Impacts of Hydrologic and Hydraulic Model Connection Schemes on Flood Simulation and Inundation Mapping in the Tar River Basin

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Thesis submitted in partial fulfillment of the requirements for the degree of Master of Science in the Department of Civil and Environmental Engineering in the Graduate School of Duke University

2012
ABSTRACT

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Abstract

Flooding is the leading cause of losses from natural disasters in the United States, responsible for over 140 deaths a year (United States Geological Survey [USGS], 2006 and 2012). Dynamic inundation mapping, or using continuous modeling in conjunction with a GIS-system to create maps of flooded areas under different scenarios, has the possibility to better predict flooding by taking into account environmental conditions preceding and during the flood (such as soil moisture, rainfall spatial heterogeneity and timing). This study examines the effect of hydrologic-hydraulic model linkage schemes in generating inundation maps and understanding the uncertainties involved, with an eye towards improving operational use.

The Hydrology Laboratory’s Research Distributed Hydrologic Model (HL-RDHM) was used to generate inputs to the Army Corps of Engineers Hydrologic Engineering Center’s River Analysis System (HEC-RAS) hydraulic model for two simulation time periods (September to December 1999 for the simulation of flooding due to Hurricane Floyd and from June to October 2006 for flooding due to Tropical Storm Alberto) in the Lower Tar River sub-basin for four different hydrologic-to-hydraulic model connection scenarios of increasing complexity and increasing number of inflow points. The HEC-GeoRAS, an extension built for use with ArcGIS to pre-process and post-process HEC-RAS geometric data, was used to generate inundation maps from the maximum water surface produced during each simulation time period.
The stage and flow simulated by the hydraulic model were compared to observed data recorded by USGS gauges and the goodness of simulation was assessed using the Nash-Sutcliffe Efficiency (NSE). The most detailed scenario slightly outperformed the less detailed scenarios for simulation of flow and stage at gauges on the main stem of the Tar River for both simulation periods (NSE >0.89). No scenario did a good job of simulating the stage or flow on the tributaries of the Tar River (NSE <0.52). For the 1999 simulation, the flood elevation predicted by the inundation map was compared to high water marks collected by the Federal Emergency Management Agency (FEMA) and the USGS after Hurricane Floyd. All scenarios generally predicted flooding well, with mean error of around 1 foot for the less complex, more calibrated models, around 1-1.5 feet for the more complex models, and mean absolute error for all scenarios around 3.5-4 feet. The time required to generate these simulations and maps was not greatly increased with increasing modeling complexity, though the models did require careful set-up to ensure stability. The results of this work suggest modest benefits in accuracy of modeling stage and flow by increasing the number of inputs from a hydrologic model to a hydraulic model. Mapping results show limited statistical gain, but some importance of a multi-stem model in order to create inundation maps which include flooding effects on tributaries. Lack of observed data to provide boundary conditions, and reliance on input data with its own set of uncertainties seem to dominate uncertainty in tributary models unforced by observed data.
Contents

Abstract ........................................................................................................................................ iv
List of Tables ................................................................................................................................. ix
List of Figures .............................................................................................................................. x
Acknowledgements .................................................................................................................... xiii
1. Introduction ................................................................................................................................. 1
   1.1 Background ........................................................................................................................... 1
   1.2 River Modeling and Inundation Mapping ............................................................................ 2
   1.3 Current Efforts ..................................................................................................................... 3
      1.3.1 Current Modeling Efforts ........................................................................................... 3
      1.3.2 Current Mapping Techniques ...................................................................................... 4
   1.4 Needs and Shortcomings of Current Methods ................................................................. 4
      1.4.1 Benefits of Hydraulic Models as Compared to Hydrologic Models ....................... 4
      1.4.2 Limitation of Models .................................................................................................. 5
      1.4.3 Issues with Static Mapping ......................................................................................... 6
   1.5 Objective ............................................................................................................................... 6
2. Literature Review ......................................................................................................................... 7
   2.1 Hydrologic Models ............................................................................................................... 7
      2.1.1 HL-RDHM ................................................................................................................. 8
   2.2 Hydraulic Models ............................................................................................................... 9
      2.2.1 Mathematical Foundations ......................................................................................... 10
      2.2.2 HEC-RAS .................................................................................................................. 11
2.3 Importance of Lateral Inflows ................................................................. 14
2.4 Coupling of Hydrologic and Hydraulic Models ........................................ 14
2.5 Inundation Mapping .............................................................................. 16
2.6 Previous Studies of the Tar River Basin .................................................. 17

3. Methods ................................................................................................. 19
  3.1 Study Area .......................................................................................... 19
  3.2 Study Time Period ............................................................................. 21
  3.3 Models Used ....................................................................................... 23
    3.3.1 HL-RDHM ..................................................................................... 24
    3.3.2 HEC-RAS .................................................................................... 25
  3.4 Processing Steps ................................................................................ 26
    3.4.1 Georeferencing and creation of a multi-stem hydraulic model .......... 26
    3.4.2 Creation of inputs to HEC-RAS .................................................... 30
  3.5 Scenario Description .......................................................................... 32
  3.6 Adjustments ....................................................................................... 40
  3.7 Generating Inundation Maps .............................................................. 41
  3.8 Validation Data .................................................................................. 41

4. Results .................................................................................................... 45
  4.1 Stage and Discharge Validation for Hurricane Floyd ................................ 45
  4.2 Flooding Extents Modeled for Hurricane Floyd ...................................... 49
  4.3 High Water Mark Validation .................................................................. 57
  4.4 Validation for 2006 (TS Alberto) Time Period ....................................... 64
  4.5 Flooding Extents for 2006 Period .......................................................... 70
List of Tables

Table 1: USGS Gauges for Observed Data........................................................................ 42

Table 2: Table of NSE, NSH, and % RMSE for the 1999 simulation period at Greenville, NC................................................................................................................... 46

Table 3: Table of NSE, NSH, and % RMSE for the 1999 simulation period on Conetoe Creek, NC. ......................................................................................................................... 48

Table 4: Mean Error and Mean Absolute Error in height of simulated high water mark by scenario, same 74 points ........................................................................................................... 62

Table 5: Mean Error in height of simulated high water mark by reach and scenario, same 74 points ............................................................................................................................................... 62

Table 6: Mean Absolute Error in height of simulated high water mark by reach and scenario, same 74 points .................................................................................................................. 62

Table 7: Mean Error and Mean Absolute Error in height of simulated high water mark by scenario, all available points for each scenario ........................................................................................................... 63

Table 8: Mean Error in height of simulated high water mark by reach and scenario, all available points for each scenario ................................................................................................................... 63

Table 9: Mean Absolute Error in height of simulated high water mark by reach and scenario, all available points for each scenario ........................................................................................................... 63

Table 10: Table of NSE, NSH, and % RMSE for the 2006 simulation period at Rock Springs, NC .......................................................................................................................................................... 65

Table 11: Table of NSE, NSH, and % RMSE for the 2006 simulation period at Greenville, NC ........................................................................................................................................................................ 67

Table 12: Table of NSE, NSH, and % RMSE for the 2006 simulation period at Grimesland, NC ........................................................................................................................................................................ 68

Table 13: Table of NSE, NSH, and % RMSE for the 2006 simulation period on Town Creek ........................................................................................................................................................................ 69

Table 14: Table of NSE, NSH, and % RMSE for the 2006 simulation period at Tranters Creek, NC ........................................................................................................................................................................ 70
List of Figures

Figure 1: The extent of the main stem model from Tarboro to Washington, NC is shown in dark blue, with the tributaries modeled shown in purple. The current basin forecast extent is shown in orange. ................................................................. 20

Figure 2: Precipitation in North Carolina due to Hurricane Floyd. Some of the heaviest precipitation occurred in parts of the Tar River basin on September 16, 1999. ...................... 21

Figure 3: Precipitation in North Carolina due to Tropical Storm Alberto, which dumped heavy rains over the central region of the state on June 14, 2006. ................................. 22

Figure 4: Precipitation in North Carolina from Tropical Storm Ernesto, which made landfall on August 31, 2006. ................................................................................... 23

Figure 5: Flowchart of models, programs, and their generated outputs used in this study. ......................................................................................................................... 24

Figure 6: The image on the left is the tributary model as obtained from NCFMP, which is not suitable for inundation mapping. The image on the right is the same model after the georeferencing process ......................................................... 27

Figure 7: The full georeferenced Tar River model as viewed in HEC-RAS, with all the channels and cross sections ................................................................. 29

Figure 8: Example of HRAP grid for RDHM, for Conetoe Creek showing different types of input points as distinct shapefiles .............................................. 31

Figure 9: A conceptual map of the Lower Tar sub-basin with inputs configured as Scenario 1 ........................................................................................................ 33

Figure 10: A conceptual map of the Lower Tar sub-basin with inputs configured as Scenario 2 ........................................................................................................ 35

Figure 11: A conceptual map of the Lower Tar sub-basin with inputs configured as Scenario 3 ........................................................................................................ 37

Figure 12: A conceptual map of the Lower Tar sub-basin with inputs configured as Scenario 4 ........................................................................................................ 39

Figure 13: Stage generated from scenarios 1, 2, 3, and 4 as compared to partial USGS observed stage at Greenville, NC from September to November 1999 .................. 45
Figure 14: Streamflow generated from scenarios 1, 2, 3, and 4 as compared to partial USGS observed stage at Greenville, NC from September to November 1999. 

Figure 15: Stage generated from scenarios 3 and 4 as compared to partial USGS observed stage at Conetoe Creek from September to November 1999. The pink line shows a post-Floyd high water mark.

Figure 16: Streamflow generated from scenarios 3 and 4 as compared to partial USGS observed stage at Conetoe Creek from September to November 1999.

Figure 17: Profile plot generated in HEC-RAS for the highest water surface at each cross section as generated from Scenario 1 for Hurricane Floyd.

Figure 18: Profile plot generated in HEC-RAS for the highest water surface at each cross section as generated from Scenario 2 for Hurricane Floyd.

Figure 19: Profile plot generated in HEC-RAS for the highest water surface at each cross section as generated from Scenario 3 for Hurricane Floyd.

Figure 20: Profile plot generated in HEC-RAS for the highest water surface at each cross section as generated from Scenario 4 for Hurricane Floyd.

Figure 21: Boundaries of flooding generated by scenarios 1 and 2 Hurricane Floyd (September 1999).

Figure 22: Boundaries of flooding generated by scenarios 2 and 3 Hurricane Floyd (September 1999).

Figure 23: Boundaries of flooding generated by scenarios 3 and 4 Hurricane Floyd (September 1999).

Figure 24: Comparison of model simulated high water at measured high water marks after Hurricane Floyd for Scenario 1.

Figure 25: Comparison of model simulated high water at measured high water marks after Hurricane Floyd for Scenario 2.

Figure 26: Comparison of model simulated high water at measured high water marks after Hurricane Floyd for Scenario 3.

Figure 27: Comparison of model simulated high water at measured high water marks after Hurricane Floyd for Scenario 4.
Figure 28: Stage generated from scenarios 1, 2, 3 and 4 as compared to USGS observed stage at Rock Springs, NC from June to September 2006. ....................................................... 65

Figure 29: Stage generated from scenarios 1, 2, 3 and 4 as compared to USGS observed stage at Greenville, NC from June to September 2006. .................................................. 66

Figure 30: Streamflow generated from scenarios 1, 2, 3 and 4 as compared to USGS observed stage at Greenville, NC for June to September 2006. ........................................... 66

Figure 31: Stage generated from scenarios 1, 2, 3 and 4 as compared to USGS observed stage at Grimesland, NC from June to September 2006. ................................................. 68

Figure 32: Stage generated from scenarios 3 and 4 as compared to USGS observed stage at Town Creek from June to September 2006. ....................................................... 69

Figure 33: Stage generated from scenarios 3 and 4 as compared to USGS observed stage at Tranter Creek from June to September 2006. .................................................... 70

Figure 34: Profile plot generated in HEC-RAS for the highest water surface at each cross section as generated from Scenario 1 from June to September 2006. ............................. 72

Figure 35: Profile plot generated in HEC-RAS for the highest water surface at each cross section as generated from Scenario 2 from June to September 2006. ....................... 73

Figure 36: Profile plot generated in HEC-RAS for the highest water surface at each cross section as generated from Scenario 3 from June to September 2006. ...................... 74

Figure 37: Profile plot generated in HEC-RAS for the highest water surface at each cross section as generated from Scenario 4 from June to September 2006 ....................... 75

Figure 38: Boundaries of flooding generated by scenarios 1 and 2 for the time period from June to September 2006. .......................................................... 76

Figure 39: Boundaries of flooding generated by scenarios 2 and 3 for the time period from June to September 2006. ...................................................... 77

Figure 40: Boundaries of flooding generated by scenarios 3 and 4 for the time period from June to September 2006. ...................................................... 78
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1. Introduction

1.1 Background

Flooding is the leading cause of losses from natural disasters in the United States, accounting for over 75% of declared Federal disasters (United States Geological Survey [USGS], 2012). These floods cause around 140 deaths each year and an estimated $6 billion in losses. Although the number of deaths due to floods has declined over the last 50 years, economic losses have increased due to population growth and urbanization, especially in coastal areas (USGS, 2006). In 2010, over half of the US population (52%) lived in coastal communities, a figure which is expected to increase 9% by 2020 (National Oceanic and Atmospheric Administration [NOAA], 2012). This projected increase in growth, combined with recent large flood events in the United States and worldwide, have led many countries to place an increased focus on mitigation of and preparing for flooding and its associated hazards (Brocca, Melone & Moramarco, 2011).

The most common causes of flooding in the Eastern United States are hurricanes and storms (USGS, 2006). Factors that affect whether rainfall from these systems will cause flooding include the conditions which precede the storm, including soil moisture conditions, and the characteristics of the storm itself, especially the rainfall duration and extent compared to the size of the basin (Bales, Oblinger & Sallenger, Jr., 2000). In order to understand the timing and magnitude of floods, as well as for contingency planning and management decisions, river modeling and inundation mapping have become increasingly important tools.
1.2 River Modeling and Inundation Mapping

Inundation mapping is a way to represent flooding for use in determining current flooding extent or areas forecasted to flood. This is useful information for emergency managers, rescue workers, and citizens who would be impacted by possible evacuations. This is commonly performed by linking GIS and hydrologic/hydraulic models, which then can be used to display the flood extent (Colby, Mulcahy & Wang, 2000). Current engineering practice is to use a rainfall-runoff model to simulate a design storm of a given recurrence interval and to convert the peak flow to stage using a steady state hydraulic model, assuming the design storm and the generated water level will have the same return period, which may not be true depending on other environmental or storm-specific conditions (Bradley, Cooper, Potter & Price, 1996). This peak stage is used to determine water levels which are used to generate a map of flooding for the design event.

Flood mapping for insurance planning, to “attempt to prevent or reduce flood damages” has been carried out by the National Flood Insurance Program of Federal Emergency Management Agency (FEMA) for over 40 years (Bales & Wagner, 2009). These maps identify areas which correspond to a few given exceedance levels, or the likelihood that an area will be flooded during a storm event. Mapping by the National Weather Service occurs at distinct forecast points and reflect a large range of multiple water levels and discharge values. These static maps are compiled to create a library which encompasses water levels considered minor flooding up to the largest flood of record, which can then be used for flood warning purposes (Real-Time (Dynamic) Inundation Mapping Evaluation (R-Time) Team, 2007).
1.3 Current Efforts

1.3.1 Current Modeling Efforts

The National Weather Service (NWS) of the National Oceanic and Atmospheric Administration (NOAA) produces forecasts at around 4,000 points across the United States, generated by 13 River Forecast Centers (RFCs). These forecasts and river observations are made available to the public through the Advanced Hydrologic Prediction Service, or AHPS at water.weather.gov/ahps. To create these forecasts, the NWS uses hydrologic models, and, to a more limited extent, hydraulic models. However, the extent of these predictions in coastal areas is quite limited. According to Van Cooten et al. (2011), of the 142 Atlantic and Gulf of Mexico coastal drainage areas, or the downstream-most basin areas that drain directly to the ocean or an estuary, over 90% do not receive any forecast information about water level or streamflow from the NWS, partially because of the burden that implementing new hydrologic models in these areas would entail, let alone hydraulic models which could accommodate tidal or backwater influences.

However, this absence does provide a big opportunity for service expansion. With this in mind, the NWS currently models 5,500 miles of river with hydraulic models. For its ease of use by forecasters and wide support among the engineering community, these models have been converted to HEC-RAS, a dynamic hydraulic model which has the ability to include several points of interest for forecasting, less reliance on observed field data as compared hydrologic models, ease in calibrating the model to reflect seasonal hydraulic property changes, and the ability to estimate values for future floods (Reed, 2010).
1.3.2 Current Mapping Techniques

Flood forecast maps provide valuable information to emergency managers and the public. The NWS currently provides static flood map libraries tied to river forecasts at about 65 points in the United States. However, many more communities could benefit from expanded flood forecast mapping services. Dynamic modeling and mapping is a promising approach to efficiently expand forecast mapping services. Dynamic models may be used to extend maps farther distances from forecast points and to tributaries where services are desired.

1.4 Needs and Shortcomings of Current Methods

Current techniques have many benefits, including that hydrologic models are widely applied, well-tested, and integrated with current forecasting systems. Static flood maps are quickly called upon, easily distributed, always available, and provide a useful resource for planning (Real-Time (Dynamic) Inundation Mapping Evaluation (R-Time) Team, 2007). However, there are many drawbacks and areas of current methods that could be improved upon by further studies. A few of these are outlined in the following section.

1.4.1 Benefits of Hydraulic Models as Compared to Hydrologic Models

Routing techniques may be classified into two major categories: simple catchment hydrologic routing involving the continuity equation, and more complex channel hydraulic routing requiring both the continuity and momentum equations. However, most comprehensive hydrologic models today incorporate hydraulic routing components.
Hydrologic routing tends to include more components (e.g., rainfall, infiltration, evapotranspiration, balancing of inflow, outflow and storage through various phases). Further distinctions are made in Section 2.

Although numerous hydraulic models are being used by the NWS for forecasting, the current extent of this practice is only a fraction of the estimated 26,000 miles which would be predicted to benefit from such hydraulic modeling. However, hydraulic models are not more widely implemented within the NWS forecasting system for a number of reasons. The three mentioned by Reed (2010) are that forecasters are not convinced that hydraulic modeling is worth the financial investment required to set-up and maintain these models, secondly, that the complexities involved in setting up and model and achieving stability give hydraulic models a reputation of being difficult to use, and third, that many forecasters may have developed their own techniques in compensating for the shortcomings of hydrologic models.

1.4.2 Limitation of Models

One of the big challenges in implementing tributary hydraulic models is that accurate hydrologic inflow forecasts may not be available. NWS River Forecast Centers (RFCs) primarily run lumped hydrologic models, producing flow information only at basin outlets. Implementing multi-reach hydraulic models on tributaries will require use of a distributed hydrologic model to provide boundary conditions.

Models must be chosen with their limitations in mind. Model complexity, while increasing resolution and possibly increasing accuracy, must be balanced with the data and computing requirements. Additionally, for operational use, the training and system implementation cost and time required are also factors in model selection and use.
1.4.3 Issues with Static Mapping

Currently NWS flood forecast maps exist only in static map libraries. These are created for the areas near 65 or so forecast points. These maps are limited in application; each map only covers a river length area of around 4 kilometers. Their effectiveness decreases with distance from the forecast point, especially in areas where high-resolution topographic data is not available or where land cover changes too rapidly in time to be reflected in maps, they may not be implemented in regions with tributary inflows or possible backwater effects, or where there are tidal influences. These are some of many reason dynamic mapping is being explored as a way of expanding the coverage of inundation maps (Real-Time (Dynamic) Inundation Mapping Evaluation (R-Time) Team, 2007).

1.5 Objective

The purpose of this study is to explore the effect of hydrologic-hydraulic model linkage schemes in generating inundation maps and understanding uncertainty involved with an eye towards improving operational use. Although numerous studies have looked at the uncertainty involved in mapping and ways to link hydrologic and hydraulic models, the most widely-used framework being used for nationwide operational forecasting has not been tested in such a way as to quantify uncertainty involved with generation of inundation maps, with the possibility of improving current or future modeling and mapping efforts. The goals of this study were to create various scenarios to represent possible linkage schemes representing different levels of detail at which distributed hydrologic model outputs are used to create boundary conditions for hydraulic models, to use these different model scenarios to evaluate the accuracy of flow simulations and
inundations maps as compared to an historic flood, and evaluate the suitability of the model in simulating other floods outside the than the calibration period.

2. Literature Review

2.1 Hydrologic Models

Hydrologic models rely on the parameterization of watershed properties and rainfall patterns and depths to produce a flood hydrograph of discharge at discrete time steps. These models have become widely used in flood forecasting, stream flow prediction, and to quantify effects of climate change, land use impacts or other spatially distributed properties. However, their limited routing methods do have some drawbacks in simulating flows in large watersheds (Mai, 2009).

Two main types are lumped hydrologic models and distributed hydrologic models. Lumped hydrologic models only consider generalized watershed characteristics, and are only able to produce streamflow hydrographs at the basin outlet. Distributed hydrologic models, on the other hand, can accommodate spatial heterogeneity of land use, soil properties, rainfall, and other inputs. This quality allows for higher resolution simulations and predictions which can improve operational forecasts, however, with this higher resolution comes more complexity and uncertainty (Carpenter & Georgakakos, 2006). The benefit of a distributed model as compared to a lumped hydrologic model is that once a distributed model is calibrated, it can be used to predict future events under changed conditions (such as different land use or spatially varying environmental parameters affected by climate change) where a lumped model would have to be completely reformulated (Nunes Correia, Castro Rego, da Graça Saraiva & Ramos, 1998).
Commonly used hydrologic models include the Hydrologic Engineering Center – Hydrologic Modeling System (HEC-HMS), the Hydrologic Simulation Program-FORTRAN (HSPF), Soil and Water Assessment Tool (SWAT), and MIKE-SHE (Bertie, et al., 2011; Debele et al., 2006; Haghizadeh et al., 2012; Lian et al., 2007).

2.1.1 HL-RDHM

The Hydrology Laboratory – Research Distributed Hydrologic Model, or HL-RDHM, is a “flexible, stand-alone tool for distributed hydrologic modeling research and development” developed by the Hydrologic Science and Modeling Branch (HSMB) of the NWS Office of Hydrologic Development (OHD) Hydrology Laboratory (HL) and is used to validate scientific techniques which then may become part of the operational software of the NWS (HL-RDHM user manual v. 3.0.0, 2009).

The HL-RDHM calculates runoff based on the distributed procedures of the Sacramento Soil Moisture Accounting model with the Heat Transfer component (SAC-SMA-HT). The forcings of SAC-SMA-HT include gridded precipitation and temperature data, along with daily or monthly potential evapotranspiration. The model calculates water storage within the soil profile in generalized upper and lower zones of soil moisture, which are further classified into free water and tension water. Tension water in the upper zone is water which when added to dry soil, does not leave the zone. Water in the upper zone enters from precipitation through permeable soil areas. Lower zone tension water is considered to be water which is held by molecular forces to soil particles (NWSRFS User’s Manual Release 81, 2005).

The effect of evapotranspiration on soil moisture is included in the model by calculating a balance of evapotranspiration demand with available soil tension water in
the upper zone, with excess moisture supplied by water in the lower zone. Free water can also be lost from the upper zone to the lower zone based on the nonlinear percolation rate

\[ I = I_0 + I_{\max}\left(1 - \frac{S_{f,2} + S_{f,3} + S_{f,4}}{S_{f,2} + S_{f,3} + S_{f,4}}\right)^\beta \frac{S_{f,1}}{S_{f,1}} \]  

(1)

where \( I_0 \) is the minimum percolation rate under fully saturated conditions, \( I_{\max} \) is the maximum percolation rate, \( \beta \) is a curve-fitting exponent, \( S_{f,i} \) is the free water storage capacity for the upper zone (1), the fast baseflow component of the lower zone (2), or the slow baseflow component of the lower zone (3), and \( S_{t,i} \) is the tension water storage for these same zones (Koren, Smith, Cui & Cosgrove, 2007).

When tension water in the upper zone is at capacity, runoff can be produced through five methods: from impervious surfaces or direct runoff from water flooded areas, from surface runoff when upper zone free water is also full and when precipitation exceeds percolation rates, interflow from lateral upper zone drainage, or from primary baseflow (RDHM User Manual v. 3.0.0, 2009). This runoff can be conceptualized as \( q_i \)

\[ q_i = \frac{r_i S_{f,i}}{\Delta t} \]  

(2)

where \( S_{f,i} \) represents the linear free water reservoirs for the i different water storage zones mentioned previously, \( r_i \) is the constant depletion coefficient for each reservoir and \( \Delta t \) is the time interval (Koren, et al., 2007). More details regarding these concepts can be found in the HL-RDHM user’s manual v. 3.0.0 (2009).

**2.2 Hydraulic Models**

Hydraulic models are based on the solutions to the St. Venant equations (described in more detail in the next section) to calculate open channel flow. The most
commonly used of these models are either one-dimensional or two-dimensional. Even one-dimensional models require inputs of cross-sectional geometry, roughness, and boundary conditions, while fully two-dimensional models require inputs of topography, roughness, wind resistance, and boundary conditions for solution of the 2-D St. Venant equations, typically via a finite element approach. Mai (2009) describes how, although 2-D hydraulic models provide satisfactory results, the data coverage requirement, the inability to rapidly adjust these models to assess catchment or river channel change, and the computational demands of 2-D models all serve as drawbacks to their implementation. Instead, a 1-D model is often used to determine stage, which is then interpolated in a GIS system to produce a flood map.

Widely used hydraulic models include Water Quality Analysis Simulation Program (WASP), CE-QUAL-W2, Environmental Fluid Dynamics Code (EFDC), EPD-RIV1, Hydrologic Engineering Center – River Analysis System (HEC-RAS), MIKE11 (a 1-D model), MIKE21 (2-D model), and SOBEK (Bertie, et al., 2011; Debele et al., 2006; Haghizadeh et al., 2012; Horritt & Bates, 2002).

2.2.1 Mathematical Foundations

The underpinnings of these models are the conservation of mass and momentum for a shallow water control volume. In one-dimension, these are expressed by the continuity equation:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0$$

(3)

where Q is the discharge, t is time, x is the channel distance, and A is the cross-sectional flow area, as well as the momentum equation:
\[ \frac{1}{A} \frac{\partial Q}{\partial t} + \frac{1}{A} \frac{\partial}{\partial x} \left( \frac{Q^2}{A} \right) + g \frac{\partial y}{\partial x} - g (S_0 - S_f) = 0 \]  

(4)

where \( g \) is gravitational acceleration, \( S_0 \) is the channel bottom slope and \( S_f \) is the friction slope.

These two equations taken together are the St. Venant equations, upon which hydraulic models are based. The first term is the local acceleration, the second accounts for convective acceleration, the third is the pressure force term, and the fourth and fifth are the gravity and friction force terms, respectively. Those models which simplify these equations to only the rightmost two terms (gravity and friction forces) are known as kinematic wave models. Diffusion wave models are those which include all but the inertial terms. Those models which solve the full St. Venant equations are known as dynamic wave models (Bedient, Huber & Vieux, 2008).

2.2.2 HEC-RAS

HEC-RAS is a product of the United States Army Corps of Engineers Hydrologic Engineering Center and is “an integrated system of software…comprised of a graphical user interface (GUI), separate hydraulic analysis components, data storage and management capabilities, graphics and reporting facilities” (HEC-RAS User’s Manual Version 4.1, 2010). The program code is public domain software available as a free download from the HEC website.

HEC-RAS computes hydraulic analysis components for steady flow which involves solution of the one-dimensional energy equation, unsteady flow through the full solution of the 1-D St. Venant equations or dynamic wave, and sediment transport.
Using implicit finite differences, Equation 3 is approximated for channel and floodplain (out-of-bank) flow as

$$\frac{\Delta Q_c}{\Delta x_c} + \frac{\Delta A_c}{\Delta t} = q_f$$  \hspace{1cm} (5)

for the exchange of water from floodplain ($q_f$), where $Q_c$ is channel flow, $A_c$ is channel; cross-sectional area, $x_c$ is the distance along the channel, and $t$ is time, and

$$\frac{\Delta Q_f}{\Delta x_c} + \frac{\Delta A_f}{\Delta t} + \frac{\Delta S}{\Delta t} = q_c + q_l$$  \hspace{1cm} (6)

for the exchange of water from channel ($q_l$), where $Q_f$ is flow in the floodplain, $S$ is storage in non-conveying portions of the floodplain, and $q_l$ is the lateral inflow per unit distance. These equations are then combined and rearranged to give

$$\Delta Q + \frac{\Delta A}{\Delta x} \Delta x + \frac{\Delta A_f}{\Delta t} \Delta x_f + \frac{\Delta S}{\Delta t} \Delta x_f - \overline{Q_l} = 0$$  \hspace{1cm} (7)

where $Q_l$ is the average lateral inflow, which is the continuity equation solved for by the HEC-RAS functions (HEC-RAS Hydraulic Reference Manual Version 4.1, 2010).

Similarly the momentum equation as given in Equation 4 is rewritten for the channel and floodplain components, assuming the same water surface elevation in the channel and in the floodplain. Approximating the momentum fluxes per unit distance for both the channel and floodplain using finite differences yields Equations 8 and 9

$$\frac{\Delta Q_c}{\Delta t} + \frac{\Delta (V_c Q_c)}{\Delta x_c} + g A_c \left( \frac{\Delta z}{\Delta x_c} + \frac{S_{fc}}{x_c} \right) = M_f$$  \hspace{1cm} (8)

$$\frac{\Delta Q_f}{\Delta t} + \frac{\Delta (V_f Q_f)}{\Delta x_c} + g A_f \left( \frac{\Delta z}{\Delta x_f} + \frac{S_{ff}}{x_f} \right) = M_f$$  \hspace{1cm} (9)
where $V$ is the velocity either in the channel or floodplain, $g$ is the acceleration due to gravity, and $S_f$ is the friction slope of the channel or the floodplain. These can be combined and simplified into Equation 10,

$$\Delta \left( Q \Delta x + Q_f \Delta x_f \right) \Delta t + \Delta \left( \beta V Q \right) + g \bar{A} \Delta z + g \bar{A} S / \Delta x_e = 0$$

where $\beta$ is the velocity distribution factor, $A$ is $A_t + A_c$, $\Delta x_e$ is the equivalent flow path, and $S_f$ represents the frictional slope for the whole cross section. At junctions, additional momentum from the entering tributaries is added to this equation as $M_t$,

$$M_t = \xi \frac{Q V_l}{\Delta x}$$

where $\xi$ is the fraction of momentum entering the main river channel, $Q_l$ is the lateral inflow, and $V_l$ is the lateral inflow average velocity. This is added to the right side of Equation 10 to form a complete momentum equation for multi-reach models. More details on these equations and the methods HEC-RAS uses to solve them can be found in the HEC-RAS Hydraulic Reference Manual.

HEC-RAS is well-tested and in use by the National Weather Service for hydraulic modeling as a part of the Community Hydrologic Prediction System (CHPS) (Roe, Dietz, Restrepo, Halquist, Hartman, Horwood, Olsen, Opitz, Shedd & Welles, 2010). However, this model makes many simplifications, including that flow can be represented by a mean velocity in a cross section, that the water surface is horizontal across the cross section, that vertical acceleration and complex floodplain flows can be neglected, and that Manning’s equation can be used to approximate uniform flow (Pappenberger, Beven, Horritt & Blazkova, 2005).
2.3 Importance of Lateral Inflows

Due to the aforementioned assumptions made by the model, many of the uncertainty quantification studies examining flows modeled in HEC-RAS have focused on the determination of roughness coefficients. However, Pappenberger, et al. (2005) mention in their study that boundary conditions do play a role in the uncertainty of a model, in that a downstream boundary can determine whether flow in the main channel is channel flow dominated or floodplain dominated in order to match the water balance at the downstream boundary condition. This has important implications for flood mapping.

Scharffenberg and Kavvas (2011) explored uncertainty in a kinematic wave hydraulic model for a stream with lateral boundary conditions. They define this lateral boundary condition as “the difference between the upstream boundary condition and the downstream result”. This can be applied uniformly along the channel, or as the authors do, modeled as a computed flow ratio. Importantly, they state that “boundary condition uncertainty is quite broadly the most significant source of uncertainty” and that “the effect of uncertainty in boundary conditions may double the total uncertainty compared to the routing parameters alone”, however, studies to quantify this effect are not widely found in the literature.

2.4 Coupling of Hydrologic and Hydraulic Models

It is common to use a one-dimensional hydraulic model coupled with GIS to produce inundation maps, but it is also quite common to couple hydrologic and hydraulic models for simulation use. There are three main levels of this coupling: full coupling, internal coupling or tight coupling and external or loose coupling (see Haghizadeh, Shui, Mirzaei, & Memarian, 2012, or Betrie, van Griensven, Mohamed, Popescu, Mynett &
Hummel, 2011). Full coupling involves complete reformulation and solution of the governing equations, while tight or internal coupling involves solution of all model equations, and then iterative updating of shared model data (Haghizadeh, et al., 2012). The most basic is loose, or external, coupling in which two models exchange data via an external intermediate program, which does not change model code and is a lower cost procedure (Betrie, et al., 2011).

According to Nunes Correia, et al., (1998) the first efforts in coupling hydrologic and hydraulic modeling for inundation mapping purposes was the creation of HEC-SAM (Spatial Analysis Methodology) by the Hydrologic Engineering Center in 1975, which combined the HEC-1 hydraulic model and kinematic wave routing with a GIS grid. Recently, combining hydrologic models with hydraulic models to simulate backwatering or looped rating curves has been examined for river system modeling.

Debele, Srinivasan & Parlange (2008) externally linked the hydrologic model SWAT with the two-dimensional hydrodynamic CE-QUAL-W2 model, by simulating upland water quality variables and using the time series outputs from this model as inputs to the hydrodynamic model. Most of the observed data were reproduced in this study, and failures to reproduce some results were attributed to poor quality of input data for some modeled water quality constituents and error propagation in calculations. Mai (2009) coupled the WetSpa distributed hydrologic model with HEC-RAS for the simulation and flood prediction of a flood-prone river basin in central Vietnam, both of which were calibrated and validated, and showed good agreement with observed peak flood levels.
Lian, Chan, Singh, Demissie, Knapp & Xue (2007) linked the hydrologic model HSPF with the one-dimensional hydraulic model UNET in order to capture the complicated hydraulics of the Illinois River. They compared the linked model and HSPF alone to see how well the models reproduced historical flows. Their work found a decrease in error of simulated flood peaks as compared to observed peaks, and slight increases in efficiency of the coupled model in daily, monthly and annual flows, though this decreased with increasing time period. These studies show the possible improvements that can be realized through model coupling.

2.5 Inundation Mapping

One of the most basic methods for inundation mapping is using a 1-D hydraulic model to generate a water surface elevation at a given location, which is then extrapolated across a digital elevation model (DEM) to determine which areas would be inundated during a flood event. Wang (2002) used measured surface water heights at USGS gauges projected over a DEM to represent flooded areas in the Tar River Basin during Hurricane Floyd. The benefits of this method were the reliability and accuracy of the flood extent, as verified by satellite imagery, simple and cost-effective methods in creating the map, and easy creation of scenarios not directly observed, but that might be of interest for planning or management decision-making. However, these data must be updated to reflect changes in land use, annual precipitation and changing inundated or dry areas during a flood event to maintain accuracy.

The next step in model coupling using a 1D model is the use of a GIS system to create inundation maps from a water surface. In a typical engineering study, Chang, Hsu, Teng & Huang (2000) linked a 1-D hydraulic model with a 2-D diffusive overland model
in the floodplain to simulate flow and inundation for 100-year and 200-year 24-hour
design rainfall, using GIS to determine parameters for the model as well as to estimate
inundation levels across the watershed. This configuration was suitable to make
engineering design recommendations for possible inundation proofing methods to be
undertaken in the watershed.

Nunes Correia, et al., (1998) coupled a GIS system with two different hydrologic
models (one lumped and one distributed), as well as a hydraulic model to correctly
represent backwater effects in the catchment. They noted a weakness of GIS systems for
mapping being that there is no easy or effective way to process or display time based
processes in GIS. This means that creation of a time series of data requires separate
processing for each time step, increasing the amount of work required, especially if the
model is coupled more tightly from start to finish, instead of more loosely.

2.6 Previous Studies of the Tar River Basin

Bales and Wagner (2009) examined the uncertainty involved in inundation
mapping, focusing on the storm event during Hurricane Floyd in 1999 in the Tar River
Basin in eastern North Carolina. They found drawbacks in using a 1-D hydraulic model,
namely that for complicated channel geometry, the assumption of a uniform water
surface across a cross section may not hold if a cross section crosses multiple channels,
and that uncertainty increases with distance from the main channel in low basins with
multiple tributaries, however good agreement between observed and simulated stage and
discharge was also found.

Colby, Mulcahy & Wang (2000) also studied inundation mapping in the Tar River
Basin after Hurricane Floyd, through generation of a water surface profile in HEC-RAS
and linkage with a GIS system through the ArcGIS extension HEC-GeoRAS, as well as through interpolation of measured water surface to create an inundation profile on a DEM. This study found that both methods better represented actual flooding as recorded by aerial photographs than did estimation of flooded areas from flood insurance maps showing probability of inundation based on recurrence interval. The authors suggest the accuracy of the mapping is limited to the estimated error of the DEM used, and qualify their results with the observation that “standard methods have not yet been developed for qualitatively evaluating the horizontal extent of floodplain boundaries.

Van Cooten et al. (2011) simulated flooding in the Tar River basin by linking HL-RDHM with the two-dimensional ADCIRC model. They found good agreement between total water level and significant wave heights between the modeled and observed data, especially at high-flow events. One difficulty encountered in this project was that HL-RDHM is a hydrologic model which cannot consider backwater effects, which is especially important in a system such as the Tar River where tidal effects can be seen far upstream, and where storm surge can play a part in flooding. The authors suggest the possibility of having the hydrologic model provide inputs to a hydraulic model, which can then be linked to the hydrodynamic model, instead of extending the hydrodynamic model far upstream of where storm surge would be predicted.

Cepero-Pérez (2009) obtained and georeferenced a HEC-RAS model for the main channel of the Tar River. This study examined the ability of a steady state hydraulic model to reproduce inundation due to Hurricane Floyd. The study found that the accuracy is higher closer to the upstream boundary and decreases in the lower parts of the reach, which was attributed to error propagation in the downstream direction.
3. Methods

3.1 Study Area

Located in eastern North Carolina, the Tar River has its headwaters near Oxford, NC, and becomes the Pamlico River at Washington, NC. According to the North Carolina Division of Water Quality [NC DWQ] (2010), the Tar-Pamlico River basin is the fourth largest in the state, and covers a total of 6,148 miles from its origin in north central North Carolina in the Piedmont physiographic area to where it empties into the Pamlico Sound. Of the land in this watershed, over half (55%) is considered wetland and forested area, with a bit less than one third (28%) of the land used for agriculture, while the remaining 7% is classified as developed land, which occurs mostly in the towns of Rocky Mount and Greenville (NC DWQ, 2010).

Only one-third of the basin is located within the Piedmont, which is characterized by easily eroding soils and underlying rock with little water holding capacity. Streams here have low summer flows and there are few natural lakes. Waters in the Coastal Plain, which is comprised mostly of deep sands with high groundwater storage capacity, comprise a majority of the watershed and are typified by swamps and estuaries, with many slow-moving streams. Even the Tar River in this region can experience saltwater intrusion, due to the combination of low topography and wind and tidal effects. There are over 2,500 miles of mapped streams within the watershed, and many more intermittent or ephemeral waterways. The Tar River basin can be subdivided into five sub-basins, which are, from most upstream: Upper Tar River sub-basin, Fishing Creek sub-basin, Lower Tar River sub-basin, Pamlico River sub-basin, and Pamlico Sound sub-basin (NC DWQ, 2010).
The focus area of this study concentrates on the 50 miles of the Tar River between Tarboro and Washington, NC, known as the Lower Tar River sub-basin. Four main tributaries of the Tar in this sub-basin were also selected for modeling as part of this study: Town Creek, Conetoe Creek, Grindle Creek and Tranters Creek. This area of the basin is shown in Figure 1 below. Current NWS hydraulic models used in forecasting only cover the basin down to Greenville, and so do not encompass the extent of the modeling used in this study. This means currently generated inflow calculations are not suitable for use in this modeling scheme. This project examined a finer scale than what is currently modeled, at a further downstream and tidally-influenced location.

![Map of Lower Tar River Watershed and Selected Tributaries](image)

**Figure 1: The extent of the main stem model from Tarboro to Washington, NC is shown in dark blue, with the tributaries modeled shown in purple. The current basin forecast extent is shown in orange. USGS gauges are also shown.**
3.2 Study Time Period

The flood of record for this basin, as well as for much of eastern North Carolina was that which resulted from Hurricane Floyd in September of 1999. During the summer of 1999, much of North Carolina was in a severe drought, with only 2.75 inches of rainfall in Wilmington and 6.39 inches in Raleigh recorded during the previous 12 months. On September 5, Tropical Storm Dennis passed across the state, dropping up to 7 inches of rainfall in parts of the Tar River basin (Bales et al., 2000). Floyd was a much larger hurricane, and made landfall only 10 days later as a Category 3 storm with 130 mile per hour winds near its eye and tropical storm force winds which extended out across its entire 580 mile span (Herring, 2000). Some parts of eastern North Carolina received around 60% of their yearly average total rainfall in only two weeks, due to the passage of Hurricanes Dennis and Floyd (Colby, Mulcahy & Wang, 2000), with over 15 inches of precipitation falling over much of the Tar River basin. A map of the precipitation from the National Weather Service Event Summary is shown in Figure 2.

![Figure 2: Precipitation in North Carolina due to Hurricane Floyd. Some of the heaviest precipitation occurred in parts of the Tar River basin on September 16, 1999.](image-url)
Hurricane Floyd resulted in 52 deaths and an estimated $6 billion in economic losses (Bales, et al, 2000). The severity of the storm has made it the storm of record and source of study for many projects involving flood modeling and mapping.

Another large and well-documented period of flooding occurred during the summer of 2006. In their Event Summary, the National Weather Service office in Raleigh, NC (2009) describes how Tropical Storm Alberto came ashore near Keaton Beach, Florida on June 13, 2006 and weakened to a tropical depression before interacting with a shortwave trough and dumping heavy rains across much of central North Carolina on June 14. A map of the precipitation from this storm event is shown in Figure 3 below. This figure shows the locally heavy rainfall that occurred, some of which was at the headwaters of the Tar River.

![Figure 3: Precipitation in North Carolina due to Tropical Storm Alberto, which dumped heavy rains over the central region of the state on June 14, 2006.](image)

Two months later, Tropical Storm Ernesto came ashore at the southern coast of North Carolina on August 31, 2006 and moved northward across the state, with some of the heaviest rainfall occurring in the Tar River watershed near Greenville, NC. The general track of the storm follows the band of heavy precipitation seen in Figure 4 below.
Figure 4: Precipitation in North Carolina from Tropical Storm Ernesto, which made landfall on August 31, 2006.

The multiple heavy storm events during the summer of 2006, although without post-event studies at the level of detail of those conducted after Hurricane Floyd, provides a good opportunity to test the ability of a hydrologic-hydraulic model system to simulate other large events.

3.3 Models Used

In this study, a hydrologic model (HL-RDHM) was loosely coupled with a hydraulic model (HEC-RAS) with different numbers of connections in four distinct scenarios to explore the effect these scenarios will have on mapping and stage and flow accuracy. A schematic of the model linkages is shown below, in Figure 5.
### 3.3.1 HL-RDHM

Outputs were generated from a run of HL-RDHM for the whole Lower Tar River sub-basin. The parameters used for HL-RDHM runs were based on a-priori parameter estimates combined with information from calibrated lumped parameters used at the Southeast River Forecast Center and some manual adjustments to potential evapotranspiration parameters to improve multi-year seasonal and overall bias statistics. It was not possible to do a comprehensive hydrologic model calibration given the resources available for this study. Two sets of data were generated: one from September 1, 1999 to December 1, 1999 for the simulation of flooding due to Hurricane Floyd and the other from June 1, 2006 to October 1, 2006 for the simulation for Tropical Storm Alberto in June and Tropical Storm Ernesto in August. These runs generated hourly...
discharge, surface flow and subsurface flow data for a ½-HRAP grid, or 2 by 2 km cells. The calculation method of these flows is described previously in Section 2.1.1.

### 3.3.2 HEC-RAS

Previous work involved development and calibration of a HEC-RAS model for the main stem of the Tar River between Tarboro and Washington, NC (Cepero-Pérez, 2011; Reed, 2010). The model contained 103 cross sections, three of which were interpolated. Cepero-Pérez (2011) explored the effects of using Manning’s $n$ values corresponding to land cover types obtained from sampling a digital map to improve simulations for the same time period by spatially varying roughness, but did not see a significant difference between the more complex scenario and using a simplified scheme where channel and floodplain roughness did not change along the reach. That study suggested using a calibrated distributed model to generate lateral inflows to the model as a way to improve simulation results. For the purpose of this study, the varied Manning’s $n$ values were used. The calculation of flow by HEC-RAS is described previously in Section 2.2.2.

Additional models were obtained for the four tributaries from engineering studies conducted by the North Carolina Floodplain Mapping Program (NCFMP). A HEC-RAS model for the upper reach, plus two HEC-2 models for the lower reach were obtained for Town Creek, which, when combined had 170 cross sections. One HEC-RAS model with 151 cross sections was obtained for Conetoe Creek, and two HEC-RAS models, one detailed and one limited in detail was obtained for Grindle Creek, with 151 cross sections. One limited detail HEC-RAS model with 168 cross sections was obtained for
Tranters Creek. Manning’s n values and bank station locations for the tributaries were assigned as a part of these engineering studies and were not modified.

3.4 Processing Steps

Although a calibrated model existed for the main stem of the Tar River for the study area, several steps had to be taken to create a multi-reach model suitable for inundation mapping. The HEC-RAS models obtained from NCFMP for Town, Conetoe, Grindle and Tranters Creeks did include surveyed cross-sections with station and elevation data, but the stream centerline was a straight line, and thus unsuitable for generating an accurately oriented flood map. These tributary models first had to be georeferenced to match the spatial orientation of the true stream channel, and then incorporated into the pre-existing main stem model to create a multi-reach model capable of simulating floods in the whole Lower Tar River sub-basin.

3.4.1 Georeferencing and creation of a multi-stem hydraulic model

Shapefiles of the tributary channels and cross sections were obtained from NCFMP and were imported into ArcGIS. Using the HEC-GeoRAS interface, the river channel and cross sections were digitized for each tributary, and then exported using the “Export RAS Data” function. In the Geometric Data window in HEC-RAS, these GIS data were imported into an existing, non-georeferenced reach, using the “Match to Existing” feature. Examples of geometry for Conetoe Creek before and after georeferencing are shown in Figure 6. A more complete explanation of georeferencing steps used here is included in Appendix A.
Figure 6: The image on the left is the tributary model as obtained from NCFMP, which is not suitable for inundation mapping. The image on the right is the same model after the georeferencing process.
Each tributary and its cross sections were added in a similar way. In the Geometric Data window of the main stem Tar model, HEC-RAS Data was selected using the Import Geometry Data function. A junction was added to connect the tributary channel to the Tar River channel, in between cross sections determined by visual inspection of the stream network in ArcGIS. A small cross section was added near the junction, in order to minimize calculation errors due to a long downstream length between the last surveyed cross section on a tributary and the junction with the main channel of the Tar River, a distance which could be several miles. This full, multi-stem model contained 800 cross sections, 59 of which were interpolated. The completed, multi-stem model is shown below in Figure 7.
Figure 7: The full georeferenced Tar River model as viewed in HEC-RAS, with all the channels and cross sections.
3.4.2 Creation of inputs to HEC-RAS

A shapefile representing the ½-HRAP grid and flow path network for the basin was obtained, on which stream centerlines and model cross sections were overlaid in ArcGIS. Cells which made up the stream and tributary channels modeled with the hydraulic model were identified, and a new shapefile with one point in each cell was created. This shapefile was used for extraction of HL-RDHM data calculating surface and subsurface flows, as described in more detail in following paragraphs. An example of this shapefile is shown as “Surface and Subsurface Flow Points” in Figure 8. Points immediately adjacent to these channel cells were also written to shapefiles for use in generating discharge from HL-RDHM, and are shown as the “Discharge Points” in Figure 8.

In the shapefile properties, a field was created named “Id”. This was manually edited and points were assigned “Id” numbers such that points sharing the same “Id” number corresponded to the same HEC-RAS input. In this way, multiple grid cells would be summed together to represent surface or subsurface flow applied to a single channel or flow from an area contributing to a single lateral inflow. “Id” categories and areas of grouped inflow were determined from visual inspection in ArcGIS. An example of the HRAP grid (color-coded to show cell type, either channel or hillslope), stream network and points for inflows of different types is shown in Figure 8.
A Python script was written to generate a .txt file containing a list of cells and their HRAP coordinates grouped by “Id” number. A second Python script then took the groups and coordinates and generated a sum of discharge, surface flow or subsurface flow, as specified by the user for all cells in a group. This time series was then saved in a HEC-DSS compatible file, which could be input as a boundary condition in a HEC-RAS simulation. For upstream boundaries and point inflows, discharge time series at a point were used. For uniform lateral inflows, surface and subsurface flow and discharge time series were summed using the Math Functions in HEC-DSS to create a single time series to be used in the model as a lateral inflow. The number of these inflows varied for the different scenarios examined, as discussed in the next section.
3.5 Scenario Description

Four different scenarios were created using different configurations of lateral inflows and different model complexities. The first and most basic scenario was the calibrated main stem model with only upstream and downstream boundary conditions from observed USGS data, no lateral inflows. These upstream and downstream boundaries were used consistently throughout the model scenarios. This was considered the simplest and most basic of the scenarios. This schematic is attached on the next page in Figure 9. This was the only scenario of the four to not have a similar total flow at Greenville for this study period.
Conceptual Map: Scenario 1
Tar River from Tarboro to Washington

Figure 9: A conceptual map of the Lower Tar sub-basin with inputs configured as Scenario 1.
The second used the same main stem model, but represented the outlets of each of the four tributaries as point inflows, represented in HEC-RAS as lateral inflow hydrographs, using data generated from RDHM. The discharge from grid cells, whose flow entered the main channel of the Tar between tributary inputs, summed with the surface and subsurface flow calculated from for the 2 km grid cells considered part of the “channel” in RDHM as uniform lateral inflows, was modeled in HEC-RAS as a uniform lateral inflow along the channel between the tributary inflows. This model included 5 lateral inflows, which added inflow from areas averaging 193.6 km$^2$ and 4 point laterals, with drainage areas averaging 358 km$^2$. This scenario most closely represented previous model testing and set-up as obtained from the NWS. This schematic is attached on the next page in Figure 10.
Figure 10: A conceptual map of the Lower Tar sub-basin with inputs configured as Scenario 2.
The third scenario used the multi-stem HEC-RAS model, and again represented the flow entering the channel between tributaries added to the surface and surface flow along the channel as uniform lateral inflows. An upstream inflow point for each tributary was designated by inspection of the grid, and the discharge calculated by RDHM from this cell was designated to be the upstream boundary at this point. Any cross section upstream of this point was eliminated. Each of the grid cells which flowed into the tributary channel, as well as the surface and subsurface flow calculated for the grid cells which made up the tributary channel in RDHM were summed and applied to the HEC-RAS tributary model as a uniform lateral inflow for each tributary. This model included runoff from 5 lateral inflows, which added inflow along the main stem from areas averaging 193.6 km$^2$, 4 upstream inflows to tributaries whose drainage areas averaged 59 km$^2$, and uniform lateral inflows along the tributary of average drainage area 299 km$^2$. A schematic of this scenario layout is attached on the next page in Figure 11.
Figure 11: A conceptual map of the Lower Tar sub-basin with inputs configured as Scenario 3.
The fourth, and most complex scenario maintained the same configuration of lateral inflows on the main stem of the Tar River. The tributary models maintained the same inflow conditions, but via visual inspection of the RDHM flow network, the cells which contained major inflows to the tributaries were selected to be point inflows, with the idea that these would represent inputs from major tributaries of Town, Conetoe, Grindle and Tranters Creeks. Three such inflows were identified for Town, Conetoe, and Tranters Creek, with one inflow along Grindle Creek. The remaining cells whose waters entered the tributaries, along with the surface and subsurface flow from the tributary channel cells were summed and applied to the reach as uniform lateral inflows along the tributary. This simulated runoff from 5 lateral inflows along the main stem from areas averaging 193.6 km$^2$, 4 tributary inflows to the upstream of each tributary model, which averaged drainage areas of 59 km$^2$, and uniform lateral inflows along the tributary of average drainage area 126 km$^2$ and tributary point inflows of an average of 98 2/3 km$^2$ of drainage area to Town Creek, 40 km$^2$ average point inflow to Conetoe Creek, one point inflow of 24 km$^2$ along Grindle Creek, and point inflows averaging 84 km$^2$ to Tranters Creek. A schematic of this scenario layout is attached on the next page in Figure 12.
Figure 12: A conceptual map of the Lower Tar sub-basin with inputs configured as Scenario 4.
3.6 Adjustments

After importing some cross sections to HEC-RAS and adding the tributary models to create a multi-reach model, it was observed that some cross sections overlapped, especially where tributaries were added to form the multi-stem model. To prevent miscalculation due to overlapping cross sections, the length of overlapping cross section was measured in ArcGIS. The length to be cut was determined from visual inspection in ArcGIS and measured using the “Measure” tool. Then, the endpoint of the cross section selected was shortened in HEC-RAS by the measured amount.

Another problem encountered was that some of the cross sections generated from the HEC-2 model for Town Creek were oriented backwards when viewed in HEC-GeoRAS. Since few cross sections were affected and no better data were available to validate how the cross section should be oriented, this was easily fixed by right clicking on a cross section in the Geometric Data window, and selecting “Reverse XS Stationing”.

In order to create a stable model run, model geometry had to be adjusted by adding interpolated cross sections along sections of reaches where surveyed cross sections were far apart and where the change in elevation was relatively steep. Seventy-three cross sections were interpolated on Town Creek, one on Conetoe Creek, and one on Tranters Creek. Another area of adjustment was setting minimum flow levels at the upstream-most cross section of some channels. The very low flow during some periods of low flow, combined with areas of steep relief can create instability in the model. By specifying minimum flows of 100 cubic feet per second at the top of Town Creek and Grindle Creek, enough flow is maintained in the channels to keep the model calculations
stable, while not enough to generate overbank flows. Minimum flows of 400 cubic feet per second, 100 cubic feet per second, and 100 cubic feet per second were applied along the first, second, and third segments of the main stem model, respectively, to ensure stability of the model for the 2006 simulation period. An explanation of model set-up and simulation is more fully described in Appendix B.

3.7 Generating Inundation Maps

After a simulation was run, post-processing was performed in HEC-RAS to generate water surface profiles. This generated a water surface profile for every selected time period, set for one hour in this study, as well as a “maximum water surface” which included the maximum water surface height at each cross section. For inundation mapping, this maximum water surface was the profile used to create the water extent. Using the Export GIS data function in HEC-RAS, the maximum water surface profile was exported for each scenario. Then, HEC-GeoRAS functions are used to generate layers. GeoRAS then imported layers, which are the cross sections, stream channel, and a bounding polygon, which was set as the extent of the mapping. The inundation mapping function then generated a water surface TIN, from which the terrain DEM was subtracted, and a raster of flood depths was created. This terrain was obtained from the 20 foot resolution LiDAR data available from the NCFMP website. A more complete description is included in Appendix C.

3.8 Validation Data

Hourly stage and discharge data, where available, were obtained for USGS stream gauges along the main stem and tributaries being modeled. The stage and flow records
on the tributaries of the Tar River (Conetoe, Town, and Tranters Creeks) were only used in the scenarios involving the multi-stem model, since this was the only time flow was simulated for these reaches. The data were adjusted to elevation using the datum provided for each gauge to allow comparison with the water surface elevation modeled in HEC-RAS. These records were used to validate stage and flow simulations performed by the model both during the Hurricane Floyd simulations and model run encompassing flooding due to Hurricane Alberto, depending on data availability. A shapefile of the locations of these stations was obtained from NOAA’s OHD, and when combined with georeferenced model data, the stream gauge was matched to the closest model cross section to allow comparison with simulated data. A listing of this information is included in Table 1.

<table>
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<tr>
<th>Gauge Name</th>
<th>Gauge Number</th>
<th>Parameters Measured</th>
<th>Dates of Operation</th>
<th>Simulations Used</th>
<th>RS Location in Models</th>
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<td>Stage</td>
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<td>Alberto</td>
<td>Main Channel 131841</td>
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<td>Floyd, Alberto</td>
<td>Main Channel 104466.6</td>
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<td>02083640</td>
<td>Stage</td>
<td>07/23/2003 – present</td>
<td>Alberto</td>
<td>Town Creek 36880</td>
</tr>
<tr>
<td>Tranters Creek at SR 1403 near Washington, NC</td>
<td>0208436195</td>
<td>Stage</td>
<td>06/24/2003 – present</td>
<td>Alberto</td>
<td>Tranters Creek 40104</td>
</tr>
</tbody>
</table>
In order to quantify the fit of the model to observed data obtained from the gauges for each simulation, the index proposed by Nash and Sutcliffe (1970) was employed to characterize the efficiency of a model in reproducing observed values. This formula is

\[
NSE = 1 - \frac{\sum (X_{\text{mod,el}} - X_{\text{obs}})^2}{\sum (X_{\text{obs}} - \bar{X}_{\text{obs}})^2}
\]

where \( X_{\text{model}} \) is the simulated stage or streamflow at each hour interval, \( X_{\text{observed}} \) is the USGS observed stage or streamflow, and \( \bar{X}_{\text{observed}} \), with values closer to 1 indicating a better “fit”.

Additionally, since this project focuses on inundation applications, a modified Nash-Sutcliffe efficiency, as proposed by Guex (2001) was used to determine the efficiency of the model at high flows or stages (as cited by Drogue, et al., 2003, p. 911). This method weights the higher discharges more heavily than the mean discharge. The formula for this is

\[
NSH = 1 - \frac{\sum \left( X_{\text{obs}} + X_{\text{obs}} \right) (X_{\text{mod,el}} - X_{\text{obs}})^2}{\sum \left( X_{\text{obs}} + X_{\text{obs}} \right) (X_{\text{obs}} - \bar{X}_{\text{obs}})^2}
\]

with values closer to 1 again signifying a better model fit to observed data.

High water marks were surveyed after the flooding from Hurricane Floyd as part of a FEMA study post-event survey. A team collected 87 high water marks in the modeled basin extent, including seed, mud, and debris lines, as well as water marks. This was a large-scale study, and the locations high water marks across eastern North Carolina extend far beyond the current forecast basins, and even the current extent of the main stem HEC-RAS model. The mean error for the high water mark points was calculated as
\[ ME = \frac{\sum X_{\text{model}} - X_{\text{obs}}}{n} \]  
\[ MAE = \frac{\sum |X_{\text{model}} - X_{\text{obs}}|}{n} \]

where \( n \) is the number of observations, and the mean absolute error was calculated as

Although satellite images were captured before and after Hurricane Floyd passed through the region, this was not used for validation or mapping verification due to a number of reasons. First was the relatively low resolution of the data, only available in 30x30 meter pixels. With some of the smallest channel cross sections spanning around 150 meters, and generation of flood maps from a DEM at a resolution of 20 feet, it was thought the coarse resolution could hinder effective comparisons between models with only a slight difference in flooding extent. Second, and perhaps more importantly, is the uncertainty that arises from processing such data. Previous studies have shown that the large amount of forested land and the inability of the radar to penetrate the forest canopy combine to create an underestimation of flooding extent (Wang, 2002).
4. Results

For each cross section for which there were observed data at a USGS stream gauge, the Nash-Sutcliffe efficiency (NSE) and the modified Nash-Sutcliffe efficiency for high flows (NSH) were calculated for each scenario as compared to the observed flows. Additionally, the percent root mean square error (percent RMSE) was calculated for each scenario with each observed gauge. For the observed high water marks, the mean error and mean absolute error were calculated.

4.1 Stage and Discharge Validation for Hurricane Floyd

Figure 13 and Figure 14 show the observed stage and flow, respectively, plotted with the stage and flow simulated by each of the four scenarios.

![Figure 13: Stage generated from scenarios 1, 2, 3, and 4 as compared to partial USGS observed stage at Greenville, NC from September to November 1999.](image-url)
Figure 14: Streamflow generated from scenarios 1, 2, 3, and 4 as compared to partial USGS observed stage at Greenville, NC from September to November 1999.

As described in the previous section, various statistics were calculated to determine the accuracy of the simulations. A table of these for the simulation from September to November 1999 for the gauges at Greenville, NC is presented in Table 2, below.

<table>
<thead>
<tr>
<th>Greenville Stage</th>
<th>Scenario 1</th>
<th>Scenario 2</th>
<th>Scenario 3</th>
<th>Scenario 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>NSE</td>
<td>0.931</td>
<td>0.978</td>
<td>0.979</td>
<td>0.979</td>
</tr>
<tr>
<td>NSH</td>
<td>0.937</td>
<td>0.978</td>
<td>0.978</td>
<td>0.978</td>
</tr>
<tr>
<td>% RMSE</td>
<td>7.64%</td>
<td>4.46%</td>
<td>4.30%</td>
<td>4.30%</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Greenville Flow</th>
<th>Scenario 1</th>
<th>Scenario 2</th>
<th>Scenario 3</th>
<th>Scenario 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>NSE</td>
<td>0.944</td>
<td>0.968</td>
<td>0.974</td>
<td>0.974</td>
</tr>
<tr>
<td>NSH</td>
<td>0.953</td>
<td>0.969</td>
<td>0.975</td>
<td>0.975</td>
</tr>
<tr>
<td>% RMSE</td>
<td>5.47%</td>
<td>4.46%</td>
<td>3.97%</td>
<td>3.94%</td>
</tr>
</tbody>
</table>
Figure 15 and Figure 16 present simulated stage and flow, respectively, as compared to the observed data for the USGS stream gauge on Conetoe Creek. There is a period of no data where the stream gauge was out of operation due to flooding from Hurricane Floyd. Figure 15 notes the level of a high water mark obtained in a post-event survey (Bales, et al., 2000). Not shown in Figure 16 is an estimated a maximum daily mean discharge 15,000 cfs for September 18, 1999 (Surface-Water Daily Statistics for the Nation, http://waterdata.usgs.gov/nwis). Backwater effects from the main stem of the Tar River can be clearly observed in both modeled scenarios around September 20, after the passage of most rainfall from Hurricane Floyd.

![Figure 15: Stage generated from scenarios 3 and 4 as compared to partial USGS observed stage at Conetoe Creek from September to November 1999. The pink line shows a post-Floyd high water mark.](image)
Figure 16: Streamflow generated from scenarios 3 and 4 as compared to partial USGS observed stage at Conetoe Creek from September to November 1999. A table of statistics calculated to determine the accuracy of the simulations for the simulation from September to November 1999 for the USGS stream gauge at Conetoe Creek, NC is presented in Table 3. There are no statistics for Scenarios 1 and 2, since these do not explicitly simulate tributary streamflow or stage.

<table>
<thead>
<tr>
<th></th>
<th>Scenario 1</th>
<th>Scenario 2</th>
<th>Scenario 3</th>
<th>Scenario 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Conetoe Stage</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>NSE</td>
<td>--</td>
<td>--</td>
<td>-3.86</td>
<td>-5.66</td>
</tr>
<tr>
<td>NSH</td>
<td>--</td>
<td>--</td>
<td>-5.43</td>
<td>-7.65</td>
</tr>
<tr>
<td>RMSE</td>
<td>--</td>
<td>--</td>
<td>8.45%</td>
<td>8.82%</td>
</tr>
<tr>
<td>Conetoe Discharge</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>NSE</td>
<td>--</td>
<td>--</td>
<td>0.142</td>
<td>0.132</td>
</tr>
<tr>
<td>NSH</td>
<td>--</td>
<td>--</td>
<td>0.255</td>
<td>0.271</td>
</tr>
<tr>
<td>RMSE</td>
<td>--</td>
<td>--</td>
<td>34.09%</td>
<td>32.04%</td>
</tr>
</tbody>
</table>


4.2 Flooding Extents Modeled for Hurricane Floyd

Maximum water surface profiles computed in HEC-RAS were exported to GIS files as described in the methods section. A profile of these water surfaces are shown in Figure 17, Figure 18, Figure 19, and Figure 20 for Scenarios 1 through 4, with the most upstream cross section on the right of the figure and the downstream outlet on the left. These serve as a qualitative check of model stability, that the water surface computed and interpolated between cross sections is physically reasonable.

Figure 21, Figure 22, and Figure 23 show the boundary of the maximum water surfaces for each scenario as calculated GeoRAS. Scenarios 1 and 2 do not explicitly model stage on the tributaries, but the TIN generated by GeoRAS for inundation mapping is of great enough extent that some or all of the three eastern tributaries are sampled. This predicts flooding by projecting the water surface of the main stem at a tributary outlet upstream along the tributary. Town Creek is not covered by the TIN in these first two models. An explanation of map generation is presented in Appendix C.

Although no validation data exist to quantify the accuracy of these boundaries, they are presented to show the difference in the extents of the main stem scenarios as compared to the multi-stem ones, and a comparison of boundaries within types. Figure 21 compares scenarios 1 and 2, which are main stem models only. Figure 22 compares scenario 3, the least complex multi-reach model with respect to lateral inflows with scenario 2. Figure 23 compares the two multi-reach models, scenarios 3 and 4. Additional maps show the flood extent overlaid on a topographic map of the city of Greenville, NC, and are an example of map generation for management decision-making. These are included in Appendix D.
Figure 17: Profile plot generated in HEC-RAS for the highest water surface at each cross section as generated from Scenario 1 for Hurricane Floyd.
Figure 18: Profile plot generated in HEC-RAS for the highest water surface at each cross section as generated from Scenario 2 for Hurricane Floyd.
Figure 19: Profile plot generated in HEC-RAS for the highest water surface at each cross section as generated from Scenario 3 for Hurricane Floyd.
Figure 20: Profile plot generated in HEC-RAS for the highest water surface at each cross section as generated from Scenario 4 for Hurricane Floyd.
Figure 21: Boundaries of flooding generated by scenarios 1 and 2 Hurricane Floyd (September 1999).
Figure 22: Boundaries of flooding generated by scenarios 2 and 3 Hurricane Floyd (September 1999).
Figure 23: Boundaries of flooding generated by scenarios 3 and 4 Hurricane Floyd (September 1999).
4.3 High Water Mark Validation

Eighty-seven high water marks were surveyed after Hurricane Floyd in the area covered by the models in this study. These were obtained in a shapefile with information about their elevation. This was compared to the inundation elevation created in a raster in ArcGIS at each point where a high water mark was sampled for each of the four scenarios modeled. Due to the shape of the TIN created by the GeoRAS processes, not every high water mark was compared for each scenario. The two main stem models are unable to simulate the left-most tributary (Town Creek) and the most upstream high water marks on the downstream-most tributary (Tranters Creek) due to the shape of the main channel and the water surface raster generated for the sub-basin, even though the other tributaries are predicted because they fall under this water surface interpolation.

Seventy-four high water marks were analyzed for Scenarios 1 and 3, and all 87 were analyzed for Scenarios 3 and 4. Figure 24 shows a map of the inundated area as predicted by the first scenario. Each point on the map represents a high water mark location. The points are color coded based on the difference between the simulated water elevation and the elevation of the surveyed high water mark, and values are shown in the legend. Figure 25 shows the same points for Scenario 2, Figure 26 shows Scenario 3, and Figure 27 is a map of high water error for Scenario 4.
Figure 24: Comparison of model simulated high water at measured high water marks after Hurricane Floyd for Scenario 1.
Figure 25: Comparison of model simulated high water at measured high water marks after Hurricane Floyd for Scenario 2.
Figure 26: Comparison of model simulated high water at measured high water marks after Hurricane Floyd for Scenario 3.
Flood Boundary Comparison

Scenario 4
High Water Marks (# in class)
Scenario 4 Difference from Observed
-9.9 - -5 ft (8)
-5 - -1 ft (28)
-1 - -0.1 ft (2)
0.1 - 0.1 ft (2)
0.1 - 1 ft (5)
1 - 5 ft (25)
5 - 10 ft (14)
10 - 15 ft (1)
15 - 25.2 ft (2)

Figure 27: Comparison of model simulated high water at measured high water marks after Hurricane Floyd for Scenario 4.
Table 4 presents the mean error, calculated by equation 14, and mean absolute error, from equation 15, which represent the accuracy of the inundation maps in predicting flood height as compared to measured high water marks. Table 8 shows the mean error and Table 6 shows mean absolute error compared to location of the mark. These statistics are only for the 74 high water marks which all simulations had in common.

Table 4: Mean Error and Mean Absolute Error in height of simulated high water mark by scenario, same 74 points

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Mean Error</th>
<th>Mean Absolute Error</th>
</tr>
</thead>
<tbody>
<tr>
<td>Scenario 1</td>
<td>0.89</td>
<td>3.56</td>
</tr>
<tr>
<td>Scenario 2</td>
<td>1.13</td>
<td>3.56</td>
</tr>
<tr>
<td>Scenario 3</td>
<td>1.72</td>
<td>3.98</td>
</tr>
<tr>
<td>Scenario 4</td>
<td>1.58</td>
<td>3.92</td>
</tr>
</tbody>
</table>

Table 5: Mean Error in height of simulated high water mark by reach and scenario, same 74 points

<table>
<thead>
<tr>
<th>Reach</th>
<th>Scenario 1</th>
<th>Scenario 2</th>
<th>Scenario 3</th>
<th>Scenario 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Town Creek</td>
<td>-4.75</td>
<td>-4.44</td>
<td>-4.42</td>
<td>-4.41</td>
</tr>
<tr>
<td>Conetoe Creek</td>
<td>2.25</td>
<td>2.55</td>
<td>3.77</td>
<td>3.74</td>
</tr>
<tr>
<td>Grindle Creek</td>
<td>-1.24</td>
<td>-1.19</td>
<td>2.52</td>
<td>1.02</td>
</tr>
<tr>
<td>Tranters Creek</td>
<td>2.01</td>
<td>2.06</td>
<td>2.86</td>
<td>3.11</td>
</tr>
<tr>
<td>Tar Main Stem</td>
<td>0.86</td>
<td>1.15</td>
<td>1.02</td>
<td>1.02</td>
</tr>
</tbody>
</table>

Table 6: Mean Absolute Error in height of simulated high water mark by reach and scenario, same 74 points

<table>
<thead>
<tr>
<th>Reach</th>
<th>Scenario 1</th>
<th>Scenario 2</th>
<th>Scenario 3</th>
<th>Scenario 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Town Creek</td>
<td>4.75</td>
<td>4.44</td>
<td>4.42</td>
<td>4.41</td>
</tr>
<tr>
<td>Conetoe Creek</td>
<td>2.93</td>
<td>3.17</td>
<td>3.77</td>
<td>3.74</td>
</tr>
<tr>
<td>Grindle Creek</td>
<td>1.24</td>
<td>1.19</td>
<td>2.54</td>
<td>1.76</td>
</tr>
<tr>
<td>Tranters Creek</td>
<td>2.01</td>
<td>2.06</td>
<td>2.86</td>
<td>3.11</td>
</tr>
<tr>
<td>Tar Main Stem</td>
<td>4.36</td>
<td>4.30</td>
<td>4.47</td>
<td>4.46</td>
</tr>
</tbody>
</table>
Scenarios 1 and 2 had 13 fewer high water marks, due to the smaller area generated by GeoRAS. For Scenarios 1 and 2, there was only one point measured on Town Creek while Scenarios 3 and 4 analyzed 11. On Tranters Creek, only 8 high water marks were within the modeled area for Scenarios 1 and 2, but 11 are included for Scenarios 3 and 4. Table 7 gives the mean error and mean absolute error by simulation, Table 8 shows the mean error and Table 9 shows mean absolute error compared to location of the mark.

<table>
<thead>
<tr>
<th>Table 7: Mean Error and Mean Absolute Error in height of simulated high water mark by scenario, all available points for each scenario</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Table 7</strong>: Mean Error</td>
</tr>
<tr>
<td>--------------------------</td>
</tr>
<tr>
<td>Scenario 1</td>
</tr>
<tr>
<td>Scenario 2</td>
</tr>
<tr>
<td>Scenario 3</td>
</tr>
<tr>
<td>Scenario 4</td>
</tr>
<tr>
<td>Mean Error</td>
</tr>
<tr>
<td>Mean Absolute Error</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 8: Mean Error in height of simulated high water mark by reach and scenario, all available points for each scenario</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Table 8</strong>: Mean Error</td>
</tr>
<tr>
<td>--------------------------</td>
</tr>
<tr>
<td>Scenario 1</td>
</tr>
<tr>
<td>Town Creek</td>
</tr>
<tr>
<td>Conetoe Creek</td>
</tr>
<tr>
<td>Grindle Creek</td>
</tr>
<tr>
<td>Tranters Creek</td>
</tr>
<tr>
<td>Tar Main Stem</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 9: Mean Absolute Error in height of simulated high water mark by reach and scenario, all available points for each scenario</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Table 9</strong>: Mean Absolute Error</td>
</tr>
<tr>
<td>--------------------------</td>
</tr>
<tr>
<td>Scenario 1</td>
</tr>
<tr>
<td>Town Creek</td>
</tr>
<tr>
<td>Conetoe Creek</td>
</tr>
<tr>
<td>Grindle Creek</td>
</tr>
<tr>
<td>Tranters Creek</td>
</tr>
<tr>
<td>Tar Main Stem</td>
</tr>
</tbody>
</table>
4.4 Validation for 2006 (TS Alberto) Time Period

Five USGS stream gauges were active during the June to September 2006 simulation period. The observed data from these gauges was compared with flow and stage simulated by HEC-RAS for each of the four different scenarios. The stage and flow time series for this time period, along with Nash-Sutcliffe efficiency (NSE), adjusted NSE (NSH), and percent root mean squared error (% RMSE) are presented in this section.

A graph showing the stage simulated near Rock Springs, NC for Scenarios 1, 2, 3, and 4 as compared to observed data from a USGS stream gauge in that area is shown in Figure 28. The high baseflows during the August 2006 period as simulated in Scenarios 3 and 4 are due to the minimum flow conditions set for stability of this model, as described in section 3.6. A table of statistics calculated to determine the accuracy of the simulations for the simulation from June to September 2006 for the USGS stream gauge at Rock Springs, NC is presented in Table 10.
Figure 28: Stage generated from scenarios 1, 2, 3 and 4 as compared to USGS observed stage at Rock Springs, NC from June to September 2006.

Table 10: Table of NSE, NSH, and % RMSE for the 2006 simulation period at Rock Springs, NC.

<table>
<thead>
<tr>
<th>Rock Springs Stage</th>
<th>Scenario 1</th>
<th>Scenario 2</th>
<th>Scenario 3</th>
<th>Scenario 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>NSE</td>
<td>0.912</td>
<td>0.973</td>
<td>0.948</td>
<td>0.958</td>
</tr>
<tr>
<td>NSH</td>
<td>0.923</td>
<td>0.970</td>
<td>0.956</td>
<td>0.962</td>
</tr>
<tr>
<td>% RMSE</td>
<td>7.71%</td>
<td>4.58%</td>
<td>5.83%</td>
<td>5.32%</td>
</tr>
</tbody>
</table>
Figure 29: Stage generated from scenarios 1, 2, 3 and 4 as compared to USGS observed stage at Greenville, NC from June to September 2006.

Figure 30: Streamflow generated from scenarios 1, 2, 3 and 4 as compared to USGS observed stage at Greenville, NC for June to September 2006.
Stage and flow simulated at Greenville, NC for the time period from June to September 2006 are shown in Figure 29 and Figure 30. Observed data are plotted, with stage and flow simulated for each of the scenarios. A table of statistics calculated to determine the accuracy of the simulations for the USGS stream gauge at Greenville, NC is presented in Table 11.

**Table 11: Table of NSE, NSH, and % RMSE for the 2006 simulation period at Greenville, NC.**

<table>
<thead>
<tr>
<th></th>
<th>Scenario 1</th>
<th>Scenario 2</th>
<th>Scenario 3</th>
<th>Scenario 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Greenville Stage</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>NSE</td>
<td>0.902</td>
<td>0.956</td>
<td>0.946</td>
<td>0.951</td>
</tr>
<tr>
<td>NSH</td>
<td>0.922</td>
<td>0.950</td>
<td>0.945</td>
<td>0.949</td>
</tr>
<tr>
<td>% RMSE</td>
<td>7.26%</td>
<td>5.51%</td>
<td>5.88%</td>
<td>5.64%</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>Scenario 1</th>
<th>Scenario 2</th>
<th>Scenario 3</th>
<th>Scenario 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Greenville Flow</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>NSE</td>
<td>0.874</td>
<td>0.910</td>
<td>0.906</td>
<td>0.911</td>
</tr>
<tr>
<td>NSH</td>
<td>0.917</td>
<td>0.908</td>
<td>0.910</td>
<td>0.915</td>
</tr>
<tr>
<td>% RMSE</td>
<td>6.87%</td>
<td>6.66%</td>
<td>6.71%</td>
<td>6.54%</td>
</tr>
</tbody>
</table>

Stage simulated at Grimesland, NC for the time period from June to September 2006 is shown in Figure 31. A table of statistics calculated to determine the accuracy of the stage for the 2006 simulation period for the USGS stream gauge is presented in Table 12.
Figure 31: Stage generated from scenarios 1, 2, 3 and 4 as compared to USGS observed stage at Grimesland, NC from June to September 2006.

Table 12: Table of NSE, NSH, and % RMSE for the 2006 simulation period at Grimesland, NC

<table>
<thead>
<tr>
<th>Grimesland Stage</th>
<th>Scenario 1</th>
<th>Scenario 2</th>
<th>Scenario 3</th>
<th>Scenario 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>NSE</td>
<td>0.825</td>
<td>0.843</td>
<td>0.900</td>
<td>0.903</td>
</tr>
<tr>
<td>NSH</td>
<td>0.879</td>
<td>0.794</td>
<td>0.890</td>
<td>0.892</td>
</tr>
<tr>
<td>% RMSE</td>
<td>7.30%</td>
<td>8.97%</td>
<td>6.63%</td>
<td>6.55%</td>
</tr>
</tbody>
</table>

Stage simulated at a stream gauge on Town Creek, a tributary of the Tar River for the time period from June to September 2006 is shown in Figure 32. A table of statistics calculated to determine the accuracy of the simulations for the simulation from for the USGS stream gauge is presented in
Table 13. There are no statistics for Scenarios 1 and 2, since these do not explicitly simulate tributary streamflow or stage.

<table>
<thead>
<tr>
<th>Town Creek Stage</th>
<th>Scenario 1</th>
<th>Scenario 2</th>
<th>Scenario 3</th>
<th>Scenario 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>NSE</td>
<td>--</td>
<td>--</td>
<td>0.365</td>
<td>0.465</td>
</tr>
<tr>
<td>NSH</td>
<td>--</td>
<td>--</td>
<td>0.427</td>
<td>0.524</td>
</tr>
<tr>
<td>% RMSE</td>
<td>--</td>
<td>--</td>
<td>11.44%</td>
<td>11.19%</td>
</tr>
</tbody>
</table>

Stage simulated at a stream gauge on Tranter Creek, a tributary of the Tar River for the time period from June to September 2006 is shown in Figure 33. A table of statistics calculated to determine the accuracy of the simulations for the simulation from for the USGS stream gauge is presented in...
Table 14. There are no statistics for Scenarios 1 and 2, since these do not explicitly simulate tributary streamflow or stage.

![Stage generated from scenarios 3 and 4 as compared to USGS observed stage at Tranters Creek from June to September 2006.](image)

Table 14: Table of NSE, NSH, and % RMSE for the 2006 simulation period at Tranters Creek, NC

<table>
<thead>
<tr>
<th>Tranters Creek Stage</th>
<th>Scenario 1</th>
<th>Scenario 2</th>
<th>Scenario 3</th>
<th>Scenario 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>NSE</td>
<td>--</td>
<td>--</td>
<td>0.076</td>
<td>0.078</td>
</tr>
<tr>
<td>NSH</td>
<td>--</td>
<td>--</td>
<td>-0.002</td>
<td>0.002</td>
</tr>
<tr>
<td>% RMSE</td>
<td>--</td>
<td>--</td>
<td>53.35%</td>
<td>53.03%</td>
</tr>
</tbody>
</table>

4.5 Flooding Extents for 2006 Period

Maximum water surface profiles computed in HEC-RAS were also computed for mapping for each scenario in the 2006 simulation period. A profile of these water
surfaces are shown in Figure 34, Figure 35, Figure 36, Figure 37 for Scenarios 1 through 4, with the most upstream cross section on the right of the figure and the downstream outlet on the left. These can be compared with the maximum water surface generated for the 1999 simulation. Since this period should simulate the maximum water surface for the storm of record for this basin, the water surfaces should be lower for this validation period.

Figure 38, Figure 39, and Figure 40 show the boundary of the maximum water surfaces for each scenario as calculated by GeoRAS. Although no validation data exist to quantify the accuracy of these boundaries, they are presented to show the difference in the extents of the main stem scenarios as compared to the multi-stem ones, a comparison of the boundaries within types, and as is true for the water surface profile, should be less than the boundaries calculated for the 1999 simulation period, which contained the storm of record. Figure 38 compares scenarios 1 and 2, which are main stem models only. Figure 39 compares scenario 3, the least complex multi-reach model with respect to lateral inflows with scenario 2. Figure 40 compares the two multi-reach models, scenarios 3 and 4. High water mark data were not available during this time period, so no uncertainty analysis of this flood event was performed.
Figure 34: Profile plot generated in HEC-RAS for the highest water surface at each cross section as generated from Scenario 1 from June to September 2006.
Figure 35: Profile plot generated in HEC-RAS for the highest water surface at each cross section as generated from Scenario 2 from June to September 2006.
Figure 36: Profile plot generated in HEC-RAS for the highest water surface at each cross section as generated from Scenario 3 from June to September 2006.
Figure 37: Profile plot generated in HEC-RAS for the highest water surface at each cross section as generated from Scenario 4 from June to September 2006
Figure 38: Boundaries of flooding generated by scenarios 1 and 2 for the time period from June to September 2006.
Figure 39: Boundaries of flooding generated by scenarios 2 and 3 for the time period from June to September 2006.
Figure 40: Boundaries of flooding generated by scenarios 3 and 4 for the time period from June to September 2006.
5. Discussion and Conclusions

5.1 Uncertainty in Models

Simulation of flows and stages on the main stem of the Tar River were very good: the Nash-Sutcliffe efficiency (NSE) and adjusted Nash-Sutcliffe efficiency (NSH) were generally very close to 1 for the comparison between simulated and observed flows and stage for both simulation periods. Average NSE for all simulations for main stem observation points was 0.898 for Scenario 1, 0.938 for Scenario 2, 0.942 for Scenario 3 and 0.946 for Scenario 4. NSH values were still very good, but generally not as high. The only scenario which showed a better ability to predict high flows or stages as compared to all flows or stages was Scenario 1, where the average NSH on the main channel of the Tar River was 0.922. Scenario 3 had the same NSH value as NSE, 0.942. The average NSH at the same points for the other scenarios were 0.928 for Scenario 2, and Scenario 4 was 0.945. Scenario 4 overall showed the best fit with observed data for points on the main stem of the Tar River.

The NSE and NSH values showed a poor model prediction of observed data on the tributaries. The NSE and NSH values for the 1999 simulation of stage on Conetoe Creek were negative, which means the modeled stage is not a good predictor of observed stage, that the average of the observed data is a better predictor of stage than the modeled data, and that the model is probably biased either in time or in stage magnitude simulated. Prediction for discharge at this point was not as bad, but was similarly low, with NSE and NSH for Scenario 3 0.142 and 0.255. Scenario 4 showed a slight increase in efficiency in prediction of high flows with NSH improving to 0.271, but NSE slightly decreasing to
0.132. It is important to keep in mind when considering these statistics at this gauge that there is a significant period of missing data, including the very highest flows. Therefore, the NSE and especially NSH at this point may not reflect the true accuracy of the model.

However, similarly poor efficiencies are found for the simulation of tributary stage during the second time period simulation with more complete data. NSE and NSH for stage at Town Creek were best of all tributary simulations, 0.365 and 0.427 for Scenario 3 and 0.465 and 0.524 for Scenario 4, respectively for the simulation during 2006. The simulation of stage at Tranter's Creek for the same time period was extremely poor, with Scenario 3 NSE and NSH 0.076 and -0.002, and Scenario 4 NSE and NSH 0.078 and 0.002. This suggests poor or no efficiency of model ability to reflect observed data.

It is encouraging that the NSH for Town Creek stage is greater than the NSE, because the focus of the use of these models is for flood forecasting. Even though this was not true for the majority of the simulation points, because the NSE and NSH are both so high on the main stem of the Tar River, the significance of any difference between these values as it relates to how well the modeled data fit the observed data may be negligible, as the whole range of stage and flows are very well predicted.

It is thought that the better prediction of flows and stage on the main stem of the Tar River as compared to the tributaries is due to observed data providing the upstream and downstream boundary conditions, which may dominate model accuracy. This was noted by Pappenberger, et al. (2005), who saw inaccuracies in flow due to the model forcing flow to meet the water balance at the downstream condition. It may be that
inaccurately predicted flow in the main stem at a junction of a tributary could represent the same problem, a downstream boundary condition which strongly affects flow or stage in the tributary. The propagation of error downstream in the main channel of this model was observed by Cepero-Pérez (2009). Future studies should examine observed data close to the inflow of a tributary to see if a better fit of simulated main channel flow to observed data near the junction of a tributary with the main stem is correlated to more accurate simulation of tributary flow.

Reed (2010) found that “a minimum standard for model calibration performance should be about a five percent RMSE error”. On average for all simulations as compared to observed data on the main channel of the Tar River, percent RSME is around 7.04% for Scenario 1, 5.77% for Scenario 2, 5.56% for Scenario 3, and 5.38% for Scenario 4. This suggests gains can be made in error reduction on the main channel of the river from increasing model complexity. Tributary percent RSME did not show such low values, with the average percent RMSE for all simulations and tributaries for Scenario 3 around 26.8% and for Scenario 4 around 26.3%. The lowest % RMSE was calculated for Scenario 3 stage on Conetoe Creek (8.45%) and the highest was a huge % RMSE of 53.35% for stage simulated by Scenario 3 on Tranter Creek. At the same time, there is a huge overprediction of stage late in the 2006 simulation for Tranter Creek, which is discussed more in the next section. This could be an error from inputs to HL-RDHM or model instability, which would not be an issue with the downstream boundary of this tributary. Regardless of the source of this particular error, it is suggested that future
studies should use more tributaries with observed data available during the simulation period to confirm the effect of boundary conditions on ungauged tributaries.

Interestingly, the errors in inundation mapping do not appear to follow this trend in error propagation as closely as the stages and streamflows. Many scenarios have some of the least error at points very near the downstream boundary of the model, and low error at points taken along the tributaries. Additionally, error was not significantly better near the upstream boundary condition, which would not be expected from the stage and streamflow error patterns. The two most overpredicted points occur near a validation point on the main stem of the model. It is thought that either the elevation of the flood at this point is recorded incorrectly, or that there are smaller scale effects (such as sheltering due to buildings or bridges) that cannot be observed by looking at a DEM which has been processed to remove some of these effects. Increasing model complexity did not seem to significantly improve accuracy of predicting water height at given points, especially along the main stem, with little to no change in mean absolute error or mean error. Town Creek showed some consistent mean error with increasing model complexity and number of high water marks, but a large increase in absolute error for Scenarios 3 and 4, when including all inundation points. The mean error for Town Creek stays about the same and the mean absolute error decreases slightly when only the points common to all simulations are compared. Conetoe Creek, Grindle Creek and Tranters Creek showed an increase in mean error and mean absolute error with increasingly model complexity, though the errors decrease from Scenario 3 to Scenario 4 for Grindle Creek. Although the main stem model can be considered well-calibrated with respect to stage and flow
prediction, there is still significant error in prediction of flood height at given high water marks, though the two highly overpredicted points mentioned previously do contribute to that error.

5.2 Ease of Modeling Other Events

As mentioned in section 5.1, NSE and NSH were very high for main stem observed data points for the simulation period during 2006, which had not previously been simulated by other studies in this basin. However, a few issues were noted that may be decreasing model accuracy. Strong overprediction of stage and flow towards the end of the 2006 simulation which cause very high stages and flows, as seen strongly in Figure 33 showing simulated and observed stage at Tranters Creek, and less so due to backwater effects along the main channel in Figure 28, Figure 29 and Figure 31. From preliminary investigation, this appears to be an over-expression of localized runoff due to Tropical Storm Ernesto, with observed data showing a very steep, sharp peak of stage or discharge during this time period, but the modeled data showing a higher and broader curve, especially at the downstream portion of the model and in the more downstream tributaries. It is unknown whether the problem lies with faulty input, in which case the error would be from HL-RDHM, either in magnitude or timing, or whether the model is overexpressing backwater effects. Identifying the source of this problem was not within the scope of this work, but it serves as an important example that a model which is well calibrated for one time period may not simulate a different time period with the same accuracy.
5.3 Lessons Learned and Recommendations

Overall, increasing complexity did increase the efficiency in modeling stage and streamflow, as well as high stage and streamflow for the main stem of the Lower Tar River sub-basin. However, the high NSE, NSH and low percent RMSE values are only true where observed data provide upstream and downstream boundary conditions. Using hydrologic model-generated upstream boundary conditions and modeled stage or flow in the main channel as downstream boundary conditions for tributaries did not produce simulations with high efficiencies or low RMSE. The error seemed to increase in the downstream direction for both the main stem and the order of tributaries, suggesting that uncertainty in the downstream boundary condition provided by the model may negatively impact the accuracy of stage or flow in tributaries without forcing by observed data.

The error in height of inundation at observed high water marks showed little trend with increasing model complexity. No obvious trends were seen correlating error to channel type (main stem or tributary) or distance to upstream or downstream boundary condition with increasing model complexity. The error in streamflow or stage prediction, which seemed to increase with distance downstream, did not seem to be correlated with error in high water height, which was often better at the downstream boundary than points farther upstream, and which varied greatly even in among points close together.

Operationally, there is not much time difference involved in running a simulation of more complex geometry: most of the time investment is taken in the post-processing component to create water surface heights of interest at different time steps. However, there is greater time involved in georeferencing and integrating all the parts to a more
complicated model and then ensuring stability. It is important to note for operational use that an overall well-calibrated model with high efficiency and low percent RMSE may still have large errors in prediction of water surface heights during flood events, so the application for this method in creating detailed inundation maps still appears to be limited.

5.4 Future Work

For future work, it is recommended to obtain aerial photography for inundation extent where available, in order to better calibrate inundation maps. Since this was not available, no quantitative conclusions could be drawn about the extent of inundation produced by the model. It would be valuable to compare the accuracy of inundation extent along with the accuracy of high water height, to see whether the extent of flooding correlates to areas of over or underprediction of with flood height, or whether the models should simply be calibrated to improve inundation height. The model could also be calibrated to produce higher NSH as compared to NSE, to see if this would produce greater accuracy in flood height generated at the high water marks.

Another recommended approach would be to isolate the effect of boundary conditions on tributary flow and stage efficiency and error. By using observed data as the upstream boundaries of a tributary, the effect of error in the downstream boundary could be quantified, and simulated discharge and stage as well as inundation mapping results for the model could be improved.
Appendix A

Georeferencing HEC-RAS model data in GIS

Set-up in GIS
GeoRAS > ApUtilities > Create New Map
Assign projection to map (or import a correctly projected terrain map)
Add terrain GRID (check projection) and desired XS (check projection), save the map
Create new River and XSCutLines layers (GeoRAS > RAS geometry > Create RAS Layers > Stream Centerline and GeoRAS > RAS geometry > Create RAS Layers > XS Cut Lines
Close ArcMap, open ArcCatalog
ArcCatalog > Select River, right click, Load > Select stream shapefile (make sure it is already projected and in one piece, not multiple segments), repeat for cross sections
Return to GIS, GeoRAS > ApUtilities > Assign UniqueID
GeoRAS > Assign RiverCode and ReachCode to River > Give the channel a name
GeoRAS > Select Flowpath and Assign LineType Attributes > Select the river shapefile as the Channel
GeoRAS > RAS geometry > Stream Centerline Attributes > All
GeoRAS > RAS geometry > XS Cut Lines Attributes > All (check names of layers in this new window)
Sort XS descending by station name, check to make sure they are in spatial order
If not, there is probably a break somewhere in the stream centerline that needs to be fixed
Editor > Modify Feature (having Snap to Vertex helps) and make the endpoints meet
Will have to re-do starting at UniqueID step
Editor > Start editing > copy and paste station names from original XS file (to match what is in HEC-RAS)
When finished, remember to save and stop editing
GeoRAS > RAS geometry > Export RAS Data (New Export Method, but I don’t know the difference)

Importing into HEC-RAS
Open HEC-RAS project file
Open Geometry Window
File > New Geometry
File > Import Geometry Data > GIS format
Check US customary units, Import stream, only check “GIS cut lines”, “reach lengths” for XS
File > Import Geometry Data > HEC-RAS format
US Customary units
Uncheck River Reach Stream Lines, Change Import River and Reach As to match GIS
Cross Sections: Increase Matching tolerance to 0.5, click Match to Existing, THEN uncheck BR/Culv, IS, LS, Names/Descriptions, Reach Lengths, and click Check Existing (order is important)
Tables>Reach Lengths > copy from channel to left and right overbanks
May need to delete empty cross sections, edit steady flow data for new plan (XS names, etc.)
Appendix B

Example HEC-RAS Set-Up and Run

1. The georeferenced models were loaded into the Geometric Data window by going to the File menu and selecting Import Geometry Data > HEC-RAS Format and choosing one of the geometries created through the georeferencing process.

2. All the default settings were accepted. Manning’s n values obtained with the models were left unchanged during this simulation. Bank stations were adjusted as needed via visual inspection. An example cross section is shown in the figure below.

3. The simulation flow file for the plan was created by returning to the main HEC-RAS window and selecting Edit > Unsteady Flow Data.

4. Under the “Add Boundary Condition Location”, “Add RS” was selected, and the cross section where the inflow would start was selected.
5. The Boundary Condition Type was selected for the appropriate inflow. Upstream boundaries were input as “Flow Hydrograph”, lateral inflows were input as “Uniform Lateral Inflow”, point laterals were characterized as “Lateral Inflow Hydr.” And the downstream-most boundary on the main stem was input as a “Stage Hydrograph”. An image of the Unsteady Flow set-up window for the actual simulation for Scenario 4 is shown below.
<table>
<thead>
<tr>
<th>River</th>
<th>Reach</th>
<th>RS</th>
<th>Boundary Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Conetoe</td>
<td>All</td>
<td>87903</td>
<td>Flow Hydrograph</td>
</tr>
<tr>
<td>Conetoe</td>
<td>All</td>
<td>86333</td>
<td>Uniform Lateral Inflow</td>
</tr>
<tr>
<td>Conetoe</td>
<td>All</td>
<td>70000</td>
<td>Lateral Inflow Hyd.</td>
</tr>
<tr>
<td>Conetoe</td>
<td>All</td>
<td>51554</td>
<td>Lateral Inflow Hyd.</td>
</tr>
<tr>
<td>Conetoe</td>
<td>All</td>
<td>27361</td>
<td>Lateral Inflow Hyd.</td>
</tr>
<tr>
<td>Grindle Creek</td>
<td>All</td>
<td>98500</td>
<td>Flow Hydrograph</td>
</tr>
<tr>
<td>Grindle Creek</td>
<td>All</td>
<td>97501</td>
<td>Uniform Lateral Inflow</td>
</tr>
<tr>
<td>Grindle Creek</td>
<td>All</td>
<td>30480</td>
<td>Lateral Inflow Hyd.</td>
</tr>
<tr>
<td>Main Stem</td>
<td>Reach - 1</td>
<td>245644.0</td>
<td>Flow Hydrograph</td>
</tr>
<tr>
<td>Main Stem</td>
<td>Reach - 1</td>
<td>245050.0</td>
<td>Uniform Lateral Inflow</td>
</tr>
<tr>
<td>Main Stem</td>
<td>Reach - 2</td>
<td>205613.0</td>
<td>Uniform Lateral Inflow</td>
</tr>
<tr>
<td>Main Stem</td>
<td>Reach - 3</td>
<td>156867.0</td>
<td>Uniform Lateral Inflow</td>
</tr>
<tr>
<td>Main Stem</td>
<td>Reach - 4</td>
<td>31238.9</td>
<td>Uniform Lateral Inflow</td>
</tr>
<tr>
<td>Main Stem</td>
<td>Reach - 5</td>
<td>24100</td>
<td>Stage Hydrograph</td>
</tr>
<tr>
<td>Town Creek</td>
<td>All</td>
<td>170012</td>
<td>Flow Hydrograph</td>
</tr>
<tr>
<td>Town Creek</td>
<td>All</td>
<td>163512</td>
<td>Uniform Lateral Inflow</td>
</tr>
<tr>
<td>Town Creek</td>
<td>All</td>
<td>144672</td>
<td>Lateral Inflow Hyd.</td>
</tr>
<tr>
<td>Town Creek</td>
<td>All</td>
<td>94000</td>
<td>Lateral Inflow Hyd.</td>
</tr>
<tr>
<td>Town Creek</td>
<td>All</td>
<td>49300</td>
<td>Lateral Inflow Hyd.</td>
</tr>
<tr>
<td>Trantoe Creek</td>
<td>All</td>
<td>134631</td>
<td>Flow Hydrograph</td>
</tr>
<tr>
<td>Trantoe Creek</td>
<td>All</td>
<td>193633</td>
<td>Uniform Lateral Inflow</td>
</tr>
<tr>
<td>Trantoe Creek</td>
<td>All</td>
<td>174286</td>
<td>Lateral Inflow Hyd.</td>
</tr>
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<td>Lateral Inflow Hyd.</td>
</tr>
<tr>
<td>Trantoe Creek</td>
<td>All</td>
<td>132386</td>
<td>Lateral Inflow Hyd.</td>
</tr>
</tbody>
</table>
6. These boundary conditions were then linked to the DSS file generated via the DSS processing by selecting “Select DSS File and Path”, navigating to the saved file, and selecting the appropriate records. An example of this selection window is shown in the image below.

7. Initial conditions were set based on observed data close to each upstream boundary, so that they wouldn’t be too high or low so as to impact model stability. These values are shown in the following figure.
8. To ensure model stability, flow minimums were also added to the flow file, by selecting “Flow Minimum and Flow Ratio Table” was selected from the “Options” menu. These values were adjusted experimentally to obtain a stable simulation. The values of these minimum flow values are shown in the figure below.
In order to compare these simulations to observed data during the simulation, observed data were added from DSS files obtained for the stream gauges listed in Table 1. To select these, the Option menu is selected on the flow file window, then “Observed (Measured) Data” and “Time Series in DSS”. In this new window, the River, Reach and cross section location (“River Sta.”) where the observed time series are located were selected from the drop down menus, and then “Add selected location to table” was clicked. The associated DSS file was then selected from the drop down menu, or by navigating through folders by clicking the folder icon, and the “Select DSS Pathname” was clicked to assign the DSS file to the cross section.

The flow file was uniquely named, and then saved.
11. To run the simulation, “Unsteady Simulation Analysis” was selected from the main HEC-RAS window Run menu. The geometry file created in steps 1 and 2 and the flow file created in steps 3-6 were added to the analysis.

12. Under Options, Flow Roughness Factors was selected. These were assigned in previous modeling efforts on this basin to increase model stability. These values were applied to every reach in the basin, and their values are shown in the figure below.
13. The Programs to Run were the Geometry Preprocessor, the Unsteady Flow Simulation, and, when mapping was to be performed, the Post Processor, which was the step which required the most time in the simulation. The Computation Interval was set to 6 minutes, the Hydrograph Interval was set to 1 hour, and the Detailed Output Interval was set to 6 hours. An example of this set-up for Scenario 4 for the 1999 simulation period is shown in the following figure.
Appendix C
Procedures for GeoRAS Mapping Using a Manually Edited Bounding Polygon

Created by: Seann Reed
Modified by Kate E. Abshire: 3/20/2012

1. Create a new directory to store GeoRAS results.
2. Export sdf file from HEC-RAS for plan and profiles of interest.
   Example:
   
   ![GIS Export interface](image)

   3. Open a blank ArcMap project
   4. Convert the sdf file to xml
   5. Save the ArcMap project. This step is required to proceed with the following steps.
   6. Run layer setup. Example:
7. Run “Import RAS Data”

8. Save the project and close ArcMap.

9. Open ArcCatalog, locate the BoundingPolygons class and “Load” data from a previously edited bounding polygon data set. Accept all of the default options from the dialog choices.

10. Re-open your ArcMap project and examine the table describing the BoundaryPolygons data set. There are two polygon records as illustrated in the screen shot below. By highlighting the records, it can be observed that one is the original bounding polygon defined by the cross-section extent and one is an edited bounding polygon.
11. Right click on the Bounding Polygon layer and select Edit Features ◊ Start Editing. Now delete the first record from the Bounding Polygon Table, save the edits, and stop editing.

12. Select Layer Setup and use the Existing Analysis option.

13. Run “Import RAS Data” again. The edited shape definition for the Bounding Polygon does not get over-written. The purpose of this step is just to get HEC-GeoRAS to re-recognize the input layers.

14. Run “Water Surface Generation”

15. Run “FloodPlain Delineation Using Rasters”. The screenshot below shows an example of the results, zoomed in on the lower part of Town Creek where the cross-sections are too narrow and the edited bounding polygon is much different from the original.
Appendix D

Inundation Mapping in Greenville, NC: High Water during Hurricane Floyd

Flood Boundary Comparison

- Scenario 1
- Scenario 2
Flood Boundary Comparison

- Scenario 2
- Scenario 3
Flood Boundary Comparison

- Scenario 3
- Scenario 4
References


Reed, S. (2010). Lessons Learned from Transitioning NWS Operational Hydraulic Models to HEC-RAS. Presented at the ASCE-EWRI World Water Congress 2010, Providence, RI.


106

