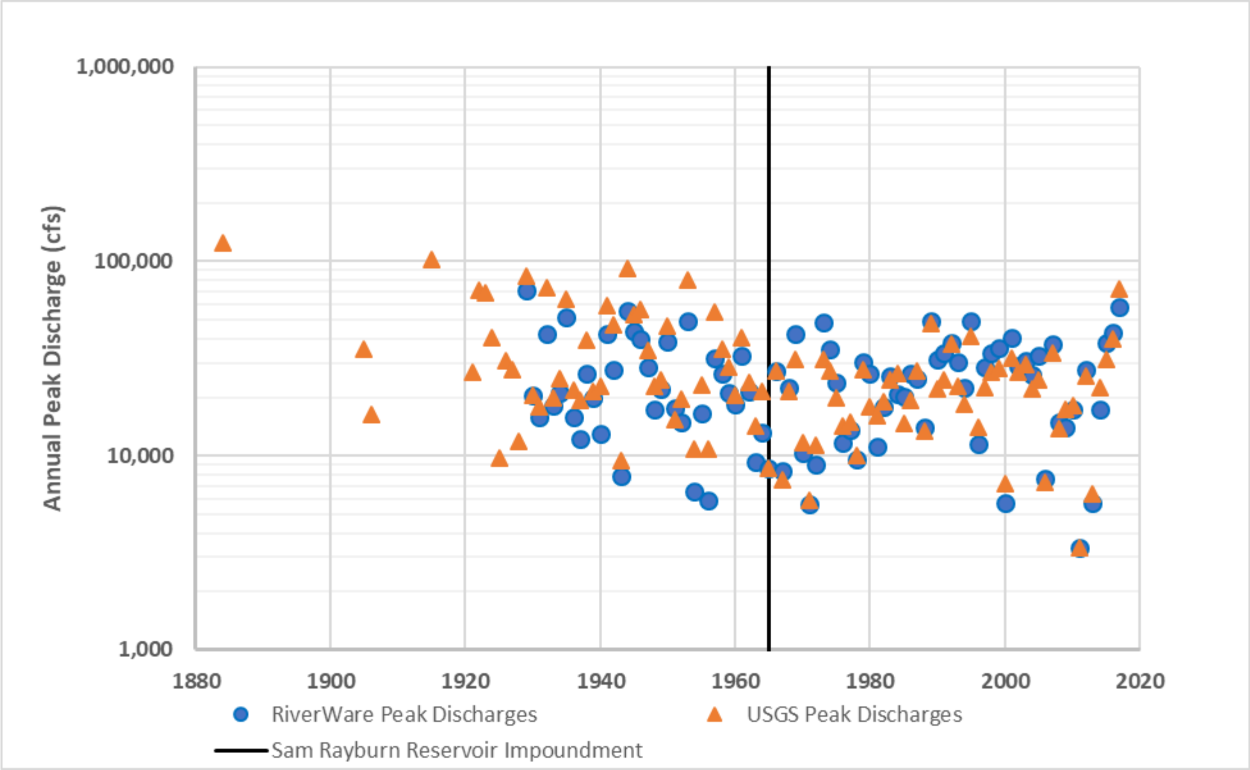
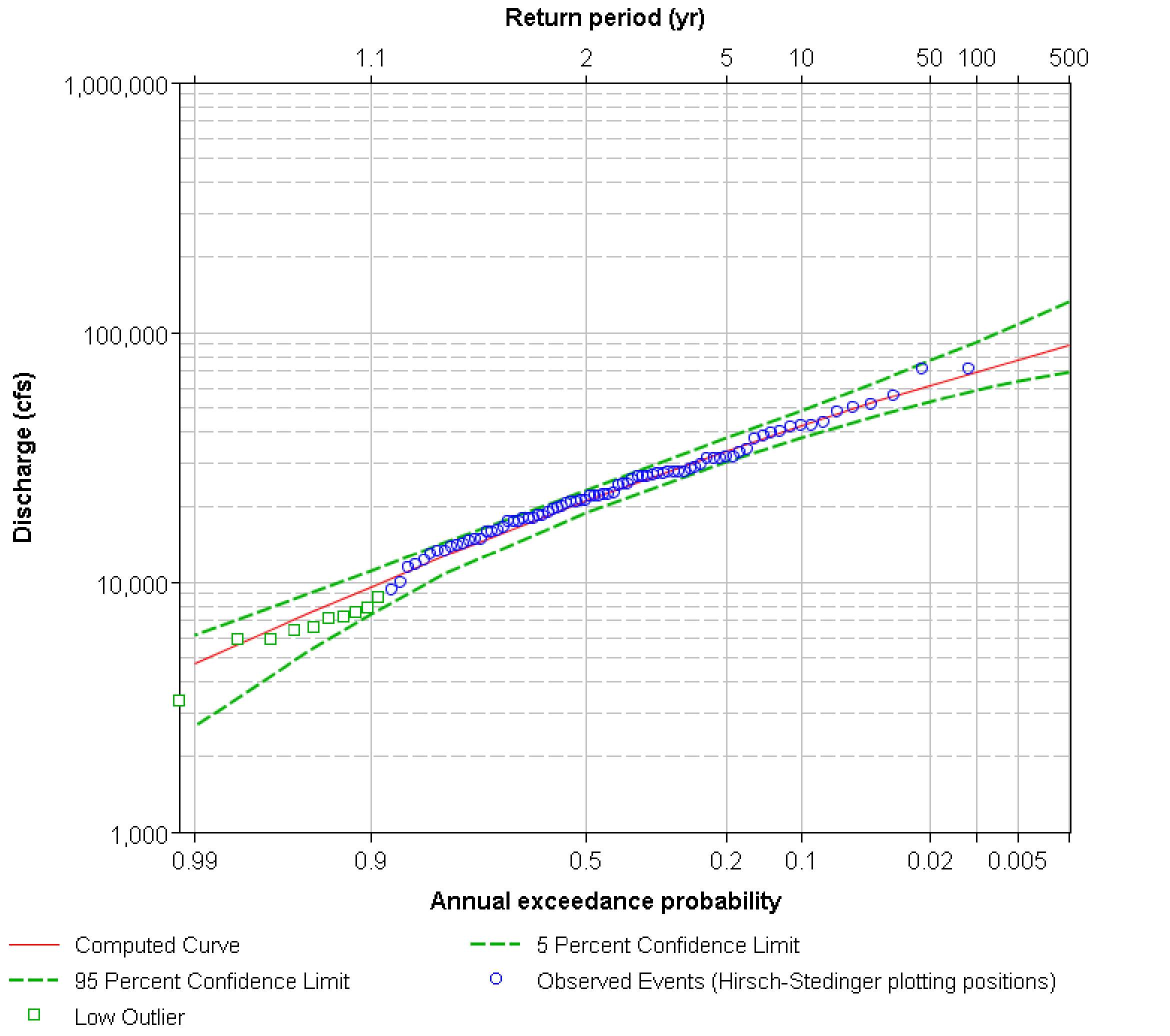


Figure 8.18: Simulated RiverWare and Observed USGS Annual Peak Discharges for USGS Streamgaging Station 08041000 Neches River at Evadale, Tex.



Note: This figure is a screenshot image obtained from U.S. Army Corp of Engineers Hydrologic Engineering Center's Statistical Software Package (HEC-SSP)

Observed events refer to the combined simulated record.

Hirsch-Stedinger plotting positions are described in Hirsch and Stedinger, 1987.

Figure 8.19: Simulated Flood Flow Frequency using log-Pearson Type III Distribution for USGS Streamgaging Station 08041000 Neches River at Evadale, Tex.

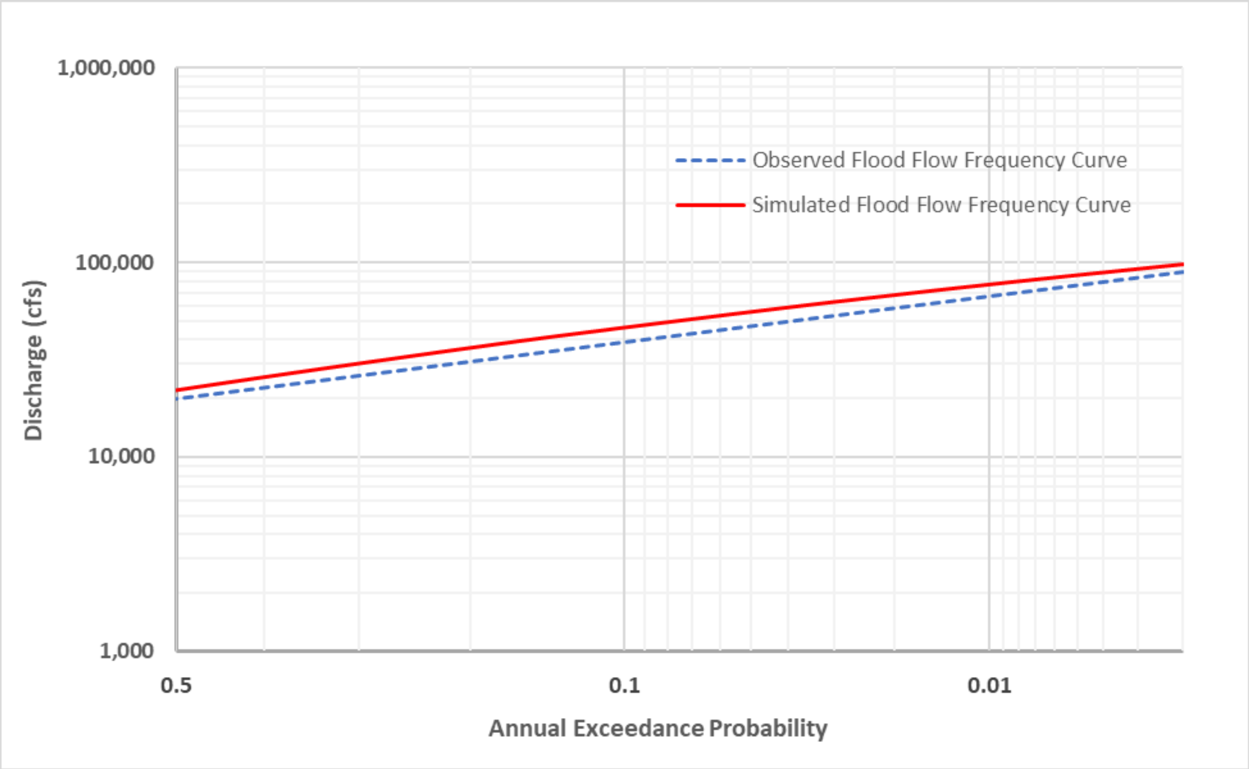


Figure 8.20: Comparison of Flood Flow Frequency Curves for the Observed and Simulated Datasets for USGS Streamgaging Station 08041000 Neches River at Evadale, Tex.

08041780 Neches River Saltwater Barrier at Beaumont, Tex.

The POR used for the flood flow frequency analysis for Neches River Saltwater Barrier gage is from 1929 through 2017, excluding 2011 (the annual peak discharge for 2011 is not available). USGS peak discharge data were substituted for RiverWare simulated data when available during 2004–2017. The largest peak in the peak discharge dataset, combining the RiverWare simulated peak discharge prior to 2004 and the USGS peak discharge after 2004, for the location is the 2017 peak discharge of 232,000 cfs, which was a result of Hurricane Harvey. The data as set up for statistical frequency analysis are shown in Figure 8.21. The flood flow frequency for the Neches River Saltwater Barrier simulated dataset is shown in Figure 8.22. A low outlier threshold of 12,000 cfs was manually set, and the skew was set to station skew (no external input).

A comparison of the simulated flood frequency analysis from this chapter and observed flood flow frequency curve from Appendix A is shown in Figure 8.23. The difference between the simulated and observed flood frequency curves are a result of this difference in peak discharge record length. The simulated RiverWare dataset adds 75 years of additional peak discharge record.

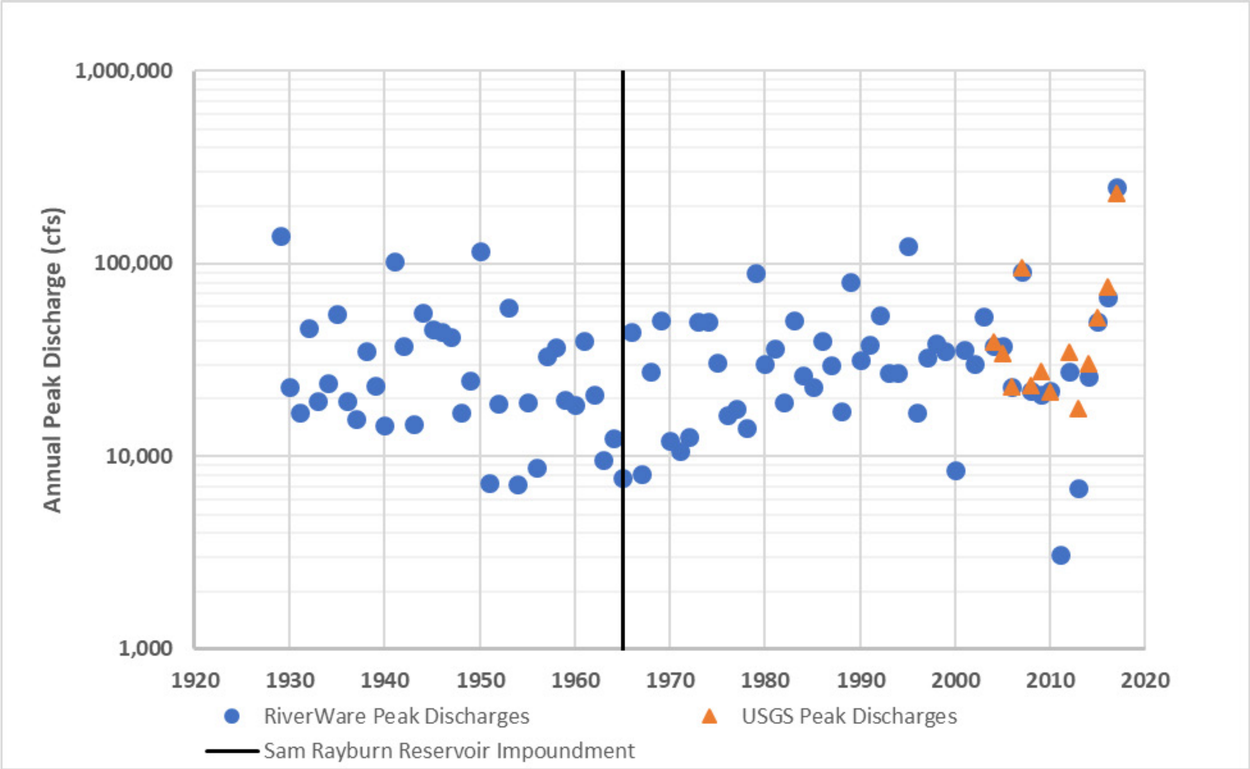
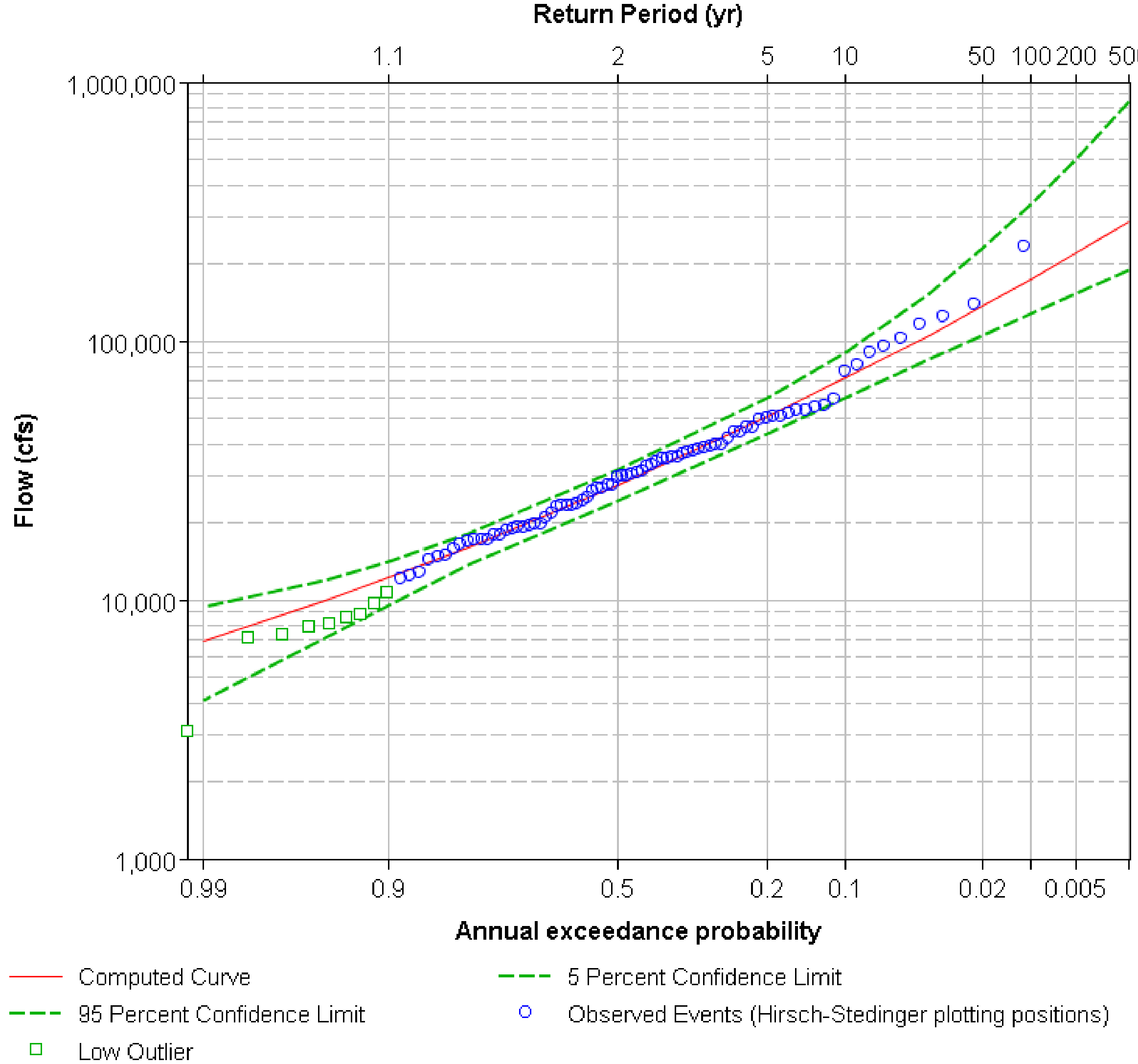


Figure 8.21: RiverWare and USGS Annual Peak Discharges for Streamgaging Station 08041780 Neches River Saltwater Barrier at Beaumont, Tex.



Note: This figure is a screenshot image obtained from U.S. Army Corp of Engineers Hydrologic Engineering Center's Statistical Software Package (HEC-SSP)

Observed events refer to the combined simulated record.

Hirsch-Stedinger plotting positions are described in Hirsch and Stedinger, 1987.

Figure 8.22: Simulated Flood Flow Frequency Determined by using a log-Pearson Type III Distribution for USGS Streamgaging Station 08041780 Neches River Saltwater Barrier at Beaumont, Tex.

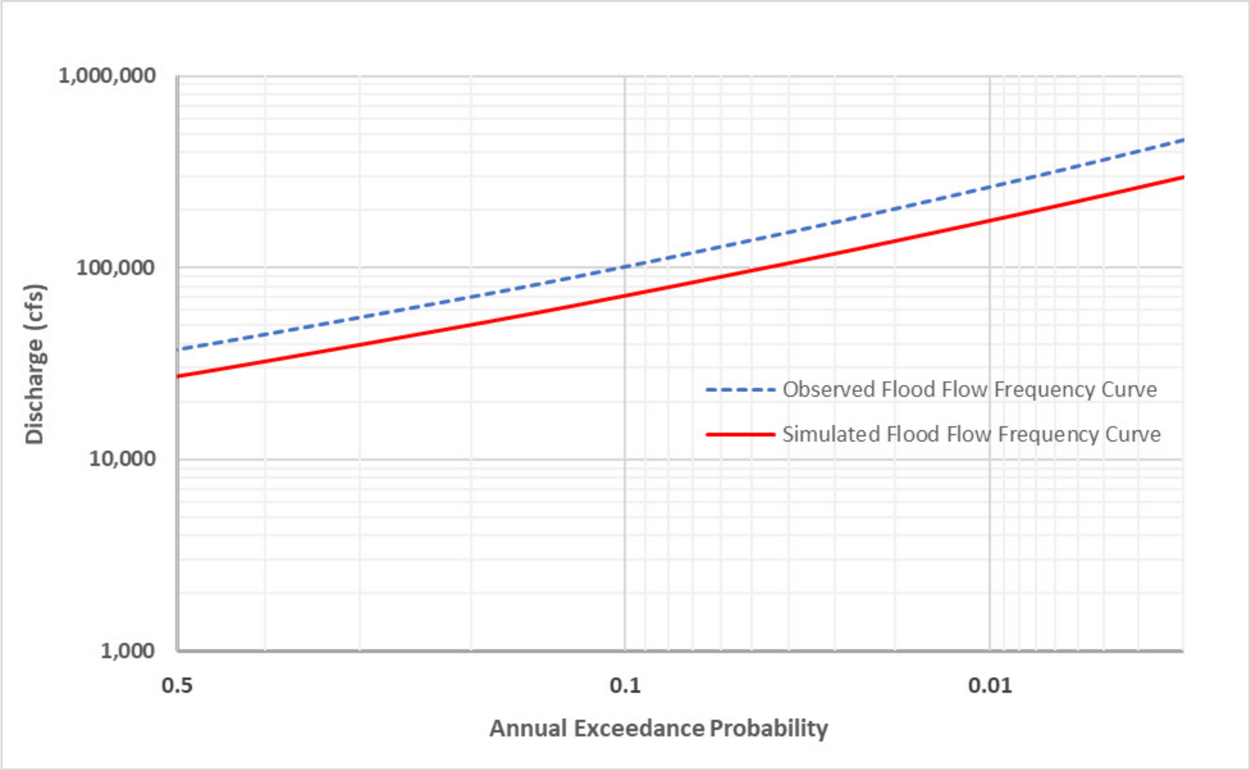


Figure 8.23: Comparison of Flood Flow Frequency Curves for the Observed and Simulated Datasets for USGS Streamgaging Station 08041780 Neches River Saltwater Barrier at Beaumont, Tex.

Table 8.8: Statistically Estimated Annual Flood Frequency Results and Confidence Intervals Simulated for Three U.S. Geological Survey Streamgaging Stations in the Neches River Basin, Texas, determined by Hydrologic Engineering Center-Statistical Software Package Software

[USGS, U.S. Geological Survey; cfs, cubic feet per second; %, percent; CI, confidence limit; Note, table contents derived from Hydrologic Engineering Center-Statistical Software Package software output (USACE, 2016). The estimates are of primary interest and are accentuated using a bold typeface.]

USGS Station number and name	Flood flow frequency by corresponding average return period (recurrence interval) in years							
	2 year (cfs)	5 year (cfs)	10 year (cfs)	25 year (cfs)	50 year (cfs)	100 year (cfs)	200 year (cfs)	500 year (cfs)
08040600 Neches River near Town Bluff, Tex.								
Lower 95%-CI	18,300	28,900	36,100	45,200	51,600	57,700	63,500	70,700
Estimate	**20,400**	**32,100**	**40,600**	**51,900**	**60,700**	**69,800**	**79,200**	**92,300**
Upper 95%-CI	22,600	36,100	46,900	63,700	78,600	95,700	115,000	146,000
08041000 Neches River at Evadale, Tex.								
Lower 95%-CI	18,900	30,200	37,700	46,700	52,800	58,500	63,600	69,800
Estimate	**21,100**	**33,600**	**42,200**	**53,300**	**61,500**	**69,800**	**78,000**	**88,900**
Upper 95%-CI	23,500	37,700	48,400	64,000	77,200	91,900	108,000	133,000
08041780 Neches River Saltwater Barrier at Beaumont, Tex.								
Lower 95%-CI	24,200	43,900	60,600	85,100	106,000	128,000	152,000	187,000
Estimate	**27,700**	**50,900**	**71,800**	**105,000**	**137,000**	**174,000**	**217,000**	**288,000**
Upper 95%-CI	31,700	60,500	90,600	153,000	228,000	338,000	501,000	839,000

9 Reservoir Analyses

9.1 INTRODUCTION

This section of the report describes the methods used to update the pool frequency curves for the Neches River Basin Reservoir projects. The reservoir projects that have been analyzed for this section are Sam Rayburn and B.A. Steinhagen. The projects are operated by the United States Army Corps of Engineers (USACE). The frequency curves were developed to represent the current reservoir control plan and watershed conditions (as of 2018). A frequency analysis is a statistical method of prediction that consists of studying past events that are characteristic of a particular hydrology process in order to determine the probabilities of occurrence of these events in the future. A Stage-Frequency curve estimates the annual chance of exceedance (ACE) for reservoir pool elevations. For example, if a reservoir pool at the spillway crest has an ACE of 1/50 (1 in 50 years on average), then the reservoir has a 2% chance of the reservoir pool elevation equaling or exceeding the spillway crest elevation in any given year. The stage-frequency curve can be determined using empirical (observed or measured) data; however, the reservoir pool elevations associated with 1% ACE (100-year) or 0.2% ACE (500-year) occurrence are typically beyond the observed reservoir pool elevation period of record (POR). Models serve the purpose of extrapolating reservoir pool elevation frequencies beyond the observed record.

For the presented study, the pool frequency curves representing current conditions were developed to evaluate the Neches River Basin projects' pool elevations resulting from the 50% ACE (2-year) to 0.2% ACE (500-year) events. This study incorporates available reservoir inflow (historical peaks – 2018) and pool data (historical peaks – 2019) into statistical software and applies statistical methods to estimate the n-day critical inflow duration and simulate inflow and elevation period of record for each project. The historical peaks may be observed and recorded by local residents or seen as water marks on bridge piers or tree trunks; those water elevation marks can be translated into peak discharge values via the use of models or by extrapolating rating curves or extrapolation of observed data points. For each project, the Hydrologic Engineering Center-Statistical Software Package (HEC-SSP) was used to compute volume duration frequency curves from the annual maximum peak reservoir inflows. An empirical pool frequency curve was developed from the available reservoir pool Annual Maximum Series (AMS). An event based stochastic Monte Carlo simulation model, Risk Management Center-Reservoir Frequency Analysis (RMC-RFA), was used to extrapolate the pool frequency curve beyond the limits of empirical pool frequency curve. RiverWare was used to develop a current condition POR for reservoir inflows and elevations. The AMS results derived from RiverWare was used to create the empirical pool frequency curve. The empirical stage-frequency curve was used to validate RFA model simulation results. The results showed adequate validation to the upper tail end of the empirical pool frequency curves and is believed to be a reasonable extrapolation for frequency of rare pool events.

No pool elevation frequency estimates are available on the effective FEMA Flood Insurance Raye Maps (FIRMs) to compare to the results documented in this chapter for Sam Rayburn and B.A. Steinhagen. However, some previous pool frequency estimates (Table 9.1) were made by the Water Management Section staff of Fort Worth District during Periodic Assessments (PA) as part of the USACE dam safety program. In this chapter, primary emphasis was put on accurately capturing the 1% ACE (100-year) and 0.2% ACE (500-year) events by utilizing the RMC-RFA program through WY 2018 for each project.

Table 9.1: Previous USACE 2009 Pool Elevation Frequency Estimates for Neches River Basin Projects

Project	USACE 2009 Pool Annual Chance of Exceedance (ACE%) /Return Interval (N-Year)					
	50% (2-yr)	20% (5-yr)	10% (10-yr)	4% (25-yr)	2% (50-yr)	1% (100-yr)
	Elevation (Feet) NGVD					
Sam Rayburn	166.7	172.1	173.3	175	176	176.5
B.A. Steinhagen	83.2	83.6	83.8	84	84.5	85

This chapter summarizes the Reservoir Analyses portion of the hydrologic analysis that was completed for the InFRM Watershed Hydrology Assessment of the Neches River Basin. Additional details on the analyses and results are available in Appendix E: Reservoir Analyses.

9.2 OBSERVED DATA

Table 9.2 and Figure 9.1 show the reservoir projects and the corresponding United State Geological Survey (USGS) gages used to develop the Neches River Basin reservoir inflows. In many instances, project inflow reads recording gage data from the nearest USGS gage upstream of the dam, especially if the project drainage area does not vary significantly from the nearest USGS gage. The nearest USGS gage rating curve can also be used to estimate the historical peak discharges for the projects. Detailed analyses for hydrology development using RiverWare can be found in the previous chapter of this report. The POR for Sam Rayburn and B.A. Steinhagen inflows were obtained from the RiverWare model for the Neches basin.

Table 9.2: USGS and USACE-SWD Observed Data

Location	Data Type (Units)	Source
Sam Rayburn Reservoir	Evaporation (inch per hour)	USACE-SWD database
B.A. Steinhagen Reservoir	Evaporation (inch per hour)	USACE-SWD database
Angelina River near Horger, TX	Discharge (cubic feet per second)	USGS 08039500*
Neches River near Town Bluff, TX	Discharge (cubic feet per second)	USGS 08040600
Neches River at Evadale, TX	Discharge (cubic feet per second)	USGS 08041000
Village Creek near Kountze, TX	Discharge (cubic feet per second)	USGS 08041500
Mud Creek near Jacksonville, TX	Discharge (cubic feet per second)	USGS 08034500
Neches River near Rockland, TX	Discharge (cubic feet per second)	USGS 08033500
Pine Island Bayou near Sour Lake, TX	Discharge (cubic feet per second)	USGS 08041700
Sam Rayburn Inflow	Discharge (cubic feet per second)	USACE-SWD database
B.A. Steinhagen Inflow	Discharge (cubic feet per second)	USACE-SWD database
Sam Rayburn gated discharge	Discharge (cubic feet per second)	USACE-SWD database
Sam Rayburn turbine release	Discharge (cubic feet per second)	USACE-SWD database
Sam Rayburn Pool	Elevation (NGVD-29 feet)	USACE-SWD database
B.A. Steinhagen Pool	Elevation (NGVD-29 feet)	USACE-SWD database

*Horger gage was discontinued in 1973. Gage was used to develop the 1915 historical inflow peak for Sam Rayburn

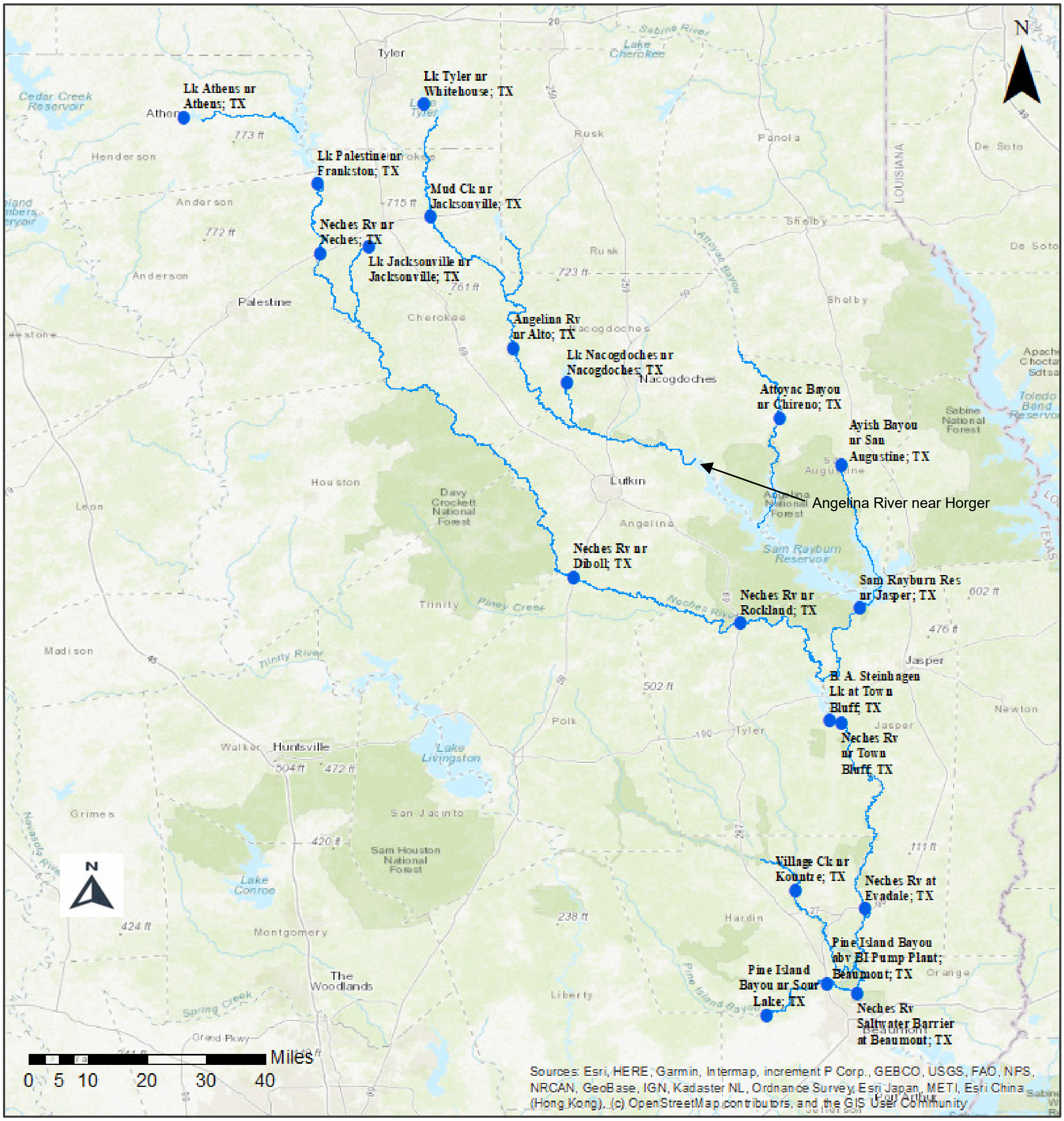


Figure 9.1: USGS Gage Locations in the Neches River Basin

9.3 METHODS OF ANALYSIS

9.3.1 Empirical Stage-Frequency

For the evaluation of a simulated reservoir pool frequency curve predictive capability, an empirical reservoir pool frequency curve is created. An empirical reservoir stage-frequency curve is constructed by ranking the observed/simulated peak annual reservoir stages, assigning the data a plotting position, and then plotting the data on probability paper using a plotting position formula. Many plotting position formulas can be used for the orientation of an empirical reservoir pool frequency curve, but a plotting position formula that is flexible and makes the fewest assumptions is preferred. The Weibull plotting position formula was selected. This formula is an unbiased estimator of expected exceedance probability for all distributions and is used to plot the series of peak annual reservoir stages. The formula for Weibull is:

$$P_i = i / (n + 1)$$

Where, i is the rank of the event, n is the sample size in years, and P_i is the exceedance probability for an event with rank i pool frequency

9.3.2 Volume-Sampling Approach

A common method for estimating a pool frequency curve for a dam is by volume-based sampling. In this method, a large number of flood events is generated using random sampling of flood volumes, the associated flood hydrographs are routed through the reservoir, and the peak reservoir elevation for each event is recorded.

The general workflow for a volume-based pool frequency analysis is as follows:

1. Choose a stage for the reservoir to begin the flood event
2. Choose an inflow flood hydrograph to scale
3. Sample a flood volume from the reservoir inflow frequency curve
4. Scale the selected flood hydrograph to match the sampled flood volume
5. Route the scaled flood hydrograph through the reservoir using an operations model
6. Record the peak stage that occurred during the event

For the stochastic model, RMC-RFA, choices made in steps 1-3 are made using random selection from a probability distribution. The choice is random in the sense that it occurs without pattern, but the relative frequency of the outcomes in the long term is defined by a probability distribution. Reservoir stages for starting the simulation come from a *pool duration curve*, which is a probability distribution for the elevation of the reservoir pool. They may be seasonally-based, in which case first the season of the flood event occurrence is selected at random, and then a starting stage is selected at random from the pool duration curve for that particular season. Sampled flood volumes come from the familiar flow frequency curve produced by fitting an analytical probability distribution to an AMS of river discharges. In the volume-based approach, instead of analyzing instantaneous peak discharge (as is typically the case in a Bulletin 17B/C-type analysis), the analysis is performed on a longer-duration volume (such as 5-, 10-, 30-, or even 60-day average discharge.)

When steps 1-6 are performed a large number of times (for example, 10,000 *samples*), the resulting peak stages are ranked and plotted, producing a stage-frequency curve for the reservoir. However, substantial uncertainty exists in several of the inputs to the model, especially the inflow frequency curve. To account for these uncertainties, steps 1-6 are performed a large number of times with different parameters for the inputs. The input parameters are varied across *realizations*, and for each realization, steps 1-6 are repeated over a large number of *samples.* Thus, the full simulation with uncertainty will contain a number of events equal to the number of realizations times the number of samples.

By varying parameters across realizations, the uncertainty in the probability of an event, for example reaching spillway crest elevation, can be better assessed. Each realization will produce an estimate of the probability of reaching this elevation based on the parameters used to drive the realization. Percentiles (for example the 5th and 95th percentiles) of these probabilities produce a confidence interval for the probability of reaching the spillway. If the mean probability of exceeding any stage is taken, then the result is the *expected frequency curve*, which is the single best estimate for the probability of exceeding a particular stage.

9.3.3 Risk Management Center - Reservoir Frequency Analysis (RMC-RFA)

RMC-RFA software was developed by the USACE Risk Management Center for use in dam safety risk assessments. It can produce a stage-frequency curve with confidence bounds using a stochastic model with the volume-sampling approach. The model functions best in situations where dam operations are relatively simple, especially when the spillway is not regulated using gates. A simplification of the operational rules is assumed through the use of an elevation-discharge table which is based on a combination of dam discharge structures and calibration to historical releases. Development of model inputs is aided by tools within the program that allow the user to estimate inputs, such as flood seasonality or pool duration curves, in a consistent and automated manner. Other inputs, such as the volume frequency curve or reservoir operations, are developed by the user independently.

9.4 DATA ANALYSIS AND MODEL INPUT

9.4.1 Inflow Hydrograph and Pool Stage

Estimate of daily average flows and pool elevations for the Neches River Basin projects were retrieved from the USACE water management database system for water year (WY) 1929 through WY 2018. Records prior to project construction were simulated using RiverWare. Pool records were extended to August of 2019 to capture high record pools at B.A. Steinhagen of 84.11 feet, which occurred on 04 January 2019, and pool record of 174.85 feet, which occurred on 28 January 2019 at Sam Rayburn. The 2019 pool records were included in the empirical frequency curves estimates. The Neches River Basin projects impoundment dates are shown in Table 9.3. RiverWare software mimics a watershed by modeling its features as linked objects, including storage or power reservoir objects, stream reach objects, groundwater storage objects, or diversion objects (see details in Section 8.1.3). In a simple model, these objects simulate basic hydrologic processes through mass balance calculations and can be linked to one another through inflow-outflow calculations. More advanced modeling is achieved by selecting object-specific methods that further define the hydrologic processes associated with each object. Additionally, RiverWare may operate under a rule-based simulation, which creates logic-based interdependency of objects through user-defined rules. These rules may look forwards and backwards in time and given priorities in one rule may supersede others depending on the importance defined by the user. These detailed yet simple modeling techniques allow RiverWare to simulate reservoirs' pool elevations and inflow efficiently.

Table 9.3: Neches River Basin Dams Deliberate Impoundment Dates

Project	Sam Rayburn	B.A. Steinhagen
Impoundment Date	29 Mar 1965	16 Apr 1951

The Water Management Section inspected the dataset for quality before being used in the analyses. The instantaneous (hourly) lake inflows were gathered. The hourly records may contain many gaps. The gaps are for times when real time recording was missing. Data with missing records were not used in the analyses.

Figures 9.2 and 9.3 display the simulated pre-dam construction daily average inflow and post-dam construction pool elevation records for Sam Rayburn Reservoir and B.A. Steinhagen Lake, respectively.

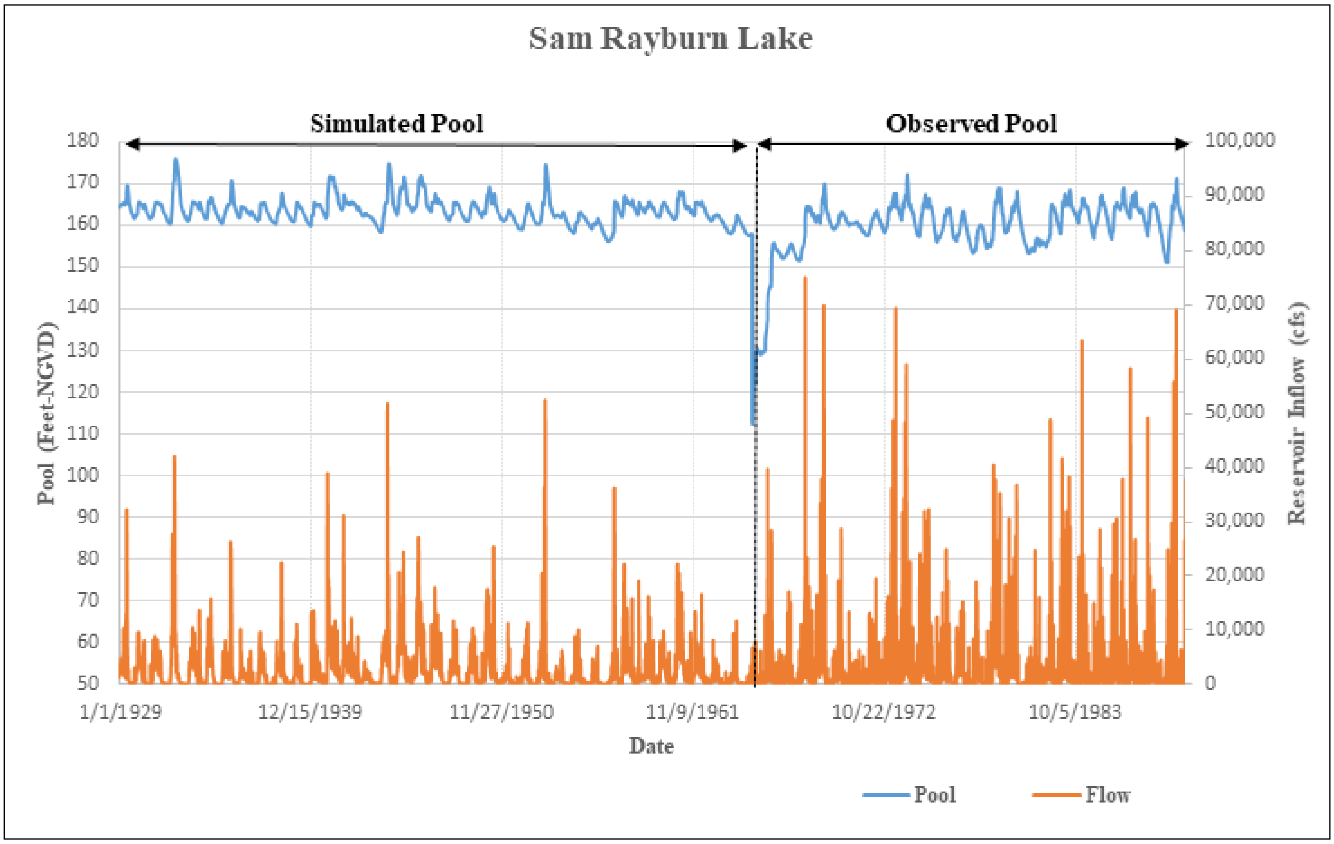


Figure 9.2: Sam Rayburn Reservoir Daily Average Inflow and Elevation

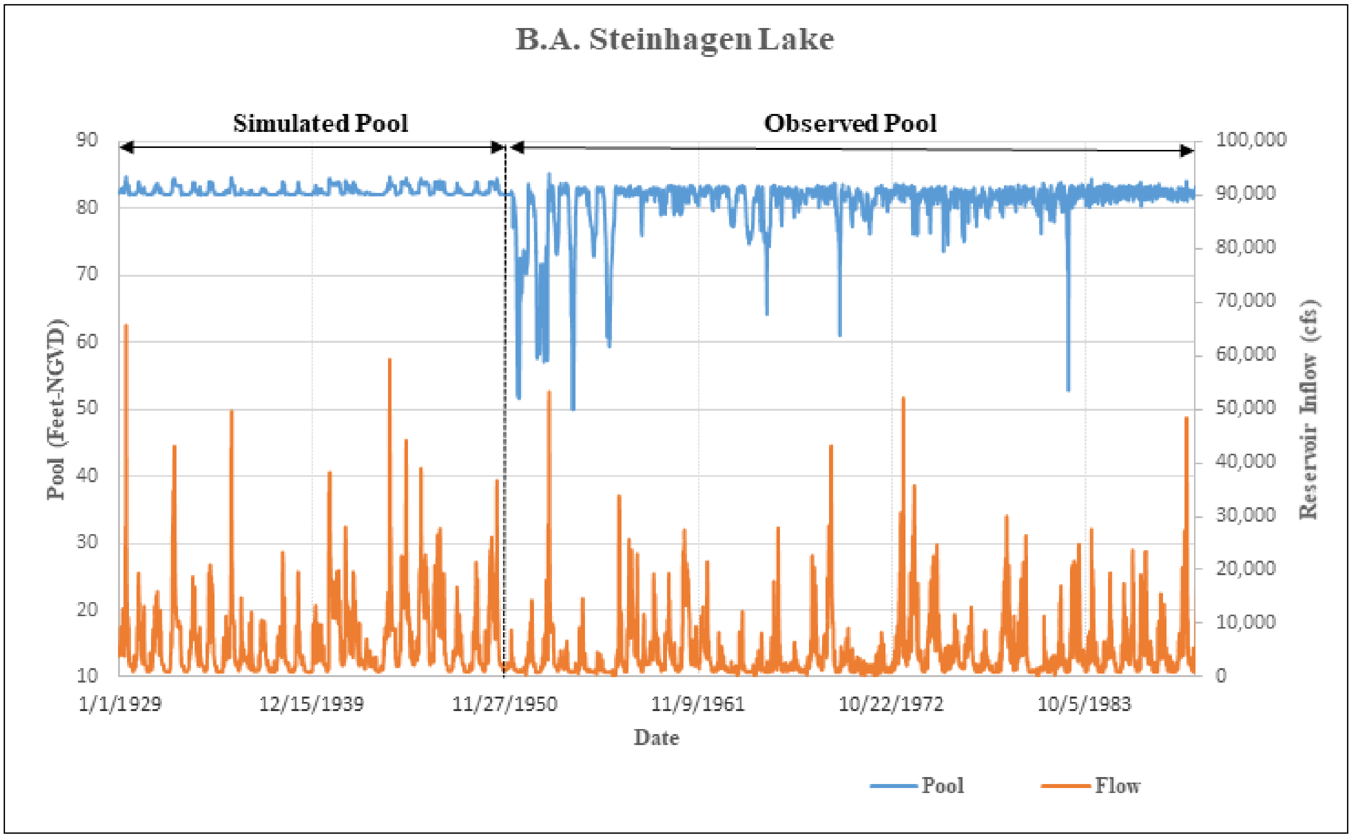


Figure 9.3: B.A. Steinhagen Lake Daily Average Inflow and Elevation

9.4.2 Instantaneous Peak Estimates

For Sam Rayburn Reservoir, two historical peaks were recorded at USGS 08039500 Angelina River near Horger (Ebenezer), TX; August 1915 (82,000cfs) and 11 April 1928 (6,940cfs). The gage has been discontinued, but its recorded historical peaks were consistent with peaks found in the Sam Rayburn Reservoir Water Control Manual (WCM) (USACE-SWF, 2018). The discontinued Horger gage is located just 5 miles downstream of the lake; it was used to develop the historical peak inflows to the lake, applying a drainage area ratio as a multiplier (Table 9.4). For B.A. Steinhagen's Lake, one historical peak was archived in the Water Management Section documents for the 1884 flood. The recorded peak is listed in Table 9.4.

Table 9.4: Neches River Basin USGS and Lake Inflow Estimated Historical Peaks

	Contibuting Drainage Area (mi^2)	Historical Year	Historical Peak (cfs)		Contibuting Drainage Area (mi^2)	*Estimated peak (cfs)
USGS 08039500	3,486	1915	82,000	Lake Sam Rayburn	3,449	81,130
		1928	6,940			6,866
Lake B.A. Steinhagen	7,573	1884	120,000			

*Estimated peak (cfs) = [Sam Rayburn drainage area (mi^2) / USGS gage drainage area (mi^2)] x [historical peak (cfs)]

9.4.3 Daily Average Annual Peak (AMS) Estimates

An extract of the 1-day average maximum annual peaks for each project was made available for the analysis. The lakes inflow systematic records were generated using RiverWare, and the lakes historical 1-day and critical duration n-day inflow peaks were generated from the historical instantaneous peaks. The critical duration best estimate in days is shown in section 9.7. Several attempts were made to better justify the best predictable n-day peaks. The n-day AMS historical peaks can be estimated using best engineering judgment once basin hydrology is well understood. For the Neches River Basin, experiences have shown that the instantaneous peaks do not tend to attenuate very much, and the travel time can stretch for a number of days, which reflects high daily average peak values.

For Sam Rayburn Reservoir, the best corresponding relationship (formula) with the strongest R^2 value among all fitting curves was utilized, and the predicted peaks followed a general power line trend, which was used to estimate the historical daily average peaks. The 1915 peak flood was adjusted slightly from the value estimated using the drainage area ratio method, which reduced the peak by 1%.

The 1-Day AMS historical peak for B.A. Steinhagen for 1884, was also estimated by applying a drainage area reduction factor of 1% from the historical instantaneous peak.

The critical duration annual peaks were estimated by establishing a correlation with the 1-Day AMS peaks. The critical duration AMS peaks are listed in Table 9.5.

Table 9.5: Neches River Basin N-Day AMS Estimated Historical Peaks

Project	N-Day Duration AMS Peak (cfs) (Historical)				Instantenous Peak (cfs)
	Year	1-Day	8-Day	33-Day	
Sam Rayburn	1915	80,318	39,507	N/A	81,130
	1928	6,826	5,548	N/A	6,866
B.A. Steinhagen	1884	118,000	N/A	74,065	120,000

9.5 CRITICAL INFLOW DURATION ANALYSIS

The critical inflow duration can be defined as the inflow duration that tends to produce most consistently the highest water surface elevation for the reservoir. The critical inflow duration accounts for the most significant storm events, which are normally selected based on a screening criterion that capture project inflow hydrographs with a minimum threshold peak determined on a case by case basis (i.e., Sam Rayburn critical inflow duration minimum threshold peak is 20,000cfs greater than B.A. Steinhagen's). Although projects located on the Neches River Basin are impacted by similar weather patterns and storms usually occur in similar seasons, B.A. Steinhagen Lake receives flows from Sam Rayburn Reservoir (Angelina River) and the Neches River tributaries. The flatter slopes and wide floodplains allow for longer critical durations. The storm duration can also impact critical durations; longer storms result in longer critical durations. For the Neches River Basin, the most critical flood season was determined to occur between June and November. In order to determine critical inflow duration of the observed rainfall-runoff events, extreme rainfall runoff (inflow) events are examined. All large inflow events are independent, meaning that different year hydrographs can be presented in one figure to determine the proper critical duration. The duration peak inflow was used to determine a reasonable value for critical inflow duration. Although this method was found accurate to produce good estimates, the critical duration can be adjusted later on during the analysis to reflect the most appropriate frequency curve. Best engineering judgment remains necessary in the final selection of the most appropriate value. For each project, a set of historical inflow events (hydrographs) with daily peak inflows greater than a certain threshold were extracted from the RiverWare simulated daily average inflow period of record (*i.e.*, examine the top 20% largest independent inflow events for each project). The best-estimate inflow duration for the reservoir is estimated by taking the average hydrograph of the major events specified. Sam Rayburn Reservoir and B.A. Steinhagen Lake inflow critical durations best estimates are demonstrated in (Figures 9.4 and 9.5).

Best estimates of the n-day critical durations are listed in Table 9.6. The best critical duration estimate produced the most conservative frequency elevation in the lake. The purpose of this analysis is to have a better understanding of the runoff response from large single rain events that helps establish what volume discharge frequency curves need to be examined.

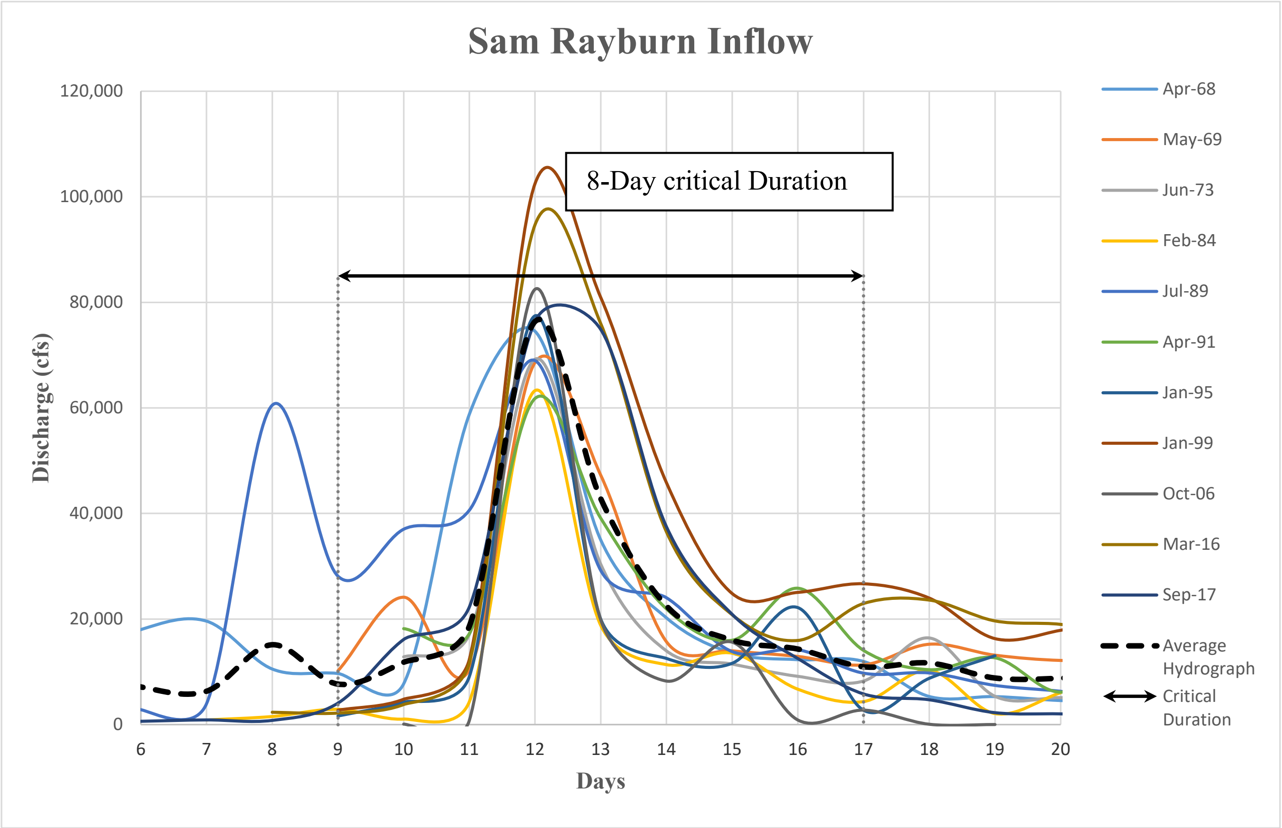


Figure 9.4: Sam Rayburn Reservoir Critical Duration Inflow Analysis

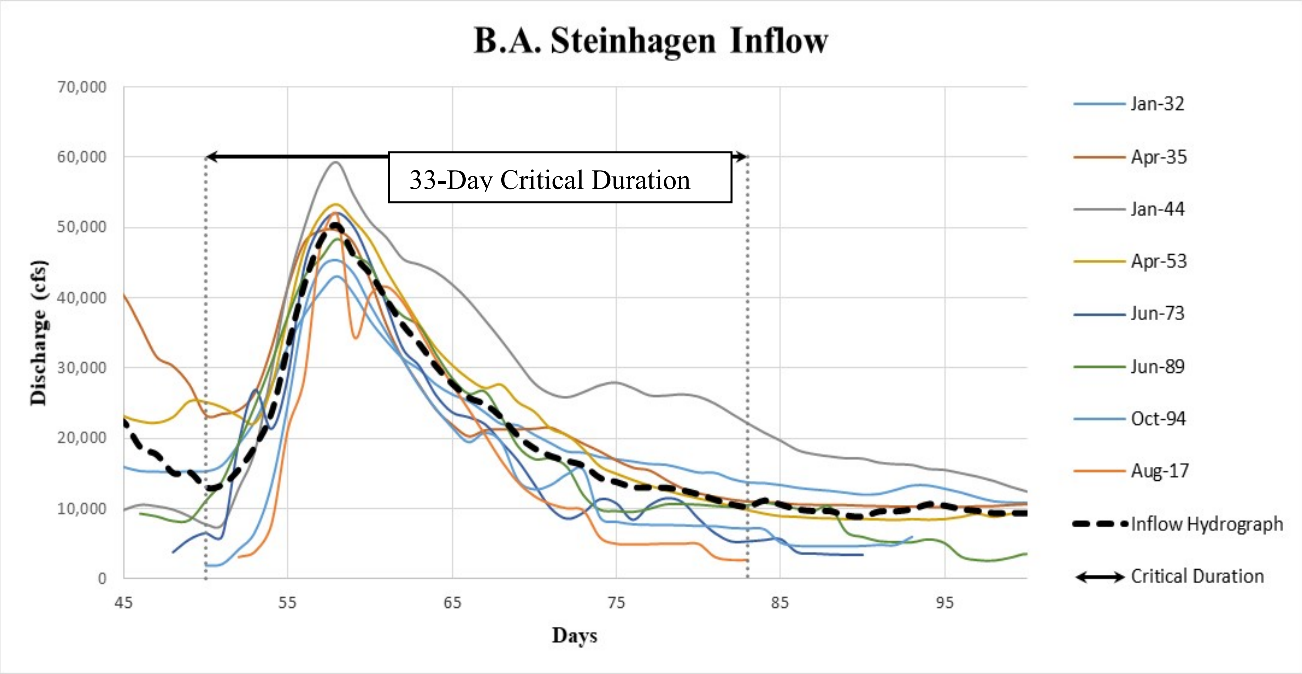


Figure 9.5: B.A. Steinhagen Lake Critical Duration Inflow Analysis

Table 9.6: Neches River Basin Inflow Duration Analysis

Project	Minimum Threshold Peak (cfs)	Number of Analyzed Inflow Events	Critical Duration (Days)
Sam Rayburn	60,000	11	8
B.A. Steinhagen	40,000	8	33

9.5.1 Volume/Flow Frequency Statistical Analysis

The volume/flow frequency analyses for the Neches River Basin lakes were estimated by following Bulletin 17C guidelines and procedures (statistical techniques) to determine exceedance probabilities associated with specific flow rates utilizing HEC-SSP 2.1.1. The observed and developed daily average annual maximum peaks were used to establish a relationship between flow magnitude and frequency. In this chapter, the term volume/flow frequency refers to the frequency with which a flow over a given duration, such as 1-, 8-, and 33-day, is expected to be equaled or exceeded. The duration range selection was based on inspecting the shape of the hydrographs such as those shown in Figures 9.4 and 9.5, and the critical durations listed in Table 9.6. To adequately assess the risk associated with the Neches River Basin Dams' structures in question, the 8-day critical duration was used to construct hypothetical inflow frequency events for Sam Rayburn Dam; the 33-day critical duration was used to construct inflow frequency events for B.A. Steinhagen dam. The events were routed through the projects to estimate reservoirs' stage-frequency curves.

9.5.2 Bulletin 17C Application

The use of Bulletin 17C guidance allows for computations of the annual exceedance probability of the instantaneous and daily average peaks, using the Expected Moments Algorithm (EMA). It estimates distribution parameters based on sample moment in a more integrated manner that incorporates non-standard, censored, or historical data at once, rather than as a series of adjustment procedures (NOAA, 2018). In this chapter, and when applicable, each project was assigned the associated historical peaks shown in Table 9.5 (i.e., B.A. Steinhagen, for a 33-day critical duration would be assigned one (1) historical peak of 74,065 cfs for the year of 1884). Values of perception thresholds from the historical peak events were set for the historical peak years for each project (*i.e.,* the year of 1884 was set for B.A. Steinhagen). The set of threshold peaks define the range of stream flow for which a flood event could have been observed; consequently, years for which an event was not observed and recorded, must have had a peak flow rate outside of the perception threshold. The use of Bulletin 17C procedures provide confidence intervals for the resulting frequency curve that incorporate diverse information appropriately, as historical data and censored values impact the uncertainty in the estimated frequency curve (NOAA, 2018). Within the Bulletin 17C EMA methodology, every annual peak flow in the analysis period, whether observed or not, is represented by a flow range that might simply be limited to the gaged value when one exists. However, it could also reflect an uncertain flow estimate, and this is the case for the Neches River Basin projects.

9.5.3 HEC-SSP Computations

A series of n-day volume duration frequency curves was developed for each of the Neches River Basin projects. The volume duration frequency results from this analysis were developed using HEC-SSP. The Multiple Grubbs-Beck algorithm was used for the low outlier test. Plotting position of the censored data is adopted from the Hirsch-Stedinger plotting position algorithm. The station skew option was used for the analysis for both projects using the systematic records. For consistency, each developed frequency curve underwent the same analysis techniques before adoption. Table 9.7 contains skews and record length for each project fed into the HEC-SSP program.

Table 9.7: Summary of HEC-SSP Input Parameters

Project	Systematic Record (years)	Historic Record (years)	Station Skew
Sam Rayburn	90	104	-0.74
B.A. Steinhagen	90	135	0.44

Note: The actual systematic record length is less than the systematic record length shown in the Table. The actual systematic record length was extended utilizing RiverWare.

The computed median inflows from HEC-SSP for the critical inflow duration are listed in Table 9.8. Only pertinent critical durations were listed for each project (*i.e.*, 8-Day and 33-Day). Additional information on the HEC-SSP analysis can be found in Appendix E.

Table 9.8: Bulletin 17C Computed Median Inflows

N	ACE	Bulletin 17C EMA Computed Average (Median) Peaks (cfs)	
Yrs	%	Sam Rayburn	B.A. Steinhagen
		8-Day	33-Day
500	0.2	52,696	66,659
200	0.5	49,107	55,689
100	1	46,025	48,239
50	2	42,569	41,435
20	5	37,303	33,299
10	10	32,648	27,679
5	20	27,194	22,386
2	50	17,875	15,439

9.6 RMC-RFA DATA INPUT

9.6.1 Inflow Hydrographs

Several inflow hydrographs were selected to route through the RMC-RFA program. The particular years of which hourly reservoir inflow hydrographs were routed are as follows:

Sam Rayburn: Available inflow hydrographs for May 2002, June 2003, April 2005, September 2005, October 2007, May 2012, and August 2017.

B.A. Steinhagen: Available inflow hydrographs for October 2009, May 2016, and December 2018.

The selected hydrographs' characteristics represent different hydrograph shapes (from peaky to large volume events) experienced at the Neches River Basin reservoirs. Figures of the selected hourly hydrographs for Sam Rayburn and B.A. Steinhagen Lakes are shown in Appendix E.

9.6.2 Volume Frequency Curve Computation

The computed volume frequency statistical parameters from HEC-SSP were fed into the RMC-RFA program to produce the n-day duration inflows for all projects. As stated in the HEC-SSP computations section, Bulletin 17C procedures and guidelines were followed to produce the volume discharge frequencies. Plots of the 8- and 33-Day discharge frequency curves for Sam Rayburn and B.A. Steinhagen are shown in Figures 9.6 and 9.7, respectively.

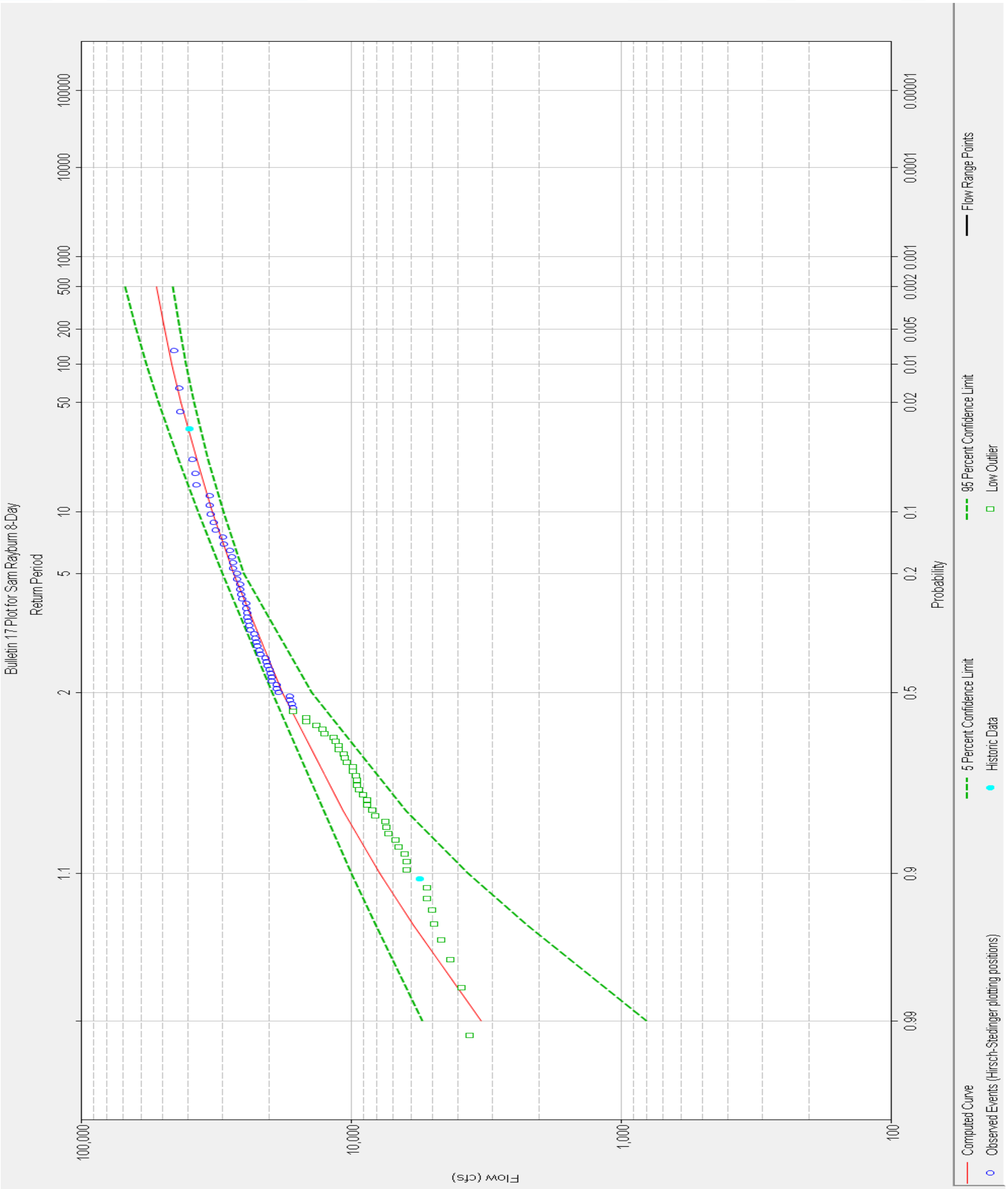


Figure 9.6: Sam Rayburn Reservoir Computed 8-Day Volume Frequency Curve

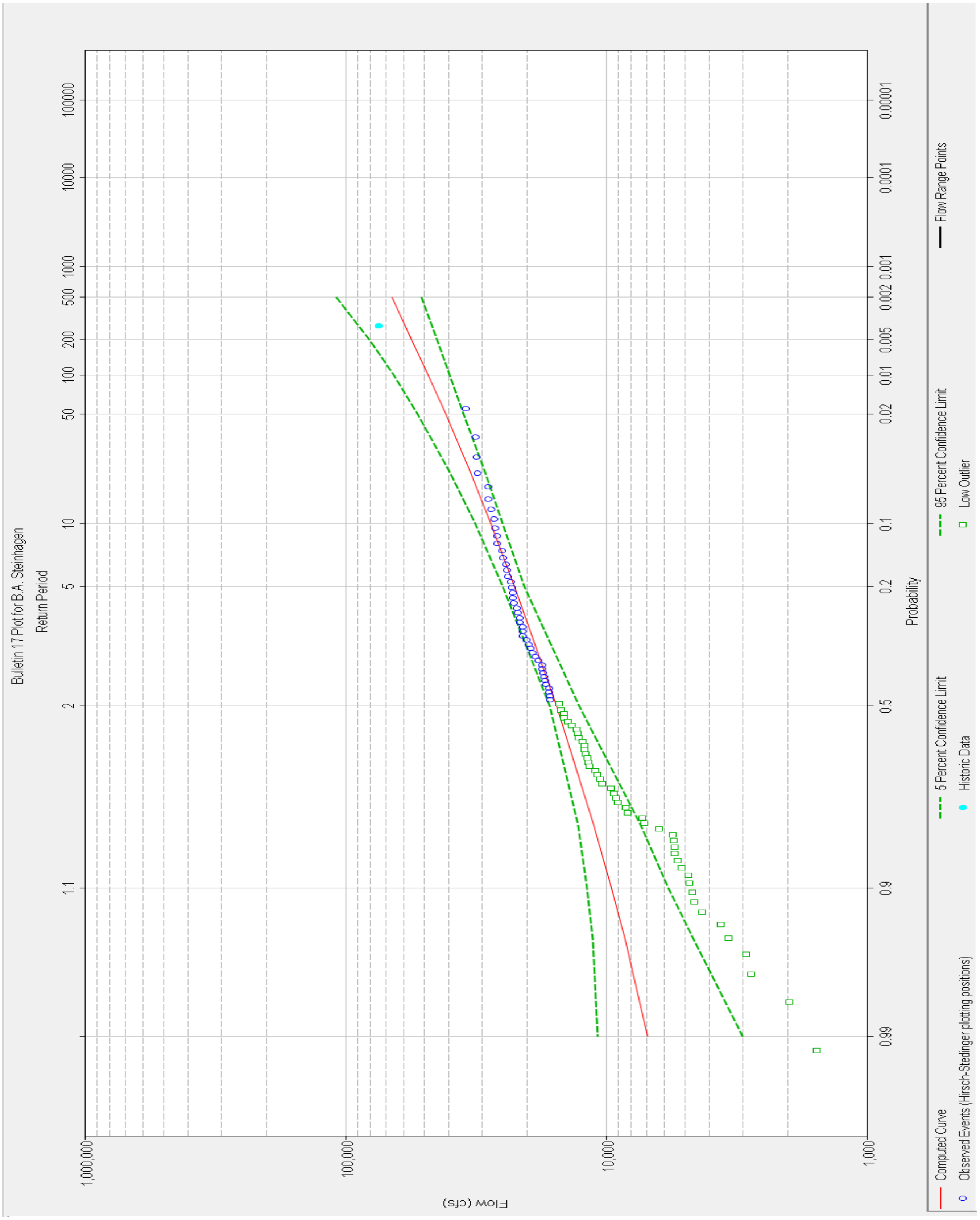


Figure 9.7: B.A. Steinhagen Lake Computed 33-Day Volume Frequency Curve

In Figure 9.7, one may observe that the1884 historical peak is by far greater than the other peaks recorded at Town Bluff, but Bulletin 17C procedures warrant the inclusion of this historic peak event in the results. For Town Bluff, incorporating the one historic peak of 1884 in the discharge frequency analysis skewed the curve upward and increased the spread of confidence bounds. Sensitivity analyses were performed by applying various low outlier thresholds to assess impacts on the results, but the some of these censoring criteria left the historic peak outside the confidence bounds. It was concluded that allowing the program to assign an option of an override low outlier threshold, following Bulletin 17C procedures and guidelines, produced more reasonable results by placing the recorded historic peak inside the confidence bounds and allowing the computed curve to fit closer to the historical peak. During more recent flood events (e.g., Hurricane Harvey), Town Bluff Dam released all inflows it received at 70,000 cfs per day. Hurricane Harvey intensified in the reaches located downstream of the project, and therefore, the observed peak at the reservoir was not as significant as the historical peak recorded in 1884.

9.7 RMC-RFA ANALYSIS

9.7.1 Flood Seasonality

Many reservoirs have operations (pool level) that vary by season in response to the cyclical changes in meteorology and hydrology throughout the year. The inflow pattern at the Neches River Basin reservoirs has three general types of flood-producing rainfall: thunderstorms, frontal rainfall, and tropical cyclones. Generally, the highest precipitation accumulations for the daily through monthly durations have occurred during tropical cyclones. However, there are some instances of heavy precipitation resulting from local thunderstorms. It should be noted that thunderstorms can occur at any time of the year and tropical storms can happen between June and November. Due to meteorological and hydrologic conditions, most significant floods occur during late spring, summer, and fall months.

The term *flood seasonality* is intended to describe the frequency of occurrence of rare floods on a seasonal basis, where a rare flood is defined as any event where the flow exceeds some user specified threshold for a specified flow duration. In the RMC-RFA model operation, a month of flood occurrence is first selected at random according to the relative frequency. Once the month of flood occurrence is specified, a starting pool elevation for the event can be determined from the reservoir stage-duration curve for that particular month. This approach ensures that seasonal variation in reservoir operations is a part of the peak-stage simulation.

The flood seasonality analysis is performed by assigning n-day flood seasonality, threshold flow, maximum events per year, and minimum days between events. With these criteria, a total number of events can be calculated. Although the flood seasonality critical duration could be different from the volume frequency curve adopted critical duration, Sam Rayburn and B. A. Steinhagen Lakes adopt the same critical duration for the criteria used to estimate flood seasonality and volume frequency curves. The flood seasonality relative frequency output for both reservoirs are illustrated in Figures 9.8 and 9.9.

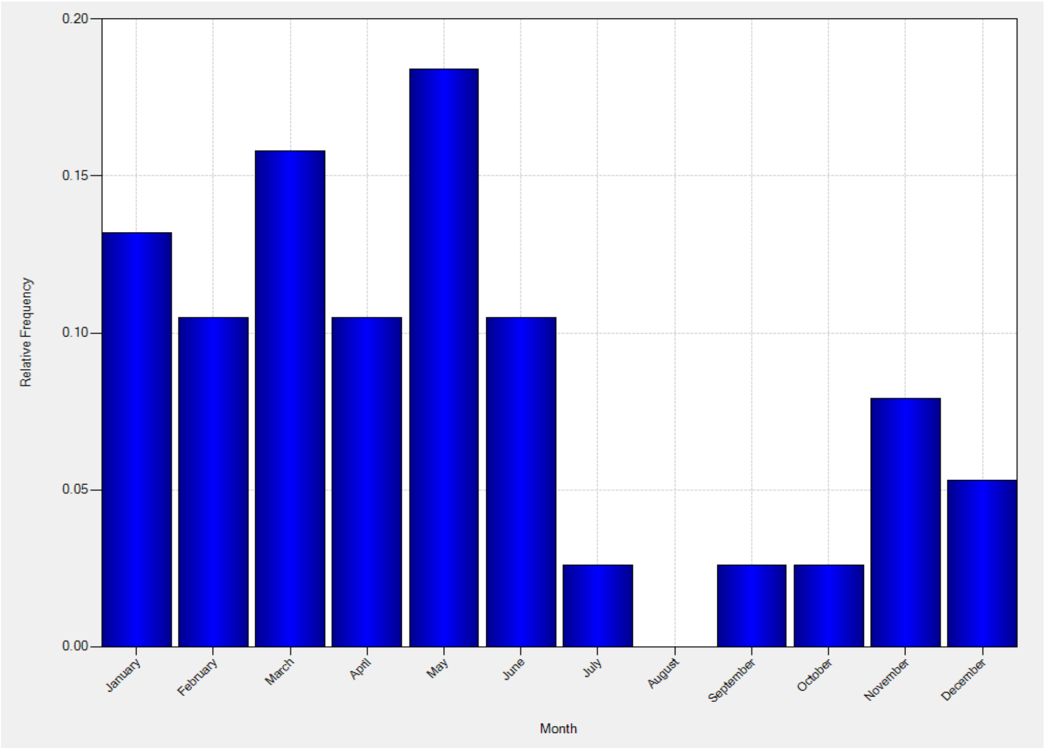


Figure 9.8: Sam Rayburn Histogram of RMC-RFA Relative Frequency Output

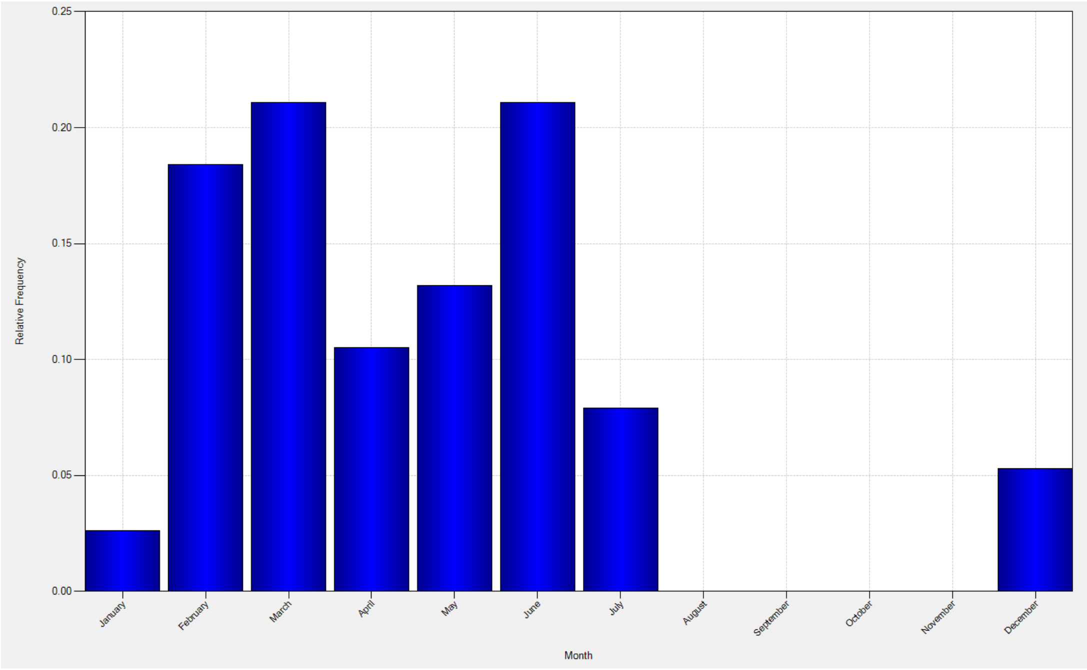


Figure 9.9: B.A. Steinhagen Histogram of RMC-RFA Relative Frequency Output

9.7.2 Reservoir Starting Stage

Pool duration curves represent the percent of time during which particular reservoir pools are exceeded and help define the range and frequency of possible starting pool elevations for the RMC-RFA analysis. For Sam Rayburn Reservoir, the reservoir starting stage was estimated by analyzing pool elevations by first filtering observed daily average pools so that they only represent typical starting pools based on a pool change threshold. Then, the filtered data set is sorted by month or season. Because RMC-RFA chooses a starting pool elevation for its simulations based on historic data, the historic data must be filtered so that it is not influenced by flooding events. Starting pool elevations should form the basis for flooding events, not be the result of said events. Therefore, historic pool elevations were filtered with a pool change threshold of 0.3 feet per day and a typical high (flood) pool duration of 177 days. This filtered stage data now forms the basis for the starting pool elevation for the RMC-RFA reservoir simulation. Figure 9.10 illustrates Sam Rayburn's filtered starting pool elevations by month and probability.

For B.A. Steinhagen Lake, an inflow threshold method was used to establish starting pool duration curves based on an inflow threshold value of 19,000 cfs. This meets the value that falls under the estimated 33-day critical duration and its most frequent event (volume) value. By doing so, all inflow hydrographs into the lake only consider rising limbs responsible for raising the pool. The project final starting pool elevation and duration curves are illustrated Figure 9.11. Low stages occurred in October and January and were associated with more frequent events. This filtered stage data now forms the basis for the starting pool elevation for the RMC-RFA reservoir simulation.

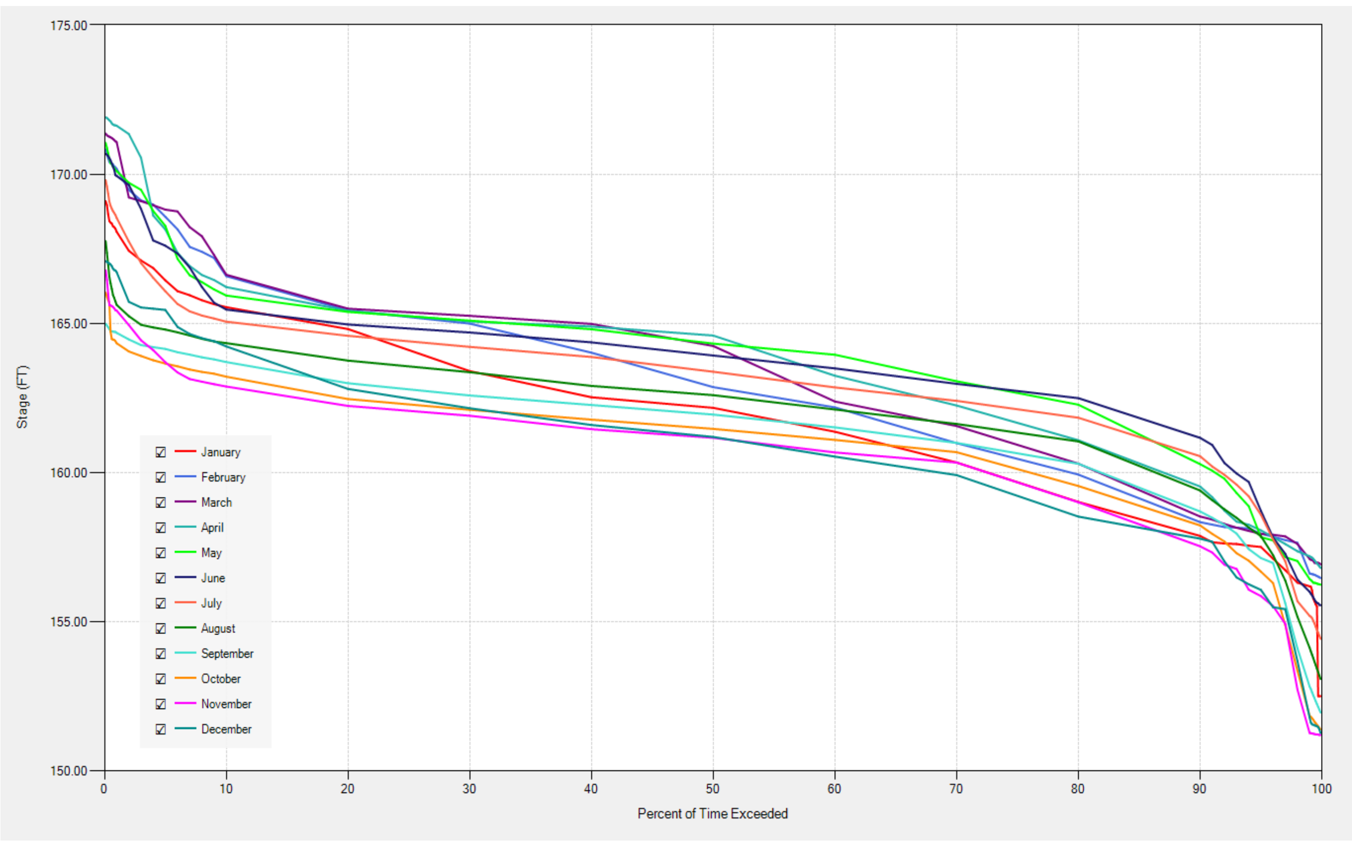


Figure 9.10: Sam Rayburn Reservoir Starting Stage Durations

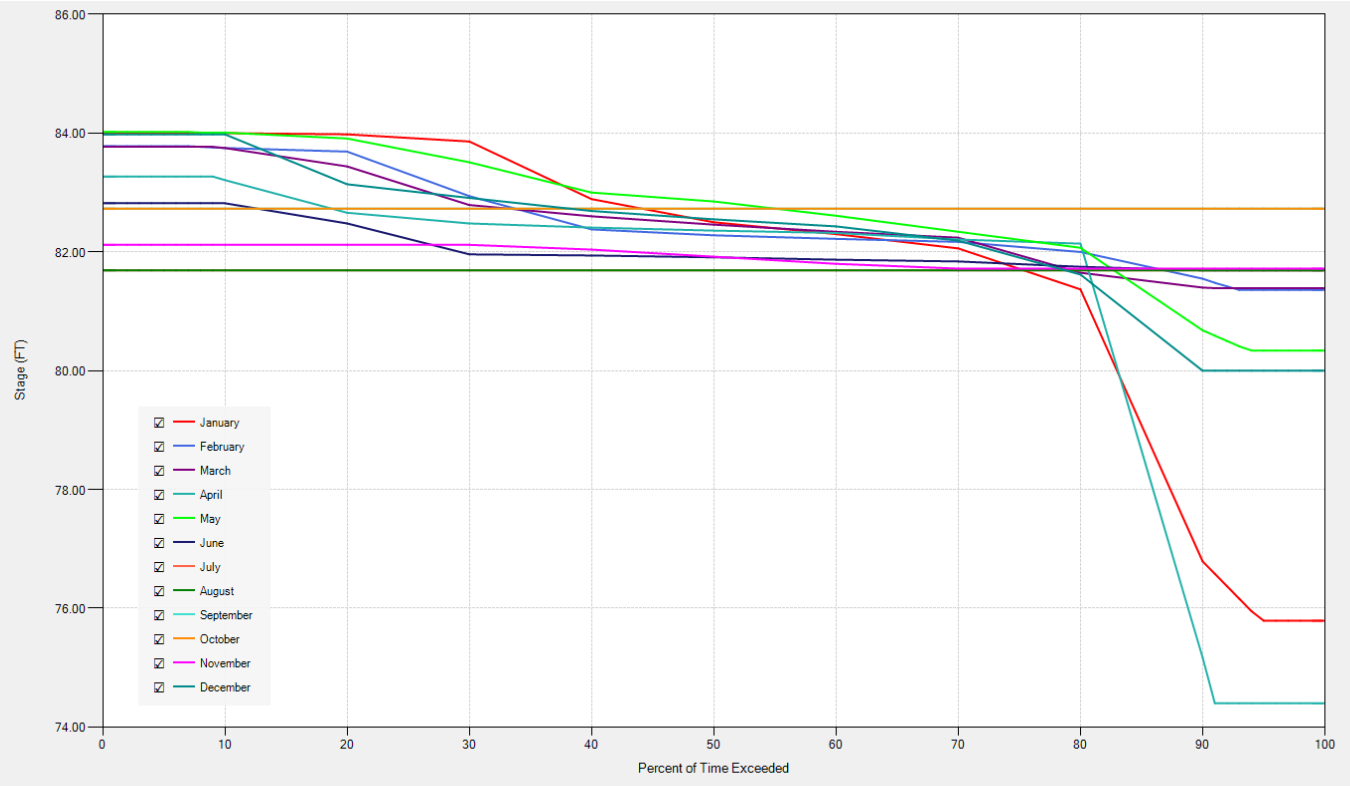


Figure 9.11: B.A. Steinhagen Lake Starting Stage Durations

9.7.3 Empirical Frequency Curve

For the evaluation of hydrologic hazards of each project, an extreme-value series of annual maximum stage was generated from the n-year systematic (RiverWare + Observed) period of record. The RiverWare simulated peaks were used prior to dam construction when the observed peaks were not available, for an intent of extending pool record. Each POR annual maximum series was extracted, the AMS was ranked, and it was plotted on log probability paper using the Weibull plotting position formula. Figures 9.12 and 9.13 show Sam Rayburn and B.A. Steinhagen's empirical pool frequency relationships when applying the Weibull plotting positions. The systematic frequency peaks for all the projects will later be plotted against the RMC-RFA expected pool frequency curves. The plotting position of the highest and lowest points are the most uncertain due to having insufficient record lengths necessary to inform accurate plotting positions at the extremes. More discussion and analysis of the empirical frequency curves is available in Appendix E.

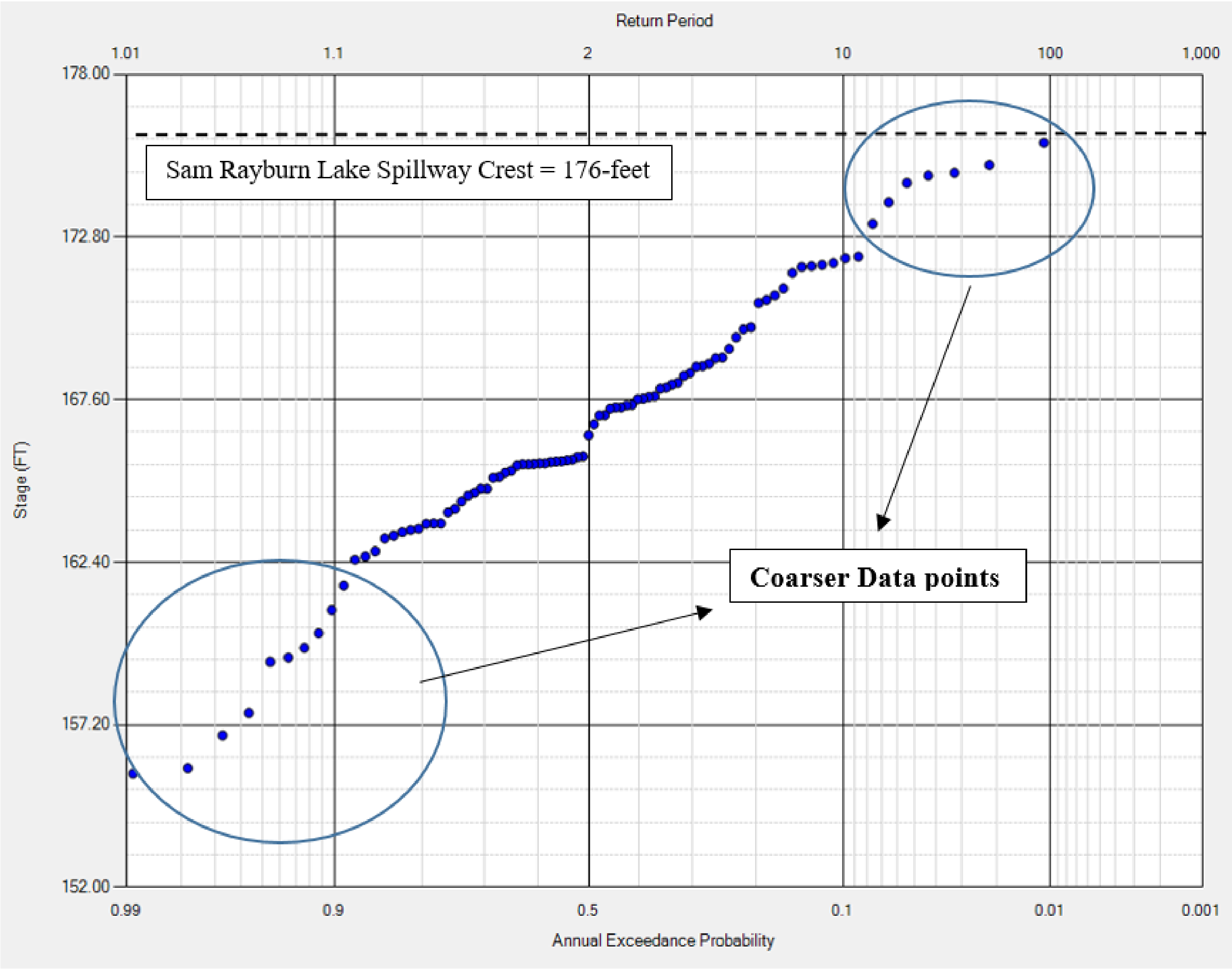


Figure 9.12: Empirical Stage Frequency for Sam Rayburn Reservoir

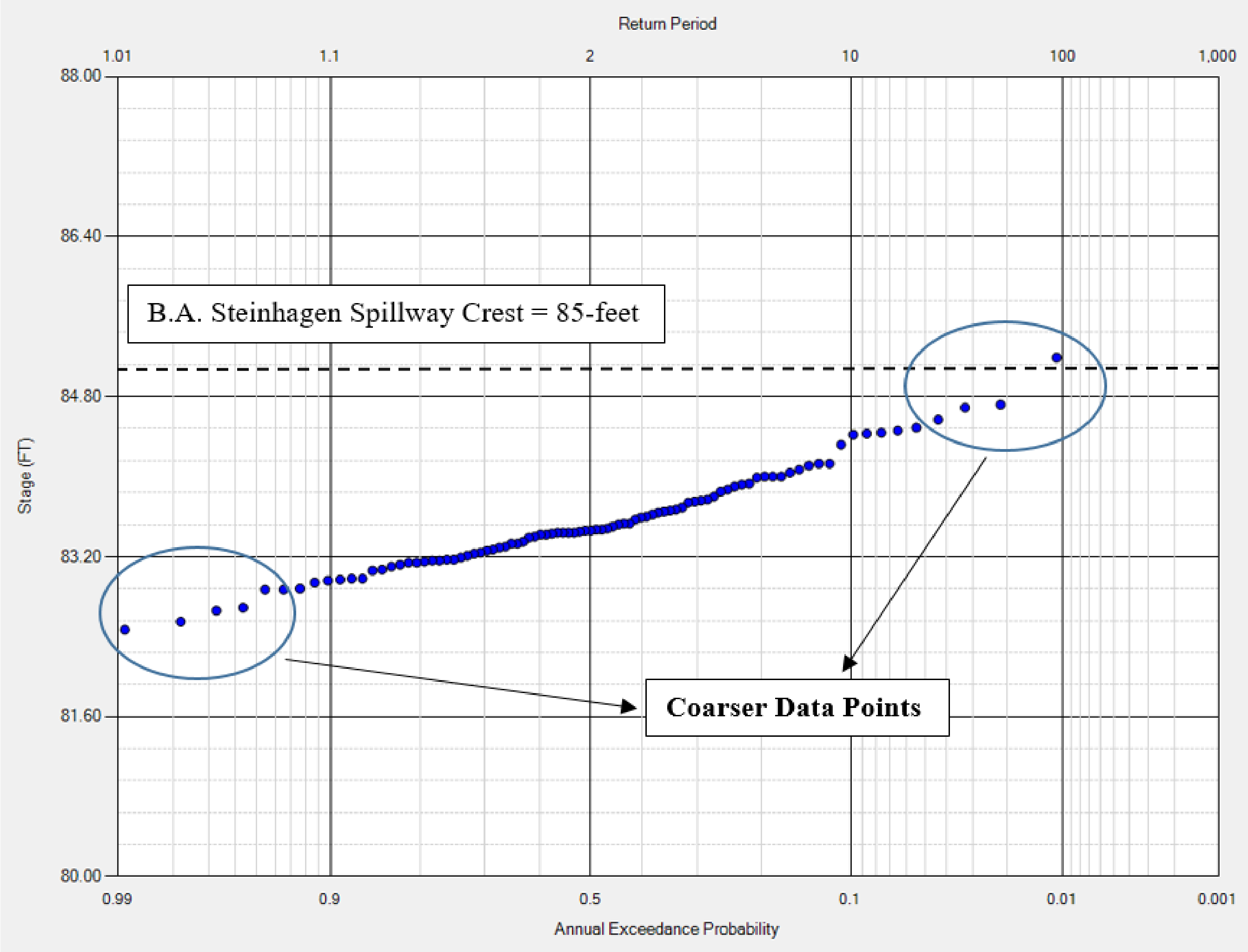


Figure 9.13: Empirical Stage Frequency for B.A. Steinhagen Lake

9.7.4 Reservoir Model

The reservoir details such as the Stage-Storage-Discharge function and top of dam, spillway, and inflow design flood elevations were obtained from the Fort Worth District USACE electronic library archived files. Volumetric surveys of both reservoirs were entered to update storage information. This was done using current GPS, acoustical depth sounder, and GIS technology. Data was then gathered and processed to generate the stage-storage curves for the reservoirs. The information is needed in order for the simulation to run. The volumetric and sedimentation survey (mostly up to conservation) of Sam Rayburn and B.A. Steinhagen lakes were completed in November 2006 and October 2011, respectively. The Texas Water Development Board (TWBD) website has the up to date surveyed lakes information. Data for portions of the surveyed lakes above conservation are obtained from the original design documents. The Neches River Basin projects' releases are stage dependent. Therefore, a stage-storage-discharge function can be estimated. The Discharge-Elevation and Storage-Elevation curves for the projects are shown in Figures 9.14 through 9.17. More details about reservoir features are listed in Table 9.9.

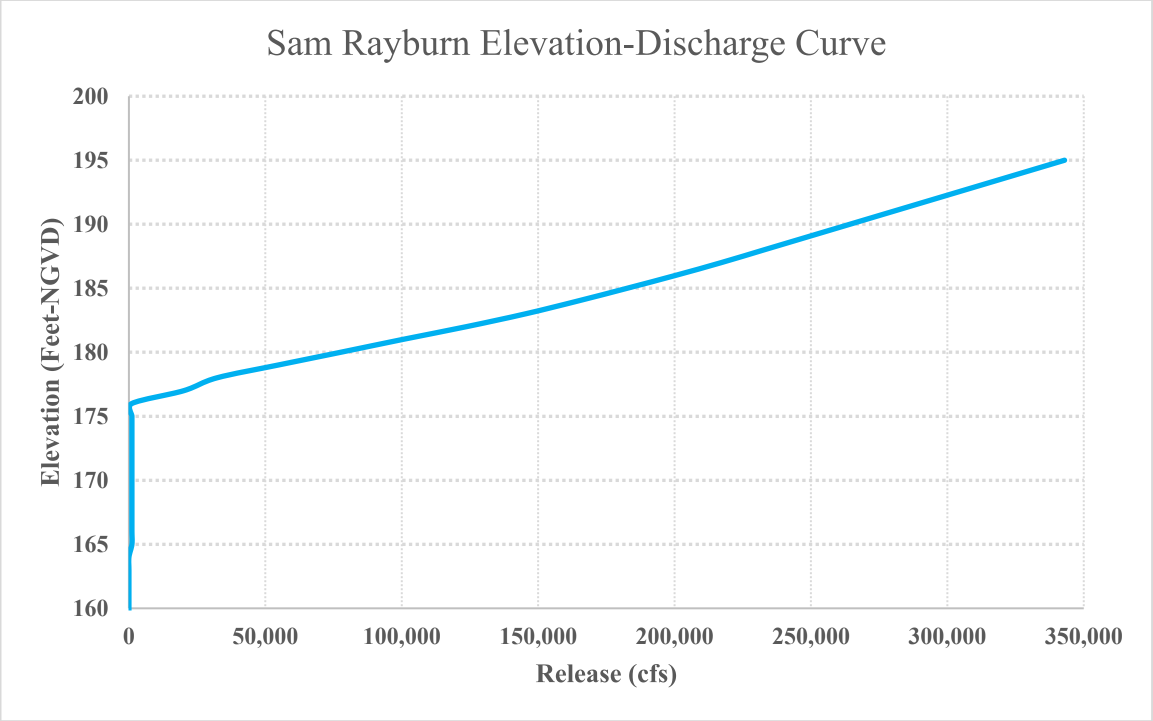


Figure 9.14: Sam Rayburn Outflow Elevation-Discharge Curve

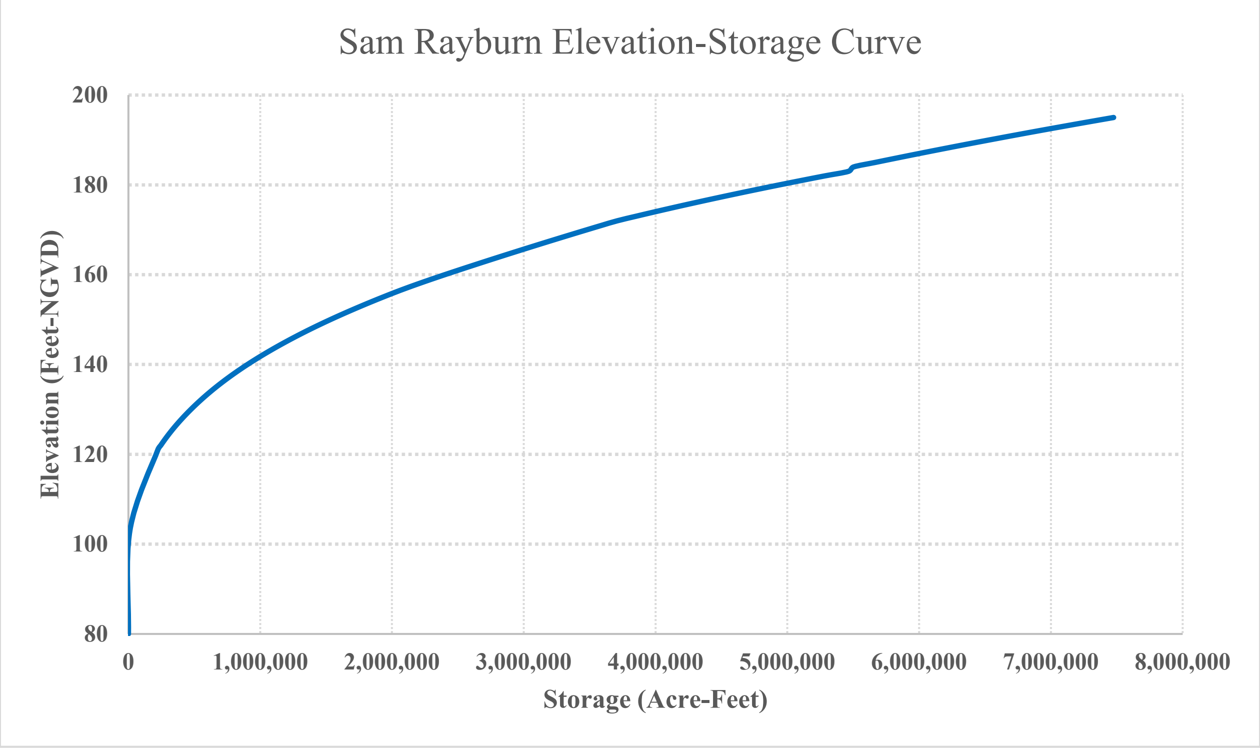


Figure 9.15: Sam Rayburn Storage-Elevation Curve

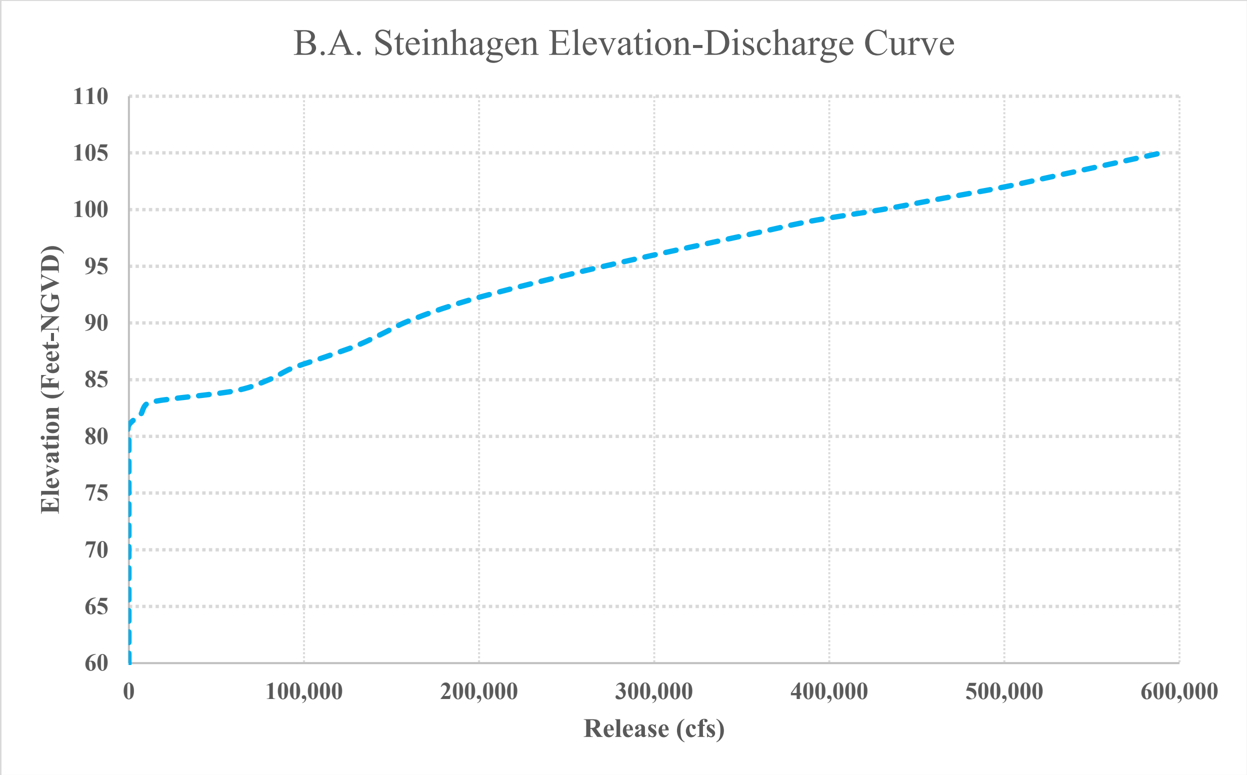


Figure 9.16: B.A. Steinhagen Lake Outflow Elevation-Discharge Curve

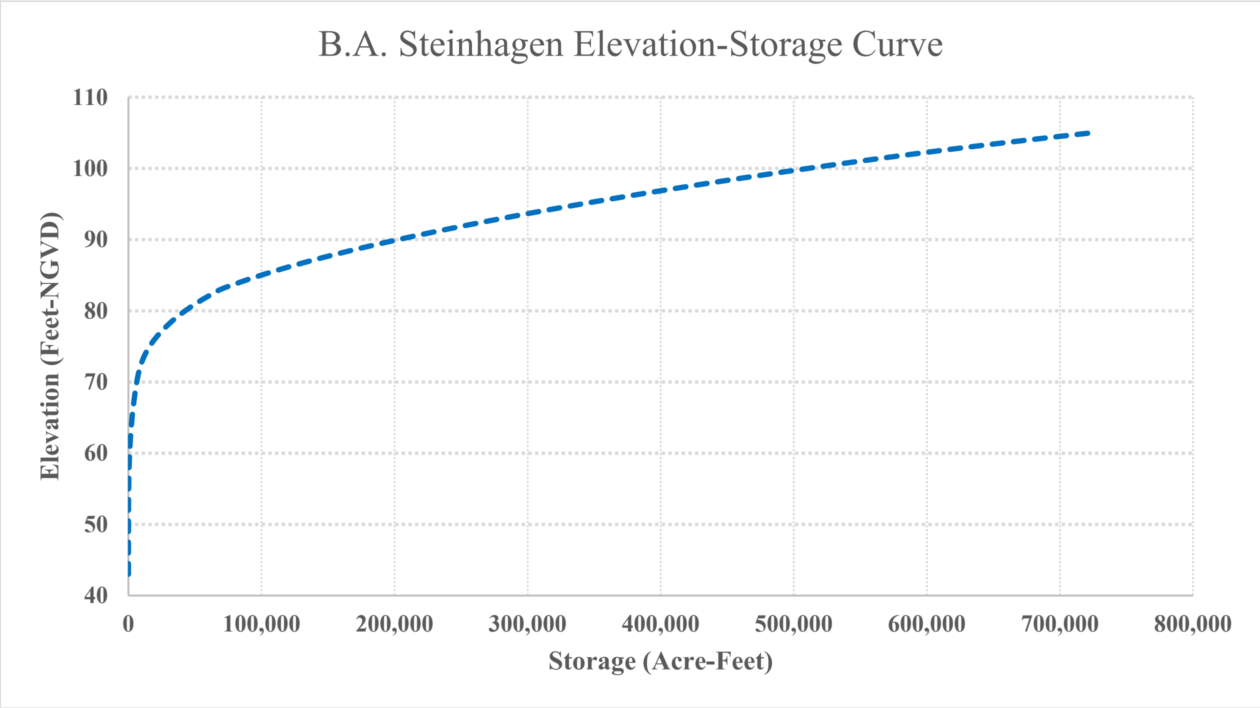


Figure 9.17: B.A. Steinhagen Lake Storage-Elevation Curve

Table 9.9: Neches River Basin Project Features

Project	Sam Rayburn	B.A. Steinhagen
Pertinent Feature	Elevation (Feet)-NGVD	
Top of Dam	193.00	95.00
Top of Flood Control Pool	173.00	N/A
Spillway Crest	176.00	85.00
Top of Conservation Pool	164.40	*83.00

* Top of conservation has fluctuated over the years between 81.5 and 83 feet. It is now set to 83 feet, per Hurricane Katrina regulations and water supply requirements with the LNVA.

The importance of using accurate Storage-Discharge-Elevation (Stage) curves is that it results in more accurate estimates of high extreme peak values associated with high degree of uncertainty (*i.e.,* 1% ACE and beyond). Such high peaks are normally observed near or above the spillway crest. Validations comparisons between the adopted discharge-elevation curves used in RMC-RFA for Sam Rayburn and B.A. Steinhagen lakes and the observed data are available in Appendix E. The plots showed that adopted curves are within range of observed operations.

9.8 RMC-RFA RESULTS

The RMC-RFA program was used to simulate rainfall-runoff floods using the inflow-frequency curve and the adopted flood seasonality. The specified hourly inflow hydrographs are weighted equally to account for each unique shape (*i.e.,* volume and peak) and to have the same probability. Long routing time windows of 30 days for Sam Rayburn and 120 days for B.A. Steinhagen Lake were specified to calculate the full size of floods routed through the reservoir on an hourly basis. In order to properly route flow events through B.A. Steinhagen, a set of peaky hydrographs were used as the basis to generate pool frequencies. Routing events with larger volumes produced higher stages than actually observed even for more frequent events (*i.e.*, 10%ACE); those events (September 2005, October 2006, August 2007, and February 2016) were not considered during the simulation to better justify results. The RMC-RFA model was simulated using the expected pool frequency curve only model option. This runs 10,000 realizations with 1,000,000 events per realization. This means RMC-RFA simulates a total of 10 billion flood events (10,000 x 1,000,000) to produce its best estimate of the expected curve. The following sections list detailed results about each project's new simulated expected stage-frequency curve. The total release from each project corresponding to each pool frequency was also developed by analyzing each project's observed releases, where annual maximum peaks were plotted using the Weibull position distribution, and applying a graphical curve, which would approximately fit through the data points.

Each federally owned project has a flowage easement elevation. The flowage easement land is privately owned land on which the Federal government (*i.e.,* USACE) has acquired certain perpetual rights. These include the right to flood it in connection with the operation of the reservoir, the right to prohibit construction of any structure for human habitation, the right to approve all other structures constructed on flowage easement land, except fencing. Having properties located above the easement elevation helps keep what would become damageable property out of the flood pool, so that the reservoir can be operated with a full focus upon downstream conditions and the concern for dam safety. To put things in perspective about the flowage easement, figures in the following sections illustrate easement elevation in relation to the reservoir pool frequencies, spillway crest elevation, and top of dam.

The RMC-RFA pool frequency curves fit well through the empirical stage data points. Greater fit was seen through the more frequent events. Frequencies between the 10% ACE (10-year) and 1.25% (80-year) events were underestimated by RMC-RFA due to the abrupt change in trend of the empirical points. Farther upward, the curves showed good fit through the coarser points near the 1% ACE (100-year) events. The curves are believed to have captured good estimates beyond the 1% ACE (100-year) events. Adopted pool frequency curves along with comparison with the Fort Worth District USACE reported pool frequencies are shown in the tables below.

9.8.1 Results for Sam Rayburn Reservoir

The detailed results of Sam Rayburn's new simulated expected stage-frequency curve are listed in Table 9.10. The total release from Sam Rayburn corresponding to each pool frequency was also developed by analyzing the dam's observed releases and the results are listed in Table 9.11. Sam Rayburn Dam tends to heavily regulate inflows for flood control and power supply purposes. Peaks are usually flattened and max out near 20,000 cfs (Figure 9.18). The observed releases were used to best estimate release frequencies below the spillway crest. High flood events that may exceed spillway crest elevation, would follow the discharge-elevation curve illustrated in Figure 9.14.

Table 9.10: Sam Rayburn RMC-RFA Computed Pool Frequency Comparison with Previous USACE Estimate

Sam Rayburn Lake		RMC-RFA Best Estimate	2009 USACE	Change in Pool	Easement Pool	Easement 1%ACE (100-yr) Freeboard
N-Year	ACE%	Feet-NGVD		Feet		
2	50	166.96	166.70	0.26		
5	20	170.35	172.10	-1.75		
10	10	172.37	173.30	-0.93		
25	4	174.27	175.00	-0.73		
50	2	175.53	176.00	-0.47		
100	1	176.40	176.50	-0.10	179.00	2.60
250	0.4	177.13				
500	0.2	177.60				

Table 9.11: Sam Rayburn Reservoir Computed Frequency Discharge Releases

Sam Rayburn Lake		RMC-RFA Best Estimate			
N-Year	ACE%	Elevation-NGVD	Spillway Release (cfs)	Gate Release (cfs)	Total Release (cfs)
2	50	166.96	0	10,000	10,000
5	20	170.35	0	15,000	15,000
10	10	172.37	0	17,000	17,000
25	4	174.27	0	19,000	19,000
50	2	175.53	0	19,500	19,500
100	1	176.40	6,440	13,560	20,000
250	0.4	177.13	12,000	8,000	20,000
500	0.2	177.60	26,275	0	26,275

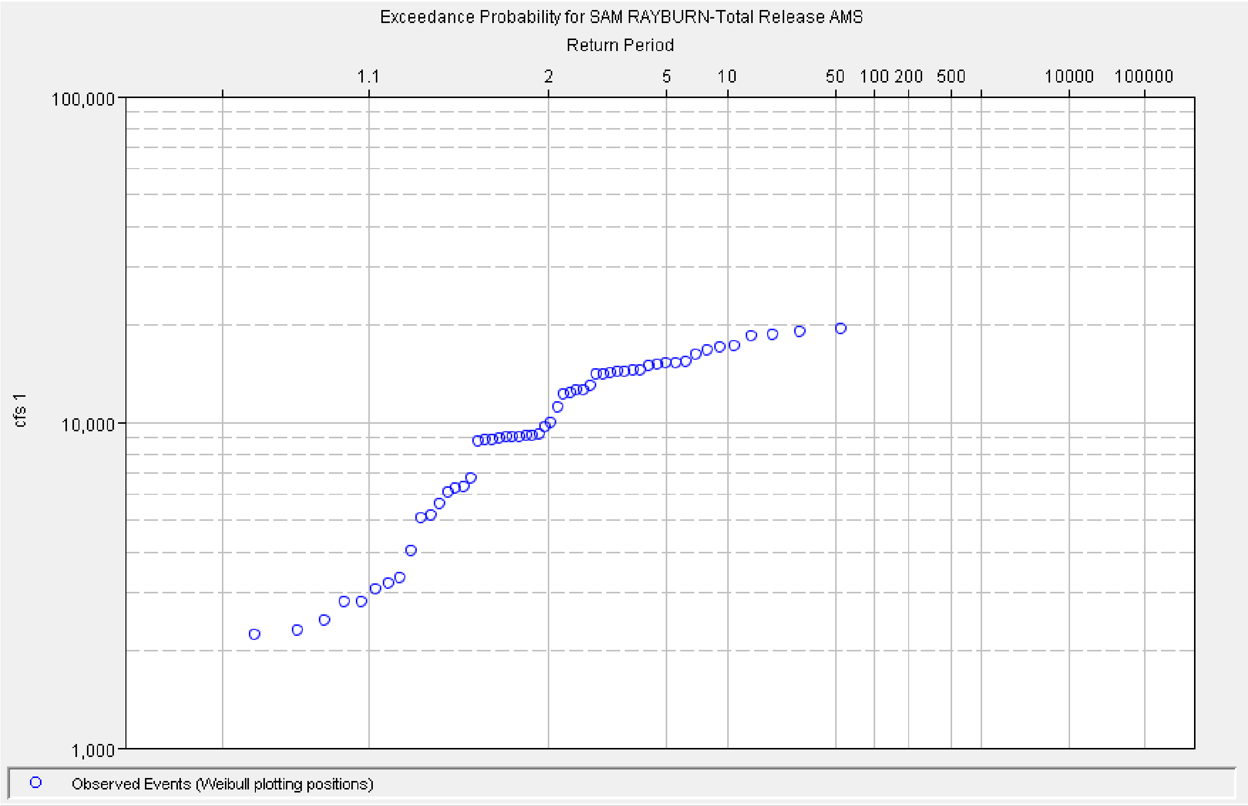


Figure 9.18: Sam Rayburn Observed Releases Following Weibull Plotting Distribution

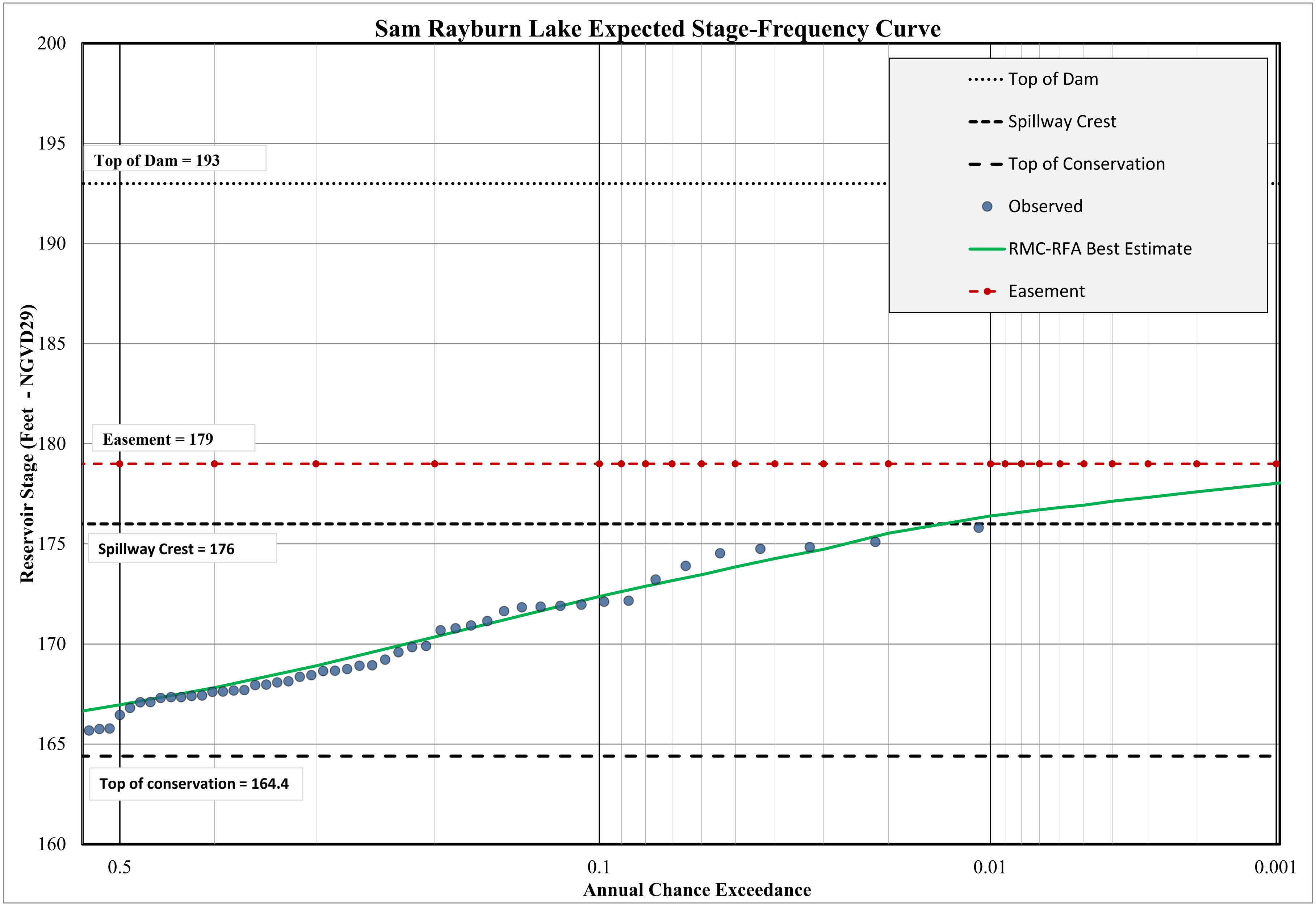


Figure 9.19: Sam Rayburn Reservoir RMC-RFA Final Stage-Frequency Curve

9.8.2 Results for B.A. Steinhagen Lake

The detailed results of B.A. Steinhagen's new simulated expected stage-frequency curve is listed in Table 9.12. The total release from B.A. Steinhagen corresponding to each pool frequency was also developed by analyzing the dam's observed releases and the results are listed in Table 9.13. B.A. Steinhagen is a run of river dam with limited storage. During flood events, the project releases all the water it receives, which allows for less peak attenuation. The plotting distribution of B.A Steinhagen's peak outflows can be analyzed following Bulletin 17B/C procedures, or by graphical distribution analysis, as shown in Figure 9.20.

Table 9.12: B.A. Steinhagen Lake Computed RMC-RFA Pool Frequency Comparison with Previous USACE Estimate

B.A. Steinhagen Lake		RMC-RFA Best Estimate	2009 USACE	Change in Pool	Easement Pool	Easement 1%ACE (100-yr) Freeboard
N-Year	ACE%	Feet-NGVD		Feet		
2	50	83.50	83.20	0.30		
5	20	83.86	83.60	0.26		
10	10	84.12	83.80	0.32		
25	4	84.55	84.00	0.55		
50	2	84.98	84.50	0.48		
100	1	85.41	85.00	0.41	88.00	2.59
250	0.4	86.23				
500	0.2	86.91				

Table 9.13: B.A. Steinhagen Lake Computed Frequency Discharge Releases

B.A. Steinhagen Lake		RMC-RFA Best Estimate			
N-Year	ACE%	Elevation-NGVD	Spillway Release (cfs)	Gate Release (cfs)	Release (cfs)
2	50	83.50	0	20,000	20,000
5	20	83.86	0	30,600	30,600
10	10	84.12	0	38,750	38,750
25	4	84.55	0	54,000	54,000
50	2	84.98	0	69,200	69,200
100	1	85.41	4,420	80,000	84,420
250	0.4	86.23	5,280	90,000	95,280
500	0.2	86.91	8,600	103,400	112,000

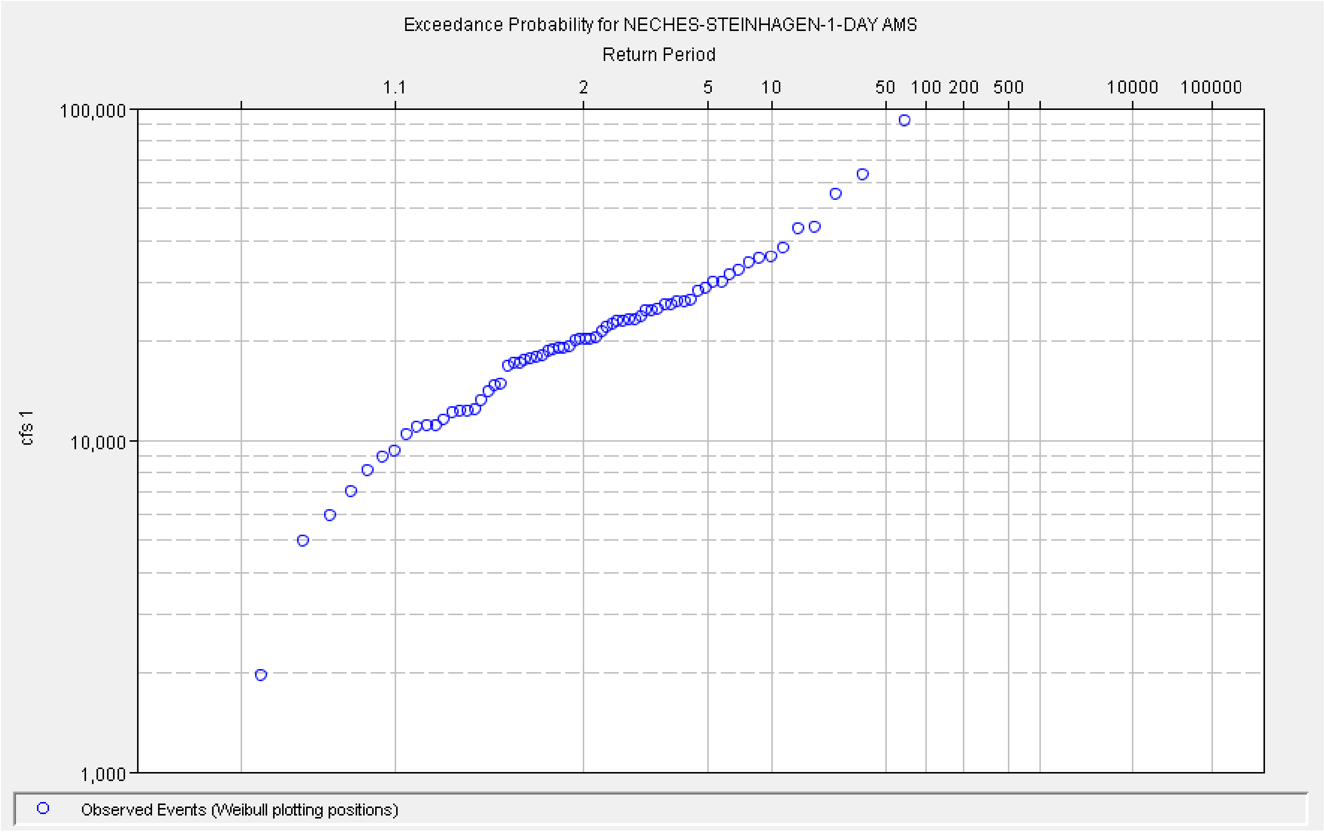


Figure 9.20: B.A. Steinhagen Observed Releases Following Weibull Plotting Distribution

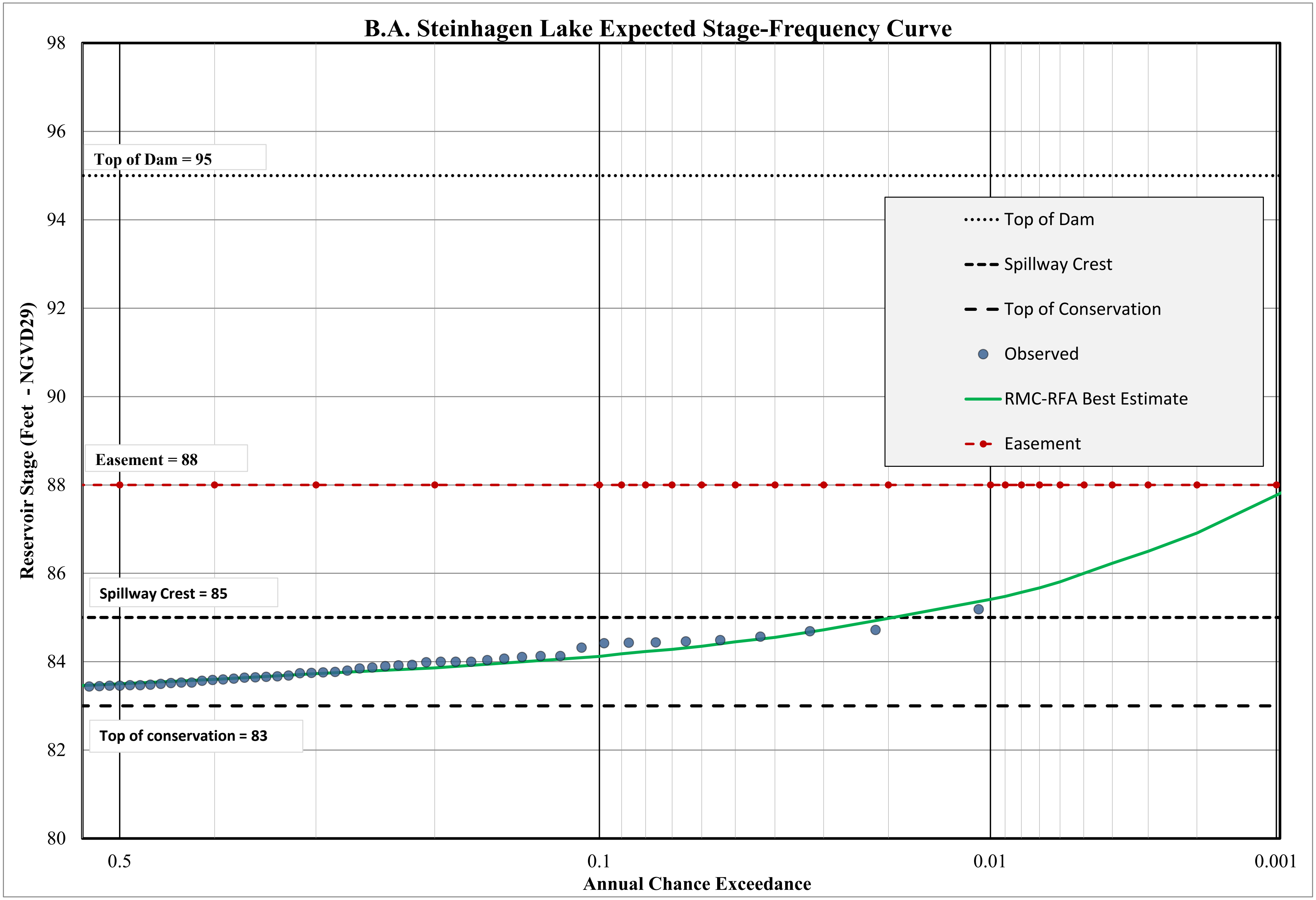


Figure 9.21: B.A. Steinhagen Lake RMC-RFA Final Stage-Frequency Curve

9.9 RESULTS VALIDATION

The pool frequency results displayed in the previous section were validated using best engineering judgment, inspection of historical observed pool records, and various sensitivity tests. B.A. Steinhagen Lake pool frequency results were not sensitive to the starting pool conditions or the critical duration used to scale inflows for routing. The produced curve showed stable transition through all frequencies. As a result, the adopted frequencies were associated with high degree of confidence. On the other hand, Sam Rayburn Reservoir pool frequencies were sensitive to the selection of the appropriate starting pool conditions. The use of the pool change threshold method and typical high pool duration produced a better curve fit than applying an inflow threshold value. The project was also found to be sensitive to the elevation-discharge curve based on turbine release conditions. Assuming high releases would lower the estimated frequency curve. Applying minimum turbine releases of 1,100 cfs tended to reflect better results, when using 8-days critical durations, while applying 4,000 cfs releases, would lower the best estimate curve significantly. The Sam Rayburn Reservoir results were not sensitive to the selected critical durations. For example, using a critical duration of 30-days, based on the extended amount of time it takes the pool to recede from its rising limb, is similar in its results to using an 8-day critical duration. The empirical points are typically used as a guide to construct the best estimate curve for more frequent events. Additional information on the sensitivity test performed and their results is included in Appendix E.

In conclusion, the results displayed in this chapter reflect the most up to date information available for Sam Rayburn Reservoir and B.A. Steinhagen Lake. It is also important to note that engineering judgment is best used to provide reliable answers when complex situations arise. This is to obtain the best defendable results.

10 2-Dimensional HEC-RAS Analysis of the Angelina River Basin above Alto

10.1 INTRODUCTION

The Upper Angelina River / Mud Creek watershed consists of largely rural land use with pockets of urbanization near Tyler, TX, Jacksonville, TX, Henderson, TX and other smaller communities (Figure 10.1). The watershed has approximately 1,276 square miles of drainage above the USGS streamflow gage Angelina Rv nr Alto, TX (08036500). It is part of the larger Neches River Basin in southeast Texas which eventually drains into the Gulf of Mexico.

While the Angelina Rv nr Alto gage has a period of record that dates back to 1940, NOAA Atlas 14 precipitation-frequency estimates in the upstream areas indicate that most observed storm events in this portion of the watershed have been uncharacteristically small, especially during the recent period for which radar rainfall data is available. During calibration of the hydrologic HEC-HMS model, most of the historic storm events available to be used for calibration were estimated to have an annual exceedance probability (AEP) of approximately 10% (10-year recurrence interval). Even with different calibration events of roughly the same magnitude, reaching a consensus of hydrologic parameter estimates in this portion of the study area proved to be difficult. The flatness of the watershed, the possibility of interbasin transfer near the confluence of the Upper Angelina River and Mud Creek, and the uncertainty of the storage volumes of the routing reaches all made this study area a prime candidate for additional 2-dimensional (2D) analysis.

Unit hydrograph theory is a commonly utilized method among the hydrologic community that transforms excess precipitation into runoff hydrographs. Historically, one of the acknowledged limitations of unit hydrograph theory is the assumption of linearity. This assumption implies that a watershed would have the same lag time receiving a very low intensity rain event as it would when receiving a high intensity event. The concerns with this assumption are reduced when the model can be calibrated to storms of similar intensity to the storm of primary interest (i.e., the 1% AEP or 100-yr recurrence interval). However, this is rarely the case, particularly in dam safety studies, but can also be true for events on the scale of the 1% AEP (i.e., 100-year recurrence interval) event. The Neches InFRM HEC-HMS hydrology model (covered in detail in Appendix B) uses the ModClark unit hydrograph method to transform excess rainfall into direct runoff hydrographs. In the case of the Angelina River / Mud Creek modeling domain where we are interested in estimating rare flow frequencies such as the 1% AEP event, there is considerable uncertainty when using calibrated unit hydrograph parameters derived from storm events that are much less intense. Literature indicates that the lag time (and consequently the time of concentration) of a unit hydrograph generally tend to decrease as storm intensity increases (Snyder, 1938 and Minshall, 1960). To account for the aforementioned shortcomings of unit hydrograph theory, USACE dam safety studies normally apply a 25-50% peaking factor to the unit hydrograph of the contributing area upstream of a dam per ER 1110-8-2(FR) “Inflow Design Floods for Dams and Reservoirs” (USACE, 1991). Due to the use of physically based routing routines/methods, HEC-RAS 2-D has been utilized by the USACE dam safety community to develop variable unit hydrograph parameters for different rainfall intensities (USACE RMC, 2017).

The primary purpose of this analysis is to utilize a HEC-RAS 2D model and equations to investigate the variability of unit hydrograph parameters used in the HEC-HMS model for the purpose of improving flood frequency estimates within the Angelina River / Mud Creek watershed. The 2D diffusion wave transform method in HEC-RAS, which is based on the momentum and continuity equations and is not tied to the assumption of linearity, is used to inform the ModClark unit hydrograph transform parameters in HEC-HMS particularly for rare, intense rainfall events that have not yet been observed. A secondary purpose of the 2D HEC-RAS analysis is to verify the storage volumes in the HEC-HMS routing reaches, particularly near the confluence of the Angelina River with Mud Creek where their respective floodplains merge.

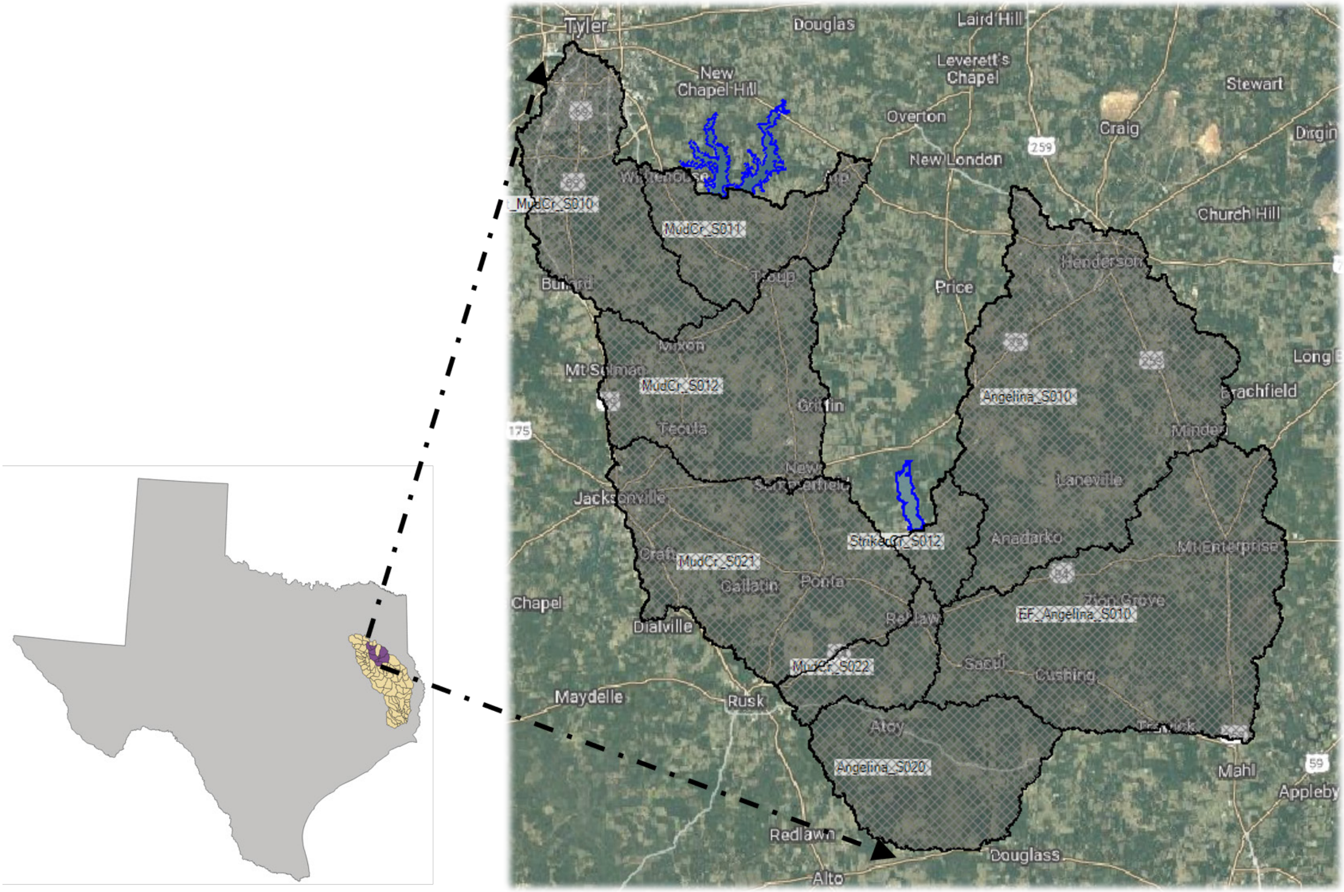


Figure 10.1: Upper Angelina River / Mud Creek Watershed and 2D Modeling Domain

This chapter summarizes the 2D HEC-RAS portion of the hydrologic analysis that was completed for the InFRM Watershed Hydrology Assessment of the Neches River Basin. Additional details on the analysis methods and results are available in Appendix F: 2-Dimensional HEC-RAS Analysis.

10.2 HEC-HMS MODEL DEVELOPMENT AND CALIBRATION

The Neches InFRM HEC-HMS model consists of subbasins that largely utilize the ModClark transform method and associated parameters (time of concentration, Tc, and storage, R) to model how excess precipitation transforms into a direct runoff hydrograph at each subbasin outlet. The Tc and R parameters in the upper portion of the Neches basin were calibrated based on several historic events including storms from the July 2007, March 2008, May 2015, March 2016, April 2016, May 2016, and August 2017 time periods. For each of the events, observed precipitation from NWS gridded hourly radar rainfall data and releases from Lake Tyler and Lake Striker were input into the HEC-HMS model. For Lake Tyler, the lake releases were calculated based on the main and auxiliary spillway rating curves and the observed lake elevations from USGS gage 08034000 Lk Tyler nr Whitehouse, TX. For Lake Striker, daily pool elevation and reservoir release data was obtained from the Angelina and Nacogdoches Counties Water Control and Improvement District Number One, which operates Lake Striker.

The losses and Tc and R parameters were then adjusted for each event until they matched the observed flows at the Mud Creek nr Jacksonville, TX (08034500) and Angelina Rv nr Alto, TX (08036500) gages. The geospatial relationships between the HEC-HMS subbasin delineations (the 2D relevant subbasins are highlighted in purple), Lake Tyler and Lake Striker, and the two USGS streamflow gages are shown in Figure 10.2. More information on the HEC-HMS model development and calibration can be found in Appendix B.

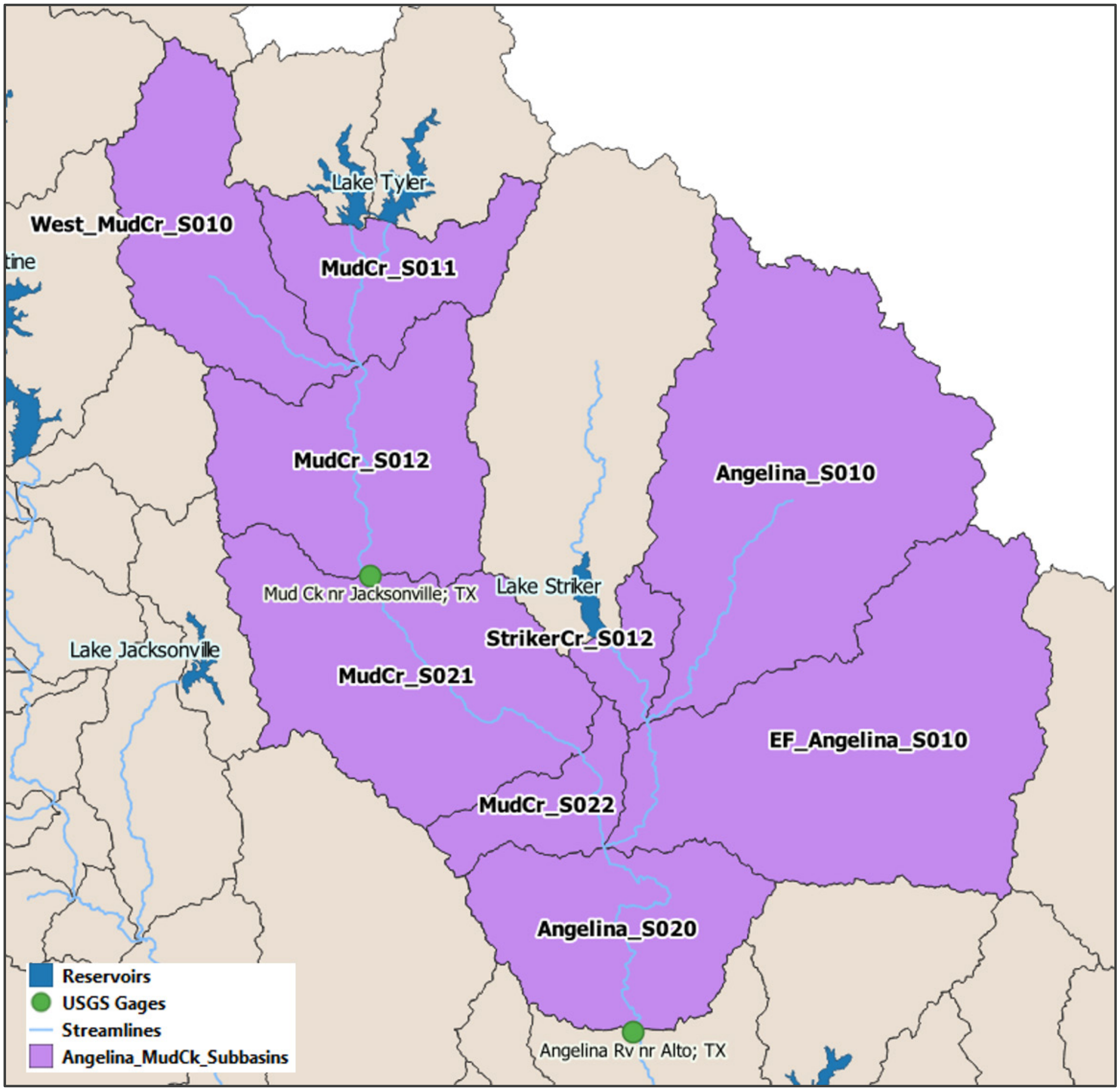


Figure 10.2: Neches InFRM HEC-HMS Model – Relevant Subbasins for the 2D Analysis

10.3 2D HEC-RAS MODEL DEVELOPMENT AND CALIBRATION

At the time of this analysis, the official HEC-RAS release version was 5.0.7. In this version, precipitation can be applied as a boundary condition to the 2D computational mesh, but losses cannot be accounted for directly in HEC-RAS (loss accounting will be a new feature starting in version 6.0). The excess precipitation applied to the HEC-RAS model was taken directly from the HEC-HMS model; other inputs to the HEC-RAS model such as subbasin delineation and observed releases from Lake Tyler and Lake Striker (used for calibration) were also taken directly from the HEC-HMS model. The primary purpose of building the 2D HEC-RAS model was to use the 2D diffusion wave method to transform excess precipitation into runoff, everything else being the same as the HEC-HMS model for a direct comparison.

10.3.1 Terrain and 2D Computational Mesh

High resolution, 1-meter LiDAR data that covered the entire modeling domain were downloaded directly from the U.S. Geological Survey (USGS) 3DEP LidarExplorer website (USGS, 2018a). Specific LiDAR projects that were used for this analysis include TX Neches B3 2016, TX Neches B4 2016, TX Neches B5 2016, and TX Red River B3. The 1-meter terrain data led to long runtimes and cumbersome file output sizes. A sensitivity analysis comparing model runs on a 1-meter terrain versus model runs on a resampled 10-foot terrain showed a negligible difference in computed values throughout the 2D model. Therefore, the resampled 10-foot terrain dataset was chosen due to computational efficiency and more manageable file output sizes. The projected coordinate system used for the 2D modeling was USA Contiguous Albers Equal Area Conic USGS version in feet.

A total of nine HEC-RAS 2D flow areas were created with perimeters that exactly matched the subbasin delineations used in HEC-HMS. Next, a 2D computational mesh was developed with 500-foot cell sizes throughout most of the model domain. A stream centerline file as well as a 2019 roadway inventory shapefile representing TX-DOT (Texas Department of Transportation) roads were inserted as break lines. The cells were snapped to the break lines and the cell sizes near the break lines were decreased to between 200 and 300-feet for increased resolution near areas of rapid relief change. Altogether there were 5,167 break lines and 158,410 cells in the model.

Flows were transferred from each 2D flow area to adjacent/downstream flow areas via the use of 2D Area connections. These connections modeled flow with the broad crested weir equation. The input weir station-elevation data were derived from the ground elevations of the terrain near the subbasin outlets. Care was taken to choose weir coefficients that resulted in headwater and tailwater differences of less than one-foot while also maintaining computational stability. An overall view of the terrain and 2D computational mesh can be seen in Figure 10.3.

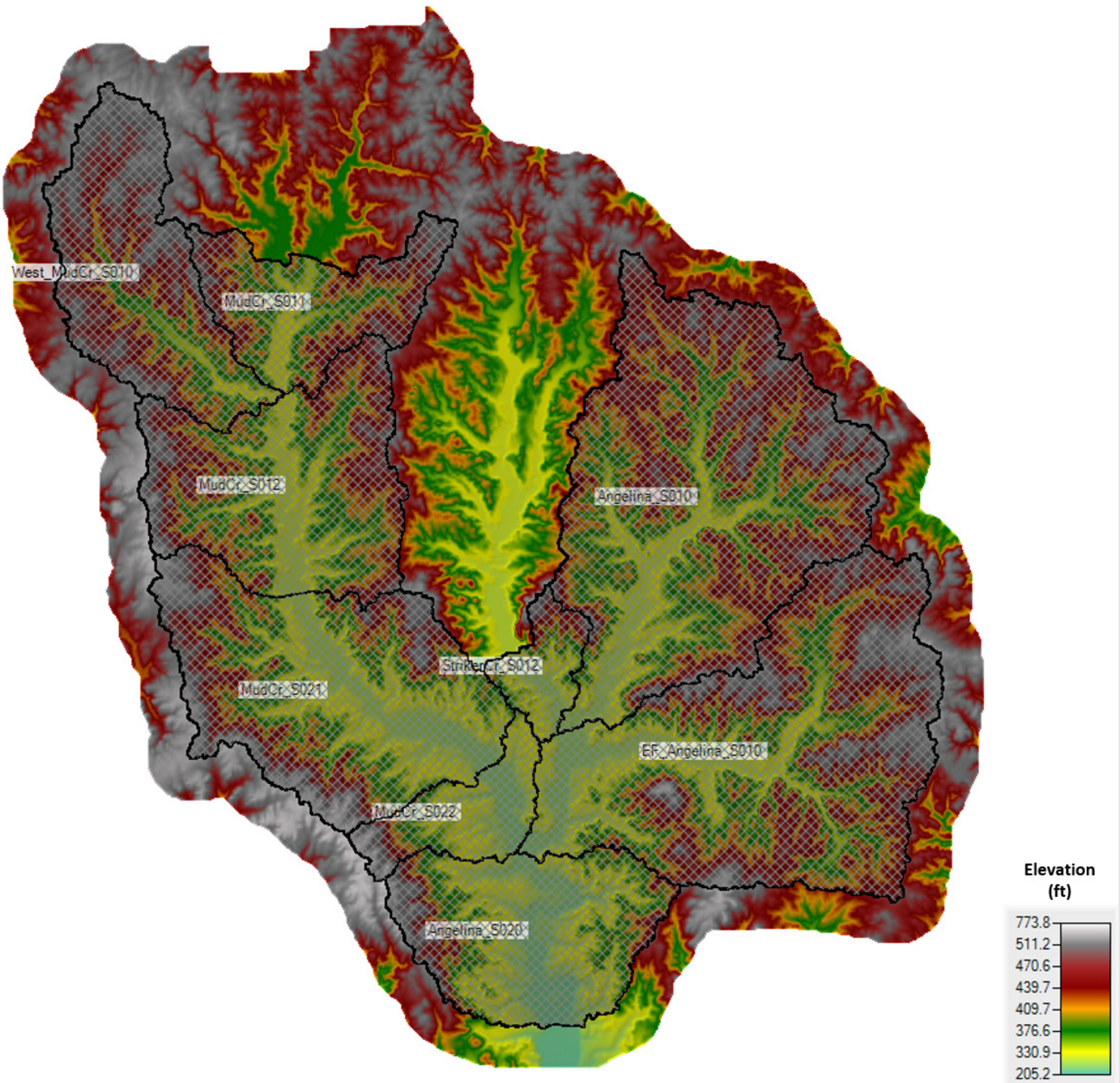


Figure 10.3: Overall Terrain Model and 2D Computational Mesh with Nine Flow Areas

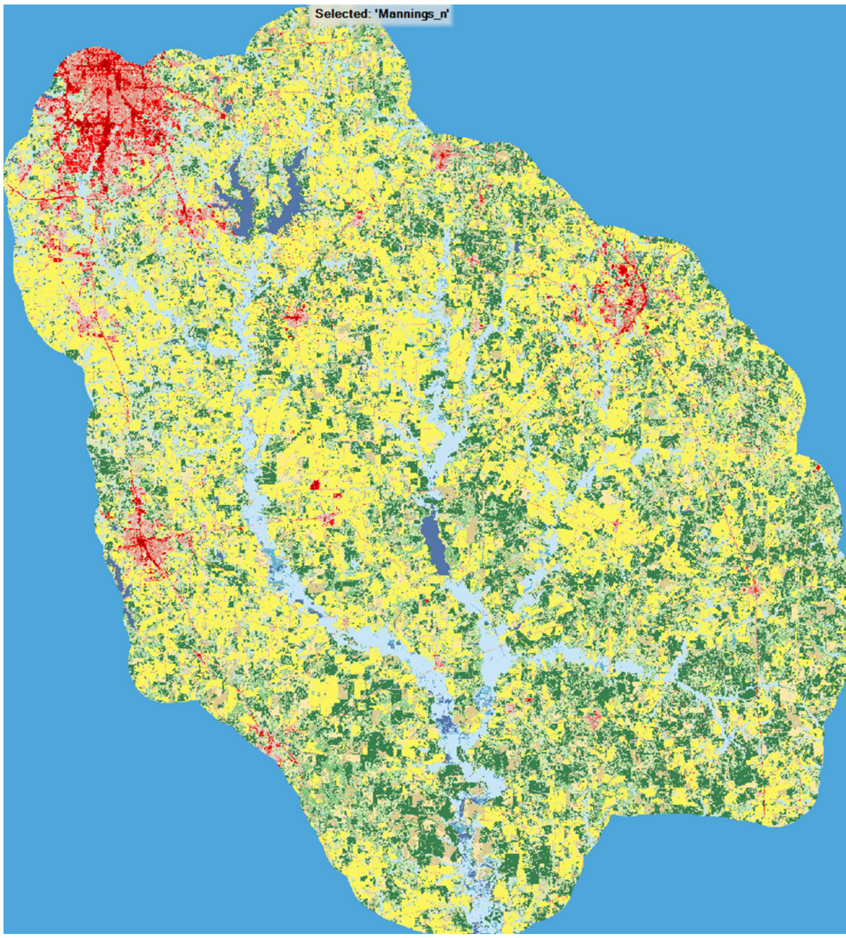
10.3.2 Boundary Conditions

For each of the nine 2D flow areas, excess precipitation from the HEC-HMS model (losses removed) was applied as a precipitation boundary condition. Furthermore, for calibration and uniform storm frequency runs in the 2D HEC-RAS model, observed and computed (from HEC-HMS) releases from Lake Tyler and Lake Striker were applied as upstream flow hydrograph boundary conditions.

A rating curve was used as the downstream boundary condition. The curve was developed from the latest available USGS data for the Angelina Rv nr Alto gage (08036500). The rating curve was extended from approximately 247.33-feet out to 250-feet (NAVD88) via linear extrapolation. This was necessary to model rarer frequency events than have been observed in this part of the watershed. A discrepancy was noted between the lower end of the rating curve and the channel elevations of the terrain dataset near the location of the gage. According to the rating curve, flow does not register until an elevation of approximately 222-feet. However, a channel cross-section near the gage shows terrain elevations of around 212-feet near the bottom of the channel. This has the unintended consequence of artificially dampening the downstream model response during low, in channel flows. However, this discrepancy does become less noticeable and less significant during large, out-of-bank flows such as the rare frequency storm events that were modeled during this study effort.

10.3.3 National Land Cover Database (NLCD)

2016 National Land Cover Database (NLCD) data were imported into RAS as a Land Cover Layer to establish initial Manning's 'n' estimates based on land cover categories (Figure 10.4). Woody wetlands and pasture/hay were the dominant land use types within the Angelina and Mud Creek floodplains; therefore, they are the categories that had the greatest impact on the routing of flows within the floodplain.



Color	Value	Name	Default Manning's n
	0	nodata	
	1	255	
	11	open water	0.05
	21	developed, open space	0.05
	22	developed, low intensity	0.055
	23	developed, medium intensity	0.065
	24	developed, high intensity	0.075
	31	barren land rock/sand/clay	0.055
	41	deciduous forest	0.21
	42	evergreen forest	0.21
	43	mixed forest	0.21
	52	shrub/scrub	0.12
	71	grassland/herbaceous	0.11
	81	pasture/hay	0.07
	82	cultivated crops	0.075
	90	woody wetlands	0.12
	95	emergent herbaceous wetla...	0.12

Figure 10.4: Angelina River – Mud Creek: NLCD Categories

10.3.4 2D HEC-RAS Calibration

The April 2016 event was used to calibrate the 2D HEC-RAS model. This same event was also used as part of the HEC-HMS model calibrations. Observed release data from Lake Tyler and Lake Striker were used as flow hydrograph inputs at upstream boundary locations. A large, uniform 10-inch rain event was first applied to the model to fill in any pits in the terrain. This precursor event served two purposes: 1) it ensured that the vast majority of the excess precipitation would drain to the outlet of the 2D model, and 2) it provided baseflow within the channel that closely matched observed baseflow levels at two USGS gages just prior to the April 2016 event. At the end of the precursor event, a “hotstart” or “restart” file was created. This was used to establish initial conditions for the calibration run where excess precipitation from the April 2016 event was applied to each 2D flow area. A graphical schematic of the calibration setup can be seen in Figure 10.5.

A sensitivity analysis was done during the calibration runs to compare how the 2D diffusion wave equations performed against the 2D full momentum equations in terms of results and performance. The 2D diffusion wave equations are simplified equations that do not account for local acceleration (changes in velocity over time) and convective acceleration (changes in velocity over distance), which sometimes can play a significant role in a flat sloping river system. Comparisons at several locations throughout the 2D model showed very little difference in computed depths and arrival times between the two equation sets. Since the 2D diffusion wave equation set was more stable and had shorter runtimes, it was chosen for this analysis.

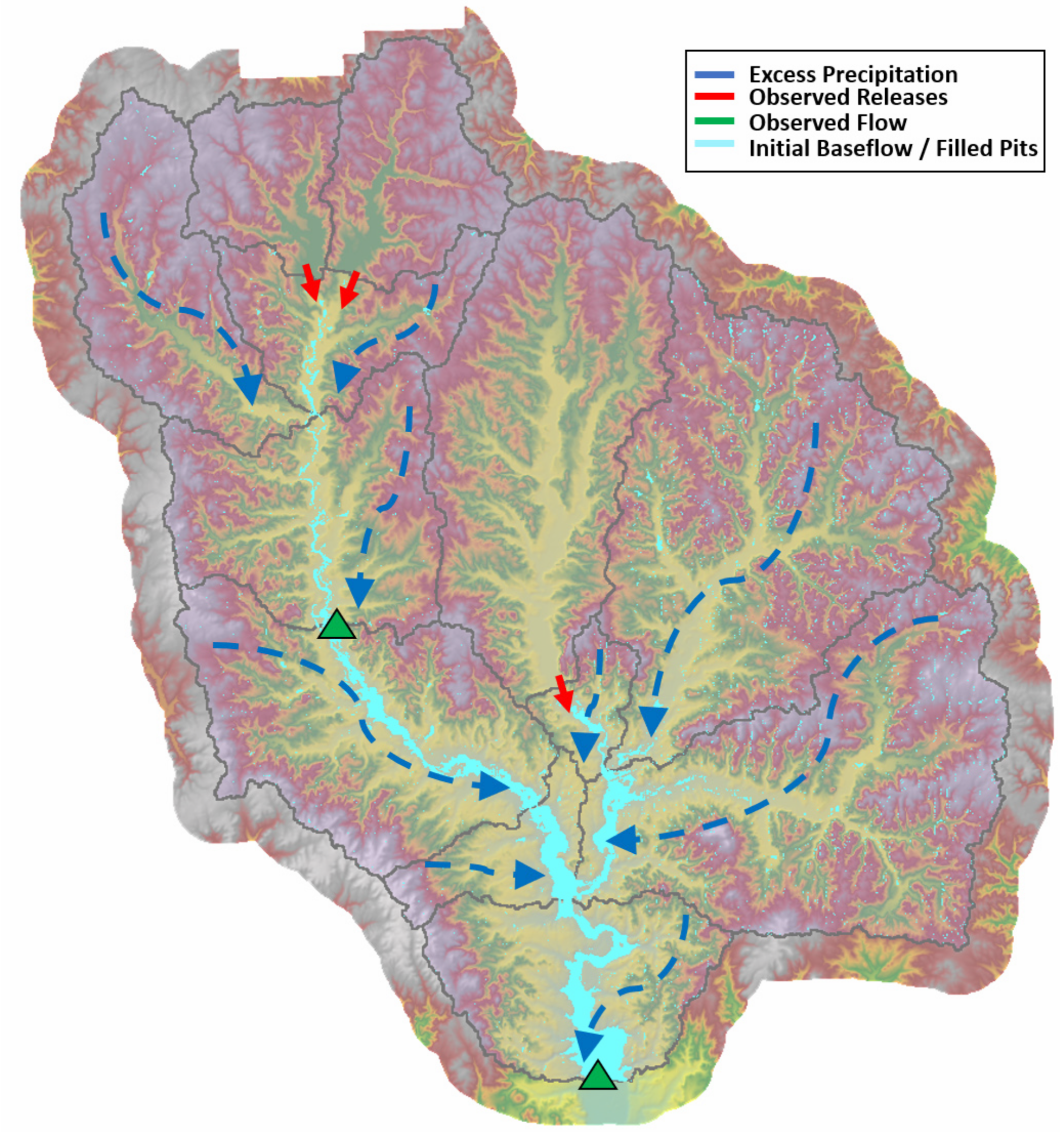


Figure 10.5: April 2016 Calibration Data Schematic

A total of nine Manning's 'n' iterations were made until computed flows largely matched the observed flows at the Mud Creek nr Jacksonville and Angelina River near Alto gages. The final calibrations included two Manning's 'n' override regions. One override region was included to increase Manning's 'n' values outside of the floodplain where areas of overland sheet flow might be expected to occur. These values were based on guidance from Technical Release 55 Urban Hydrology for Small Watersheds (USDA, 1996). Secondly, another Manning's 'n' override region was specified within the floodplain of the Upper Angelina River. This reach of the model needed higher 'n' values than the Mud Creek floodplain to better match the timing of the observed flow data. The final calibrated 'n' values and overland zones can be seen in Table 10.1 and Figure 10.6.

Once the Manning's 'n' values were finalized, excess precipitation and observed flows from a secondary, June 2007 event were input into the 2D model for a validation run. The June 2007 computed flows and stages at the gaged locations also closely matched the observed data.

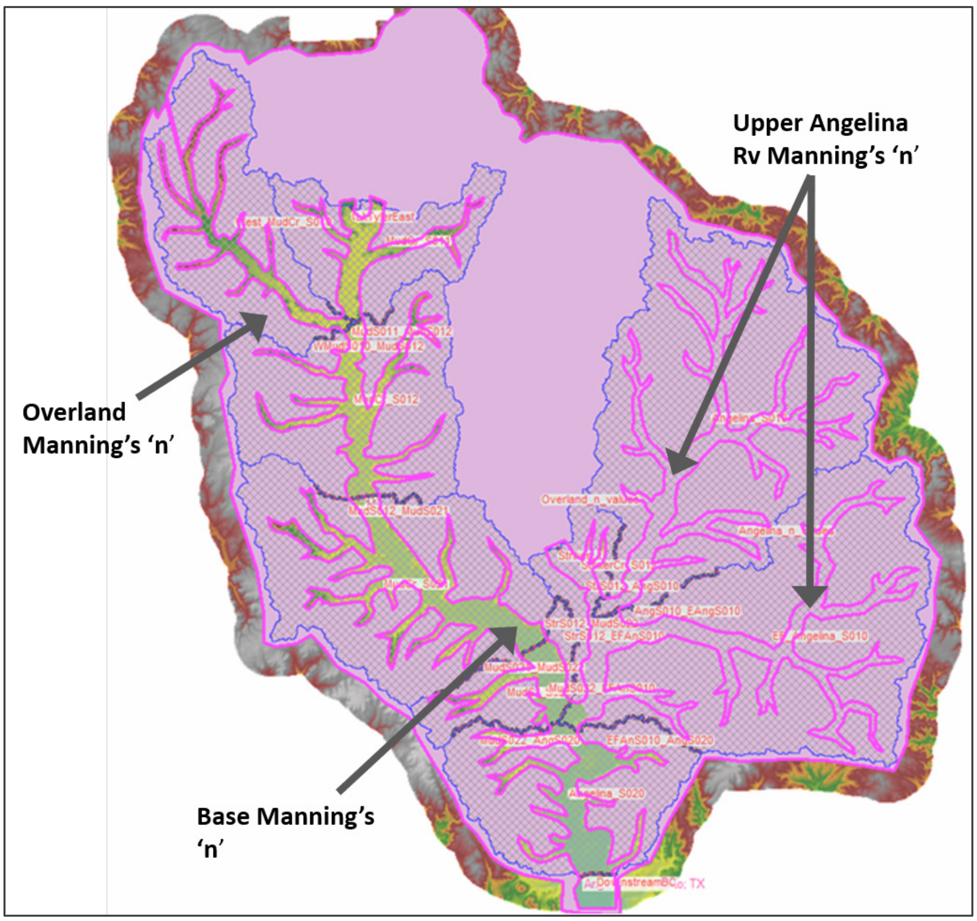


Figure 10.6: Calibrated Manning's 'n' Override Regions

Table 10.1: Final Calibrated Manning's 'n' Values

Land Use Category	Base Mann 'n'	Overland Mann 'n'	Upper Angelina Mann 'n'
Barren Land Rock/Sand/Clay	0.055	--	0.0825
Cultivated Crops	0.075	0.12	0.1125
Deciduous Forest	0.21	0.6	0.315
Developed, High Intensity	0.075	--	0.1125
Developed, Low Intensity	0.055	--	0.0825
Developed, Medium Intensity	0.065	--	0.0975
Developed, Open Space	0.05	--	0.075
Emergent Herbaceous Wetlands	0.12	0.2	0.18
Evergreen Forest	0.21	0.6	0.315
Grassland/Herbaceous	0.11	0.2	0.165
Mixed Forest	0.21	0.6	0.315
Open Water	0.05	--	0.075
Pasture/Hay	0.07	0.2	0.105
Shrub/Scrub	0.12	0.2	0.18
Woody Wetlands	0.12	0.2	0.18

10.3.5 Model Calibration Metrics and Results

In addition to simple graphical comparisons comparing simulated to observed flow hydrographs, statistical tests were also employed in evaluating model performance. The statistical metrics used to evaluate the 2D HEC-RAS model performance included the Nash-Sutcliffe Efficiency (NSE), the Root Mean Square Error – Observed Standard Deviation Ratio (RSR), and the Percent Bias (PBIAS).

The model performance metrics were evaluated for the model calibration and validation effort. For the purposes of this analysis, performance statistics are evaluated using performance ratings which are consistent with standard practice in industry (Moriasi, 2007). A summary of flow performance statistics for the April 2016 calibration event and for the June 2007 validation event are included below in Table 10.2. More details on these performance metrics and ratings are available in Appendix F.

Table 10.2: Summary of Flow Performance Statistics and Ratings for Simulated Events

Event	NSE	RSR	PBIAS	Performance Rating
April 2016 Calibration Mud Creek nr Jacksonville	0.9675	0.1802	4.0102	Very Good
April 2016 Calibration Angelina Rv nr Alto	0.9394	0.2463	-7.7426	Very Good
June 2007 Validation Mud Creek nr Jacksonville	0.8758	0.3524	-15.2486	Good
June 2007 Validation Angelina Rv nr Alto	0.9703	0.1724	7.0854	Very Good

Figures 10.7 and 10.8 show comparisons of the modeled versus observed flow hydrographs for the April 2016 calibration event. Figures 10.9 and 10.10 show comparisons of the modeled versus observed flow hydrographs and peak stage for the June 2007 validation event.

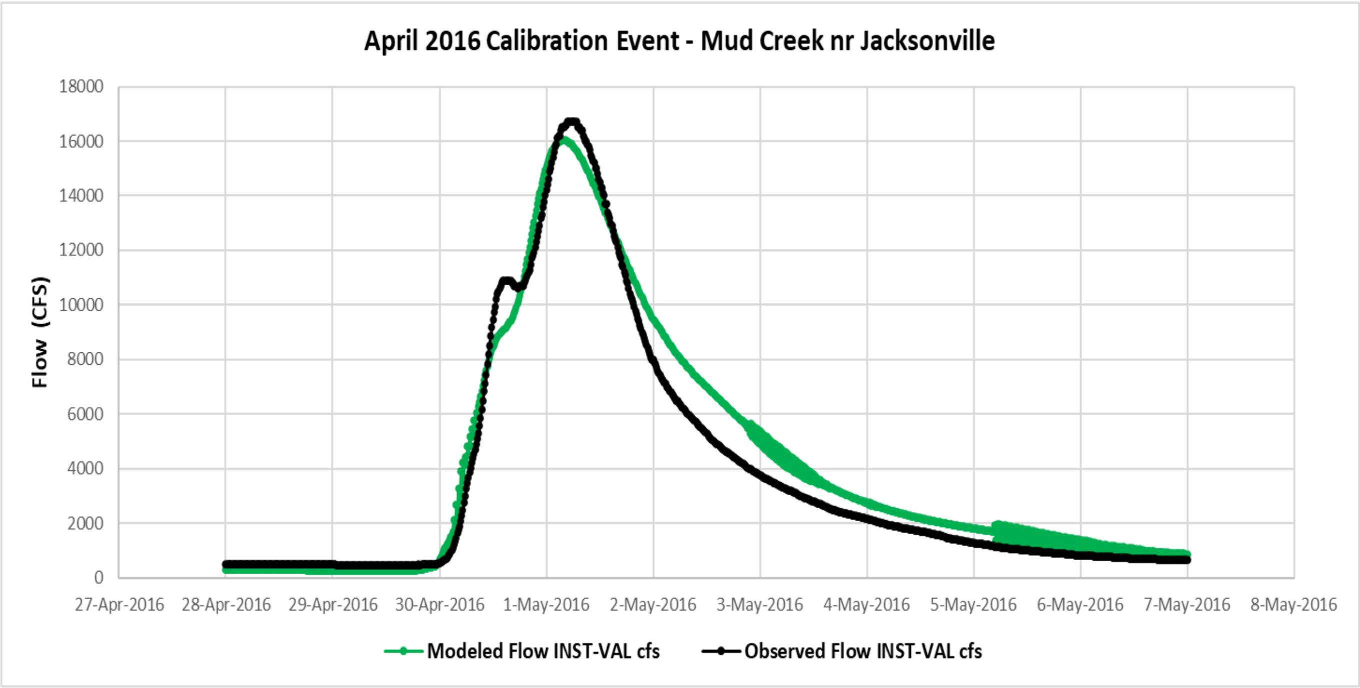


Figure 10.7: April 2016 Modeled vs Observed Flows – Mud Creek nr Jacksonville

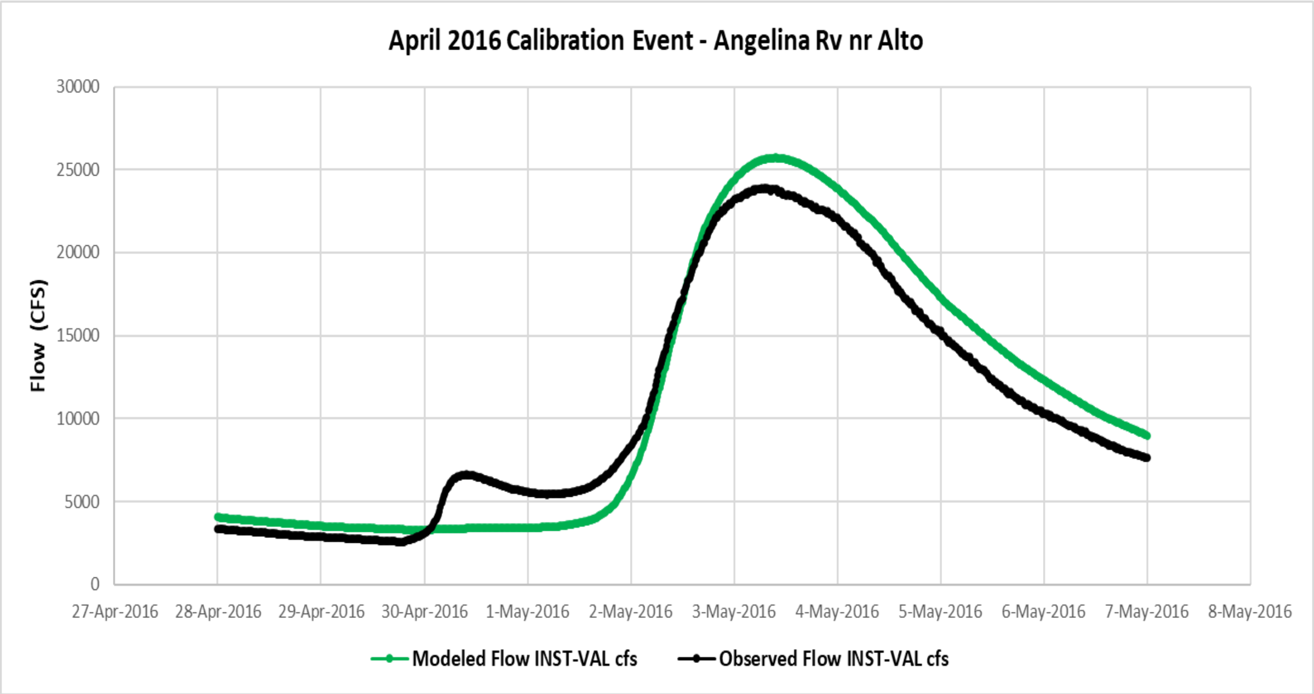


Figure 10.8: April 2016 Modeled vs Observed Flows – Angelina Rv nr Alto

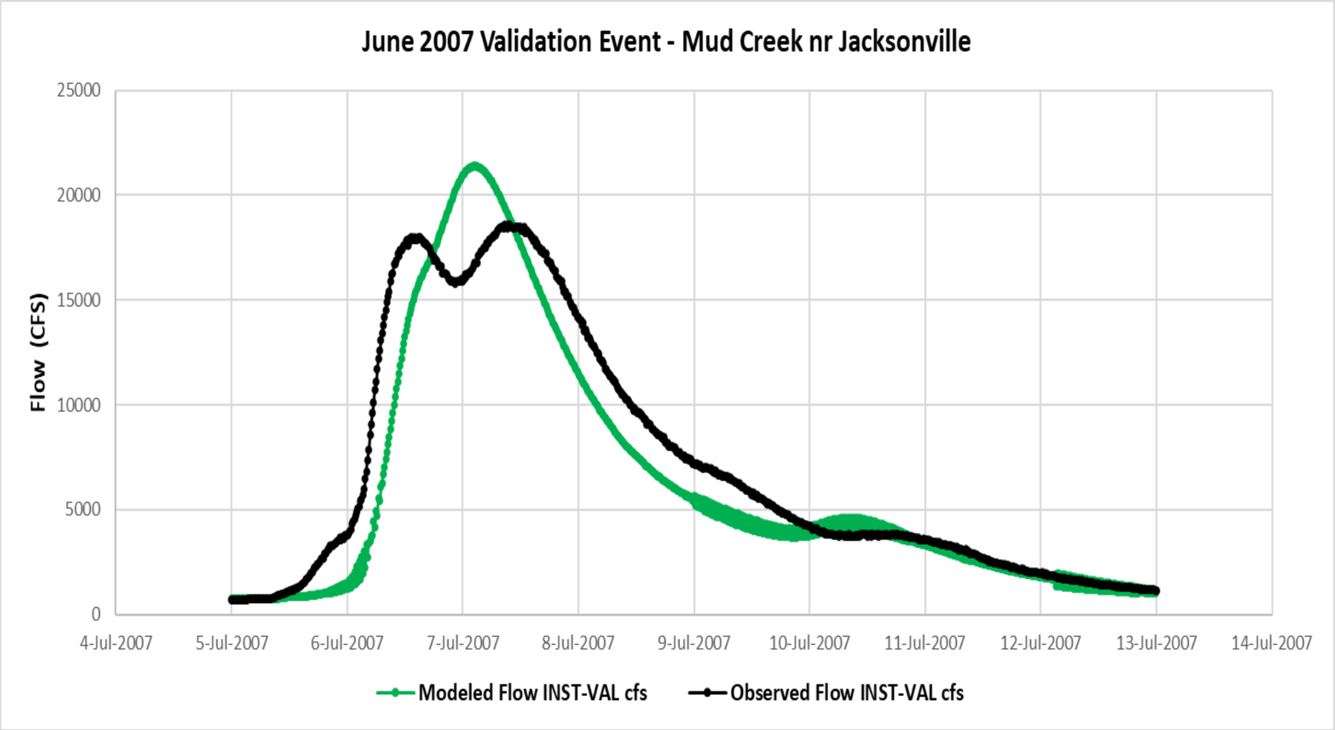


Figure 10.9: June 2007 Modeled vs Observed Flow – Mud Creek nr Jacksonville

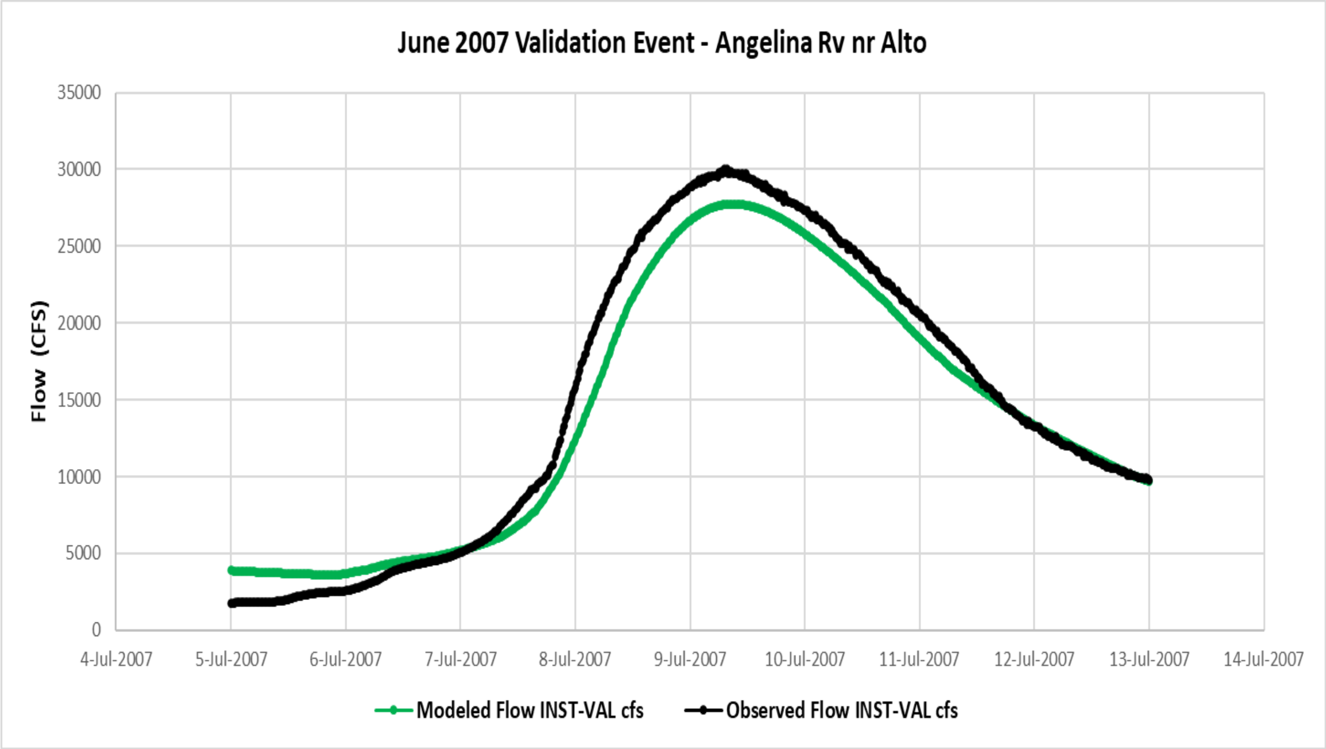


Figure 10.10: June 2007 Modeled vs Observed Flow – Angelina Rv nr Alto

10.4 2D FREQUENCY ANALYSIS AND RESULTS

10.4.1 2D Transform Frequency Runs

After the HEC-RAS 2D diffusion wave model was calibrated and validated, various frequency runs were set up. Similar to the calibration runs, a large, uniform 10-inch rain event was first applied to the model to fill in any pits in the terrain; at the end of the precursor event, a “hotstart” or “restart” file was created. This restart file differed from the calibration restart file in that baseflow in the main channel was minimized as much as possible so that all flow at each subbasin (2D flow area) outlet could be associated with direct runoff. Plans for the 5-year, 25-year, 50-year, and 100-year events were created and executed for each subbasin. The applied excess precipitation data were based on NOAA Atlas 14 depth estimates and were taken directly from the InFRM HEC-HMS model with losses already accounted for.

While time of concentration values and storage values (Tc and R) are not directly computed by HEC-RAS, they can be estimated by iteratively adjusting the ModClark Tc and R parameters in HEC-HMS until the ModClark transform hydrographs match the 2D transform hydrographs. Figure 10.11 demonstrates how the estimated 2D time of concentration values decrease with increasing intensity. An example of the final frequency transform hydrographs for a subbasin is shown in Figures 10.12. From this figure, one may observe that the runoff travels faster and peaks sooner for the larger, more intense frequency events, which is consistent with expectations. Additional subbasin results are available in Appendix F.

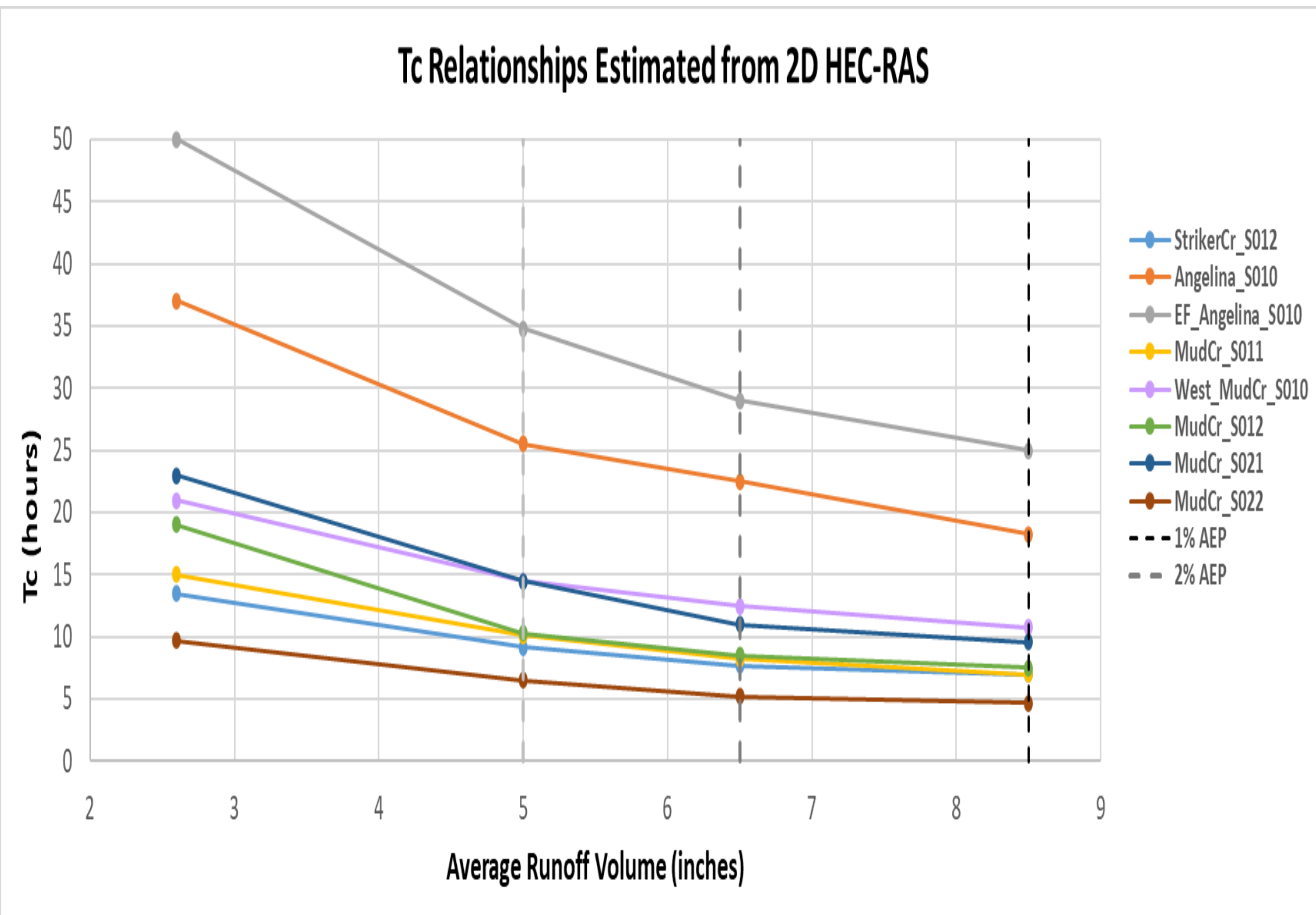


Figure 10.11: Time of Concentration Estimates from 2D HEC-RAS

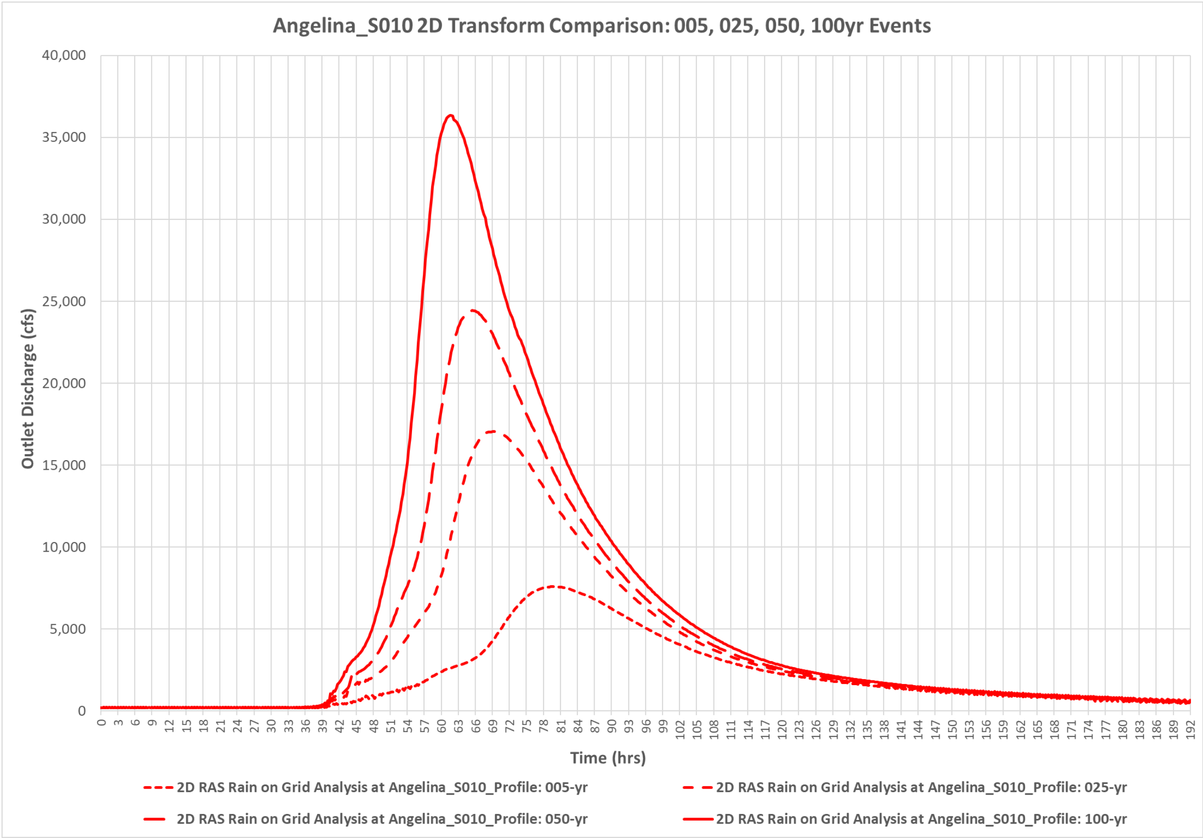


Figure 10.12: Example Subbasin 2D Frequency Results for Angelina_S010

10.4.2 2D Routing Analysis

Prior to this 2D analysis, the HEC-HMS modified-puls routing reaches within the Angelina River and Mud Creek watersheds used storage-discharge curves that were generated from a 1D HEC-RAS model. During calibration of the HEC-HMS model, there was speculation that the routing reaches, particularly for the Upper Angelina River, may not be capturing the full storage potential of the floodplain.

In addition to the transform analysis, the calibrated HEC-RAS 2D diffusion wave model was used to generate updated storage discharge curves for five routing reaches (MudCr_R022, MudCr_R031, MudCr_R032, Angelina_R010, and AngelinaR020) which corresponded to five of the subbasins within the 2D modeling domain. For each of these five subbasins, a steady flow hydrograph was applied as an upstream boundary condition and normal depth was applied as the downstream boundary condition. A range of steady flows from 200 to 200,000 cfs were applied to the subbasins, and at the end of each simulation the ending volume was recorded from the .bco HEC-RAS output file. Results of the routing analysis are available in Figure 10.13. For most reaches, the 2D analysis resulted in a 5 to 10 percent increase in storage volume over the 1D HEC-RAS analysis, but for the Angelina River reach just above the confluence with Mud Creek (Angelina_R010), the 2D analysis storage volumes were 20 to 25 percent higher than the 1D storages. These increases in storage are due to the 2D model's ability to better capture the full volume of the floodplain.

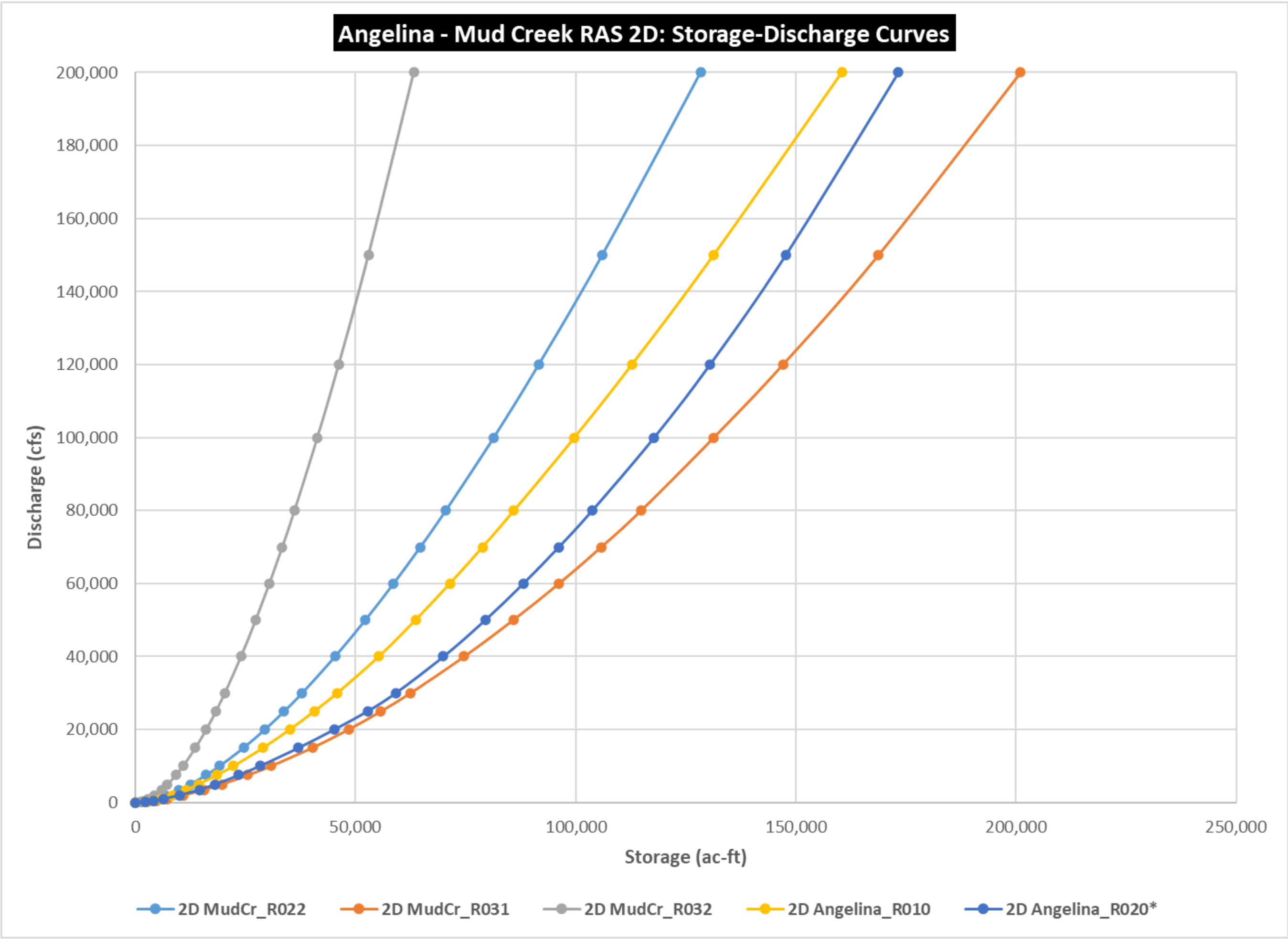


Figure 10.13: 2D HEC-RAS Storage-Discharge Curves

10.4.3 2D-Informed Updates to the InFRM HEC-HMS Model

The five storage-discharge curves computed from the 2D HEC-RAS model were adopted in the final InFRM HEC-HMS model as parameters to the mod-puls routing reaches in the Upper Angelina River watershed. These curves replaced those computed from a 1D HEC-RAS model that were previously applied to this portion of the watershed. As discussed in the prior section, the 2D curves indicate notably more storage than what was previously modeled in the Upper Angelina.

After updating the routing reaches, the 2D transform results were used to update the ModClark transform parameter estimates in HEC-HMS. It is important to note that the 2D diffusion wave equations in HEC-RAS have no direct output of Time of Concentration and Storage (Tc and R) parameters which are specific to the HEC-HMS ModClark transform method. However, the peak magnitude, peak timing, and overall shape of the 2D transform hydrographs can be used to inform Tc and R parameters. The Tc and R parameters were adjusted in HEC-HMS until the ModClark transform hydrographs more closely matched the 2D transform hydrographs. While adjusting the Tc and R parameters in HEC-HMS, R/(Tc+R) ratios were constrained to fall between the values of 0.5 and 0.7 which are commonly observed in this area of East Texas. Table 10.3 shows a comparison of the preliminary HEC-HMS transform parameters, the 100-yr estimated transform parameters from the 2D analysis, and the final adopted InFRM HEC-HMS transform parameters for each Upper Angelina / Mud Creek subbasin. The final adopted HEC-HMS transform parameters were adjusted to (1) keep a consistent ratio of R to Tc across the watersheds, and (2) to closely match the peak frequency flows results from 2D HEC-RAS at the stream gages. The

InFRM Uniform HEC-HMS model was then recomputed with the adopted 2D storage-discharge curves and the adopted Tc and R parameters.

Table 10.3. Subbasin Tc and R Comparison Table

	Preliminary Calibrated HEC-HMS Parameters			Calibrated 2D HEC-RAS 100yr Estimated Parameters			Final Adopted HEC-HMS Parameters		
Subbasin	Tc (hrs)	R (hrs)	R/(Tc+R)	Tc (hrs)	R (hrs)	R/(Tc+R)	Tc (hrs)	R (hrs)	R/(Tc+R)
StrikerCr_S012	20.7	34.57	0.63	7	13	0.65	7	13.86	0.66
Angelina_S010	46.18	77.12	0.63	18.25	23.5	0.56	24.5	48.51	0.66
EF_Angelina_S010	47.83	79.88	0.63	25	35	0.58	31	61.38	0.66
MudCr_S011	11.12	14.9	0.57	7	12	0.63	8	13.36	0.63
West_MudCr_S010	18.21	24.4	0.57	10.75	18	0.63	12	20.04	0.63
MudCr_S012	9.18	15.33	0.63	7.5	11.8	0.61	8.5	14.20	0.63
MudCr_S021	27.89	46.58	0.63	9.6	26	0.73	12	23.76	0.66
MudCr_S022	21.44	35.8	0.63	4.7	17	0.78	5.5	10.89	0.66

Figures 10.14 and 10.15 compare the final adopted HEC-HMS 1% AEP (100-yr) peak flow results with the 2D HEC-RAS model results and the preliminary InFRM Uniform HEC-HMS model results at the two gages. In general, the adopted ModClark and routing parameters result in earlier and slightly higher peaks at the two gages and come closer to the 2D HEC-RAS results.

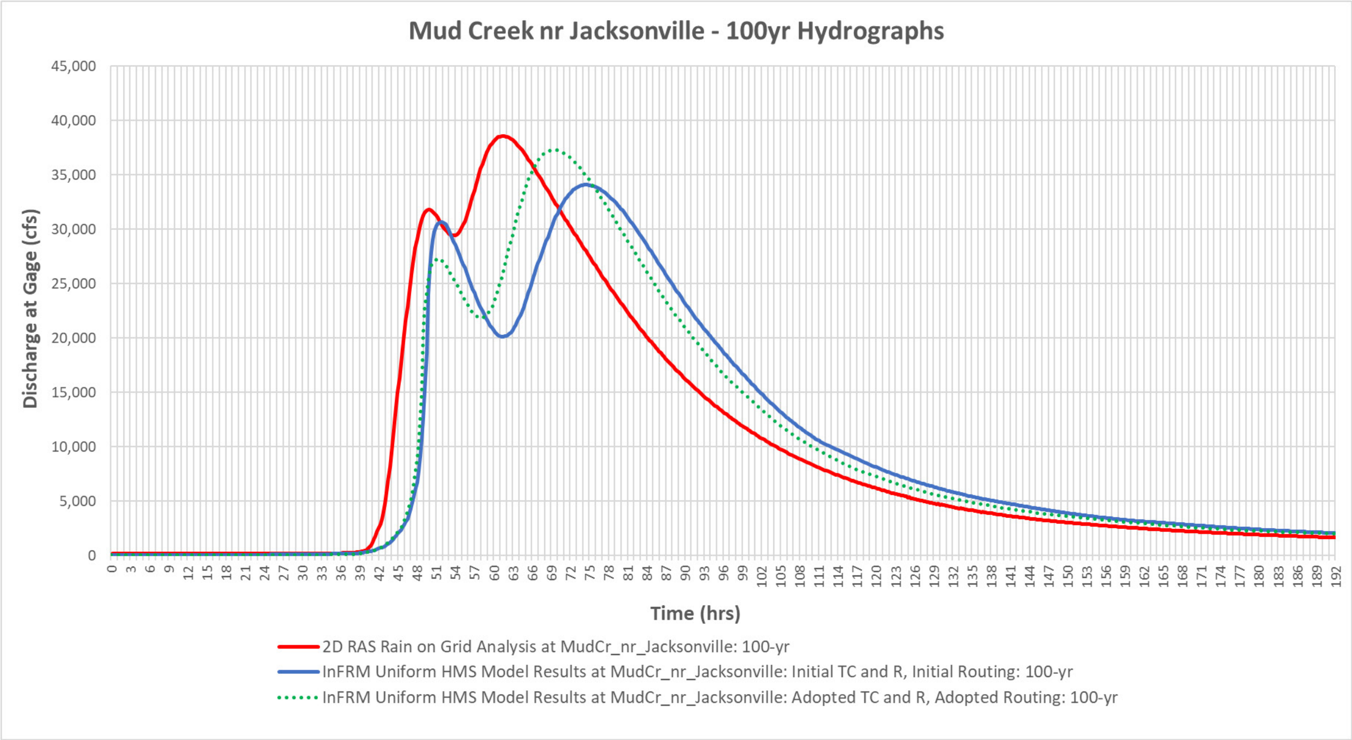


Figure 10.14: Mud Creek nr Jacksonville 100yr Flow Comparison

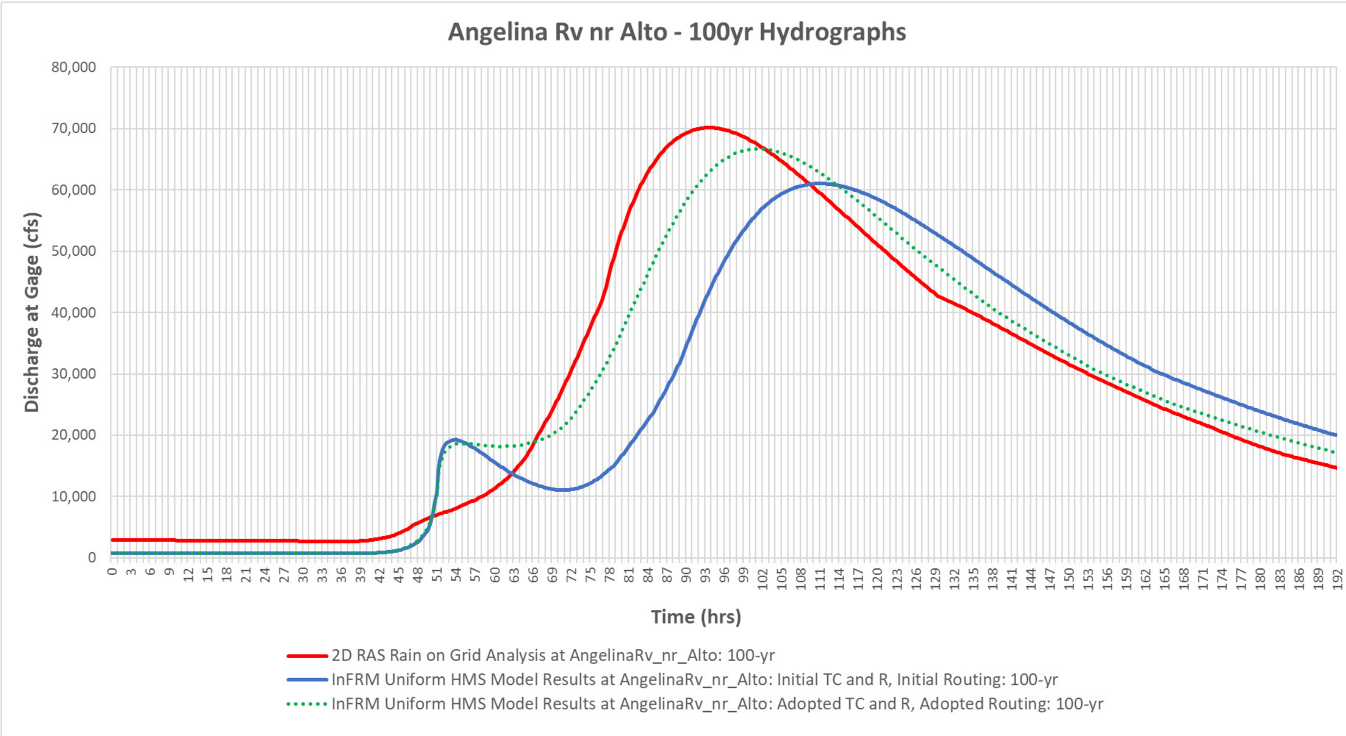


Figure 10.15: Angelina Rv nr Alto 100yr Flow Comparison

10.5 CONCLUSIONS

One of the acknowledged limitations of unit hydrograph theory is the assumption of linearity, which implies that a watershed would have the same time of concentration when receiving a very low intensity rain event as it would when receiving a high intensity rainfall event. Concerns with this assumption can be reduced by calibrating the model to storms of similar intensity to the storm of primary interest (i.e., the 1% AEP or 100-yr recurrence interval). However, for the Angelina River Basin above Alto, NOAA Atlas 14 precipitation-frequency estimates indicated that the recent storm events available for calibration were uncharacteristically small, having an AEP of approximately 10% (a 10-year recurrence interval).

In this analysis, the 2D diffusion wave transform method in HEC-RAS, which is based on the momentum and continuity equations and is not tied to the assumption of linearity, was used to inform the ModClark unit hydrograph transform parameters in HEC-HMS particularly for rare, intense rainfall events that have not yet been observed. The results of this analysis indicated that as rainfall intensity increases within the watershed, the time of concentration decreases. In fact, the results of this analysis led to an average decrease of 45% in the ModClark times of concentration for the 1% AEP storm event on the Angelina watershed. These decreases in time of concentration generally led to higher peak discharges downstream. The results from this analysis were also consistent with those found in literature such as Snyder and Minshall (Snyder, 1938 and Minshall, 1962).

The 2D HEC-RAS analysis was also used to update the storage volumes in the HEC-HMS routing reaches of the Angelina River Basin above Alto, particularly near the confluence of the Angelina River with Mud Creek where their respective floodplains comingle. The analysis showed that 2D model of the Angelina watershed was better able to capture the full volume of the floodplain. For most reaches, the 2D analysis resulted in a 5 to 10 percent increase in storage volume over the previous 1D HEC-RAS analysis, but for the Angelina River reach just above the confluence with Mud Creek), the 2D analysis storage volumes were 20 to 25 percent higher than the 1D storages.

The results of this 2D analysis were used to update the transform and routing reach parameters in the final InFRM HEC-HMS model. This analysis helped to overcome the known limitations of unit hydrograph theory and helped to reduce the uncertainty in the flood frequency estimates of the HEC-HMS model for rare events such as the 1% AEP (100-yr) storm.

11 Comparison of Frequency Flow Estimates

As each of the hydrologic analyses was completed, their results were compared to one another in terms of frequency peak discharge estimates at the USGS stream gage locations. These comparisons of frequency flow estimates were made in table format as well as graphs of peak discharge versus probability. The estimated frequency curves from each method were plotted along with their associated confidence limits and the previous published discharges from the effective FEMA Flood Insurance Studies (FIS) or the Base Level Engineering (BLE) data for the Neches River basin. For gages where a statistical change over time plot was generated, as described in Section 5.3, the results from the other methods were also compared against the range of flow values in those graphs.

Wherever there were significant differences in the resulting flood magnitudes, the InFRM team made an effort to investigate and understand the reasons for those differences to the extent practicable. The investigation process often uncovered one or more adjustments that should be made to the assumptions in a particular method that improved the results. These adjustments may or may not have led to better agreement in the results, but at the very least, the strengths and weaknesses of each method at a particular location were more fully understood through the process of investigation.

11.1 FREQUENCY FLOW COMPARISONS

The final comparisons of the frequency flow estimates are given in Tables 11.1 to 11.16. Blank cells indicate data was not available at that specific location. Figures 11.1 through 11.16 include plots of the estimated frequency curves at each gage along with their confidence limits and the previous published discharges from the BLE data and the effective FEMA Flood Insurance Studies (FIS). Where available, the statistical change over time comparison plots are also included in the figures of this section. Additional discussion of the results is included with the figures for that location.

Table 11.1: Frequency Flow (cfs) Results Comparison for the Neches River nr Neches, TX

Annual Exceedance Probability (AEP)	Return Period (years)	Currently Effective FEMA FIS	BLE Data - Statistical Analysis	Statistical Analysis of the Observed Gage Record (56 years)	HEC-HMS Uniform Rain Frequency Storm	HEC-HMS Elliptical Frequency Storm
0.002	500		46,840	45,800	72,200	67,400
0.005	200			37,500	53,000	53,300
0.01	100		31,502	31,600	42,200	43,800
0.02	50		25,664	26,100	32,600	34,500
0.04	25		20,300	20,900	25,000	25,000
0.1	10		13,922	14,500	15,700	15,100
0.2	5			10,100	10,300	10,100
0.5	2			4,810	4,680	4,690

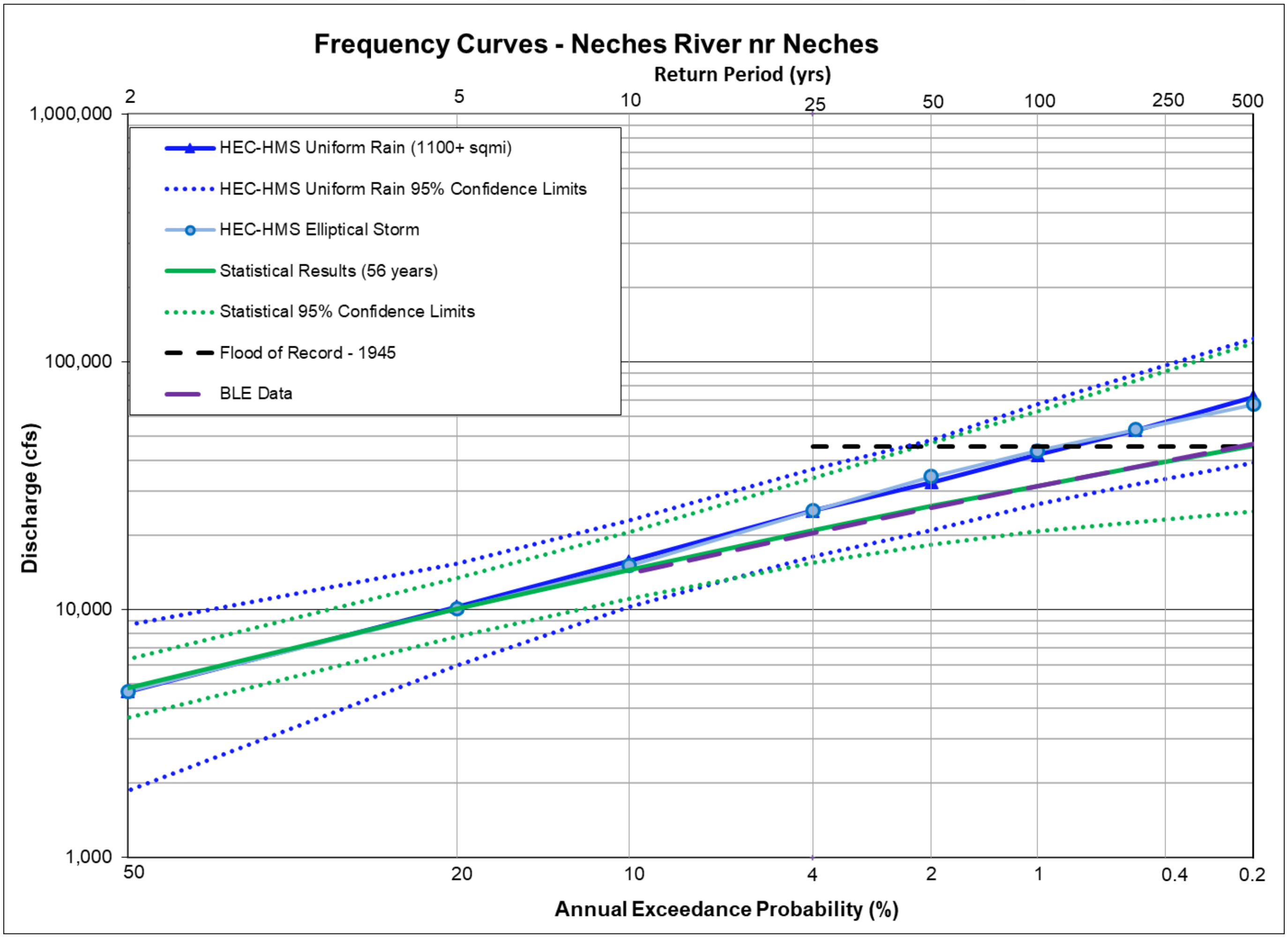


Figure 11.1a: Flow Frequency Curve Comparison for the Neches River nr Neches, TX

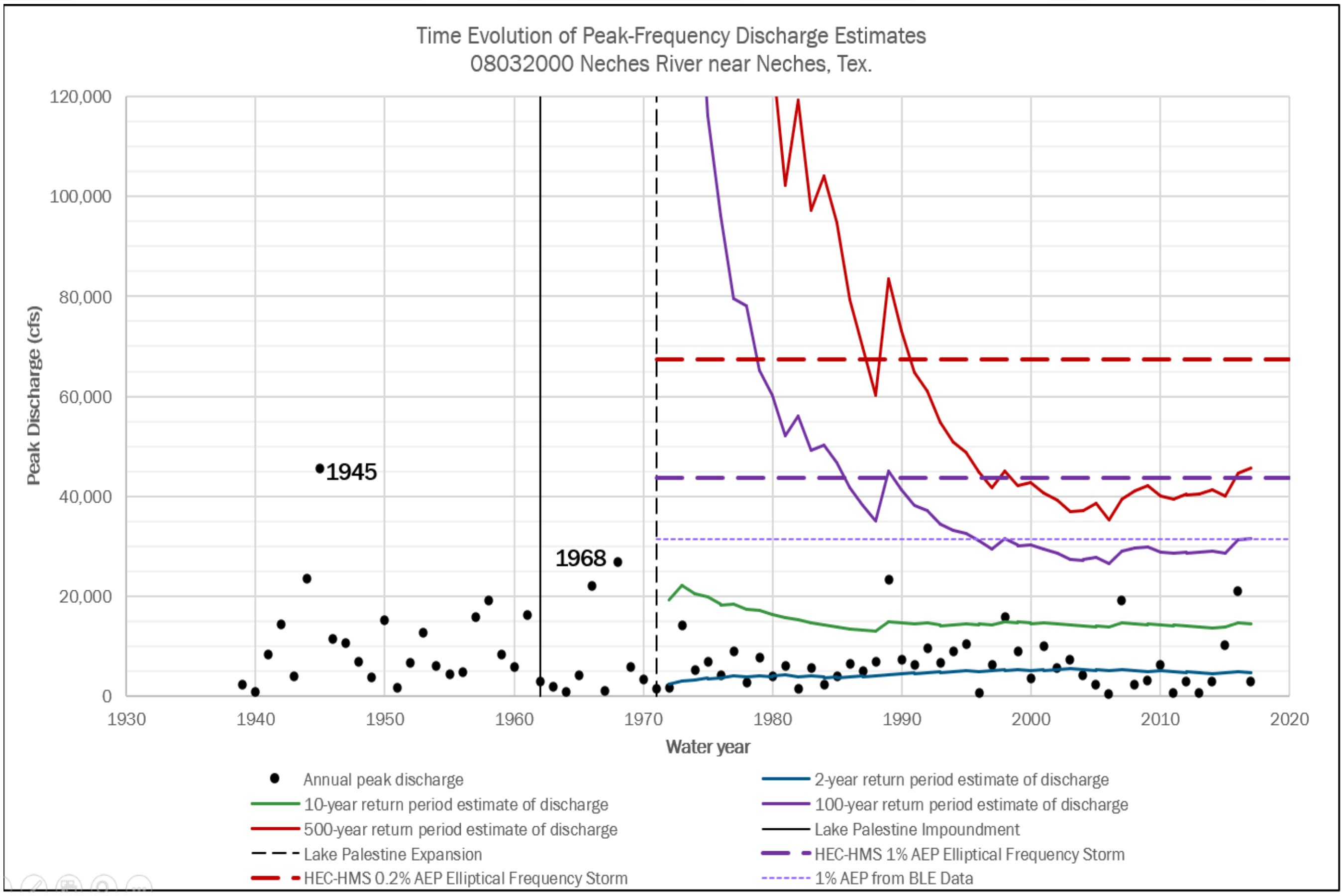


Figure 11.1b: Statistical Change Over Time Comparison for the Neches River nr Neches, TX

The Neches River near Neches, Texas is the most upstream USGS stream gage on the Neches River. It has a total drainage area of 1,145 square miles, but only 300 square miles of that is below Lake Palestine. Lake Palestine was completed in 1962, but its capacity was significantly expanded in 1971, and it has a significant dampening effect on downstream flows. The USGS gage at this location has 56 years of record, which is a moderate length of record. The published BLE data for this location was based off a statistical analysis of the gage record, which is why the BLE flows almost match the statistical analysis flows.

Figure 11.1a shows that the HEC-HMS results from both uniform rain and the elliptical storms are significantly higher than the statistical analysis of the gage record at the 1% AEP event, but still well within one another's confidence bounds. Figure 11.1b shows that while the statistical flow estimates have been relatively stable in recent decades, they are at a relative low point compared to the earlier part of the gage record. The largest flood on record occurred in 1945, and the more recent floods have been relatively small by comparison. An investigation into the historic floods for this location revealed that the April 1945 flood had rainfall totals of up to 11 inches, but was located mostly upstream of where Lake Palestine would later be constructed. The second highest event on record was May 1968, which had about 8 inches of rain, but was also located primarily upstream of Lake Palestine. The 1% AEP elliptical storm analysis, on the other hand, revealed that the critical storm centering for this location was on the 300 square miles of uncontrolled area downstream of Lake Palestine. This uncontrolled area has not experienced any large storm events similar to a 2% or 1% AEP (100-yr) event, which indicates that the current statistical record could be underestimating the 1% AEP flood potential. The 1% AEP elliptical storm resulted in a peak flow similar to the 1945 flood of record and may be a more reliable indicator of the flood potential if a large storm event were to hit this area.

Table 11.2: Frequency Flow (cfs) Results Comparison for the Neches River nr Alto, TX (at the TX-21 Bridge)

Annual Exceedance Probability (AEP)	Return Period (years)	Currently Effective FEMA FIS	BLE Data - Regression Equations	Statistical Analysis of the Observed Gage Record (35 years)	HEC-HMS Uniform Rain Frequency Storm	HEC-HMS Elliptical Frequency Storm
0.002	500		51,956	87,600	119,400	107,300
0.005	200			68,400	87,700	82,100
0.01	100		37,694	55,600	69,200	66,300
0.02	50		31,945	44,100	52,600	50,800
0.04	25		26,436	34,000	38,100	36,100
0.1	10		19,500	22,500	23,700	22,000
0.2	5			15,100	15,200	14,700
0.5	2			6,800	6,800	6,600

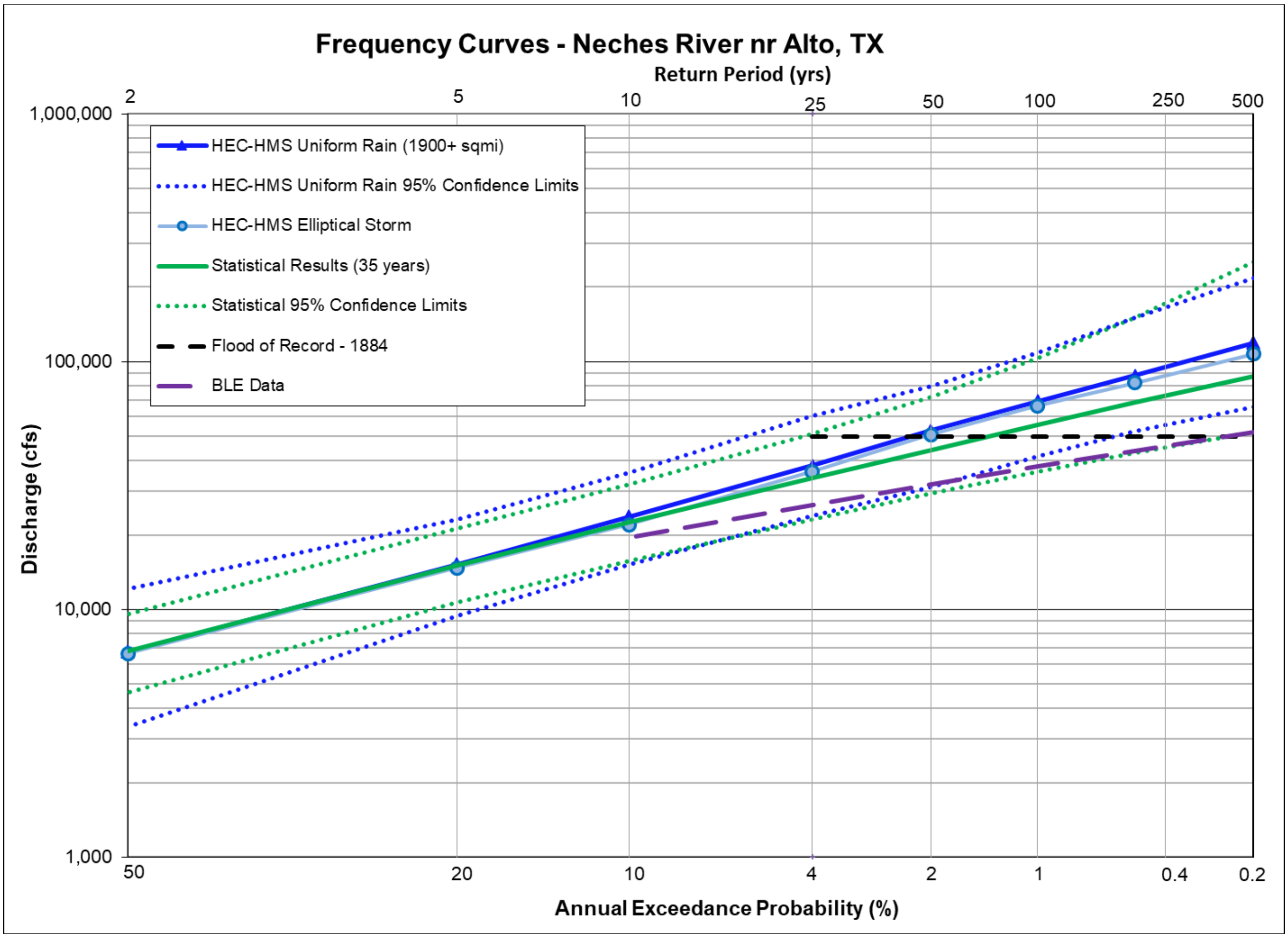


Figure 11.2: Flow Frequency Curve Comparison for the Neches River nr Alto, TX (at theTX-21 Bridge)

The Neches River near Alto, Texas is a USGS gage at the TX-21 bridge over the Neches River. It has a total drainage area of over 1,900 square miles. The USGS gage at this location has only 35 years of record. Although the gage record began in the 1940s, the gage was out of commission for almost 40 years (from 1978 – 2017). The relatively short length of record at this gage yields more uncertainty in its statistical analysis estimates. The published BLE data for this location was based off of regression equations, which is a simple, approximate method of hydrology. As shown in Figure 11.2, the BLE 1% AEP flow is at or below the 95% confidence bounds from the other analyses. This means that there is at least a 95% chance that the BLE data is underestimating the flood potential at this location for a 1% or 0.2% AEP event.

Figure 11.2 shows that the HEC-HMS results from both uniform rain and the elliptical storms are significantly higher than the statistical analysis of the gage record at the 1% AEP event, but the results are still well within one another's confidence bounds. No statistical change over time plot is available at this location due to its relatively short gage record. The flood of record for this location was the historic peak of 1884; however, not much is known about the rainfall that caused that flood. The second highest event in the record was the April 1945 flood, whose rainfall was mostly located upstream of where Lake Palestine would later be constructed. The 1% AEP elliptical storm analysis, on the other hand, revealed that the critical storm centering for this location was on the 1,000 square miles of uncontrolled drainage area downstream of Lake Palestine. Due to the relatively short record at this gage and the absence of large known flood events, the 1% AEP elliptical storm may be a more reliable estimate of the flood potential for this location.

Table 11.3: Frequency Flow (cfs) Results Comparison for the Neches River nr Diboll, TX

Annual Exceedance Probability (AEP)	Return Period (years)	Currently Effective FEMA FIS	BLE Data - Regression Equations	Statistical Analysis of the Observed Gage Record (79 years)	HEC-HMS Uniform Rain Frequency Storm	HEC-HMS Elliptical Frequency Storm
0.002	500		55,492	80,300	149,400	107,000
0.005	200			67,300	97,100	76,700
0.01	100		42,245	57,900	71,700	57,600
0.02	50		36,711	48,700	49,600	45,400
0.04	25		31,265	39,800	40,100	40,100
0.1	10		24,155	28,700	29,400	27,900
0.2	5			20,700	21,100	20,500
0.5	2			10,400	10,100	10,200

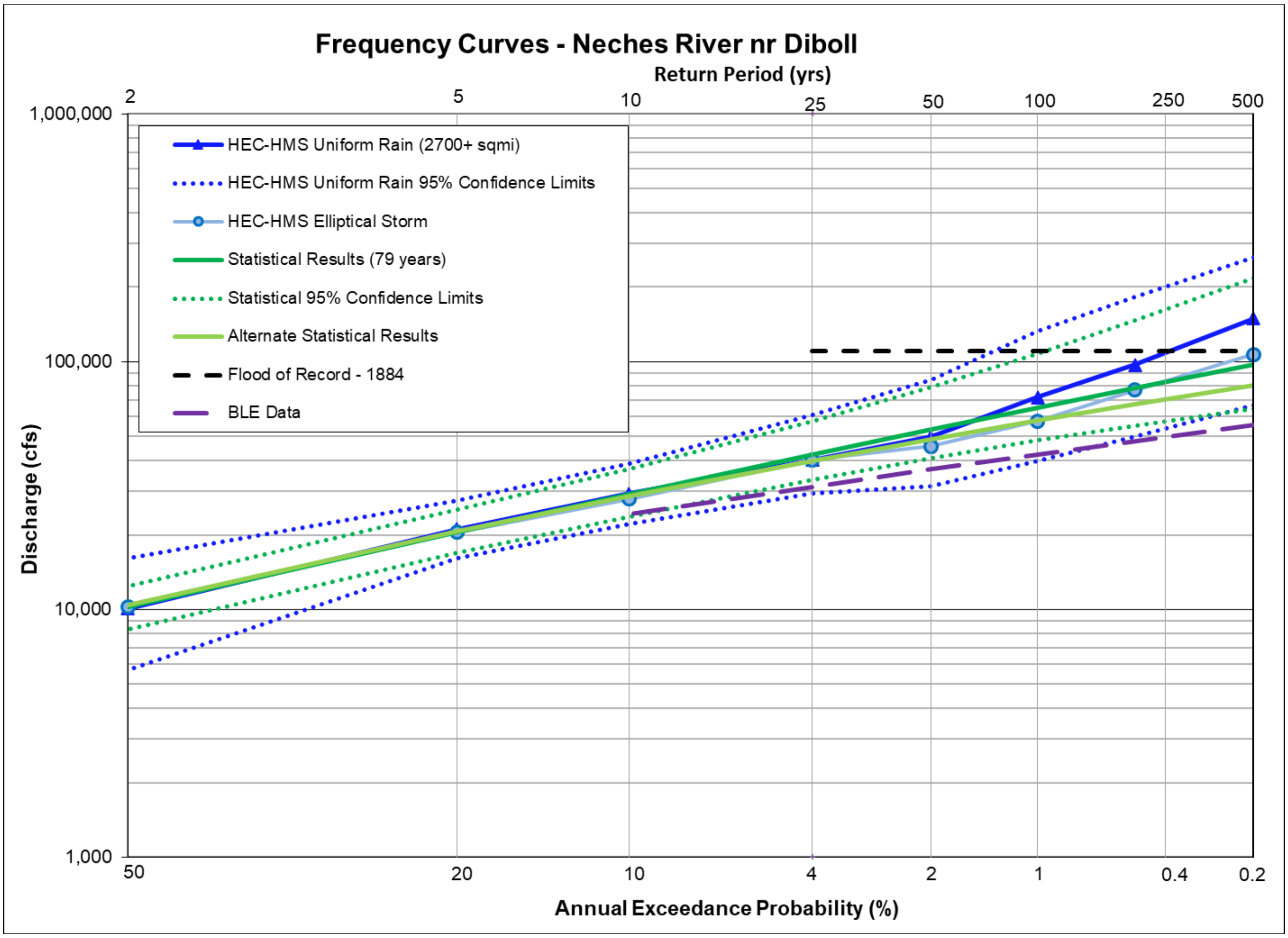


Figure 11.3a: Flow Frequency Curve Comparison for the Neches River nr Diboll, TX

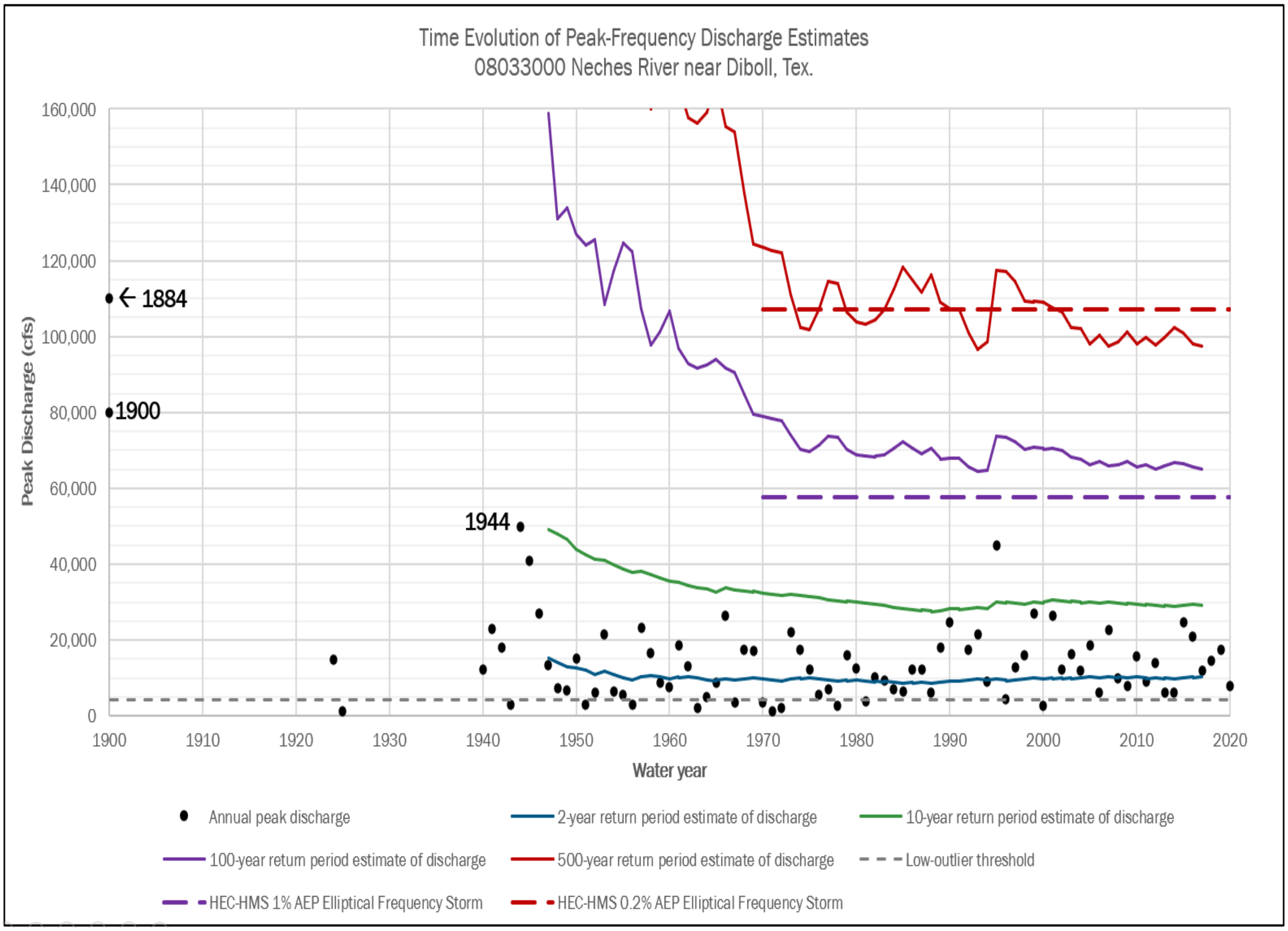


Figure 11.3b: Statistical Change Over Time Comparison for the Neches River nr Diboll, TX

The Neches River near Diboll, Texas is a USGS gage with a total drainage area of over 2,700 square miles. The USGS gage at this location has 79 years of record, which is a fairly long record for Texas. The published BLE data for this location was based off of regression equations, which is an approximate method of hydrology. As shown in Figure 11.3a, the BLE flows are at or below the 95% confidence bounds from the other analyses. This means that there is at least a 95% chance that the BLE data is underestimating the flood potential at this location for a 1% or 0.2% AEP event.

Figure 11.3a shows that there is a greater difference between the uniform rain and elliptical storm HEC-HMS results at the 1% AEP than was observed at the upstream gages. This is due to the uniform rain method's tendency to overestimate the total rainfall volume for larger drainage areas. Figure 11.3a also shows that there is a general agreement between the elliptical storm and both sets of statistical analyses results, the primary difference between the two statistical curves being the assumptions of uncertainty surrounding the 1884 flood.

The statistical change over time plot for this location (Figure 11.3b) shows that aside from a small bump in 1995, the statistical frequency estimates have been at a relative low point for the past four decades. This is because the largest floods on record occurred during the historic period prior to the systematic gage record. The flood of record for this location was the historic peak of 1884; however, not much is known about the rainfall that caused that flood. Another historic flood event occurred in 1900, which is still the second highest event on record. Since then, the largest recorded event was the May 1944 flood, which resulted from 6 to 9 inches of rain over a period of two days. According to the NOAA Atlas 14 rainfall depths for this area, the 1944 flood resulted from about a

25-yr rainfall. The elliptical storm results, on the other hand, were a bit lower than the statistical results at the 1% AEP and a bit higher than the statistics at the 0.2% AEP. The 0.2% AEP elliptical storm was also similar in magnitude to the historic 1884 flood.

Table 11.4: Frequency Flow (cfs) Results Comparison for the Neches River nr Rockland, TX

Annual Exceedance Probability (AEP)	Return Period (years)	Currently Effective FEMA FIS	BLE Data - Regression Equations	Statistical Analysis of the Observed Gage Record (114 years)	HEC-HMS Uniform Rain Frequency Storm	HEC-HMS Elliptical Frequency Storm
0.002	500		58,720	84,900	147,000	106,400
0.005	200			72,900	104,300	76,800
0.01	100		46,589	63,900	77,700	58,600
0.02	50		41,368	55,000	54,000	50,200
0.04	25		36,110	46,200	45,600	45,200
0.1	10		29,029	34,600	34,800	33,500
0.2	5			25,900	25,900	25,100
0.5	2			14,100	14,000	13,800

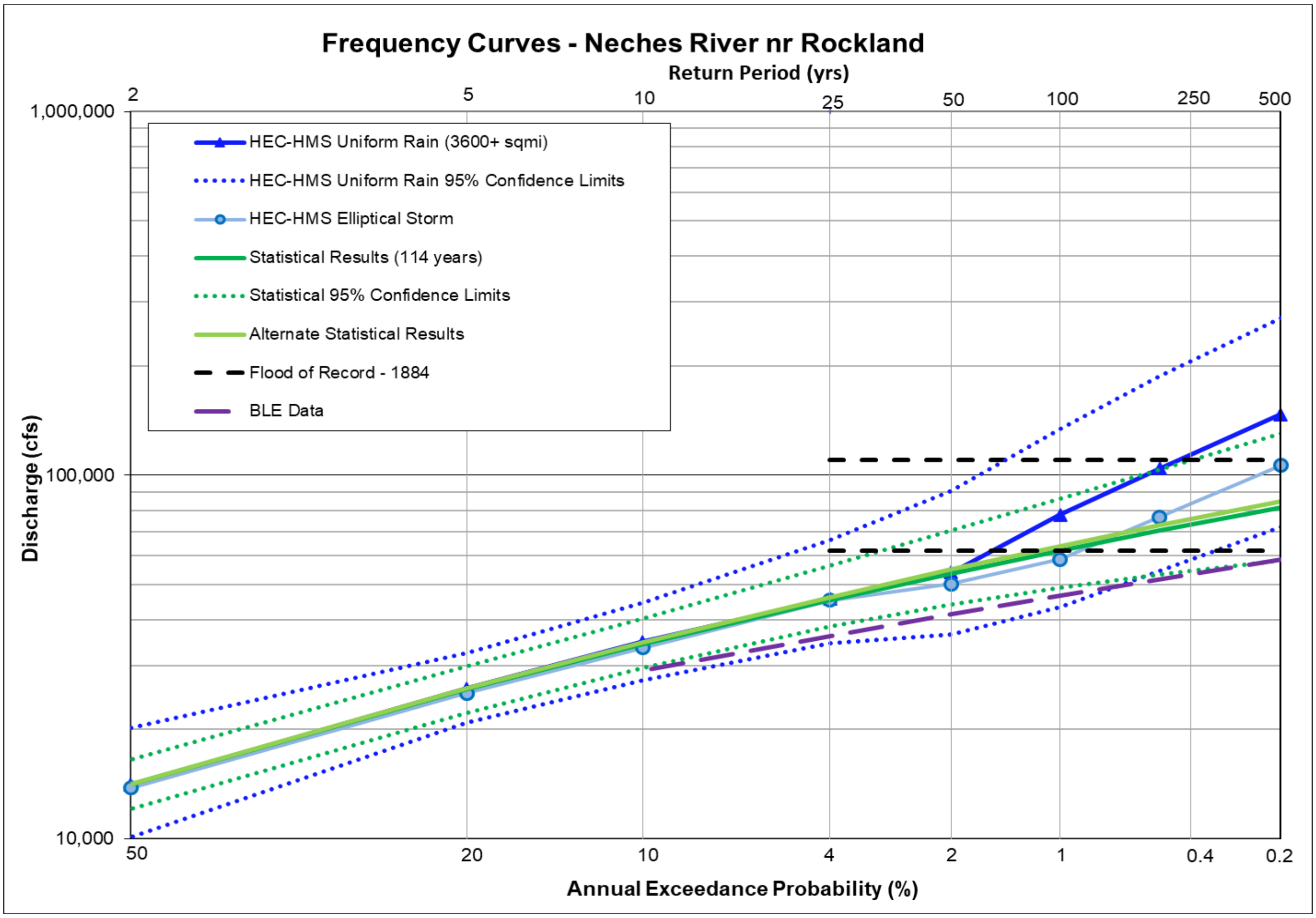


Figure 11.4a: Flow Frequency Curve Comparison for the Neches River nr Rockland, TX

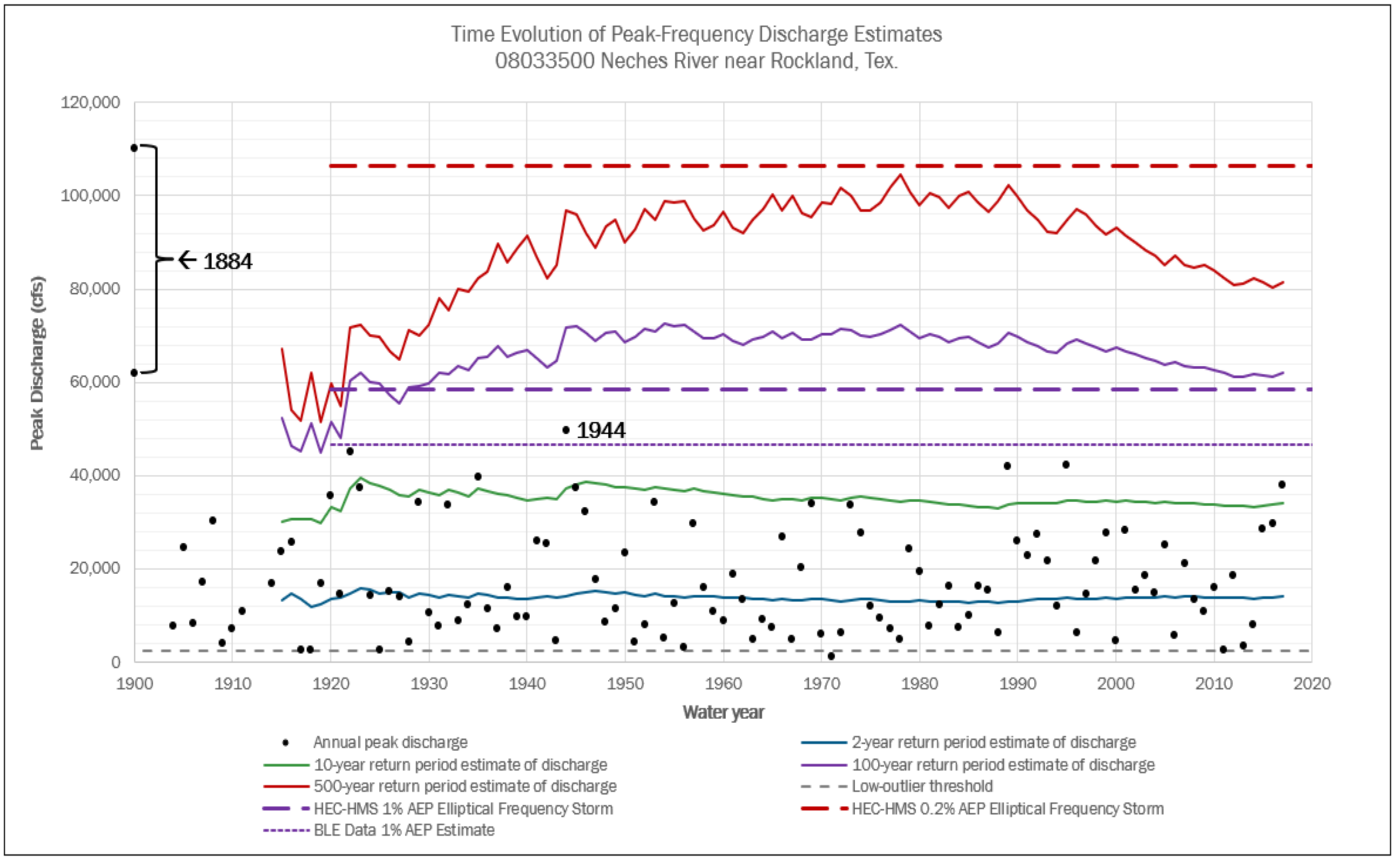


Figure 11.4b: Statistical Change Over Time Comparison for the Neches River nr Rockland, TX

The Neches River near Rockland, Texas is a USGS gage with a total drainage area of over 3,600 square miles. The USGS gage at this location has 114 years of record, which is the longest continuous gage record in the Neches River basin that was not significantly affected by regulation. The published BLE data for this location was based off of regression equations, which is an approximate method of hydrology. As shown in Figure 11.4a, the BLE flows are at or below the 95% confidence bounds from the other analyses. This means that there is at least a 95% chance that the current BLE data is underestimating the flood potential at this location for a 1% or 0.2% AEP event.

Figure 11.4a shows that there is a significant difference between the uniform rain and elliptical storm HEC-HMS results at the 1% through 0.2% AEP events. This is due to the uniform rain method's tendency to overestimate the total rainfall volume for larger drainage areas. Figure 11.4a also shows that there is a very good agreement between the elliptical storm results and both sets of statistical results up through the 0.5% AEP event. The primary difference between the two statistical curves are the assumptions surrounding the 1884 flood. The official peak discharge estimate of the historic 1884 flood near Rockland was 60,000 cfs. However, the gages upstream and downstream of this location (Diboll and Town Bluff), both had peak discharge estimates over 100,000 cfs for the 1884 flood. This means that it is very likely that the true peak discharge at Rockland from the 1884 flood was at least 100,000 cfs. Therefore, while the first statistical analysis used the 60,000 cfs peak discharge, in the alternate statistical analysis, an interval estimate between 60,000 and 110,000 cfs was used to represent the 1884 flood.

The statistical change over time plot for this location (Figure 11.4b) shows that the 1% AEP discharge estimate has been relatively stable over the past 90 years, generally varying between 55,000 and 75,000 cfs. The 0.2% AEP statistical estimate, however, has varied more widely between 70,000 and 105,000 cfs. The flood of record for this location was the historic flood of 1884; however, not much is known about the rainfall that caused that flood. The second largest recorded event was the May 1944 flood, which resulted from 6 to 9 inches of rain over a period of two days. According to the NOAA Atlas 14 rainfall depths for this area, the 1944 flood resulted from about a 25-yr rainfall. The elliptical storm results, on the other hand, were a bit lower than the current statistical results at the 1% AEP and a bit higher than the statistics at the 0.2% AEP. The 0.2% AEP elliptical storm was also similar in magnitude to the upper estimate of the historic 1884 flood.

Table 11.5: Frequency Flow (cfs) Results Comparison for Mud Creek near Jacksonville, TX

Annual Exceedance Probability (AEP)	Return Period (years)	Currently Effective FEMA FIS	BLE Data – Statistical Analysis	Statistical Analysis of the Observed Gage Record (57 years)	HEC-HMS Uniform Rain Frequency Storm	HEC-HMS Elliptical Frequency Storm	2D HEC-RAS Analysis with Uniform Rain-on-Grid
0.002	500		41,589	38,600	66,000	51,700	
0.005	200			30,800	48,700	37,700	
0.01	100		27,236	25,400	37,300	30,100	38,600
0.02	50		21,899	20,500	26,300	24,400	26,900
0.04	25		17,056	16,000	18,100	17,400	18,200
0.1	10		11,421	10,800	11,300	10,700	
0.2	5			7,390	7,600	7,100	7,700
0.5	2			3,420	3,800	3,200	

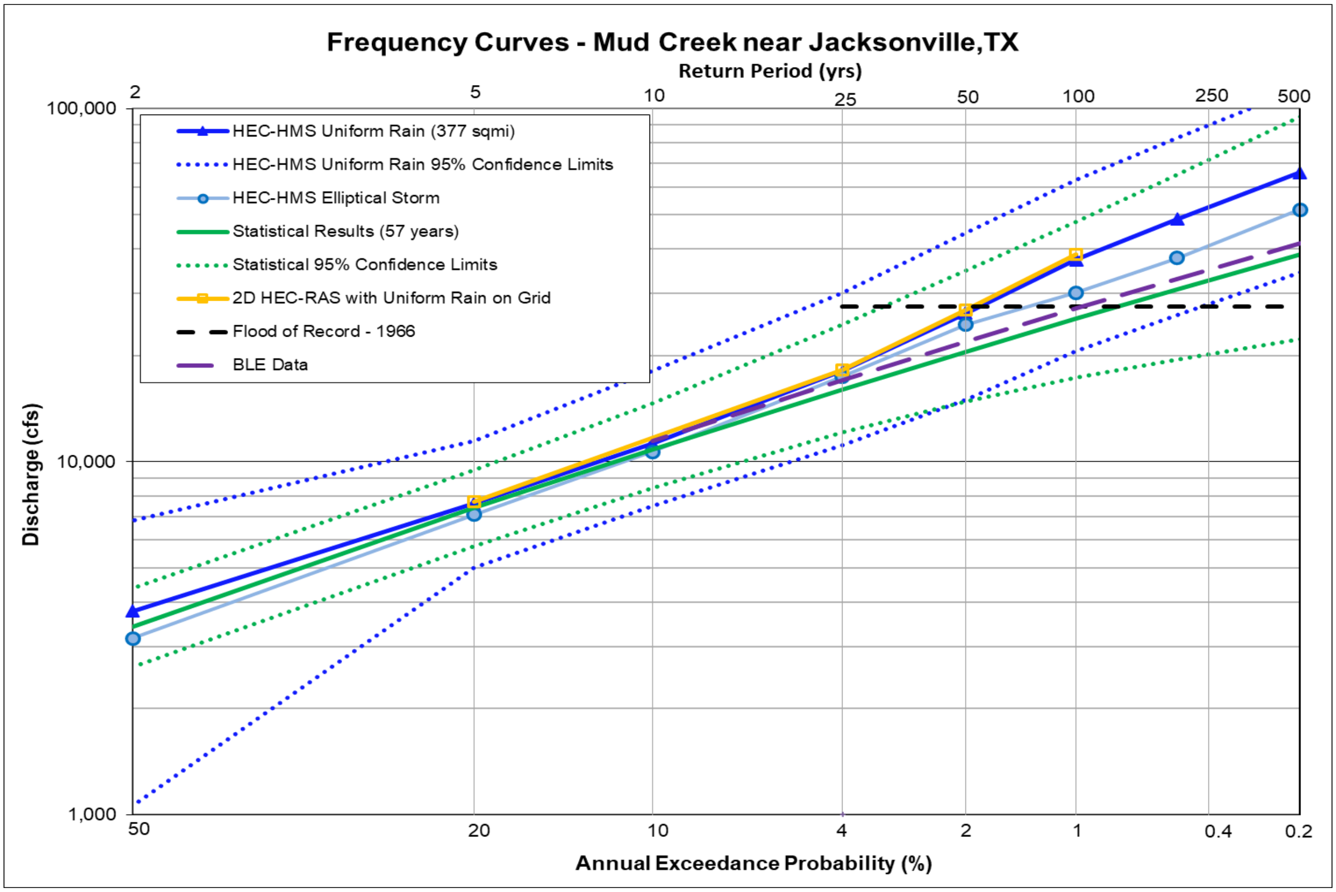


Figure 11.5a: Flow Frequency Curve Comparison for Mud Creek near Jacksonville, TX

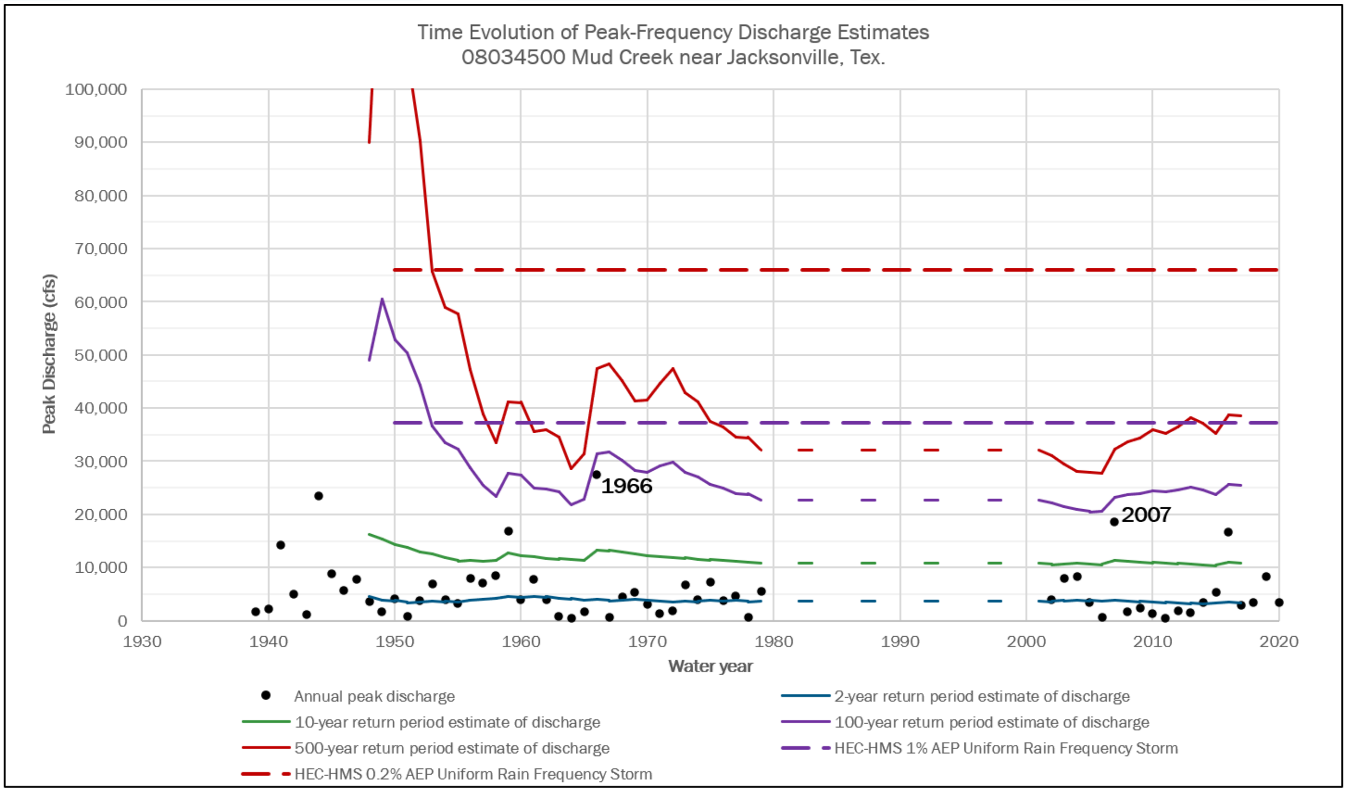


Figure 11.5b: Statistical Change Over Time Comparison for the Mud Creek near Jacksonville, TX

Mud Creek near Jacksonville, TX is a USGS gage in the upper Angelina River watershed with about 377 square miles of total drainage area, with about 113 square miles regulated by Lake Tyler and East Lake Tyler. The USGS gage at this location has 57 years of record, which is a moderate length of record. Its record started in the late 1930s, but the gage was out of service for over 20 years during the 1980s and 1990s. The published BLE data for this location was based off a statistical analysis of the gage record, which is why the BLE flows almost match the current statistical analysis flows.

Figure 11.5a shows that the HEC-HMS results from both uniform rain and the elliptical storms are significantly higher than the statistical analysis of the gage record at the 1% AEP event, but still well within one another's confidence bounds. Figure 11.5b shows that there has still been significant variation in the statistical flow estimates, partially due to the missing record. The largest flood on record occurred in 1966, which had rainfall totals of 9 to 11 inches on Mud Creek. According to the NOAA Atlas 14 rainfall depths for this area, this would equate to about a 25 to 50-yr rainfall event. Therefore, one would expect that 1% AEP peak discharge would likely be higher than the 1966 peak. However, Figure 11.5b shows that the current statistical 1% AEP estimate is lower than the 1966 peak, while the HEC-HMS 1% AEP results are higher than the 1966 peak.

The second highest event on record was 2007, which had about 9 inches of rain below Lake Tyler and equates to about a 25-yr rainfall event. The 1% AEP point rainfall depth from NOAA Atlas 14, on the other hand, was about 13.6 inches. Since this location has not experienced a large flood event similar to a 100-yr event, it appears that the current statistical record could be underestimating the 1% AEP flood potential at this location.

An additional analysis was performed for this location using a 2D HEC-RAS model. Figure 11.5a shows that the calibrated 2D HEC-RAS model with rain-on-grid frequency storms produced very similar peak discharge results to the HEC-HMS model with uniform rain. The HEC-HMS with uniform rain results are also more consistent with the magnitudes of the observed floods of 1966 and 2007 and their estimated rainfall frequencies. Therefore, the

HEC-HMS model, which was calibrated to the 2D HEC-RAS results, is likely yielding a more reliable estimate of the rare flood frequencies than the statistical results.

Table 11.6: Frequency Flow (cfs) Results Comparison for the Angelina River nr Alto, TX

Annual Exceedance Probability (AEP)	Return Period (years)	Currently Effective FEMA FIS	BLE Data - Regression Equations	Statistical Analysis of the Observed Gage Record (60 years)	HEC-HMS Uniform Rain Frequency Storm	HEC-HMS Elliptical Frequency Storm	2D HEC-RAS Analysis with Uniform Rain on Grid
0.002	500		69,666	60,000	120,000	96,900	
0.005	200			49,000	87,900	70,400	
0.01	100		50,388	41,200	66,700	52,800	70,100
0.02	50		41,334	34,000	46,000	37,400	45,700
0.04	25		33,145	27,400	31,300	26,300	31,000
0.1	10		23,429	19,300	20,400	17,900	
0.2	5			13,800	13,500	12,500	
0.5	2			6,960	6,700	6,500	

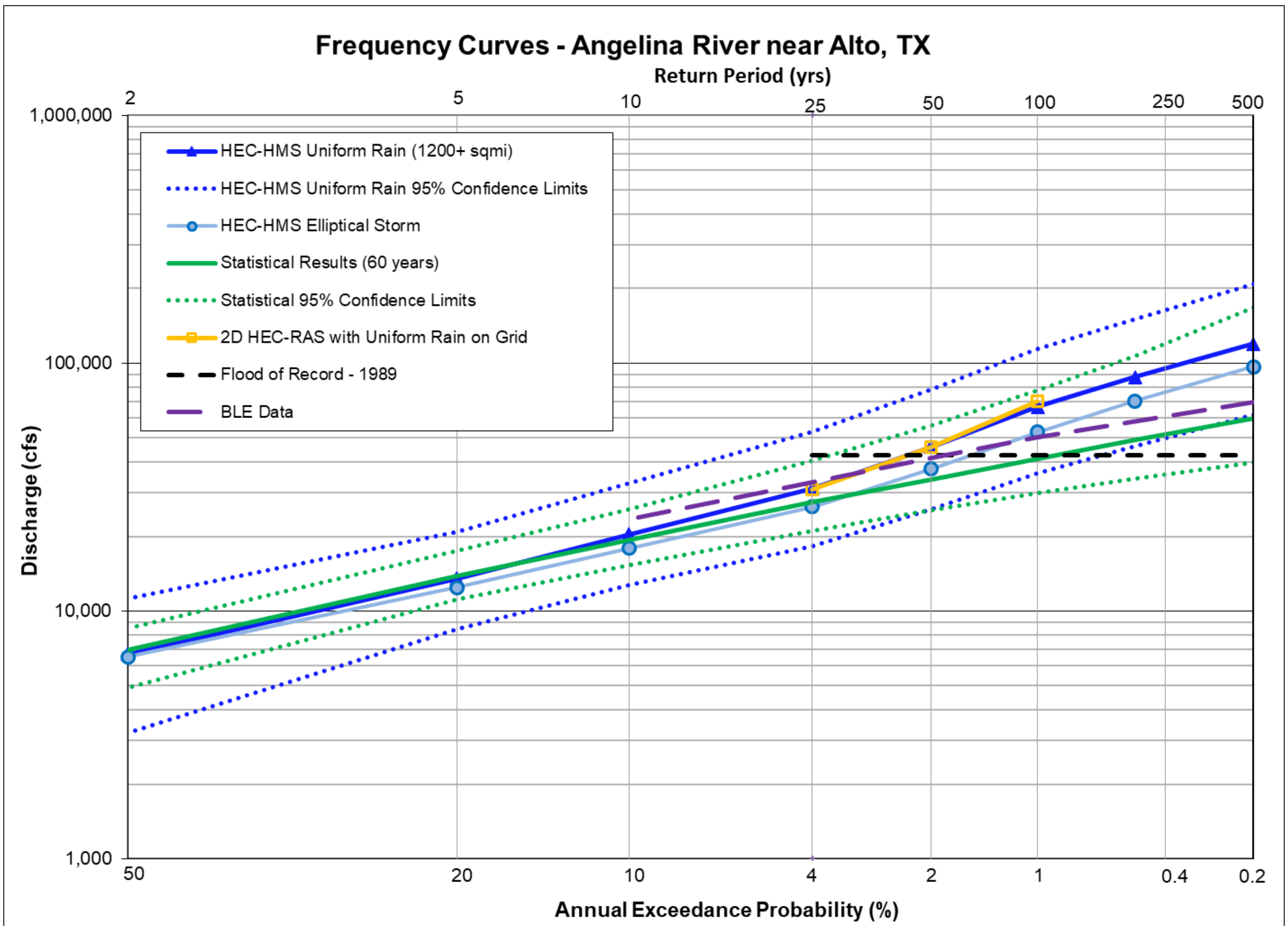


Figure 11.6a: Flow Frequency Curve Comparison for the Angelina River nr Alto, TX

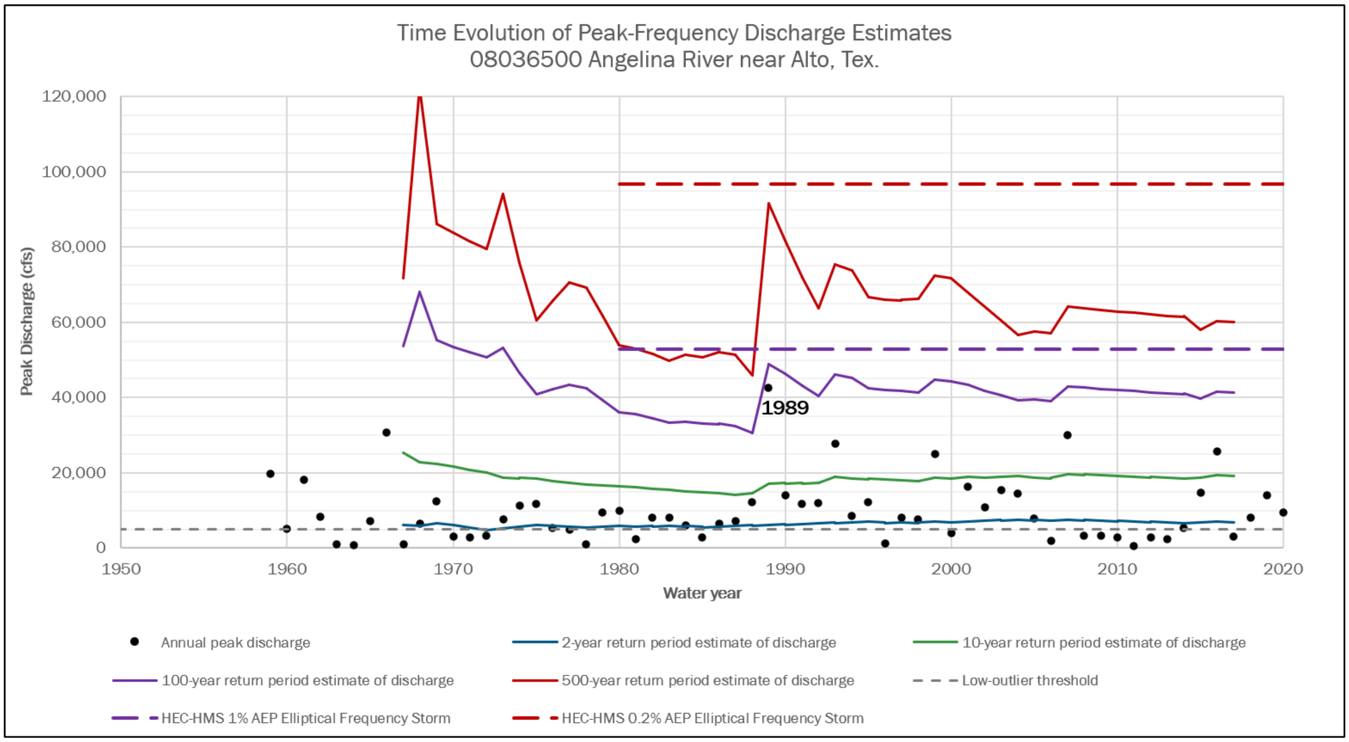


Figure 11.6b: Statistical Change Over Time Comparison for the Angelina River nr Alto, TX

The Angelina River near Alto, TX is a USGS gage in the upper Angelina River watershed with over 1,200 square miles of total drainage area. It is located about 20 miles downstream of the confluence of the Angelina River with Mud Creek. The USGS gage at this location has 60 years of record, which is a moderate length of record. The published BLE data for this location was based off regression equations, which is an approximate method of hydrology. Coincidentally, the BLE 1% AEP discharge is very close to the 1% AEP discharge from the HEC-HMS elliptical storms.

Figure 11.6a shows that the HEC-HMS results from both uniform rain and the elliptical storms are significantly higher than the statistical analysis of the gage record at the 1% AEP event, but still well within one another's confidence bounds. Figure 11.6b shows that there has still been significant variation in the statistical flow estimates over time. The largest flood on record occurred in 1989, which had rainfall totals of 9 to 11 inches over 2 days. According to the NOAA Atlas 14 rainfall depths for this area, this would equate to about a 25-yr rainfall event. Therefore, one would expect that 1% AEP peak discharge should be higher than the 1989 peak. However, Figure 11.6b shows that the current statistical 1% AEP estimate is lower than the 1989 peak, while the HEC-HMS 1% AEP results are higher than the 1989 peak.

A 2D HEC-RAS analysis was also performed for this location. Figure 11.6a shows that the calibrated 2D HEC-RAS model with rain-on-grid uniform rain frequency storms produced very similar peak discharge results to the HEC-HMS model with uniform rain. However, the uniform rain method tends to overestimate the rainfall volume for larger drainage areas, such as this location. Therefore, the HEC-HMS elliptical storm results are likely a more reliable estimate of the rare flood frequencies, while still accurately representing the response of the watershed to a large storm event.

Table 11.7: Frequency Flow (cfs) Results Comparison for the Angelina River nr Lufkin, TX

Annual Exceedance Probability (AEP)	Return Period (years)	Currently Effective FEMA FIS	BLE Data - Regression Equations	Statistical Analysis of the Observed Gage Record (51 years)	HEC-HMS Uniform Rain Frequency Storm	HEC-HMS Elliptical Frequency Storm
0.002	500		59,303	65,500	122,400	96,300
0.005	200			54,300	88,700	68,600
0.01	100		41,168	46,300	66,200	52,000
0.02	50		34,090	38,600	46,500	36,800
0.04	25		27,453	31,200	31,500	26,000
0.1	10		19,358	22,100	20,700	17,900
0.2	5			15,700	14,800	12,600
0.5	2			7,690	7,700	6,500

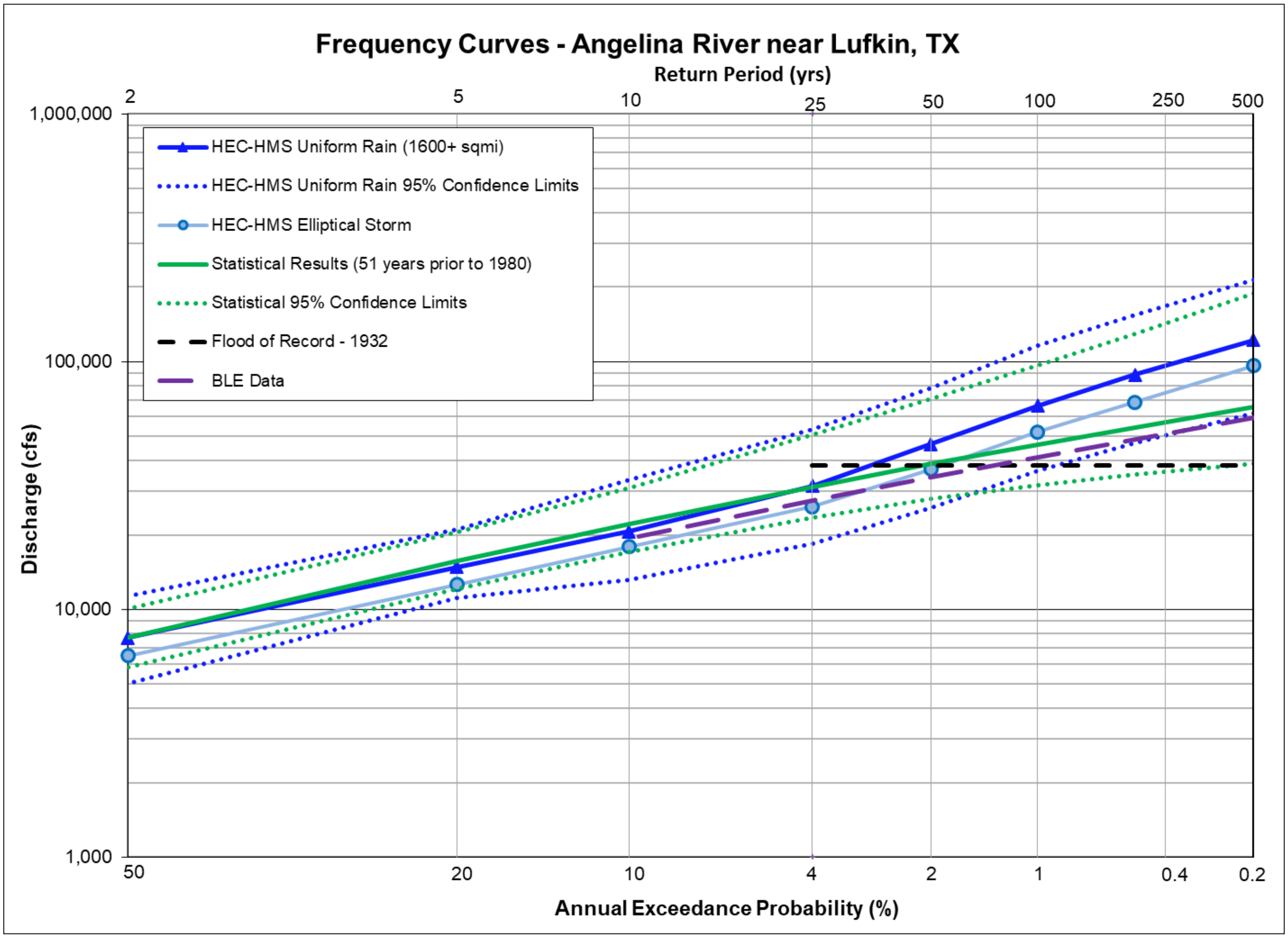


Figure 11.7: Flow Frequency Curve Comparison for the Angelina River nr Lufkin, TX

The Angelina River near Lufkin, Texas is a recently re-instated USGS gage with a drainage area of over 1,600 square miles. The USGS gage at this location has 51 years of record, all of which occurred prior to 1980. The published BLE data for this location was based off of regression equations, which is a simple, approximate method of hydrology. As shown in Figure 11.7, the BLE 1% AEP flow is lower than both the statistical and HEC-HMS results, indicating that the current BLE data may be underestimating the flood potential at this location for a 1% or 0.2% AEP event.

Figure 11.7 shows that the HEC-HMS elliptical storm results are fairly consistent with the statistical results up through the 1% AEP frequency. For the 0.5% and 0.2% AEP events the HEC-HMS results trend higher than the statistical results. No statistical change over time plot is available at this location due to its older and relatively short gage record.

The flood of record for this location was the 1932 peak, which is the highest in 50 years of record. Coincidentally, both the statistical results and the HEC-HMS elliptical storm results place the 1932 peak discharge at about a 2% AEP (50-yr) frequency. Since the elliptical storm results agree with the statistical results through the 2% AEP, and the gage record is not long enough to accurately estimate the rarer events, the HEC-HMS elliptical storms considered the most reliable estimate of the flood potential for this location.

Table 11.8: Frequency Flow (cfs) Results Comparison for Attoyac Bayou near Chireno, TX

Annual Exceedance Probability (AEP)	Return Period (years)	Currently Effective FEMA FIS	BLE Data–Statistical Analysis	Statistical Analysis of the Observed Gage Record (78 years)	HEC-HMS Uniform Rain Frequency Storm	HEC-HMS Elliptical Frequency Storm
0.002	500		52,180	49,100	84,800	77,300
0.005	200			43,900	63,300	58,100
0.01	100		37,250	39,500	49,700	45,400
0.02	50		31,350	34,600	34,500	31,900
0.04	25		25,760	29,200	29,400	27,300
0.1	10		18,820	21,400	20,000	19,100
0.2	5			15,100	14,300	13,700
0.5	2			6,500	6,500	6,300

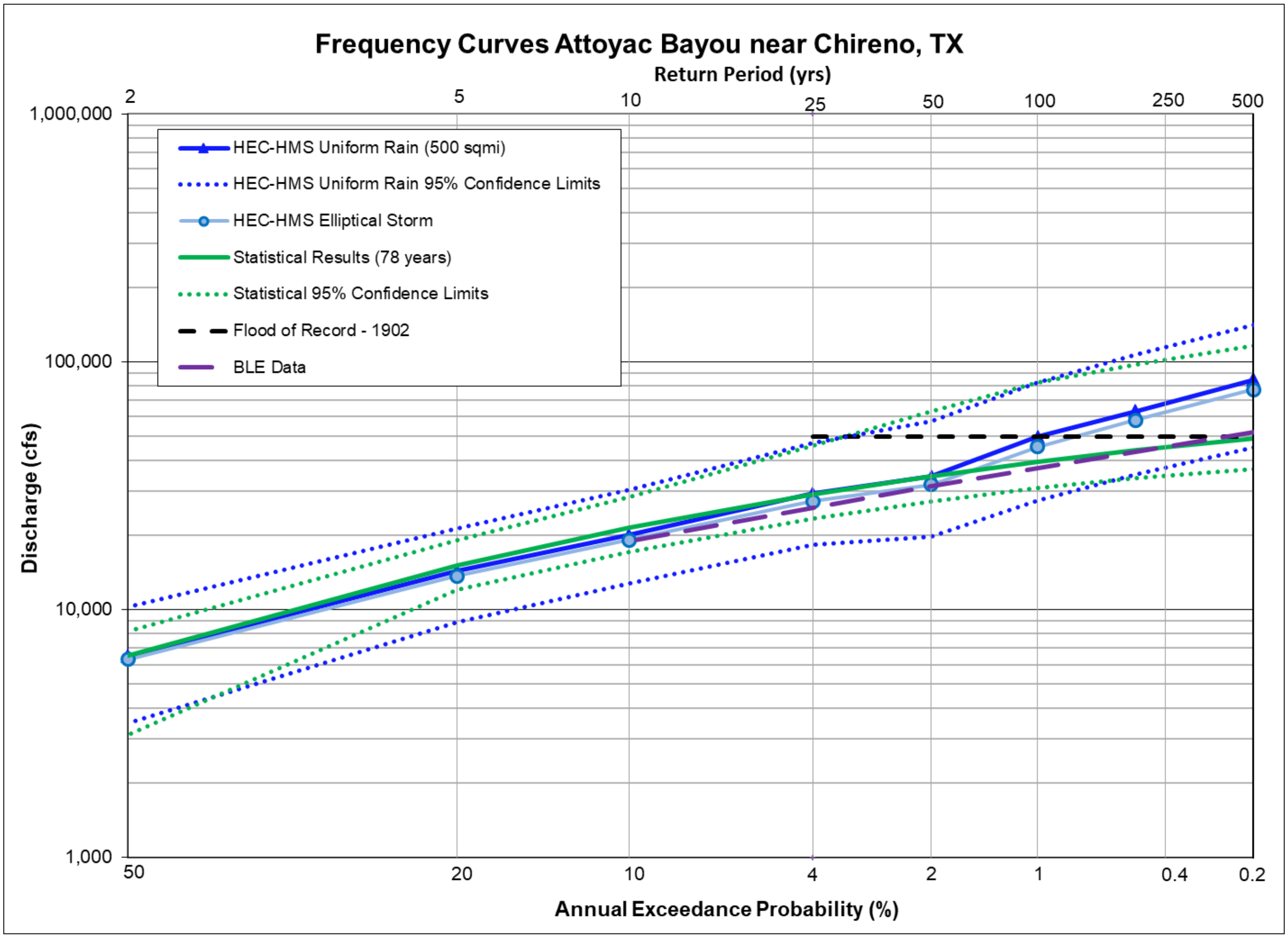


Figure 11.8a: Flow Frequency Curve Comparison for Attoyac Bayou near Chireno, TX

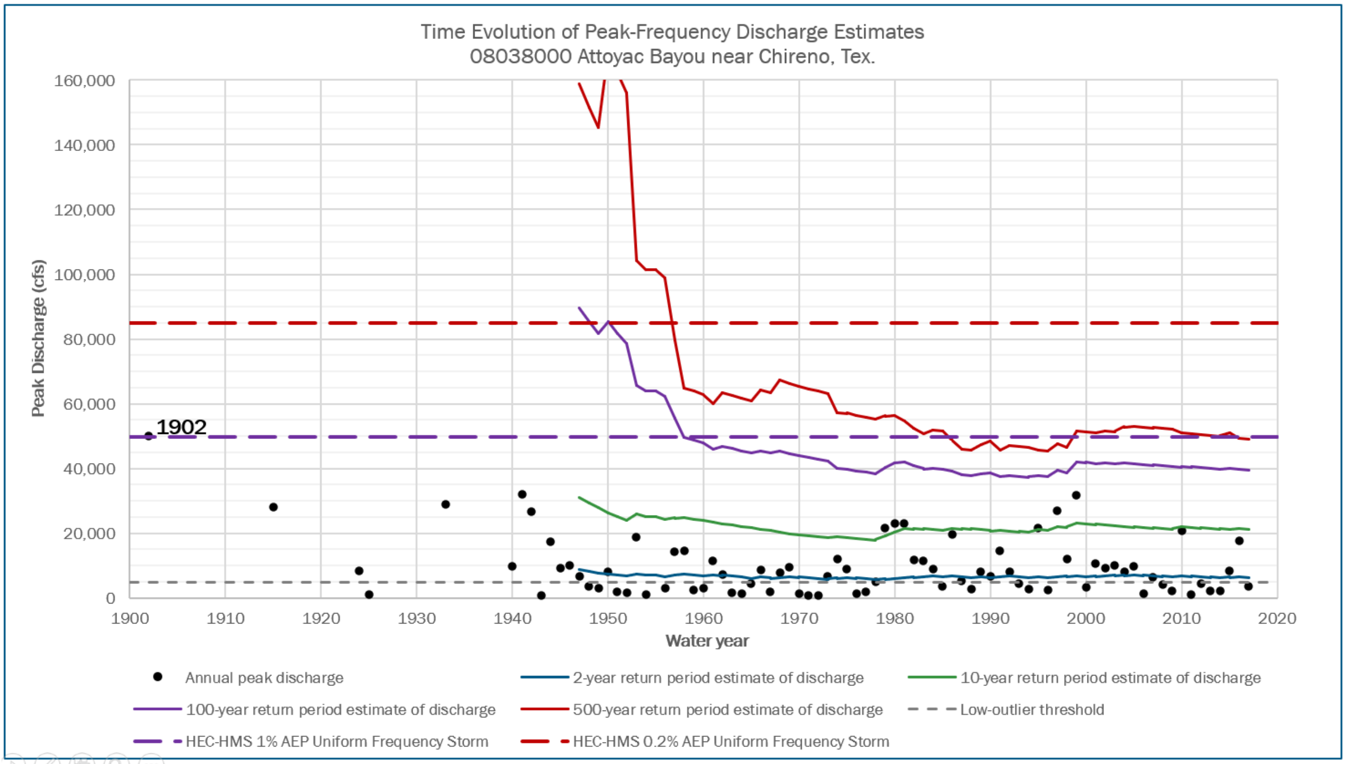


Figure 11.8b: Statistical Change Over Time Comparison for Attoyac Bayou near Chireno, TX

Attoyac Bayou near Chireno, Texas is a USGS gage with a total drainage area of about 500 square miles, which is located on a tributary of the Angelina River directly north of Sam Rayburn Reservoir. In addition, there are about 13 NRCS flood detention structures located upstream of this gage which were built in the 1970s and collectively control about 100 square miles of the watershed. The USGS gage at this location has about 78 years of record, which is a fairly long period of record. The published BLE data for this location was based off of a statistical analysis of the gage record, which is why its numbers are so similar to the statistical hydrology results, as shown in Figure 11.8a.

Figure 11.8a shows that there is not much difference between the HEC-HMS results from uniform rain and elliptical frequency storms, but the HEC-HMS results are significantly higher than the statistical hydrology results at the 1% through 0.2% AEP events. However, the HEC-HMS 1% AEP results are very similar to the 1902 flood of record for this location, while the current statistical results would put the 1902 flood at a 500-yr return interval. During the 1902 storm event, about 14 inches of rain was recorded in one day in Nacogdoches, Texas, but the exact rainfall amounts on Attoyac Bayou are unknown.

The statistical change over time plot for this location (Figure 11.8b) shows that while the 1% AEP discharge estimate has been relatively stable over the past 40 years, it is also at a relative low point compared to its earlier history. This means that one new large storm event could cause a significant increase in the rare frequency discharge estimates. The HEC-HMS results, on the other hand, are less likely to change over time and are more consistent with the 1902 flood, which is the highest event in 78 years of record.

Table 11.9: Frequency Flow (cfs) Results Comparison for the Ayish Bayou nr San Augustine, TX

Annual Exceedance Probability (AEP)	Return Period (years)	Currently Effective FEMA FIS	BLE Data – Statistical Analysis	Statistical Analysis of the Observed Gage Record (60 years)	HEC-HMS Uniform Rain Frequency Storm	HEC-HMS Elliptical Frequency Storm
0.002	500		43,169	39,100	37,100	
0.005	200			31,800	30,200	
0.01	100		27,730	26,700	25,500	
0.02	50		22,220	21,900	20,500	
0.04	25		17,330	17,400	16,900	
0.1	10		11,750	12,000	12,100	
0.2	5			8,350	8,500	
0.5	2			3,940	4,100	

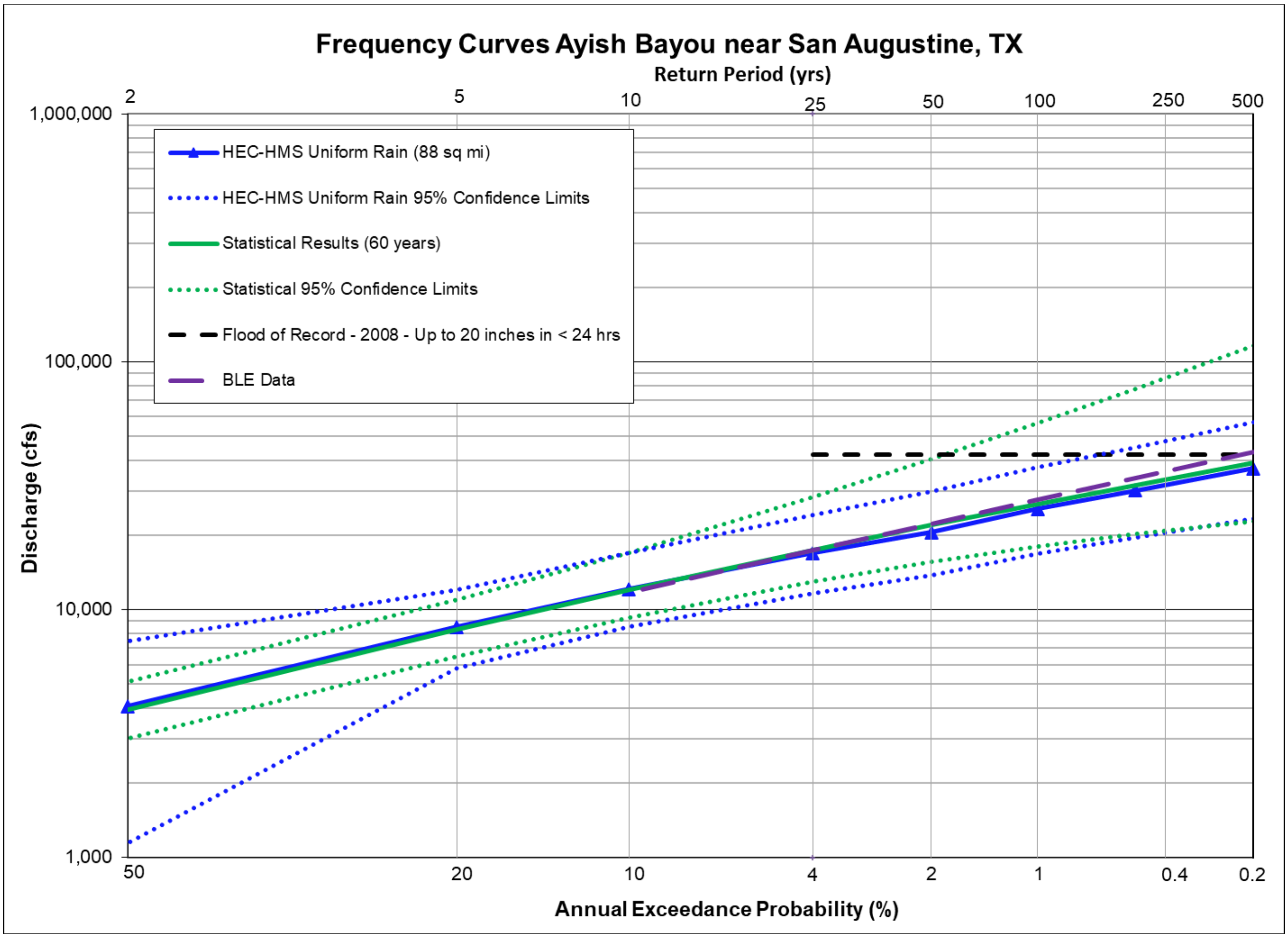


Figure 11.9: Flow Frequency Curve Comparison for the Ayish Bayou nr San Augustine, TX

Ayish Bayou, near San Augustine, Texas is a USGS gage with a drainage area of only 88 square miles, which is located on a tributary to the Angelina River directly north of Sam Rayburn Reservoir. The USGS gage at this location has a moderate period of record of 60 years. The published BLE data for this location was based off of a statistical analysis of the gage record, which is why its numbers are so similar to the statistical hydrology results, as shown in Figure 11.9.

Figure 11.9 shows that the HEC-HMS results agree very closely with the statistical results all the way up through the 0.2% AEP event. The agreement between the results at this location occurred coincidentally, rather than by force. Figure 11.9 also shows similar confidence limits between the two results, with the HEC-HMS results having narrower confidence bands at the 1% through 0.2% AEP levels. No statistical change over time plot is available at this location.

The flood of record for this location was the 2008 event, which is the highest in 60 years of record. However, the frequency curves in Figure 11.9 show that the peak discharge from 2008 corresponds to about a 0.2% AEP (500-year) event according to the statistics and the HEC-HMS results. An investigation into the rainfall for this storm revealed that up to 20 inches of rain was reported in less than 24 hours in the area of Ayish Bayou. Flooding in the county was so severe that many homes were under five feet of water and many bridges were washed out. The 2008 flood on Ayish Bayou also caused one fatality. The rainfall and severity of flooding caused by the 2008 event are consistent with the 500-year recurrence interval indicated by the HEC-HMS results.

Table 11.10: Frequency Flow (cfs) Results Comparison for the Angelina River below Sam Rayburn Dam

Annual Exceedance Probability (AEP)	Return Period (years)	Currently Effective FEMA FIS	BLE Data - Regression Equations	Statistical Analysis of the Observed Gage Record	Reservoir Analysis in RMC-RFA	HEC-HMS Uniform Rain Frequency Storm	HEC-HMS Elliptical Frequency Storm
0.002	500		34,300		26,300	43,900	17,000
0.005	200				20,000	18,600	17,000
0.01	100		27,700		20,000	17,000	15,400
0.02	50		24,730		19,500	15,700	14,000
0.04	25		21,670		19,000	14,000	14,000
0.1	10		17,420		17,000	14,000	11,700
0.2	5				15,000	14,000	9,800
0.5	2				10,000	8,400	8,400

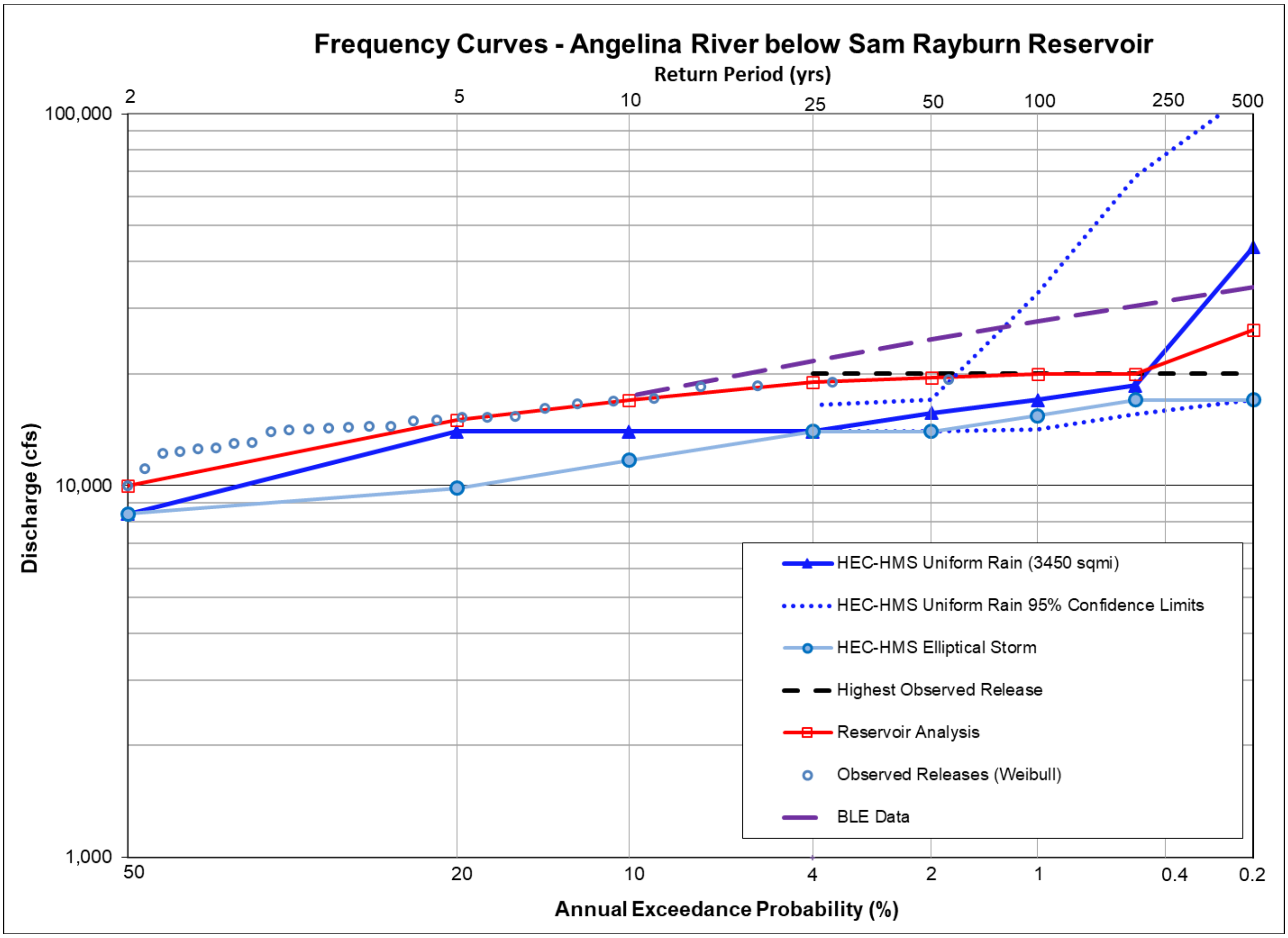


Figure 11.10: Flow Frequency Curve Comparison for the Angelina River below Sam Rayburn Dam

The Angelina River below Sam Rayburn Dam has a drainage area of about 3,450 square miles, but it does not have an observed USGS stream gage. Instead, USACE Fort Worth District maintains records of the dam's releases in their water management database. In lieu of a Bulletin 17C statistical analysis, which would not be appropriate for this highly regulated location, Figure 11.10 shows the peak annual observed releases with their Weibull plotting positions. The highest observed release in the dam's 55 year history is about 20,000 cfs. These have all been gated releases, as the labyrinth spillway at Sam Rayburn Dam has never been activated (as of December 2021). The observed pool elevation has come within about one foot of the spillway crest on more than one occasion.

The published BLE data for this location was based off of regression equations, which is an approximate method of hydrology that ignores the reservoir. Figure 11.10 shows that the BLE discharge estimates trend much higher than the other results at the 2% through 0.2% AEP events. This is because the BLE data essentially calculated an unregulated discharge that did not take into account the major flood storage capacity of Sam Rayburn Reservoir.

Figure 11.10 also shows that the Reservoir Analysis results from RMC-RFA plot very consistently with the observed Weibull plotting positions. The HEC-HMS results, on the other hand, plot significantly lower than the observed data. This is because the RMC-RFA reservoir analysis gives a more complete picture of the operations of the reservoir, including the full range of possible inflow volumes and starting pool elevations, whereas HEC-HMS only uses a single inflow event and starting pool elevation. Therefore, the reservoir analysis is considered the most complete and reliable analysis for this location.

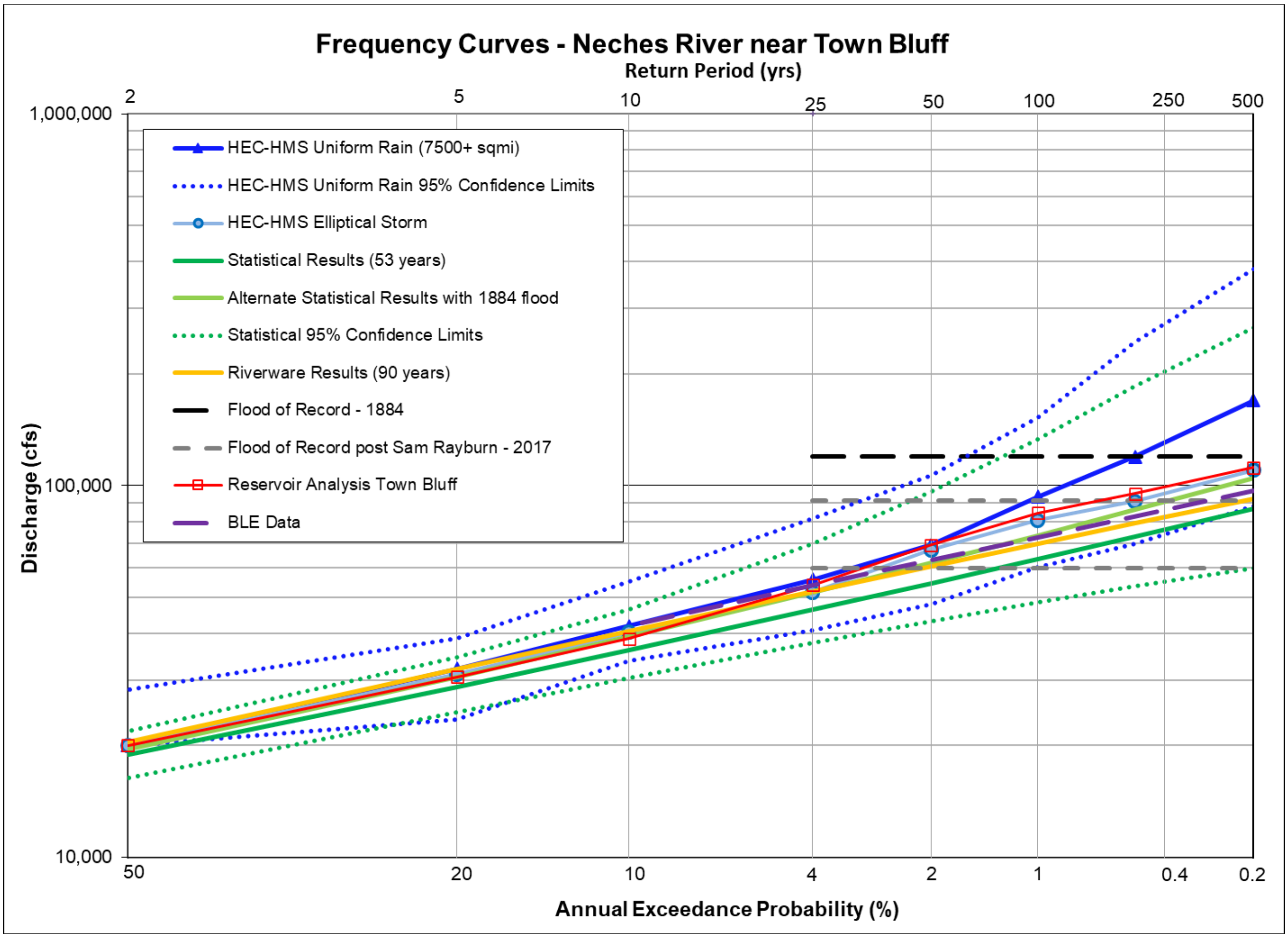


Figure 11.11a: Flow Frequency Curve Comparison for the Neches River nr Town Bluff, TX

Table 11.11: Frequency Flow (cfs) Results Comparison for the Neches River nr Town Bluff, TX

Annual Exceedance Probability (AEP)	Return Period (years)	BLE Data – Statistical Analysis	Statistical Analysis of the Gage Record (53 years)	Alternate Statistical Analysis with the 1884 flood included	Statistical Analysis of the Extended RiverWare Record (90 years)	HEC-HMS Uniform Rain Frequency Storm	HEC-HMS Elliptical Frequency Storm	Reservoir Analysis in RMC-RFA
0.002	500	97,105	86,700	105,000	92,300	169,600	110,000	112,000
0.005	200		73,000	86,100	79,200	119,500	90,700	95,300
0.01	100	72,522	63,500	73,500	69,800	93,400	80,700	84,400
0.02	50	62,908	54,600	62,100	60,700	69,200	67,200	69,200
0.04	25	53,948	46,300	51,700	51,900	55,900	51,600	54,000
0.1	10	42,067	36,100	39,200	40,600	41,900	40,300	38,800
0.2	5		28,700	30,600	32,100	32,200	31,100	30,600
0.5	2		18,900	19,500	20,400	20,000	20,000	20,000

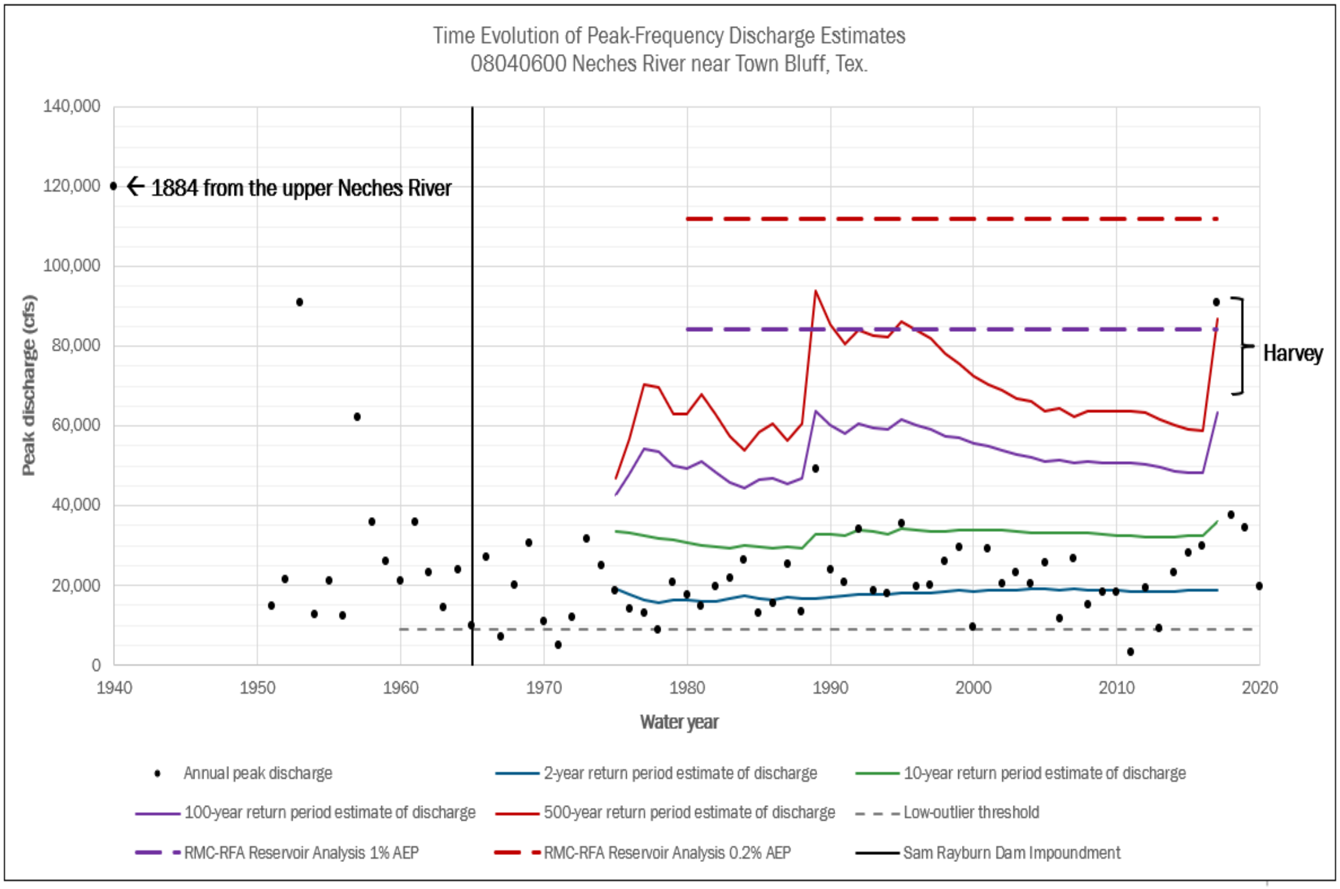


Figure 11.11b: Statistical Change Over Time Comparison for the Neches River nr Town Bluff, TX

The Neches River near Town Bluff, Texas is a USGS stream gage with over 7,500 square miles of drainage area. It is located directly downstream of B.A. Steinhagen Lake and just downstream of the confluence of the Angelina River with the Neches River. While the USGS gage has a period of record dating back to the 1951, only the period of record after 1965 was used in the statistical analysis, which corresponds to when Sam Rayburn Dam was completed. Sam Rayburn Reservoir controls approximately half the drainage area above B.A. Steinhagen; therefore, its completion had a noticeable effect on the flows at Town Bluff. B.A. Steinhagen Lake, on the other hand, is primarily a pass-through lake with very little volume allocated to storing flood waters.

Figure 11.11a shows that while a wide variety of analyses were completed for this location, most of the results fell within a relatively narrow band of discharges. The published BLE data for this location was based off of a statistical analysis of the gage record, which is why its numbers are so similar to some of the statistical hydrology results. Figure 11.11a also shows that there is a significant difference between the uniform rain and elliptical storm HEC-HMS results at the 1% through 0.2% AEP events. This is due to the uniform rain method's tendency to overestimate the total rainfall volume for large drainage areas like this one.

Figure 11.11a also shows that there is a very good agreement between the HEC-HMS elliptical storm results, the reservoir analysis, and the alternate statistical analysis. The primary difference between the two statistical curves are the assumptions surrounding the 1884 flood of record. While the first statistical analysis excluded the 1884 flood as being prior to Sam Rayburn dam, the alternate statistical analysis included the 1884 event when it was found that the flood primarily originated on the unregulated portion of the Neches River. In fact, the historic 1884 peak discharge at Diboll of 110,000 cfs was very similar to the historic peak at Town Bluff of 120,000 cfs.

The statistical change over time plot for this location (Figure 11.11b) shows that the 1% and 0.2% AEP statistical discharge estimates are significantly lower than the results of the reservoir analysis. However, these change over time estimates do not include any of the floods prior to 1965. This plot would look very different if some of the earlier floods, such as 1884, were included. While not much is known about the rainfall that caused the 1884 flood, it is the highest known flood event on this portion of the Neches River. Both the 0.2% AEP reservoir analysis and elliptical storm results were similar in magnitude to the historic 1884 flood at this location.

Table 11.12: Frequency Flow (cfs) Results Comparison for the Neches River at Evadale, TX

Annual Exceedance Probability (AEP)	Return Period (years)	BLE Data - Statistical Analysis	Statistical Analysis of the Observed Gage Record (53 years)	Alternate Statistical Analysis with the 1884 flood included	Statistical Analysis of the Extended RiverWare Record (90 years)	HEC-HMS Uniform Rain Frequency Storm	HEC-HMS Elliptical Frequency Storm
0.002	500	99,839	89,700	104,000	88,900	170,200	115,000
0.005	200		76,600	87,800	78,000	119,300	90,900
0.01	100	74,203	67,300	76,200	69,800	93,500	74,700
0.02	50	64,253	58,400	65,300	61,500	69,300	57,700
0.04	25	55,034	49,900	55,000	53,300	56,200	46,000
0.1	10	42,834	39,000	42,200	42,200	42,200	36,600
0.2	5		31,000	33,000	33,600	32,500	29,700
0.5	2		19,900	20,600	21,100	21,200	21,000

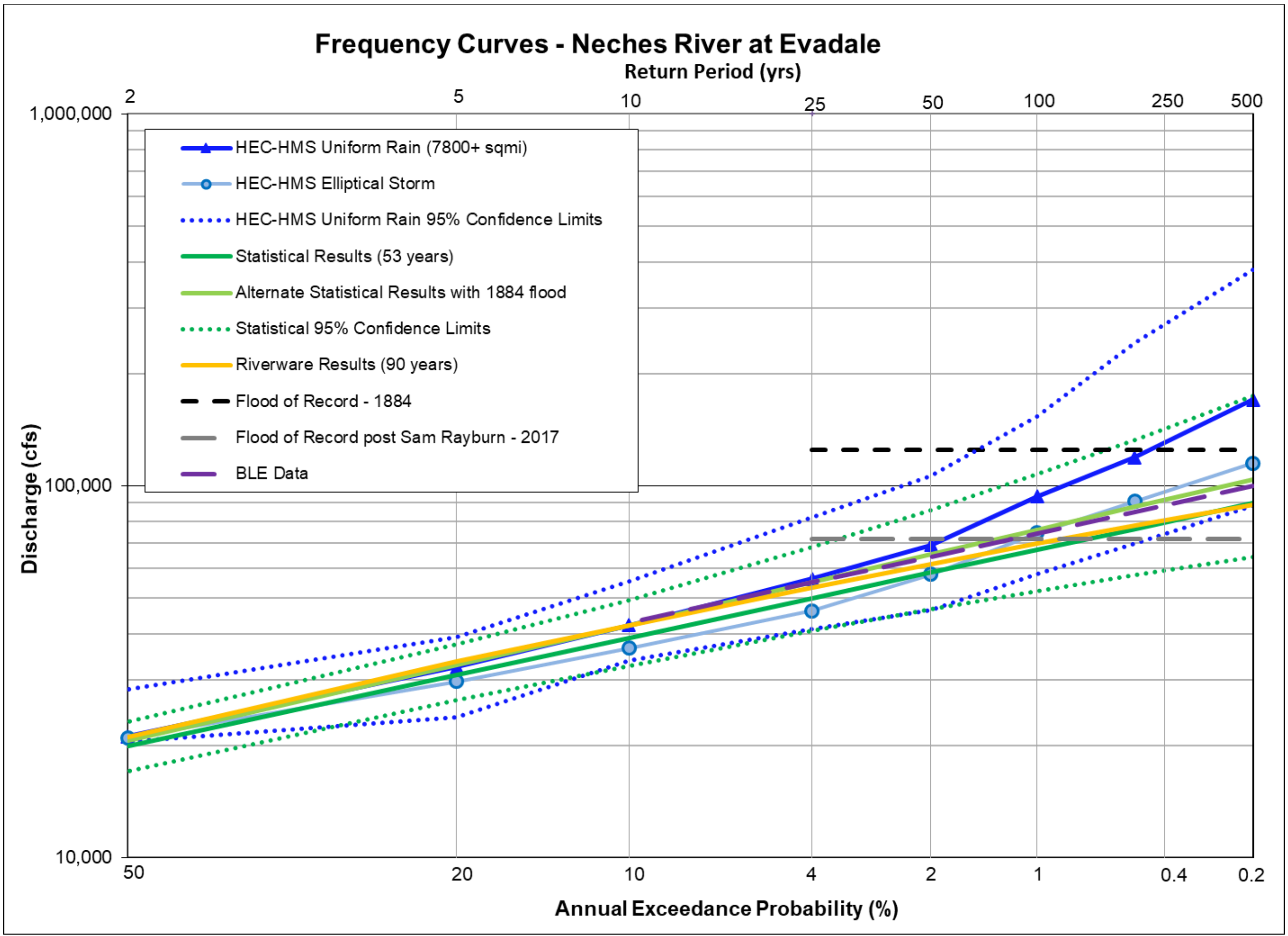


Figure 11.12a: Flow Frequency Curve Comparison for the Neches River at Evadale, TX

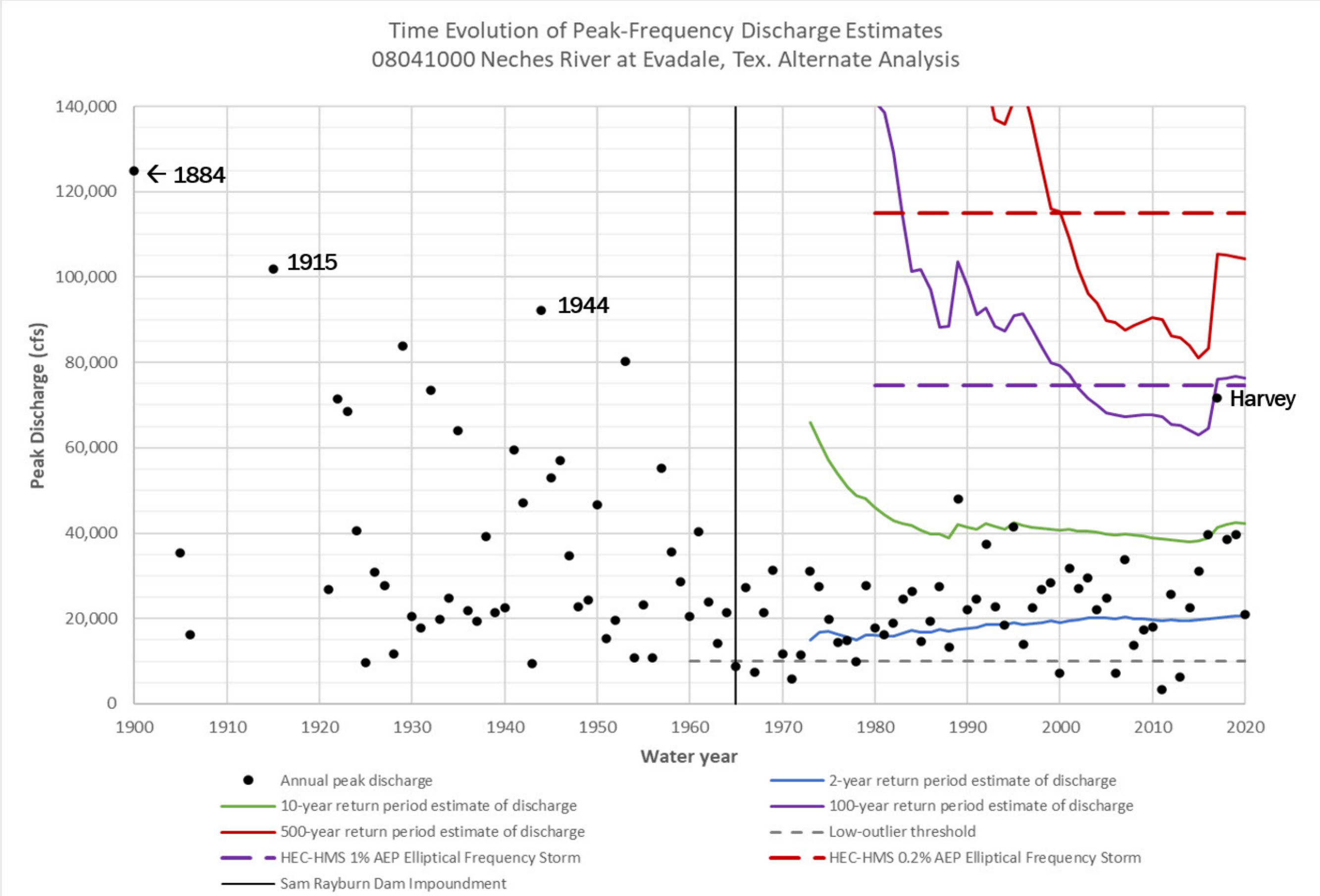


Figure 11.12b: Statistical Change Over Time Comparison for the Neches River at Evadale, TX

The Neches River at Evadale, Texas is a USGS gage with almost 7,900 square miles of drainage area, which is located about 30 miles south of Town Bluff Dam and B.A. Steinhagen Lake, and it has a similar hydrology to the USGS gage near Town Bluff. While the Evadale gage has a period of record that dates back to the early 1900s, the completion of Sam Rayburn dam in 1965 significantly affected the hydrology of this location.

Figure 11.12a shows that while a wide variety of analyses were completed for this location, most of the results fell within a relatively narrow band of discharges. The published BLE data for this location was based off of a statistical analysis of the gage record, which is why its numbers are so similar to the statistical results. Figure 11.12a also shows that there is a significant difference between the uniform rain and elliptical storm HEC-HMS results at the 1% through 0.2% AEP events. This is due to the uniform rain method's tendency to overestimate the total rainfall volume for large drainage areas like this one.

Figure 11.12a also shows that there is fairly good agreement between the elliptical storm results and the two sets of statistical results. The primary difference between the two statistical curves are the assumptions surrounding the 1884 flood of record. While the first statistical analysis excluded the 1884 flood along with all of the other data prior to the completion of Sam Rayburn dam, the alternate statistical analysis included the 1884 historic event when it was found that the flood primarily originated on the unregulated portion of the Neches River. In fact, the historic 1884 peak discharge at Diboll of 110,000 cfs was very similar to the historic peaks near Town Bluff and Evadale of 120,000 and 125,000 cfs, respectively.

The statistical change over time plot for this location (Figure 11.12b) is based on the first statistical analysis, and it shows that there has been significant variation in the 1% and 0.2% AEP statistical discharge estimates over time. However, it also shows that the HEC-HMS elliptical storm results are very similar to the current the statistical results at the 1% AEP frequency. The 1% and 0.2% AEP elliptical storm peak discharges are also well within the range of the statistical estimates that have been experienced over time for those frequencies.

The 1% AEP peak discharge estimate from the HEC-HMS elliptical storm has been significantly exceeded by flood events in 1884, 1915 and 1944. While not much is known about the rainfall that caused the 1884 flood, it is well known as the highest flood event on this portion of the Neches River, and the 0.2% AEP elliptical storm results were similar in magnitude to the historic 1884 flood at this location. Further investigation of the 1915 flood revealed that it originated primarily on the upper Angelina River. The original water control manual for Sam Rayburn Dam and Reservoir indicated that if Sam Rayburn Dam had been in place in 1915, the regulated peak for that event would have been approximately one fourth of its recorded magnitude (USACE Fort Worth District, 1971). Similarly, the 1944 flood magnitude would have been cut in half with Sam Rayburn Dam in place (USACE Fort Worth District, 1971). The 1884 flood, on the other hand, would have been largely unchanged since it originated on the upper Neches River. Given the current regulated condition of the watershed, the HEC-HMS elliptical storm results are less likely to change and are consistent with the known information concerning the largest observed floods at Evadale.

Table 11.13: Frequency Flow (cfs) Results Comparison for **Village Creek near Kountze, TX**

Annual Exceedance Probability (AEP)	Return Period (years)	Currently Effective FEMA FIS	BLE Data - Regression Equations	Statistical Analysis of the Gage Record (94 years)	Alternate Statistical Analysis with Harvey's uncertainty	HEC-HMS Uniform Rain Frequency Storm	HEC-HMS Elliptical Frequency Storm
0.002	500	120,000	196,363	225,000	191,000	165,600	142,500
0.005	200			152,000	134,000	126,600	107,300
0.01	100	73,880	103,234	111,000	101,000	98,100	80,400
0.02	50	58,400	75,945	80,100	74,700	71,000	59,200
0.04	25		54,458	56,200	53,700	54,100	45,200
0.1	10	28,930	33,121	33,200	32,800	33,000	32,400
0.2	5			20,900	21,000	21,100	20,600
0.5	2			9,190	9,410	9,400	8,900

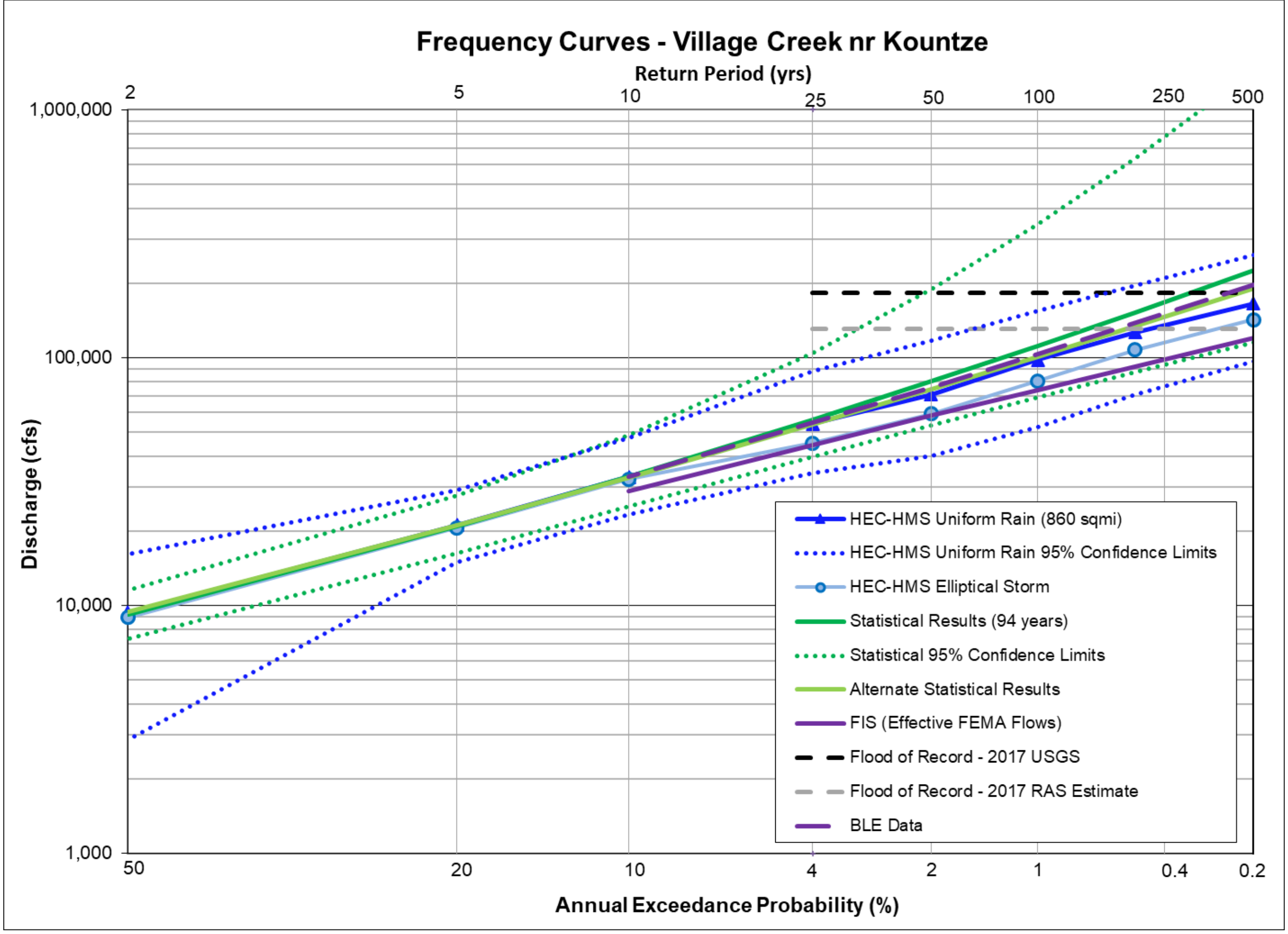


Figure 11.13a: Flow Frequency Curve Comparison for Village Creek near Kountze, TX

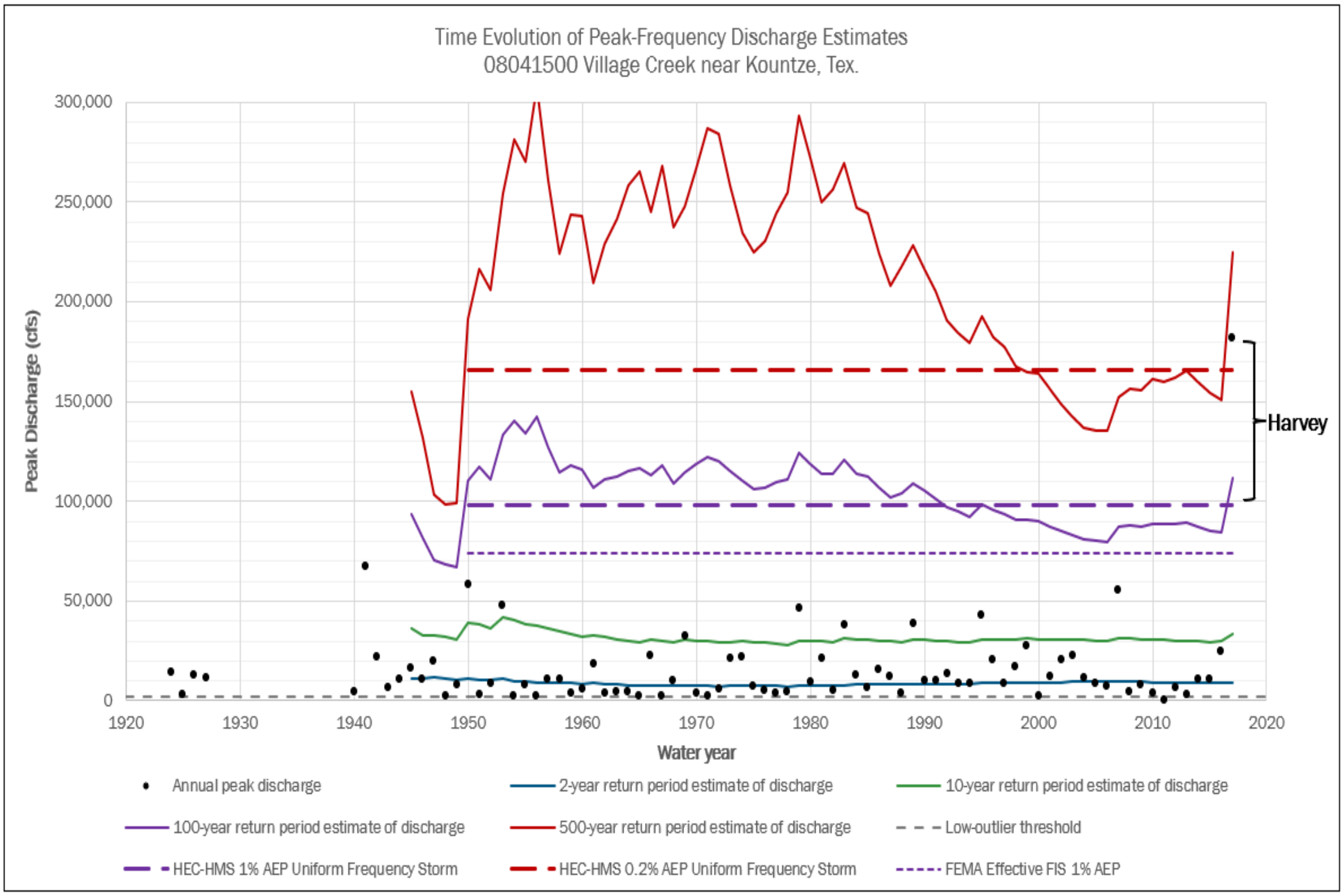


Figure 11.13b: Statistical Change Over Time Comparison for Village Creek near Kountze, TX

Village Creek near Kountze, Texas is a USGS stream gage with about 860 square miles of drainage area, which is located on a tributary to the Neches River. The USGS gage has a period of record of 94 years, which is a fairly long period of record for Texas. The published BLE data for this location was based off of a statistical analysis of the gage record, which is why its numbers are very close to the current statistical results.

Figure 11.13a shows that there is a relatively small difference between the uniform rain and elliptical storm HEC-HMS results at the 2% through 0.2% AEP events. This difference is primarily due to the uniform rain method's higher rainfall volume. However, the drainage area for this location is small enough that both rainfall methods produce reasonable results in HEC-HMS.

Figure 11.13a also shows that the HEC-HMS uniform rain results are similar to the alternate statistical results that accounted for some uncertainty in the peak discharge for Hurricane Harvey. There was a high degree of uncertainty in the USGS peak discharge estimate of 182,000 cfs for Harvey at this location. The peak observed gage height during Hurricane Harvey exceeded any previous observed event at that location by over 8 feet and was well beyond the existing rating curve for the gage. The USGS estimated the peak flow for Hurricane Harvey indirectly after the fact, and they assigned an uncertainty of +/- 20% to that flow value. An alternate estimate of the Hurricane Harvey peak discharge on Village Creek was made by the InFRM team using an existing HEC-RAS model from the BLE data (FEMA, 2019). By applying a range of low to high roughness n-values in the hydraulic model, an interval estimate of 100,000 to 175,000 cfs was determined as the range of peak discharges that corresponded to the recorded peak gage height during Hurricane Harvey. The alternate statistical analysis used this interval estimate to account for Harvey's peak discharge uncertainty.

Figure 11.13b shows that the 1% and 0.2% AEP HEC-HMS uniform rain peak discharges are well within the range of the statistical estimates that have been experienced over time for those frequencies. Figure 11.13b also shows that the 1% and 0.2% AEP discharge estimates were significantly affected by Hurricane Harvey. In fact, the HEC-HMS results were in close agreement with the statistical results for the two decades prior to Harvey. Hurricane Harvey produced a basin average rainfall of 27 inches in 5 days in the Village Creek watershed, which equates to about a 0.2% AEP (500-yr) rainfall according to NOAA Atlas 14. Both the high rainfall amounts and the uncertainty in the peak discharge for Hurricane Harvey point to a likelihood that the current statistical analysis is overestimating the rare discharges. Figure 11.13b also shows the dramatic impact that a single large flood event can have on the statistical results, even after 94 years worth of record.

Table 11.14: Frequency Flow (cfs) Results Comparison for Pine Island Bayou near Sour Lake, TX

Annual Exceedance Probability (AEP)	Return Period (years)	Preliminary FEMA FIS for Jefferson County	BLE Data - Regression Equations	Statistical Analysis of the Observed Gage Record (53 years)	HEC-HMS Uniform Rain Frequency Storm	HEC-HMS Elliptical Frequency Storm
0.002	500	40,700	80,840	147,000	72,217	67,850
0.005	200			91,700	53,632	50,667
0.01	100	26,300	44,790	63,200	41,780	39,184
0.02	50	21,400	33,680	42,800	30,508	28,346
0.04	25		24,650	28,500	23,081	21,611
0.1	10	11,400	15,360	15,800	14,654	14,413
0.2	5			9,600	9,364	9,221
0.5	2			4,190	4,387	4,249

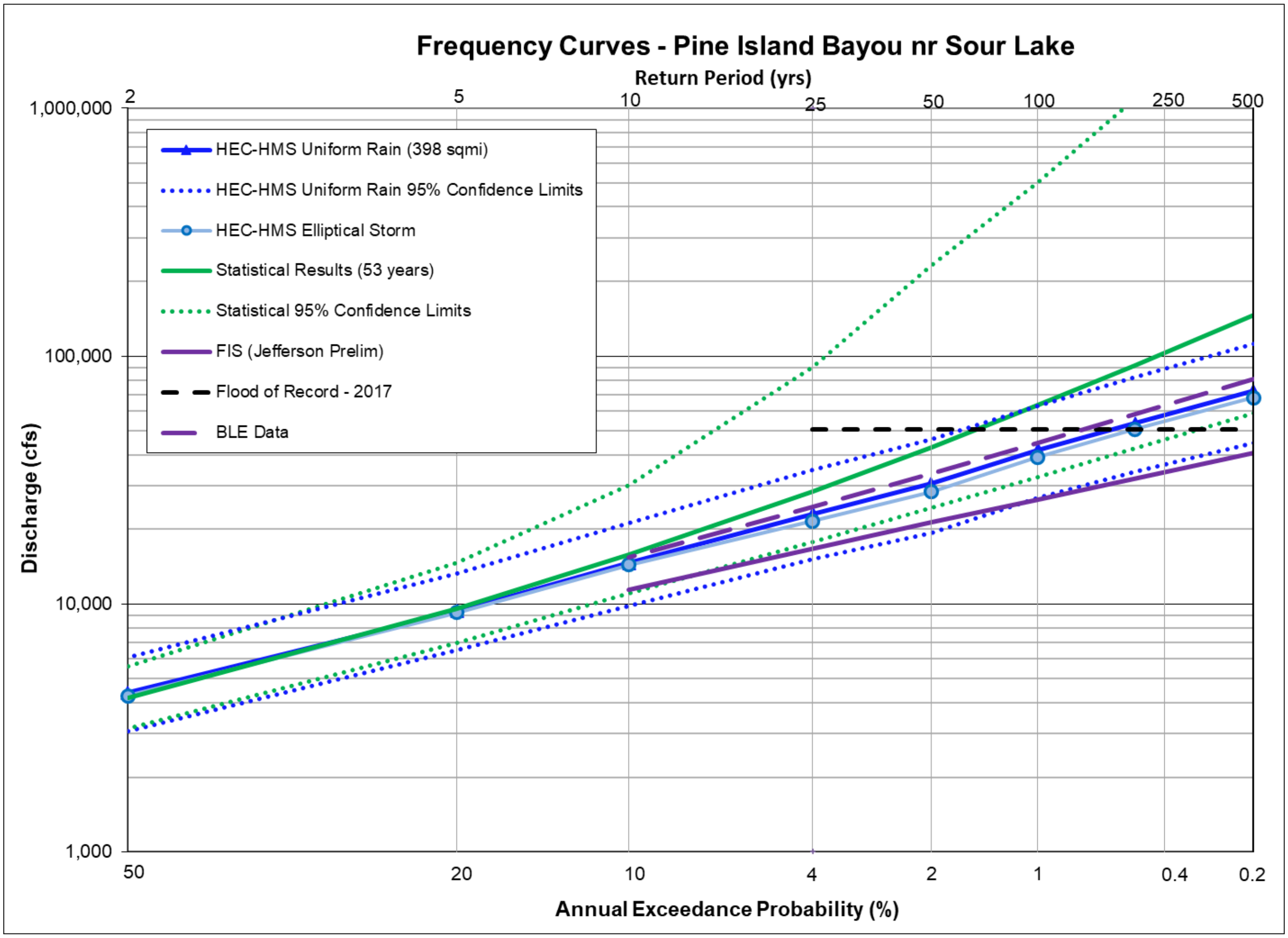


Figure 11.14a: Flow Frequency Curve Comparison for Pine Island Bayou near Sour Lake, TX

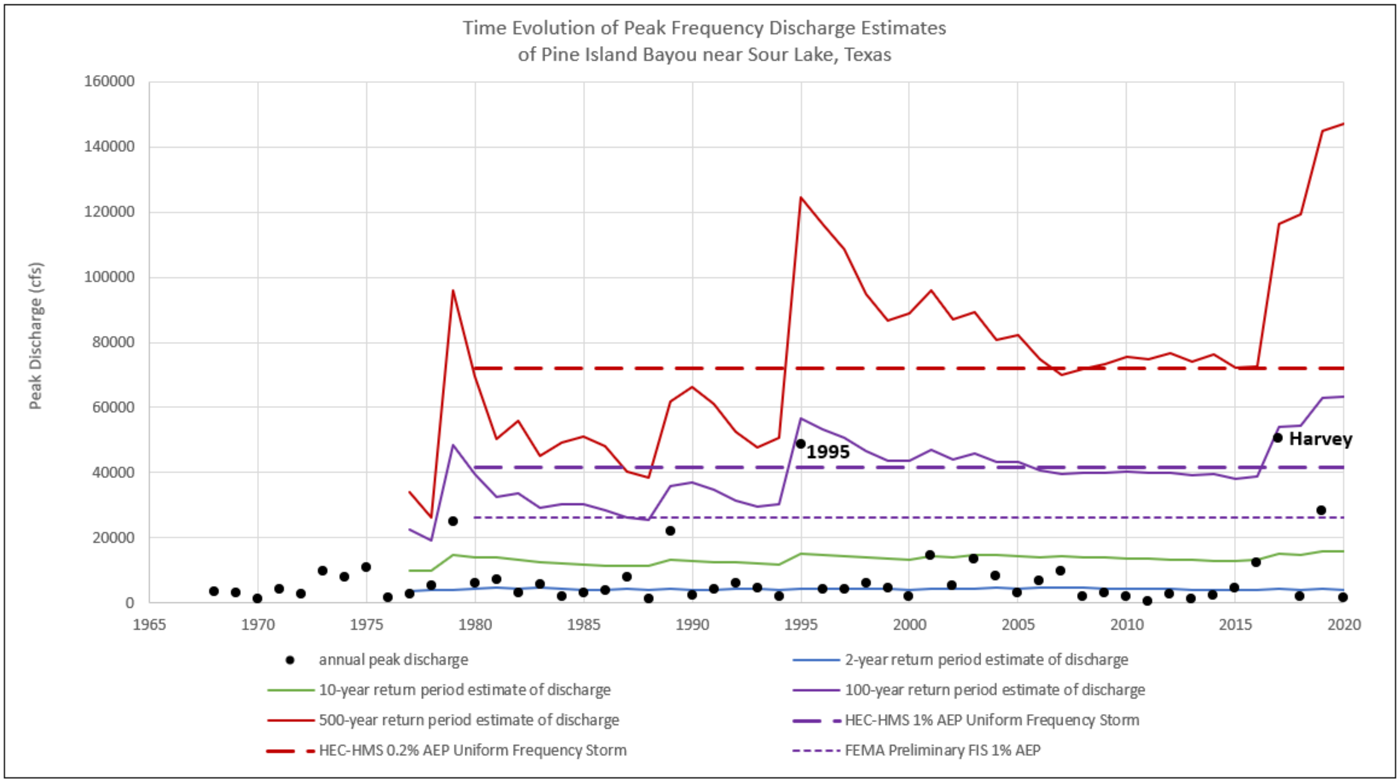


Figure 11.14b: Statistical Change Over Time Comparison for Pine Island Bayou near Sour Lake, TX

Pine Island Bayou near Sour Lake, Texas is a USGS stream gage with just under 400 square miles of drainage area, which is located on a tributary to the Neches River. The USGS gage at this location has a moderate length of record of 53 years. The published BLE data for this location was based off of regression equations, which are an approximate method of hydrology. Coincidentally, the BLE discharge estimates were very close to the HEC-HMS results at this location, as shown in Figure 11.14a. This location also includes published discharge estimates from the Jefferson County Preliminary Flood Insurance Study (FIS) (FEMA, 2011). Unfortunately, the preliminary FIS discharges are at or below the 95% confidence limits from both the statistical analysis and the HEC-HMS results. This means that there is at least a 95% chance that the preliminary FIS discharges are underestimating the flood risk at this location.

Figure 11.14a shows that there is very little difference between the uniform rain and elliptical storm HEC-HMS results at this location, which is to be expected due to its relatively small drainage area. This means that both rainfall methods can be considered to produce reasonable frequency discharge estimates in HEC-HMS.

Figure 11.14a also shows that the HEC-HMS results are significantly lower than the statistical results at the 1% through 0.2% AEP frequencies. However, the statistical change over time plot for this location (Figure 11.14b) shows that the 1% and 0.2% AEP discharge estimates were significantly affected by Hurricane Harvey, while little to no effect is seen in the 10% and 50% AEP statistical estimates. Figure 11.14b also shows that the 1% and 0.2% AEP HEC-HMS peak discharges are well within the range of the statistical estimates that have been experienced over time for those frequencies. Figure 11.14b shows that the HEC-HMS results were in close agreement with the statistical results for the ten years prior to Harvey. Hurricane Harvey produced a basin average rainfall of 36 inches in 5 days in the Pine Island Bayou watershed. This equates to about a 0.2% AEP (500-yr) rainfall according to NOAA Atlas 14. Similarly, the 1995 annual peak resulted from an October 1994 storm which recorded over 28 inches of rain on Pine Island Bayou in four days. This equates to greater than a 1% AEP (100-yr) rainfall according to NOAA Atlas 14. The high rainfall amounts and dramatic impact of Hurricane

Harvey on the statistical results point to a likelihood that the current statistical results are likely overestimating the rare frequency discharges. Therefore, the HEC-HMS results are considered more reliable than the statistical results for the rare frequency events such as the 1% AEP.

Table 11.15: Frequency Flow (cfs) Results Comparison for Pine Island Bayou above BI Pump Plant

Annual Exceedance Probability (AEP)	Return Period (years)	Currently Effective FEMA FIS, Hardin Co	BLE Data - Regression Equations	Statistical Analysis of the Observed Gage Record (17 years)	HEC-HMS Uniform Rain Frequency Storm	HEC-HMS Elliptical Frequency Storm
0.002	500	49,910	147,102	189,000	107,600	94,700
0.005	200			128,000	79,200	69,200
0.01	100	39,680	82,441	93,900	61,200	52,900
0.02	50	33,900	62,151	67,500	43,800	36,600
0.04	25		45,520	47,400	32,600	27,600
0.1	10	23,610	28,270	28,100	20,300	17,600
0.2	5			17,700	13,000	11,400
0.5	2			7,860	6,200	6,100

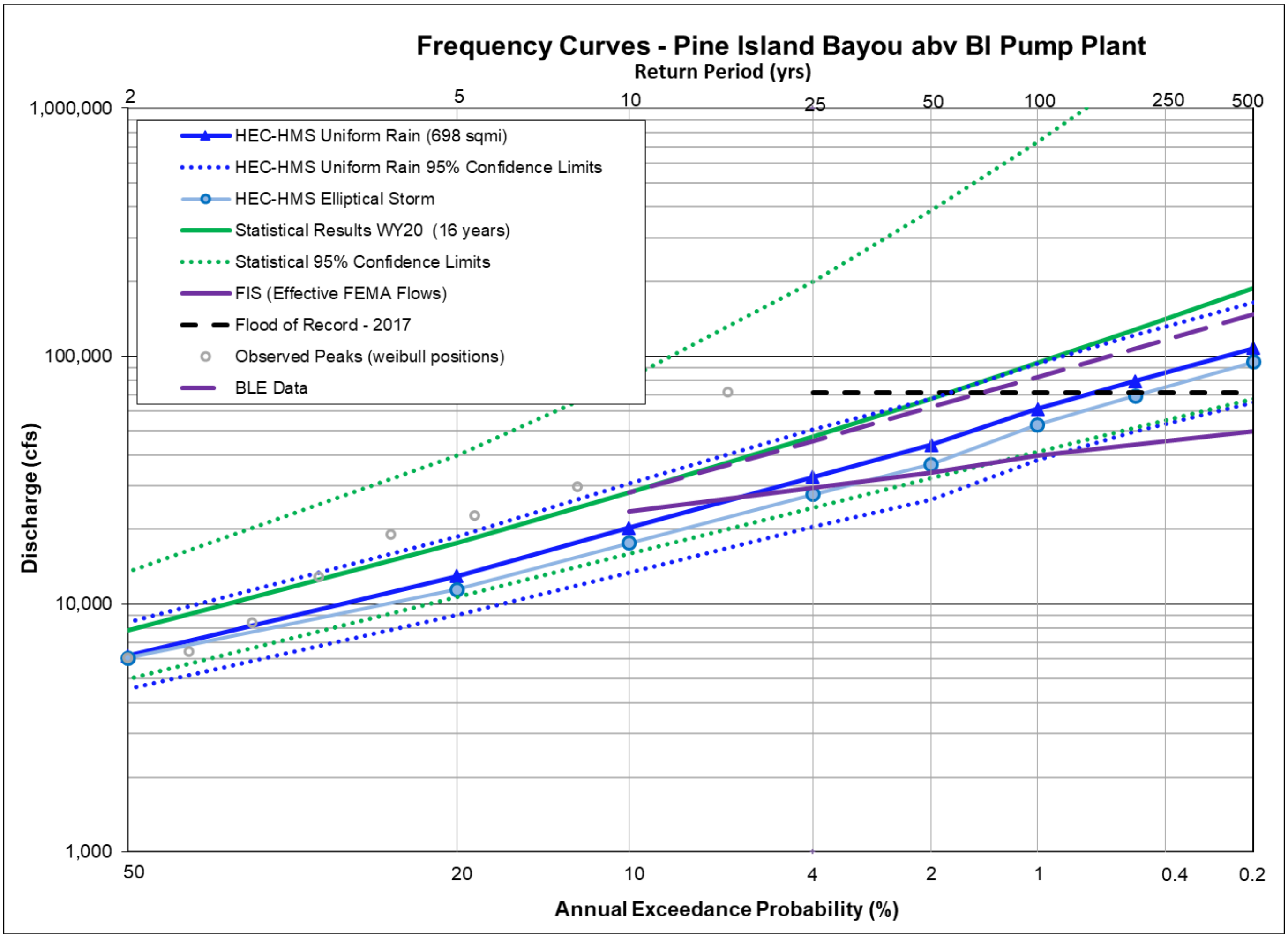


Figure 11.15: Flow Frequency Curve Comparison for Pine Island Bayou above BI Pump Plant

Pine Island Bayou above the BI Pump Plant near Beaumont, Texas is a relatively new USGS stream gage with almost 700 square miles of drainage area, which is located on a tributary to the Neches River. The USGS gage at this location has a very short length of record of less than 20 years, which means that the statistical results can only be considered trustworthy for the very frequent events, such as the 50% AEP (2-yr) discharge. The published BLE data for this location was based off of a statistical analysis, which is why those discharges are relatively close to the statistical results, as shown in Figure 11.15. This location also includes published discharge estimates from the Hardin County effective Flood Insurance Study (FIS) (FEMA, 2010). Unfortunately, Figure 11.15 also shows that the effective FIS discharges are at or below the 95% confidence limits from both the statistical analysis and the HEC-HMS results for the 1% and 0.2% AEP. This means that there is at least a 95% chance that the effective FIS discharges are underestimating the flood risk at this location.

Figure 11.15 shows that there is a slightly larger difference between the uniform rain and elliptical storm HEC-HMS results at this location, which is to be expected due to the increase in drainage area between Sour Lake and BI Pump Plant. Figure 11.15 also shows that both HEC-HMS results are significantly lower than the statistical results. However, with only 20 years of record, the statistical results cannot be relied on for a 1% AEP (100-yr estimate). In fact, Figure 11.15 shows that the current statistical results are at the upper 95% confidence limit of the HEC-HMS results. Due to the short period of record, there is no change over time plot for this location, but we still know that the statistical results were significantly affected by Hurricane Harvey, which produced a basin average rainfall of 36 inches in 5 days in the Pine Island Bayou watershed and had a peak observed flow of 71,500 cfs. The magnitude of this event far exceeds what would normally be expected in a gage with only 20 years of record, which is why the statistical curve does not fit well with the observed peaks shown in Figure 11.15. Therefore, the HEC-HMS results are considered more reliable than the statistical results at this location.

Table 11.16: Frequency Flow (cfs) Results Comparison for the Neches River at the Saltwater Barrier, TX

Annual Exceedance Probability (AEP)	Return Period (years)	Currently Effective FEMA FIS Jefferson County	BLE Data - Regression Equations	Statistical Analysis of the Observed Gage Record (16 years)	Statistical Analysis of the Extended RiverWare Record (90 years)	HEC-HMS Uniform Rain Frequency Storm	HEC-HMS Elliptical Frequency Storm
0.002	500	240,000	115,604	399,000	288,000	359,900	270,000
0.005	200			298,000	217,000	265,400	197,600
0.01	100	136,000	83,747	235,000	174,000	203,900	150,400
0.02	50	107,000	71,849	184,000	137,000	146,800	112,900
0.04	25		61,140	141,000	105,000	112,900	91,500
0.1	10	60,000	47,123	95,800	71,800	77,500	68,900
0.2	5			68,200	50,900	55,000	50,500
0.5	2			37,800	27,700	33,300	29,200

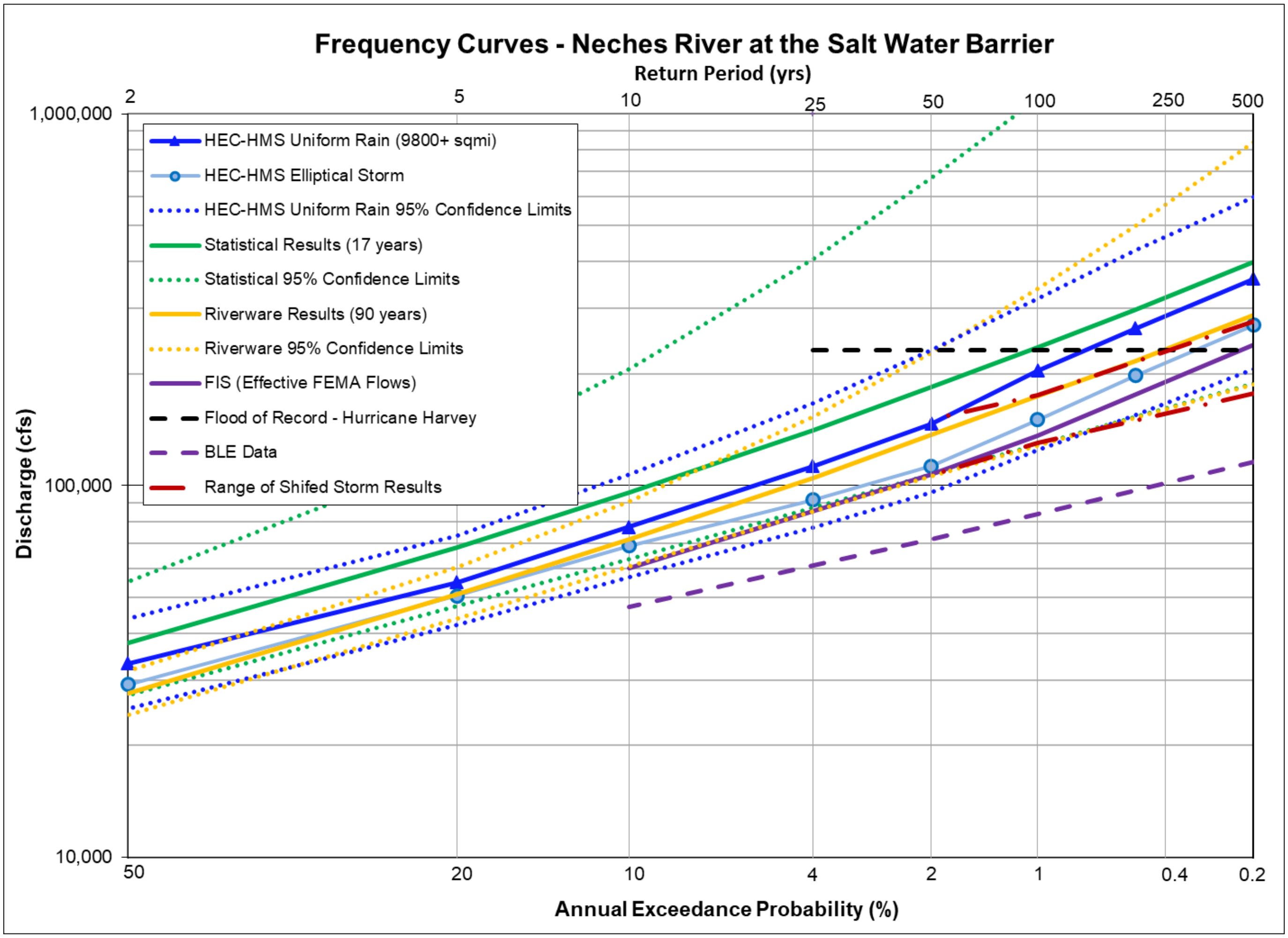


Figure 11.16a: Flow Frequency Curve Comparison for the Neches River at the Saltwater Barrier, TX

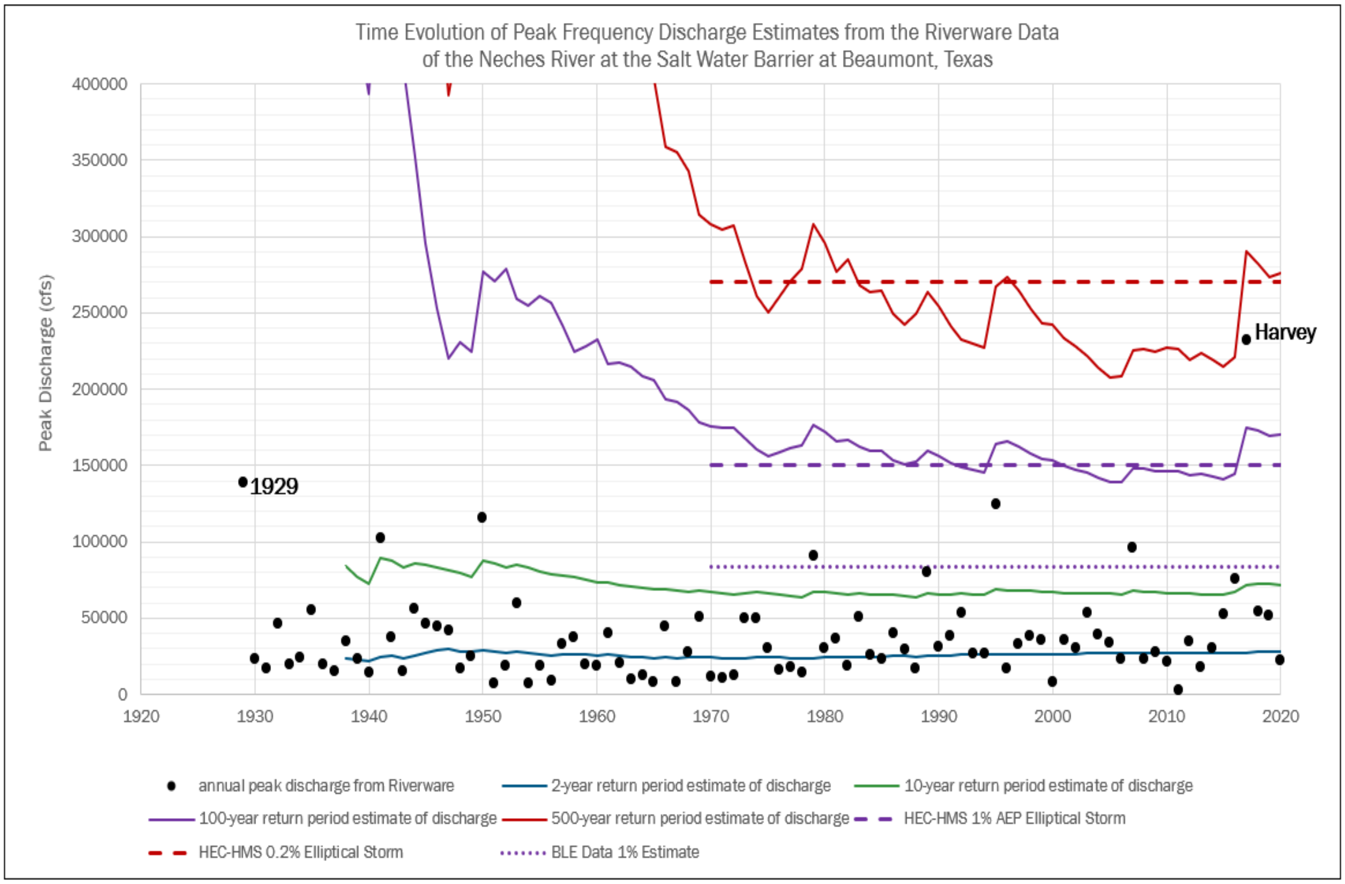


Figure 11.16b: Statistical Change Over Time Comparison for the Neches River at the Saltwater Barrier, TX

The Neches River at the Saltwater Barrier is the most downstream USGS stream gage in the study area, with a drainage area of over 9,800 square miles. The USGS gage at this location has a very short length of record of less than 20 years, which means that the statistical results can only be considered trustworthy for the very frequent events, such as the 50% AEP (2-yr) discharge. The published BLE data for this location was based off of regression equations, which are an approximate method of hydrology (FEMA, 2019). This location also includes published discharge estimates from the Jefferson County effective Flood Insurance Study (FIS) (FEMA, 2002). Unfortunately, Figure 11.16a shows that the BLE discharges are well below the 95% confidence limits from all the other analyses, and the effective FIS discharges are close to the lower 95% confidence limits. This means that there is greater than a 95% chance that the BLE discharges are underestimating the flood risk at this location.

Figure 11.16a also shows that there is a significant difference between the uniform rain and elliptical storm HEC-HMS results at the 1% through 0.2% AEP events. This is due to the uniform rain method's tendency to overestimate the total rainfall volume for very large drainage areas like this one. Figure 11.16a also shows that both HEC-HMS elliptical storm results are significantly lower than the statistical results. However, with less than 20 years of record, the statistical results cannot be relied on for a 1% AEP (100-yr estimate). That period of record was extended to 90 years using data from the RiverWare model for the early part of the record, and Figure 11.16a shows that the RiverWare results are still slightly higher than the elliptical storm results for the rare events like the 1% and 0.2% AEP.

As an additional validation of the results at this location, a storm shifting analysis was performed, as described in Appendix G. The storm shifting analysis took recent storms from nearby watersheds, including October 2006, May 2016, and August 2017 (Hurricane Harvey), and shifted them to an optimized location above the Saltwater

Barrier. A range of loss rates were then applied to the shifted storms in HEC-HMS, ranging from the 50% to the 1% AEP frequency loss rates. The red lines on Figure 11.16a illustrate the range of discharges that resulted from the shifted storms. The HEC-HMS elliptical storm results were well within the range of results from the shifted storms, which is another validation that the elliptical storm results are reasonable.

Figure 11.16b is a change over time statistical plot that was based off of the 90 years of RiverWare data. This plot shows that the current RiverWare statistical results were significantly affected by Hurricane Harvey, which produced rainfall totals of 26 to 40 inches in 5 days over the lower portion of the Neches River basin and had a peak observed flow of 232,000 cfs. Hurricane Harvey exceeded the previous record stage and the existing rating curve by over 10 feet at this location; therefore, the USGS estimated that there is at least +/- 20% uncertainty in the actual peak discharge for that event. Figure 11.16b also shows that the 1% and 0.2% AEP elliptical storm peak discharges are well within the range of the statistical estimates that have been experienced over time for those frequencies. In fact, the 1% AEP elliptical storm peak discharge aligns very well with the RiverWare 1% AEP estimates for the three decades prior to Hurricane Harvey.

11.2 LAKE ELEVATION COMPARISONS

Sam Rayburn Dam and Reservoir is a USACE reservoir on the Angelina River. It has a normal surface area of over 112,000 acres and a drainage area of about 3,450 square miles. The reservoir has a very large flood storage capacity of over 1.5 million acre-feet between the top of conservation pool and the spillway crest. The dam has hydropower turbines, a gated outlet works, and a labyrinth uncontrolled spillway. The spillway at Sam Rayburn has never been engaged as of this writing, but the observed pool elevations have come within about one foot of the spillway crest on more than one occasion. Figure 11.17 compares the observed pool elevation data to the results from the RMC-RFA reservoir analysis, the HEC-HMS modeling, and the previous water surface elevations from the BLE data and the 2009 USACE estimates. This figure shows that the RMC-RFA results clearly have the best fit to the observed data. This is because the RMC-RFA methodology better accounts for the variable starting pool elevations and inflow volumes that can be experienced by the reservoir during a flood event. The HEC-HMS modeling, on the other hand, assumes a single starting pool elevation at the top of conservation pool for each frequency storm. Since Sam Rayburn has such a large flood storage capacity, high pool elevations are often the result of a series of storms rather than a single storm, and the RMC-RFA analysis better accounts for these conditions.

Town Bluff Dam and B.A. Steinhagen Lake is a USACE reservoir on the Neches River downstream of the confluence of the Neches River with the Angelina River, near the town of Town Bluff, Texas. B.A. Steinhagen Lake has a drainage area of over 7,500 square miles, but the lake has very little capacity allocated to storing flood waters. There are only two feet of elevation between the top of normal pool and the uncontrolled spillway and only 57,700 acre-feet of flood storage in that range. By comparison, Sam Rayburn has 1.5 million acre-feet of flood storage and only half the drainage area of B.A. Steinhagen. At normal pool and below, B.A. Steinhagen Lake tries to maintain releases at or below the downstream channel capacity of 20,000 cfs, but during large inflow events, such as a 1% AEP storm, the dam is operated with outflows essentially equal to inflows in order to keep the pool at a relatively level elevation. Figure 11.18 compares the observed pool elevation data at B.A. Steinhagen to the results from the RMC-RFA reservoir analysis, the HEC-HMS modeling, and the previous water surface elevations from the BLE data and the 2009 USACE estimates. This figure shows that the RMC-RFA results have the best fit to the observed data, but the HEC-HMS elliptical storm results are a close second. The elliptical storm frequency pool elevations are very close to the RMC-RFA results for the larger storms such as the 1% through 0.2% AEPs, but they are about half a foot lower than the RMC-RFA results at the frequent end of the curve (50% through 4% AEPs). This difference is likely due to the starting pool elevation assumptions. The RMC-RFA methodology uses variable starting pool elevations in a monte carlo analysis, while the HEC-HMS modeling assumes a single starting pool elevation at the top conservation pool for each frequency storm. However, since B.A. Steinhagen Lake has very limited flood storage, this difference only amounts to about half a foot in elevation.

Both Figure 11.17 and Figure 11.18 also show that the published BLE water surface elevations for these reservoirs are below the normal conservation pool elevation and far below the observed data. This is because the BLE data does not account for hydraulic structures like dams and reservoirs. Therefore, the BLE water surface elevations were calculated based on a natural valley assumption that did not account for the effects of the reservoirs.

Table 11.17: Frequency Pool Elevation (ft NAVD88) Comparison for Sam Rayburn Reservoir

Annual Exceedance Probability (AEP)	Return Period (years)	Currently Effective FEMA FIS	BLE Data - Elevations upstream of the dam	Previous 2009 USACE Estimate	HEC-HMS Uniform Rain Frequency Storm	HEC-HMS Elliptical Frequency Storm	Reservoir Analysis in RMC-RFA
0.002	500		162.0		178.5	175.9	177.6
0.005	200				176.5	173.3	177.1
0.01	100		161.6	176.5	174.2	171.4	176.4
0.02	50		161.5	176.0	171.7	169.4	174.5
0.04	25		161.3	175.0	169.9	168.2	174.3
0.1	10		161.0	173.3	167.9	167.0	172.3
0.2	5			172.1	166.7	166.2	170.3
0.5	2			166.7	165.7	165.2	166.9

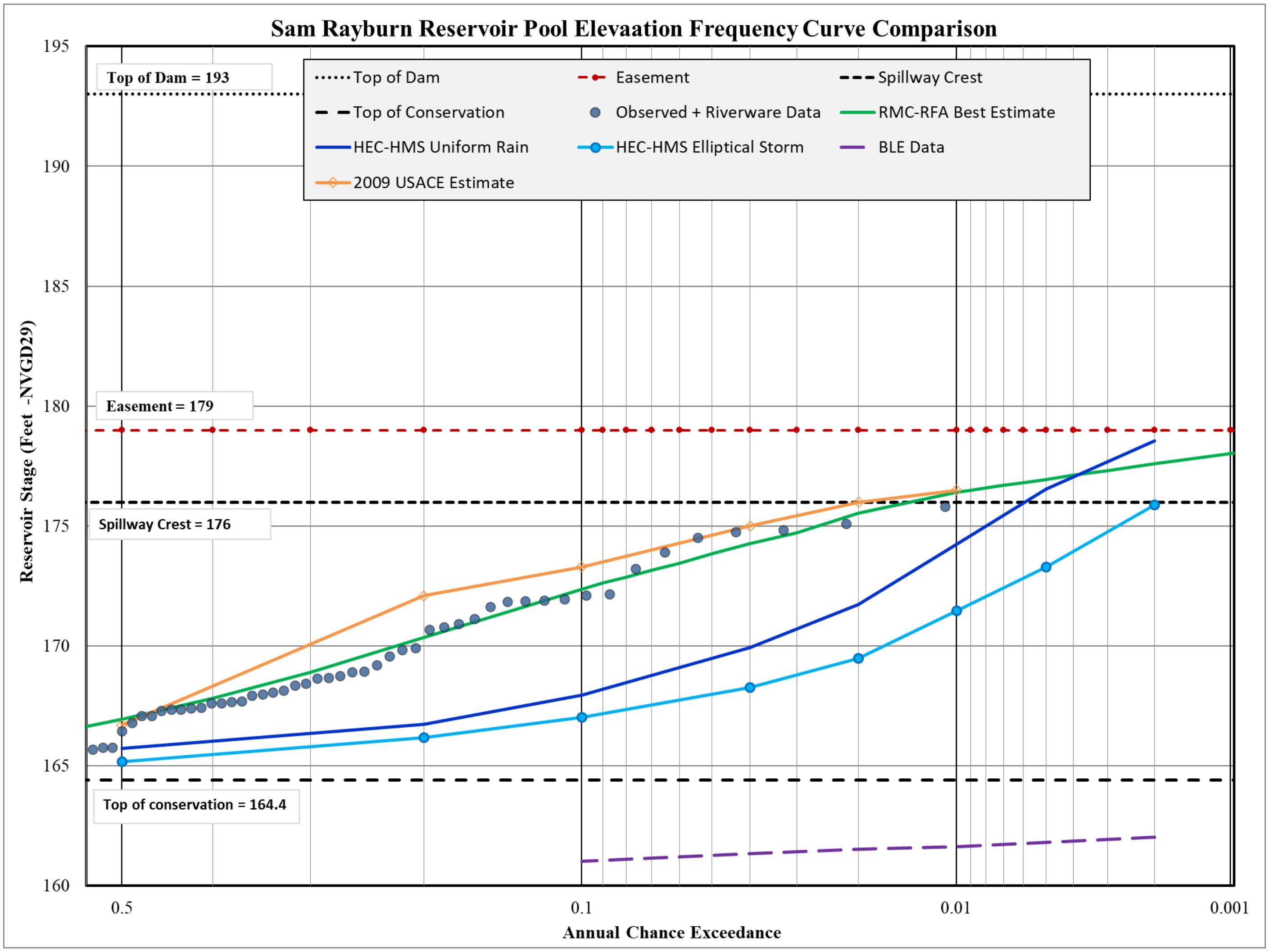


Figure 11.17: Pool Elevation Frequency Curve Comparison for Sam Rayburn Reservoir

Table 11.18: Frequency Pool Elevation (feet NAVD88) Comparison for B.A. Steinhagen Lake

Annual Exceedance Probability (AEP)	Return Period (years)	Currently Effective FEMA FIS	BLE Data - Elevations upstream of the dam	Previous 2009 USACE Estimate	HEC-HMS Uniform Rain Frequency Storm	HEC-HMS Elliptical Frequency Storm	Reservoir Analysis in RMC-RFA
0.002	500		83.1		90.7	86.9	86.9
0.005	200				87.4	85.8	86.2
0.01	100		81.6	85.0	86.0	85.0	85.4
0.02	50		81.2	84.5	84.5	84.3	85.0
0.04	25		80.9	84.0	83.9	83.8	84.5
0.1	10		80.5	83.8	83.6	83.6	84.1
0.2	5			83.6	83.3	83.3	83.9
0.5	2			83.2	83.0	83.0	83.5

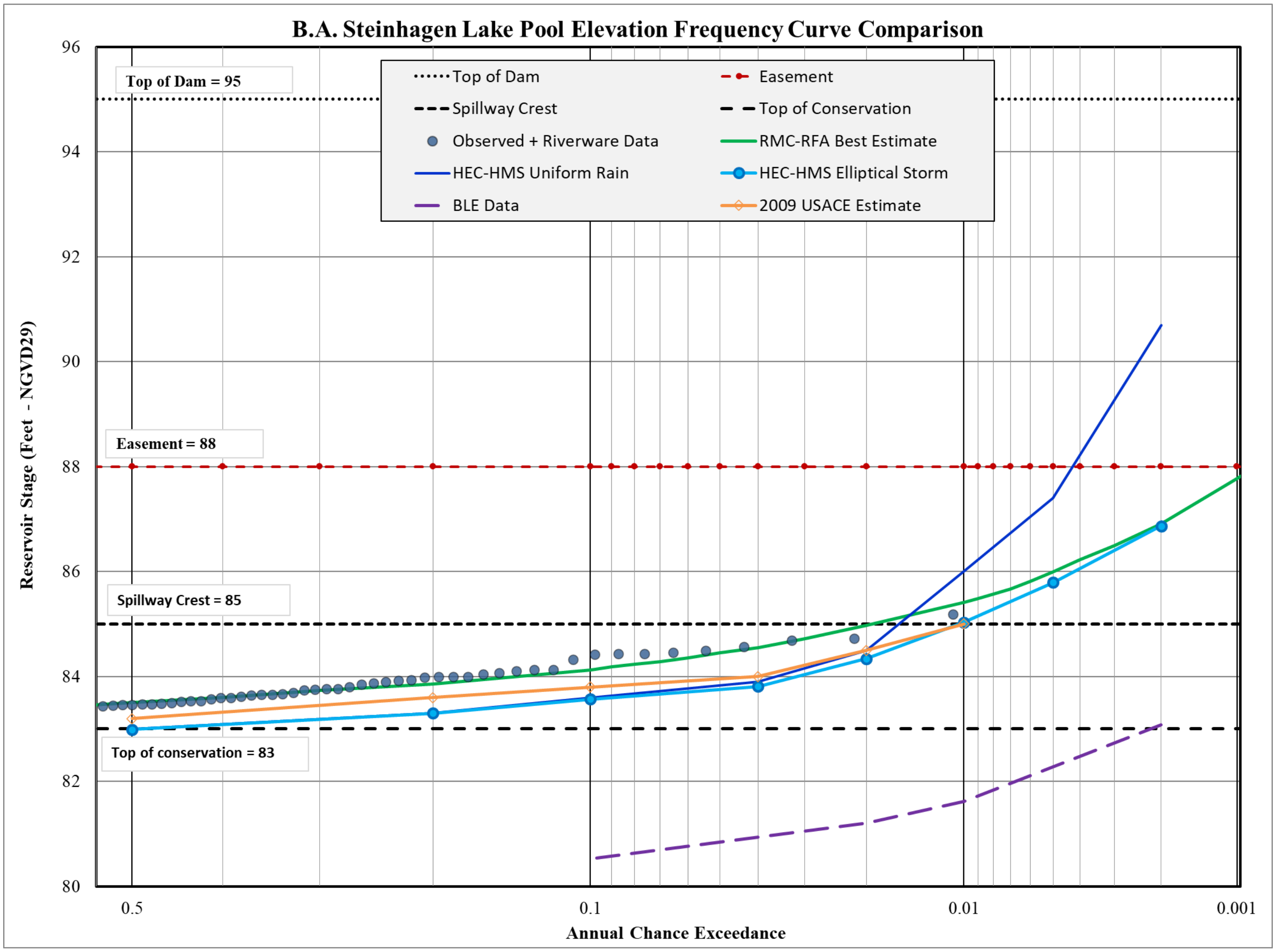


Figure 11.18: Pool Elevation Frequency Curve Comparison for B.A. Steinhagen Lake

12 Frequency Flow Recommendations

The final recommendations for the InFRM Watershed Hydrology Assessments are formulated through a rigorous process which requires technical feedback and collaboration between all of the InFRM subject matter experts. This process includes the following steps at a minimum: (1) comparing the results of the various hydrologic methods to one another, (2) performing an investigation into the reasons for any significant differences in results at each location in the watershed, (3) selecting the draft recommended methods, (4) performing internal and external technical reviews of the hydrologic analyses and the draft recommendations, and finally, (5) finalizing the study recommendations.

After completing this process for the Neches River basin, the frequency discharges that were recommended for adoption by the InFRM team were a combination of the results from the following methods: HEC-HMS NOAA Atlas 14 uniform rain frequency storms (Chapter 6), HEC-HMS NOAA Atlas 14 elliptical frequency storms (Chapter 7), and RMC-RFA Reservoir Analyses (Chapter 9). Detailed breakouts of the recommended frequency discharges and pool elevations for each location in the watershed are given in Tables 12.1 and 12.2.

The statistical results from Chapter 5 and the RiverWare statistical results from Chapter 8 were used as points of comparison, especially at the frequent end of the curves, but the InFRM team chose not to adopt the statistical flow frequency results directly. One reason for this decision was the tendency of the statistical results to change after each significant flood event, as demonstrated in the statistical change over time comparison figures in Chapter 11. In addition, climate variability from wet to dry conditions can result in non-representative samples in the gage record. The statistical frequency analyses and RiverWare results support the HEC-HMS results by demonstrating that they are generally within one another's confidence limits, especially for the 1% and 0.2% AEP events of interest for FEMA floodplain mapping.

Rainfall runoff modeling, on the other hand, is based on physical watershed characteristics, such as drainage area and stream slope, that do not tend to change as much over time. Climate variability can also be accounted for in the watershed model by using regional rainfall information from NOAA Atlas 14 and by adjusting soil loss rates to be consistent with observed storms and appropriate for the rarity of the event in question. Another reason for the selection of the HEC-HMS modeling discharges was the ability to directly calculate frequency discharges for locations within the Neches River watershed that do not coincide with a stream gage.

Rainfall-runoff modeling in HEC-HMS was used to simulate the physical processes that occur in the Neches watershed during intense storm events, including the movement of water across the land surface and through the streams and rivers. The HEC-HMS model for the Neches River basin underwent extensive calibration to accurately simulate the response of the watershed to a range of observed flood events, including large events similar to a 1% ACE (100-yr) flood. In fact, a total of eight recent storm events were used to fine tune the HEC-HMS model; thereby bestowing a high degree of confidence in the HEC-HMS model's results. For the Mud Creek and Upper Angelina River, the 2D HEC-RAS analysis from Chapter 10 was also used to inform the HEC-HMS model parameters and ensure that the model was accurately simulating the response of the watershed to intense storm events.

Chapter 11 discusses examples in the Neches basin where one gage location has been hit heavily by large storms and another is missed almost entirely. These non-representative samples can bias the statistical frequency flow results up or down, depending on the storms that have occurred above that particular gage.

In addition to extensive calibration, best available precipitation frequency estimates from NOAA Atlas 14 (NOAA, 2018) were used to build frequency storms within the HEC-HMS model. There are a couple factors that make NOAA Atlas 14 the most accurate, up-to-date, and comprehensive study of rainfall depths in Texas. First, the

NOAA Atlas 14 study contained an additional 23 years of rainfall data compared to the previous precipitation product developed by the USGS in 2004, which only included data through 1994. Some of the largest storms on record in the Neches River basin have occurred within the last 23 years, most notably Hurricane Harvey in 2017 on the lower portion of the Neches River basin. Secondly, NOAA Atlas 14 used a regional statistical approach that incorporated at least 1,000 cumulative years of daily data and 500 cumulative years of sub-daily data into each station's rainfall frequency estimate. This regional approach yielded better estimates of rare rainfall depths such as the 1% and 0.2% AEP (100-yr and 500-yr) depths. For these reasons, the calibrated HEC-HMS watershed modeling with the NOAA Atlas 14 rainfall depths was adopted as having the most complete accounting of both the historic rainfall data and the physical processes at work in the watershed.

Between the uniform rain and the elliptical frequency storms in HEC-HMS, the uniform rain method is simpler and well suited for smaller drainage areas, while the elliptical storm method is more complex and better suited for larger drainage areas. Both this study and the previous InFRM Watershed Hydrology Assessments have confirmed that the results of the uniform rainfall method are generally reasonable up to at least 1,000 square miles (InFRM, 2019) (InFRM, 2021). For larger drainage areas in the Neches River basin, which ranged up to nearly 10,000 square miles, the elliptical storm results from HEC-HMS did a better job of producing reasonable runoff volumes and subsequently peak stream flows. Table 12.1 indicates the locations where the recommended results transitioned from uniform rainfall results to elliptical storm results on each stream and river. The exact locations of the transitions between uniform and elliptical storms generally occurred at locations with drainage areas between 400 and 1,100 square miles and were placed at significant confluences to avoid any jumps or dips in the peak flows due to a change in the rainfall method.

For the USACE reservoirs in the Neches River basin, the recommended frequency pool elevations and releases were calculated in the RMC-RFA reservoir analyses from Chapter 9. These reservoir analyses were performed for the two USACE reservoirs within the basin: Sam Rayburn Reservoir and B.A. Steinhagen Lake. The RMC-RFA analyses utilized stochastic techniques and had the most comprehensive accounting for the operations of the dam, the frequency of its inflow volumes, and the range of its starting pool elevations. This type of detailed reservoir analysis lends a higher level of confidence to the resulting frequency estimates of its pool elevations. The resulting recommended frequency pool elevations are shown in Table 12.2. This table also contains recommended frequency pool elevations using methods less comprehensive than the RMC-RFA analysis method. While these results do represent a picture of flood risk using best available scientific modeling and information, they are unable to fully account for all the variables such as starting pool elevation, variances in reservoir operation, and inflow hydrograph shape and duration variation that make up the true or actual frequency pool elevations for a reservoir, which are better accounted for in the reservoir analysis methodology presented in Chapter 9. These less comprehensive methods are recommended as a starting point for pool frequency information where there is no existing information or where the information is less detailed. It is recommended that more detailed and comprehensive analyses be performed, such as in the reservoir analysis methods applied within this study, when possible. The corresponding frequency outflows from the reservoir analyses as well as frequency peak flows for the rest of the watershed are presented in Table 12.1.

For the reaches downstream of USACE dams, there are two distinct sources of flooding: (1) a large release from the dam and (2) rainfall runoff from the local drainage area downstream of the dam. The first flooding source was analyzed through the RMC-RFA reservoir analysis methods. For the second flooding source, peak flows from the local rainfall runoff were calculated in the HEC-HMS model with the NOAA Atlas 14 rainfall patterns of Chapters 6 and 7. The frequency peak flows from these two flooding sources were then compared to one another for each reach of the river, and the higher of the two peak flows were recommended for adoption. In general, the results showed that releases from USACE dams dominate the Angelina and Neches River discharges immediately downstream of the dam, but that the flows from the local rainfall runoff quickly become dominant as one moves further downstream.

Table 12.1: Summary of Recommended Frequency Peak Discharges (cfs) for the Neches River Basin

Location Description	Drainage Area	50%	20%	10%	4%	2%	1%	0.50%	0.20%	Hydrologic Method
	sq mi	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	200-YR	500-YR	
Neches River above Prairie Creek	127.0	4,800	9,300	12,600	17,300	21,400	26,400	31,200	37,800	HEC-HMS Uniform Rainfall
Prairie Creek above Neches River	89.8	6,000	10,400	13,700	18,300	22,200	26,800	31,400	37,900	HEC-HMS Uniform Rainfall
Neches River below Prairie Creek	216.7	8,800	16,500	22,300	31,000	38,400	47,500	56,100	68,300	HEC-HMS Uniform Rainfall
Neches River above Kickapoo Creek	281.0	7,800	15,700	22,500	33,000	42,000	54,200	65,500	81,400	HEC-HMS Uniform Rainfall
Kickapoo Creek above Neches River	289.6	5,600	10,800	15,000	21,400	26,900	33,900	40,500	50,200	HEC-HMS Uniform Rainfall
Neches River below Kickapoo Creek	570.6	12,100	22,300	30,200	42,000	52,300	65,900	78,800	97,400	HEC-HMS Uniform Rainfall
Lake Athens Inflow	21.6	2,200	3,700	4,900	6,500	7,900	9,500	11,100	13,500	HEC-HMS Uniform Rainfall
Flat Creek below Lake Athens	21.6	300	500	800	1,200	1,200	1,200	1,300	2,000	HEC-HMS Uniform Rainfall
Flat Creek above Lake Palestine	118.6	4,600	8,600	11,700	16,300	20,200	25,100	29,900	36,800	HEC-HMS Uniform Rainfall
Lake Palestine Inflow	838.1	13,500	28,100	40,600	58,500	75,300	96,100	117,400	147,800	HEC-HMS Elliptical Frequency Storm
Neches River below Lake Palestine	838.1	2,500	6,900	10,000	15,700	21,100	28,300	36,400	48,300	HEC-HMS Elliptical Frequency Storm
Neches River at US-175	882.5	2,500	7,000	10,100	15,900	21,400	28,600	36,700	48,700	HEC-HMS Elliptical Frequency Storm
Neches River above Caddo Creek	901.9	2,600	7,000	10,200	15,900	21,500	28,700	36,900	48,900	HEC-HMS Elliptical Frequency Storm
Caddo Creek	64.8	2,400	5,200	7,500	10,800	13,400	16,600	19,800	24,400	HEC-HMS Uniform Rainfall
Neches River below Caddo Creek	966.7	3,500	7,600	11,500	18,100	24,400	30,900	37,400	46,900	HEC-HMS Elliptical Frequency Storm
Neches River above Brushy Creek	1020.4	2,600	7,000	10,300	16,100	21,900	29,400	37,900	50,400	HEC-HMS Elliptical Frequency Storm
Brushy Creek above Neches River	84.0	3,300	6,600	9,200	13,300	16,500	20,400	24,300	30,000	HEC-HMS Uniform Rainfall
Neches River below Brushy Creek	1104.4	5,300	10,400	14,800	22,900	30,200	37,300	44,600	58,500	HEC-HMS Elliptical Frequency Storm
Neches River nr Neches, TX USGS Gage 08032000 at US-79 bridge	1145.8	4,700	10,100	15,100	25,000	34,500	43,800	53,300	67,400	HEC-HMS Elliptical Frequency Storm
Neches River above Hurricane Creek	1171.2	2,700	5,900	9,400	16,900	25,100	33,900	44,300	59,700	HEC-HMS Elliptical Frequency Storm
Hurricane Creek above Neches River	103.8	4,100	8,700	12,000	16,800	20,800	25,600	30,400	37,400	HEC-HMS Uniform Rainfall
Neches River below Hurricane Creek	1275.0	4,500	9,700	13,400	20,800	30,500	41,000	53,700	72,800	HEC-HMS Elliptical Frequency Storm
Neches River above Stills Creek	1289.5	3,200	7,100	11,200	20,200	29,700	40,200	52,600	71,300	HEC-HMS Elliptical Frequency Storm
Stills Creek above Neches River	56.0	3,300	6,700	9,400	13,000	15,900	19,400	22,900	27,900	HEC-HMS Uniform Rainfall
Neches River below Stills Creek	1345.5	3,300	7,300	11,500	20,900	30,600	41,400	54,300	73,800	HEC-HMS Elliptical Frequency Storm

Location Description	Drainage Area	50%	20%	10%	4%	2%	1%	0.50%	0.20%	Hydrologic Method
	sq mi	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	200-YR	500-YR	
Neches River above Tails Creek	1358.7	3,100	7,100	11,200	20,000	29,100	40,600	54,000	73,400	HEC-HMS Elliptical Frequency Storm
Lake Jacksonville Inflow	39.6	6,600	11,100	14,400	19,200	23,000	27,700	32,400	39,200	HEC-HMS Uniform Rainfall
Tails Creek below Lake Jacksonville	39.6	1,200	1,200	1,300	1,300	1,300	1,500	3,400	6,900	HEC-HMS Uniform Rainfall
Tails Creek above Neches River	107.0	3,000	6,000	8,300	11,700	14,500	17,900	21,400	26,500	HEC-HMS Uniform Rainfall
Neches River below Tails Creek	1465.8	3,800	8,800	13,100	22,300	31,700	44,000	58,600	81,500	HEC-HMS Elliptical Frequency Storm
Neches River above Ioni Creek	1497.3	3,400	8,000	12,600	21,600	31,000	42,600	56,500	78,900	HEC-HMS Elliptical Frequency Storm
Ioni Creek above Neches River	104.3	4,900	10,200	14,100	19,900	24,400	30,100	35,700	44,000	HEC-HMS Uniform Rainfall
Neches River below Ioni Creek	1601.6	5,000	11,300	15,800	23,200	31,900	44,000	58,700	81,900	HEC-HMS Elliptical Frequency Storm
Neches River above San Pedro Creek	1637.6	3,400	7,900	12,500	21,600	31,100	42,700	57,300	80,200	HEC-HMS Elliptical Frequency Storm
San Pedro Creek	134.9	5,200	11,000	15,400	21,800	27,000	33,500	39,900	49,100	HEC-HMS Uniform Rainfall
Neches River below San Pedro Creek	1772.6	8,400	16,300	21,800	31,200	39,400	48,200	57,400	70,800	HEC-HMS Elliptical Frequency Storm
Neches River at TX-21 Bridge, former USGS gage near Alto 08032500	1943.4	6,600	14,700	22,000	36,100	50,800	66,300	82,100	107,300	HEC-HMS Elliptical Frequency Storm
Neches River above Hickory Creek	2008.3	5,600	12,500	18,600	30,600	42,200	56,000	73,100	99,700	HEC-HMS Elliptical Frequency Storm
Hickory Creek above Neches River	90.5	4,400	9,200	13,300	17,500	21,000	25,900	31,100	37,600	HEC-HMS Uniform Rainfall
Neches River below Hickory Creek	2098.8	6,000	13,300	19,600	32,200	44,200	58,600	77,300	105,700	HEC-HMS Elliptical Frequency Storm
Neches River at TX-7 bridge near Pollok, TX	2236.5	8,000	15,000	19,200	32,800	43,600	59,600	77,600	105,400	HEC-HMS Elliptical Frequency Storm
Neches River at TX-94 bridge near Apple Springs, TX	2433.3	12,000	22,100	28,800	39,300	45,700	55,100	73,400	103,900	HEC-HMS Elliptical Frequency Storm
Neches River nr Diboll, USGS gage 08033000 at US-59 bridge	2726.2	10,200	20,500	27,900	40,100	45,400	57,600	76,700	107,000	HEC-HMS Elliptical Frequency Storm
Neches River above Piney Creek	2941.0	11,600	15,700	18,700	27,900	37,800	52,300	69,000	96,500	HEC-HMS Elliptical Frequency Storm
Piney Creek above Neches River	247.7	4,800	9,500	14,600	19,300	23,400	29,800	36,200	45,300	HEC-HMS Uniform Rainfall
Piney Creek at US-59 bridge near Corrigan, TX	247.7	4,800	9,500	14,600	19,300	23,400	29,800	36,200	45,300	HEC-HMS Uniform Rainfall
Piney Creek above Neches River	374.4	3,700	7,200	12,400	17,600	22,100	31,300	40,100	52,900	HEC-HMS Uniform Rainfall
Neches River below Piney Creek	3315.4	15,700	22,400	27,300	35,000	41,800	58,400	77,200	106,900	HEC-HMS Elliptical Frequency Storm

Location Description	Drainage Area	50%	20%	10%	4%	2%	1%	0.50%	0.20%	Hydrologic Method
	sq mi	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	200-YR	500-YR	
Neches River near Rockland, USGS gage 08033500 at US-69 bridge	3633.1	13,800	25,100	33,500	45,200	50,200	58,600	76,800	106,400	HEC-HMS Elliptical Frequency Storm
Neches River above the Angelina River	3791.1	10,400	14,700	19,900	31,400	40,300	56,300	73,500	102,300	HEC-HMS Elliptical Frequency Storm
Inflow to Lake Striker	182.0	3,800	6,900	9,600	13,800	17,400	22,100	26,700	33,500	HEC-HMS Uniform Rainfall
Striker Creek below Lake Striker	182.0	3,100	6,000	8,600	12,600	16,000	20,100	23,800	28,600	HEC-HMS Uniform Rainfall
Striker Creek above Angelina River	201.8	2,300	4,300	6,300	10,000	13,400	18,000	22,400	28,500	HEC-HMS Uniform Rainfall
Angelina River above Striker Creek	224.4	4,200	6,700	9,000	11,600	15,100	19,500	23,700	29,700	HEC-HMS Uniform Rainfall
Angelina River below Striker Creek	426.3	5,600	9,800	13,700	19,900	26,900	36,000	44,000	56,100	HEC-HMS Uniform Rainfall
Angelina River above Mud Creek	638.6	5,800	10,300	14,600	20,700	28,500	39,300	49,700	64,900	HEC-HMS Uniform Rainfall
Mud Creek above West Mud Creek	172.0	2,600	5,200	7,100	9,700	12,700	18,200	23,800	32,200	HEC-HMS Uniform Rainfall
West Mud Creek above Mud Creek	92.5	3,200	6,200	8,300	11,200	14,300	17,700	21,100	26,100	HEC-HMS Uniform Rainfall
Mud Creek below West Mud Creek	264.5	4,900	10,000	13,900	19,400	25,700	33,200	41,200	54,400	HEC-HMS Uniform Rainfall
Mud Creek near Jacksonville, USGS gage 08034500 at US-79 bridge	377.4	3,800	7,600	11,300	18,100	26,300	37,300	48,700	66,000	HEC-HMS Uniform Rainfall
Mud Creek at US-84 bridge, near Reklaw, TX	523.3	5,200	8,300	11,100	15,500	23,400	34,400	46,400	64,900	HEC-HMS Uniform Rainfall
Mud Creek above the Angelina River	556.3	2,700	5,500	9,100	15,200	22,900	33,900	45,700	64,200	HEC-HMS Uniform Rainfall
Angelina River below Mud Creek	1194.8	8,200	14,200	20,300	30,200	42,600	59,100	78,000	105,500	HEC-HMS Elliptical Frequency Storm
Angelina River near Alto, USGS gage 08036500 at TX-21 bridge	1286.4	6,500	12,500	17,900	26,300	37,400	52,800	70,400	96,900	HEC-HMS Elliptical Frequency Storm
Angelina River above Bayou Loco	1415.8	6,200	12,000	17,100	25,100	35,700	50,300	67,100	94,600	HEC-HMS Elliptical Frequency Storm
Inflow to Lake Nacogdoches	89.0	5,900	12,000	15,000	18,700	22,300	27,600	33,200	40,300	HEC-HMS Uniform Rainfall
Bayou Loco below Lake Nacogdoches	89.0	1,000	2,400	2,800	2,900	3,400	6,200	9,400	13,600	HEC-HMS Uniform Rainfall
Bayou Loco above Angelina River	102.5	1,000	2,300	2,800	3,300	3,900	6,100	9,500	14,000	HEC-HMS Uniform Rainfall
Angelina River above Bayou Loco	1518.3	6,500	12,700	18,000	26,400	37,400	53,000	70,000	97,900	HEC-HMS Elliptical Frequency Storm
Angelina River at Hwy 59 near Lufkin USGS gage, above Bayou La Nana	1621.5	6,500	12,600	17,900	26,000	36,800	52,000	68,600	96,300	HEC-HMS Elliptical Frequency Storm

Location Description	Drainage Area	50%	20%	10%	4%	2%	1%	0.50%	0.20%	Hydrologic Method
	sq mi	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	200-YR	500-YR	
Bayou La Nana above the Angelina River	83.3	4,000	7,000	9,200	12,700	15,700	19,500	23,200	28,400	HEC-HMS Uniform Rainfall
Angelina River below Bayou La Nana	1704.9	6,500	12,600	18,000	26,100	36,900	52,300	69,000	97,100	HEC-HMS Elliptical Frequency Storm
Angelina River above Bayou Carrizo	1842.3	6,300	12,100	17,300	25,100	35,000	49,700	66,000	92,400	HEC-HMS Elliptical Frequency Storm
Bayou Carrizo above Angelina River	110.2	5,800	9,800	12,900	17,800	22,100	27,600	32,800	40,200	HEC-HMS Uniform Rainfall
Angelina River below Bayou Carrizo	1952.4	12,600	22,200	28,900	38,900	47,500	58,700	69,300	84,600	HEC-HMS Elliptical Frequency Storm
Attoyac Bayou below West Creek	314.2	12,000	20,800	26,600	35,200	40,100	51,000	61,200	75,600	HEC-HMS Uniform Rainfall
Attoyac Bayou above Big Iron Ore Creek	388.1	6,000	13,200	18,400	26,900	31,300	44,900	56,600	73,500	HEC-HMS Uniform Rainfall
Big Iron Ore Creek above Attoyac Bayou	97.2	6,000	9,800	12,400	16,200	18,400	23,000	27,400	33,600	HEC-HMS Uniform Rainfall
Attoyac Bayou below Big Iron Ore Creek	485.2	7,400	14,900	20,900	30,900	36,200	52,300	66,400	87,100	HEC-HMS Uniform Rainfall
Attoyac Bayou nr Chireno, USGS gage 08038000 at TX-21 bridge	503.1	6,500	14,300	20,000	29,400	34,500	49,700	63,300	84,800	HEC-HMS Uniform Rainfall
Attoyac Bayou above Angelina River	670.7	8,100	13,800	19,600	29,500	35,300	51,900	67,400	90,400	HEC-HMS Uniform Rainfall
Angelina River below Attoyac Bayou	2808.0	26,600	47,600	62,500	84,700	104,300	129,900	154,000	188,900	HEC-HMS Elliptical Frequency Storm
Ayish Bayou near San Augustine, USGS gage 08039100 at TX-103 bridge	88.6	4,100	8,500	12,100	16,900	20,500	25,500	30,200	37,100	HEC-HMS Uniform Rainfall
Ayish Bayou above Sam Rayburn Lake	202.1	9,000	14,300	19,000	26,100	32,200	39,900	47,300	58,100	HEC-HMS Uniform Rainfall
Total Inflow to Sam Rayburn Lake	3451.8	51,100	88,500	116,400	159,600	197,700	248,200	297,200	365,300	HEC-HMS Elliptical Frequency Storm
Angelina River below Sam Rayburn	3451.8	10,000	15,000	17,000	19,000	19,500	20,000	20,000	26,275	RMC-RFA Reservoir Analysis
Angelina River above the Neches River	3566.9	10,000	15,000	17,000	19,000	21,700	26,400	30,800	37,300	HEC-HMS Elliptical Frequency Storm (2% to 0.2% AEP) & RMC-RFA Reservoir Analysis (50% to 4% AEP)
Total Inflow to Town Bluff Dam	7569.3	25,400	40,300	50,200	61,400	81,300	100,400	117,800	143,000	HEC-HMS Elliptical Frequency Storm
Neches River below Town Bluff Dam, USGS gage 08040600	7569.3	20,000	30,600	38,750	54,000	69,200	84,420	95,280	112,000	RMC-RFA Reservoir Analysis
Neches River below Big Creek	7673.6	22,300	34,800	43,600	53,800	67,800	82,800	96,000	117,700	HEC-HMS Elliptical Frequency Storm
Neches River below Mill Creek at FM 1013 bridge	7716.9	22,100	32,400	40,000	50,000	64,900	81,200	95,800	117,800	HEC-HMS Elliptical Frequency Storm

Location Description	Drainage Area	50%	20%	10%	4%	2%	1%	0.50%	0.20%	Hydrologic Method
	sq mi	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	200-YR	500-YR	
Neches River below Black Branch	7784.9	21,400	30,800	38,200	47,900	61,500	79,300	95,200	118,600	HEC-HMS Elliptical Frequency Storm
Neches River at Evadale	7894.7	21,000	29,700	36,600	46,000	57,700	74,700	90,900	115,000	HEC-HMS Elliptical Frequency Storm
Neches River below Evadale	7950.3	21,100	30,100	37,100	46,800	58,700	76,200	92,900	117,800	HEC-HMS Elliptical Frequency Storm
Neches River above Village Creek	8001.7	20,700	29,600	36,300	45,600	56,800	73,600	90,200	115,200	HEC-HMS Elliptical Frequency Storm
Village Creek at US-69 bridge	265.0	4,700	10,900	17,000	25,300	30,600	38,600	47,800	61,100	HEC-HMS Uniform Rainfall
Village Creek above Turkey Creek	423.1	5,600	13,500	21,100	35,600	43,100	56,400	71,700	93,000	HEC-HMS Uniform Rainfall
Big Cypress Creek at US-69	85.1	3,700	8,000	11,500	16,100	19,000	23,000	27,500	34,000	HEC-HMS Uniform Rainfall
Turkey Creek at FM 1943	141.4	3,600	9,300	15,700	23,900	28,800	35,500	43,000	53,600	HEC-HMS Uniform Rainfall
Turkey Creek above Village Creek	166.3	2,800	6,900	11,700	19,400	24,300	31,700	40,400	53,000	HEC-HMS Uniform Rainfall
Village Creek below Turkey Creek	589.4	8,100	19,000	29,300	47,100	56,800	73,300	93,500	121,600	HEC-HMS Uniform Rainfall
Village Creek above Beech Creek	601.7	7,300	16,600	26,200	43,800	56,100	73,000	93,000	121,000	HEC-HMS Uniform Rainfall
Beech Creek above Village Creek	213.0	5,200	10,800	16,300	23,300	27,600	33,800	41,100	51,700	HEC-HMS Uniform Rainfall
Village Creek below Beech Creek	814.7	9,600	21,300	33,900	57,600	74,900	98,400	125,800	163,900	HEC-HMS Uniform Rainfall
Village Creek near Kountze, USGS gage 08041500 at FM 418 bridge	861.1	9,400	21,100	33,000	54,100	71,000	98,100	126,600	165,600	HEC-HMS Uniform Rainfall
Village Creek above Cypress Creek	864.4	8,400	20,300	32,900	53,200	69,000	90,300	123,200	163,400	HEC-HMS Uniform Rainfall
Cypress Creek above Village Creek	199.7	1,500	4,900	7,000	10,700	14,300	19,400	24,500	32,400	HEC-HMS Uniform Rainfall
Village Creek below Cypress Creek	1064.0	9,300	23,400	37,400	61,000	80,500	107,500	146,500	194,800	HEC-HMS Uniform Rainfall
Village Creek at US-96 bridge near Lumberton, TX	1104.4	7,900	20,700	33,200	56,200	71,800	95,300	121,600	172,700	HEC-HMS Uniform Rainfall
Village Creek above Neches River	1113.9	7,800	20,500	32,900	55,900	70,700	93,800	118,500	172,300	HEC-HMS Uniform Rainfall
Neches River below Village Creek	9115.6	25,100	42,900	59,100	78,400	97,400	128,200	163,900	213,200	HEC-HMS Elliptical Frequency Storm
Neches River above Pine Island Bayou	9132.5	24,800	41,800	56,800	73,600	90,300	118,400	153,800	206,400	HEC-HMS Elliptical Frequency Storm
Willow Creek above Pine Island Bayou	206.1	4,400	8,800	12,600	18,400	23,300	30,400	38,400	50,600	HEC-HMS Uniform Rainfall
Pine Island Bayou above Willow Creek	171.8	2,300	4,400	6,500	9,600	12,300	16,300	20,700	27,500	HEC-HMS Uniform Rainfall
Pine Island Bayou below Willow Creek	377.9	5,400	10,800	15,900	23,700	30,700	41,400	53,000	70,600	HEC-HMS Uniform Rainfall

Location Description	Drainage Area	50%	20%	10%	4%	2%	1%	0.50%	0.20%	Hydrologic Method
	sq mi	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	200-YR	500-YR	
Pine Island Bayou near Sour Lake, USGS gage 08041700 at Old Beaumont Rd bridge	397.7	4,400	9,400	14,700	23,100	30,500	41,800	53,600	72,200	HEC-HMS Uniform Rainfall
Pine Island Bayou above Little Pine Island Bayou	417.3	3,900	8,200	13,400	21,900	29,400	41,200	53,200	71,900	HEC-HMS Uniform Rainfall
Little Pine Island Bayou above Pine Island Bayou	134.8	2,100	3,700	5,000	6,800	8,400	11,100	14,000	18,600	HEC-HMS Uniform Rainfall
Pine Island Bayou below Little Pine Island Bayou	552.2	5,400	11,300	17,800	28,300	37,600	52,100	67,000	90,200	HEC-HMS Uniform Rainfall
Pine Island Bayou above BI Pump Plant	697.7	6,200	13,000	20,300	32,600	43,800	61,200	79,200	107,600	HEC-HMS Uniform Rainfall
Pine Island Bayou above the Neches River	726.2	6,300	13,200	20,700	33,400	45,000	62,900	81,500	110,900	HEC-HMS Uniform Rainfall
Neches River below Pine Island Bayou	9858.6	29,200	50,600	68,900	91,600	112,800	150,400	197,400	269,700	HEC-HMS Elliptical Frequency Storm
Neches River at the Saltwater Barrier	9858.7	29,200	50,500	68,900	91,500	112,900	150,400	197,600	270,000	HEC-HMS Elliptical Frequency Storm

Table 12.2: Recommended Frequency Peak Pool Elevations (feet NAVD88) for Reservoirs in the Neches River Basin

Reservoir Name	Drainage Area	50%	20%	10%	4%	2%	1%	0.50%	0.20%	Hydrologic Method
	sq mi	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	200-YR	500-YR	
Lake Athens	21.6	441.1	441.7	442.2	442.9	443.6	444.6	445.7	446.9	HEC-HMS Uniform Rain Frequency Storms
Lake Palestine	838.1	346.2	347.4	348.1	349.1	349.9	350.9	351.9	353.2	HEC-HMS Elliptical Frequency Storms
Lake Jacksonville	39.6	423.4	424.8	426.0	427.8	429.5	431.4	432.8	434.4	HEC-HMS Uniform Rain Frequency Storms
Lake Striker	182.0	293.3	293.6	293.8	294.2	294.5	295.1	295.8	297.1	HEC-HMS Uniform Rain Frequency Storms
Lake Tyler	113.3	376.5	377.2	377.7	378.6	379.3	380.1	381.0	382.1	HEC-HMS Uniform Rain Frequency Storms
Lake Nacogdoches	89.0	280.5	282.6	283.8	285.2	286.7	288.4	289.9	291.3	HEC-HMS Uniform Rain Frequency Storms
Sam Rayburn Reservoir	3451.8	166.9	170.3	172.3	174.3	174.5	176.4	177.1	177.6	RMC-RFA Reservoir Analysis
B.A. Steinhagen Lake	7569.3	83.5	83.9	84.1	84.5	85.0	85.4	86.2	86.9	RMC-RFA Reservoir Analysis

13 Conclusions

This report summarizes new analyses that were completed as part of an InFRM Watershed Hydrology Assessment (WHA) to estimate the 1% annual chance (100-yr) flow, along with other frequency flows, for various stream reaches throughout the Neches River Basin in Texas. In addition to the partnered federal agencies of the InFRM team, regional stakeholders such as the Lower Neches Valley Authority (LNVA), the Angelina & Neches River Authority (ANRA), and the Texas Water Development Board (TWDB) also participated in the updates and review process for this study. This study represents a significant step forward towards increasing resiliency against flood hazards in the Neches River basin.

The flow results that were recommended for adoption came from a combination of the watershed model results using NOAA Atlas 14 uniform rain, elliptical storms, and reservoir analysis techniques. Other methods, such as the statistical and RiverWare results, were used as points of comparison to fine tune the model for the frequent storms, but they were not adopted directly due to their tendency to change after each significant flood event. Since the calibrated watershed model simulates the physical processes that occur during a storm event, it can produce more reliable and consistent estimations of the flow expected during a 1% annual chance (100-yr) storm. In addition, NOAA Atlas 14's recent study of rainfall depths in Texas shed new light on the depths and frequency of rainfall that could be expected in the Neches River basin. Both uniform rain and elliptical shaped frequency storms were run in the watershed model. The elliptical frequency storm results were generally recommended for river reaches with large drainage areas, while the uniform rain results were recommended for the smaller drainage areas. The expected impacts of reservoir operations for Sam Rayburn Reservoir and B.A. Steinhagen Lake were also analyzed in detail for this study, and the frequency dam releases and pool elevations that resulted from the reservoir analyses were recommended for the reaches immediately upstream and downstream of the dams.

Previously published frequency discharges from effective FEMA Flood Insurance Studies (FIS) and Base Level Engineering (BLE) data in the Neches River Basin differ from the new flow frequency results of this study in many locations. The new flow frequency results are higher than the previously published results in some areas, while they are lower in other areas. Figures 13.1 and 13.2 compare the recommended 1% annual chance (100-yr) results from this Watershed Hydrology Assessment with previously published flows at some key locations within the basin. Similarly, Figures 13.3. and 13.4 compare the recommended 100-yr pool elevation results from this study with the previously published water surface elevations at Sam Rayburn Reservoir and B.A. Steinhagen Lake.

For most areas of the upper Neches and Angelina River watersheds, the recommended results of this study are either similar to or slightly higher than the previously published 100-yr flows from the Base Level Engineering (BLE) data, as shown in Figure 13.1. Similarly, the statistical analyses of the gage records are generally consistent with the recommended results from this study but may be slightly higher or lower than the recommended results at a given location, depending on whether or not large storms have hit that particular watershed during the observed gage record. The upper Neches and Angelina River watersheds generally have not been studied in detail; therefore, there are no effective FEMA FIS flows available for comparison in these areas.

For the lower Neches River and its tributaries, there is more variation between the recommended results of this study and the previously published 100-yr flows from the effective FEMA FIS and the 2020 BLE data, as shown in Figure 13.2. The changes in these flow frequency estimates can primarily be attributed to a combination of factors including (1) additional gage record length, (2) improved calibration of the rainfall runoff model, and (3) increased rainfall depths near the Gulf from NOAA Atlas 14. First, the new flow frequency results from this study differ from the effective flood insurance values because there have been new floods in the gage record, which caused some of the current statistical estimates to be very different than they were when the previous FEMA FIS flow frequency estimates were developed. While the effective FEMA FIS maps in the lower Neches basin were updated between 2002 and 2011, the hydrology behind those flood insurance maps has not been updated since the 1980s or early 1990s. In addition, the current study found that the extreme magnitude of Hurricane Harvey caused many of the current statistical 100-yr estimates to be overestimated in the lower portions of the Neches watershed. Second, the rainfall-runoff watershed model underwent extensive calibration to accurately simulate the response of the watershed to a range of recent observed flood events, including large events similar to a 1% annual chance (100-yr) flood and even more extreme events like Hurricane Harvey. The frequency flow results of the calibrated rainfall-runoff watershed model exposed that some of the FIS flows calculated in the past using statistical hydrology or uncalibrated rainfall-runoff modeling did not accurately reflect the response of the watershed to a 1% annual chance (100-yr) storm event. Finally, NOAA Atlas 14 revealed that previous estimates of the 100-yr rainfall near Gulf coast had been underestimated by up to 3 inches for the 24-hour duration and up to 6 inches for the 4-day duration. This additional rainfall led to higher peak flows on portions of Pine Island Bayou and the lower Neches River, as shown in Figure 13.2.

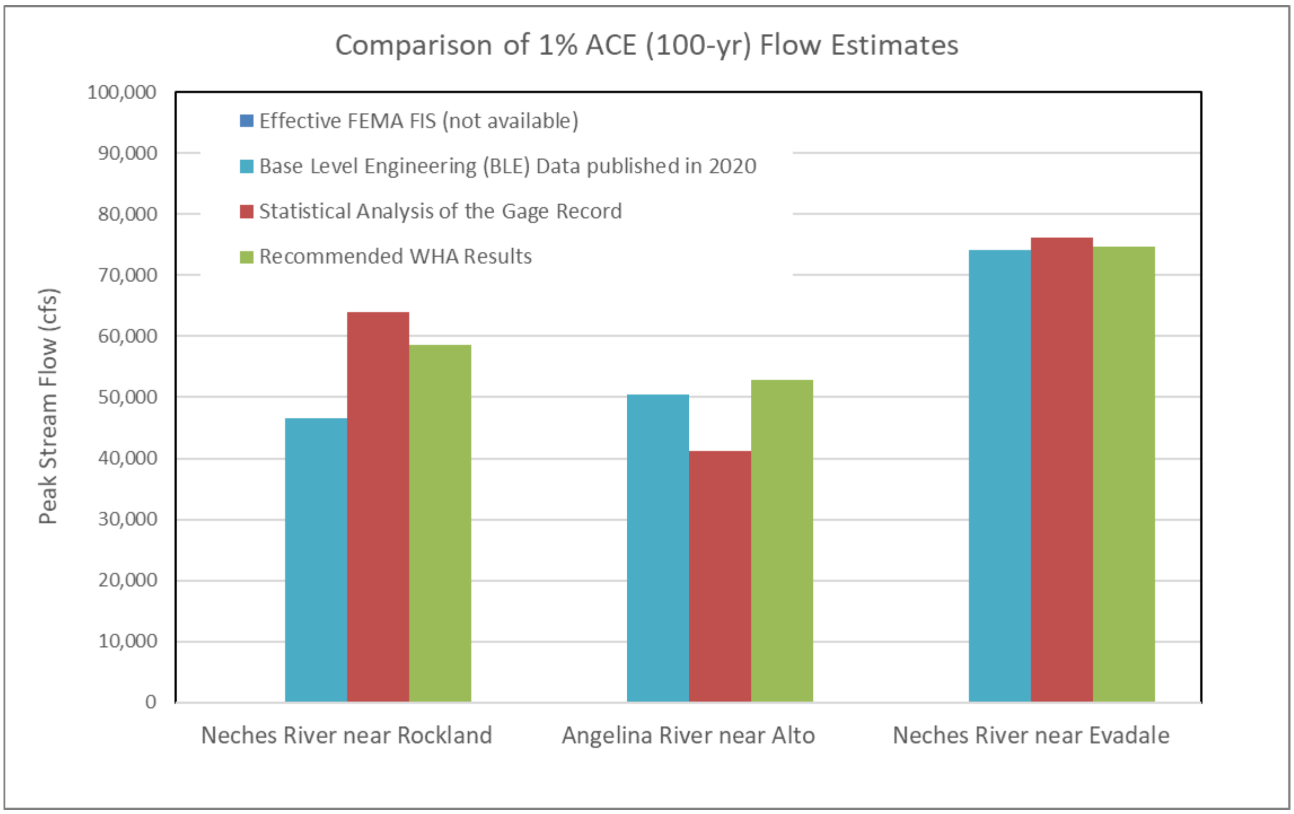


Figure 13.1: Comparison of 1% Annual Chance (100-yr) Flow Results on the Upper Neches and Angelina Rivers

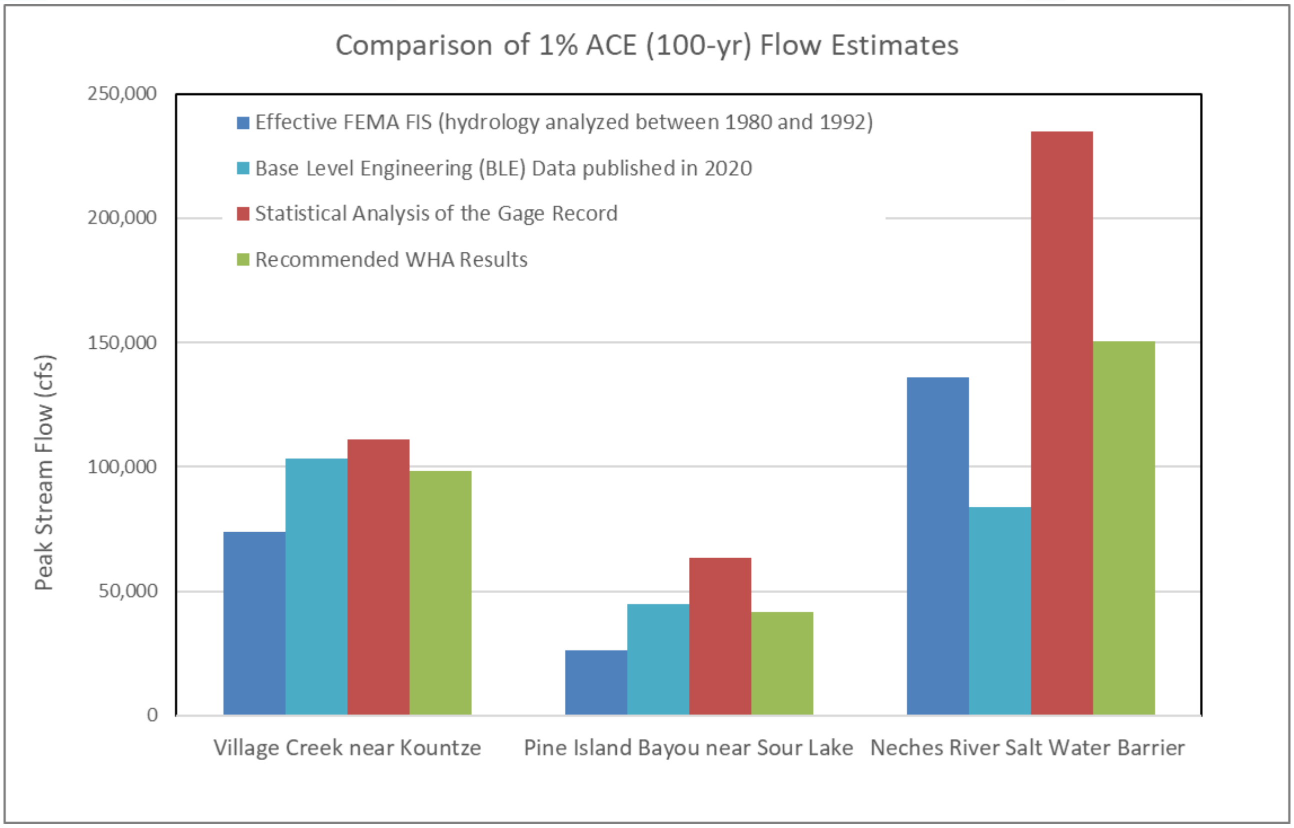


Figure 13.2: Comparison of 1% Annual Chance (100-yr) Flow Results on the Lower Neches Watershed

For the two USACE reservoirs in the basin, Sam Rayburn and B.A. Steinhagen, Figures 13.3. and 13.4 show that the recommended 100-yr pool elevations from this study are generally just above the emergency spillway crest and the flood of record for each respective reservoir. These figures also show that the recommended results of this study are much higher than the previously published 100-yr elevations from the Base Level Engineering (BLE) data, which are actually below the reservoirs' normal pool elevations. This is because the BLE data is based on approximate methods that do not account for the effects of the reservoirs. The recommended results of this study, on the other hand, came from detailed reservoir analyses that utilized stochastic techniques to account for the operations of the dam, the frequency and volume of its inflows, and the possible range of its starting pool elevations. Once again, the areas near these reservoirs have not been previously studied in detail; therefore, there are no effective FEMA FIS pool elevations available for comparison.

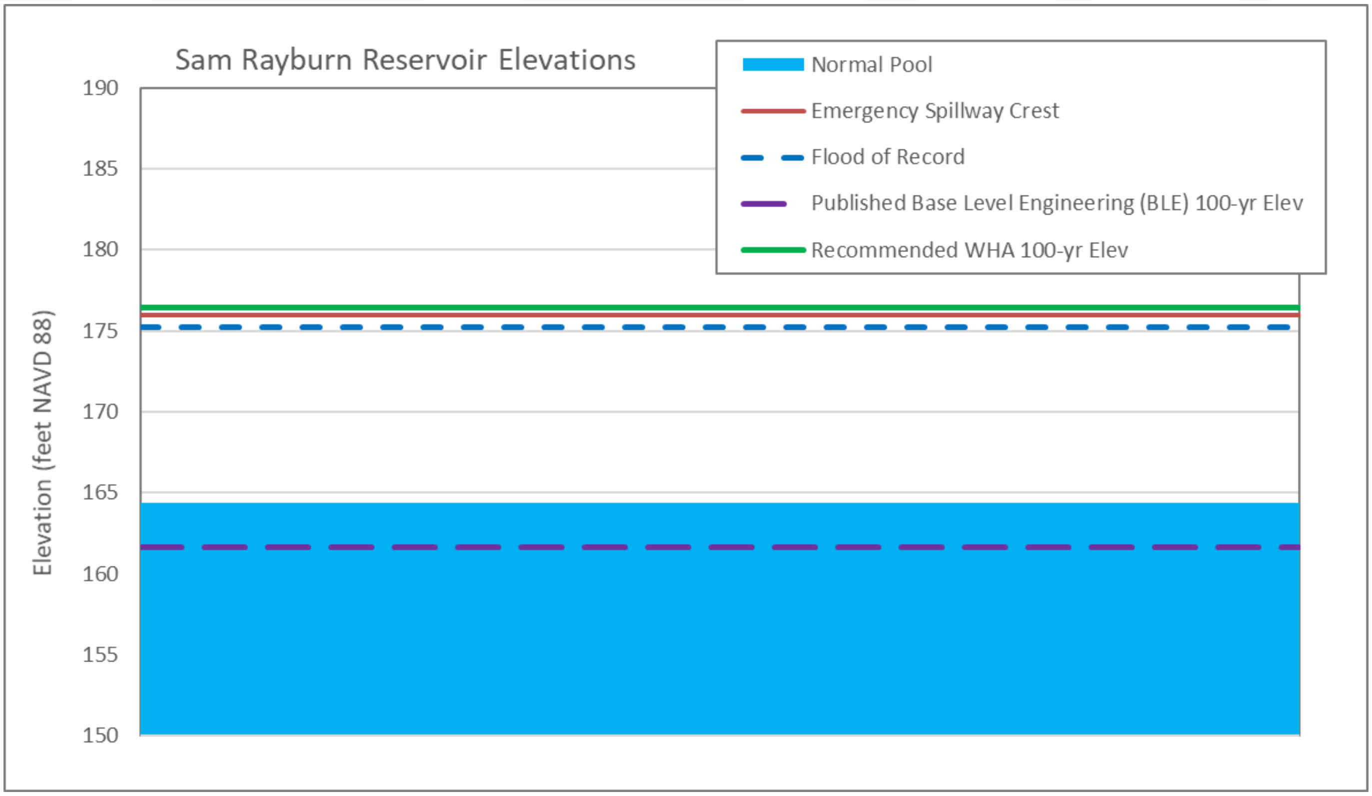


Figure 13.3: Comparison of Pool Elevations for Sam Rayburn Reservoir

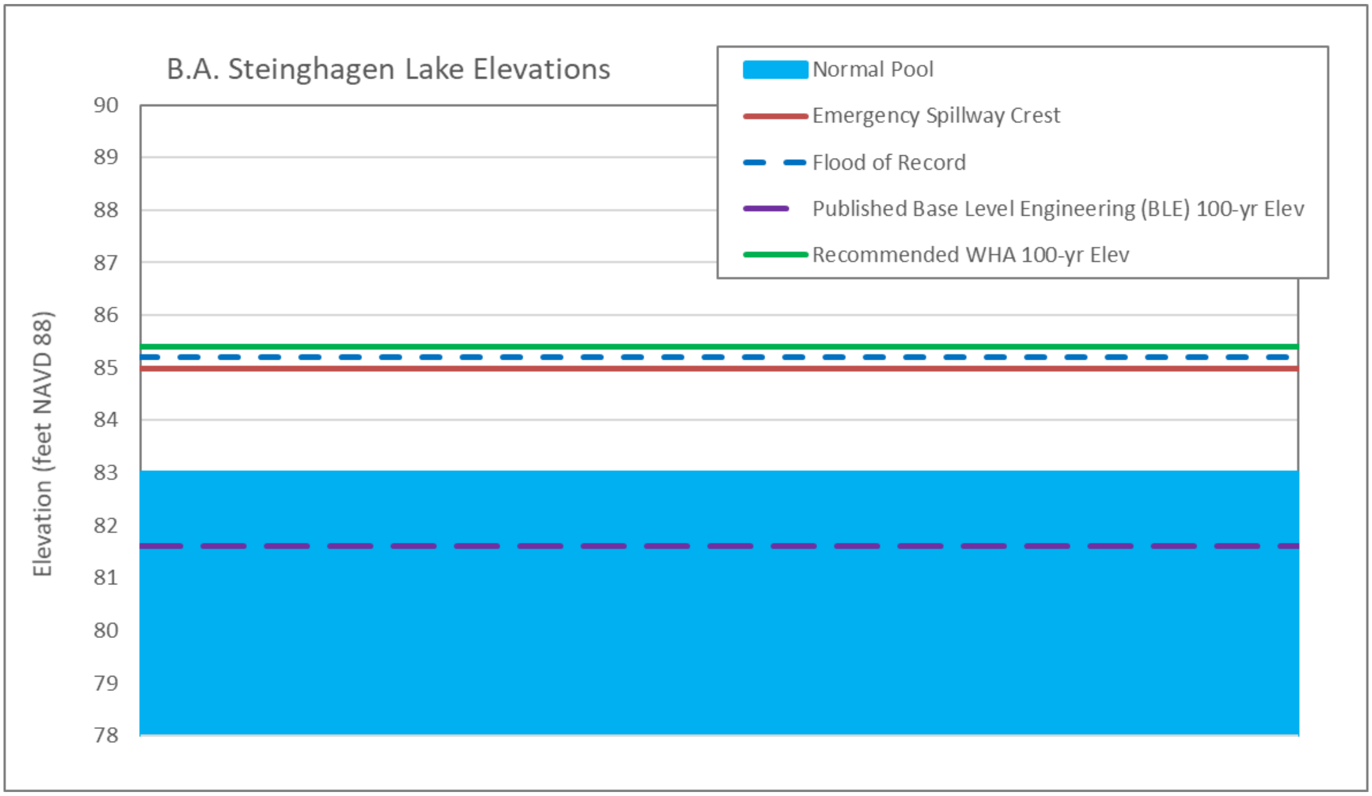


Figure 13.4: Comparison of Pool Elevations for B.A. Steinhagen Lake

Given the severe loss of life and property that has occurred during recent floods within the State of Texas, it is imperative that future updates to the published flood insurance rate maps for the Neches River Basin accurately reflect the known levels of flood risk in the basin. The recommended results from this study represent the best available estimate of flood risk for the larger streams in the Neches River basin, based on a range of hydrologic methods performed by an expert team of engineers and scientists from multiple federal agencies. For smaller tributaries in the Neches basin, the recommended results from the watershed model provide a good starting point which could be further refined by adding additional subbasins and using methodologies that are consistent with this study.

As a result of the level of investment, analyses, and collaboration that went into this Watershed Hydrology Assessment, the flood risk estimates contained in this report are recommended as the basis for future NFIP studies or other federal flood risk studies within the Neches River basin. These federally developed modeling results form a consistent understanding of hydrology across the Neches watershed, which is a key requirement outlined in FEMA's General Hydrologic Considerations Guidance. Furthermore, the models and data used to produce these flood risk estimates are available upon request, at no charge, to communities, local stakeholders, and architecture engineering firms. Requests for the models should be sent to the InFRM team through the InFRM website at www.InFRM.us.

While the results from this study should be considered the best available estimates of flood risk for many areas of the Neches River basin, significant uncertainty still remains, as it does in any hydrologic study. Because of this uncertainty and because of the potential impacts these estimates can have on life and property, the InFRM team strongly recommends and supports local communities that implement higher standards, such as additional freeboard requirements, floodplain management practices based on standards greater than the 1% annual chance flood, and/or "no valley storage loss" criteria.

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15 Terms of Reference

Acronym	Definition
2D	two-dimensional
3DEP	three-dimensional Elevation Program
AEP	annual exceedance probability
BFE	base flood elevations
cfs	cubic feet per second
CWMS	Corps Water Management System
DDF	Depth Duration Frequency
DEM	digital elevation model
DSS	data storage system
EM	Engineering Manual
ER	Engineering Regulation
EMA	expected moment algorithm
ERDC	Engineering Research & Development Center of USACE
FEMA	Federal Emergency Management Agency
FIS	flood insurance study
GeoHMS	Geospatial Hydrologic Model System extension
GIS	Geographic Information Systems
HEC	Hydrologic Engineering Center
HMS	Hydrologic Modeling System
IACWD	Interagency Advisory Committee on Water Data
InFRM	Interagency Flood Risk Management
LiDAR	Light (Laser) Detection and Range
LOC	Line of organic correlation
LPIII	Log Pearson III
MMC	Modeling, Mapping, and Consequences Production Center
NA14	NOAA Atlas 14
NAD 83	North American Datum of 1983
NCDC	National Climatic Data Center
NED	National Elevation Dataset
NGVD 29	National Geodetic Vertical Datum of 1929
NHD	National Hydrography Dataset
NID	National Inventory of Dams
NLCD	National Land Cover Database
NOAA	National Oceanic and Atmospheric Administration
NRCS	Natural Resources Conservation Service
NSE	Nash Sutcliffe Efficiency
NWIS	National Water Information System
NWS	National Weather Service
PDSI	Palmer Drought Severity Index
PeakFQ	Peak Flood Frequency
PFDS	Precipitation Frequency Data Server
PMP	Probable Maximum Precipitation
QPF	Quantitative Precipitation Forecast
RAS	River Analysis System
ResSim	Reservoir System Simulation
RFA	Reservoir Frequency Analysis
RFC	River Forecast Center
RMC	Risk Management Center
RMSE	root mean square error
RSR	observed standard deviation ratio

Acronym	Definition
SCS	Soil Conservation Service
SHG	Standard Hydrologic Grid
SME	subject matter expert
SOP	Standard Operating Procedures
sq mi	square miles
SSURGO	Soil Survey Geographic Database
TxDOT	Texas Department of Transportation
USACE	U.S. Army Corps of Engineers
USDA	U.S. Department of Agriculture
USGS	U.S. Geological Survey
WCM	Water Control Manual
WGRFC	West Gulf River Forecast Center