Hydrologic Analysis of Peak Stage Measurements in Church Creek at Crosstowne Christian Church

Prepared for
Crosstowne Christian Church

Final Report

January 2019

Hydrologic Analysis of Peak Stage Measurements in Church Creek at Crosstowne Christian Church

Prepared for

Crosstowne Christian Church

Prepared by

Joshua L. Robinson, MS, PE
Nolan C. Williams, EIT
Philip A. Ellis, MS, PE
Robinson Design Engineers

and

Anand D. Jayakaran, PhD, PE

January 2019

Robinson Design Engineers
10 Daniel Street, Charleston, SC 29407
www.robinsondesignengineers.com

ROBINSON DESIGN ENGINEERS[SM]
Land.Water.Ecology.

Hydrologic Analysis of Peak Stage Measurements in Church Creek at Crosstowne Christian Church

Prepared for

Crosstowne Christian Church

SOUTH CAROLINA
ROBINSON DESIGN ENGINEERS
No. 4593
CERTIFICATE OF AUTHORIZATION

SOUTH CAROLINA
PROFESSIONAL ENGINEER
REGISTERED
No. 26831
JOSHUA L. ROBINSON

1-14-19

ROBINSON DESIGN ENGINEERS[SM]
Land.Water.Ecology.

TABLE OF CONTENTS

EXECUTIVE SUMMARY 5

1. INTRODUCTION 6
   1.1. Project Description 6
   1.2. Study Area 7
   1.3. Background 8
   1.4. Previous Studies 8

2. DATA SOURCES 12
   2.1. Continuous Water Level Data 12
   2.2. Flood Crest Elevation Data 13
   2.3. Rainfall Data 14

3. ANALYSIS & RESULTS 15
   3.1. Bees Ferry vs. Crosstowne Stage 15
   3.2. Ashley River Stage vs. Church Creek Stage 15
   3.3. Rainfall Averaging 18
   3.4. Rainfall vs. Peak Stage 19
   3.5. Stage Increase Due to Rainfall 21
   3.6. Compound Flooding Function 22

4. CONCLUSIONS & RECOMMENDATIONS 24
   4.1. Stormwater Flooding 24
   4.2. Compound Flooding 24
   4.3. Future Land Development 25
   4.4. Assumptions & Limitations 25
   4.5. Future Work 26

5. REFERENCES 27

FIGURES 30

LIST OF EQUATIONS

Equation 1: Dry Weather Regression Equation Relating Peak WSE in the Ashley River and Peak WSE in Church Creek **17**

Equation 2: Inverse Distance Weighted Average Formula for Daily Rainfall Totals **19**

Equation 3: Additional Stage Function for Relating 48-hour Cumulative Rainfall Total and Observed – Predicted WSE in Church Creek **22**

Equation 4: Compound Flooding Function for Predicting Church Creek WSE **23**

LIST OF TABLES

Table 1: Example Derivation of Cumulative Rainfall Totals for an Observed Peak Water Surface Event on 08/31/2015. **20**

EXECUTIVE SUMMARY

This study was conducted by Robinson Design Engineers, in collaboration with Anand Jayakaran, PhD, PE, on behalf of Crosstowne Christian Church in Charleston, SC. Crosstowne Christian Church is located along Church Creek, a heavily hydromodified tidal creek system that is part of the Ashley River basin. Since 2015, Crosstowne has suffered three separate catastrophic flooding events, each of which inflicted hundreds of thousands of dollars in water damage and required the congregation to temporarily relocate. A fourth event produced flood levels within an inch of the building. Prior to 2015, the Crosstowne building had not been flooded. In response to the sudden and recurring flooding problem, Crosstowne retained Robinson Design Engineers in the Spring of 2018 to conduct an independent hydrologic analysis of the flood events, the probability of their occurrence, and the likely source or sources of flooding. Robinson Design Engineers installed a water level gage in Church Creek at Crosstowne and performed a series of statistical analyses of data collected at the new gage and data collected by federal agencies at other gaging stations. These analyses reveal the isolated effects of rainfall and Ashley River tide levels on the peak stage of Church Creek at Crosstowne. This report presents a “Compound Flooding Function” that predicts the peak water surface elevation of Church Creek at Crosstowne based upon a 48-hour rainfall total and the peak tide level in the Ashley River.

This study reveals that under current hydrologic conditions, Crosstowne has approximately 4 percent annual chance of flooding (“25-year” recurrence interval) by the combined effects of rainfall and tide, and between 2 and 3 percent annual chance of flooding (“50-year” to “33-year” recurrence interval) by rainfall only. These analyses and predictive tools strongly suggest that the flooding at Crosstowne is much more influenced by rainfall than by tidal surge. Storm event data reveal that peak water surface elevation of Church Creek at Crosstowne is strongly correlated to the 48-hour rainfall total, but weakly correlated to the 24-hour rainfall total. Furthermore, from the start of rainfall, the time-to-peak is usually greater than 24 hours.

This study confirms that any land development activities in the Church Creek basin will likely increase the flood frequency and flood depth at Crosstowne Christian Church. In addition, this study reveals that current City of Charleston stormwater design standards, which are based on the 24-hour storm and do not include significant volume retention requirements, are insufficient and will not safeguard Crosstowne from additional flooding.

Previous hydrologic studies of the basin have been based on computer simulations that were neither calibrated nor properly verified with field measurements. As a result, these studies are potentially inaccurate and even misleading. Future flood mitigation activities in the watershed that are based on these studies will likely not reflect true flooding conditions at Crosstowne Christian Church. Furthermore, the results of this study indicate that tidal surge protection measures would only slightly reduce the likelihood of flooding at Crosstowne.

1. INTRODUCTION

1.1 Project Description

Crosstowne Christian Church is situated along a channelized segment of Church Creek in suburban Charleston, South Carolina. In the past three years, Crosstowne has suffered three separate catastrophic flooding events, each of which has inflicted hundreds of thousands of dollars in water damage and required the congregation to temporarily relocate. Each of the tropical storms—Hurricanes Joaquin (2015), Matthew (2016), and Irma (2017)—delivered a combination of heavy rainfall and tidal surge.

The Church Creek basin has been fraught with flooding for many years. While the Crosstowne building has flooded three times in recent years, on many other occasions Church Creek has flooded the Crosstowne site and water levels have come within inches of the building. Although these major floods have most often been associated with tropical systems, the Creek poses a flooding threat during any heavy rainfall event. Since the late 1990's, the City of Charleston has been aware of the flood risks and has undertaken a series of modeling studies to guide their management of the watershed and its permitted land uses.

In early 2018, Crosstowne retained Robinson Design Engineers to perform an independent study of the flood hydrology of Church Creek in the vicinity of its property. Whereas the previous and on-going basin-wide hydrology studies and flood studies conducted by others have been based on predictive models, Crosstowne requested that Robinson Design Engineers perform a field-based study to characterize existing hydrology and to establish a "baseline" of the current flow regime. In particular, the goals of this study were to 1) provide a better understanding of tidal influence on flooding at Crosstowne, and 2) determine whether future land development in the basin could worsen flooding at Crosstowne.

This report presents the work and findings of Robinson Design Engineers, and documents the present hydrologic behavior of Church Creek in the vicinity of Crosstowne based upon data collected from October 2015 to October 2018. This information is presented with the goal of informing Crosstowne as they face difficult decisions with regard to their future flooding potential, on-going land development activities in the watershed, and future flood management efforts by the City of Charleston.

1.2 Study Area

Church Creek is a tidal tributary of the Ashley River in Charleston, South Carolina. In its current state, the waterway drains between 8.5 square miles (Woolpert, 2001) and approximately 15.9 square miles (Weston & Sampson, 2017) of the suburban community of West Ashley. Shallow groundwater and the low relief of the landscape and make it difficult to determine the actual contributing drainage area. Figure 1.1 is a current map of the Church Creek watershed and the surrounding areas. Near its confluence with the Ashley River, Church Creek exhibits the characteristics of a tidal creek and salt marsh typical of the region. Approximately 4.3 miles upstream of the confluence, the Creek and its landscape transition to bottomland forest and freshwater wetlands historically known as “Bear Swamp” (USGS, 1919). The historical or natural headwaters of the system have been significantly altered by mining, logging, ditching, and land development over several generations. Direct modifications to the creek and wetlands have included channelization for drainage and navigation, excavation and strip mining, and filling of large portions of the original freshwater and saltwater wetlands. Civil War era maps document an extensive network of navigation channels connecting a series of artillery batteries throughout the basin. Postbellum strip mining significantly altered the network of military canals. The existing network of ditches and canals reached its current state by the mid-1900s during the suburbanization of West Ashley (USGS, 1958).

Historically, Church Creek was hydrologically connected to an adjacent tidal system now known as Long Branch. Long Branch, which drains to the Stono River, is a historic tidal creek with channel and watershed characteristics similar to those of Church Creek. The historic connection between these two creeks provided navigation between the Ashley and Stono Rivers and connected an extensive network of artillery batteries west of the Charleston peninsula. During the development of the two watersheds in the 20th century, the connection between the two Creeks was severed by the construction of a series of embankments that now form a freshwater body known as Lake Dotterer. Annotated historic maps of the Church Creek – Long Branch system from the years 1862, 1919, and 1958 have been included as Figures 1.2, 1.3, and 1.4, respectively.

The specific area of interest for this study is the property of Crosstowne Christian Church, which is located along the eastern bank of a channelized portion of Church Creek. This area of interest is approximately 4.9 miles upstream of the Creek’s confluence with the Ashley River, as measured along the creek. This reach of Church Creek is not part of the historic stream channel, but is part of the channel system that was excavated to drain the bottomland hardwood forests upstream. Crosstowne is fronted by Bees Ferry Road, a four-lane thoroughfare with a bridge crossing over Church Creek directly upstream of the Crosstowne property. Roughly 1,250 feet downstream of Crosstowne, the Creek passes through a collection of several very large culverts beneath an embankment supporting an active rail line. A map of the study area and major points of interest has been included as Figure 1.5.

1.3 Background

In the 1990's, property owners and residents of the Church Creek basin began to experience recurring flooding. Initially, these flood events occurred once every several years and were typically associated with unusually large rainfall events. However, since 2015 the basin has experienced one or more catastrophic flood events each year. The largest events have resulted from tropical cyclone storm systems, but flooding has also occurred in the basin in the absence of tropical storms. The flood-prone nature of the basin has confused and concerned both residents and governing officials for many years, but the catastrophic flood events of the last few years have created a crisis.

The City of Charleston has studied the basin since the early 2000's, and has endeavored to reduce flooding and to appropriately regulate land development activities in the basin. The City has engaged several engineering consulting firms to perform extensive studies of the basin, with deliverables including hydrology and hydraulic models, stormwater design standards, and policy recommendations. However, the models were neither calibrated nor properly verified (as outlined in Sturm, 2001, for example). As a result, the various modeling studies of the basin offer contradictory conclusions, leaving citizens and City officials with no clear path towards resolving the causes or effects of flooding. As a result, despite the numerous recommendations and proposed solutions, little has been implemented. This has led to an increasing public perception that flooding is worsening and that the causes and implications of the problem remain largely unknown.

1.4 Previous Studies

1.4.1 FEMA Flood Insurance Study

Crosstowne is within a Special Flood Hazard Area (SFHA) as defined by the Federal Emergency Management Agency (FEMA) and depicted on the Flood Insurance Rate Map (FIRM) 45019C0480J. The most recent Flood Insurance Study (FIS) is dated 2016 and is currently provisional. The current effective FIS is dated 2004. The 2016 study included new analyses of the coastal flooding hazard.

The FEMA FIS assumes that the most important flooding source in the vicinity of Crosstowne is a tidal surge event. Accordingly, the Base Flood Elevation (BFE) at Crosstowne is based upon the predicted tidal surge in the Ashley River and the resultant wave propagation into the Church Creek basin. The FIS does not include flooding from stormwater runoff; in this sense, the FEMA analysis represents a "dry weather" prediction representing only tidal surge (FEMA, 2004; FEMA, 2016).

1.4.2 City of Charleston: Woolpert Studies

The first hydrology study of the Church Creek Basin was undertaken in late-2000 by Woolpert, an engineering consultant hired by the City of Charleston. The purpose of this original study, entitled "Church Creek Stormwater Master Plan, was to "1) identify existing stormwater flooding problems, 2) analyze and recommend potential solutions to those identified problems, 3) analyze flooding impacts due to future development, 4) analyze the current stormwater detention requirements, and 5) determine if land use restrictions or modifications to the detention requirements should be made" (Woolpert, 2001). The report states that the study was initiated by the City of Charleston in response to recurring stormwater flooding that began in the 1990's as the basin began to undergo rapid residential development.

The primary product of the initial Woolpert study was a basin-wide model created using the Interconnected Channel and Pond Routing (ICPR) modeling software (Woolpert, 2001). ICPR is a coupled hydrology and hydraulic model that operates using a network of connected "links" and "nodes" that are created with user-defined data representing elements including pipes, ponds, natural channels, etc. These links and nodes are used to route stormwater runoff hydrographs based on the characteristics of user-defined sub-basin elements.

To build the model, the consulting team conducted a range of hydrologic analyses including: identifying the extent of the Church Creek watershed, delineating sub-catchments within the watershed, estimating the distribution of different soil types, and calculating the distribution of various forms of land cover throughout the watershed. These analyses, along with survey data and field observations, were then used to estimate Time of Concentration and Runoff Curve Number for each of the sub-catchments within the basin according to the NRCS TR-55 methodology (Cronshey et al, 1983). To simulate hypothetical rainfall events, 24-hour design storms ranging in magnitude from a 2-year to a 500-year return period were created according to the SCS Type III Rainfall Distribution. Along with the results of the TR-55 calculations, these design storms formed the basis of the hydrology component of the ICPR model. Hydraulic elements and their properties were based on survey data, field observations, and various maps relating to the dimensions and characteristics of channels and stormwater control measures throughout the basin.

The modeling domain was defined as the basin from its inland headwaters to the US Hwy. 61 bridge crossing of Church Creek. The downstream hydraulic boundary condition was defined by a time series of tailwater elevations intended to represent a large tidal cycle. These tailwater elevations were estimated by projecting the MHHW and MLW levels in the Ashley River to the extents of the modeling domain near the mouth of Church Creek. The time series of the tailwater boundary condition was set such that the maximum water level in Church Creek at the railroad crossing would occur approximately at the same time as peak rainfall from the design storm would occur. This was done in attempt to simulate the maximum possible effects of backwater conditions at the culvert crossing under the railroad.

Once constructed, the ICPR model was used to simulate various design storms to assess the extent of flooding based on the existing watershed conditions. The ICPR model results were checked against the FEMA Flood Insurance Study, other stormwater studies within the basin, and USGS regression equation flows. However, the ICPR model was neither calibrated nor verified using measured storm event data. In fact, when the model was used to simulate actual flood events, the results under-predicted flood elevations reported by residents. These inherent limitations were described clearly in section "5.1.4 Historical Structure Flooding" (Woolpert, 2001), which states:

> "In order to produce similar flooding in the Shadowood location, parameters had to be adjusted to unreasonable values and in return would produce unreasonable water surface elevations downstream. Therefore, it is assumed that other factors not included in the ICPR model contribute to flooding in the Shadowood area."

Woolpert then used the model to test a series of conceptual solutions and to evaluate the existing stormwater management design standards for new development in the basin.

An important conclusion of Woolpert's 2001 Master Plan Report is that flooding in the Church Creek basin is primarily caused by stormwater runoff, not tidal effects. Based on their analysis, Woolpert determined that the existing stormwater management design standards were insufficient. Woolpert recommended a new set of standards, including a requirement that new developments should control peak outflow rates and should detain excess runoff volumes for at least 24 hours.

Throughout the 2000's and 2010's, Woolpert periodically updated the watershed ICPR model as new developments were completed, new surveys were conducted, etc. In 2015, Woolpert undertook a major revision and update of the model to reflect several large-scale infrastructure projects completed in the basin, most notably the widening of Bees Ferry Road and the construction of the West Ashley Traffic Circle. Around the time that the report accompanying the update was published, Tropical Storm Erika and Hurricane Joaquin produced multi-day storm events in the region. Both of these storms caused significant flooding in the Church Creek basin.

Following these events, Charleston County and the City of Charleston requested that Woolpert run the updated model to simulate the recent storm events for two reasons: 1) to verify that the infrastructure projects described above had not increased flood elevations, and 2) to determine the influence of tidal surge on the flood events. Based on the results of the model, Woolpert confirmed that neither of the major infrastructure projects had contributed to flooding and that "higher tides did not contribute to the structural flooding in the Church Creek Watershed" (Woolpert, 2016).

1.4.3 City of Charleston: Weston & Sampson Study

In late 2016, Charleston was impacted by another large tropical storm, Hurricane Matthew, which caused widespread flooding in the Church Creek basin. As a result, in early 2017 the City of Charleston retained another engineering consultant, Weston & Sampson, to conduct an independent assessment and study of the basin. Weston & Sampson was tasked with reviewing the previous studies, conducting their own analyses, and developing an updated set of recommendations to reduce flooding in the basin (Weston & Sampson, 2017). In late 2017, while the study was underway, the Church Creek basin was yet again impacted by widespread, catastrophic flooding resulting from Hurricane Irma.

One of the primary outcomes of the Weston & Sampson study was an updated delineation of the Church Creek watershed based on a Digital Elevation Model (DEM) derived from LiDAR collected by the South Carolina Department of Natural Resources between 2007 and 2009 (Weston & Sampson, 2017). Using the DEM, Weston & Sampson determined that the area draining to Church Creek was much larger than had been assumed in previous studies. Earlier estimates of the drainage area were approximately 8.5 square miles, while Weston & Sampson estimated the area to be approximately 15.9 square miles. Another outcome of the study was upgrading the existing ICPR model of the basin to the latest version of the software, ICPR4. As part of these updates, Weston & Sampson added additional sub-catchments to represent the expanded watershed, and updated model elements to reflect recent changes in the watershed.

Using the new model, Weston & Sampson simulated various storm events and correlated the model results to their field observations with "acceptable accuracies." The model was used to simulate the August 2015 event, and predicted water levels within 0.2 feet of the observed water levels at the Bridge Point townhomes (Weston & Sampson, 2017). The report does not provide any other documentation of model calibration, verification, or comparison with previous studies of the basin.

Weston & Sampson then used the updated ICPR model to test a series of conceptual solutions. Their final recommendations include a stormwater pump station, tidal protection measures, and additional surface storage areas throughout the basin. An important implication of the study is that, in the opinion of Weston & Sampson, flooding in the basin is strongly linked to tidal surge. This implication is included throughout the report but appears most evidently in the "Recommendations for Structural Improvements" section, which states: "It is important to note here that surge protection is a necessity when any scenario is used that includes pumping within the basin" (Weston & Sampson, 2017).

2. DATA SOURCES

As introduced previously, Crosstowne Christian Church charged Robinson Design Engineers with conducting a field-based study of measured hydrology data. The hydrology data used in this study included measurements of water levels at multiple locations throughout the basin and in the Ashley River, rainfall totals in the vicinity of Church Creek basin, and peak stage data from flood events in the basin.

2.1 Continuous Water Level Data

2.1.1 Charleston Harbor

The most downstream point in the larger Church Creek / Ashley River system is the Charleston Harbor. The United States National Oceanographic and Atmospheric Administration (NOAA) operates the Charleston Harbor tidal gage, Station 8665530 Charleston, Cooper River Entrance, SC. This gage has been in continuous operation since 1899 and is the longest running and most continuous source of water level data for the Charleston Harbor. Data for periods of interest were downloaded via the station's online portal for historic water levels. The downloaded water level data are referenced to the North American Vertical Datum of 1988 (NAVD88) and were provided at a 15-minute interval.

2.1.2 Ashley River at I-526

The United States Geologic Survey (USGS) operates a gage several miles upstream of the Charleston Harbor, USGS 021720869 Ashley River Near North Charleston, SC. This gage was installed in October of 2007, and data were downloaded through the USGS online portal for accessing historic data. Water level data for this gage are presented as a "gage height," which was converted to a water surface elevation according the National Geodetic Vertical Datum of 1929 (NGVD29) given the published elevation of the gage site. Water level data collected from this gage were provided at a 15-minute time interval. Data from this gage are particularly relevant because the gage is located just downstream of the mouth of Church Creek as it empties into the Ashley River. Because of this close proximity, data from this gage effectively represent water level at the mouth of Church Creek. Additionally, this gage location provides insight on the timing and magnitude of tidal propagation upriver from the Harbor.

2.1.3 Church Creek at Bees Ferry Road

The upstream-most point of data collection in the Church Creek system is at the Bees Ferry Road bridge crossing directly upstream of Crosstowne. At this location the National Weather Service (NWS) and Woolpert, a consultant for the City of Charleston, operate a stream gage on the Creek. The BEES1 gage was installed in early 2017 and is operated primarily as an early flood warning tool. Robinson Design Engineers began downloading real-time data from the gage's online portal beginning in June of 2018. In addition, the online portal provides a record of the 10 highest water levels measured by the gage. Water levels for this gage are presented as a water surface elevation in reference to NGVD29. Measurements collected at this gage were recorded at a five-minute interval.

2.1.4 Church Creek at Crosstowne Christian Church

In June 2018, Robinson Design Engineers installed a gage station on Church Creek at Crosstowne Christian Church. This station is equipped with a HOBO MX2001 pressure transducer and data logger, and is located approximately 330 feet downstream of the NWS-Woolpert gage at Bees Ferry Road. Due the close proximity of the two gages, the Robinson Design Engineers gage was primarily used for verifying water level measurements observed at the Bees Ferry gage and for collecting data during periods in which the Bees Ferry gage was not operating, or data were unavailable. Initially, Robinson Design Engineers set the recording interval at this gage as five-minutes, but later reduced the collection interval to fifteen minutes. Figure 2.1 is a map of water level data collection points.

2.2 Flood Crest Elevation Data

In addition to the data sets of continuous water levels, a key component of this study was incorporating data relating to historic events that have flooded Crosstowne into the analyses. The building that Crosstowne occupies was built in 1998. When constructed the finished floor elevation (FFE) of the Crosstowne building was set at 8.10 NGVD29 to match the FFE of an adjacent building constructed in 1985. Neither of the buildings experienced flooding until 2015. Since then, the building has been affected by flooding three times and has been within approximately one inch of flooding in another instance. The near flood event occurred in August of 2015 in conjunction with Tropical Storm Erika. The three instances where the Church has flooded occurred in October of 2015, October of 2016, and September of 2017. These flood events are associated with Hurricane Joaquin, Hurricane Matthew, and Hurricane Irma, respectively.

The USGS Ashley River and NOAA Cooper River Entrance gages measured and recorded water level data for the large magnitude events that have caused flooding at Crosstowne. However, because the NWS-Woolpert gage at Bees Ferry and the Robinson Design Engineers gage at Crosstowne were installed in early-2017 and mid-2018, respectively, there are no gage data available for Church Creek in the immediate area of Crosstowne for the flooding events associated with Tropical Storm Erika and Hurricanes Matthew and Joaquin. As a result, peak water levels associated with these flood events were derived from measurements of high-water marks in or near the Crosstowne building in relation to the FFE. These peak water levels were then converted to water surface elevations using the Church's known FFE. A peak WSE in Church Creek was recorded at the BEES1 gage for Hurricane Irma.

2.3 Rainfall Data

2.3.1 National Weather Service Rain Gages

The two closest reliable sources of rainfall data in the proximity of the study area are a pair of gages operated by the National Weather Service: KCHS at Charleston International Airport and KCXM in Downtown Charleston. The primary data collected from these two gages were daily precipitation totals which were accessed and downloaded through online gage records maintained by the NWS Charleston Weather Forecast Office. A map showing the location of these two gages relative to the position of Crosstowne can be seen in Figure 2.2.

2.3.2 NWS-Woolpert Gage

In addition to the two primary NWS rain gages, the NWS-Woolpert gage at Bees Ferry measures and records precipitation data. However, because the gage was installed in early-2017, no prior precipitation data are available. Furthermore, records for this gage are only publicly accessible through NWS/NOAA online resources for a limited time (approximately one week) after which they are published. As a result, precipitation data from this gage were collected beginning in June 2018.

2.3.3 CoCoRAHS

The data from the NWS gages were the only rainfall data which were used in conducting the actual hydrologic analysis of the system; however, rainfall data were also collected from Community Collaborative Rain, Hail, and Snow Network (CoCoRAHS) gages in the area of Crosstowne. Many of the gages that are part of the CoCoRAHS Network are owned and operated by nearby community members at their residences. Because the accuracy of these community gages are unknown and their measurements are difficult to verify, these data were not used in any quantitative analyses. However, the data collected from these gages were used for qualitative comparisons between daily rainfall totals at gages in the immediate vicinity of Crosstowne and daily rainfall totals obtained from the official NWS gages.

2.3.4 Precipitation Frequency Data

Statistical rainfall data for a range of rainfall depths, storm event durations, and recurrence intervals were downloaded from NOAA's Precipitation Frequency Data Server. These data are derived for Crosstowne's exact location based on extensive precipitation records and statistical relationships established by NOAA (Bonnin et al., 2006).

3. ANALYSIS & RESULTS

The objective of the analysis was to ascertain linkages between the hydrologic drivers (rainfall and tidal inflow) and resulting change in water levels along Church Creek in the vicinity of Crosstowne. In particular, the analysis aimed to isolate the effects of rainfall from the effects of tidal inflow on Church Creek stage at Crosstowne.

3.1 Bees Ferry vs. Crosstowne Stage

As previously described, the National Weather Service and Woolpert operate a gage station at the Bees Ferry Road bridge crossing, just upstream of Crosstowne. A short distance downstream, the Robinson Design Engineers gage station collects duplicate water level data. To verify the similarity of the two data sets, measurements recorded by the Robinson Design Engineers gage were compared to measurements recorded at the NWS-Woolpert gage. The intent of this comparison was to establish the equivalence of the stage data at these two gage stations. By establishing this relationship, data from either gage could be used in the event that either of the gages malfunctioned. The importance of this duplicate data collection is revealed by Figure 3.1, which shows that NWS-Woolpert gage malfunctioned and failed to collected data in several instances, especially during periods of high stage.

The data used in this analysis were collected between May 4 and June 12, 2018, with measurements recorded at a five-minute time interval. Figure 3.1 depicts a Creek stage hydrograph measured at each of the gages. In addition, a simple linear regression relationship between the two data sets, as depicted in Figure 3.2, displays a slope of 0.997 with a y-intercept of 0 and a R-squared value of 0.997. The slope, y-intercept, and coefficient of determination collectively suggest that the hypothesis that data from the gages are practically equivalent, is accurate. It should be noted that, although both gages collected water level measurements at a five-minute interval, there was approximately a one-and-a-half-minute time difference between the measurements. Additionally, the two data sets were recorded using different sets of instrumentation, which could have different levels of accuracy and sensitivity. These factors likely account for the negligible variation between measurements at the two gages.

3.2 Ashley River Stage vs. Church Creek Stage

In the vicinity of Crosstowne, Church Creek has been observed to exhibit tidal bidirectional flow and stage fluctuation. Robinson Design Engineers performed a field investigation of the waterway downstream of Crosstowne and confirmed that Church Creek is hydraulically connected to the Ashley River.

Water level data collected from Church Creek at Crosstowne were then compared to water level data from the USGS gage at the I-526 bridge in the Ashley River to explore the wave celerity and change in tidal amplitude as the tidal wave propagates from the Ashley River inland / upstream to Crosstowne (Julien, 2002; Leopold, Collins, & Collins, 1993). This comparison reveals several noteworthy characteristics regarding tidal connectivity between the Ashley River and Church Creek at Crosstowne:

- Wave celerity, when measured as a peak delay from the Ashley River to Church Creek at Crosstowne, ranges from 3 to 8 hours;
- Wave celerity is related to tidal amplitude such that a large tidal amplitude (measured as peak to trough) exhibits a higher wave celerity (i.e. shorter travel time), whereas a small tidal amplitude exhibits a lower wave celerity (i.e. longer travel time);
- Tidal amplitude is significantly diminished as the tidal wave moves inland;
- Significant rain events and resultant stormwater runoff overwhelm the tidal hydrograph at Crosstowne, and the effects of runoff on the hydrograph at Crosstowne can persist for several days.

3.2.1 Peak Stage Comparison

The relationship between Ashley River stage and Church Creek stage was further analyzed by comparing the daily peak water surface elevations, which represent the highest of the semidiurnal tides measured at each gage. However, as shown in Figure 3.3, this comparison exhibits significant scatter with clustered data points.

3.2.2 Wet Weather vs. Dry Weather - Page 43

The peak water surface elevation data set was separated into “wet weather” data sets and “dry weather” data sets to isolate the effects of rainfall-runoff from the effects of tidal influence on the water surface elevation in Church Creek. Data were divided into the dry and wet categories based upon the occurrence of rainfall in the days preceding the observed peak stage event. In particular, if no rainfall had occurred on the day of or on either of the two days leading up to an observed peak, the observation was classified as being a dry weather observation. However, if any rainfall had occurred on the day of or the two days prior to the observed peak, the observation was classified as being a wet weather observation. The dry weather data set is comprised of data collected from the Robinson Design Engineers gage at Crosstowne. The wet weather data set is comprised of data from the Robinson Design Engineers gage at Crosstowne, along with data from the flood crest elevation data collected by the NWS-Woolpert gage at Bees Ferry Road (see section 2.2).

Regression analysis of the dry weather data set exhibits a strong linear relationship (R-squared of 0.84) between peak Ashley River WSE and peak Church Creek WSE as defined below as Equation 1 and shown in Figure 3.4.

Equation 1: Dry Weather Regression Equation Relating Peak WSE in the Ashley River and Peak WSE in Church Creek.

$$y = 0.45x + 0.44$$

where: y = Church Creek Peak WSE (ft, NAVD88)
x = Ashley River Peak WSE (ft, NAVD88)

Similarly, regression analysis of the wet weather data set exhibits a linear relationship (R-squared of 0.69) between peak Ashley River WSE and peak Church Creek WSE. While this relationship is weaker than the dry weather relationship, the wet weather data exhibit two distinct clusters. One data cluster represents the large magnitude tropical events and the other cluster represents the low- to average-magnitude storm events.

It should be noted in Figure 3.4 that both the dry weather and wet weather linear regression equations exhibit a y-intercept of 0.44. Theoretically, this y-intercept represents the peak WSE in feet (NAVD88) in Church Creek if the peak WSE in the Ashley River were 0 feet (NAVD88). Physically. This value likely represents the baseflow WSE in Church Creek at Crosstowne, and is non-zero because of the channel gradient from the Ashley River to the Crosstowne gaging station.

3.2.3 Dry Weather + FEMA Model Results

As shown in Figure 3.4 when the data points derived from the Existing and Provisional FEMA FIRMs are included as part of the dry weather data set, the linear regression improves markedly while exhibiting a very similar slope as the original, measured dry weather data. As previously described, the FEMA predictions of Base Flood Elevation are based upon a modeling study of a storm surge propagating from the Charleston Harbor, into the Ashley River, and inland along the Church Creek corridor. As a result, the FEMA study represents a “dry weather” prediction because it does not include the contribution of rainfall-runoff to the flood event. At a minimum, the modeled FEMA data provide a physically-based, approximate validation of the dry weather linear regression results. Furthermore, the similar slope and intercept of the data suggest that the FEMA solutions of the shallow water wave equations and the measured dry weather data both represent the relationship between Ashley River peak WSE and the WSE of Church Creek at Crosstowne in a similar manner.

3.3 Rainfall Averaging

Precipitation patterns vary spatially such that the total depth and intensity of rainfall vary even within a small area (Noori et al., 2014; Taesombat & Sriwongsitanon, 2009; Segond et al, 2007, Berne et al., 2004). This phenomenon is especially true in the Church Creek basin, which is situated just a few miles inland from the Atlantic Ocean and as a result is often affected by both maritime and inland weather systems. This pattern was observed on several occasions at the two primary rain gages used in this study. The KCHS station at Charleston International Airport is roughly 14 miles inland from the mouth of Charleston Harbor, while the KCXM station located in Downtown Charleston is less than 5 miles from the mouth of the Harbor and the Atlantic Ocean. For several large storm events that affected both gages, the daily totals at the two gages varied by could vary by more than 1 inch of total rainfall.

To account for these variations, cumulative rainfall depths were estimated using an Inverse Distance Weighted (IDW) average of the two primary gage stations. An IDW average is based on the assumption that when determining an unknown value at a given location, that value is inversely related to a known value at another location with respect to the distance between the two points. Simply put, when averaging two or more values to determine the value at another point, the unknown value is more likely to be similar to the value of the point closest to it. The formula used for calculating the IDW average rainfall totals is included below as Equation 2.

In the base form of the IDW equation, the variable “p” is referred to as the power parameter (also commonly designated as the alpha or α parameter). The value of this parameter must be a real, positive number and it is used to control the influence of known values on the unknown value based on their proximity. Specifically, the higher the power parameter, the more influence a point closer to the point of interest has. The default value of this parameter is 2.0 (Zhu & Jia, 2004), which was used in this study. While several studies concerning the most appropriate value of “p” have found that the default value of 2.0 is rarely the optimal choice for generating the most accurate estimation of rainfall at an unknown location (Noori et al., 2014; Chen & Liu, 2012; Tung, 1983; Simanton & Osborn, 1980), Dirks et al. (1998) found that small changes in the value of “p” between 1.5 and 4 had minimal effect on the observed level of error in estimations. The decision to use the default value of this parameter was therefore made because neither of the known values are particularly close to the point of interest, and because the two gages occupy somewhat different landscape positions that experience some difference in weather patterns.

Equation 2: Inverse Distance Weighted Average Formula for Daily Rainfall Totals

$$P = \frac{\sum P_i * \left(\frac{1}{d_i}\right)^p}{\sum \left(\frac{1}{d_i}\right)^p} = \frac{\left[P_{KCHS} * \left(\frac{1}{d_{KCHS}}\right)^2\right] + \left[P_{KCXM} * \left(\frac{1}{d_{KCXM}}\right)^2\right]}{\left(\frac{1}{d_{KCHS}}\right)^2 + \left(\frac{1}{d_{KCXM}}\right)^2}$$

where: P = Unknown Daily Rainfall Total at Crosstowne (in)
P_{KCHS} = Known Daily Rainfall Total at KCHS Station (in)
P_{KCXM} = Known Daily Rainfall Total at KCXM Station (in)
d_{KCHS} = Distance between Crosstowne and KCHS (mi)
d_{KCXM} = Distance between Crosstowne and KCXM Total (mi)

In the context of this study, the unknown value is the daily rainfall total at the location of Crosstowne. Given the known daily rainfall totals at the KCHS and KCXM gages and the straight-line distance from these two gages to Crosstowne, an IDW average daily rainfall total can be calculated to represent the daily rainfall total at Crosstowne. Based on the form of Equation 2 and the knowledge that Crosstowne is located approximately 5.15 miles from KCHS and 8.75 miles from KCXM, the calculated IDW daily rainfall totals calculated for Crosstowne will be closer in magnitude to those of KCHS. The calculated IDW totals represent a simple method of estimating the actual rainfall totals at this location.

3.4 Rainfall vs. Peak Stage Page 47

To study the effects on rainfall on the water level in Church Creek, cumulative rainfall totals were paired with observed peak water surface events. The peak WSE data of interest in this analysis are the those associated with the events described in the historic crests data set as described in Section 2.2. Cumulative daily rainfall totals were estimated using daily rainfall totals as described above in section 3.3.

For an observed peak water surface event, cumulative rainfall totals were estimated for each of the five days preceding the event. Accordingly, a 24-hour (or 1-day) cumulative total represents the total rainfall that occurred on the calendar day of the observed peak water surface event. A 48-hour (or 2-day) cumulative total is based on the total rainfall that occurred on the calendar day of the observed peak and the total rainfall that occurred on calendar day prior to the day of the observed peak. Similarly, totals were developed for 72-hour (3-day), 96-hour (4-day), and 120-hour (5-day) periods. An example of how these cumulative rainfall totals were developed has been included below as Table 1.

Table 1: Example Derivation of Cumulative Rainfall Totals for an Observed Peak Water Surface Event on 08/31/2015.

Date	Daily Rainfall Total (in)	Cumulative Rainfall Total (in)
09/01/2015	0.00	-
08/31/2015	6.43	24-hr Total = 6.43
08/30/2015	0.97	48-hr Total = 7.40 (6.43 + 0.97)
08/29/2015	0.00	72-hr Total = 7.40 (7.40 + 0.00)
08/28/2015	0.08	96-hr Total = 7.48 (7.40 + 0.08)
08/37/2015	0.07	120-hr Total = 7.55 (7.48 + 0.07)
08/26/2015	0.00	-

These five different cumulative rainfall totals were analyzed for several reasons. The 24-hour total was included because a storm duration of 24 hours is the typical rainfall event used for most stormwater management studies. Like many municipalities, the City of Charleston requires the use of SCS 24-hour storm depths and synthetic unit hydrographs for stormwater design calculations. Accordingly, 24-hour storm events have been the standard form of rainfall data used in all previous studies of the Church Creek basin.

The 120-hour (5-day) total was included because this rainfall total is commonly used as a metric for approximating antecedent moisture condition (Silveira et al., 2000; Kent, 1973; Mockus & Ogrosky, 1964). Antecedent moisture condition (AMC) is used to approximate the amount of available rainfall storage within the soil. Thus, AMC has a large influence on a landscape's ability to infiltrate rainfall and generate runoff, and by using the 120-hour cumulative rainfall total, it is assumed that a fairly accurate representation of the AMC and its effects on a landscape's hydrology have been captured (Kent, 1973).

While 24-hour and 120-hour rainfall totals are two of the most commonly used in hydrologic analyses, the intermediate daily totals were also included in this study to examine the relationship between these rainfall totals and observed peak water levels in Church Creek at Crosstowne. Regression analyses were thus performed using a subset of the "wet weather" data, which included the ten largest recorded water surface elevations at the NWS-Woolpert gage, along with the extreme events that flooded Crosstowne. Although several of these events occurred prior to the installation of the BEES1 gage, the peak water surface elevations were measured on the Crosstowne site or within the building. These events, which are associated with Tropical Storm Erika, Hurricane Joaquin, and Hurricane Matthew, left distinctive high-water marks on the church building or site features, which were subsequently measured relative to the FFE of the building.

Cumulative daily rainfall totals corresponding to these events were estimated according to the IDW methodology described in section 3.3 for the following durations: 24-hour (1-day), 48-hour (2-day), 72-hour (3-day), 96-hour (4-day), and 120-hour (5-day). Figures 3.5–3.9 depict each of the cumulative rainfall totals plotted against the corresponding peak WSE in Church Creek.

The linear regression analyses performed on the cumulative rainfall totals of varying durations for the historic crests data set exhibited a strong relationship (R-squared ranging from 0.74 to 0.81), with the exception of the 24-hour total (R-squared of 0.31). Each of the linear regressions exhibited a default y-intercept of approximately 3.00. Theoretically, this intercept represents the peak WSE in Church Creek with a zero rainfall depth during the duration of interest. The “dry weather” data support this approximation; therefore, the y-intercept for the regression analyses was set to 3.00 to provide a realistic comparison among the regression analyses of cumulative rainfall totals.

The 48-hour rainfall total exhibited the strongest relationship between rainfall and peak WSE in Church Creek with a R-squared value of 0.81 and a predicted R-squared value of 0.73. A more detailed version of the plot seen in Figure 3.6 can be seen in Figure 3.10. While some literature suggest that a five-day rainfall total is the best approximation of a landscape’s AMC (Kent, 1973; Mockus & Ogrosky, 1964), the goal of the present analyses to determine the time scale at which the peak WSE of the Creek is most influenced by rainfall. So while a five-day rainfall total may be the most appropriate for basin-wide analysis of water balance, these results suggest that the 48-hour rainfall is best correlated with peak water surface elevations measured in Church Creek at Crosstowne.

Although these results exhibit a strong correlation between 48-hour cumulative rainfall and peak WSE in Church Creek, the data also demonstrates that the rainfall parameter alone is not an accurate predictor of peak WSE at Crosstowne. For small and large events alike, the regression relationship both over- and under-estimates the peak WSE in the Creek. For example, this regression equation would over-predict the peak WSE resulting from Hurricane Joaquin by approximately one foot based solely on the 48-hour rainfall total associated with the storm. Similarly, the regression equation would under-predict the peak WSE in the Creek by nearly two feet for Hurricane Irma. These inconsistencies confirm the complexities of compound flooding, and indicate that rainfall depth is not a sufficient predictor of the peak WSE at Crosstowne.

3.5 Stage Increase Due to Rainfall

As described in section 3.2, Church Creek at Crosstowne exhibits a clear tidal influence driven by its direct connection with the Ashley River. Figure 3.4. demonstrates that “dry weather” peak WSE data exhibit a very strong, predictable trend. Furthermore, as described in the previous section, during “wet weather” the peak WSE of Church Creek at Crosstowne is strongly correlated to 48-hour rainfall total. It follows that during wet weather, the peak WSE of Church Creek at Crosstowne is influenced by both rainfall and the tide in the Ashley River.

More explicitly, the dry weather data and trendline describe the correlation between peak WSE in the Ashley River and peak WSE in Church Creek at Crosstowne. Although this relationship is displayed mostly clearly in isolation during dry weather, the peak WSE in the Ashley River contributes to the peak WSE at Crosstowne during wet weather as well. Accordingly, we hypothesize that the influence of the Ashley River peak WSE persists during wet weather, but that this influence is simply obscured by the effects of rainfall. The form of the “dry weather” and “wet weather” correlations described in section 3.2 promote a direct comparison to explore this hypothesis.

For each observation of peak WSE in the “wet weather” data set in the Ashley River, the predicted peak WSE in Church Creek was calculated using the “dry weather” regression equation. The difference between the predicted peak WSE in Church Creek and the measured observation of peak WSE at that location could then be understood as the “additional stage”, contributed by the 48-hour rainfall total, beyond the stage increase contributed by the peak WSE of the Ashley River. This additional stage difference provides a simple method for isolating the cumulative effects of stage increase due to rainfall.

As Figure 3.11 illustrates, this calculated set of stage increase data can be represented by a power regression function of additional stage versus 48-hour rainfall. This power function is also presented below as Equation 3, and exhibits an R-squared value of approximately 0.76 and a predicted R-squared value of 0.73.

Equation 3: Additional Stage Function for Relating 48-hour Cumulative Rainfall Total and Observed – Predicted WSE in Church Creek.

$$y = 5.81x^{0.54}$$

where: y = Church Creek WSE Difference (ft, NAVD88)
x = 48-hour Cumulative Rainfall Total (ft)

It should be noted that this method of isolating stage increase is relatively crude, and that fully isolating the effects of rainfall from tide is difficult considering that the catastrophic flooding events have occurred from heavy rainfall and some degree of tidal surge.

3.6 Compound Flooding Function

The “Additional Stage” function (Equation 3) was then combined with the “Dry Weather” function (Equation 1) to create a function that represents the effects of tide, rainfall, or both. This “Compound Flooding Function,” listed below as Equation 4, synthesizes the independent regression analyses of peak WSE contributed by tide and peak WSE contributed by rainfall.

Equation 4: Compound Flooding Function for Predicting Church Creek Peak WSE

$$y = [0.45x_1 + 0.44] + 5.81x_2^{0.54}$$

where: y = Peak WSE: Church Creek at Crosstowne (ft, NAVD88)
x_1 = Peak WSE: Ashley River at I-526 (ft, NAVD88)
x_2 = 48-hour Cumulative Rainfall Total (ft)

The peak WSE data calculated by the Compound Flooding Function were then plotted against the observed peak WSE data. A simple linear regression analysis of the predicted and observed data provided a means of approximating the "goodness of fit" of the function. If the function perfectly predicted the measured values, a linear regression test would exhibit a slope of unity, a y-intercept of zero, and an R-squared value close to one. This goodness-of-fit plot, along with a regression analysis comparing the "predicted" versus observed data sets has been included as Figure 3.12.

The slope of the linear regression is approximately one and the y-intercept is zero. The R-squared and predicted R-squared values calculated for this linear regression model are both 0.95. These results indicate that the predicted values are very similar to the measured observations. For example, the function predicts the peak WSE of Church Creek at Crosstowne within one foot of the Hurricane Irma flood, and within less than a tenth of a foot for Hurricanes Matthew and Joaquin. However, for the mid-magnitude events (storm events with relatively small amounts of rainfall) the predictions exhibit significantly more error between the observed and predicted data.

The Compound Flooding Function reveals the relative influence of both rainfall and tide on the peak WSE of Church Creek at Crosstowne across the full range of observed peak WSE data. Furthermore, the Function exhibits a significant predictive value when considering historic flood data and dry weather data that were used in its development. Whereas the data sets described herein have been used to calibrate the Compound Flooding Function, future data will be used to verify the Compound Flooding Function to validate its relevance and accuracy.

4. CONCLUSIONS & RECOMMENDATIONS

4.1 Stormwater Flooding

The results of this study demonstrate that the peak WSE of Church Creek at Crosstowne Christian Church is influenced by both stormwater runoff and tidal levels in the Ashley River. However, the results strongly suggest that for flooding events, the peak WSE of Church Creek at Crosstowne is overwhelmingly driven by rainfall. In fact, Crosstowne has between a 2 and 3 percent annual chance of flooding (“50-year” to “33-year” recurrence interval) by rainfall only.

A primary conclusion of this study is that the peak WSE of Church Creek at Crosstowne is strongly correlated to cumulative rainfall over 48 hours (Figure 3.6). Furthermore, the peak WSE of Church Creek at Crosstowne is only weakly correlated to cumulative rainfall over 24 hours (Figure 3.5). These correlations and individual storm event hydrographs suggest that the hydrologic response of the Church Creek watershed is normally longer than 24 hours, such that the runoff hydrograph peak occurs more than 24 hours following the start of rainfall. Current City of Charleston stormwater design standards for new land developments only require analyses of the 24-hour rain event and runoff hydrograph, and thus are unlikely to safeguard Crosstowne from additional flooding from new land developments.

4.2 Compound Flooding

While the combined effects of stormwater runoff and storm surge pose a significant risk, Crosstowne is highly susceptible to flooding from normal rain events in the absence of storm surge. Because Crosstowne is situated several miles inland from the tidal forcing of the Atlantic Ocean, much of the tidal energy entering the creek system is lost before it reaches Crosstowne. Nevertheless, during a seasonal “King Tide” event, the Compound Flooding Function (Equation 4) predicts that Crosstowne would be flooded by the 48-hour rainfall event with the 10-year recurrence interval. Furthermore, during a normal high tide event, the Compound Flooding Function predicts Crosstowne would be flooded by the 48-hour rainfall event with the 50-year recurrence interval. Figure 4.1 provides a graphical representation of the various combinations of tides and 48-hour rainfall totals that would cause flooding at Crosstowne. Figure 4.2 displays the same graphical representation of data presented in Figure 4.1, but with an additional overlay representing the joint probability of exceedance for different combinations of tide and 48-hour rainfall. Under current hydrologic conditions, Crosstowne has approximately 4 percent annual chance of flooding (“25-year” recurrence interval) by the combined effects of rainfall and tide. Compared to the probability of flooding from rainfall only, tidal surge protection measures would only slightly reduce the likelihood of flooding at Crosstowne.

4.3 Future Development

This study confirms that any new land development or land clearing activities in the Church Creek basin directly threaten to increase the flood frequency and flood depth at Crosstowne Christian Church. As stated above, data from individual storm event hydrographs suggest that the hydrologic response of the Church Creek watershed is normally longer than 24 hours, such that the runoff hydrograph peak occurs more than 24 hours following the start of rainfall. Current City of Charleston stormwater design standards for new land developments only require analyses of the 24-hour rain event and runoff hydrograph, and thus are unlikely to safeguard Crosstowne from additional flooding from new land developments. To maintain the “status quo” of flood frequency and probability at Crosstowne, the data suggest that any new land development activities in the Church Creek basin must retain all excess stormwater runoff volume generated on-site, or detain excess runoff for no less than 48 hours from the start of rainfall.

4.4 Assumptions & Limitations

One of the primary assumptions of this study is that the hydrologic regime of the Church Creek basin remained relatively constant throughout the data collection period. More specifically, it was assumed that no major changes to land cover occurred and that no major hydrologic modifications occurred during the period of study. These assumptions should be valid because, from early 2017 to late 2018, a land development moratorium throughout the Church Creek basin was imposed by the City of Charleston in response to flooding in late 2016. As a result, the measured data related to large magnitude peak WSE events in the Creek all occurred under similar conditions, and the only variables affecting the water levels in the Creek are rainfall and tide.

Another primary assumption of this study is that measuring and calculating peak water surface elevation is an appropriate descriptor of flood behavior in Church Creek in the vicinity of Crosstowne. By definition, flooding is associated with an elevated water level, but in the context of hydraulic analyses, flooding is typically characterized or quantified by discharge (volumetric flow rate). For inland watersheds, flooding can be reasonably predicted by relating discharge to the resultant flood elevation at certain point along a waterway (Sturm, 2001). However, in the context of a tidally-influenced system like Church Creek, the relationship between water surface elevation and discharge is variable and nonstationary (Leopold, Collins, & Collins, 1993). This means that a specific discharge value cannot be easily or directly correlated to a specific water surface elevation, and that a single discharge value can be related to numerous water surface elevations throughout the tidal cycle based upon the magnitude of the tide, the absolute difference between low and high tides, etc. Therefore, under these physical conditions, it is assumed that a predictive analyses of peak water surface elevation at a point along the Creek is valid and more reliable than discharge-based relationships.

4.5 Future Work

Most importantly, long term monitoring and data collection should continue at the various gage sites located throughout the study area. Although some gages have long-running records (e.g. NOAA-Charleston Harbor and USGS-Ashley River), the gages installed to monitor Church Creek itself have only recently been installed in response to the recurring floods of the past several years. As a result, the localized record of data for parameters of interest within Church Creek and its watershed is very limited.

Future storm-event data will also enable further validation of the Combined Flooding Function. Although the results of this study indicate that the Combined Flooding Function is a strong fit of the existing data set, additional dry weather and flood data are necessary to properly validate the function. Similarly, another step that should be taken is to improve the accuracy of the rainfall data used in the various analyses by incorporating the complete daily precipitation records of the NWS-Woolpert BEES1 gage, or by installing a rain gage at the Crosstowne gaging station. A third, off-site rain gage with a long period of record similar to KCHS and KCXM could also be used as a third point in calculating IDW average daily totals. A third gage, ideally in the southern portion of West Ashley or on Johns Island, could supplement and triangulate the measurements at KCHS and KCXM in the IDW calculations.

What are unsteady flow dynamics?

Time-dependent flow is known as unsteady

Previous studies of the Church Creek basin have been based on computer simulations using one-dimensional, quasi-unsteady models that were neither calibrated nor verified. While these studies may offer some utility for qualitative land planning efforts, the results of these models are unreliable at best and misleading at worst. Future efforts to simulate basin-wide hydrologic processes models must account for shallow groundwater storage, unsteady flow dynamics, and two-dimensional flows. Delineating watershed boundaries within low-relief, coastal drainage basins is notoriously difficult. The Church Creek drainage basin and the surrounding landscape is characterized by bottomland forested wetlands within sandy soils underlain by clay. So while a high resolution DEM reveals the surface topography and surface runoff pathways, subsurface groundwater connectivity does not necessarily correspond to the surface topography. As a result, it is very difficult to assess the actual land area and stormwater runoff volume that drain to the creek. Computational models can easily be “verified” to match an observed water surface elevation at one location; however, calibrating a model to accurately simulate observations across an entire watershed is a much more rigorous process (Sturm, 2001). The magnitude and frequency of flooding at Crosstown Christian Church, and across the Church Creek basin, demand physically-valid hydrologic and hydraulic analyses that are based upon field measurements of water surface elevation and discharge at multiple points throughout the watershed.

Robinson Design Engineers has begun collecting storm-event discharge measurements of Church Creek at Crosstowne Christian Church. The results of these measurements and subsequent analyses will be documented in a separate report.

5. REFERENCES

Berne, A., Delrieu, G., Creutin, J., & Obled, C. (2004). Temporal and spatial resolution of rainfall measurements required for urban hydrology. *Journal of Hydrology,* 299, 166-179. https://doi.org/10.1016/j.jhydrol.2004.08.002

Bonnin, G.M., Martin, D., Lin, B., Parzybok, T., Yekta, M., & Riley, D. (2006). NOAA Atlas 14: Precipitation-Frequency Atlas of the United States. Volume 2 Version 3.0: Delaware, District of Columbia, Illinois, Indiana, Kentucky, Maryland, New Jersey, North Carolina, Ohio, Pennsylvania, South Carolina, Tennessee, Virginia, West Virginia. Silver Spring, Maryland: U.S. Department of Commerce, National Oceanic and Atmospheric Administration (NOAA).

Chen, F., & Liu, C. (2012). Estimation of the spatial rainfall distribution using inverse distance weighting (IDW) in the middle of Taiwan. *Paddy and Water Environment,* 10(3), 209-222. doi: 10.1007/s10333-012-0319-1.

Cronshey, R., McCuen, R.H., Miller, N., Rawls, W., Robbins, S., & Woodward, D. (1986). Technical Release 55: Urban Hydrology for Small Watersheds. Washington, D.C.: U.S. Department of Agriculture, Natural Resources Conservation Service (NRCS).

Dirks, K.N., Hay, J.E., & Harris, S.D. (1998). High-resolution studies of rainfall on Norfolk Island Part II: Interpolation of rainfall data. *Journal of Hydrology,* 208(3), 187-193. https://doi.org/10.1016/S0022-1694(98)00155-3.

Hartkamp, A.D., K. De Beurs, A. Stein, & J.W. White. (1999). *Interpolation Techniques for Climate Variables*. NRG-GIS Series 99-01. Mexico, D.F.: CIMMYT.

Juien, P.Y. (2002). *River Mechanics*. New York, NY: Cambridge University Press.

Kent, K.M. (1973). SCS-TP-149: A Method for Estimating Volume and Rate of Runoff in Small Watersheds. Washington, D.C.: U.S. Department of Agriculture, Soil Conservation Service (SCS).

Leopold, L.B., Collins, J.N., Collins, L.M. (1993). Hydrology of Some Tidal Channels in Estuarine Marshland Near San Francisco. *Catena*, 20, 469-493. Retrieved from https://eps.berkeley.edu/people/lunaleopold/.

Naoum, S., & Tsanis, I.K. (2004). Ranking Spatial Interpolation Techniques Using a GIS-Based DSS. *Global NEST Journal,* 6(1), 1-20. https://doi.org/10.30955/gnj.000224.

Noori, M.J., Hassan, H.H., & Mustafa, Y.T. (2014). Spatial Estimation of Rainfall Distribution and Its Classification in Duhok Governorate Using GIS. *Journal of Water Resource and Protection,* 6, 75-82. doi: 10.4236/jwarp.2014.62012.

Ogrosky, H. O., and Mockus, V. (1964). Hydrology of agricultural lands. Handbook of applied hydrology, Ven te Chow, ed., (sec. 21), 97 p. McGraw-Hill Book Co., New York.

Segond, M., Neokleous, N., Makropoulos, C., Onof, C., & Maksimovic, C. (2007). Simulation and spatio-temporal disaggregation of multi-site rainfall data for urban drainage applications. *Hydrological Sciences Journal,* 52(5), 917-935. doi: 10.1623/hysj.52.5.917.

Silveira, L., Charbonnier, F., & Genta, J.L. (2000). The antecedent soil moisture condition of the curve number procedure. *Hydrological Sciences-Journal,* 45(1), 3-12. https://doi.org/10.1080/02626660009492302

Simanton, J.R., & Osborn, H.B. (1980). Reciprocal-distance estimate of point rainfall. *Journal of the Hydraulics Division,* 106(7), 1242-1246. Retrieved from https://www.tucson.ars.ag.gov/unit/publications/PDFfiles/329.pdf

Sturm, T.W. (2001). *Open Channel Hydraulics*. Textbook Series in Water Resources and Environmental Engineering, McGraw-Hill, New York.

Taesombat, W., & Sriwongsitanon, N. (2009). Areal rainfall estimation using spatial interpolation techniques. *Science Asia,* 35, 268-275. doi: 10.2306/scienceasia1513-1874.2009.35.268.

Tung, Y. (1983). Point Rainfall Estimation for a Mountainous Region. *Journal of Hydraulic Engineering,* 109(10). https://doi.org/10.1061/(ASCE)0733-9429(1983)109:10(1386)

U.S. Federal Emergency Management Agency (FEMA) (2004). Flood Insurance Study 45019CV000A: Charleston County, South Carolina and Incorporated Areas. Retrieved from https://msc.fema.gov/portal/home.

U.S. Federal Emergency Management Agency (FEMA) (2016). Flood Insurance Study 45019CV000B: Charleston County, South Carolina and Incorporated Areas. Retrieved from https://msc.fema.gov/portal/home.

U.S. Geological Survey (USGS) (1919). Topography Map: John’s Island, South Carolina. Retrieved from https://ngmdb.usgs.gov/topoview/.

U.S. Geological Survey (USGS) (1958). Topography Map: John’s Island, South Carolina. Retrieved from https://ngmdb.usgs.gov/topoview/.

Viero, D. P., & Defina, A. (2018). Consideration of the mechanisms for tidal bore formation in an idealized planform geometry. *Water Resources Research*, 54. https://doi.org/10.1029/2018WR022937.

Woolpert (2001). Church Creek Stormwater Master Plan: Technical Report (Volume 2). Charleston, SC: The City of Charleston, South Carolina.

Woolpert (2015). 2015 Church Creek Watershed ICPR Model Addition/Revision: Final Revised Report. Charleston, SC: The City of Charleston, South Carolina.

Weston & Sampson (2017). Church Creek Drainage Basin Study: Final Report. Charleston, SC: The City of Charleston, South Carolina.

Zhu, H.Y., & Jia, S.F. (2004). Uncertainty in the spatial interpolation of rainfall data. *Progress in Geography,* 23(2), 34–42. doi: 10.11820/dlkxjz.2004.02.005.

FIGURES

Figure 1.1: Map of the Church Creek Watershed
Figure 1.2: 1862 Map of Church Creek – Long Branch System
Figure 1.3: 1919 Map of Church Creek – Long Branch System
Figure 1.4: 1958 Map of Church Creek – Long Branch System
Figure 1.5: Map of the Area Surrounding Crosstowne Christian Church

Figure 2.1: Map of Water Level Data Collection Points
Figure 2.2: Map of Rainfall Data Collection Points

Figure 3.1: Church Creek stage hydrograph recorded at Bees Ferry Road and Crosstowne Christian Church for May 3, 2018 to June 13, 2018.
Figure 3.2: Linear regression plot comparing the Church Creek WSE as measured at Bees Ferry Road versus the Church Creek WSE as measured at Crosstowne Christian Church.
Figure 3.3: Ashley River WSE versus Church Creek WSE.
Figure 3.4: Ashley River WSE versus Church Creek WSE with data separated into "Wet Weather" and "Dry Weather" data sets.
Figure 3.5: 24-hour IDW Cumulative Rainfall Total versus Church Creek WSE.
Figure 3.6: 48-hour IDW Cumulative Rainfall Total versus Church Creek WSE.
Figure 3.7: 72-hour IDW Cumulative Rainfall Total versus Church Creek WSE.
Figure 3.8: 96-hour IDW Cumulative Rainfall Total versus Church Creek WSE.
Figure 3.9: 120-hour IDW Cumulative Rainfall Total versus Church Creek WSE.
Figure 3.10: 48-hour IDW Cumulative Rainfall Total versus Church Creek WSE.
Figure 3.11: Additional Stage Function Relating 48-hour Cumulative Rainfall Total and Observed – Predicted WSE Difference in Church Creek.
Figure 3.12: Goodness of Fit for Compound Flooding Function for Predicting Church Creek Peak WSE.

Figure 4.1: Predicted flooding of Crosstowne Christian Church building based on scenarios of 48-hour cumulative rainfall total and peak WSE in the Ashley River.
Figure 4.2: Joint probability of annual exceedance for predicted flooding of Crosstowne Christian Church building based on scenarios of 48-hour cumulative rainfall total and peak WSE in the Ashley River.

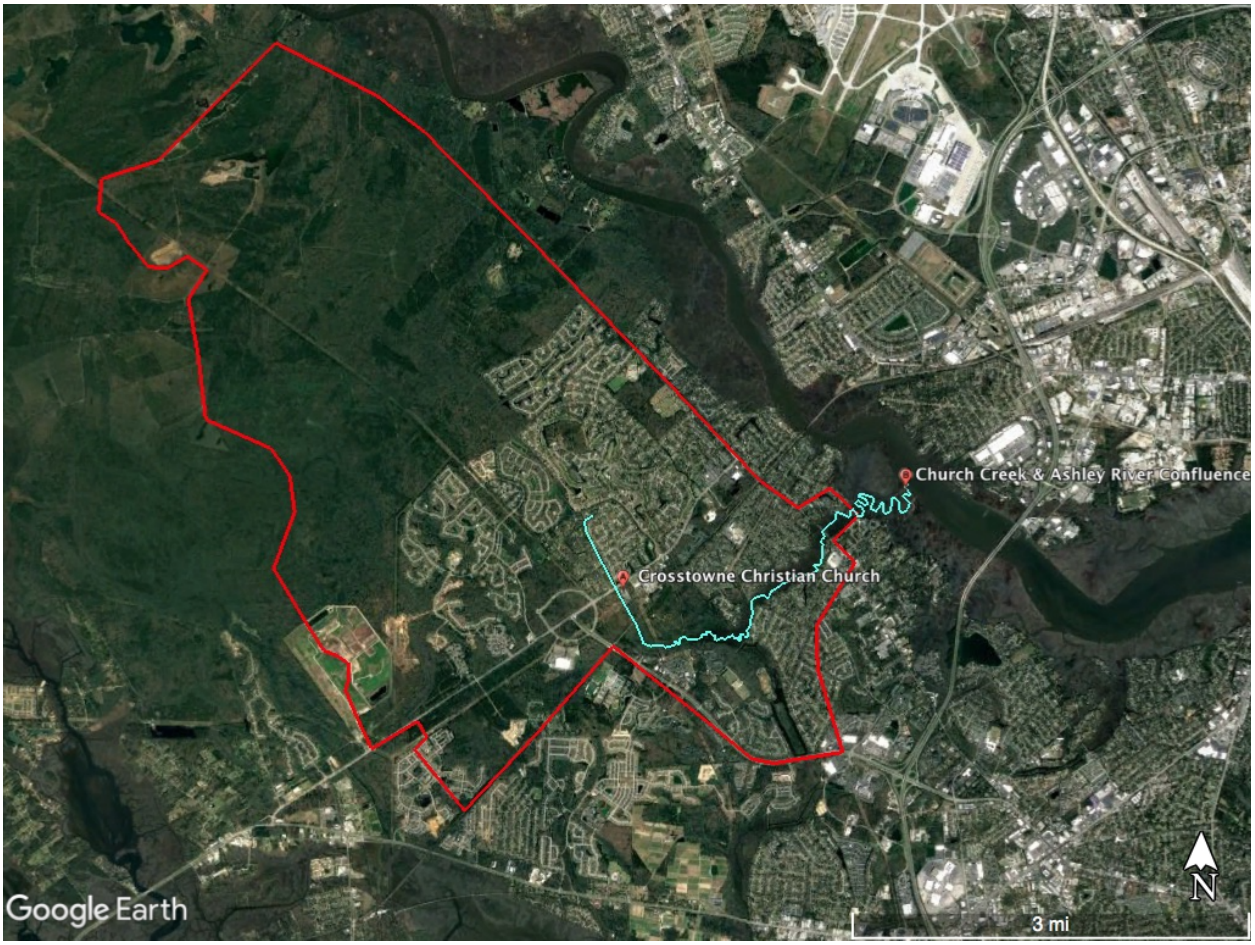

Figure 1.1: Map of the Church Creek Watershed

Figure 1.2: 1862 Map of Church Creek – Long Branch System

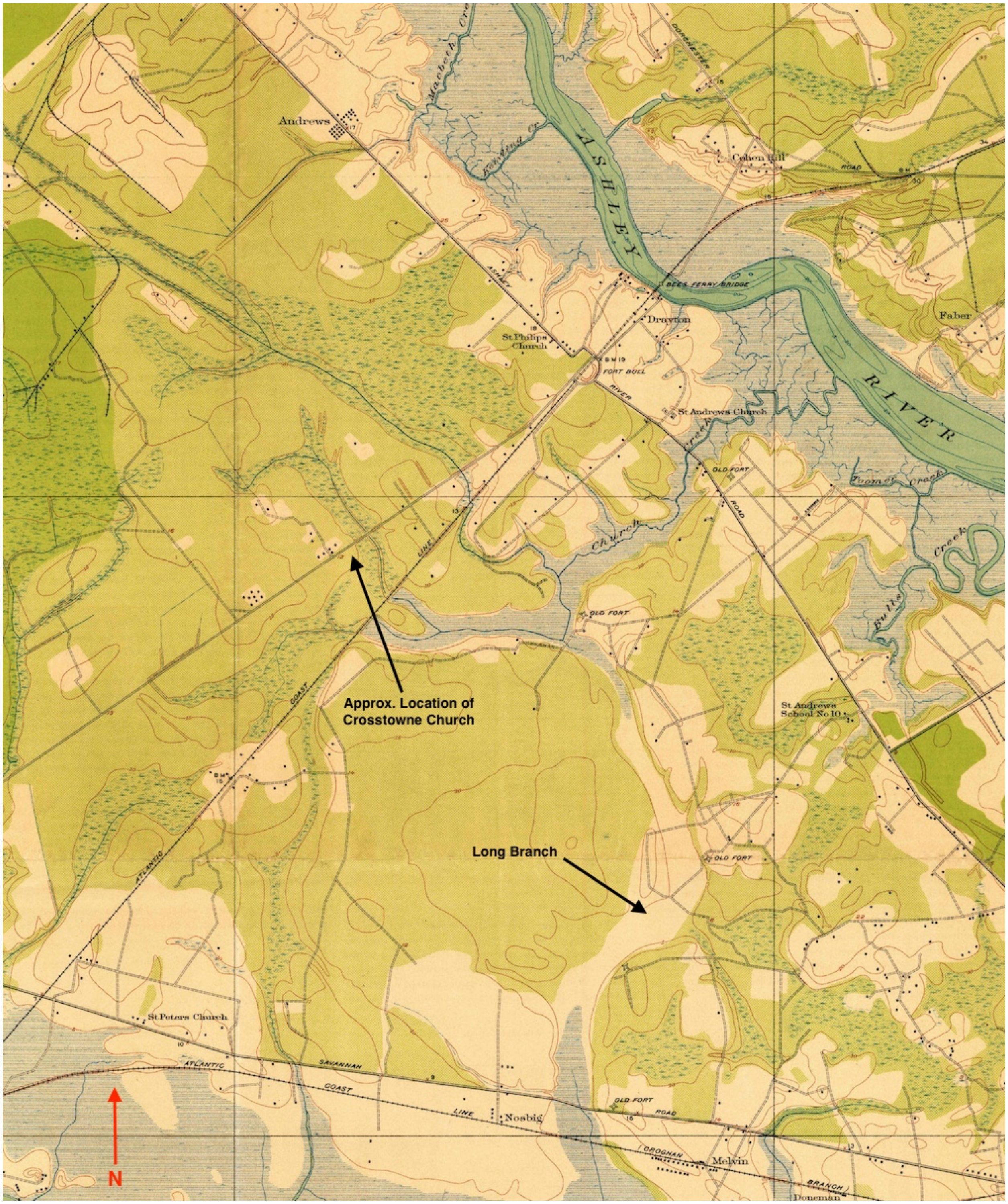

Figure 1.3: 1919 Map of Church Creek – Long Branch System (USGS, 1919).

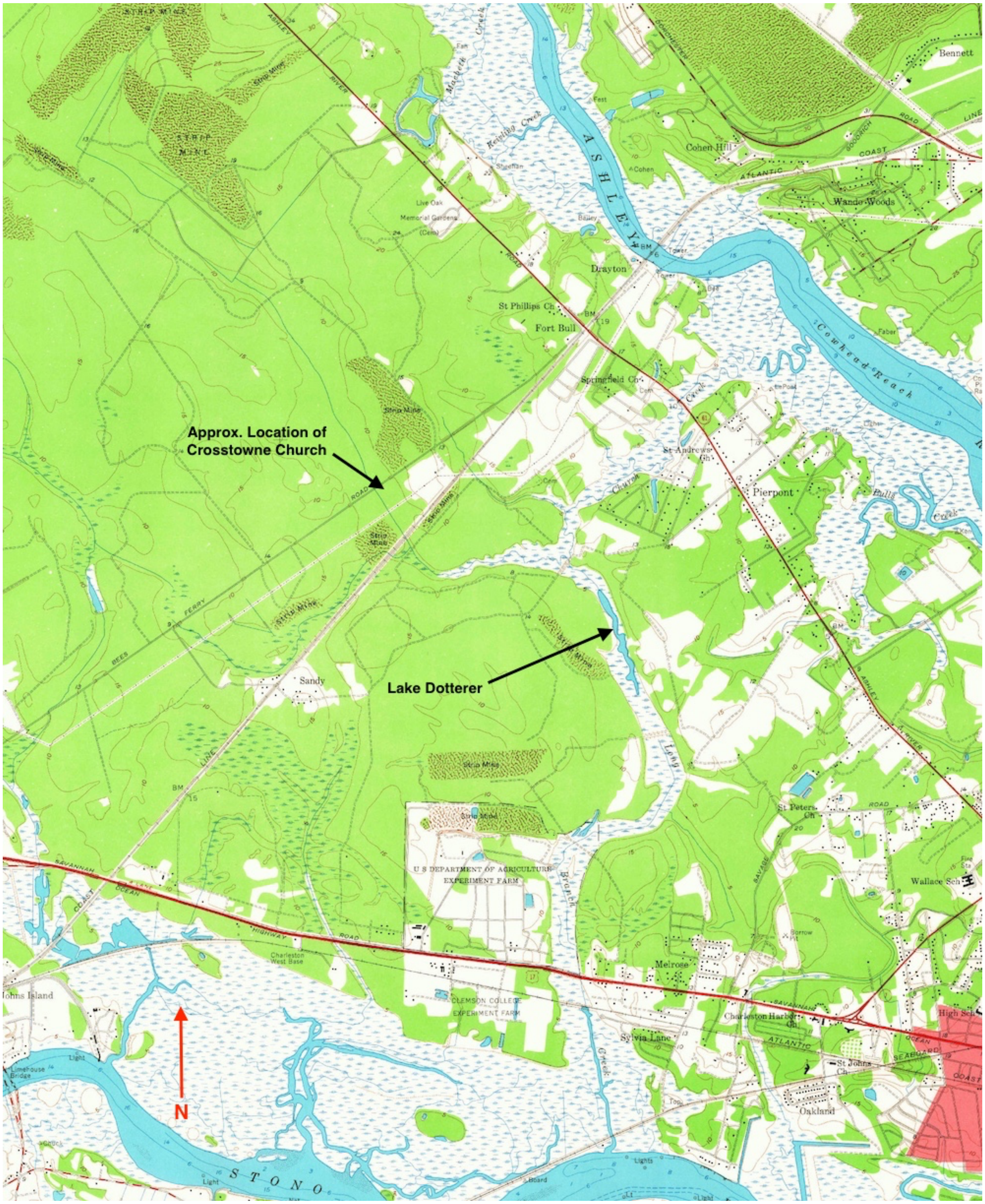

Figure 1.4: 1958 Map of Church Creek – Long Branch System (USGS, 1958).

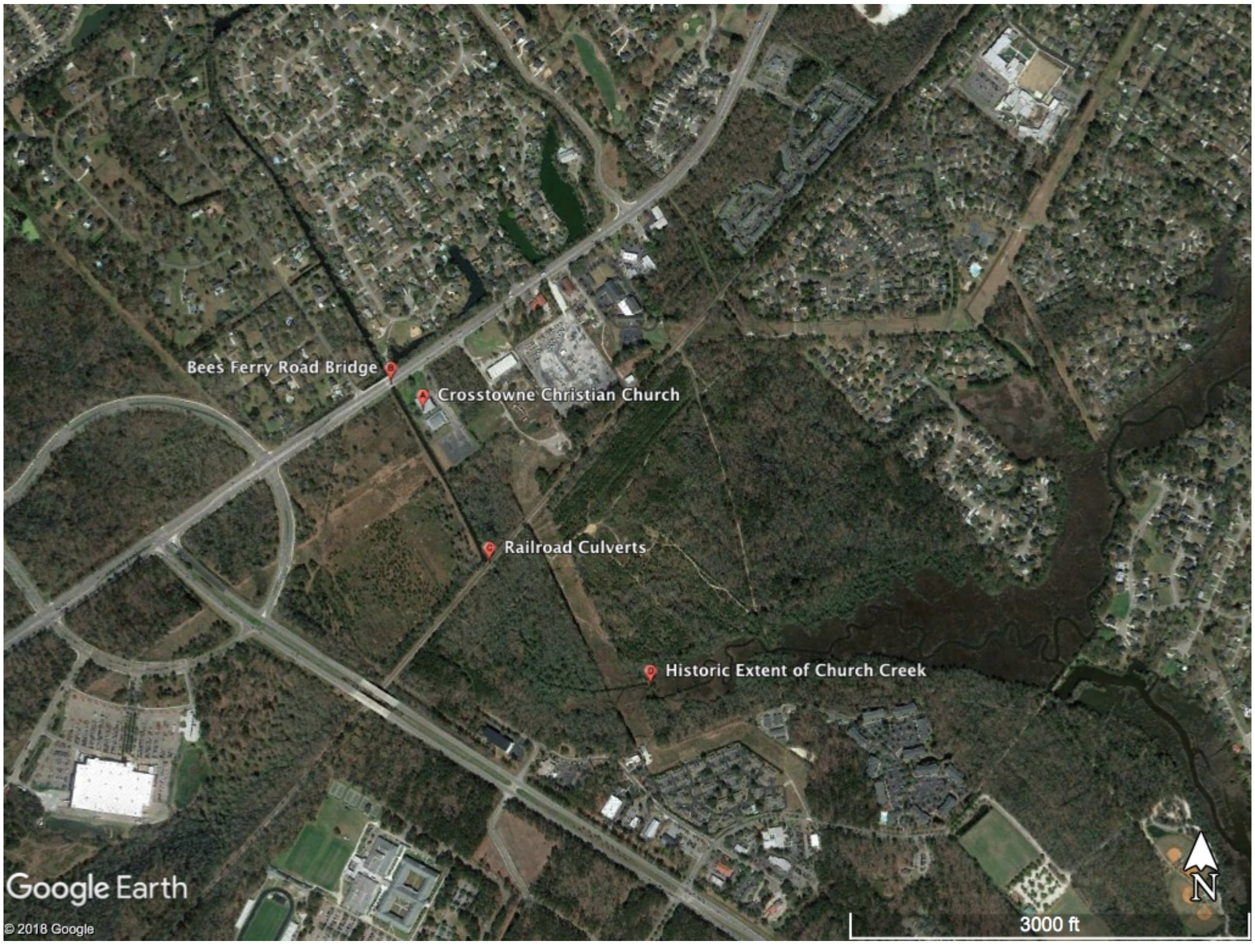

Figure 1.5: Map of the Area Surrounding Crosstowne Christian Church

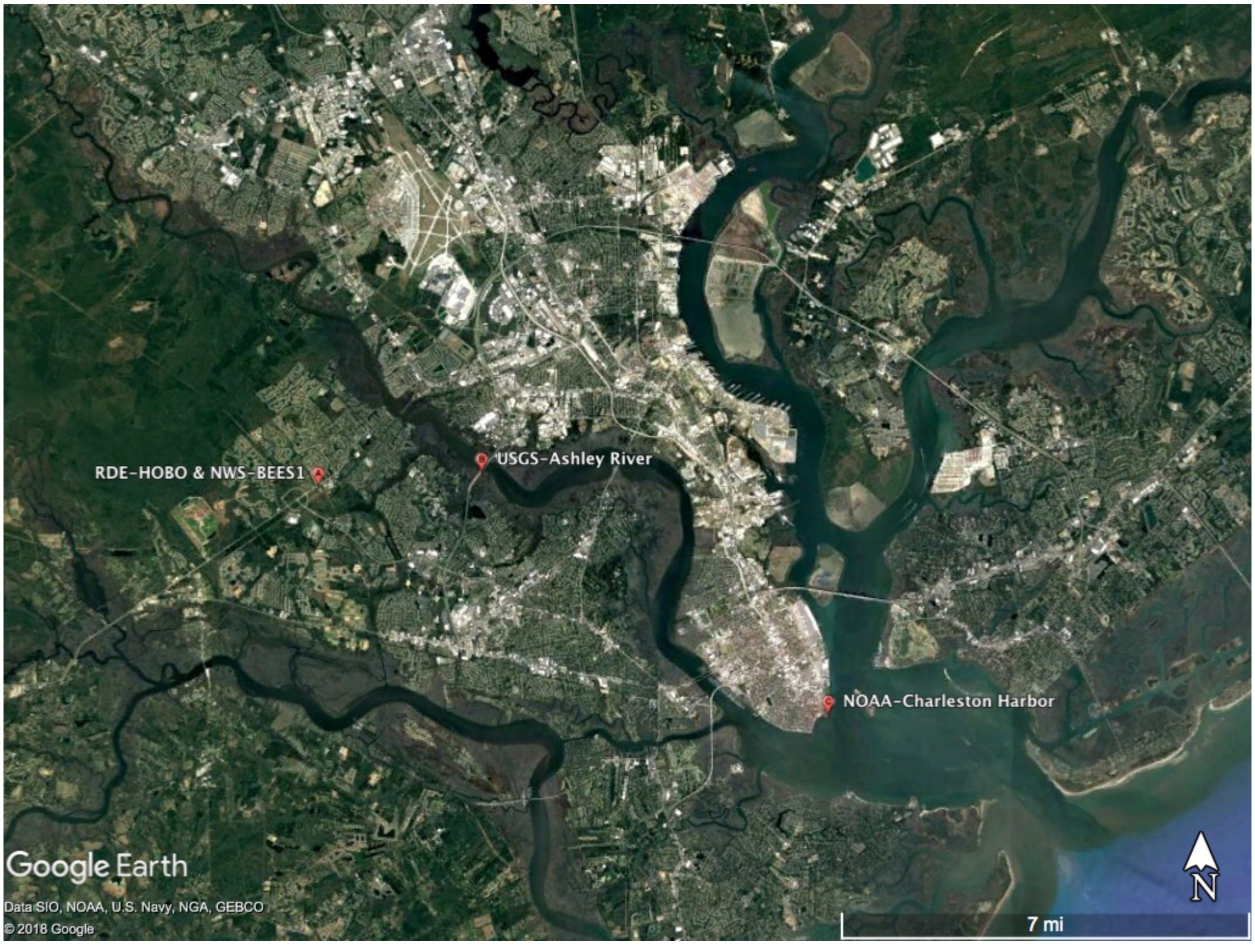

Figure 2.1: Map of Water Level Data Collection Points

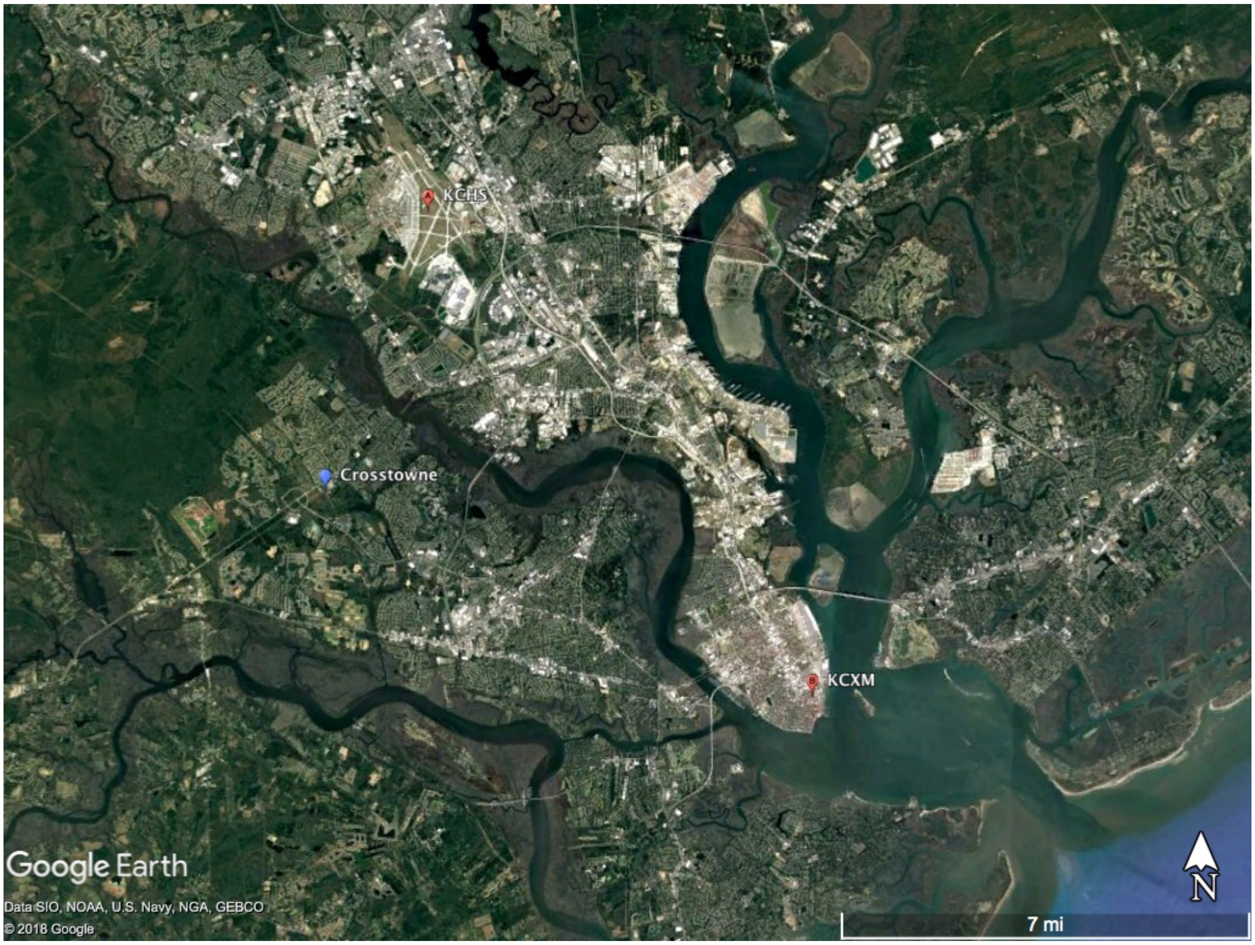

Figure 2.2: Map of Rainfall Data Collection Points

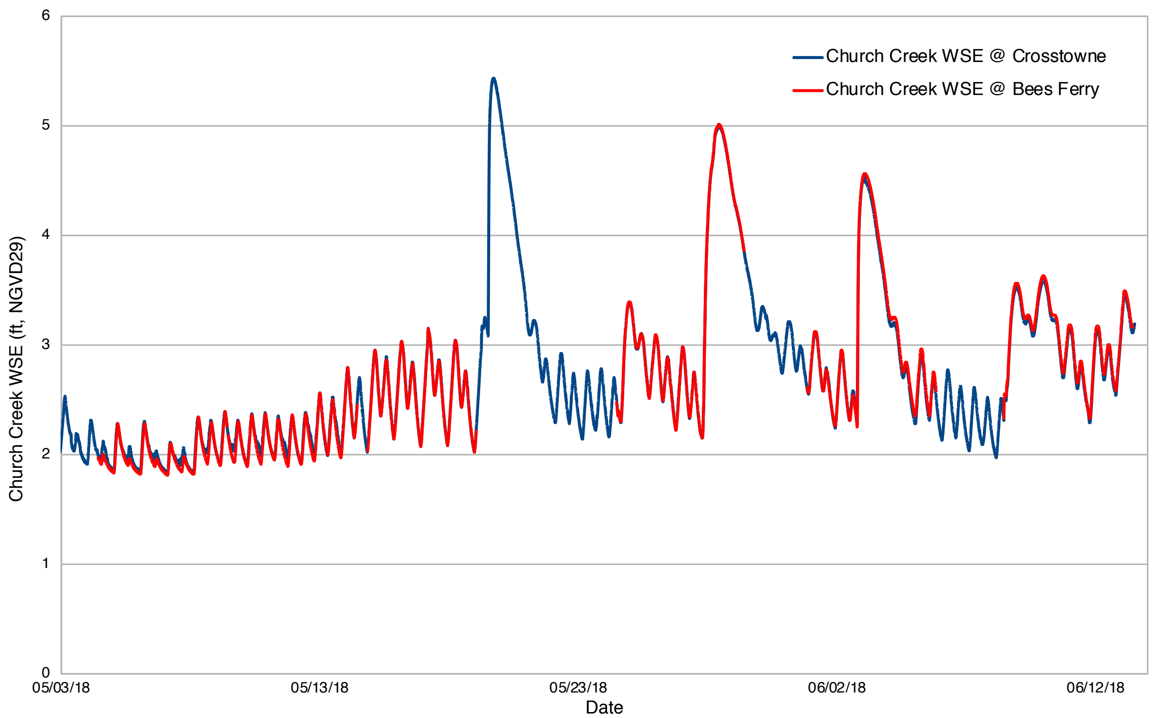

Figure 3.1: Church Creek stage hydrograph recorded at Bees Ferry Road and Crosstowne Christian Church for May 3, 2018 to June 13, 2018. WSE data for Bees Ferry Road were collected from the NWS-Woolpert BEES1 gage. WSE data for Crosstowne Christian Church were collected from the Robinson Design Engineers' gage at Crosstowne.

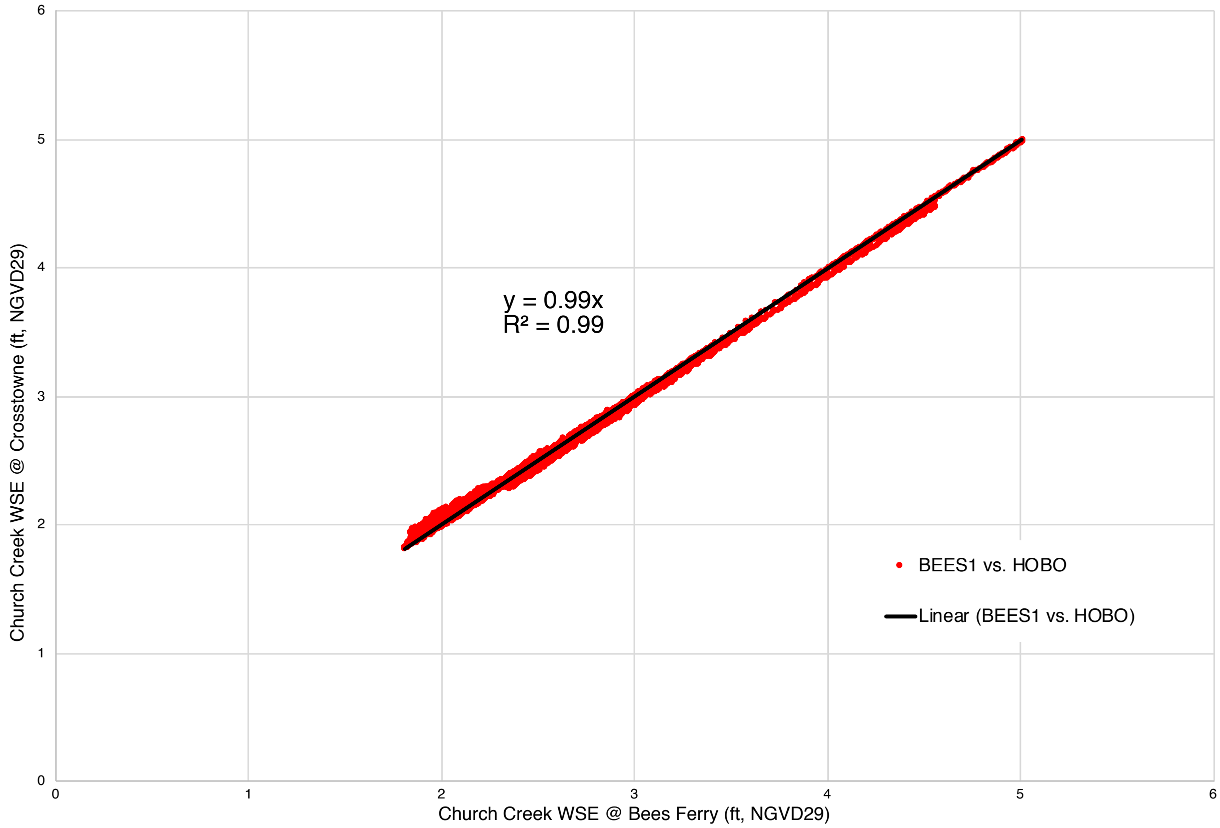

Figure 3.2: Linear regression plot comparing the Church Creek WSE as measured at Bees Ferry Road versus the Church Creek WSE as measured at Crosstowne Christian Church. WSE data for Bees Ferry Road were collected from the NWS-Woolpert BEES1 gage. WSE data for Crosstowne Christian Church were collected from the Robinson Design Engineers' gage at Crosstowne.

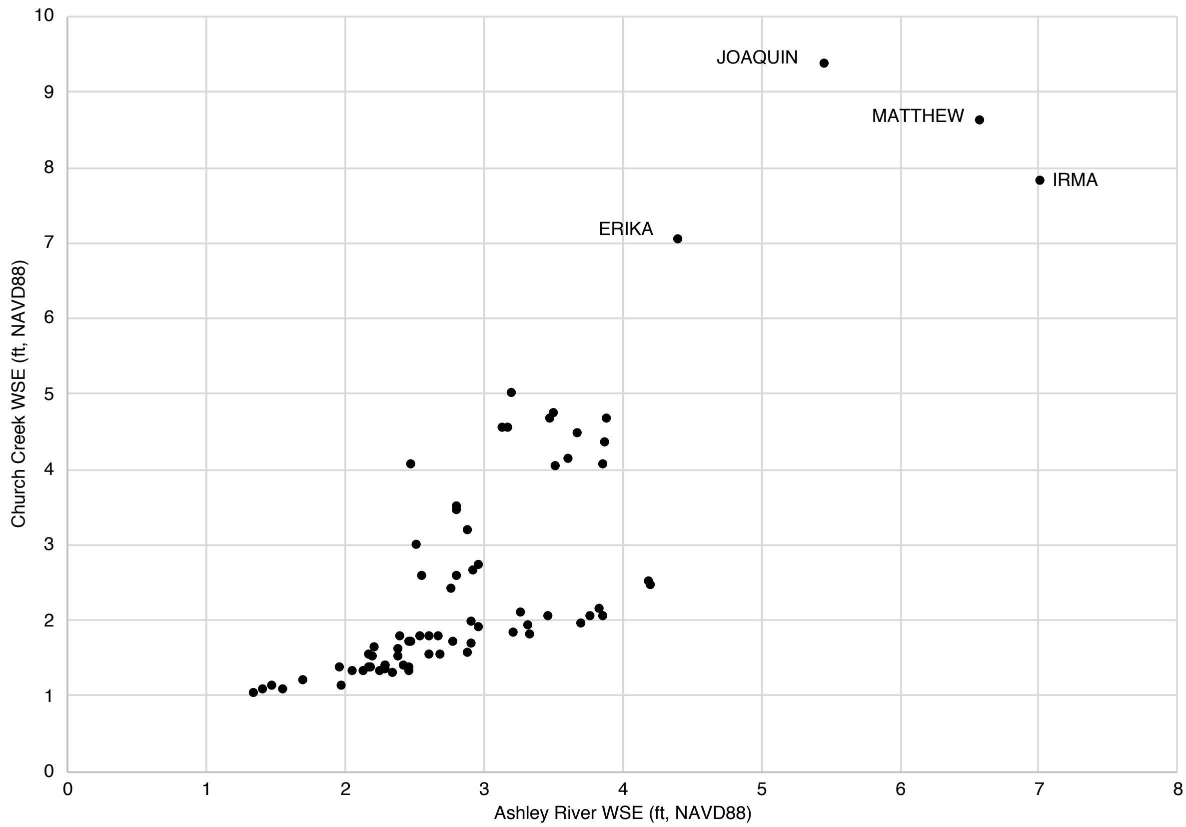

Figure 3.3: Ashley River WSE versus Church Creek WSE. Ashley River WSE data were collected from the USGS 021720869 Ashley River Near North Charleston, SC gage. Church Creek WSE data were collected from the NWS-Woolpert BEES1 gage and the Robinson Design Engineers' gage at Crosstowne Christian Church.

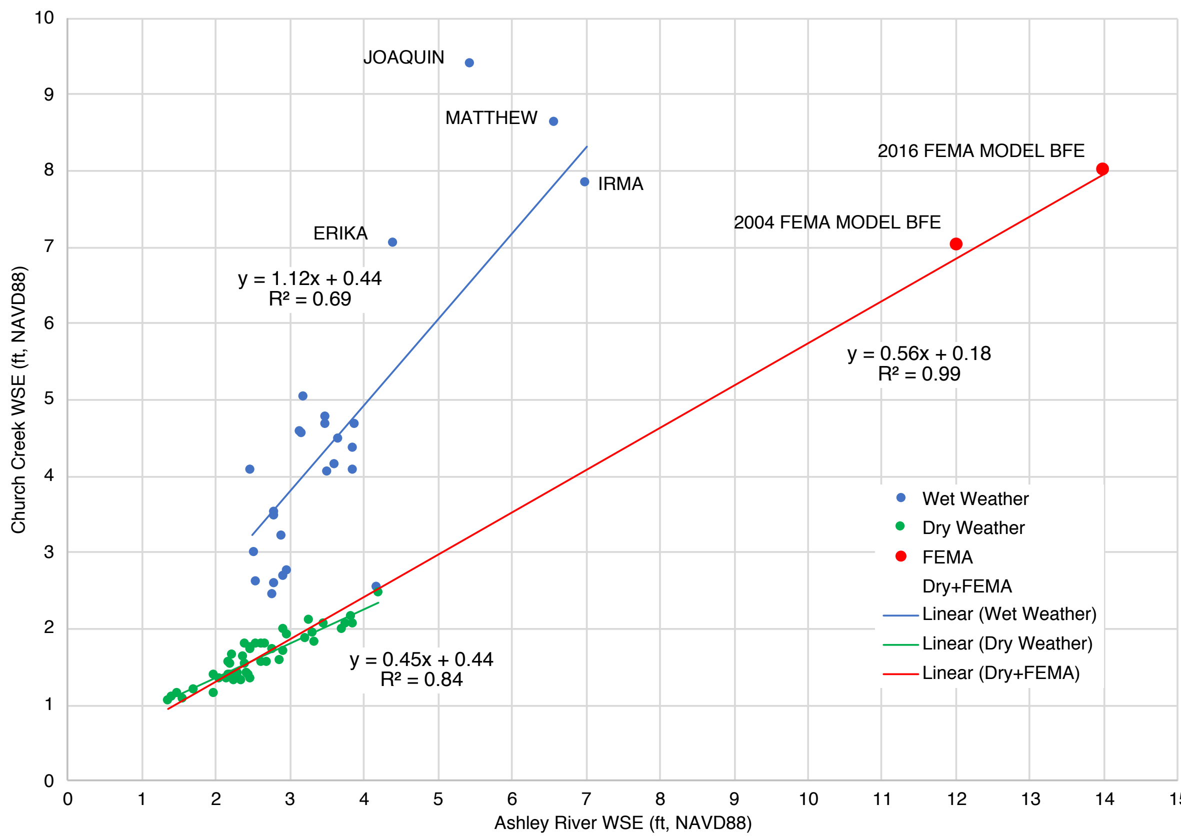

Figure 3.4: Ashley River WSE versus Church Creek WSE with data separated into "Wet Weather" and "Dry Weather" data sets (see Section 3.2.2). Wet Weather Church Creek WSE data were collected from the NWS-Woolpert BEES1 gage. Dry Weather Church Creek WSE data were collected from the Robinson Design Engineers' gage at Crosstowne Christian Church. FEMA WSE data were collected from FIRMs 45019C0480J and 45019C0479K.

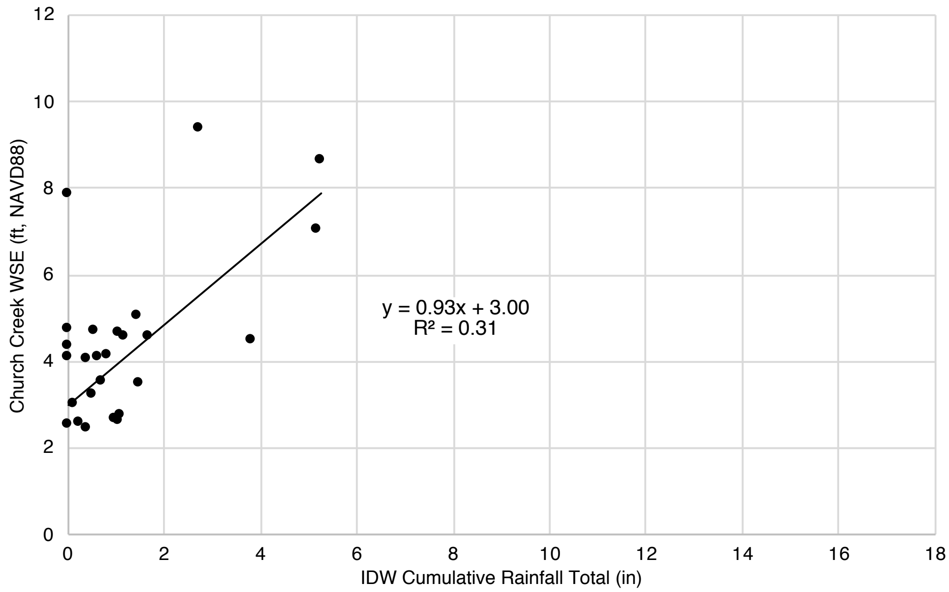

Figure 3.5: 24-hour IDW Cumulative Rainfall Total versus Church Creek WSE. Daily rainfall data were collected from NWS stations KCHS and KCXM. Church Creek WSE data were collected from the NWS-Woolpert BEES1 gage.

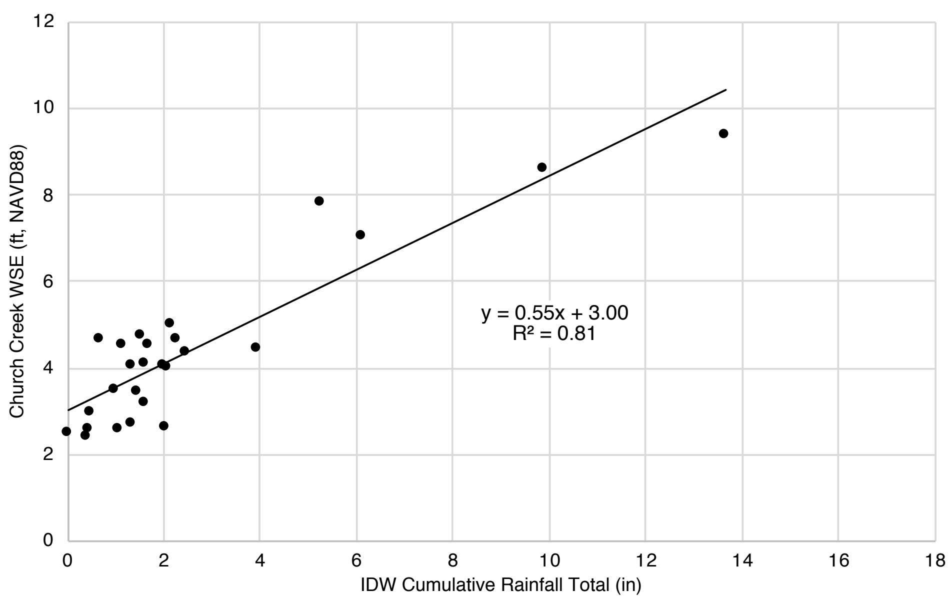

Figure 3.6: 48-hour IDW Cumulative Rainfall Total versus Church Creek WSE. Daily rainfall data were collected from NWS stations KCHS and KCXM. Church Creek WSE data were collected from the NWS-Woolpert BEES1 gage.

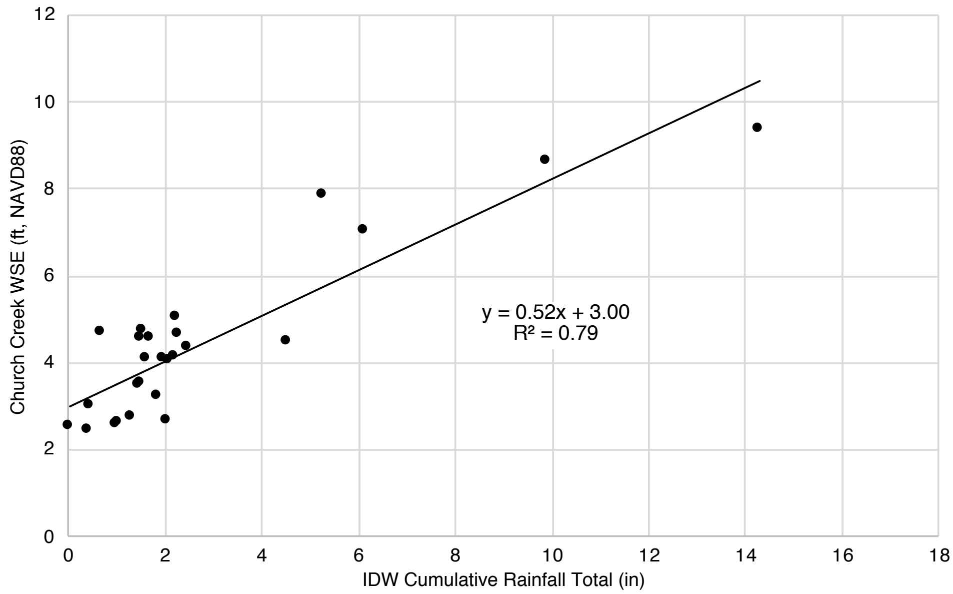

Figure 3.7: 72-hour IDW Cumulative Rainfall Total versus Church Creek WSE. Daily rainfall data were collected from NWS stations KCHS and KCXM. Church Creek WSE data were collected from the NWS-Woolpert BEES1 gage.

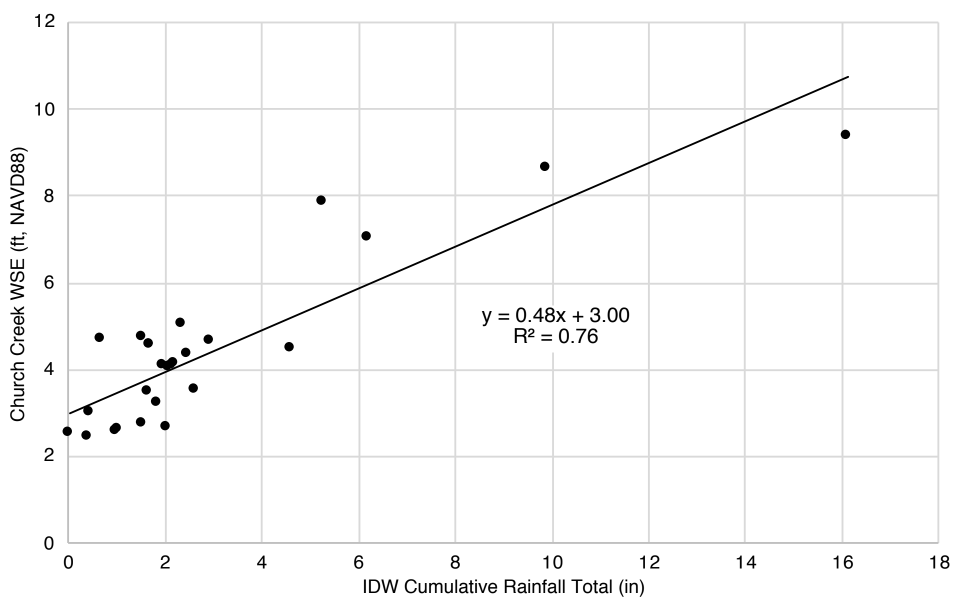

Figure 3.8: 96-hour IDW Cumulative Rainfall Total versus Church Creek WSE. Daily rainfall data were collected from NWS stations KCHS and KCXM. Church Creek WSE data were collected from the NWS-Woolpert BEES1 gage.

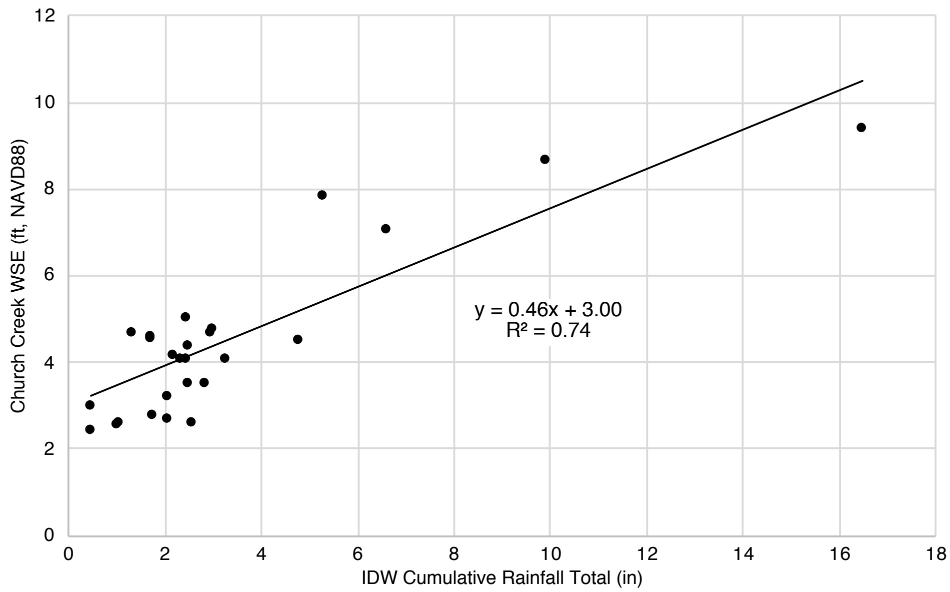

Figure 3.9: 120-hour IDW Cumulative Rainfall Total versus Church Creek WSE. Daily rainfall data were collected from NWS stations KCHS and KCXM. Church Creek WSE data were collected from the NWS-Woolpert BEES1 gage.

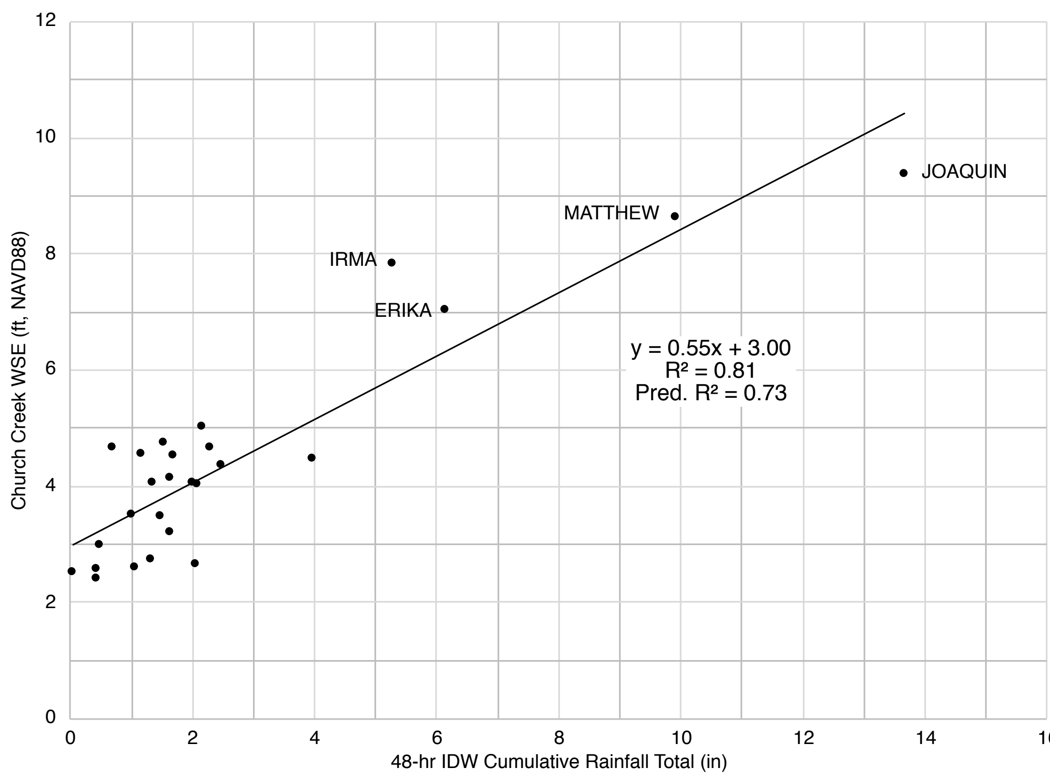

Figure 3.10: 48-hour IDW Cumulative Rainfall Total versus Church Creek WSE. Daily rainfall data were collected from NWS stations KCHS and KCXM. Church Creek WSE data were collected from the NWS-Woolpert BEES1 gage.

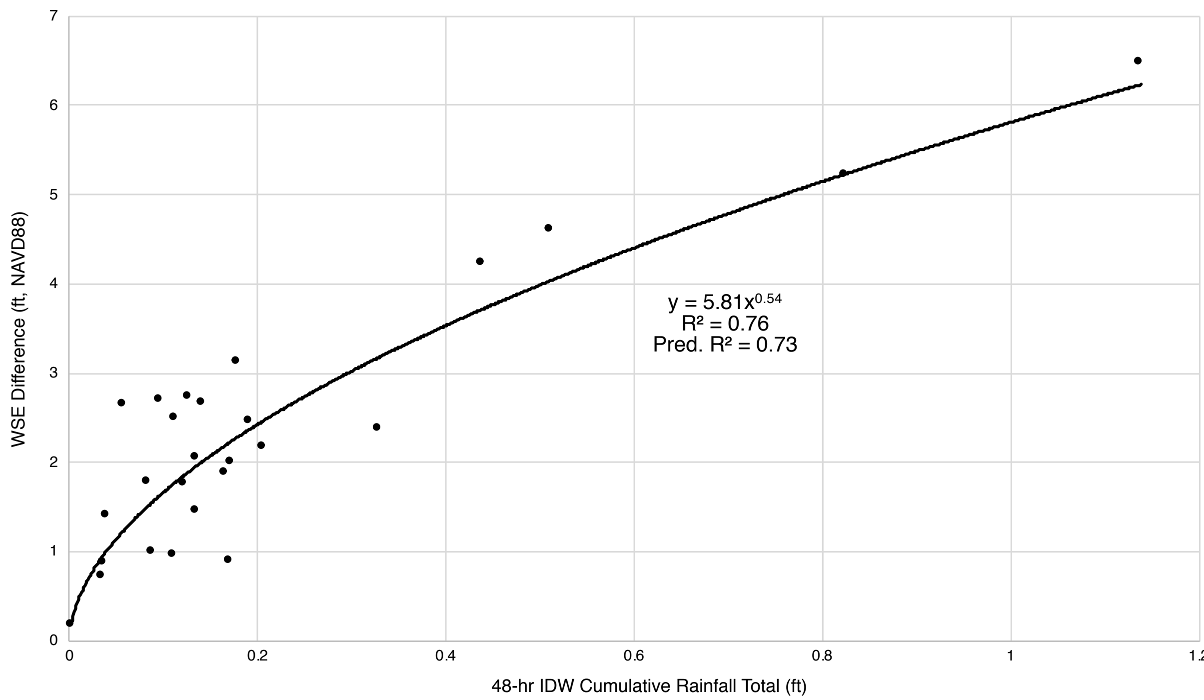

Figure 3.11: Additional Stage Function Relating 48-hour Cumulative Rainfall Total and Observed – Predicted WSE Difference in Church Creek. Daily rainfall data were collected from NWS stations KCHS and KCXM. Observed Church Creek WSE data were collected from the NWS-Woolpert BEES1 gage. Predicted Church Creek WSE data were generated based on the Dry Weather regression relationship listed as Equation 1 in Section 3.2.2.

This shows that the dry weather relationship between the Ashley and Church Creek falls apart strikingly in hurricanes and wet weather, meaning the 48-hour rainfall make a lot more flooding.

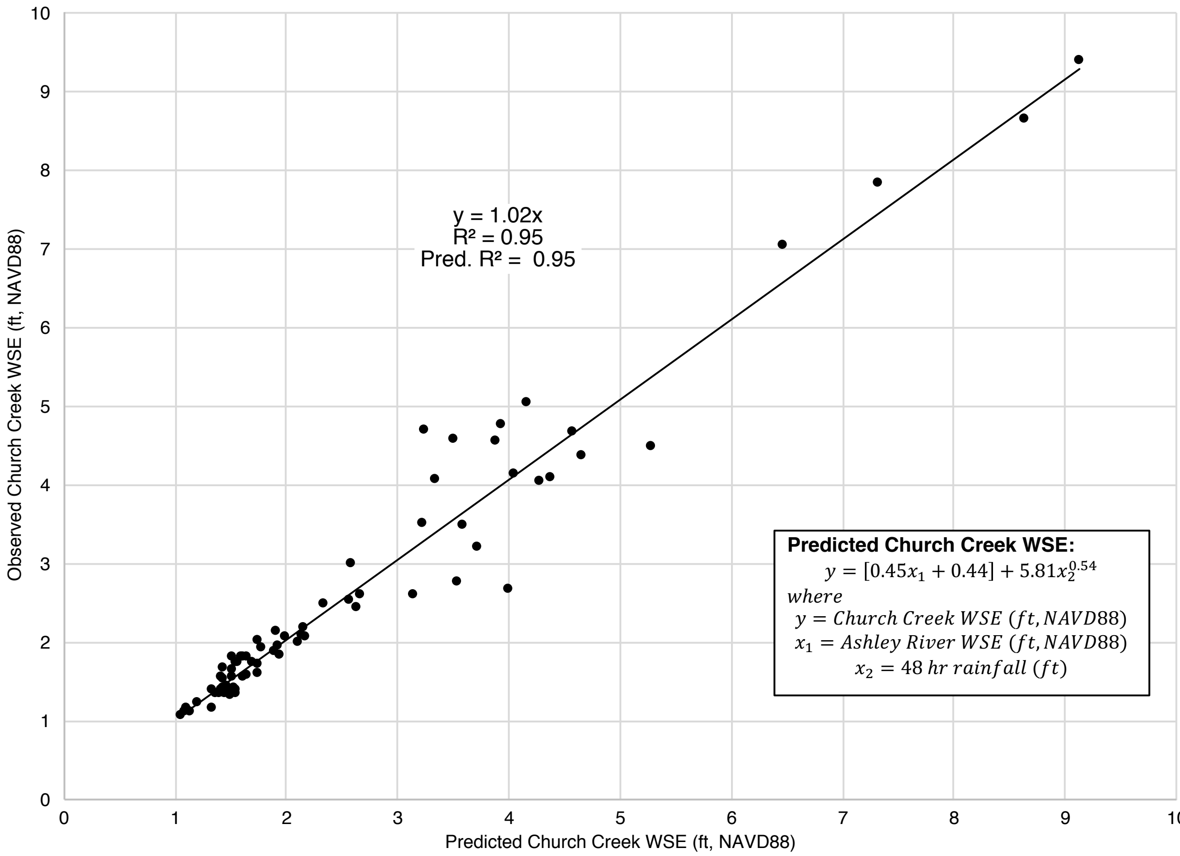

Figure 3.12: Goodness of Fit for Compound Flooding Function for Predicting Church Creek Peak WSE. Observed Church Creek WSE data were collected from the NWS-Woolpert BEES1 gage and the Robinson Design Engineers' gage at Crosstowne Christian Church.

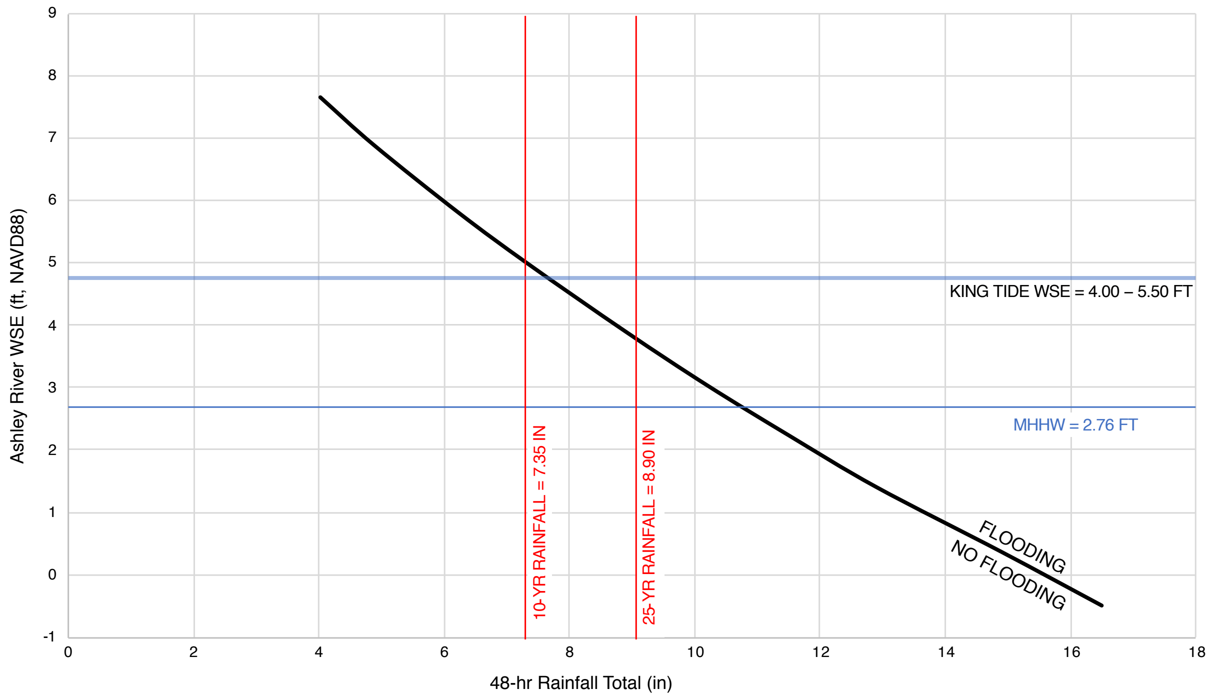

Figure 4.1: Predicted flooding of Crosstowne Christian Church building based on scenarios of 48-hour cumulative rainfall total and peak WSE in the Ashley River.

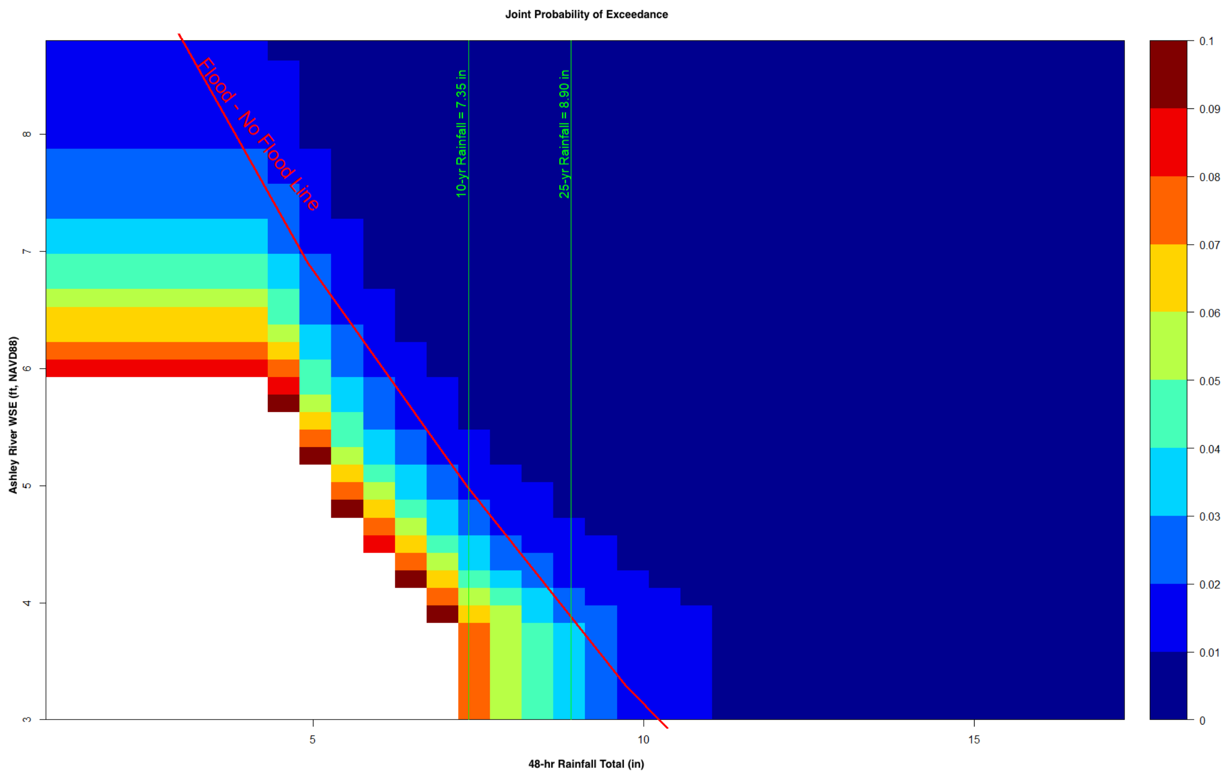

Figure 4.2: Joint probability of annual exceedance for predicted flooding of Crosstowne Christian Church building based on scenarios of 48-hour cumulative rainfall total and peak WSE in the Ashley River (color gradient expresses annual probability of exceedance).

Robinson Design Engineers
10 Daniel Street
Charleston, SC 29407
www.robinsondesignengineers.com