Analysis of major hydrologic events in Ascension Parish, LA

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ANALYSIS OF MAJOR HYDROLOGIC EVENTS IN ASCENSION PARISH, LA

A Thesis

Submitted to the Graduate Faculty of the Louisiana State University and Agricultural and Mechanical College in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering

in

The Department of Civil & Environmental Engineering

by

Jessica Irene Mason
B.S., University of Mississippi, 2007
May 2011
To God, my family and friends
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# Table of Contents

Dedication.................................................................................................................. ii

Acknowledgments...................................................................................................... iii

List of Tables .............................................................................................................. vi

List of Figures ............................................................................................................. vii

Abstract ....................................................................................................................... x

Chapter 1 - Introduction .............................................................................................. 1
  1.1 Frequency Analysis ............................................................................................ 1
  1.2 Hydrologic Modeling ......................................................................................... 1
  1.3 Hydraulic Modeling .......................................................................................... 2
  1.4 Study Objectives .............................................................................................. 2
  1.5 Overview of Thesis .......................................................................................... 3

Chapter 2 - Background ............................................................................................. 4
  2.1 Literature Review ............................................................................................ 4
     2.1.1 Frequency Analysis .................................................................................. 4
     2.1.1.1 The TP-40 Method ............................................................................ 4
     2.1.1.2 The SRCC Method .......................................................................... 7
     2.1.2 Hydrologic Models .................................................................................. 9
        2.1.2.1 Transform Methods ....................................................................... 9
           2.1.2.1.1 The Soil Conservation Service (SCS) Method ......................... 9
           2.1.2.1.2 The Snyder Method ................................................................ 11
           2.1.2.1.3 The Clark Unit Hydrograph Method ..................................... 12
        2.1.2.2 Hydrologic Flow Routing ............................................................. 13
        2.1.2.3 Muskingum Flow Routing ............................................................. 14
     2.1.3 Hydraulic Models ...................................................................................... 15
        2.1.3.1 Hydraulic Routing ........................................................................ 15
        2.1.3.2 HEC-RAS Unsteady Modeling ....................................................... 15
     2.2 Geography of Study Site .............................................................................. 16

Chapter 3 - Methods .................................................................................................. 20
  3.1 Frequency Analysis ......................................................................................... 20
  3.2 Hydrologic Modeling ....................................................................................... 24
     3.2.1 Previous Study ....................................................................................... 24
     3.2.2 Current Study ......................................................................................... 28
        3.2.2.1 Transform Methods ....................................................................... 29
        3.2.2.2 Baseflow ....................................................................................... 30
        3.2.2.3 HEC-HMS Control Specifications ................................................. 30
  3.3 Hydraulic Modeling ......................................................................................... 31
3.3.1 Hydraulic Modeling of Grand Goudine Bayou ......................................................... 31
3.3.2 Calibration of the HEC-RAS model ......................................................................... 38

Chapter 4 - Results .......................................................................................................... 41
  4.1 Hydrologic Analysis ........................................................................................................ 41
  4.2 Calibration ..................................................................................................................... 51
  4.3 Frequency Analysis ....................................................................................................... 60

Chapter 5 - Summary and Recommendations .................................................................... 64
  5.1 Summary ........................................................................................................................ 64
  5.2 Future Recommendations .............................................................................................. 65

References .......................................................................................................................... 67

Appendix A– 24-hour Frequency/Magnitude Atlas Maps for the SRCC Method ................. 69
Appendix B - 24-hour Frequency/Magnitude Atlas Maps for the TP-40 Method ................ 72
Appendix C - SCS Method Rainfall Distributions ................................................................. 75
Appendix D - CUH Method Ratio Calculation .................................................................... 76
Appendix E - Manning’s n Values for Natural Streams ......................................................... 77
Vita ....................................................................................................................................... 79
List of Tables

Table 2-1—Worst Floods in Ascension Parish, Louisiana, Prior to 1980................................. 19
Table 3-1—Comparison of Rainfall Depths for the SRCC Method and TP-40 Method............. 22
Table 3-2—Grand Goudine Watershed SCS Curve Numbers and Lag Times...................... 28
Table 3-3—Subcritical Flow Contraction and Expansion Coefficients ................................. 37
Table 4-1—CUH Calculated Storage Coefficients ............................................................. 42
Table 4-2—Comparison of Flows for Transform Methods .................................................... 42
Table 4-3—Percent Difference of Flows as Compared to the SCS Method ............................ 42
Table 4-4—Mean Square Error for Transform Methods ....................................................... 51
Table 4-5—Mean Square Error for Baseflow Simulations ................................................... 55
Table 4-6—Rainfall Depth Comparison of SRCC vs. TP-40 Method for 5-, 10-, and 100-yr Storms ......................................................................................................................... 60
Table 4-7—Flow Comparison of SRCC vs. TP-40 Method for 5-, 10-, and 100-yr Storms....... 61
Table 4-8—Maximum Water Surface Elevation Comparison of SRCC vs. TP-40 Method for 5-, 10-, and 100-yr Storms ........................................................................................................... 61
Table E-1—Manning’s n Values for Natural Streams ............................................................... 77
List of Figures

Figure 2.1—Method Within TP-40 for Determining Rainfall Depth for Durations Between 1-hour and 24-hours .............................................................................................................................................6

Figure 2.2—Differences Between the Huff-Angel log-log Method and the SRCC log-linear Method ........................................................................................................................................8

Figure 2.3—Estimation of Direct Runoff from Curve Number ........................................................................................................................................10

Figure 2.4—Decreasing Amite River Elevation .............................................................................................................................................17

Figure 2.5—Tidal Influence in Ascension ..................................................................................................................................................18

Figure 3.1—24-hour, 10 year TP-40 Frequency/Magnitude Atlas for Southern Louisiana ...........21

Figure 3.2—24-hour, 10 year SRCC Frequency/Magnitude Atlas for Southern Louisiana........21

Figure 3.3—Estimated Rainfall Frequency/Magnitude Values for the SRCC Method at the gage in Gonzales, Ascension Parish, LA for years: 1978-2009 ......................................................................................23

Figure 3.4—Delineated Watersheds in Ascension Parish, LA .......................................................................................................................................25

Figure 3.5—Basin Model of Grand Goudine for HEC-HMS .......................................................................................................................................27

Figure 3.6—HEC-RAS Geometric Profile for Grand Goudine Bayou ........................................................................................................................................33

Figure 3.7—HEC-RAS Unsteady Flow Editor for Grand Goudine Bayou ........................................................................................................................................34

Figure 3.8—Initial Manning's n Values for Grand Goudine Bayou ........................................................................................................................................36

Figure 3.9—Aerial of Grand Goudine Bayou with USGS Gage ........................................................................................................................................39

Figure 4.1—Comparison of Flow Hydrographs for Subbasin W1 ........................................................................................................................................44

Figure 4.2—Comparison of Flow Hydrographs for Subbasin W2 ........................................................................................................................................44

Figure 4.3—Comparison of Flow Hydrographs for Subbasin W3 ........................................................................................................................................45

Figure 4.4—Comparison of Flow Hydrographs for Subbasin W4 ........................................................................................................................................45

Figure 4.5—Simulated vs Observed Stage for Grand Goudine for a Snyder Transform Method with Cp of 0.5 ...........................................................................................................................................47
Figure 4.6—Simulated vs Observed Stage for Grand Goudine for a Snyder Transform Method with Cp of 0.6 .................................................................47

Figure 4.7—Simulated vs Observed Stage for Grand Goudine for a Snyder Transform Method with Cp of 0.7 .................................................................48

Figure 4.8—Simulated vs. Observed Stage for Grand Goudine for a SCS Transform Method .................................................................48

Figure 4.9—Simulated vs. Observed Stage for Grand Goudine for a CUH Transform Method with ratio of 0.5 .................................................................49

Figure 4.10—Simulated vs. Observed Stage for Grand Goudine for a CUH Transform Method with ratio of 0.6 .................................................................49

Figure 4.11—Simulated vs. Observed Stage for Grand Goudine for a CUH Transform Method with ratio of 0.65 .................................................................50

Figure 4.12—Simulated vs. Observed Stage for Grand Goudine for a CUH Transform Method with ratio of 0.7 .................................................................50

Figure 4.13—Simulated vs. Observed Stage for Grand Goudine for a Baseflow of 5 cfs ...............52

Figure 4.14—Simulated vs. Observed Stage for Grand Goudine for a Baseflow of 10 cfs ...........53

Figure 4.15—Simulated vs. Observed Stage for Grand Goudine for a Baseflow of 12 cfs ...........53

Figure 4.16—Simulated vs. Observed Stage for Grand Goudine for a Baseflow of 15 cfs ...........54

Figure 4.17—Simulated vs. Observed Stage for Grand Goudine for a Baseflow of 18 cfs ...........54

Figure 4.18—Simulated vs. Observed Stage for Grand Goudine for a Baseflow of 20 cfs ...........55

Figure 4.19—Comparison of Manning’s n values for Grand Goudine ........................................56

Figure 4.20—Comparison of Altering Contraction and Expansion Coefficients for Bridge Sections on Grand Goudine .................................................................57

Figure 4.21—Model Comparison Event -- October 6, 2006 .........................................................58

Figure 4.22—Model Comparison Event -- September 2, 2008 .....................................................59

Figure 4.23—Comparison of SRCC vs. TP-40 Methods at the Outlet (XS 0) for the 5 yr Storm ...........................................................................................................62
Figure 4.24—Stage Comparison of SRCC vs. TP-40 Methods at the Outlet (XS 0) for the 10 yr Storm ..........................................................63

Figure 4.25—Stage Comparison of SRCC vs. TP-40 Methods at the Outlet (XS 0) for the 100 yr Storm ..................................................................................................................63

Figure A.1—24-Hour, 2-year Recurrence Frequency/Magnitude Atlas for SRCC Method ......69
Figure A.2—24-Hour, 5-year Recurrence Frequency/Magnitude Atlas for SRCC Method ....69
Figure A.3—24-Hour, 10-year Recurrence Frequency/Magnitude Atlas for SRCC Method ......70
Figure A.4—24-Hour, 25-year Recurrence Frequency/Magnitude Atlas for SRCC Method ......70
Figure A.5—24-Hour, 50-year Recurrence Frequency/Magnitude Atlas for SRCC Method ......71
Figure A.6—24-Hour, 100-year Recurrence Frequency/Magnitude Atlas for SRCC Method ....71
Figure B.1—24-hour, 2-year Frequency/Magnitude Atlas for TP-40 Method.......................72
Figure B.2—24-hour, 5-year Frequency/Magnitude Atlas for TP-40 Method.......................72
Figure B.3—24-hour, 10-year Frequency/Magnitude Atlas for TP-40 Method.......................73
Figure B.4—24-hour, 25-year Frequency/Magnitude Atlas for TP-40 Method.......................73
Figure B.5—24-hour, 50-year Frequency/Magnitude Atlas for TP-40 Method.......................74
Figure B.6—24-hour, 100-year Frequency/Magnitude Atlas for TP-40 Method.......................74
Figure C.1—Approximate SCS Rainfall Distributions Map ..................................................75
Abstract

Ascension Parish is located in Southern Louisiana and is characterized by low slope and low elevation. This combination allows for much hydrologic storage during a flood event. Because Ascension Parish is one of the most rapidly growing areas in the United States, it is necessary to accurately predict the hydrologic and hydraulic properties of flow and accurately model them. Several processes within a hydrologic model greatly influence the output flow hydrographs so selecting correct methods and parameters is important. Transform methods describe how excess precipitation is transformed into runoff. There are several methods that can be used as a transform method, including the SCS method, Snyder method and Clark Unit Hydrograph method. Each method uses a separate set of equations and processes to determine runoff. By estimating method parameters and using these in a hydrologic model, the effects of varying transform method can be quantified. Results showed significant differences amongst the three methods. The Clark Unit Hydrograph method, which accounts for storage within the watershed, resulted in the most accurate peak flows. Accuracy of these outflow hydrographs was tested through an unsteady hydraulic model for Grand Goudine Bayou. Parameters within the hydraulic model were also tested for accuracy by varying factors, channel Manning’s n value and baseflow, until the simulated stage resembled the actual observed stage. A Manning’s n value of 0.090 and a baseflow of 20 cfs resulted in the most accurate model.

Another important hydrologic design parameter is the rainfall depth determined for the area given a particular return period. There are several different methods and distributions which predict rainfall depths, including the TP-40 method, which utilizes the Gumbel distribution, and the SRCC method. For most areas, the SRCC method offers greater rainfall depth estimates of the higher return periods, (i.e. the 50- and 100- year storms); however, for Ascension Parish, the
SRCC method yielded lower rainfall depth estimates for these return periods. When compared after hydraulic modeling, the differences between the SRCC method and TP-40 method were relatively insignificant.
Chapter 1 - Introduction

1.1 Frequency Analysis

Because hydrologic systems are subject to extreme events, including storms and flooding, it is necessary to understand the likelihood of an event happening in any given year (Chow, Maidment, and Mays 1988). These probabilities are utilized in distributions that are used to relate frequency and magnitude of events. These distributions generally suggest a given return period for an event, which is an estimation of the time interval between storm events.

These distributions are developed upon a series of collected data over a period of time. These precipitation data are generally ranked from highest to lowest value and depending on which distribution is being used these data are defined by either an Annual Maximum Series or a Partial Duration Series. An Annual Maximum Series is defined roughly by selecting the largest storm event for a given year, whereas a Partial Duration Series (PDS) includes the largest storms no matter which year they occur. Ranking of these data is necessary if the distribution is to be plotted. Plotting probability data can be useful to interpolate or extrapolate a certain return period (Chow, Maidment, and Mays 1988). Extrapolation is necessary if the record of available event data is particularly short.

There are different methods for determining a proper distribution function and plotting method to correctly determine a return period for a given location. Because of the different methods available, there is much discourse about the accuracy for any method for a given region.

1.2 Hydrologic Modeling

Hydrologic modeling is the use of mathematics to represent the flow of water in a watershed. This watershed generally receives water in the form of precipitation. Most of the water moves through the watershed via a stream network and exits at an outlet. Water can also be
accumulated in storage areas within the watershed. Modeling a hydrologic system can be useful to simulate these various precipitation-runoff processes in a watershed. Data can be collected and used as important inputs for a model, such as precipitation and stream gage data. These input data are combined with various hydrologic processes including routing methods, loss and transform methods. These models produce an estimate of outflow volume and flow at the different watershed elements.

1.3 Hydraulic Modeling

Hydraulic modeling is the use of mathematics to symbolize a system and use calculations to make estimates regarding the components of the waters movement through the system, including flow, water surface elevations, and velocity. Unlike a hydrologic model, a hydraulic model can incorporate various existing structures (e.g. a bridge/culvert, weir, or pump station) along waterways and can calculate the flow through or around the structure, which is useful in determining the relative performance of the structure. A hydraulic model is also useful to understand what is occurring at distinct profiles within the system and also at different time intervals, as in unsteady modeling.

1.4 Study Objectives

Watershed runoff can be heavily affected by many variables, including precipitation and also storage effects within a watershed. It is therefore necessary to understand how high precipitation on a watershed can be transformed into direct runoff. Selecting an appropriate transform method for hydrologic process modeling becomes important because the method affects the outflow reported for a specific subbasin or reach. If these flows are further used for hydraulic routing, then the water surface elevation can be simulated for the stream. Overall, this
means that better prediction for flows through a watershed and water surface elevations in a stream can be analyzed and can be accounted for when developing a drainage plan.

It is important to understand how the overall process for determining frequency analysis can influence the estimated precipitation over a watershed for a given time period. Due to the different scopes of various rainfall frequency estimation methods, it is uncertain the overall effect that these different estimations have on water surface elevations and other hydrologic and hydraulic outputs. By examining the differences between two methods of frequency analysis and using the resultant rainfall estimates as inputs in both hydrologic and hydraulic models, the effects on drainage and overall flood risk predictions can be analyzed. These hydrologic and hydraulic models can be verified for relative accuracy by comparing the simulated peak outflow and water surface elevations with actual observed data collected from a stream gage.

1.5 Overview of Thesis

The first chapter provides the introduction to frequency analysis as well as hydrologic and hydraulic modeling. Also present in Chapter One is the study objectives of the thesis. Chapter Two contains relative background information and contains a literature review for the focus of this thesis. It also contains the study site’s geography. Chapter Three details the methods used in the study, including the methods for determining the appropriate transform method, hydrologic routing methods, unsteady hydraulic modeling, and the comparison of the Southern Regional Climate Center (SRCC) method and the Technical Paper No. 40 (TP-40) method used for frequency analysis. Chapter Four discusses the results from the transform methods, model calibration and validation, and the results from the frequency analysis study. Chapter Five concludes the study and offers some recommendations for future work in relation to the study.
Chapter 2 Background

2.1 Literature Review

2.1.1 Frequency Analysis

There are different forms of probability distributions that can be utilized for frequency analysis of storm events. Of specific interest are the TP-40 method, which uses a Gumbel distribution to determine rainfall depths for a particular return period, and the SRCC method, which uses a log-linear relationship. These two methods are discussed in the following sections.

2.1.1.1 The TP-40 Method

In 1961, the Weather Bureau published Technical Paper No. 40 (TP-40), which outlined rainfall frequency estimates for the 48 contiguous United States (Hershfield 1961). TP-40 was developed as an extension to previously published technical papers because the Weather Bureau had more weather station data available as a series of new stations were implemented across the United States. This TP-40 method was developed so that simple and low frequency storm events along with several existing maps could be used to develop relationships for additional return periods and durations (Hershfield 1961).

Data used for the analysis and development of the rainfall frequency atlas were based on 200 stations that had sufficient records. These 200 stations had duration of 30-minutes to 24 hours and also had an average length of record for 48 years. There were over 1350 stations of daily recording data, but the average length of record was only 16 years, which was believed to be too short to draw conclusions (Hershfield 1961).

Since some of the data was assessed every 24 hours, a 1.13 factor must be applied to all daily rainfall data. This factor is necessary because most 24-hour data is published on an
observational-day basis and typically, the maximum rainfall in any 24 hour (1440 min) rainfall event is 13 percent higher than the observational day (Hershfield and Wilson 1957).

There are two methods for analyzing rainfall series, the Annual Maximum series, which only includes the largest data value for a given year, and the Partial Duration Series (PDS), where the largest rainfall data values are taken despite the year they occurred. This PDS is required for the TP-40 method because sometimes “the second highest [value] of some year occasionally exceeds the highest of some other year” (Hershfield 1961).

Duration and frequency analysis were determined using a diagram that allows for the various durations to be determined based upon an empirical relationship between the 1-hour and 24-hour storms and 2-yr and 100-yr storms, respectively.

To use the diagram in Figure 2.1, a straight line is drawn from the rainfall depth at 1-hour duration to the rainfall depth at 24-hour duration; the depths for the other durations can be determined from where the drawn line intersects the specific duration line. While this chart shows hourly duration, the same method is applied to find various return periods based off the 2-year and 100-year storms. Essentially, this chart is only used to determine storms from 1 to 10 years. For larger storms above the 20-year return period, the Gumbel procedure was used for fitting annual series to the Fisher-Tippett type 1 distribution (Hershfield 1961). In general, the Gumbel procedure is a special case of the Fisher-Tippett type 1 extreme value distribution.

Though the TP-40 method is still widely used, many have started to doubt the accuracy of the TP-40 method for estimates of higher return periods. Though the TP-40 method is considered generally to be a good two-parameter distribution used to represent extreme precipitation data, this distribution was developed based on annual extreme data with short record length (Wilks 1993), which means that questions have been raised about the events
extrapolated beyond this record length. Hershfield (1961) indicates that “100 year values which were computed for 3500 selected points…are the product of the values from the 2-year maps and the 100-year to 2-year ratio maps.” This indicates that the 100-year return period was determined from a ratio developed from data with a relatively short record. Hershfield (1961) even comments that due to length of record, most stations under 60 years, and also the parametric form of the distribution “raises the question of the predictive value of the results—particularly, for the longer return periods.”

![Figure 2.1 Method Within TP-40 for Determining Rainfall Depth for Durations Between 1-hour and 24-hours (Hershfield 1961)](image)

Adding to the questions raised about use of the TP-40 method is the proposition that no single probability distribution could accurately fit all of the conterminous United States. In recent years, development of regional rainfall frequency/magnitude maps has become common (Keim and Faiers 2000). Another area of concern with the TP-40 method is that it may not
accurately represent the data that has been collected since 1961 (Faiers, Keim, and Muller 1997). By adding these data to data collected prior to 1961, there is the potential for better estimations of the higher return periods.

2.1.1.2 The SRCC Method

In 1997, the Southern Regional Climate Center (SRCC) at Louisiana State University developed a method commonly referred to as that SRCC method. This method was developed to re-examine the existing rainfall frequency atlas produced by using the Gumbel distribution in the TP-40 method (Faiers, Keim, and Muller 1997).

An initial study was performed in response to the TP-40 method’s possible limitations described earlier. The initial study was performed across the state of Louisiana and yielded similar, yet “more complex spatial pattern[s],” (Faiers, Keim, and Muller 1997) which lead to the determination of rainfall frequency-magnitude estimates for the entire six-state region of the South, including, Texas, Oklahoma, Arkansas, Louisiana, Mississippi and Tennessee. The pilot study consisted of 27 stations within the six-state region. Data were collected from each station with a minimum period of record of 35 years (Faiers, Keim, and Muller 1997). These data, organized in PDS, were then used to investigate multiple distributions and their fit for the southern states. The SRCC method and the method upon which it was similarly based, the Huff–Angel method, were compared to these distributions and showed that both methods resulted in larger estimations for higher return periods (Faiers, Keim, and Muller 1997).

As mentioned, the SRCC method was developed similarly to the Huff-Angel method, which was originally developed in 1992 for the Midwest region, including nine-states. Arranged in PDS, the SRCC method used the empirical factor of 1.13 to account for the 24-hour moving window (Hershfield and Wilson 1957; Huff and Angel 1992; Faiers, Keim, and Muller 1997).
In the Huff-Angel method, a log-log graphical analysis was used to determine the frequency magnitudes (Huff and Angel 1992). The SRCC method uses a similar graphical analysis; however, it uses a log-linear analysis (Faiers, Keim, and Muller 1997). Figure 2.2 shows the comparison between the Huff-Angel method and the SRCC method.

Though the SRCC method was developed similarly to the Huff-Angel method, the log-linear regression produced lower rainfall estimates for the higher return periods (50-year and 100-year) than the Huff-Angel method, which seemed to produce excessively high rainfall estimates (Faiers, Keim, and Muller 1997). However, for lower return periods (2-, 5-, 10-, and 25-year), the Huff-Angel log-log regression and the SRCC log-linear regression produced similar rainfall estimates.

![Figure 2.2 Differences between the Huff-Angel log-log Method and the SRCC log-linear Method](image)

(Faiers, Keim, and Muller 1997)
2.1.2 Hydrologic Models

Hydrologic models are used to simulate precipitation and runoff processes occurring in a watershed. The model is a physical representation of the watershed composed of the various hydrologic elements as well as a meteorological data and control specifications. The models also generally have different options for methods for transforming excess rainfall to runoff, baseflow representation, and hydrologic routing (Wu & Xu 2006).

2.1.2.1 Transform Methods

Within hydrologic process routing, certain hydrologic processes such as loss methods, transform methods, and routing methods are represented. These user-defined parameters are used to better represent the actual hydrologic process within the watershed. Transform methods specifically are used to define how excess precipitation is transformed into runoff. There are several different methods that are commonly used. These include the Soil Conservation Service method, the Clark Unit Hydrograph method, and the Snyder method.

2.1.2.1.1 The Soil Conservation Service (SCS) Method

The Soil Conservation Service (SCS) method was developed as a transform method for calculating runoff by determining abstractions from rainfall. An abstraction is the water lost by either infiltration or surface storage (Chow, Maidment, and Mays 1988). In this method, storm runoff is denoted \( Q \) and can be calculated by using **Equation 2.1** below.

\[
Q = \frac{(P - 0.2S)^2}{P + 0.8S} \quad \text{EQ. 2.1}
\]

In this equation, \( P \) is equal to the depth of precipitation, and \( S \) is the maximum retention in a watershed (U.S. Department of Agriculture 1986). This method requires the calculation of a Curve Number (CN) that is used to define properties of runoff. According to the SCS Technical
Report 55 (TR-55), the main parameters that determine CN are hydrologic soil group (HSG), cover type, treatment, hydrologic condition and antecedent runoff condition (ARC). The ARC is defined as the index of runoff potential before a storm event (U.S. Department of Agriculture 1986). The CN is calculated using the equation below.

\[ CN = \frac{1000}{S + 10} \]  \hspace{1cm} \text{EQ. 2.2}

Figure 2.3 shows the relationship defined above.

**Figure 2.3 Estimation of Direct Runoff from Curve Number**  
(U.S. Department of Agriculture 1986)

After calculating the curve number, runoff is calculated by using a dimensionless unit hydrograph. This hydrograph is influenced by the Time of Concentration \( T_c \). The \( T_c \) is the time it takes for the runoff to reach the outlet from the most hydraulically distant point of the watershed (U.S. Department of Agriculture 1986).
There are two types of graphs that are used to determine the shape of these unit hydrographs. They are the Standard shape and the Delmarva shape. Generally, the Standard shape is applied most places in the United States, whereas the Delmarva shape, as the name indicates, has been mostly used in Delaware, Maryland and Virginia (HEC-HMS User’s Manual 2008). Associated with each graph type is a specific peaking coefficient, which determines hydrograph steepness. The Standard type graph has a peaking coefficient of 484, while the Delmarva type graph has a peaking coefficient of 284.

2.1.2.1.2 The Snyder Method

The Snyder method was developed to calculate runoff by utilizing a standard unit hydrograph. More important than this standard unit hydrograph are the five characteristics that define the required unit hydrograph. These characteristics are the peak discharge, basin lag, the base time, and widths of the hydrograph at both 50% and 75% of the peak discharge (Chow, Maidment, and Mays 1988).

Basin lag is implemented into Equation 2.3 that is used to determine peak discharge. This equation is shown below.

\[ Q = \frac{C_2 AC_p}{t_p} \quad \text{EQ. 2.3} \]

Where \( Q \) is the peak discharge, \( C_2 \) is equal to 2.75 (Metric) or 640 (English), \( C_p \) is the peaking coefficient, and \( t_p \) is the lag time (Chow, Maidment, and Mays 1988).

There are several alternative equations to determine basin lag and peaking coefficient. Different equations and relationships can be used to determine lag times for different regions. While there are many different ways to determine lag time, the lag time determined by the SCS
method is 0.6*Tc. The lag time determined by the SCS method is considered acceptable to use for the Snyder method. For this thesis, a range of 0.5 to 0.7 was used for the peaking coefficient.

2.1.2.1.3 The Clark Unit Hydrograph Method

Unlike the unit hydrographs, mentioned above, that require a specified duration, the Clark Unit Hydrograph (CUH) was developed using a time-area method (Viessman and Lewis 2003). This method is considered an instantaneous unit hydrograph, which is similar to a regular unit hydrograph except that it is assumed that effective precipitation is applied to a drainage basin in an infinitesimally short period of time.

Essentially, the technique acknowledges the fact that runoff at any point in time relates directly to characteristics of a given watershed. These characteristics include both storage and translation characteristics, which includes the combination of overland and channel travel times with the watershed’s storage (Viessman and Lewis 2003).

Translation through the watershed is accounted for with the watershed’s travel time or Tc. This Tc is defined in the same way as the SCS method mentioned previously. Storage effect or attenuation is accounted for by a built-in storage coefficient. “Many studies have found that the storage coefficient, divided by the sum of time of concentration and storage coefficient, is reasonably constant over a region” (HEC-HMS User’s Manual 2008). Equation 2.4 shows this relationship.

\[
\frac{R}{R + T_c} \quad \text{EQ. 2.4}
\]

The storage coefficient (R) measures the storage of rainfall in the watershed before it can drain to the outlet point. It is measured in units of time. The higher the storage coefficient is in comparison to the time of concentration, the higher the storage within the watershed (Sabol 1988).
While there are numerous methods for determining Clark’s storage coefficient, there is an acceptable way to estimate this parameter. According to Viessman and Lewis (2003), this coefficient can be estimated by multiplying $T_c$ by a constant, which generally ranges from 0.6-2.0. This calculation yields a ratio that can be used in Equation 2.5 to find the storage coefficient, $R$. From this estimation one can require a range of storage coefficients, which is useful in determining the best coefficient to use for a particular watershed.

$$R = \frac{\text{ratio} \times T_c}{1 - \text{ratio}} \quad \text{EQ. 2.5}$$

The CUH method is also easily implemented into computer programs. This ease is due to the fact that the CUH technique already accounts for hydrologic routing procedures, meaning that “a unit hydrograph need not be developed as an intermediate step in converting rainfall excess to a storm hydrograph” (Sabol 1988). This process makes it easy for computerized models to create an outflow hydrograph.

Because of its easy computerized application, the CUH procedure is used in several different modeling programs, including appearing as an option in the Hydrologic Engineering Center’s Hydrologic Modeling System (HEC-HMS), as developed by the United States Army Corps of Engineers (USACE).

2.1.2.2 Hydrologic Flow Routing

Hydrologic routing is a mathematical method for predicting the changing size and speed of a flood wave as it propagates down rivers or through reservoirs (Tewolde and Smithers 2006). Hydrologic routing is useful for understanding the entire movement of water from “rainfall to runoff” (Chow 1959). Essentially, routing is used to determine the unsteady flow rate, along a watercourse. In most cases, hydrologic routing is used for basins and storage reservoirs (Chin
Essentially, the continuity equation is applied to indicate storage between upstream and downstream conditions (Chow, Maidment, and Mays 1988). There are many different methods that have been developed. Depending on the routing method being used either for reservoir or channel routing, the procedure for routing is subject to change (Chin 2000). A commonly used model for channel routing is the Muskingum method.

### 2.1.2.3 Muskingum Flow Routing

Developed for the Muskingum Conservancy District in a flood study, the Muskingum routing method approximates storage in a system according to a wedge method (Chin 2000). The development of the wedge is due to a flood wave that causes inflow to exceed outflow and creates a wedge of storage (Chow, Maidment, and Mays 1988). This method is represented by Equation 2.6.

\[
S = K[XI + (1 - X)O] \quad \text{EQ. 2.6}
\]

Where \( S \) is storage, \( I \) is Inflow, \( O \) is outflow. \( K \) and \( X \) values are determined using characteristics of the channel, where \( K \) is essentially a proportionality constant and \( X \) is a weighting factor that ranges from \( 0 \leq X \leq 0.5 \) (Chow 1959). The \( X \) parameter must range from 0 to 0.5 and defines the shape of the wedge storage (Chow, Maidment, and Mays 1988). An \( X \) value close to 0 would represent almost a linear reservoir, which means there is no backwater; whereas an \( X \) value closer to 0.5 would represent a full wedge. The Muskingum method considers a linear relationship between inflow and outflow. While the Muskingum method adequately approximates the storage within a river reach because of its linear nature the formula shown in Equation 2.6 may not apply to all hydrologic systems.
2.1.3 Hydraulic Models

2.1.3.1 Hydraulic Routing

According to Chow (1959), the hydraulic method from routing is distinguished from the hydrologic routing method in that the hydraulic method uses differential equations for unsteady flow for the solution, while hydrologic routing does not utilize these equations.

2.1.3.2 HEC-RAS Unsteady Modeling

Unsteady flow routing is routinely used because generally, it is more accurate than steady models. Also, unsteady models generally represent more closely what is occurring in a river or natural environment, and they afford a more detailed understanding of river behavior and simulation of real flow events.

Use of steady flow modeling can lead to an overestimation in flow and stage predictions. This overestimation is mostly because flow within the reach is considered constant and completely ignores the potential for storage, which means that flow around bridges and culverts can be miscalculated for a steady flow model (Dyhouse et al. 2006).

HEC-RAS unsteady flow computations are performed by a HEC modified version of the UNET (Unsteady NETwork model) program developed by Dr. Robert L Barkau (Barkau 1992, as cited in HEC-RAS 4.0 User’s Manual 2006). Generally, the flow simulation is performed in a series of three steps. The input data is read, the program is run performing the unsteady flow calculations, and the output is written to a HEC-DSS file. A post processor is then run, which provides significantly detailed hydraulic information for specified timelines (HEC-RAS 4.0 User’s Manual 2006).

It is also suggested that smaller time steps will lead to greater model accuracy and improved stability (Dyhouse et al. 2006). If the model shows instability or a pattern of
nonconvergence, then HEC-RAS will display warning messages. If necessary, the time-step length should be reduced until the model is stable. If stability is not achieved with very small time steps (less than one minute), this may indicate there may be other problems with the model.

The user can specify the computation interval to which this post-processor will run. Hydraulic computations are then performed for each time profile that is requested, beginning with the time interval selected by the user. These computations are performed based upon a computation interval selected by the user. In general, it is necessary to select a computation time that is small enough “to accurately describe the rise and fall of the hydrographs being routed” (HEC-RAS 4.0 User’s Manual 2006). This time is important because if the time step is not small enough, the model can go unstable. If the model runs and is successfully stable, then HEC-RAS will report the results in a DSS file, where the user can view water surface elevations and profile plots.

2.2 Geography of Study Site

Ascension Parish is located in Southeastern Louisiana, located directly southeast of Baton Rouge. The Mississippi River divides the southwest border of Ascension Parish and actually separates Ascension Parish into two parts. East Ascension Drainage encompasses all areas that lie to the east of the Mississippi River. This area is the main focus for this thesis.

The elevation in the parish ranges from about 30 feet above sea level at the north portion of the parish to almost 0 feet (or at sea level) at the south portion of the parish. This means that roughly the slope of the parish is very gentle at only 2-2.5 feet per mile. The slope also decreases from west to east across the parish as well. This is due mostly to the levees bordering the Mississippi River that eventually slopes downward to swamp area in the East. This change in elevation can be seen in Figure 2.4.
The lower east corner of Ascension Parish is bordered almost completely by waterways, including the Amite River to the east and the Blind River to the south. Both of these rivers flow directly into Lake Maurepas. Lake Maurepas is tidally connected to the Gulf of Mexico through a series of connected lakes, including Lake Pontchartrain and Lake Bourne. This connectivity can be seen in Figure 2.5. Tidewaters in the Gulf of Mexico move into Lake Pontchartrain from Lake Bourne. Once the tidewater is in Lake Pontchartrain, it passes through Pass Manchac into Lake Maurepas. These high tides in Lake Maurepas cause increased water surface elevations in both the Blind River and the Amite River. Because these rivers drain most of Ascension Parish, the high water surface elevations can cause serious drainage problems in the low ground elevation areas of Ascension Parish.

Figure 2.4 Decreasing Amite River Elevation
Essentially, elevated water levels in Lake Maurepas cause draining water in reaches to slow, which causes water to back up in Northern Ascension parish (Prescott-Follett Plan 1980). Table 2-1 shows how tidal influence can cause small magnitude storms to become critical storms. It depicts the worst floods in Ascension Parish to have occurred before 1980. Most of the rainfall events were not considered significant but were disastrous because lake levels were elevated. Due to its indirect connection to the Gulf of Mexico, Lake Maurepas is also subject to high lake levels due to storm surge. Overall, this tidal influence adds to the low slopes and low elevations of the terrain to cause major flooding in Ascension Parish even with small precipitation events.

![Figure 2.5 Tidal Influence in Ascension Parish (Braud 2009)](image-url)
Table 2-1 Worst Floods in Ascension Parish, Louisiana, Prior to 1980  
(Prescott Follett Plan 1980)

<table>
<thead>
<tr>
<th>Tide Level</th>
<th>Date</th>
<th>Rainfall depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>+ 1.6 ft</td>
<td>Nov 1961</td>
<td>14.47”</td>
</tr>
<tr>
<td>+ 2.38 ft</td>
<td>Feb 1966</td>
<td>4.26”</td>
</tr>
<tr>
<td>+2.16 ft</td>
<td>April 1966</td>
<td>3.10”</td>
</tr>
<tr>
<td>+ 3.8</td>
<td>April 1977</td>
<td>7.5”</td>
</tr>
<tr>
<td>+4.43</td>
<td>Sept 1977</td>
<td>7.5”</td>
</tr>
</tbody>
</table>
Chapter 3-Methods

3.1 Frequency Analysis

Though there are many different methods available for rainfall frequency/magnitude determination, the two methods used here were the commonly used TP-40 method as well as the SRCC method. Details about these two methods were described in the previous chapter. Each method was reported in a technical report, Technical Paper 40 (Hershfield 1961), and a paper detailing the SRCC method (Faiers, Keim, and Muller 1997). Rainfall magnitude atlases were contained in these publications; these atlases were analyzed carefully to identify differences between the two methods. Figure 3.1 and Figure 3.2 show the 24-hr 10 year storm event for the TP-40 method and the SRCC method, respectively. The red arrows indicate the Ascension Parish area.

Figure 3.1 and Figure 3.2 show significant differences for the estimated rainfall depth for South Louisiana. For the study area, the SRCC method atlas indicates a rainfall depth of 7-7.5 inches for a 24-hour 10 year event; whereas the TP-40 Method suggests a higher value of roughly 8.5 inches for the same event. Frequency/Magnitude maps also exist for the 2-, 5-, 10-, 25-, 50- and 100- year storms for the 3-, 6-, 12- and 24- hour storms (Hershfield 1961; Faiers, Keim, and Muller 1997). The 24 hour storm frequency/magnitude maps for the SRCC method and TP-40 method are represented in Appendix A and Appendix B, respectively.

Table 3-1 shows the comparable values for the different storm events for the SRCC method and the TP-40 method. The values represented here are taken directly from the various return frequency/magnitude atlases and therefore are subject to personal interpolation. Because the map is only measured to the nearest half inch, there is possibility of human error of ± 0.5 inch for the SRCC method and ± 1 inch for the TP-40 method.
Figure 3.1 24-hour, 10 year TP-40 Frequency/Magnitude Atlas for Southern Louisiana
(Hershfield 1961)

Figure 3.2 24-hour, 10 year SRCC Frequency/Magnitude Atlas for Southern Louisiana
(Faiers, Keim, and Muller 1997)
### Table 3-1 Comparison of Rainfall Depths for the SRCC Method and TP-40 Method

<table>
<thead>
<tr>
<th>Storm Event (24 hr)</th>
<th>SRCC Method Value (inches)</th>
<th>TP-40 Method Value (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 yr</td>
<td>4.5</td>
<td>5.25</td>
</tr>
<tr>
<td>5 yr</td>
<td>6.0</td>
<td>7.0</td>
</tr>
<tr>
<td>10 yr</td>
<td>7.0</td>
<td>8.5</td>
</tr>
<tr>
<td>25 yr</td>
<td>8.5</td>
<td>10</td>
</tr>
<tr>
<td>50 yr</td>
<td>10</td>
<td>11</td>
</tr>
<tr>
<td>100 yr</td>
<td>11</td>
<td>12.5</td>
</tr>
</tbody>
</table>

It is noticeable from Table 3-1 that there is considerable difference between the two methods. As discussed in the previous chapter, the justification for developing the SRCC method was that the TP-40 method under estimated the higher return periods, mostly the 50-year and the 100-year return period (Wilks 1993; Faiers, Keim, and Muller 1997). Upon studying the atlases, both the TP-40 method and SRCC method have similar spatial patterns; however, the area surrounding the study site varies greatly. The SRCC method, which generally predicts higher values for most areas, actually reports lower values for Ascension Parish and the surrounding area (Faiers, Keim, and Muller 1997). The reduction in the SRCC method values in Table 3-1 is primarily due to data differences and not selected distribution. Typically the Gumbel distribution used in the TP-40 method underestimates the 50-year and 100-year return period as compared to the SRCC method when developed for the same data record.

These low SRCC method values that surround Ascension Parish can be justified with the data from a gage located in central Ascension Parish at the community of Gonzales. **Figure 3.3**
is a plot showing estimated return period values for Ascension Parish based upon roughly 31 years of rainfall data, ranging from 1978-2009 from NOAA gage, # 163695, which monitors weather data daily. Data were retrieved from the SRCC website (climod.srcc.lsu.edu) and was fitted to the Weibull formula which can be shown by Equation 3.1.

\[ P = \frac{R}{(n+1)} \]  \hspace{1cm} \text{EQ. 3.1}

Where P = probability, R = rank, and n=number of storms in a series (Faiers, Keim, and Muller 1997). Employing a log-linear scale as the SRCC method dictates, the Figure 3.3 shows the current frequency/magnitude relationship in Gonzales, Ascension Parish, LA.

![Figure 3.3 Estimated Rainfall Frequency/Magnitude Values for the SRCC Method at the gage in Gonzales, Ascension Parish, LA for years: 1978-2009](image)

The distribution shown above is only useful to demonstrate the plotting technique for the SRCC method. The values obtained above were not used for actual hydrologic or hydraulic modeling explained below. While used only for demonstration of the SRCC method, it is
noticeable from Figure 3.3 that the return period rainfall depths are slightly higher than suggested in Table 3-1. This increase is due to more than 10 additional years of precipitation data available at this gage since the SRCC method was created, which supports the premise that the TP-40 method is out-dated and the need for constant update of any rainfall frequency/magnitude method.

3.2 Hydrologic Modeling

Although there are various models available to use for hydrologic modeling, the model chosen for this study was HEC-HMS, developed by the Corps of Engineers. HEC-HMS was selected because it is capable of simulating a number of different types of dendritic watersheds that cover most types of geographic areas. Its capabilities include hydrology for large river basins and even for small urban or natural watershed runoff (HEC-HMS User's Manual 2008).

HEC-HMS models for significant watersheds in Ascension Parish were developed. These watersheds were based on a detailed stream network for Ascension Parish, developed using an automated watershed delineation module within the free GIS based program, BASINS (Braud 2009).

Physical characteristics of the particular stream or streams, including river length, river slope, basin slope and longest flow path, were implemented into the BASINS model (Braud 2009). Loss method, transform method, and baseflow type were also applied to the watershed sub-basins and the routing method was applied to the reach elements.

3.2.1 Previous Study

The specific watershed of interest for this thesis is Grand Goudine Bayou (Subbasin 11 in Figure 3.4). In Braud (2009), a HEC-HMS model was created for Grand Goudine Bayou. The basin watershed was divided into four subbasins. Subbasins were labeled starting at the most
downstream subbasin, W1, and increasing as the subbasins extended upstream. These subbasins were connected to the watershed outlet through a series of two reaches, denoted R10 and R20. Three junctions were also added. These junctions were located where, subbasin W4 enters the stream, where subbasins W2 and W3 combine and enter the stream, and the outlet where subbasin W4 also enters the stream. These watersheds can be seen in Figure 3.5.

**Figure 3.4 Delineated Watersheds in Ascension Parish, LA (Braud 2009)**

For each model developed, three main components must be identified. These necessary components are the basin model, the meteorologic model and the control specifications. In Braud (2009), the SCS method was selected as the meteorologic model. The SCS method requires two inputs, storm type and depth. The storm type can be determined for a specific region. The
different regions are displayed in the map found in Appendix C. For the study region, Type 2 was selected. Since this storm type requires input of precipitation depth in inches, the values obtained for the 5-, 10- and 100-year events from the SRCC and the TP-40 methods were used as inputs for the 24 hour duration storm.

The next main component to be determined was the control specifications. These specifications control the start and stop time and date of a simulation and also controls the time interval for which calculations are made. One limitation to the HEC-HMS program is that the time interval, \( T < 0.29 \times TLAG \), where \( TLAG \) is the shortest lag time in hours. For Braud (2009) the time interval was set as 1 minute. The simulation was set to start at 1:00 a.m. on January 1, 2009 and end at 2:00 p.m. on January 2, 2009. Though the time stamp given to the model is insignificant, the fact that the simulation time spans for 37 hours is important. This simulation time period was chosen into ensure that the entire storm event could run and the complete outflow hydrograph could be computed.

After determining the main components of a HEC-HMS model, other important parameters must be specified by the user. As mentioned previously, loss methods, transform methods and routing method are necessary parameters. In Braud (2009), both the loss and transform method utilized the SCS method for the modeling runs. For the SCS loss method, user inputs include subbasin curve number. This curve number (CN) was determined from a generated curve number grid for Ascension Parish, LA. For the study region, the CN grid was developed by incorporating soil conditions as well as land cover identified in the region (Braud 2009). Braud (2009) utilized soil type data for Grand Goudine downloaded from the Natural Resources Conservation Service (NRCS) website (soildatamart.nrcs.usda.gov) and land use data downloaded from the USGS website (seamless.usgs.gov).
For the transform method, the SCS unit hydrograph method was used. For the SCS unit hydrograph method, the user must first determine between using the “Standard” or “Delmarva” graph type. The Delmarva graph type was selected because the Standard graph type seemed to yield outflow values that were too high for the area. This graph type has a peaking coefficient of 284. Generally, this graph type was created for areas with low slope and thus high storage capacity, like Ascension Parish, LA (Braud 2009). The inputs for the SCS method are lag times for each subbasin. In Braud (2009), lag times are calculated by determining the longest flow path in the subbasin by utilizing the NRCS curve number method. The lag times determined in Braud
(2009) were used in this thesis study. Table 3-2 shows both the curve number and lag time for each Grand Goudine subbasin.

Table 3-2 Grand Goudine Watershed SCS Curve Numbers and Lag Times

<table>
<thead>
<tr>
<th>Subbasin ID</th>
<th>SCS Curve Number</th>
<th>Lag time (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>W1</td>
<td>78.99</td>
<td>135.30</td>
</tr>
<tr>
<td>W2</td>
<td>79.47</td>
<td>84.97</td>
</tr>
<tr>
<td>W3</td>
<td>82.95</td>
<td>75.39</td>
</tr>
<tr>
<td>W4</td>
<td>83.19</td>
<td>70.06</td>
</tr>
</tbody>
</table>

For the routing method, the Muskingum routing method was chosen for the reach elements. It requires two user input parameters: X and K. These variables are a weighting parameter and reach travel time, respectively. The Muskingum X parameter of 0.25 was used for Grand Goudine Bayou. While the values for the Muskingum X parameter can range from 0-0.5, where a 0 value represents natural channels and a 0.5 value represents straightened and improved channels (Braud 2009). While Grand Goudine is maintained, it is still a natural channel. The Muskingum K parameter represents the travel time through the reach. In the previous study, this value was calculated for each individual reach where Muskingum Routing was selected using the following equation: \( K = \text{Reach Length (ft)} / (2 \text{ ft/sec } \times 3600 \text{ sec/HR}) \). This 2 ft/sec velocity was chosen due to the slight slope for Ascension Parish. The K values calculated for reach R10 and R20 were 5.44 HR and 9.07 HR, respectively (Braud 2009).

3.2.2 Current Study

Because this study was complementary to the previous study performed on Grand Goudine Bayou, many settings and parameters for the hydrologic model were kept. For this
study, the basin area and delineation were kept. The loss method was kept as the SCS Curve Number method, and the routing method was kept as the Muskingum method. All previously defined meteorologic data were kept as well. The parameters that were changed were the transform method, the baseflow and the control specifications. The transform method was studied in order to determine how alternatives affected the hydraulic model. Baseflow was altered in order to provide a constant flow to help the unsteady hydraulic model run more smoothly. The control specifications were lengthened in order to ensure the entire storm event was modeled.

3.2.2.1 Transform Methods

In addition to the SCS method used in Braud (2009), two additional transform methods were tested as alternatives for use in model calibration. The methods tested were the Snyder method and the CUH method. The Snyder method requires two user inputs: the lag time and the Snyder peaking coefficient. Three values (0.5, 0.6 and 0.7) were tested for Snyder peaking coefficient, Cp. The lag times were entered as they appear in Table 3-2.

The CUH method was also tested. The inputs for the CUH method are $T_c$ and storage coefficient, R. Storage coefficients used in this thesis study were determined from Equation 2.5.

The ratio shown in Equation 2.5 is determined based upon specific watershed characteristics. As mentioned in Section 2.1.2.1.3, this ratio can be estimated by multiplying the $T_c$ from each subbasin by a constant between 0.6 and 2.0. For this study, the shortest $T_c$ time was multiplied by 2.0 and the longest $T_c$ time was multiplied by 0.6. By doing this, the maximum storage ratio was determined for the watershed with the shortest $T_c$ time, and the minimum ratio was determined for the watershed with the longest $T_c$ time. Using this approach, all other watershed ratios would fall between these. This approach provided a range of storage
ratios needed to determine watershed storage coefficients. The range of ratio was 0.375--0.667. For the calculation details, see Appendix D. Four specific ratios were used to determine different storage coefficients: 0.5, 0.6 and 0.65. These were selected for detailed analysis because the ratios seemed to account for higher amount of storage. The value of 0.70 was also tested, even though it is out of the range. The different storage coefficients as well as the $T_c$ were input into HEC-HMS.

### 3.2.2.2 Baseflow

In the previous study, no baseflow was used for individual subbasins. For this study a low baseflow was assigned to each subbasin. The purpose of using a low baseflow was to aid in the hydraulic modeling performed for this study. It helped the hydraulic model run more smoothly by creating a small and constant inflow. The amount of baseflow added was determined using a series of trial runs. In this study, for each trial a different baseflow was used. Values of 5, 10, 12, 15, 18 and 20 cfs were added individually to each subbasin in HEC-HMS. Other values yielded similar results as the six values mentioned above and were therefore not included in this study. The selected baseflow was applied in HEC-HMS as a constant monthly scale, meaning a constant value for baseflow was added to each month. The HEC-HMS models were run, and the resulting subbasin outflow hydrographs were input into HEC-RAS. Once the hydrographs were implemented, the HEC-RAS unsteady model was run. The stage hydrograph output was compared to the observed stage, with particular attention paid to the water surface elevations prior to the storm event.

### 3.2.2.3 HEC-HMS Control Specifications

The simulation time of Braud (2009) was set to run for 37 hours, starting at 01:00 on January 1, 2009 and ending at 14:00 on January 2, 2009. Here, the simulation time was
lengthened to ensure that the model adequately simulated the entire storm event. In addition, the simulation duration was lengthened to ensure that the hydraulic modeling duration was long enough to allow the initial simulated water surface elevation to become level and therefore to better represent the channel prior to the precipitation event. The simulation time for this study starts at 01:00 on December 31, 2008 and ends at 14:00 January 4, 2009, lengthening the simulation period from 37 hours to 109 hours.

This study used a time interval of one minute, which is the same as the previous study. This interval was calculated using the same equation described in Section 3.2.1. The same time interval was used due to the fact that the subbasins and lag times were also unchanged.

3.3 Hydraulic Modeling

The program selected for hydraulic modeling was the HEC-RAS program developed by the United States Army Corps of Engineers (USACE). This program is useful in that it is developed with a graphical user interface and is capable of performing steady flow, unsteady flow, sediment transport and water quality functions (HEC-RAS 4.0 User’s Manual 2006). Here, mostly unsteady flow analysis was utilized. The purpose for the hydraulic modeling was to simulate the unsteady flow through Grand Goudine Bayou.

3.3.1 Hydraulic Modeling of Grand Goudine Bayou

Cross sections for Grand Goudine Bayou were obtained from the East Ascension Drainage Works Department, as part of the cross sections originally developed by BCG Engineering Company. These cross sections were originally created as a detailed Flood Insurance Study (FIS) in Ascension Parish, Louisiana. As per detailed FIS studies, these cross sections take into account actual surveyed data. Though much of Ascension Parish’s waterways have been improved, this model was developed previous to those improvements. This fact
means that the survey data may no longer reflect actual existing geometry for Grand Goudine but does reflect conditions for the events modeled and was useful as a basis for comparing the SRCC and TP-40 methods. These cross sections were also modified from the original BCG design, in that some cross sections were eliminated from this study. The reason is that using the four subbasins delineation as determined by Braud (2009) resulted in the lack of hydrologic data for the most upstream portion of the stream.

The geometric file data used for this study is represented in **Figure 3.6.** The portion of Grand Goudine that was studied is from where subbasin W4 enters Grand Goudine to the junction of Grand Goudine with New River Canal. For this study, 54 cross sections were used along the channel. Two cross sections are required immediately upstream and downstream of a bridge or culvert. In cases where these did not exist, cross sections were interpolated.

As mentioned earlier, Grand Goudine was divided into four subbasins. At the outlet of each subbasin, a hydrograph was produced by HEC-HMS. For the HEC-RAS unsteady flow data, DSS files from these HEC-HMS output hydrographs for each subbasin were inserted at geographically correct locations. This process was done by using the unsteady flow editor which lets the user insert specific DSS files at a specific cross section.

The geographically correct cross section, where flows were added, was determined by measuring the river reach length of the watershed from HEC-HMS and measuring that exact distance along the reach in HEC-RAS. If a cross section was not present at this exact distance then a new cross section was created using a method of cross section interpolation. Cross section interpolation between two cross sections requires for the identification of the two bounding cross sections as well as the maximum distance that should be between them. In this case, cross sections were interpolated for two areas, upstream station 19165.5, where W4 enters Grand
Goudine Bayou, and station 7304.66, where subbasins W2 and W3 enter. These stations will now be referred to as 19165 and 7304, respectively.

Once new cross sections were developed, the flow data was entered into the model. As mentioned earlier the unsteady flow data is entered via the use of DSS files of output hydrographs from HEC-HMS. This form of data is useful in order to guarantee that the models are congruent. Data are input into an unsteady flow editor shown in Figure 3.7 where each DSS file is assigned to a particular cross section. The flow files are assigned one cross section upstream from where the flow actually enters.

![Figure 3.6 HEC-RAS Geometric Profile for Grand Goudine Bayou](image)
For this study, several different flow data profiles were created. Each flow data file consisted of the flow hydrographs for a particular HEC-HMS run. These runs include all the trials for each separate transform method (SCS, CUH, and Snyder) as well as the 5-, 10- and 100 yr return periods for storms using both the SRCC and TP-40 methods.

![Figure 3.7 HEC-RAS Unsteady Flow Editor for Grand Goudine Bayou](image)

Unsteady flow data inputs are entered into the model as boundary conditions, either an upstream boundary condition, downstream boundary condition or internal boundary condition. Upstream or downstream selection dictates what type or boundary condition should be entered. Upstream boundary condition types include flow hydrograph, stage hydrograph or a combination flow/stage hydrograph. For a downstream boundary condition, the data can be entered not only
as a flow hydrograph, stage hydrograph or a combination flow/stage hydrograph but also a rating curve or normal depth (HEC-RAS 4.0 User’s Manual 2006). For internal boundary conditions, there are lateral inflow hydrographs, lateral uniform inflow or groundwater interflow.

The upstream boundary condition selected for this model was a flow hydrograph. This hydrograph is the flow output hydrograph from HEC-HMS for subbasin W4. It was implemented at the most upstream cross section, 19165. Two internal boundary conditions were necessary for this model. A lateral hydrograph was defined at cross section 7304 where subbasins W2 and W3 combine and at cross section 422 where subbasin W1 enters. The hydrographs from both subbasins W2 and W3 were added together prior to being put into the model.

If normal depth is used for the downstream boundary condition, Manning’s equation is used to determine downstream stage levels. This boundary condition requires that an energy slope be entered for the reach. If the energy slope is unknown then the user can approximate it by entering the water surface slope or the slope of the channel bottom (HEC-RAS 4.0 User’s Manual 2006). For Grand Goudine Bayou a normal depth friction slope of 0.0002 was used. This value was selected based upon the extreme low slopes that are found within the vicinity of the outlet or the slope of the channel of roughly the last two to three cross sections. The slope measured for the last three cross sections in Grand Goudine Bayou was 0.0002. There is much sensitivity to this value. If the normal depth friction slope is increased or decreased, the water surface elevation changes by lowering or rising, respectively.

After these data are entered there are singularly only a few edits that should be made. Manning’s n values should be entered for the left overbanks (LOB), channel, and right overbank (ROB). Initially, the Manning’s n values were determined by BCG engineers, but for this thesis
study Manning’s n values for overbanks were based upon visual assessment, which means that roughness was determined based upon what land cover could be seen from arial view. The BCG determined values are represented below in Figure 3.8. The n#2 values that are highlighted in green represent the Manning’s n value for the channel.

Figure 3.8 Initial Manning’s n Values for Grand Goudine Bayou

Manning’s n values, which account for roughness of the channel, can vary depending on the conditions of the channel. According to Chow (1959), Manning’s n values for a channel similar to Grand Goudine Bayou, a major stream with a top width of greater than 100 feet at flood stage, can range from 0.035 to 0.100. As shown in Appendix E, a Manning’s n value of 0.035 typically represents a channel that is “clean, straight, full stage, [with] no rifts or deep...
pools,” whereas a Manning’s n value of 0.100 would be a channel with “very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush” (Chow 1959). According to the existing FIS report, Manning’s n values of 0.035 to 0.08 were used for Grand Goudine Bayou (Federal Emergency Management Agency 2007). For this study, Manning’s n values of 0.035 to 0.09 were analyzed to see how the changes in Manning’s n affected the overall simulated stage output hydrograph.

The final edit made to the hydraulic model were contraction and expansion coefficients. Contraction and expansion coefficients are used when flow expands or contracts due to significant changes in cross sections that result in energy losses (HEC RAS Hydraulic Reference Manual 2002). The default values for contraction and expansion coefficients are 0.1 and 0.3, respectively. According to Table 3-3, contraction and expansion coefficients of 0.3 and 0.5 should be assigned to bridge sections. Coefficients are assigned to two cross sections upstream of a bridge and one cross section downstream of a bridge. Contraction and expansion coefficient default values were initially tested for this thesis study, but contraction and expansion coefficients of 0.3 and 0.5, respectively, were tested at bridge sections.

**Table 3-3 Subcritical Flow Contraction and Expansion Coefficients**  

<table>
<thead>
<tr>
<th></th>
<th>Contraction</th>
<th>Expansion</th>
</tr>
</thead>
<tbody>
<tr>
<td>No Transition loss computed</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Gradual Transitions</td>
<td>0.1</td>
<td>0.3</td>
</tr>
<tr>
<td>Typical Bridge Sections</td>
<td>0.3</td>
<td>0.5</td>
</tr>
<tr>
<td>Abrupt Transitions</td>
<td>0.6</td>
<td>0.8</td>
</tr>
</tbody>
</table>
After editing the model, the unsteady flow analysis was run. A simulation start time was selected to match the same start time at which the HEC-HMS models were run. A computation of 30 seconds was selected due to large number of bridges/culverts in the model and the need for model stability and accuracy. Because multiple models were being run to compare the SRCC method and TP-40 method, it was necessary to make sure that both models could run at the same computation interval.

### 3.3.2 Calibration of the HEC-RAS model

It is necessary to calibrate a model to ensure its accuracy. A common way to calibrate a HEC-RAS model is by using observed hydrologic data available from sites along the stream. These data can be in the form of stage data or flow hydrographs. Grand Goudine Bayou has an active USGS stream gage, #0738022295, located approximately 2.7 miles upstream of the outlet; this gage is located in the model by cross section 14214. Figure 3.9 shows the aerial of Grand Goudine Bayou, as well as the gage located around the middle of the reach. The gage is indicated by a green marker.

Stage data were collected from the USGS for this stream gage for known 24-hour storm events, occurring on April 30, 2006; October 22, 2006; and September 02, 2008. The model was calibrated to the April event and then tested to the two other events. For comparison, the observed data are input into HEC-RAS via the Options tab in the Unsteady Flow Editor where the user has the option to add Observed (Measured) Data. This data can be input as one of three forms including, Time Series (in DSS), High Water Marks, and Ratings Curves (Gages). For this calibration, the Time Series option was used. This selection requires user input of observed flow or stage value DSS files. These DSS files contained the stage data mentioned previously.
While these stage data were originally recorded with the actual dates for the events (in April, October, or September), a common time interval must be set for this observed data and the simulation. Because the existing HEC-RAS model was already set at a start time of December 31, 2008 at 1:00 AM, the observed gage data for the 24 hour storm event was modified to fit this time and duration.

It was assumed that the precipitation event must be a significant storm with duration of only 24 hours. The three events used in this study were selected because the majority of the precipitation in those events occurred in a 24 hour span. To ensure that there had not been significant rainfall prior to the chosen storm event, data from three days before and after the
event were also collected. Precipitation was analyzed in order to determine when the isolated
24-hour event actually occurred. After the start and end times of the precipitation event were
determined, the stage data corresponding to this time were then collected.

Precipitation was added as Time Series gage data in HEC-HMS. Rather than being
added directly at the simulation start time, it was added 24 hours after the simulation was set to
begin. As mentioned before, this ensured that the unsteady HEC-RAS simulation would have
time for the initial flow to become level. The resulting outflow hydrographs were then used as
input boundary condition in the HEC-RAS model. The stage data were then fit to the simulation
start time in the same manner as described above. The observed stage data were input into the
HEC-RAS model and the model was run.

Mentioned earlier, Manning’s n value for the channel that was used for all simulations
was determined from these calibration steps. The method for determining the Manning’s n
equation involved a systematic process. The original Grand Goudine model, developed by BCG
engineers, used channel Manning’s n values of 0.06. While this Manning’s n value is relatively
high, it did not seem to adequately measure the roughness, which caused a steeper hydrograph
tail that did not approximate the observed data sufficiently. Other Manning’s n values were
tested to see the effect on the hydrograph tail. These values include 0.035, 0.05, 0.06, 0.08 and
0.09. These Manning’s n values were coupled with changes in baseflow until the simulated and
observed stage hydrograph matched. Matching simulated and observed stage hydrographs were
referred to as the “best fit.” This fit was determined by comparing the mean square error for
each trial. The least mean square error was determined to be the “best fit.” This means that the
simulation with a particular Manning’s n and specific baseflow adequately simulated what was
actually occurring in Grand Goudine.
Chapter 4 - Results

4.1 Hydrologic Analysis

The hydrologic transform methods described previously were utilized to determine a “best fit” model, as described previously. This technique is useful in analyzing the different rainfall frequency/magnitude methods. While the transform methods all have similar fundamental equations, results differed greatly. Graphs and tables below show the differences between the flow hydrographs of the different transform methods as well as the comparison of resulting stage hydrographs from using these different hydrographs as inputs.

Mentioned earlier, three different transform methods were selected for testing: the SCS method, Snyder method and CUH method. Within each of these, several input parameters were tested. For the SCS method, parameters determined in the previous study were kept. In the Snyder Method, peaking coefficients of 0.5, 0.6, and 0.7 were tested. For the CUH Method, storage ratios of 0.5, 0.6, 0.65, and 0.7 were tested. Each method was tested individually and the mean square error result of each trial was compared to others methods.

For the CUH method, storage coefficients were calculated from the equation shown in Section 3.2.2.1. Table 4-1 shows the calculated storage coefficients in relation to the subbasin area (mi²) and Tc (HR). As expected, a ratio of 0.5 yields storage coefficients equivalent to the Tc in hours. The ratio and the storage coefficient are proportional. Therefore, as the ratio increases, so does the storage coefficient. Each method was input into HEC-HMS and the resulting outflow hydrographs compared. Table 4-2 shows a comparison of peak outflow values for each subbasin using each method. Each method was compared to the original SCS method. The SCS method was chosen as the control method because it was used in the previous study. The percent differences between each method and the SCS Method are shown in Table 4-3.
### Table 4-1 CUH Calculated Storage Coefficients

<table>
<thead>
<tr>
<th>Subbasin</th>
<th>Area (mi²)</th>
<th>Tc (HR)</th>
<th>R from Ratio = 0.5</th>
<th>R from Ratio = 0.6</th>
<th>R from Ratio = 0.65</th>
<th>R from Ratio = 0.7</th>
</tr>
</thead>
<tbody>
<tr>
<td>W1</td>
<td>3.145</td>
<td>3.758</td>
<td>3.758</td>
<td>5.637</td>
<td>6.980</td>
<td>8.769</td>
</tr>
<tr>
<td>W2</td>
<td>1.322</td>
<td>2.360</td>
<td>2.360</td>
<td>3.540</td>
<td>4.383</td>
<td>5.507</td>
</tr>
<tr>
<td>W3</td>
<td>1.653</td>
<td>2.094</td>
<td>2.094</td>
<td>3.141</td>
<td>3.889</td>
<td>4.886</td>
</tr>
<tr>
<td>W4</td>
<td>2.592</td>
<td>1.946</td>
<td>1.946</td>
<td>2.919</td>
<td>3.614</td>
<td>4.540</td>
</tr>
</tbody>
</table>

### Table 4-2 Comparison of Flows for Transform Methods

<table>
<thead>
<tr>
<th>Subbasin</th>
<th>Snyder 0.5</th>
<th>Snyder 0.6</th>
<th>Snyder 0.7</th>
<th>SCS</th>
<th>CUH 0.5</th>
<th>CUH 0.6</th>
<th>CUH 0.65</th>
<th>CUH 0.7</th>
</tr>
</thead>
<tbody>
<tr>
<td>W1</td>
<td>843.0</td>
<td>1001.8</td>
<td>1162.5</td>
<td>693.0</td>
<td>669.1</td>
<td>512.6</td>
<td>444.7</td>
<td>376.3</td>
</tr>
<tr>
<td>W2</td>
<td>526.9</td>
<td>616.0</td>
<td>700.9</td>
<td>433.4</td>
<td>433.6</td>
<td>340.1</td>
<td>295.6</td>
<td>252.6</td>
</tr>
<tr>
<td>W3</td>
<td>799.6</td>
<td>929.2</td>
<td>1051.0</td>
<td>671.4</td>
<td>664.3</td>
<td>521.6</td>
<td>453.3</td>
<td>386.8</td>
</tr>
<tr>
<td>W4</td>
<td>1315.5</td>
<td>1522.5</td>
<td>1716.3</td>
<td>1119.0</td>
<td>1097.1</td>
<td>861.9</td>
<td>748.2</td>
<td>637.4</td>
</tr>
</tbody>
</table>

### Table 4-3 Percent Difference of Flows as Compared to the SCS Method

<table>
<thead>
<tr>
<th>Subbasin</th>
<th>Snyder 0.5</th>
<th>Snyder 0.6</th>
<th>Snyder 0.7</th>
<th>SCS</th>
<th>CUH 0.5</th>
<th>CUH 0.6</th>
<th>CUH 0.65</th>
<th>CUH 0.7</th>
</tr>
</thead>
<tbody>
<tr>
<td>W1</td>
<td>21.6%</td>
<td>44.6%</td>
<td>67.7%</td>
<td>0.0%</td>
<td>-3.6%</td>
<td>-35.2%</td>
<td>-55.8%</td>
<td>-84.2%</td>
</tr>
<tr>
<td>W2</td>
<td>21.6%</td>
<td>42.1%</td>
<td>61.7%</td>
<td>0.0%</td>
<td>0.1%</td>
<td>-27.4%</td>
<td>-46.6%</td>
<td>-71.6%</td>
</tr>
<tr>
<td>W3</td>
<td>19.9%</td>
<td>38.4%</td>
<td>56.6%</td>
<td>0.0%</td>
<td>-1.1%</td>
<td>-28.7%</td>
<td>-48.1%</td>
<td>-73.6%</td>
</tr>
<tr>
<td>W4</td>
<td>17.6%</td>
<td>36.1%</td>
<td>53.4%</td>
<td>0.0%</td>
<td>-2.0%</td>
<td>-29.8%</td>
<td>-49.6%</td>
<td>-75.6%</td>
</tr>
</tbody>
</table>
Table 4-2 and Table 4-3 clearly show that the Snyder method yields higher flow peaks than both the SCS method and the CUH method. The CUH method yields lower peak flows than the SCS method. Differences are emphasized with increased ratio/peaking coefficient value. The difference between the CUH method and the SCS method can be explained by storage. The CUH Method differs from other methods by not only using the Tc to develop a translation hydrograph built upon a time-area curve, but also by routing the resulting translational hydrograph through a linear reservoir to account for storage (Sabol 1998; HEC-HMS User’s Manual 2008). In comparison, the SCS method accounts for abstractions, but these are mostly based upon land use and soil group and less upon storage. The Snyder method yielded higher flows than the others because outflow is directly proportional to the peaking coefficient. As mentioned in Section 2.1.2.1.2, the peaking coefficient determines the steepness of the outflow hydrograph. The direct runoff volume is constant for each subbasin no matter the transform method used. Therefore, if the steepness of the hydrograph is increased by increasing the peaking coefficient, then the only way to keep the volume constant is to increase the peak outflow. This explains why the Snyder method has higher peak outflow values than the other trials. Figures 4.1, 4.2, 4.3 and 4.4 show the comparison of outflow hydrographs for each subbasin.

For all subbasins, the Snyder method with peaking coefficient of 0.7 produces the highest peak flow. For all subbasins, the CUH method with ratio of 0.7 produces the lowest peak flow. As seen in Table 4-3, the CUH method is as expected, the Snyder Method with peaking coefficient of 0.7 also produces the steepest hydrograph and the CUH method with ratio of 0.7 produces the hydrograph with the gentlest slope. In fact, as expected, the trend was consistent for all subbasins.
Figure 4.1 Comparison of Flow Hydrographs for Subbasin W1

Figure 4.2 Comparison of Flow Hydrographs for Subbasin W2
Figure 4.3 Comparison of Flow Hydrographs for Subbasin W3

Figure 4.4 Comparison of Flow Hydrographs for Subbasin W4
For this study, the proper transform method was determined by implementing the subbasin outflow hydrographs from HEC-HMS as boundary condition inputs into HEC-RAS. After running the HEC-RAS model, the resulting simulated stage was compared to the actual observed stage, and then the method that produced the best overall match was selected as the appropriate transform method. The simulated stage versus the observed stage is shown for each of the tested transform methods.

It is clear from Figure 4.5, 4.6, 4.7, 4.8, 4.9, 4.10, 4.11 and 4.12 that both the SCS method and the Snyder method overestimate peak flow, where as the CUH method has a less extreme peak and appears to represent observed flow more accurately. Visually, the CUH method with storage coefficient of 0.7 provides a simulated stage closest to the actual observed stage. For a storage ratio of 0.7, the peak is reduced the most; however, it is noticeable that the initial stage of the simulated stage (in blue) is higher than the observed stage (in black). This difference is due to the baseflow that was assigned to the subbasins in the hydrologic model. As part of the calibration process, lower baseflow values were tested to see if the simulated initial stage could be reduced; however, the lower baseflow values caused a significant increase in the steepness of the tail of the stage hydrograph. Results of these tested baseflow values can be seen in the next section.

In order to verify the goodness of fit of each trial, the mean square error was calculated following each simulation. From this, the least mean square error was determined. The trial with the least mean square error was determined to be the most accurate and the “best fit”. These results are presented in Table 4-4. From this table it can be seen that the CUH method with ratio of 0.7 results in the most accurate model, with a MSE of 0.223. As the CUH ratio increases the MSE decreases.
Figure 4.5 Simulated vs. Observed Stage for Grand Goudine for a Snyder Transform Method with Cp of 0.5

Figure 4.6 Simulated vs. Observed Stage for Grand Goudine for a Snyder Transform Method with Cp of 0.6
Figure 4.7 Simulated vs. Observed Stage for Grand Goudine for a Snyder Transform Method with Cp of 0.7

Figure 4.8 Simulated vs. Observed Stage for Grand Goudine for a SCS Transform Method
Figure 4.9 Simulated vs. Observed Stage for Grand Goudine for a CUH Transform Method with ratio of 0.5

Figure 4.10 Simulated vs. Observed Stage for Grand Goudine for a CUH Transform Method with ratio of 0.6
Figure 4.11 Simulated vs. Observed Stage for Grand Goudine for a CUH Transform Method with ratio of 0.65

Figure 4.12 Simulated vs. Observed Stage for Grand Goudine for a CUH Transform Method with ratio of 0.7
<table>
<thead>
<tr>
<th>Transform Method</th>
<th>Mean Square Error (MSE)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Snyder 0.5</td>
<td>0.335</td>
</tr>
<tr>
<td>Snyder 0.6</td>
<td>0.374</td>
</tr>
<tr>
<td>Snyder 0.7</td>
<td>1.24</td>
</tr>
<tr>
<td>SCS</td>
<td>0.476</td>
</tr>
<tr>
<td>CUH 0.5</td>
<td>0.287</td>
</tr>
<tr>
<td>CUH 0.6</td>
<td>0.247</td>
</tr>
<tr>
<td>CUH 0.65</td>
<td>0.231</td>
</tr>
<tr>
<td>CUH 0.7</td>
<td>0.223</td>
</tr>
</tbody>
</table>

4.2 Calibration

Several factors were varied to calibrate the model. These were Manning’s n of the channel, initial baseflow, and transform method. The transform method was already described. Both Manning’s n and baseflow greatly influenced the overall calibration.

In brief, the baseflow is a parameter added to each subbasin in the hydrologic aspect of the modeling. In Braud (2009), no baseflow was added to the subbasins. However, in this study, baseflow was added to all subbasins to help the hydraulic model run more smoothly by creating a small and almost negligible constant inflow. Several values ranging from 5 cfs to 20 cfs were tested. Any values larger than 20 cfs drastically overestimated the initial flow by overestimation. Values less than 5 cfs caused low inflow values, and the model did not run properly. A comparison of flows can be seen in the Figures 4.13, 4.14, 4.15, 4.16, 4.17 and 4.18.

Each trial was run using the CUH method with ratio 0.7. Of all the trials, there were four baseflow values that could have been used: 12 cfs, 15 cfs, 18 cfs and 20 cfs. Baseflow of 12 cfs has an initial stage that aligned with the initial observed stage, but the simulated hydrograph tail was steeper than the observed. Baseflow of 15 cfs resulted in a higher initial simulated stage but
the hydrograph tail more resembled the observed. Baseflow of 18 cfs resulted in a hydrograph similar to the hydrograph of 15 cfs where the initial simulated stage was higher but the hydrograph tail more closely resembled the tail of the observed stage hydrograph. The simulations with baseflow of 12 cfs or even 15 cfs did not provide enough flow to raise the tail of the simulated stage hydrograph to match the tail of the observed hydrograph. Baseflow of 20 cfs resulted in a high initial simulated stage but relatively matched the hydrograph tail. Baseflow of 20 cfs was chosen for two reasons. First, a baseflow of 20 cfs provides enough flow to extend the tail of the stage hydrograph, and second the elevated initial stage does not seem to change the overall peak. To verify goodness of fit, the mean square error was calculated for each trial. The least mean square error was then determined, and the trial with the least mean square error was determined the most accurate. This error comparison is shown in Table 4-5. From the table it can be see that baseflow of 20 cfs results in the most accurate model with MSE of 0.223.

![Figure 4.13 Simulated vs. Observed Stage for Gr and Goudine for a Baseflow of 5 cfs](image)
Figure 4.14 Simulated vs. Observed Stage for Grand Goudine for a Baseflow of 10 cfs

Figure 4.15 Simulated vs. Observed Stage for Grand Goudine for a Baseflow of 12 cfs
Figure 4.16 Simulated vs. Observed Stage for Grand Goudine for a Baseflow of 15 cfs

Figure 4.17 Simulated vs. Observed Stage for Grand Goudine for a Baseflow of 18 cfs
Figure 4.18 Simulated vs. Observed Stage for Grand Goudine for a Baseflow of 20 cfs

Table 4-5 Mean Square Error for Baseflow Simulations

<table>
<thead>
<tr>
<th>Baseflow (cfs)</th>
<th>Mean Square Error</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>6.57</td>
</tr>
<tr>
<td>10</td>
<td>2.20</td>
</tr>
<tr>
<td>12</td>
<td>1.50</td>
</tr>
<tr>
<td>15</td>
<td>0.882</td>
</tr>
<tr>
<td>18</td>
<td>0.591</td>
</tr>
<tr>
<td>20</td>
<td>0.223</td>
</tr>
</tbody>
</table>

The next parameter used for calibration was Manning’s n value for the channel. This parameter is important because it affects the tail of the hydrograph. Manning’s n parameters were tested over a large range, using the following values: 0.035, 0.05, 0.06, 0.08, and 0.09.
These values were selected due to a large range of values that were originally used in the FIS report for Ascension Parish. In the FIS, values ranged from 0.035 to 0.08 (Federal Emergency Management Agency 2007). These values were determined to provide an adequate sample. **Figure 4.19** shows the graphical comparison of the Manning’s n values.

Changing the channel Manning’s n had three primary effects. It caused a change in the tail of the hydrograph and also caused a change in the initial stage and therefore, the peak stage. This relationship is directly proportional. As the Manning’s n increases, the initial stage increases and also as Manning’s n increases, the overall tail of the hydrograph increases. While a Manning’s n of 0.090 predicts a high initial stage, its tail best matches the observed stage’s tail. A Manning’s n of 0.090 was selected over a Manning’s n of 0.080 because the differences between the increased initial stage and peak stage were minimal, while the Manning’s n of 0.090 has a better match in the tail section. Therefore, a Manning’s n of 0.090 was applied to all runs.

![Figure 4.19 Comparison of Manning’s n values for Grand Goudine](image)

56
The next parameter tested was the contraction and expansion coefficients for bridge sections. The values tested were 0.1 and 0.3 for contraction and expansion coefficients, respectively. Figure 4.20 shows the comparison of water surface elevations computed using the default (0.1 and 0.3 for contraction and expansion, respectively) and suggested values for bridge sections (0.3 and 0.5). The plot altering the values does not significantly alter the overall stage hydrograph. Because the difference between the two runs was insignificant, the default values were used.

![Figure 4.20 Comparison of Altering Contraction and Expansion Coefficients for Bridge Sections on Grand Goudine](image)

Once these parameters were determined, the model was assumed to be calibrated. To ensure that the calibration was justifiable, it was tested against two other known events, October 6, 2006 and September 2, 2008. Using the same methods as discussed in Chapter 3 and in the
same manner as the April 30\textsuperscript{th} event described in Section 3.3.2, the October 6\textsuperscript{th} and September 2\textsuperscript{nd} events were run. Results are presented in Figures 4.21 and 4.22.

There was a great difference between these two events and the corresponding observed data. While the same parameters were used as in the April 30\textsuperscript{th} model, the simulations did not result in as accurate stage hydrographs. The October 6\textsuperscript{th} model shown in Figure 4.21 is shown to have much visual error. While the same general shape is achieved by the simulation, the initial stage was lower than the observed, and the simulated peak was higher. The September 2\textsuperscript{nd} event represented in Figure 4.22 also takes the generalized shape of the observed stage, yet the simulation stage peaks at roughly 1.7 feet higher. The MSE was calculated for both comparison events. The MSE calculated for the October 6\textsuperscript{th} event was 2.19. The MSE for the September 2\textsuperscript{nd} event was 0.526.

![Figure 4.21 Model Comparison Event -- October 6, 2006](image)
If the canals were dredged between the April 30th event and the other two tested events, this would account for the differences in the simulations. Canal dredging (draining) means that the channel could have been deepened, widened and cleaned which would mean that the cross sections used in the HEC-RAS study no longer actually reflected what was happening in the channel. Channel clearing could result in a change in Manning’s n value of the channel. It is probable that East Ascension Drainage Works cleared or mowed Grand Goudine between April and October of 2006 (between the two events in 2006) as these are typical maintenance activities needed in summer. It is also probable that East Ascension Drainage Works dredged Grand Goudine Bayou during this time period, as they were in the midst of performing capital improvements, which includes canal dredging (10 Year Drainage Plan 2007). Second, storm rainfall depths (2.96 inches for the October event, 3.37 inches for the September event and 4.04
inches for the April event) are all measured at a single gage over an almost 9 square mile watershed. This storm could actually have only had this intensity and duration at the gage site and other areas of the watershed did not receive as much rain. HEC-HMS applies rainfall uniformly across the entire watershed. Therefore some subbasins could be overestimating the amount of runoff and therefore overestimating the stage. These explanations apply to both the October and September event.

4.3 Frequency Analysis

After calibrating the model, the known parameters were set for all runs. Both the TP-40 and SRCC methods were modeled with the baseflow of 20 cfs and with a channel Manning’s n value of 0.09. This consistency ensures that the results are directly comparables.

While there is a difference in the estimated rainfall values for the TP-40 method and the SRCC method, results are somewhat similar. Table 4-6 shows the comparison of rainfall depths for the SRCC method and TP-40 method for the 5-, 10- and 100 year storms.

Table 4-6 Rainfall Depth Comparison of SRCC vs. TP-40 Method for 5-, 10-, and 100-yr Storms

<table>
<thead>
<tr>
<th>Storm Event (24 hr)</th>
<th>SRCC Method Value (inches)</th>
<th>TP-40 Method Value (inches)</th>
<th>Percent Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 yr</td>
<td>11</td>
<td>12.5</td>
<td>-13.64</td>
</tr>
<tr>
<td>10 yr</td>
<td>7.0</td>
<td>8.5</td>
<td>-21.43</td>
</tr>
<tr>
<td>5 yr</td>
<td>6.0</td>
<td>7.0</td>
<td>-16.67</td>
</tr>
</tbody>
</table>

Table 4-7 shows the comparison of outflows for all four subbasins in Grand Goudine Bayou. As is expected, the SRCC method yields lower outflows than the TP-40 method.

Figures 4.23, 4.24, and 4.25 are a series of graphs that show the comparison of the simulated and stages for the methods. The stage levels indicated in these figures is measured at the outlet of Grand Goudine Bayou. Visually, it is apparent that there is some difference between the two
methods. The difference is seemingly minimal and from Table 4-8 the WSEs for each method are shown at four important areas: Junction J20 (where W1 enters), the gage site, Junction J10 (where W2 and W3 enter), and Junction JOutlet.

Table 4-7 Flow Comparison of SRCC vs. TP-40 Method for 5-, 10-, and 100-yr Storms

<table>
<thead>
<tr>
<th>Return Period</th>
<th>SRCC Method Flow (cfs)</th>
<th>TP-40 Method Flow (cfs)</th>
<th>Percent Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 yr</td>
<td>W1</td>
<td>1169.5</td>
<td>1371.5</td>
</tr>
<tr>
<td></td>
<td>W2</td>
<td>733.8</td>
<td>857.9</td>
</tr>
<tr>
<td></td>
<td>W3</td>
<td>1058.5</td>
<td>1229.9</td>
</tr>
<tr>
<td></td>
<td>W4</td>
<td>1753.0</td>
<td>2037.8</td>
</tr>
<tr>
<td>10 yr</td>
<td>W1</td>
<td>641.9</td>
<td>836.9</td>
</tr>
<tr>
<td></td>
<td>W2</td>
<td>408.2</td>
<td>528.9</td>
</tr>
<tr>
<td></td>
<td>W3</td>
<td>604.5</td>
<td>773.8</td>
</tr>
<tr>
<td></td>
<td>W4</td>
<td>988.2</td>
<td>1279.8</td>
</tr>
<tr>
<td>5 yr</td>
<td>W1</td>
<td>515.1</td>
<td>680.5</td>
</tr>
<tr>
<td></td>
<td>W2</td>
<td>329.5</td>
<td>432.1</td>
</tr>
<tr>
<td></td>
<td>W3</td>
<td>493.1</td>
<td>638.2</td>
</tr>
<tr>
<td></td>
<td>W4</td>
<td>812.6</td>
<td>1054.2</td>
</tr>
</tbody>
</table>

Table 4-8 Maximum Water Surface Elevation Comparison of SRCC vs. TP-40 Method for 5-, 10-, and 100 yr Storms

<table>
<thead>
<tr>
<th>Return Period</th>
<th>SRCC Method Stage (ft)</th>
<th>TP-40 Method Stage (ft)</th>
<th>Percent Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 yr</td>
<td>J20 (XS 19165)</td>
<td>15.69</td>
<td>16.16</td>
</tr>
<tr>
<td></td>
<td>Gage(XS 14214)</td>
<td>12.42</td>
<td>12.91</td>
</tr>
<tr>
<td></td>
<td>J10 (XS 7304)</td>
<td>12.34</td>
<td>12.84</td>
</tr>
<tr>
<td></td>
<td>Joutlet (XS 0)</td>
<td>12.22</td>
<td>12.73</td>
</tr>
<tr>
<td>10 yr</td>
<td>J20 (XS 19165)</td>
<td>14.52</td>
<td>15.07</td>
</tr>
<tr>
<td></td>
<td>Gage(XS 14214)</td>
<td>10.85</td>
<td>11.50</td>
</tr>
<tr>
<td></td>
<td>J10 (XS 7304)</td>
<td>10.79</td>
<td>11.43</td>
</tr>
<tr>
<td></td>
<td>Joutlet (XS 0)</td>
<td>10.67</td>
<td>11.30</td>
</tr>
<tr>
<td>5 yr</td>
<td>J20 (XS 19165)</td>
<td>13.99</td>
<td>14.64</td>
</tr>
<tr>
<td></td>
<td>Gage(XS 14214)</td>
<td>10.32</td>
<td>11.00</td>
</tr>
<tr>
<td></td>
<td>J10 (XS 7304)</td>
<td>10.26</td>
<td>10.93</td>
</tr>
<tr>
<td></td>
<td>Joutlet (XS 0)</td>
<td>10.14</td>
<td>10.81</td>
</tr>
</tbody>
</table>
Comparing stage and flow results with the SRCC method and the TP-40 method rainfall depths showed that the SRCC method consistently yielded lower results than the TP-40 method. The flow values suggest that the SRCC method and TP-40 method yield flow values that are more similar for higher return periods as compared to lower return periods where the difference between the two methods is greater. The difference in the resulting water surface elevations is minimal, at a maximum difference of 0.75 feet. This small difference indicates that there is little overall affect of choosing one method over another. It should be noted, however, that in terms of floodplain mapping, Base Flood Elevations (BFEs), which are determined from water surface elevations, are measured to the nearest foot. This fact means that depending on the BFE value, the variance between the SRCC method and TP-40 method could affect the overall floodplain map, resulting in higher BFEs for stream reaches.

Figure 4.23 Stage Comparison of SRCC vs. TP-40 Methods at the Outlet (XS 0) for the 5 yr Storm
Figure 4.24 Stage Comparison of SRCC vs. TP-40 Methods at the Outlet (XS 0) for the 10 yr Storm

Figure 4.25 Stage Comparison of SRCC vs. TP-40 Methods at the Outlet (XS 0) for the 100 yr Storm
Chapter 5 - Summary and Recommendations

5.1 Summary

Hydrologic and hydraulic analyses were performed for Grand Goudine Bayou in Ascension Parish, Louisiana. Hydrologic analysis of Grand Goudine Bayou involved selecting an appropriate transform method that adequately modeled outflow. Three transform methods, the Snyder method, the SCS method and the CUH method, were tested. The resulting outflows from each method were then used in an unsteady hydraulic model of Grand Goudine Bayou. Gage data were used to validate the hydrologic and hydraulic analyses.

While all three transform methods are widely used, only one seemed to accurately represent the direct runoff flows for the low slope and low elevation in the Grand Goudine watershed. While there was no direct way to calibrate the flows and ensure that the hydrologic model was accurate, the outflow hydrographs were somewhat validated in the hydraulic model by comparing water surface elevations to gage data. This validation also encompassed watershed characteristics, including the watershed baseflow and Manning’s n value. Because three variables were being validated at once, iteration was necessary. Comparing the transform methods based upon the calibrated results showed that the Snyder method often overestimated peak flow for all watersheds, that the SCS method also overestimated peak flow and that the CUH method provided reasonable outflows. Because the CUH method accounts for storage within the watershed, the overall peak flow was lessened, which contributed to the simulated water surface elevation more adequately resembling the observed water surface elevation.

Two frequency/magnitude determination methods were analyzed. While analysis revealed that there is some difference between the SRCC method and the TP-40 method, it fails
to show enough difference to determine any sound judgment regarding use of one method over
the other.

5.2 Future Recommendations

Model calibration is important, as it ensures that results are reasonable. For a calibration
to be performed, accurate data must be abundant. The calibration techniques provided for this
study were developed in a HEC-RAS model and therefore became dependent on three separate
variables. If the hydrologic model must be validated by means of the hydraulic model, then
adequate data regarding the channel’s natural characteristics should be known, e.g., knowing
whether or when Grand Goudine Bayou had last been dredged would be helpful in understanding
differences between simulated and observed values.

It would be more useful to calibrate the hydrologic model directly. This flow calibration
would require one of two things: gage data and/or regression equations. To more adequately
predict flows, more gages would be necessary. These gages would need to report flow instead of
just stage, like the USGS gage on Grand Goudine. Regression equations would be useful tools in
determining estimated flow values. These equations could help by showing reasonable flow
values for a watershed in a specific region with a specific watershed area.

Other potential work could be based around rainfall data collection. For this study,
rainfall data were collected for the three known events for calibration and validation. Collected
gage data were scoured to find a storm event that had duration of 24 hours and no longer.
Coupled with this requirement, this event could not be led or followed immediately by another
event. Furthermore, it was determined that the storm event be considered significant. These
requirements significantly reduced the amount of available data that could be considered for this
study. With more data available now, perhaps better storm events could be selected; storm
events occurring in or near the same month, which would reduce the number of potential errors
associated with artificial changes in the channel. These, along with the other recommendations,
could offer more insight into this study.
References


East Ascension Drainage Works Telephone Interview. 30 Mar. 2010.


(climod.srcc.lsu.edu)

(seamless.usgs.gov).

(soildatamart.nrcs.usda.gov)
Appendix A – 24-hour Frequency/Magnitude Atlas Maps for the SRCC Method

Figure A.1 24-Hour, 2-year Recurrence Frequency/Magnitude Atlas for SRCC Method

Figure A.2 24-Hour, 5-year Recurrence Frequency/Magnitude Atlas for SRCC Method
Figure A.3 24-Hour, 10-year Recurrence Frequency/Magnitude Atlas for SRCC Method

Figure A.4 24-Hour, 25-year Recurrence Frequency/Magnitude Atlas for SRCC Method
Figure A.5 24-Hour, 50-year Recurrence Frequency/Magnitude Atlas for SRCC Method

Figure A.6 24-Hour, 100-year Recurrence Frequency/Magnitude Atlas for SRCC Method
Appendix B - 24-hour Frequency/Magnitude Atlas Maps for the TP-40 Method

Figure B.1 24-hour, 2-year Frequency/Magnitude Atlas for TP-40 Method

Figure B.2 24-hour, 5-year Frequency/Magnitude Atlas for TP-40 Method
Figure B.3 24-hour, 10-year Frequency/Magnitude Atlas for TP-40 Method

Figure B.4 24-hour, 25-year Frequency/Magnitude Atlas for TP-40 Method
Figure B.5 24-hour, 50-year Frequency/Magnitude Atlas for TP-40 Method

Figure B.6 24-hour, 100-year Frequency/Magnitude Atlas for TP-40 Method
Appendix C – SCS Method Rainfall Distributions

Figure C.1  Approximate SCS Rainfall Distributions Map (U.S. Department of Agriculture 1986)
Appendix D – CUH Method Ratio Calculation

\[ 0.6 \times T_c = K_1 \]
\[ 2.0 \times T_c = K_2 \]

\[ \text{Ratio}_1 = \frac{K_1}{T_c + K_1} \]
\[ \text{Ratio}_2 = \frac{K_2}{T_c + K_2} \]
\[ \text{Ratio}_1 < \text{Ratio} < \text{Ratio}_2 \]

\[ \text{Ratio} = \frac{R}{T_c + R} \]

\[ R = \frac{\text{Ratio} \times T_c}{1 - \text{Ratio}} \]

Where, \( R \) is storage Coefficient, \( T_c \) is Time of Concentration and \( K_1 \) and \( K_2 \) are variables.
## Appendix E – Manning’s n Values for Natural Streams

### Table E-1 Manning’s n Values for Natural Streams

<table>
<thead>
<tr>
<th>Type of Channel and Description</th>
<th>Minimum</th>
<th>Normal</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Natural Streams</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>1. Minor streams (top width at floodstage &lt; 100 ft)</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a. Clean, straight, full stage, no rifts or deep pools</td>
<td>0.025</td>
<td>0.030</td>
<td>0.033</td>
</tr>
<tr>
<td>b. Same as above, but more stones and weeds</td>
<td>0.030</td>
<td>0.035</td>
<td>0.040</td>
</tr>
<tr>
<td>c. Clean, winding, some pools and shoals</td>
<td>0.033</td>
<td>0.040</td>
<td>0.045</td>
</tr>
<tr>
<td>d. Same as above but some weeds and stones</td>
<td>0.035</td>
<td>0.045</td>
<td>0.050</td>
</tr>
<tr>
<td>e. Same as above, lower stages, more ineffective slopes and sections</td>
<td>0.040</td>
<td>0.048</td>
<td>0.055</td>
</tr>
<tr>
<td>f. Same as &quot;d&quot; with more stones</td>
<td>0.045</td>
<td>0.050</td>
<td>0.060</td>
</tr>
<tr>
<td>g. Sluggish reaches, weedy, deep pools</td>
<td>0.050</td>
<td>0.070</td>
<td>0.080</td>
</tr>
<tr>
<td>h. Very weedy reaches, deep pools or floodways with heavy stand of timber and underbrush</td>
<td>0.075</td>
<td>0.100</td>
<td>0.150</td>
</tr>
<tr>
<td><strong>2. Mountain Streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a. bottom: gravels, cobbles, and a few boulders</td>
<td>0.030</td>
<td>0.040</td>
<td>0.050</td>
</tr>
<tr>
<td>b. bottom: cobbles with large boulders</td>
<td>0.040</td>
<td>0.050</td>
<td>0.070</td>
</tr>
<tr>
<td><strong>3. Floodplains</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a. Pasture, no brush</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. short grass</td>
<td>0.025</td>
<td>0.030</td>
<td>0.035</td>
</tr>
<tr>
<td>2. high grass</td>
<td>0.030</td>
<td>0.035</td>
<td>0.050</td>
</tr>
<tr>
<td>b. Cultivated areas</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. no crop</td>
<td>0.020</td>
<td>0.030</td>
<td>0.040</td>
</tr>
</tbody>
</table>
2. mature row crops  |  0.025  |  0.035  |  0.045  \\
3. mature field crops |  0.030  |  0.040  |  0.050  \\

c. Brush

1. scattered brush, heavy weeds  |  0.035  |  0.050  |  0.070  \\
2. light brush and trees, in winter |  0.035  |  0.050  |  0.060  \\
3. light brush and trees, in summer |  0.040  |  0.060  |  0.080  \\
4. medium to dense brush, in winter |  0.045  |  0.070  |  0.110  \\
5. medium to dense brush, in summer |  0.070  |  0.100  |  0.160  \\

d. Trees

1. Dense willows, summer, straight |  0.110  |  0.150  |  0.200  \\
2. Cleared land with tree stumps, no sprouts |  0.030  |  0.040  |  0.050  \\
3. Same as above, but with heavy growth of sprouts |  0.050  |  0.060  |  0.080  \\
4. Heavy stand of timber, a few down trees, little undergroth, flood stage below branches |  0.080  |  0.100  |  0.120  \\
5. Same as 4, with flood stage reaching branches |  0.100  |  0.120  |  0.160  \\

4. Major streams (top width at flood stage > 100 ft)

a. Regular section with no boulders or brush |  0.025  |  .....  |  0.060  \\
b. Irregular and rough section |  0.035  |  .....  |  0.100  
(Chow 1959)
Vita

Jessica Mason was born in Oxford, Mississippi, in January of 1984 to Gary and Debra Mason. She attended the University of Mississippi in Oxford, Mississippi, for her undergraduate education and received a Bachelor of Science in Civil Engineering degree in May, 2007. In August, 2007, she entered graduate school at Louisiana State University. While attending LSU, she worked on a hydrologic study of Ascension Parish sponsored by the East Ascension Drainage Board. She has made presentations at American Water Resources Association conferences regarding the Incorporating of Coastal Storms in a Southern Louisiana Parish and assisted with Application of Accurate Vertical Controls to LIDAR DEMs and the Impact on Hydrologic Modeling in Flat Landscapes. She expects to receive a Master of Science in Civil Engineering in May, 2011.