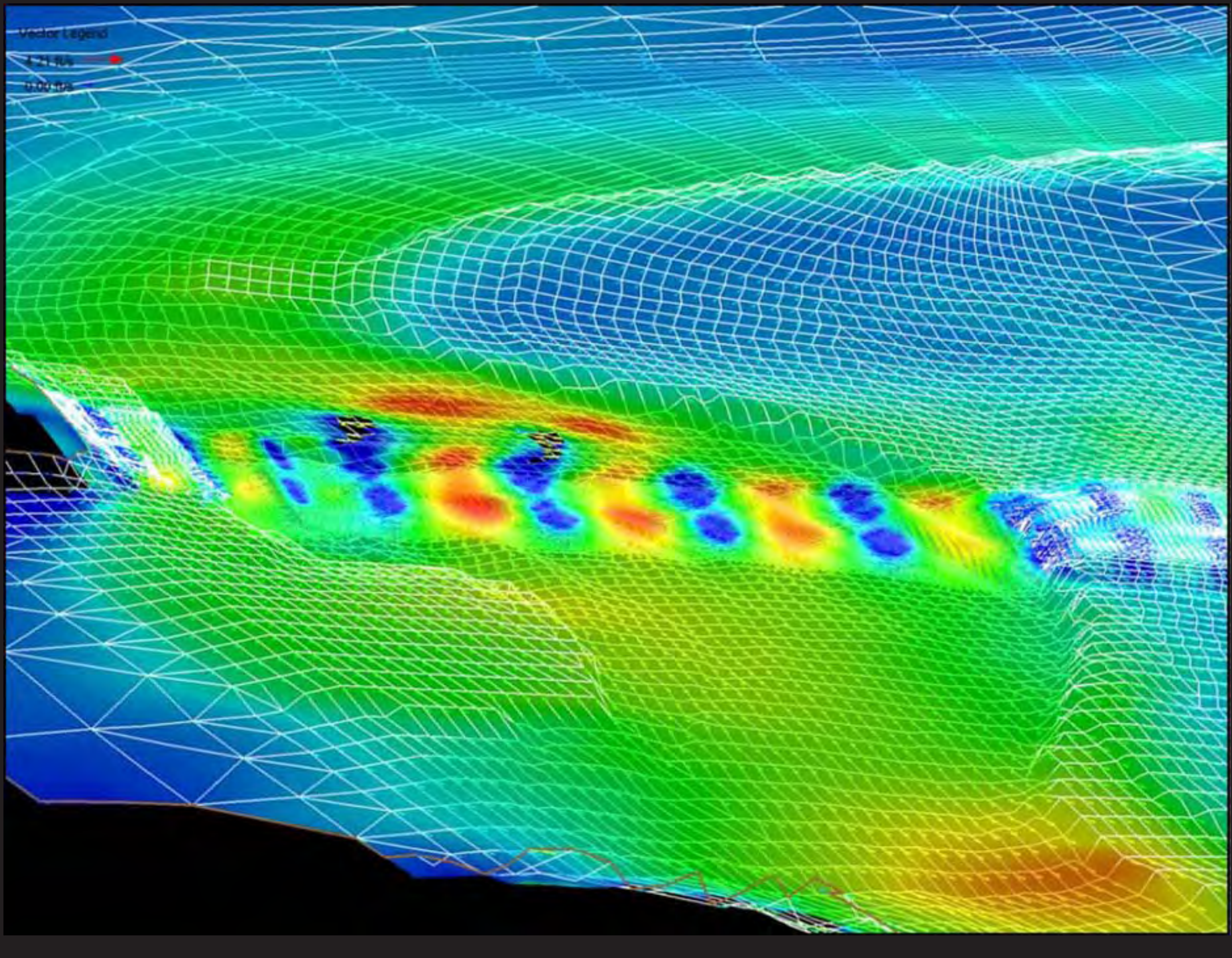
Prepared in cooperation with the North Carolina Department of Transportation

# Simulation of Water-Surface Elevations and Velocity Distributions at the U.S. Highway 13 Bridge over the Tar River at Greenville, North Carolina, Using One- and Two-Dimensional Steady-State Hydraulic Models



Scientific Investigations Report 2007–5263

U.S. Department of the Interior
U.S. Geological Survey

**Cover.** Three-dimensional rendering of the Tar River bathymetry through the U.S. Highway 13 bridge near Greenville, North Carolina, overlain with a simulated two-dimensional velocity field output from the Finite Element Surface-Water Modeling System (FESWMS).

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**U.S. Department of the Interior**
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# Conversion Factors

Inch/Pound to SI

| Multiply | By | To obtain |
|---|---|---|
| | Length | |
| inch (in.) | 2.54 | centimeter (cm) |
| foot (ft) | 0.3048 | meter (m) |
| mile (mi) | 1.609 | kilometer (km) |
| | Area | |
| square foot ($ft^2$) | 0.09290 | square meter ($m^2$) |
| square mile ($mi^2$) | 2.590 | square kilometer ($km^2$) |
| | Flow rate | |
| foot per second (ft/s) | 0.3048 | meter per second (m/s) |
| cubic foot per second ($ft^3/s$) | 0.02832 | cubic meter per second ($m^3/s$) |

Vertical coordinate information is referenced to the North American Vertical Datum of 1988 (NAVD 88).

Horizontal coordinate information is referenced to the North American Datum of 1983 (NAD 83).

Elevation, as used in this report, refers to distance above the vertical datum.

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# Simulation of Water-Surface Elevations and Velocity Distributions at the U.S. Highway 13 Bridge over the Tar River at Greenville, North Carolina, Using One- and Two-Dimensional Steady-State Hydraulic Models

By Chad R. Wagner

## Abstract

The use of one-dimensional hydraulic models currently is the standard method for estimating velocity fields through a bridge opening for scour computations and habitat assessment. Flood-flow contraction through bridge openings, however, is hydrodynamically two dimensional and often three dimensional. Although there is awareness of the utility of two-dimensional models to predict the complex hydraulic conditions at bridge structures, little guidance is available to indicate whether a one- or two-dimensional model will accurately estimate the hydraulic conditions at a bridge site.

The U.S. Geological Survey, in cooperation with the North Carolina Department of Transportation, initiated a study in 2004 to compare one- and two-dimensional model results with field measurements at complex riverine and tidal bridges in North Carolina to evaluate the ability of each model to represent field conditions. The field data consisted of discharge and depth-averaged velocity profiles measured with an acoustic Doppler current profiler and surveyed water-surface profiles for two high-flow conditions. For the initial study site (U.S. Highway 13 over the Tar River at Greenville, North Carolina), the water-surface elevations and velocity distributions simulated by the one- and two-dimensional models showed appreciable disparity in the highly sinuous reach upstream from the U.S. Highway 13 bridge. Based on the available data from U.S. Geological Survey streamgaging stations and acoustic Doppler current profiler velocity data, the two-dimensional model more accurately simulated the water-surface elevations and the velocity distributions in the study reach, and contracted-flow magnitudes and direction through the bridge opening.

To further compare the results of the one- and two-dimensional models, estimated hydraulic parameters (flow depths, velocities, attack angles, blocked flow width) for measured high-flow conditions were used to predict scour depths at the U.S. Highway 13 bridge by using established methods. Comparisons of pier-scour estimates from both models indicated that the scour estimates from the two-dimensional model were as much as twice the depth of the estimates from the one-dimensional model. These results can be attributed to higher approach velocities and the appreciable flow angles at the piers simulated by the two-dimensional model and verified in the field.

Computed flood-frequency estimates of the 10-, 50-, 100-, and 500-year return-period floods on the Tar River at Greenville were also simulated with both the one- and two-dimensional models. The simulated water-surface profiles and velocity fields of the various return-period floods were used to compare the modeling approaches and provide information on what return-period discharges would result in road overtopping and(or) pressure flow. This information is essential in the design of new and replacement structures.

The ability to accurately simulate water-surface elevations and velocity magnitudes and distributions at bridge crossings is essential in assuring that bridge plans balance public safety with the most cost-effective design. By compiling pertinent bridge-site characteristics and relating them to the results of several model-comparison studies, the framework for developing guidelines for selecting the most appropriate model for a given bridge site can be accomplished.

## Introduction

The use of one-dimensional hydraulic models currently is the standard method for estimating velocity fields through a bridge opening for scour computations and habitat assessments. Flood-flow contraction through bridge openings, however, is hydrodynamically two dimensional and often three dimensional. One of the most important factors in using numerical models to simulate flow contraction at bridges is the ability of the model to accurately represent the velocity distribution laterally across the stream and floodplain.

There is a growing emphasis on development and use of two-dimensional numerical models that produce more detailed and accurate analyses of river systems and bridge crossings. Until recently, the use of two-dimensional models required extensive field surveys and significant increases in personnel and computer resources compared with traditional one-dimensional approaches. With high-resolution topographic data becoming more readily available, the advent of graphical user interfaces for two-dimensional modeling software, and continued improvements in computer hardware, significant advances have been made in the ability to apply two-dimensional models to solve practical problems. Although there is awareness of the utility of two-dimensional models to predict the complex hydraulic conditions at bridge structures, little guidance is available to indicate whether a one- or two-dimensional model will accurately estimate the hydraulic conditions at a bridge site. Criteria also are needed for simulating hydraulics through bridges for evaluating alternative designs of single or multiple bridge openings in relation to overall structural cost, scour, backwater, and flood hazard mapping.

The U.S. Geological Survey (USGS), in cooperation with the North Carolina Department of Transportation (NCDOT), initiated a study in 2004 to compare one- and two-dimensional model results with field measurements at complex riverine and tidal bridges in North Carolina to evaluate the ability of each model to represent field conditions. The results of these model comparisons will provide an initial basis for development of modeling guidelines that will ensure cost-effective hydraulic analysis at bridges in other parts of the country.

## Background

The NCDOT Hydraulics Unit is responsible for providing preliminary designs for bridges, culverts, and other drainage features throughout the State, as well as providing information on wetlands, permit requirements, and water quality. During the design stage, the Hydraulics Unit develops detailed design recommendations for hydraulic structures, including bridges, box culverts, pipes, ditches, channels, stream relocations, and storm drainage systems. The Unit also is responsible for providing field data and hydraulic design recommendations to the NCDOT Bridge Maintenance Unit for the ongoing bridge replacement program.

In order to provide hydraulic design recommendations for the construction of new bridges and(or) embankments, the NCDOT uses numerical models to simulate bridge hydraulics for various flow conditions. One-dimensional step-backwater models, such as the Hydraulic Engineering Center-River Analysis System (HEC-RAS; U.S. Army Corps of Engineers, 2003) and Water-Surface Profile Computations (WSPRO; Shearman, 1990), are used almost exclusively by NCDOT to simulate the hydraulics for bridge design regardless of the site conditions. Many bridges, however, have site-specific characteristics that produce lateral and(or) vertical variability in the velocity distribution that cannot accurately be represented with a one-dimensional step-backwater model.

Site characteristics that can produce hydraulic complexities at a bridge include upstream channel alignment (meanders), geomorphic setting of the river valley, and floodplain alignment relative to the channel and hydraulic structures (levees, dikes). For example, flow distributions at flood stages can significantly be altered when flows leave a channel and enter a floodplain at a channel bend. When the channel bend is just upstream from a bridge, concentrated channel flows can be directed to a section of a bridge opening that would not typically experience this magnitude of flow if the channel were straight. Under these conditions, the flow fields are inherently two dimensional and cannot be accurately represented with a one-dimensional model. Guidance for selecting the appropriate model to simulate bridge hydraulics would improve the ability to accurately simulate velocity magnitudes and distributions at bridge crossings, which is essential in assuring that bridge plans balance public safety with the most cost-effective design.

## Purpose and Scope

The purpose of this report is to describe the development of one- and two-dimensional models to simulate hydraulic conditions at a complex riverine bridge site and to compare the modeling results to field data as a means of evaluating the capability of each model to accurately simulate field conditions. The results of this and other similar studies will build the knowledge base from which modeling guidelines can be developed. Such modeling guidelines can be valuable in directing engineers in the selection of the most appropriate modeling approach when new bridges are being designed.

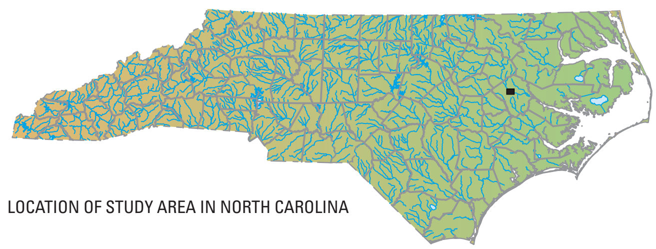
The selected study site was the U.S. Highway 13 (U.S.-13) bridge over the Tar River at Greenville, North Carolina, and the study reach extended approximately 2.9 miles upstream and 1 mile downstream from the U.S.-13 bridge. Detailed bathymetry data were collected and merged with light detection and ranging (LiDAR) topographic data for model development and velocity. Discharge and water-surface elevations were collected in the study reach during two high-flow events for model calibration and comparison. The two models used in the study are HEC-RAS (U.S. Army Corps of Engineers, 2003) to simulate one-dimensional hydraulics and the Finite Element Surface-Water Modeling System (FESWMS; Froehlich, 2002) to simulate two-dimensional, vertically averaged hydraulics. Scour estimates also were compared by using the scour prediction equations from Hydraulic Engineering Circular-18 (HEC-18; Richardson and Davis, 2001) and input parameters derived from the HEC-RAS and FESWMS models. This information will help provide a basis for ongoing development of modeling guidelines that will ensure cost-effective hydraulic analysis.

## Study Site and Data

The U.S.-13 bridge over the Tar River at Greenville, North Carolina, was selected for study because of (1) its abutment slope stability problems, which stem from a highly skewed channel configuration directly upstream from the bridge (fig. 1), and (2) plans by the NCDOT for a replacement bridge, for which model results would be a valuable design tool. The U.S.-13 bridge is skewed approximately 60 degrees to the direction of flow in the Tar River. The U.S.-13 bridge over the Tar River spans 540 feet (ft), has an average roadway elevation of approximately 27 ft (North American Vertical Datum of 1988 (NAVD 88)), and has a low-steel elevation of approximately 24.5 ft, which induces pressure-flow conditions at the structure for the 500-year recurrence-interval flow (hereafter referred to as 500-year flood) that is estimated to be 73,000 cubic feet per second ($ft^3/s$).

There are two overflow bridges associated with U.S.-13 in the study area, both located on the left floodplain (fig. 1). Overflow bridge number 730057 begins approximately 760 ft to the northeast of the U.S.-13 bridge over the Tar River, has an average roadway elevation of approximately 24.5 ft and a low-steel elevation of approximately 20.5 ft, and spans approximately 255 ft. Overflow bridge number 730070 begins approximately 2,560 ft to the northeast of the U.S.-13 bridge over the Tar River, has an average roadway elevation of approximately 24.5 ft and a low-steel elevation of approximately 21.5 ft, and spans approximately 125 ft. The low point (21.5 ft above NAVD 88) on U.S.-13 in the study reach is located approximately 2,000 ft northeast of overflow bridge 730070.

Two high-flow hydrographic surveys were conducted in the study reach to collect discharge data, cross-sectional



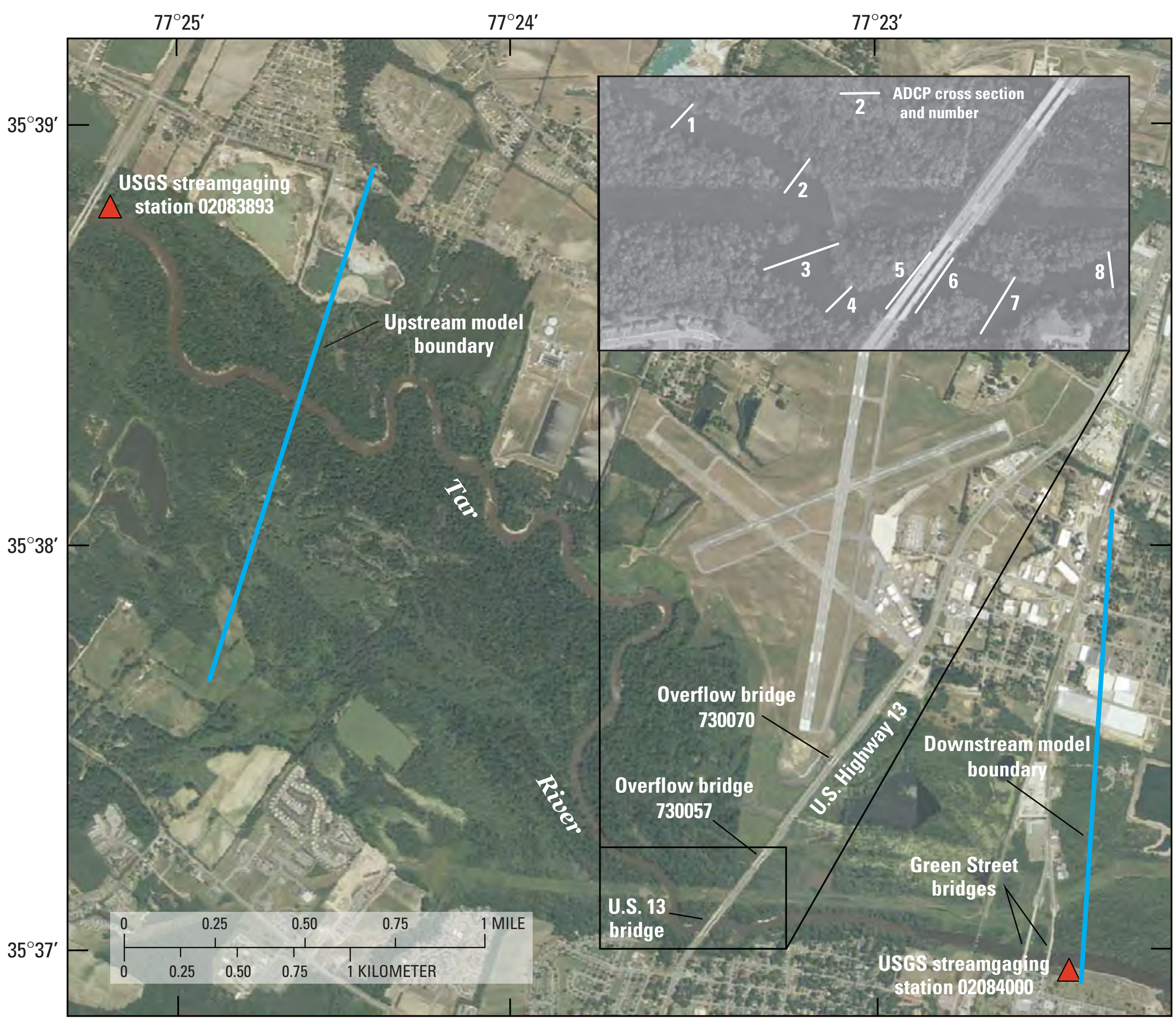




**Figure 1.** Aerial view of the study area on the Tar River at Greenville, North Carolina, 2003. [ADCP, acoustic Doppler current profiler]

velocity profiles (fig. 1 inset), and water-surface elevation data concurrently for model calibration and to provide a basis for model comparisons. The first measured high-flow condition was caused by precipitation associated with Hurricane Charley in August 2004 and the second was caused by precipitation from Tropical Storm Alberto in June 2006. The survey in 2004 was made approximately 18 hours prior to the storm's peak flow at Greenville, North Carolina, and the survey in 2006 captured the storm's peak flow. The measured discharges during the 2004 and 2006 surveys were 11,500 $ft^3/s$ and 21,000 $ft^3/s$, respectively, both of which are less than the estimated 10-year return-period flow. Large floating debris rafts between pile bent 2 and 5 were observed during both high-flow hydrographic surveys (fig. 2). The general location of the debris accumulation was consistent for both high-flow surveys; however, the spatial extent of the debris varied. The debris raft extended from the upstream face to the downstream face of the parallel bridges during the Tropical Storm Alberto (21,000 $ft^3/s$) condition but was more restricted to the upstream bridge face during the Hurricane Charley (11,500 $ft^3/s$) condition. It was evident from the field measurements that the debris had a major effect on flow through the bridge; therefore, the general debris configurations observed in the field were incorporated into the models. Methods and specific information regarding the data collection and how the data were used to develop, calibrate, and compare the models are discussed in the Methods section.

A.

B.


**Figure 2.** Debris accumulation (A) upstream from U.S.-13 and (B) between the parallel bridges of U.S.-13 over the Tar River at Greenville, North Carolina, on June 20, 2006.

# Methods

The availability of reliable bathymetric and topographic information is key to the development of reliable hydraulic models. The bathymetric and topographic data that were collected and processed for the model development are described in this section. Field data-collection techniques used to collect model calibration and comparison data (water-surface profiles, velocity distribution, and discharge) are also described. Comparisons of the one- and two-dimensional modeling results to the field data provided the basis for evaluating the differences between the two models and the implications of the differences. The model comparisons also provide valuable information for the development of guidelines for applying two-dimensional models at bridge sites.

## Data Collection

Channel bathymetry data were collected from a moving boat by using a 200-kilohertz (kHz) single-beam echo sounder and differentially corrected global positioning system (DGPS). The data were collected over a 2-day period in December 2004. The echo sounder is specified by the manufacturer to have an accuracy of 1 centimeter (cm) at one standard deviation. The DGPS unit is specified by the manufacturer to be accurate to 3.3 ft at two standard deviations; tests and prior use of this unit indicate that typically about 80 percent of the data are within 3.3 ft of the true location. Water-surface elevations, discharge, and detailed water velocities were measured at two different high-flow conditions (11,500 and 21,000 $ft^3/s$). The 11,500 $ft^3/s$ flow condition was measured on August 19, 2004, and was a result of rainfall produced by the passage of Hurricane Charley over eastern North Carolina. The water surface measured at USGS streamgaging station 02084000 (located at the downstream boundary of the modeled reach) during the survey on August 19, 2004, was 10.7 ft (NAVD 88), which is 1.3 ft above the Tar River flood stage at Greenville, North Carolina (table 1). The 21,000 $ft^3/s$ flow condition was measured on June 20, 2006, and was a result of rainfall from the remnants of Tropical Storm Alberto passing over North Carolina. The water surface measured at USGS streamgaging station 02084000 during the survey on June 20, 2006, was

**Table 1.** Summary of field data collection in the study reach at the U.S. Geological Survey streamgaging station on the Tar River at Greenville, North Carolina.

[USGS, U.S. Geological Survey; NAVD 88, North American Vertical Datum of 1988; ft, feet; ft/ft, foot per foot; $ft^3/s$, cubic feet per second]

| Measurement date | Water-surface elevation at USGS streamgaging station 02084000 (ft above NAVD 88) | Distance above flood stage (ft) | Water-surface slope in study reach (ft/ft) | Range in measured discharge ($ft^3/s$) |
|---|---|---|---|---|
| 8/19/04 | 10.7 | 1.3 | 0.00017 | 11,300 – 11,600 |
| 6/20/06 | 14.7 | 5.3 | 0.00015 | 20,900 – 21,700 |

14.7 ft (NAVD 88), which is 5.3 ft above the Tar River flood stage at Greenville, North Carolina (table 1). Water-surface elevations for both flow conditions were measured at three locations on the Tar River, two of which are located within the study reach and the other located 1 mile upstream from the study reach. Detailed water-velocity measurements and channel-bathymetry data were collected at eight cross sections during each of the hydraulic surveys (fig. 1 inset).

## Bathymetry and Topography

Bathymetry data were collected along cross sections spaced approximately one channel width apart (200 ft) throughout the study area. The raw bathymetry data were collected and processed to filter out (1) problems related to the echo sounder processing a multiple-return acoustic signal in shallow water (which causes the measured depth to be twice the actual depth), (2) GPS problems, and (3) redundant areas along the banks caused by failure to properly end data collection at various cross sections. Analog printouts of the bottom profiles were produced as part of the data-collection process and used as a quality-assurance measure because multiple-return errors can easily be identified in the analog printouts. The processed bathymetry data (including edge of water points) were exported into a text file that included geographic coordinates and a corresponding depth for 182,449 surveyed data points on 118 cross sections. The water depths were then subtracted from the water-surface elevation to establish an elevation for the streambed. The water-surface profile through the surveyed reach was established using tapedowns from a reference mark on the U.S.-13 bridge and measured water levels at USGS streamgaging stations 02083893 and 02084000 located approximately 1 mile upstream and at the downstream extent of the study reach, respectively (fig. 1). An average water-surface slope for the survey period (December 2–3, 2004) was computed and used to adjust the water surface from which the streambed elevations were computed throughout the reach.

Development of the model required accurate land-surface elevation data. In January 2001, contractors for the North Carolina Floodplain Mapping Program (FMP) used LiDAR systems to acquire elevation data along with the horizontal position of each elevation data point for the Tar River basin. Raw LiDAR data subsequently were processed to remove returns from objects, such as trees, buildings, and other structures. The initial data set, known as the bare earth mass points data, is distributed by the FMP free of charge through their Floodplain Mapping Information System (FMIS; *http://www.ncfloodmaps.com*). The root mean square error of the LiDAR data for all of the counties in the Tar River basin is less than 20 cm (7.9 inches), which is the vertical accuracy required by the FPM for acceptance of LiDAR data measured in coastal plain counties of North Carolina. LiDAR data for the floodplains in the study area were available from the FMP. Bare earth mass point data received from the FMP were a collection of irregularly spaced points. These data were reprocessed into a digital elevation model (DEM) with regularly spaced, 10-ft by 10-ft cells. An analysis was done by the USGS in North Carolina to determine the resolution for which to reprocess the bare earth LiDAR data that best balanced topographic accuracy and efficiency of use. For the coastal plain of North Carolina, a 10-ft by 10-ft grid was determined to be the best resolution. The DEM was created by first generating a triangulated irregular network (TIN). A TIN maintains the exact horizontal and vertical positions of the source data at the vertices of each triangle in the TIN, which maintains the integrity of the original data. A representation of major streams (known as breaklines) provided by the FMP was used to guide the interpolation along the edges of the triangles. The processed LiDAR data (10-ft by 10-ft grid) for the floodplain and overbank topography consisted of more than 2.5 million data points. The 10-ft by 10-ft overbank and floodplain DEM was merged with the bathymetry data using routines provided in the Surface-Water Modeling System (SMS), version 9.0 (Brigham Young University, 2005), modeling interface to develop a comprehensive topographic map for the model reach.

## Water-Surface Elevations

Water-surface elevations in the study reach were determined from USGS streamgaging stations 02083893 and 02084000 on the Tar River and by tapedown from a surveyed reference mark on the downstream guardrail of the U.S.-13 bridge (fig. 1). USGS streamgaging station 02084000 (Tar River at Greenville) was used as the downstream boundary for all modeling efforts.

## Velocity and Discharge

Water-velocity and discharge data were collected from a moving boat. The horizontal position of the boat was measured by use of a DGPS receiver. The DGPS system received its differential corrections from a local U.S. Coast Guard beacon.

Advances in velocity-measurement technology allow three-dimensional velocities to be measured from a moving boat by using an acoustic Doppler current profiler (ADCP) (Oberg and Mueller, 1994; Mueller, 1996; Oberg and others, 2005). All velocities were measured with an ADCP. Three-dimensional velocities were measured from approximately 3.5 ft beneath the water surface to within 6 percent of the depth to the bottom. Depths ranged from 11.5 to 29 ft for the August 19, 2004, survey and from 18 to 31.5 ft for the June 20, 2006, survey. Established methods were used to estimate the discharge in the unmeasured top and bottom portions of the profile (Simpson and Oltmann, 1991). Depth-averaged velocities were computed from the three-dimensional field data at each cross section; these discrete depth-averaged velocities were computed as an average of the measured velocity, so no velocities were computed in the unmeasured portions of the water column. Two cross-sectional measurements were made in succession at each of the eight velocity profile sections for quality assurance.

Discharge was also measured by the ADCP for each velocity profile section; however, these measurements were not representative of the total discharge because dense floodplain vegetation limited data collection by the ADCP to only the main channel. A separate total discharge measurement was collected during the surveys at a contracted bridge opening for the Highway 264 bypass (located approximately 2.5 miles downstream from the study reach) through which all of the Tar River flow passed. In order to compensate for unsteadiness in flow during the surveys, discharge and water-surface elevation measurements made at the beginning and end of the velocity data collection were averaged to produce a time-averaged flow rate and water-surface profile that were representative of the survey period (table 1).

## Scour Observations

Debris accumulation, high velocities, and shallow depths limited the ability to collect velocity and depth data from a moving boat directly adjacent to the abutments and piers. For the two high-flow hydrographic surveys, however, comparison of the measured cross sections upstream and downstream from U.S.-13 showed that the deepest section of the channel is located directly upstream from the right abutment. The actual streambed elevation directly upstream from the right abutment differed by less than 0.25 ft for the two high-flow surveys, which is within the error of the instrument and data-collection method in turbulent conditions. The streambed upstream from the U.S.-13 bridge near the right abutment was 4 to 5 ft deeper than the streambed directly downstream from the right abutment. The depths associated with the June 20, 2006, flood measurements were deep enough to collect data within approximately 25 ft of the left abutment, and the streambed at this location was about 1.25 ft deeper than the adjacent streambed. Data near the left abutment were not collected during the August 19, 2004, flood measurements because water depths were too shallow to collect ADCP data. The streambed downstream from the U.S.-13 bridge on the left side of the main channel (the area between piers 5 and 8, numbered from right to left looking downstream; fig. 3) was 5 to 6 ft deeper than the streambed upstream from the bridge, which indicates a combination of contraction and pier scour through the bridge. The streambed elevations through the bridge opening showed no change between the two high-flow measurements.

# Modeling

A two-dimensional and one-dimensional model of the study reach were developed separately using the field data described previously, 2-ft pixel black and white aerial photography, and bridge plans from the NCDOT. The domain for both models was identical for all simulated conditions. The modeled reach is 4 miles long, with floodplain widths that vary from 5,400 to 1,050 ft for the surveyed high-flow conditions. The lateral extent of the modeled reach coincides with the approximate peak water-surface elevation (30 ft above NAVD 88 at U.S.-13) measured following Hurricane Floyd in 1999, which was estimated to be a 500-year flood. Depths represented by the models ranged from 0.01 to 39.25 ft.

## Two-Dimensional Model

The two-dimensional, vertically averaged Finite Element Surface-Water Modeling System (FESWMS), version 3.0, was implemented by the USGS for the selected bridge site over the Tar River. FESWMS is currently supported by the Federal Highway Administration (FHWA) and has been described as being "designed specifically to analyze flow at bridge crossings where natural processes and fabricated structures have created complicated hydraulic conditions that are difficult to evaluate using conventional methods" (Froehlich, 2002). An earlier version of the model was successfully used to simulate bridge hydraulics and help compare predicted scour with measured scour at the U.S. Highway 70 bridge over Bear Creek near Mays Store, North Carolina, during Hurricane Floyd in 1999 (Wagner and others, 2005). Previous research has shown that the simulation of pressure flow at bridges is problematic in FESWMS, version 3.0 (Zevenbergen and others, 2002). Even for relatively simple cases, such as low discharges and flow velocities, numerical instability causes the model to diverge when the water surface reaches the bottom of a bridge deck. Similar problems were encountered during this study, which limited the accuracy of the hydraulics in the study reach for conditions that induced pressure flow. Increased roughness coefficients of 0.55–0.65 were used in an attempt to simulate the energy losses associated with pressure flow at the main channel and overflow bridges of U.S.-13.

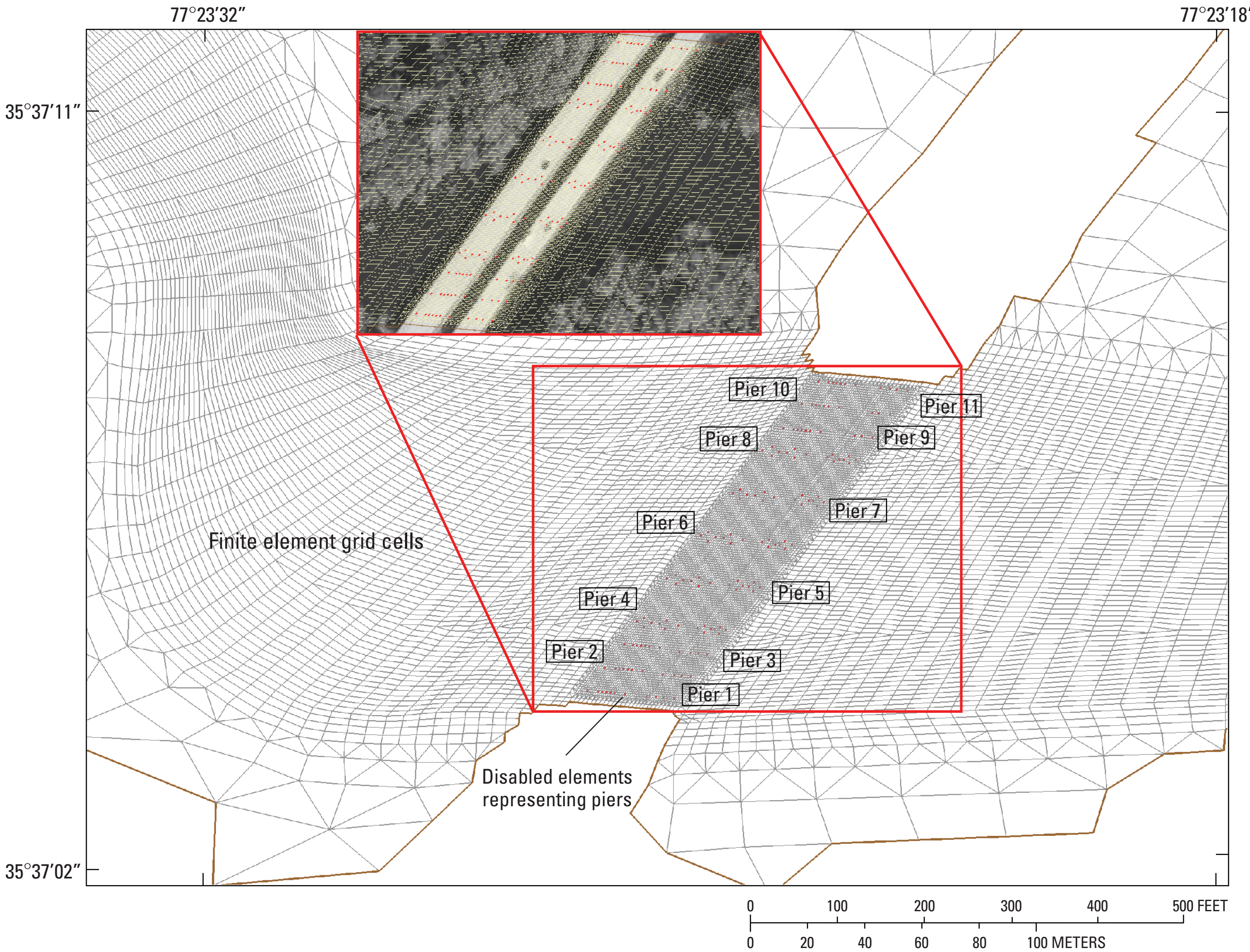



**Figure 3.** Finite element grid configuration through the U.S.-13 bridge over the Tar River at Greenville, North Carolina.

## Computational Grid

The finite-element mesh for the two-dimensional model consisted of 27,450 to 28,160 active grid cells (elements) ranging in size from 378,900 square feet ($ft^2$) on the floodplains to 235 $ft^2$ in the main channel upstream and downstream from U.S.-13 and 7 $ft^2$ in the main channel through the U.S.-13 bridge opening. The resolution of the grid was refined in the area surrounding the U.S.-13 bridge to improve simulation of small-scale hydraulic complexities caused by the structure (fig. 3).

Each element within the computational mesh was assigned a material type, which is associated with a unique set of hydraulic characteristics (Manning's *n*, eddy viscosity, marsh porosity, pressure-flow potential) that directly influences how water moves through an element. Aerial photography was used to assign material types to elements based on similar land-use characteristics (forest, residential, wetland, industrial). The actual hydraulic characteristics associated with each material type were initially assigned on the basis of published guidance and engineering judgment, then slightly adjusted during the calibration process. The final Manning's *n* roughness values ranged from 0.038 in the main channel to 0.14 on the floodplains, and base eddy viscosity values ranged from 9 pound-second per square foot (lb-sec/$ft^2$) for the main channel and overbanks to 15 lb-sec/$ft^2$ for all other material types. Eddy viscosity was allowed to vary from cell to cell according to the bed shear velocity, depth, and the horizontal dimensions of the cell using equation 1.

$$v_t = v_{t0} + c_{\mu 1}\mu^* H + c_{\mu 2}\left|J\right|\sqrt{\left(\frac{\delta U}{\delta x}\right)^2 + \left(\frac{\delta V}{\delta y}\right)^2 + \frac{1}{2}\left(\frac{\delta U}{\delta y} + \frac{\delta V}{\delta x}\right)^2}, \quad (1)$$

where:

$v_t$ = depth-averaged kinematic eddy viscosity;

$v_{t0}$ = base kinematic eddy viscosity;

$c_{\mu 1}, c_{\mu 2}$ = dimensionless coefficients;

$H$ = water depth;

$\mu^*$ = bed shear velocity;

$\left|J\right|$ = determinant of the jacobian matrix of element coordinate transformations, which provides point-wise measures of element area;

$\frac{\delta U}{\delta x}$ = partial differential of depth-averaged velocity in the *x* direction;

and

$\frac{\delta V}{\delta y}$ = partial differential of depth-averaged velocity in the *y* direction.

The dimensionless coefficients, $c_{\mu 1}$ and $c_{\mu 2}$, were assigned to be 0.3 and 0.1, respectively, for the channel and overbank elements and 0.9 and 0.5, respectively, for all other elements.

To correctly simulate bridge hydraulics, the roadways in the study area had to be positioned accurately in the model. A geo-referenced digital map of the roadway alignments and pier locations was unavailable and had to be developed in AutoCad using existing NCDOT bridge plans and aerial photography. The spatial orientation of the U.S.-13 and Green Street roadways and associated piers in the model was established using the developed map. The elements through the bridge section were sized to correspond with the footprint of the bridge piles (1.67 ft diameter). The 144 individual piles supporting the U.S.-13 bridge were modeled with no-slip conditions as disabled elements, which designate an element as a barrier to flow. Disabled elements and increased Manning's roughness values were used to simulate the floating debris accumulated through the U.S.-13 bridge opening. The elements directly in front of piers 2–5 (numbered from right to left, looking downstream; fig. 3) were disabled to represent the heavy debris that was accumulated against the piers. To simulate the observed floating debris, the elements through the bridge opening between piers 2–5 were assigned Manning's *n* values that varied by depth. The top 2.5 to 3.5 ft of water in these areas was assigned a Manning's *n* value that ranged from 0.075 to 0.20, whereas the remaining area of the water column below a depth of 3.5 ft was assigned a roughness value of 0.045.

### Boundary Conditions

Measured steady-state discharges of 11,500 and 21,000 $ft^3/s$ were used as the upstream boundary condition for the model, and the inflow distribution across the upstream boundary was based on depth. The downstream model boundary coincided with the USGS streamgaging station 02084000 (Tar River at Greenville), and the water-surface elevation was determined from water-level data collected at USGS streamgaging station 02084000.

## One-Dimensional Model

The one-dimensional, step-backwater model HEC-RAS was used for hydraulic modeling at the study site (U.S. Army Corps of Engineers, 2003). (Version 3.1.2, released in April 2004, was used for this application.) HEC-RAS is widely used for simulating steady-flow water-surface profiles in stream reaches and for one-dimensional hydraulic analysis at bridge crossings. The one-dimensional energy equation is solved within HEC-RAS for determination of water-surface profiles. The momentum equation can be included in the solution for situations in which the water-surface profile changes markedly with distance, such as at hydraulic jumps, bridges and culverts, and stream junctions. The effects of obstructions, such as bridges, culverts, weirs, and structures in the floodplain, are included in the hydraulic computations (U.S. Army Corps of Engineers, 2003). Unsteady flow simulations also can be performed using HEC-RAS.

### Model Development

The development of the HEC-RAS model relied entirely on the data sets used to develop the two-dimensional FESWMS model. Cross sections were extracted directly from the two-dimensional model's topographic data sets and imported to the HEC-RAS geometric data editor. Surveyed geometry and LiDAR data were used to incorporate the roadway and bridges associated with U.S.-13 and Green Street into the one-dimensional model by using the bridge design module within HEC-RAS. Manning's *n* values that were associated with the calibrated two-dimensional model were also transferred into HEC-RAS. The HEC-RAS model was calibrated using the two hydraulic conditions that were measured in the field and used in the FESWMS model.

The U.S.-13 bridge was modeled as one bridge in HEC-RAS with multiple openings (one main channel and two relief openings). The areas between the bridge openings were modeled as ineffective flow areas. Ineffective flow areas were also assigned in the bridge opening and contraction and expansion cross sections. A 1:1 contraction ratio (for example, if the model cross section is 10 ft upstream from the bridge, the effective flow area is 10 ft wider than the bridge opening on either bank) was used to assign the ineffective flow areas upstream from the bridge, and a 2:1 expansion ratio was applied downstream from the bridge. These ratios were applied using guidance from the HEC-RAS hydraulic reference manual (U.S. Army Corps of Engineers, 2003). Contraction and expansion coefficients (0.3 and 0.5, respectively) were elevated at the approach and exit sections and through the bridge opening relative to the coefficients in the other cross sections in the model (0.1 and 0.3, respectively).

Modeled flow conditions with water-surface elevations that were below the low chord of the U.S.-13 bridge were computed using the momentum, energy equation (standard step), and WSPRO bridge modeling routines. The routine that computed the highest energy loss through the bridge was used for the final solution. HEC-RAS also has the ability to compute flows that come into contact with the maximum low-chord elevation of the bridge by either the energy equation or by using separate equations for pressure and(or) weir flow. The energy-based method performs all computations as though they are open channel flow; however, the area obstructed by bridge piers, abutments, and bridge deck are subtracted from the flow area, and additional wetted perimeter is added. For the pressure and weir-flow method, pressure flow is computed using a sluice gate type of equation when only the upstream side of the bridge is in contact with the water, and the standard full-flowing orifice equation is used when both the upstream and downstream sides of the bridge are submerged. Flow over the bridge and(or) roadway is calculated using the standard weir equation. When the weir becomes highly submerged, HEC-RAS will automatically switch to calculating the upstream water surface by the energy equation instead of using the pressure and weir-flow equations. The criteria for when the model switches to the energy equation is based on percentage submergence of the bridge (submergence is defined as the depth of water above the minimum weir elevation on the downstream side of the bridge divided by the height of the energy gradeline above the minimum weir elevation on the upstream side) (U.S. Army Corps of Engineers, 2003). The default submergence criterion (95 percent) was used for the study. The energy equation solution was used to compute the energy loss through the U.S.-13 bridge for all simulated flow conditions, including flows that overtopped the roadway because the water-surface elevations of those overtopping conditions fully submerged the roadway/weir.

HEC-RAS allows the user to specify up to 45 locations in each cross section for which the program will compute flow distribution output. At each cross section where a flow distribution is specified, HEC-RAS will compute the flow, area, wetted perimeter, percentage of conveyance, and average velocity for each of the user-defined slices, referred to hereafter as conveyance tubes. Velocity distributions were extracted from the one-dimensional model and compared to two-dimensional model results and field data using the mean velocity output from 45 conveyance tubes distributed on cross sections that coincided with locations of field measurements.

### Boundary Conditions

The same steady-state discharges used in the two-dimensional model (11,500 and 21,000 $ft^3/s$) were used in the one-dimensional model. These discharges were used as the upstream-boundary condition for the model. The downstream head-boundary condition was determined from water-level data collected at USGS streamgaging station 02084000, and the downstream model extent coincided with the location of USGS streamgaging station 02084000.

# Simulation of Water-Surface Elevations and Velocity Distributions

Water-surface elevations and velocity distributions of both models were compared to measured field data to evaluate the ability of each model to represent field conditions. A summary of the calibration process and model comparisons is presented.

## Model Calibration

The two-dimensional model was initially calibrated to water-surface slope by using published roughness values and uniform eddy viscosity values, and not accounting for the accumulated debris through the bridge. Although measured and simulated water-surface slopes agreed for the August 19, 2004, (11,500 $ft^3/s$) conditions, there was disagreement between the measured and simulated velocity distributions. This is a noteworthy finding and demonstrates the values of site-specific velocity data for model calibration and testing. The roughness values and eddy viscosity terms were adjusted, and the mesh configuration through the U.S.-13 bridge was refined and incorporated the floating debris (as described previously) until the modeled water-surface elevations and velocities for the steady flow of 11,500 $ft^3/s$ matched those measured in the field. The steady-flow condition of 21,000 $ft^3/s$ was then simulated without changing the computational mesh or model parameters, and the simulated water-surface elevations and velocities were compared with those observed in the field. Minor adjustments were made to some material types and hydraulic parameters (mainly for those elements representing the floating debris accumulated through the bridge) such that model results displayed good agreement with field data for both flow conditions. Calibration results are presented in the following sections.

The Manning's *n* values used for the FESWMS model were also used initially to calibrate the HEC-RAS model to the measured water-surface elevation for both steady-flow conditions. Manning's *n* values in the main channel were increased slightly for the one-dimensional model to better match the measured water-surface elevation. The distribution of mean velocities in the conveyance tubes extracted from the one-dimensional model output were insensitive to reasonable changes (based on published values and engineering judgment) in Manning's *n* values and cross-sectional contraction and expansion coefficients; therefore, no adjustments could be made to the one-dimensional model during the calibration process to improve agreement between simulated and measured velocity distributions.

## Water-Surface Elevations

Although water-surface elevations were measured at three locations during each high-flow survey, the location of the two USGS streamgaging stations precluded them from being used to compare measured and simulated water-surface elevations. USGS streamgaging station 02083893 was useful in establishing the measured water-surface slope in the study reach; however, it is located approximately 1 mile upstream from the upstream model boundary and, therefore, cannot be compared directly to simulated results. Water-surface elevation data from USGS streamgaging station 02084000 are used as the downstream model boundary condition and hence does not provide a measure of model performance. Thus, the only location in which measured and simulated water-surface elevations were able to be compared was at the U.S.-13 bridge. A comparison of the simulated and measured water-surface elevations at the downstream side of the U.S.-13 bridge over the Tar River for the one- and two-dimensional hydraulic models is presented in table 2.

## Flow Continuity

Flow continuity was checked throughout the models to assure that (1) mass was being conserved within the two-dimensional model and (2) flow was being accurately distributed through the main channel and overflow bridges of U.S.-13 for both models. The two-dimensional model conserved mass throughout the domain for both flow conditions and provided better agreement with measured flow through the U.S.-13 main channel bridge than the one-dimensional model (table 3). Discharges for cross sections 4–7 in table 3 represent main channel flow only; vegetation and shallow depths prevented data collection on the floodplains. A tolerance of +/–3 percent in mass conservation discrepancy is typically acceptable for most hydraulic models (Donnell and others, 2005).

**Table 2.** Summary of water-surface elevation calibration for the Tar River at Greenville study reach.

[Values are in feet above North American Vertical Datum of 1988. 1D, one dimensional; 2D, two dimensional]

| Location | Hurricane Charley, August 2004 | | | Tropical Storm Alberto, June 2006 | | |
|---|---|---|---|---|---|---|
| | Measured water-surface elevation | 1D model water-surface elevation | 2D model water-surface elevation | Measured water-surface elevation | 1D model water-surface elevation | 2D model water-surface elevation |
| Downstream side of U.S.-13 bridge | 11.80 | 11.47 | 11.67 | 15.96 | 15.40 | 15.70 |

| Measured event | Water-surface elevation at the downstream side of U.S.-13 bridge | | |
|---|---|---|---|
| | Measured | 1D model | 2D model |
| Hurricane Charley (8/19/04) | 11.80 | 11.47 | 11.67 |
| Tropical Storm Alberto (6/20/06) | 15.96 | 15.40 | 15.70 |

**Table 3.** Summary of continuity checks for the modeled reach of the Tar River at Greenville, North Carolina.

[Values are in cubic feet per second. Cross section #, cross section numbers are referenced to the inset of figure 1]

| Location | Hurricane Charley, August 2004 | | | Tropical Storm Alberto, June 2006 | | |
|---|---|---|---|---|---|---|
| | Measured discharge | One-dimensional model discharge | Two-dimensional model discharge | Measured discharge | One-dimensional model discharge | Two-dimensional model discharge |
| Upstream model boundary | 11,500 | 11,500 | 11,500 | 21,000 | 21,000 | 21,000 |
| Cross section # 4 | 7,800 | 6,950 | 6,400 | 7,360 | 10,050 | 6,600 |
| Cross section # 5 (upstream side of U.S.-13 bridge) | 8,650 | 10,380 | 8,660 | 12,300 | 18,100 | 13,000 |
| Cross section # 6 (downstream side of U.S.-13 bridge) | 9,800 | 10,230 | 9,000 | 12,700 | 18,270 | 13,400 |
| Cross section # 7 | 5,900 | 7,050 | 5,000 | 10,970 | 10,700 | 10,560 |
| Downstream model boundary | 11,500 | 11,500 | 11,500 | 21,000 | 21,000 | 21,000 |

### Two-Dimensional Model Velocity Distribution

The two-dimensional model calibration process consisted of comparing measured and simulated cross-sectional velocity profiles at the four cross sections located adjacent to the U.S.-13 bridge (lines 4–7 in fig. 1 inset). The cross section along the downstream face of the bridge (line 6) is most indicative of the hydraulics through the bridge opening; therefore, the calibration process focused on matching the field data at that location. The simulated velocity distribution for line 6 displayed good agreement with field measurements for both flow conditions as illustrated by figure 4. The largest discrepancy between the simulated and measured velocity distribution exists near the right bank in an area influenced by heavy debris accumulation. The debris constricts the cross-sectional area of the channel in that region and, therefore, increases the flow velocity. The complex hydrodynamics induced by the debris accumulation are three dimensional in nature and cannot accurately be represented by a two-dimensional, vertically averaged hydraulic model. A three-dimensional model would provide a more representative simulation of the hydraulics through the U.S.-13 bridge opening for conditions in which accumulated debris is present. Despite the limitations of the two-dimensional model regarding simulation of the three-dimensional hydraulics around the accumulated debris, the shape of the field- and modeled-velocity distributions were similar, whereas the cross-sectional median difference between measured and simulated velocity magnitudes ranged from 0.40 to 0.69 feet per second (ft/s).

### Effects of Debris

Debris accumulation was determined to have an appreciable effect on the velocity distributions through the U.S.-13 bridge. Large floating debris rafts between piers 2 and 5 were observed during both hydrographic surveys (fig. 2). The general location of the debris accumulation was the same for both surveys, but the spatial extent of the debris varied. The debris raft extended from the upstream face to the downstream face of the parallel bridges during the June 20, 2006 (21,000 $ft^3/s$), condition but was more restricted to the upstream bridge face during the August 19, 2004 (11,500 $ft^3/s$), condition. The two-dimensional model calibration process initially did not take into account the debris accumulation at U.S.-13, but comparisons of modeled and measured velocity distributions made it evident that the debris had a major effect on flow through the bridge. Therefore, the general debris configurations observed in the field were incorporated into the two-dimensional model by disabling elements and increasing roughness values through the bridge, as previously described. As a result, measured and simulated velocities were more consistent with measured data (fig. 4). Debris was incorporated into the one-dimensional model by specifying a floating debris height and width for piers 2–5 in the pier data editor within the HEC-RAS graphical user interface that was consistent with the debris configuration used in the two-dimensional model.

The NCDOT current (2007) designation of debris potential at bridges is “high, moderate, or low” based on field observations and interviews with bridge maintenance personnel. The field observations during this study indicate that debris potential is moderate to high at the U.S.-13 bridge, which was shown to appreciably affect the velocity magnitudes and lateral distribution through the bridge.

## Model Uncertainty

Hydraulic models require three types of data: (1) topographic data for the hydraulic model computational grid; (2) effective friction values (Manning’s *n*) and eddy viscosity terms for each computational segment (one-dimensional model) or element (two-dimensional model); and (3) model validation data of some type (Bates and others, 2004). Uncertainties exist in each of these data types (Bales and others, 2006). More specific factors that introduce uncertainty into the Tar River at Greenville model are the debris configuration through the U.S.-13 bridge and potentially reduced floodplain storage from accumulated debris from past floods and passage of tropical systems (hurricanes and tropical storms).

### Topographic Data

Bathymetric data were collected for the model along cross sections spaced less than one channel width apart and with point spacing less than 0.25 ft. Despite the dense bathymetric data, uncertainty is introduced during the creation of a continuous bathymetric map of the channel from cross-sectional data. Surveyed bathymetric data do not fall on perfectly straight cross sections because of inconsistencies in the boat course across the river. Internal triangulation routines by the FESWMS interface software package Surface-Water Modeling System (SMS), version 9.0 (Brigham Young University, 2005), did not properly interpolate the collected bathymetry data (Wagner and Mueller, 2001). Instead, a uniform grid of the raw bathymetry data was developed using the channel template routine included in the Multi-Dimensional Surface-water Modeling System (MD-SWMS) interface developed by the USGS (McDonald and others, in press). The bathymetric data used to build the computational mesh are an interpolated representation of the raw data, which introduces model uncertainty, especially in the areas between measured cross sections and the region between the last measured bathymetric point and the bank point.

Uncertainty also exists in merging the bathymetric data with the LiDAR data. For many overbank areas within the model domain, the bathymetric data overlapped the LiDAR data. In these cases, visual inspection and orthophotography were used to determine which data to preserve. The interpretation of these overlapping regions introduced additional geometric uncertainty.

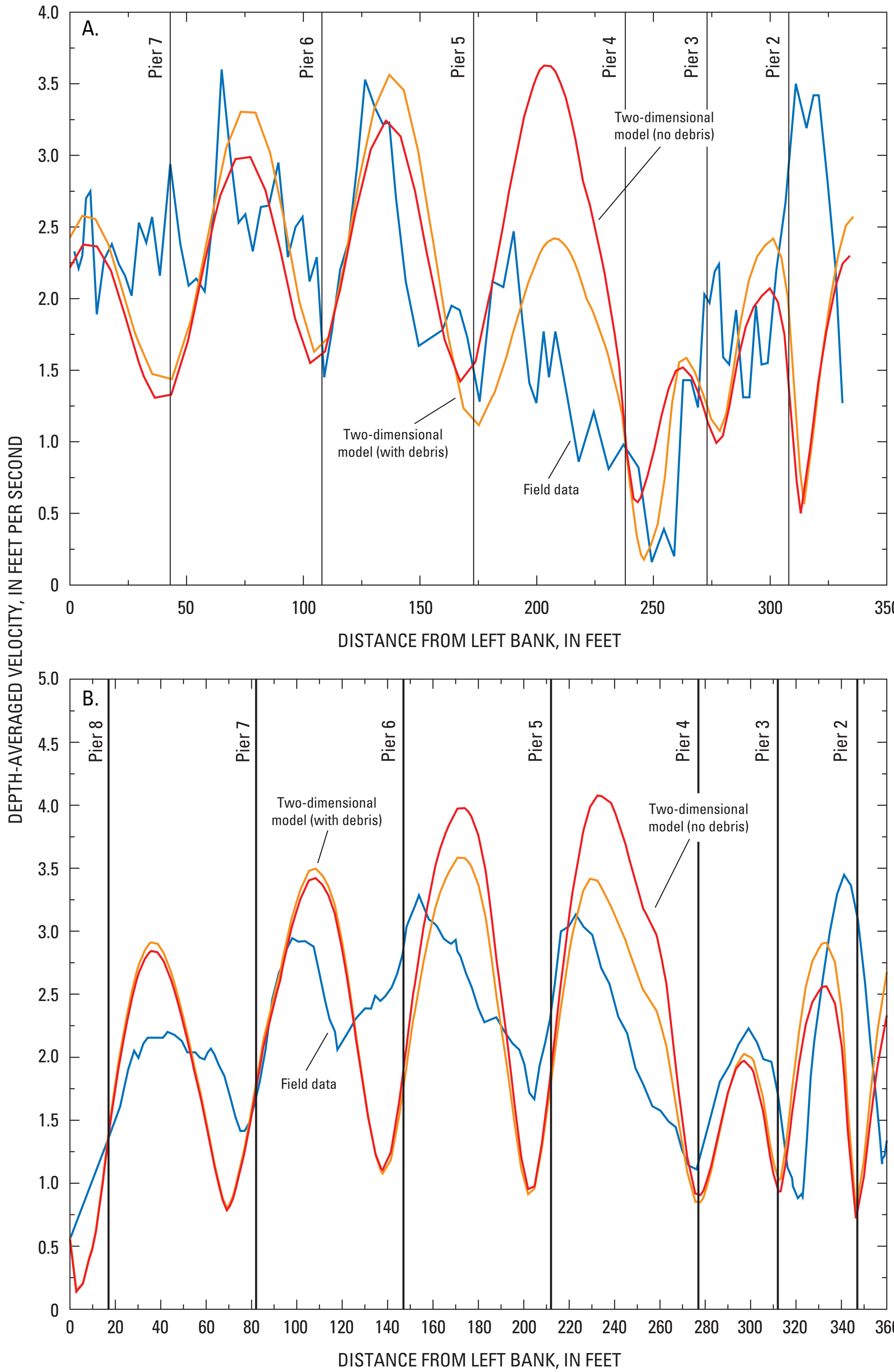



**Figure 4.** Measured and simulated velocity distributions through the U.S.-13 bridge over the Tar River at Greenville, North Carolina, for high-flow conditions associated with Hurricane Charley, August 2004, and (B) Tropical Storm Alberto, June 2006.

## Hydraulic Parameters

Friction values (Manning's *n*) and eddy viscosity values were assigned to the model on the basis of engineering judgment and in accordance with published literature. Digital orthophotography from 2003 was used to assign the hydraulic parameters to the elements (two-dimensional model) and cross sections (one-dimensional model) according to various land uses, such as residential, forest, wetland, and industrial. The orthophotography does not provide the resolution that windshield surveys and photographs can offer for estimating the hydraulic characteristics of each particular land use and, thus, can limit the accuracy of approximating model parameters.

## Model Calibration Data

The Tar River through the study reach was out of its banks for both high-flow calibration surveys. There are no contracted sections within the model domain in which to measure a total discharge with a manned boat; therefore, the survey crew had to go downstream approximately 2.5 miles to a fully contracted opening (U.S. Highway 264 Bypass) to measure the total discharge. There are no major tributaries to the Tar River between the model's upstream boundary and the location of the discharge measurement. However, depending on where on the flood hydrograph the discharge measurements were indicated to be made, water may be moving into or out of storage between the model's inflow boundary and the measurement location. Velocity profiles collected with the ADCP included random noise associated with the measurement of the Doppler shift off moving particles in the water column.

## Debris

Debris accumulation at the U.S.-13 bridge was observed by USGS personnel during the study to occur at high flows in excess of at least the estimated 1.5-year recurrence interval flow and had an appreciable effect on velocity distributions. Although debris was noted during both field surveys, photographs of the debris during the 11,500 $ft^3/s$ event were not taken. The actual debris configuration and depth below the water surface incorporated into the models required some estimation from survey crew field notes and bridge inspection reports from 2003.

Although the general location of the debris accumulation was similar between the two surveyed events, the extent and quantity of debris were different. Thus, there is uncertainty associated with modeling conditions that have not been surveyed because of the natural variability in debris supply and configuration for various flows.

There is also uncertainty associated with the technique used to model the debris in the two-dimensional model. The use of disabled elements (creates a barrier to flow in the model) to represent debris accumulated directly upstream from the piers may not be realistic because flow can convey through and under the floating woody debris. The true depth of floating debris was unknown; therefore, there is uncertainty associated with the procedure of assigning roughness values by depth to simulate the debris through the bridge.

## Overbank Flow and Debris

The flooding and wind damage throughout the Tar River basin as a result of Hurricanes Fran and Floyd in the late 1990s left large accumulations of debris on the floodplains of the Tar River and the main channels of most tributaries in the basin. Stage-discharge ratings at many USGS streamgaging stations in the Tar River basin underwent shifts as a result of the debris accumulation from Hurricanes Fran and Floyd. Following Hurricane Floyd in 1999 through 2002 and from mid-2004 through early 2006, much of eastern North Carolina was experiencing below-average precipitation or drought conditions, which reduced the opportunity for the removal of accumulated debris by high-flow events. The accumulated debris on the floodplains in the model reach may have affected the model calibration process by increasing roughness values above published values and decreasing the storage volume in the floodplains, which would reduce the natural tendency of flow to leave the channel and spread out across the floodplain. Comparisons of simulated and measured main channel discharge upstream and downstream from U.S.-13 show that the measured flow is higher for both surveyed conditions, which is consistent with increased roughness and reduced storage volume in the floodplains. Roughness values on the floodplains were increased above published values in the models, but the potential reduction in floodplain storage volume as a result of accumulated debris was not accounted for in the modeling process.

# Model Scenarios for Existing Bridge and Pre-Roadway Conditions

Computed flood-frequency estimates of the 10-, 50-, 100-, and 500-year return-period floods on the Tar River at Greenville were simulated with both the one- and two-dimensional models. The simulated water-surface profiles and velocity fields were used to compare the modeling approaches and provide information on what return-period discharges would result in road overtopping and(or) pressure flow, which is essential in the design of new and replacement structures. A pre-roadway construction scenario (excluded the U.S.-13 bridge and roadway) was simulated with both models for the 10-, 50-, 100-, and 500-year return-period floods and compared to simulations of the existing bridge conditions.

## Flood-Frequency Calculations

Flood-frequency estimates for the 10-, 50-, 100-, and 500-year discharges were taken from the Flood Insurance

Study Report for Pitt County, North Carolina, submitted to the Federal Emergency Management Agency (FEMA) in 2002 (Federal Emergency Management Agency, 2002). The published flood-frequency discharges for the Tar River in Pitt County did not directly coincide with USGS streamgaging station 02084000 (drainage area 2,660 square miles), but discharges were provided for locations upstream and downstream from the study reach (drainage areas of 2,521 and 2,757 square miles, respectively). The 10-, 50-, 100-, and 500-year return-period flows in the study reach (at USGS streamgaging station 02084000) were estimated by interpolating the locations of FEMA-published flood-frequency discharges using the drainage area ratio correction shown in equation 2 (table 4).

$$Q_G = Q_{222}(DA_G / DA_{222})^{C_{RP}} \qquad (2)$$

where:

$Q_G$ = x-year discharge on Tar River at Greenville,

$Q_{222}$ = published x-year discharge on Tar River at State Route (SR) 222,

x-year = specific flood-frequency return period (10, 50, 100, 500),

$DA_G$ = drainage area for Tar River at Greenville (2,660 square miles),

$DA_{222}$ = published drainage area for Tar River at SR 222 (2,521 square miles), and

$C_{RP}$ = coefficient from published (Pope and others, 2001) flood-frequency equations for x-year discharge in the Coastal Plain hydrologic area.

**Table 4.** Flood-frequency estimates for U.S. Geological Survey streamgaging station 02084000 (Tar River at Greenville, North Carolina).

[$ft^3/s$, cubic feet per second]

| Return period | Estimated discharge ($ft^3/s$) |
|---|---|
| 500-year | 73,000 |
| 100-year | 52,000 |
| 50-year | 44,000 |
| 10-year | 29,000 |

## Boundary Conditions

The flood-frequency discharges computed for USGS streamgaging station 02084000 were applied as the upstream boundary conditions for the 10-, 50-, 100-, and 500-year modeled scenarios. The water-surface elevations for the downstream boundary conditions were determined from the stage-discharge rating curve at USGS streamgaging station 02084000.

## Model Scenario Results

The results of the one- and two-dimensional models of the study area for existing and pre-roadway conditions were summarized to (1) provide information on the effect of U.S.-13 on hydraulics in the study area; (2) determine what return-period discharges would result in road overtopping and(or) pressure flow; and (3) compare the results of the one- and two-dimensional models.

### Water-Surface Elevations

There was generally good agreement between the water-surface elevations simulated by the one- and two-dimensional models downstream from the U.S.-13 bridge (fig. 5). However, appreciable disparity occurred between the one- and two-dimensional models in simulating water-surface elevations upstream from the U.S.-13 bridge. The water-surface elevations at the approach section of the U.S.-13 bridge for all modeled scenarios are summarized in table 5. The model results for the 500-year flood showed that aside from the U.S.-13 bridge, most of the U.S.-13 roadway in the study area is overtopped and that pressure flow occurs at the U.S.-13 bridge. For the 100-year return-period flow (hereafter referred to as the 100-year flood), the models did not indicate pressure flow at the U.S.-13 bridge, and roadway overtopping was confined to a region beginning approximately 2,000 ft northeast of the overflow bridge 730070 and extended to the model boundary on the left floodplain. The two-dimensional simulated water-surface elevation for the 100-year flood at the U.S.-13 upstream bridge face varied from 23.5 to 23.9 ft, which provides approximately 1.0 to 0.6 ft of freeboard below the low-steel elevation. Neither model indicated any road overtopping or pressure flow for the 50- and 10-year return-period discharges.

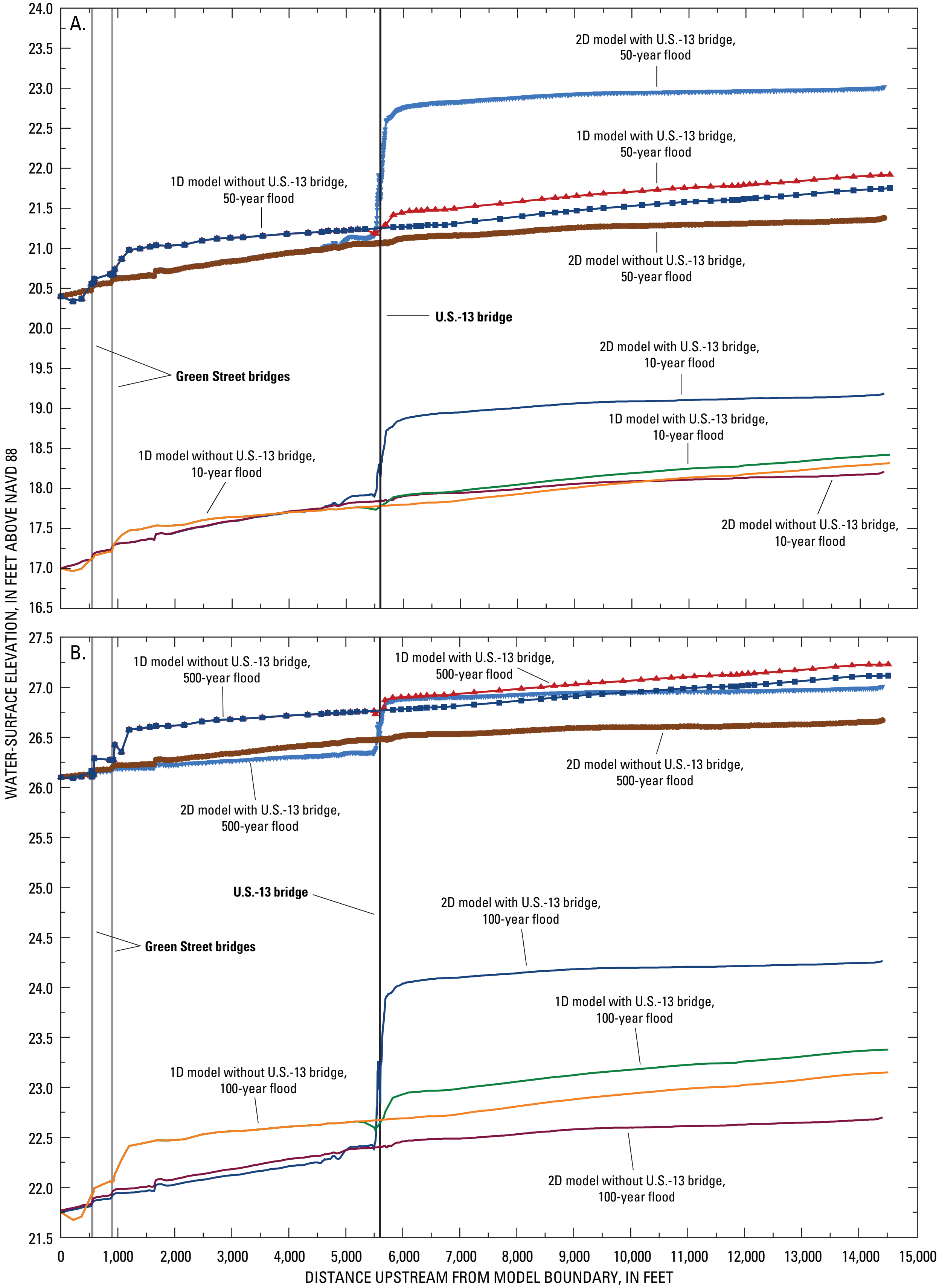



**Figure 5.** Modeled water-surface elevations for existing bridge and pre-roadway scenarios for (A) 50- and 10-year floods and (B) 500- and 100-year floods for the Tar River at Greenville, North Carolina, study area.

**Table 5.** Modeled water-surface elevations at the approach section of U.S.-13 bridge over the Tar River at Greenville, North Carolina, for the existing bridge and pre-roadway scenarios.

[Values are in feet. 2D, two dimensional; 1D, one dimensional]

| Return-period flood | 2D modeled water-surface elevation at approach with U.S.-13 bridge | 1D modeled water-surface elevation at approach with U.S.-13 bridge | 2D modeled water-surface elevation at approach without U.S.-13 bridge | 1D modeled water-surface elevation at approach without U.S.-13 bridge |
|---|---|---|---|---|
| 500-year | 26.88 | 26.90 | 26.52 | 26.78 |
| 100-year | 24.03 | 22.93 | 22.45 | 22.69 |
| 50-year | 22.74 | 21.44 | 21.12 | 21.27 |
| 10-year | 18.86 | 17.91 | 17.90 | 17.80 |

### Two-Dimensional Model Flow Fields

The flow fields associated with the two-dimensional simulations of the scenarios with and without the U.S.-13 bridge and roadway show the largest disparity in the areas around the bridges and along the roadway with the maximum velocity differences (ranging from 3 to 6.5 ft/s for the modeled floods) occurring at the location of the right abutment of overflow bridge 730057. Difference maps of the simulated flow fields for the model scenarios with and without the U.S.-13 bridge and roadway are illustrated in the appendix.

# Comparisons of One-Dimensional and Two-Dimensional Modeling Results to Field Data

The results of the one-dimensional (HEC-RAS) and two-dimensional (FESWMS) models of the U.S.-13 bridge over the Tar River at Greenville were compared to determine whether the two-dimensional model is a more appropriate tool for simulating site conditions than the one-dimensional model. Water-surface elevations, velocity distributions, and scour estimates are compared in the following sections.

## Water-Surface Elevations

The water-surface elevations simulated by the two-dimensional and one-dimensional models were within 0.2 and 0.6 ft, respectively, of the measured water-surface elevations on the downstream side of the U.S.-13 bridge, and the models are generally in good agreement downstream from the U.S.-13 bridge (fig. 6). However, appreciable disparity occurred between the one- and two-dimensional models in simulating water-surface elevations upstream from the U.S.-13 bridge. The disparity upstream from the U.S.-13 bridge can be attributed mainly to differences in momentum loss at U.S.-13. The simulated water-surface slopes for the two models are similar upstream from U.S.-13; however, the two-dimensional model simulates more energy loss through the U.S.-13 main channel bridge, U.S.-13 overflow bridges, and channel bend directly upstream from U.S.-13. The flow conditions directly upstream and through the U.S.-13 bridge are two dimensional in nature, and the associated energy losses are not fully represented by a one-dimensional model, which explains the large discrepancies in simulating the water-surface elevations (fig. 6).

To evaluate model performance in predicting water-surface elevations upstream from the U.S.-13 bridge, simulated water levels at the upstream model boundary for both high-flow events were compared to stage data collected at USGS streamgaging station 02083893 (Tar River at U.S. 264 Bypass near Rock Springs, North Carolina), located approximately 1 mile upstream from the upstream model boundary (fig. 1). The water-surface slope in the upper 3,500 ft of both models was used to extrapolate simulated water-surface elevations upstream to USGS streamgaging station 02083893. The comparison indicated that the extrapolated water levels from the one- and two-dimensional models were within 1.7 and 0.8 ft of the measured gage data, respectively. Therefore, based on the available data from USGS streamgaging station 02083893, the two-dimensional model more accurately simulated the water-surface elevations in the study reach.

To quantify the effect of debris on the water-surface elevations, the two-dimensional model was applied to the study reach to account for both the presence and absence of debris accumulation at the U.S.-13 bridge. For both high-flow events, the two-dimensional model that simulated debris resulted in slightly higher water-surface elevations upstream from the U.S.-13 bridge relative to the two-dimensional model that did not simulate debris (fig. 6). The debris (simulated with disabled elements and increased roughness values) impedes flow through the U.S.-13 bridge, thereby creating a damming effect that raises the upstream water level and is more in line with recorded stage data from USGS streamgaging station 02083893.

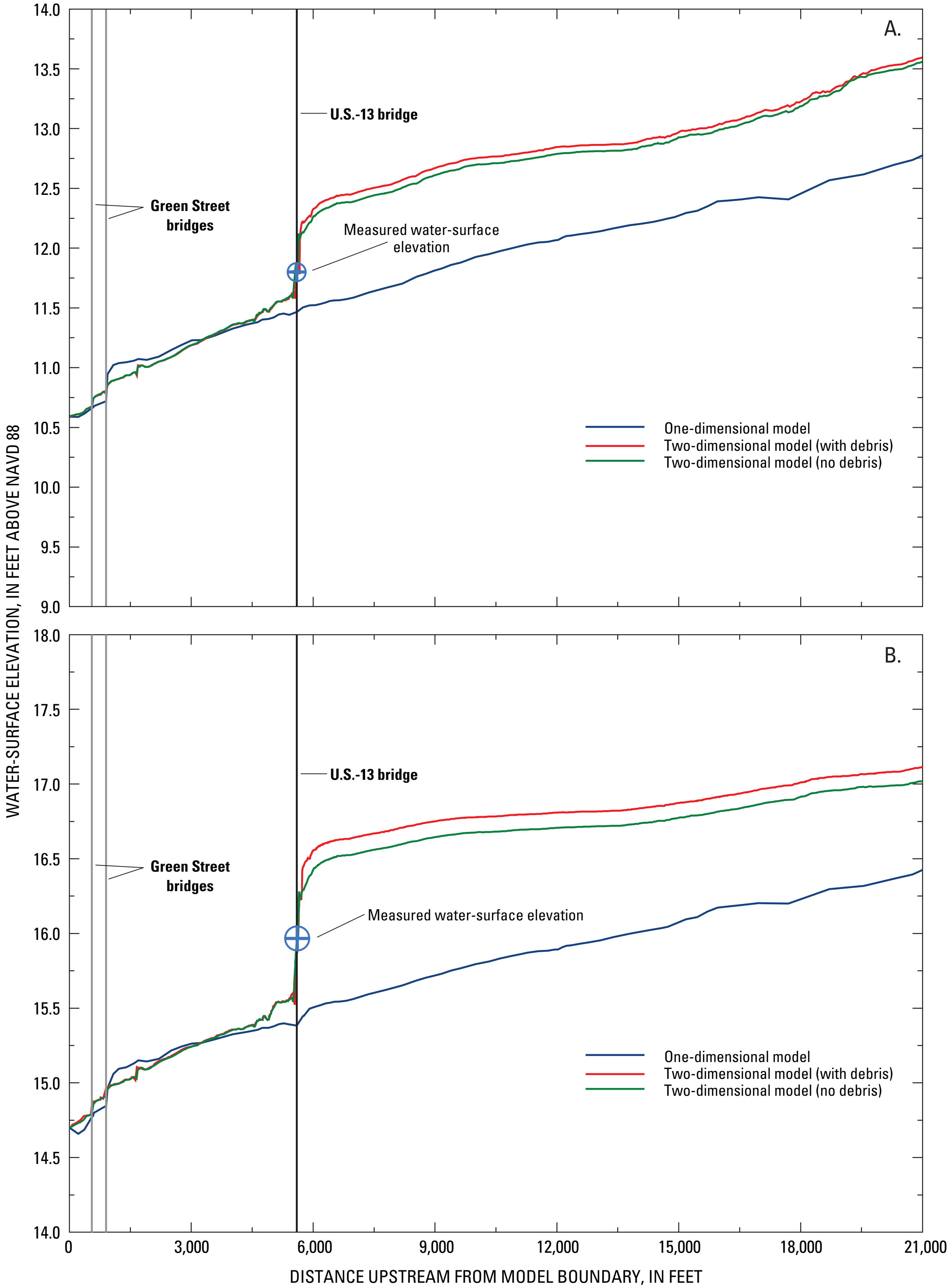



**Figure 6.** Modeled water-surface elevations for high-flow conditions associated with (A) Hurricane Charley in August 2004 and (B) Tropical Storm Alberto in June 2006.

## Velocity Distributions

Velocity distributions through bridge openings can have the greatest effect on the location of potential scour; therefore, accurate representations of these distributions are essential for cost-effective bridge design. To evaluate the capability of each model to accurately represent velocity distributions at the U.S.-13 bridge over the Tar River, simulated one- and two-dimensional velocity distributions were compared to field data (fig. 7). Velocity distributions were extracted from the one-dimensional model using the velocity from 45 conveyance tubes uniformly distributed through the U.S.-13 bridge opening and from the two-dimensional model based on 200 values extracted from the grid cells distributed across the bridge opening. The raw field data consisted of approximately 200 depth-averaged velocity measurements across the bridge opening. It is evident from the comparisons that the one-dimensional model is less representative than the two-dimensional models of measured velocity magnitudes and lateral distributions for the surveyed cross sections. In general, the velocities from the one-dimensional model were biased low and more uniformly distributed than velocities from the two-dimensional model and measured data. The mean differences in velocity between the two-dimensional model and measured field data for the Hurricane Charley (August 19, 2004) and Tropical Storm Alberto (June 20, 2006) conditions were 0.66 and 0.47 ft/s, respectively. The mean differences in velocity between the one-dimensional model and field data for the Hurricane Charley and Tropical Storm Alberto conditions were 0.84 ft/s and 0.93 ft/s, respectively.

A comparison of the maximum velocities simulated by the one- and two-dimensional models is summarized in table 6. The approach velocity is an important parameter used to predict scour at bridges. To further compare the simulated velocities, a summary of the approach velocities upstream from the U.S.-13 piers simulated by the one- and two-dimensional models is presented in table 7.

Debris accumulation at the U.S.-13 bridge had an appreciable effect on the velocity distributions simulated by the two-dimensional model (fig. 4). The difference was negligible between the one-dimensional model results with and without debris; therefore, both results are not depicted in figure 7 for the sake of legibility. The two-dimensional model, with debris factored in, most accurately simulated the contracted-flow distribution (fig. 7) and direction (figs. 8, 9) through the bridge opening, which makes it a better tool for safe and efficient bridge design.

The graphical and statistical comparisons of the modeled velocity distributions are slightly biased by the fact that there are different sample sizes between the output of the two models. Output from HEC-RAS is limited to a maximum of 45 individual velocity points or conveyance tubes in a cross section. Reducing the number of velocity points output from the two-dimensional model to be consistent with the one-dimensional model would greatly diminish the resolution provided by the two-dimensional simulation and, therefore, is not a practical approach. The quality of the statistical and graphical comparisons between the one- and two-dimensional models is limited by the difference in output capabilities of the model interfaces.

## Scour Estimates

To further compare the results of the one- and two-dimensional models, simulated hydraulic parameters (flow depths, velocities, attack angles, blocked flow width) for both measured high-flow conditions were used to predict scour depths at the U.S.-13 bridge by using the HEC-18 (Richardson and Davis, 2001) methods. The absence of available data on sediment size ($D_{50}$) in the study reach was a limiting factor for scour calculations; therefore, contraction scour was not computed because the corresponding HEC-18 equations require sediment size. Pier and abutment scour were computed because the prediction equations in HEC-18 do not require precise sediment-size information (aside from computing the $K_4$ factor for pier scour, which is included to account for armoring in course-grained soil but is not applicable to the Tar River at Greenville).

Pier-scour estimates using HEC-18 methods (Richardson and Davis, 2001) are sensitive to the attack angle of flow (the angle of flow directly upstream from the pier, measured in degrees from parallel or in line with the pier). Measured field data and the two-dimensional model results at the upstream bridge face were in good agreement (figs. 8, 9) and showed appreciable attack angles at 5 of the 11 piers, ranging from 30 to 80 degrees. The one-dimensional model is not capable of computing flow angles, yet the HEC-18 computations within HEC-RAS require manual input of those angles in order to compute pier scour. Without the field data and(or) two-dimensional model results, the accuracy of flow angles for pier-scour analysis using a one-dimensional model is limited to engineering judgment and visual estimations based on channel and floodplain alignment relative to the bridge. The flow angles used in the one-dimensional model scour computations were based on visual estimations of channel and floodplain alignment from 2-ft black and white aerial photography of the site. Pier-scour estimates using the hydraulic parameters derived from the one- and two-dimensional models for the two high-flow surveys are summarized in tables 8 and 9. Comparisons of pier-scour estimates from both models for the U.S.-13 bridge indicate that the two-dimensional scour estimates are more often deeper than those from the one-dimensional model. The two-dimensional pier-scour estimates differ by as much as 4.4 ft with a mean difference of 1.5 ft for the measured flow conditions relative to the one-dimensional model estimates. To account for potential shifting of the thalweg, NCDOT often uses the highest velocity flow tube in HEC-RAS to compute scour for all piers. This procedure was applied using the results of the HEC-RAS simulations and is summarized in tables 8 and 9. The use of the highest velocity tube estimate for all pier-scour estimates resulted in

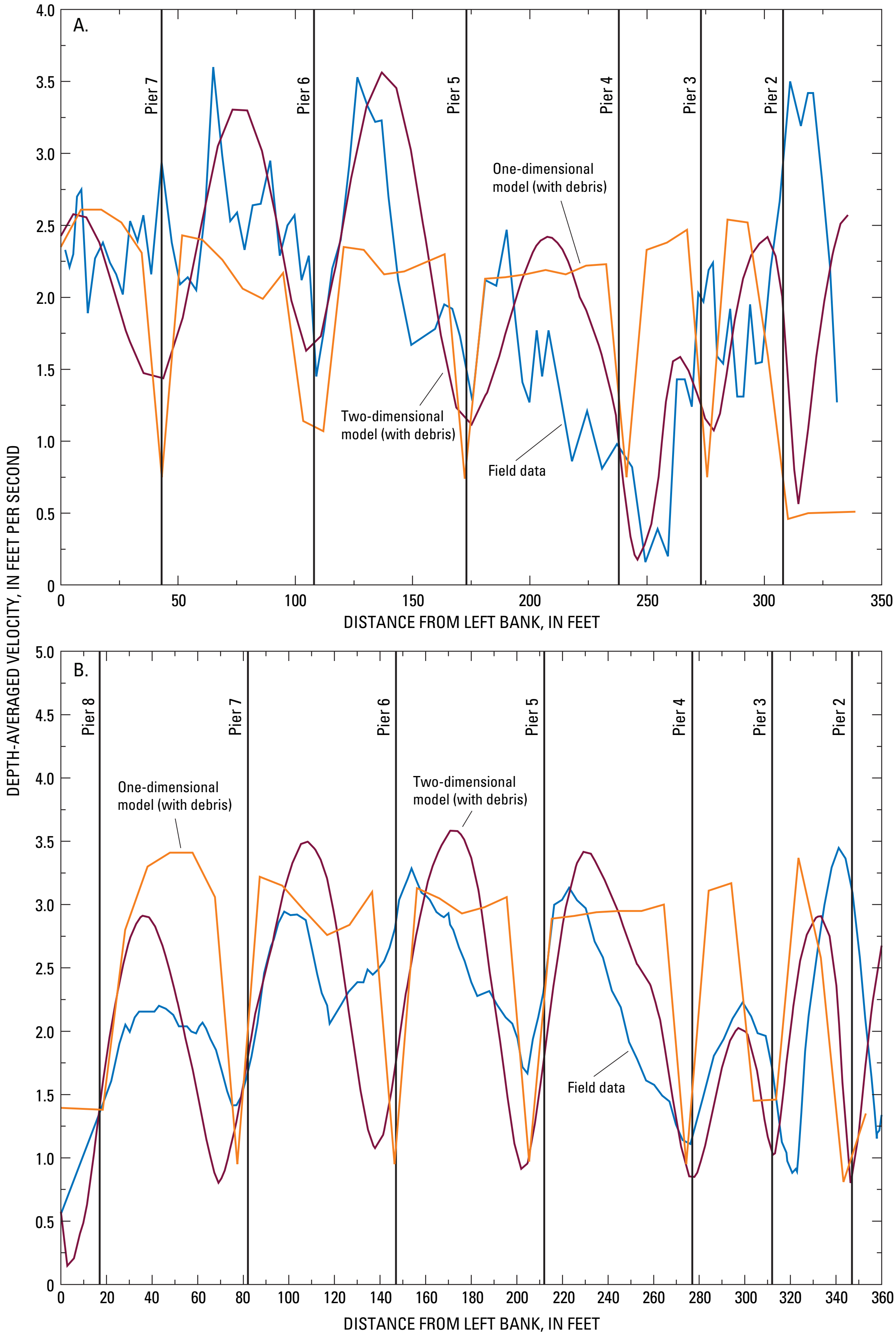



**Figure 7.** Measured and simulated velocity distributions through the U.S.-13 bridge over the Tar River at Greenville, North Carolina, for high-flow conditions associated with (A) Hurricane Charley, August 2004, and (B) Tropical Storm Alberto, June 2006.

**Table 6.** One- and two-dimensional model simulated maximum velocities for the main channel and overflow U.S.-13 bridges over the Tar River at Greenville, North Carolina.

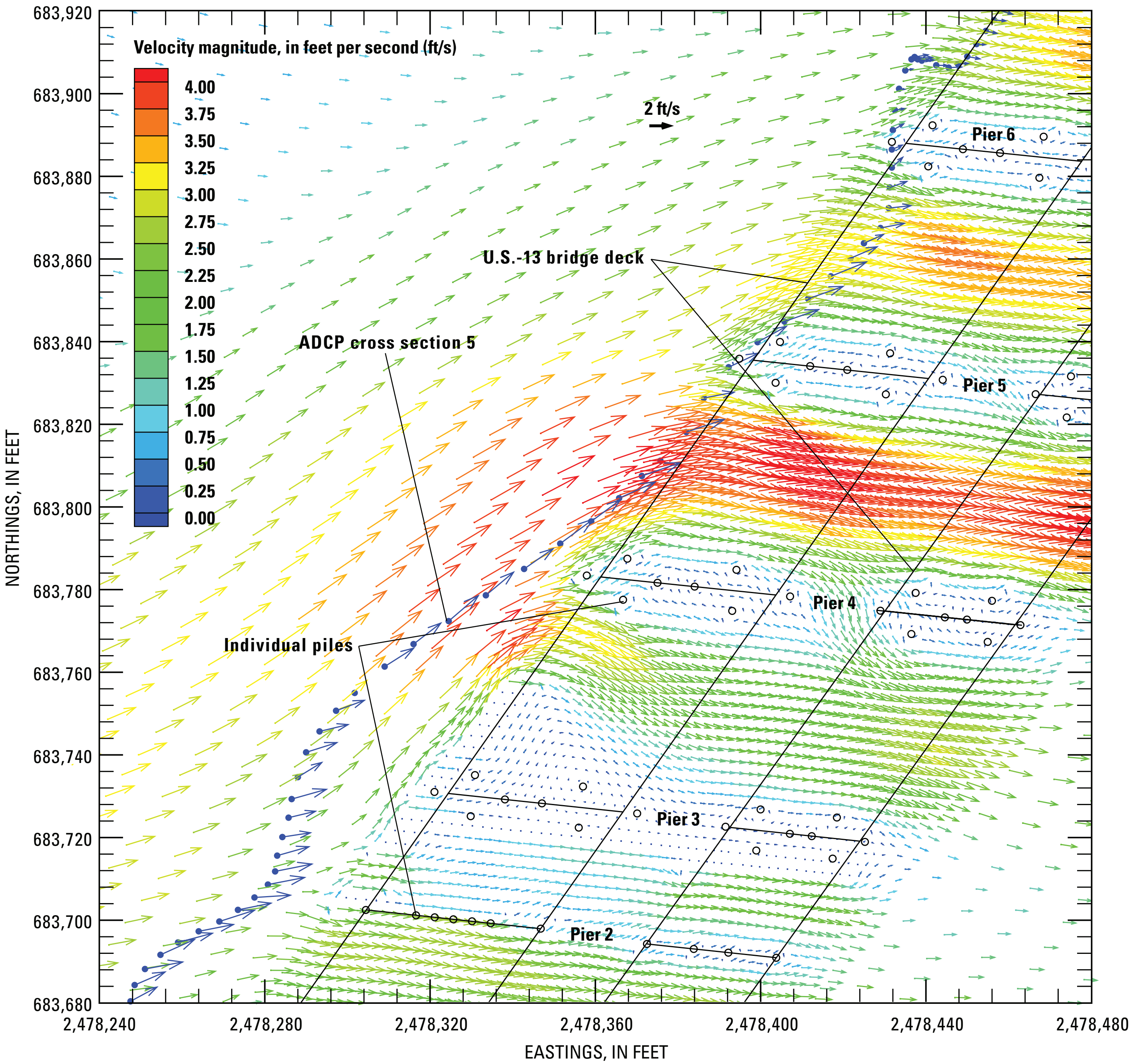
[Values are in feet per second. 2D, two dimensional; 1D, one dimensional]

| Location | Hurricane Charley, August 2004 | | Tropical Storm Alberto, June 2006 | |
|---|---|---|---|---|
| | 2D model maximum velocity | 1D model maximum velocity | 2D model maximum velocity | 1D model maximum velocity |
| U.S.-13 bridge main channel | 4.2 | 2.6 | 4.0 | 3.4 |
| U.S.-13 bridge left overbank | 1.1 | 0.94 | 1.9 | 1.5 |
| U.S.-13 bridge right overbank | 2.5 | 1.1 | 3.2 | 1.4 |
| Overflow bridge 730057 | 2.2 | 0.5 | 4.0 | 0.9 |
| Overflow bridge 730070 | 2.1 | 0.4 | 4.0 | 0.7 |

**Table 7.** One- and two-dimensional model simulated approach velocities for the piers supporting the U.S.-13 bridge over the Tar River at Greenville, North Carolina.

[Values are in feet per second. 2D, two dimensional; 1D, one dimensional]

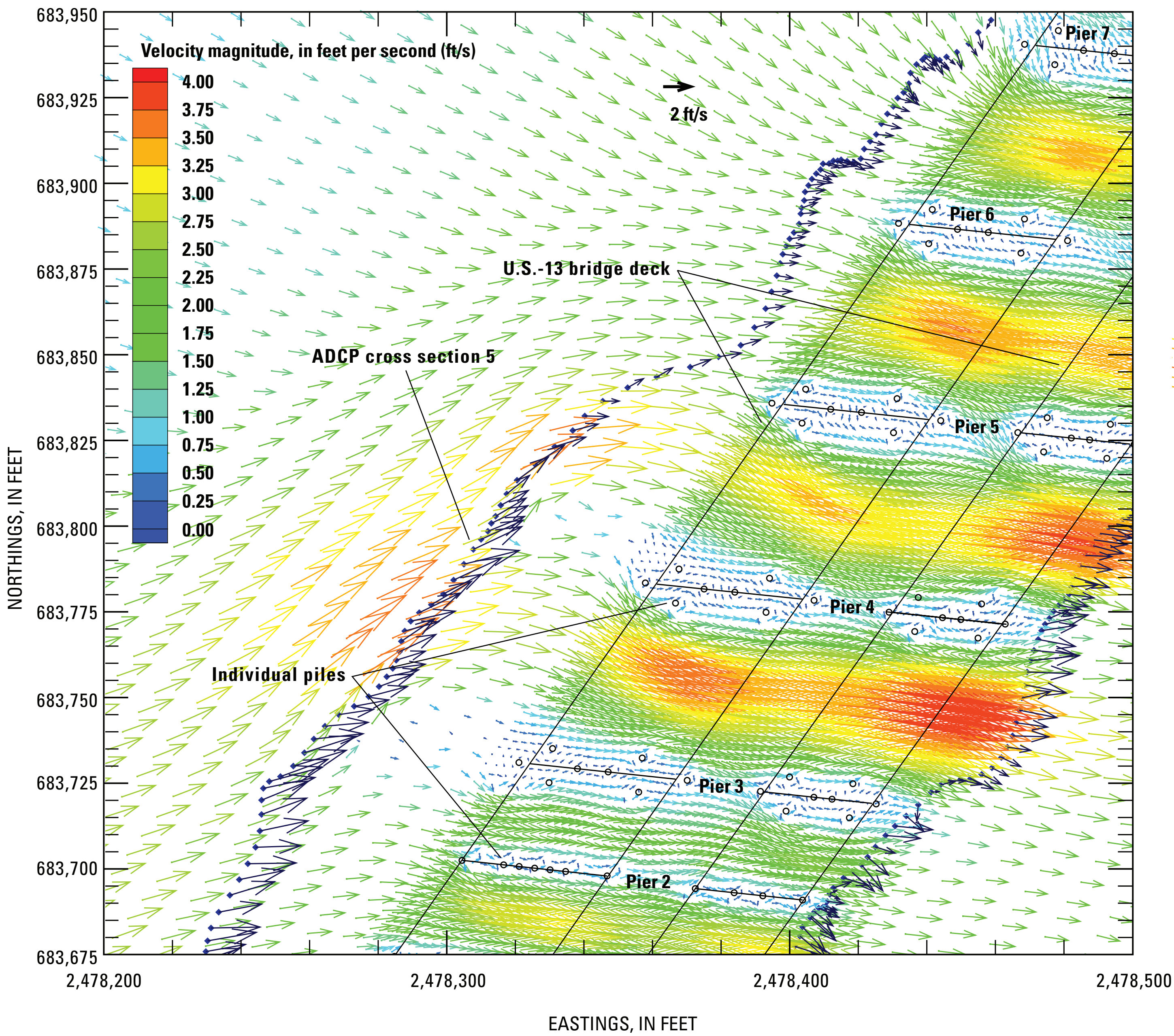
| Pier number (from right to left, looking downstream, fig. 3) | Hurricane Charley, August 2004 | | Tropical Storm Alberto, June 2006 | |
|---|---|---|---|---|
| | 2D model approach velocity | 1D model approach velocity | 2D model approach velocity | 1D model approach velocity |
| Pier 1 | 0.8 | 1.3 | 0.8 | 0.6 |
| Pier 2 | 1.5 | 1.6 | 1.2 | 2.3 |
| Pier 3 | 1.7 | 1.9 | 1.6 | 2.6 |
| Pier 4 | 2.2 | 1.9 | 2.8 | 2.6 |
| Pier 5 | 3.3 | 1.6 | 2.8 | 2.3 |
| Pier 6 | 2.6 | 1.5 | 1.6 | 2.2 |
| Pier 7 | 1.7 | 1.4 | 1.4 | 2.0 |
| Pier 8 | 0.7 | 0.7 | 1.2 | 1.1 |
| Pier 9 | 0.6 | 0.7 | 1.2 | 1.1 |
| Pier 10 | 0.2 | 0.7 | 1.0 | 1.1 |
| Pier 11 | 0.04 | 0.7 | 0.6 | 1.1 |




**EXPLANATION**

| | |
|---|---|
| Blue arrows | Acoustic Doppler current profiler (ADCP) field data |
| Multi-colored arrows | Model data contoured by velocity magnitude |
| Horizontal datum | State plane coordinates, North Carolina |

**Figure 8.** Measured and simulated velocity vectors at the upstream side of the U.S.-13 bridge over the Tar River at Greenville, North Carolina, for high-flow conditions associated with Hurricane Charley, August 2004.




**EXPLANATION**

| | |
|---|---|
| Blue arrows | Acoustic Doppler current profiler (ADCP) field data |
| Multi-colored arrows | Model data contoured by velocity magnitude |
| Horizontal datum | State plane coordinates, North Carolina |

**Figure 9.** Measured and simulated velocity vectors at the upstream side of the U.S.-13 bridge over the Tar River at Greenville, North Carolina, for high-flow conditions associated with Tropical Storm Alberto, June 2006.

**Table 8.** Pier-scour estimates for the U.S.-13 bridge over the Tar River at Greenville, North Carolina, using modeled hydraulic data for high-flow conditions associated with Hurricane Charley, August 2004.

[Values are in feet. 2D, two dimensional; 1D, one dimensional; —, not applicable (no debris)]

| Pier number (from right to left, looking downstream, fig. 3) | 2D model scour depth | 2D model highest approach velocity scour depth | 2D model debris scour depth | 1D model scour depth | 1D model highest velocity tube scour depth | 1D model debris scour depth |
|---|---|---|---|---|---|---|
| Pier 1 | 1.7 | 2.9 | — | 2.2 | 2.6 | — |
| Pier 2 | 2.6 | 3.3 | 7.0 | 2.7 | 2.9 | 7.3 |
| Pier 3 | 2.8 | 3.4 | 10.0 | 2.9 | 3.0 | 10.4 |
| Pier 4 | 7.3 | 7.9 | 20.3 | 3.0 | 3.0 | 19.2 |
| Pier 5 | 7.7 | 7.7 | — | 4.5 | 4.8 | — |
| Pier 6 | 6.5 | 6.6 | — | 5.2 | 5.7 | — |
| Pier 7 | 2.7 | 3.3 | — | 4.1 | 4.7 | — |
| Pier 8 | 4.0 | 7.1 | — | 1.7 | 2.6 | — |
| Pier 9 | 3.2 | 6.3 | — | 3.2 | 4.9 | — |
| Pier 10 | 2.0 | 6.3 | — | 3.1 | 4.9 | — |
| Pier 11 | 0.4 | 2.6 | — | 1.4 | 2.1 | — |

**Table 9.** Pier-scour estimates for the U.S.-13 bridge over the Tar River at Greenville, North Carolina, using modeled hydraulic data for high-flow conditions associated with Tropical Storm Alberto, June 2006.

[Values are in feet; 2D, two dimensional; 1D, one dimensional; —, not applicable (no debris)]

| Pier number (from right to left, looking downstream, fig. 3) | 2D model scour depth | 2D model highest approach velocity scour depth | 2D model debris scour depth | 1D model scour depth | 1D model highest velocity tube scour depth | 1D model debris scour depth |
|---|---|---|---|---|---|---|
| Pier 1 | 1.8 | 3.2 | — | 1.6 | 3.1 | — |
| Pier 2 | 2.4 | 3.5 | 9.4 | 3.2 | 3.4 | 12.6 |
| Pier 3 | 2.8 | 3.6 | 17.4 | 3.4 | 3.5 | 21.3 |
| Pier 4 | 7.9 | 7.9 | 20.7 | 3.5 | 3.5 | 20.0 |
| Pier 5 | 7.3 | 7.3 | 18.4 | 5.4 | 5.6 | 17.0 |
| Pier 6 | 2.7 | 3.5 | — | 6.1 | 6.7 | — |
| Pier 7 | 2.5 | 3.4 | — | 5.0 | 5.5 | — |
| Pier 8 | 5.2 | 7.6 | — | 2.2 | 3.1 | — |
| Pier 9 | 4.9 | 7.0 | — | 4.3 | 6.2 | — |
| Pier 10 | 4.2 | 6.5 | — | 4.3 | 6.2 | — |
| Pier 11 | 1.5 | 2.9 | — | 1.9 | 2.8 | — |

an increase in one-dimensional and two-dimensional model scour estimates that ranged from 0.1 to 1.9 ft and 0.1 to 4.3 ft, respectively. Separate pier-scour estimates also were computed for the piers with observed debris accumulation by using the interim procedure outlined in HEC-18 (Richardson and Davis, 2001, appendix D). The accumulated debris nearly tripled the original pier-scour estimates of both models. The differences in pier-scour estimates can be attributed mainly to the appreciable flow angles simulated by the two-dimensional model, as previously discussed, at piers 5–11, which were included in the two-dimensional scour computations.

Abutment scour was estimated using the hydraulic parameters derived from the one- and two-dimensional models for the two high-flow surveys (table 10). The characteristics of the right and left U.S.-13 bridge abutments are different. The Froehlich prediction equation was applied to the right abutment because the site conditions do not fit the criteria for using the HIRE equation (Richardson and Davis, 2001), which was based on field data of scour at the end of spur dikes in the Mississippi River and is applicable when the ratio of projected abutment length (L) to the flow depth at the abutment (y1) is greater than 25. In contrast, the left abutment blocks an appreciable length of flow and, therefore, is more appropriate for application of the HIRE equation. The scour estimates of the two models for the left abutment are identical for the Hurricane Charley high-flow conditions and within 1.6 ft for the Tropical Storm Alberto high-flow conditions. The one-dimensional model overpredicted scour at the right abutment relative to the two-dimensional model for both high-flow conditions, which is attributed mainly to the higher velocities simulated by the one-dimensional model on the right overbank at the approach section.

## Suggestions for Selecting Appropriate Modeling Approach

Little guidance is available to indicate whether a one- or two-dimensional model would provide a more accurate estimate of the hydraulic conditions at a bridge site. Because of the additional resources associated with two-dimensional models, the absence of guidelines to aid in determining when a two-dimensional model is likely to improve a design could lead to unnecessary modeling expenditures and time delays. One goal of this study is to build the knowledge base from which modeling guidelines can be developed. Such modeling guidelines can be valuable in directing engineers in the selection of the most appropriate modeling approach when new bridges are being designed.

During the planning stages of a new bridge design, little or no information is available regarding the hydraulics of the river until a predictive model is developed and applied. Hydrologic, topographic, and physiographic characteristics of the area around a bridge site can be useful in determining if a one- or two-dimensional model is needed to accurately estimate the hydraulic conditions. By compiling pertinent site characteristics and relating them to the results of several model-comparison studies, the framework for developing guidelines for selecting the most appropriate model for a given bridge site can be accomplished. Potentially relevant bridge-site characteristics and definitions are listed in table 11, including a summary of site characteristics for the U.S.-13 bridge over the Tar River at Greenville. As additional model comparisons are completed, additional relevant site characteristics can be identified and added to those listed in table 11. The results of this study indicate that for bridges having skews

**Table 10.** Abutment-scour estimates for the U.S.-13 bridge over the Tar River at Greenville, North Carolina, using hydraulic data from two high-flow events.

[2D, two dimensional; 1D, one dimensional; ft, feet]

| Scour equation[a] | Abutment | 2D model sour depth (ft) | 1D model sour depth (ft) |
|---|---|---|---|
| Hurricane Charley, August 2004 | | | |
| HIRE | Left | 11.7 | 11.8 |
| Froehlich | Right | 15.5 | 19.5 |
| Tropical Storm Alberto, June 2006 | | | |
| HIRE | Left | 20.3 | 18.7 |
| Froehlich | Right | 19.7 | 29.1 |

[a] Richardson and Davis, 2001.

**Table 11.** Definitions of potentially relevant bridge-site characteristics for the development of modeling guidelines at bridges, including site characteristics for the U.S.-13 bridge over the Tar River at Greenville, North Carolina.

[>, greater than; <, less than]

| Bridge-site characteristic | Definition | U.S.-13 bridge over Tar River value |
|---|---|---|
| Geometric-contraction ratio ($GC_r$) | $GC_r = 1-(b/B)$, where b = contracted-bridge opening width and B = 100-year floodplain width at the approach section. | 0.9 |
| Degree of sinuosity | Ratio of length of stream reach measured along it centerline to length measured along valley centerline [Straight (1–1.05), Sinuous (1.06–1.25), Meandering (1.26–2.0), Highly Meandering (>2.0)]. | 1.41 |
| Stream size | Width of channel measured along perpendicular line drawn between opposing banks in a straight section or inflection point in a bend. | 200 feet |
| Valley setting | Relief measured from valley bottom to top of nearest adjacent divide. | 27 feet |
| Debris potential | High, Medium, Low based on upstream land use/vegetation and visual site inspections. | High |
| Stream slope | Slope of the stream in feet per feet estimated from surveys or topographic maps. | 0.000064 |
| Radius of curvature of bend(s) | The radius of curvature of a channel bend within five channel widths upstream from the bridge. | 225 feet |
| Ratio of floodplain to channel width | Floodplain width divided by channel width. Floodplains are defined as the surface presently under construction by a stream that is flooded with a frequency of about 1 1/2 years. [Little or None (<2 x channel width), Narrow (2–10 x channel width), Wide (>10 x channel width)]. | 15 |
| Floodplain eccentricity | Ratio of the general left and right floodplain widths at the approach section ($F_1/F_2$, where $F_1$ is always the smaller of the two widths). | 0.055 |
| Braiding | The degree of braiding of the channel. A braided stream is one that consists of multiple and interlacing channels that are generally formed as bars of sediment are deposited within the main channel, causing the overall channel system to widen. [None (Not braided to <5 percent), Locally (5–35 percent), Generally (>35 percent)]. | None |
| Anabranching | A description of the degree of anabranching of the channel. An anabranched stream differs from a braided stream in that the flow is divided by islands rather than bars, and the islands are relatively large in relation to the channel width. The anabranches, or individual channels, are more widely and distinctly separated and more fixed in position than the braids of a stream. [Not Anabranched (<5 percent), Locally Anabranched (5–35 percent), Generally Anabranched (>35 percent)]. | None |
| Drainage area | The contributing drainage area of the stream at bridge location in square miles. | 2,660 square miles |
| Bridge length | The length of the bridge (abutment to abutment). | 540 feet |
| Bridge width | The width, railing to railing, of the bridge deck. | 31.25 feet |
| Parallel bridges | Are there separate parallel bridges at the site? | Yes |
| Distance between centerlines | Stream distance between the centerlines of parallel bridges. | 65 feet |
| Bridge skew | The acute angle a bridge makes with a perpendicular to flow. Skew is positive if rotated clockwise from perpendicular (the left abutment is pointing downstream). | 60 |
| Abutment/Contracted opening type | Define the type of contracted bridge opening based on classifications found in Shearman (1990).<br>Type I - Vertical embankments and vertical abutments, with or without wingwalls.<br>Type II - Sloping embankments and vertical abutments without wingwalls.<br>Type III - Sloping embankments and sloping spillthrough abutments.<br>Type IV - Sloping embankments and vertical abutments with wingwalls.<br>Other - If above definitions are not adequate. | Type III |
| Abutment setback | Maximum distance measured from abutment toe to bank of main channel. | 70 feet |
| Guidebanks | The presence of any guidebanks defined as Straight, Elliptical, None, Other. Guidebanks (also referred to as spur dikes) guide approach flows through the opening, to reduce abutment scour potential and increase bridge conveyance efficiency. | None |
| Pier type | The type of piers on the bridge. Single (single column or wall piers) or Group (form multi-column or pile bent piers). | Group of piles |
| Pier width | The actual pier/pile width. For tapered piers, enter a representative width. | 1.67 feet |
| Pier contraction ratio | Ratio = Ap/(Area of bridge opening), where Ap = Area of piers/piles projected onto a plane defined by the bridge opening. | 0.087 |
| Number of piers | List the number of piers for the bridge(s). If piers consist of multiple-piles, list total number of piles in parentheses. | 22 (146) |
| Number of relief bridges/ openings | How many relief bridges or culverts are proposed or present in the floodplains. | Two |

of 60 degrees, seven piers in the main channel, debris potential that is more than three times the pier widths, and a meander bend directly upstream, a two-dimensional model provides a more accurate representation of water-surface elevations and velocity magnitudes and directions through the bridge.

## Summary

One- and two-dimensional models of the hydraulically complex U.S.-13 bridge over the Tar River at Greenville, North Carolina, were developed, and detailed water-surface elevations, discharge, and velocity distributions were collected in the field for two high-flow events—Hurricane Charley in August 2004 and Tropical Storm Alberto in June 2006. Comparisons of the field data relative to the results of the one- and two-dimensional models provided the basis for evaluating the differences between the two models as well as the implications of these differences. Water-surface profiles, velocity magnitudes and distributions, and scour estimates from the one- and two dimensional models were compared.

The water-surface elevations for both models generally agreed downstream from the U.S.-13 bridge; however, appreciable disparity occurred between the one-dimensional and two-dimensional models upstream from the U.S.-13 bridge. The disparity upstream from the U.S.-13 bridge mainly can be attributed to differences in momentum loss at U.S.-13. The simulated water-surface slopes for the two models are similar upstream from U.S.-13; however, the two-dimensional model simulates more energy loss through the U.S.-13 main channel bridge, U.S.-13 overflow bridges, and channel bend directly upstream from U.S.-13. The flow conditions directly upstream and through U.S.-13 are two dimensional in nature, and the associated energy losses are not fully represented by a one-dimensional model, which explains the large discrepancies in simulating the water-surface elevations. Based on a comparison of measured stage at USGS streamgaging station 02083893 (located approximately 1 mile upstream from the upstream model boundary) and simulated water-surface elevations extrapolated upstream to the gage location, the two-dimensional model more accurately simulated the water-surface elevations in the study reach.

Comparisons of the modeled and measured velocity distributions through the U.S.-13 bridge opening revealed that the one-dimensional model results are less representative, relative to the two-dimensional model, of the measured velocity magnitudes and distributions. In general, the velocities from the one-dimensional model are biased low and are more uniformly distributed than the velocities from the two-dimensional model and measured data. The two-dimensional model most accurately simulates the contracted-flow magnitudes and direction through the bridge opening, which makes it a better tool for safe and efficient bridge design.

Large floating debris rafts between piers 2 and 5 were observed during both high-flow hydrographic surveys. The general location of the debris accumulation was consistent in both high-flow surveys, although the extent and quantity of debris were different. Uncertainty, therefore, is associated with modeling hypothetical scenarios and(or) conditions that have not been surveyed because of the natural variability in debris supply and configuration at various flows, which can greatly affect the simulated flow fields through a bridge. The NCDOT current (2007) designation of debris potential at bridges is “high, moderate, or low” based on field observations and interviews with bridge maintenance personnel. The results of this study indicate that the potential for debris accumulation is moderate to high at the U.S.-13 bridge, which appreciably affects the velocity magnitudes and distribution through the bridge. A three-dimensional model would provide a more representative simulation of the hydraulics through the U.S.-13 bridge opening for conditions in which accumulated debris is present.

To further compare the results of the one- and two-dimensional models, hydraulic parameters estimated by the models for the two measured high-flow conditions were used to predict scour depths at the U.S.-13 bridge using HEC-18 methods. Pier and abutment scour were computed at the U.S.-13 bridge; however, contraction scour was not computed because sediment grain-size data were not available.

Comparisons of pier-scour estimates from both models indicated that the two-dimensional scour estimates are deeper than those from the one-dimensional model. The two-dimensional pier-scour estimates differed by as much as 5.0 ft, with a mean difference of 1.8 ft for the measured conditions relative to the one-dimensional model estimates. The inconsistencies in the pier-scour estimates can be attributed mainly to the appreciable flow angles simulated by the two-dimensional model at piers 5–11, which were included in the two-dimensional scour computations.

The scour estimates of the two models for the left abutment are identical for the Hurricane Charley condition and within 1.6 ft for the Tropical Storm Alberto condition. The one-dimensional model overpredicts scour at the right abutment relative to the two-dimensional model for both high-flow conditions, which is attributed mainly to the higher velocities simulated by the one-dimensional model on the right overbank at the approach section.

Little or no information is available during the planning and design stages of a new bridge regarding the hydraulics of the river until a predictive model is developed and applied. Hydrologic, topographic, and physiographic characteristics of the area around a bridge site can be useful in determining whether a one- or two-dimensional model is needed to accurately estimate the hydraulic conditions. By compiling pertinent site characteristics and relating them to the results of several model-comparison studies, the framework for developing guidelines for selecting the most appropriate model for a given bridge site can be accomplished. Potentially relevant bridge-site characteristics for the development of modeling guidelines have been defined, including a summary of the characteristics of the U.S.-13 bridge over the Tar River at Greenville.

The results of this study indicate that for bridges having skews of 60 degrees, seven piers in the main channel, debris potential that is more than three times the pier widths, and a meander bend directly upstream, a two-dimensional model provides a more accurate representation of water-surface elevations and velocity magnitudes and directions through the bridge. By combining information from several similar projects and incorporating comparisons of existing models from other DOTs in the Southeast, NCDOT engineers and staff can be provided with the tools and data that are essential in improving bridge design and mitigating structural hazards at a range of hydraulically complex sites.

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# Appendix

Difference maps of simulated velocity magnitudes for the existing and pre-roadway model scenarios for U.S.-13 over the Tar River at Greenville, North Carolina.

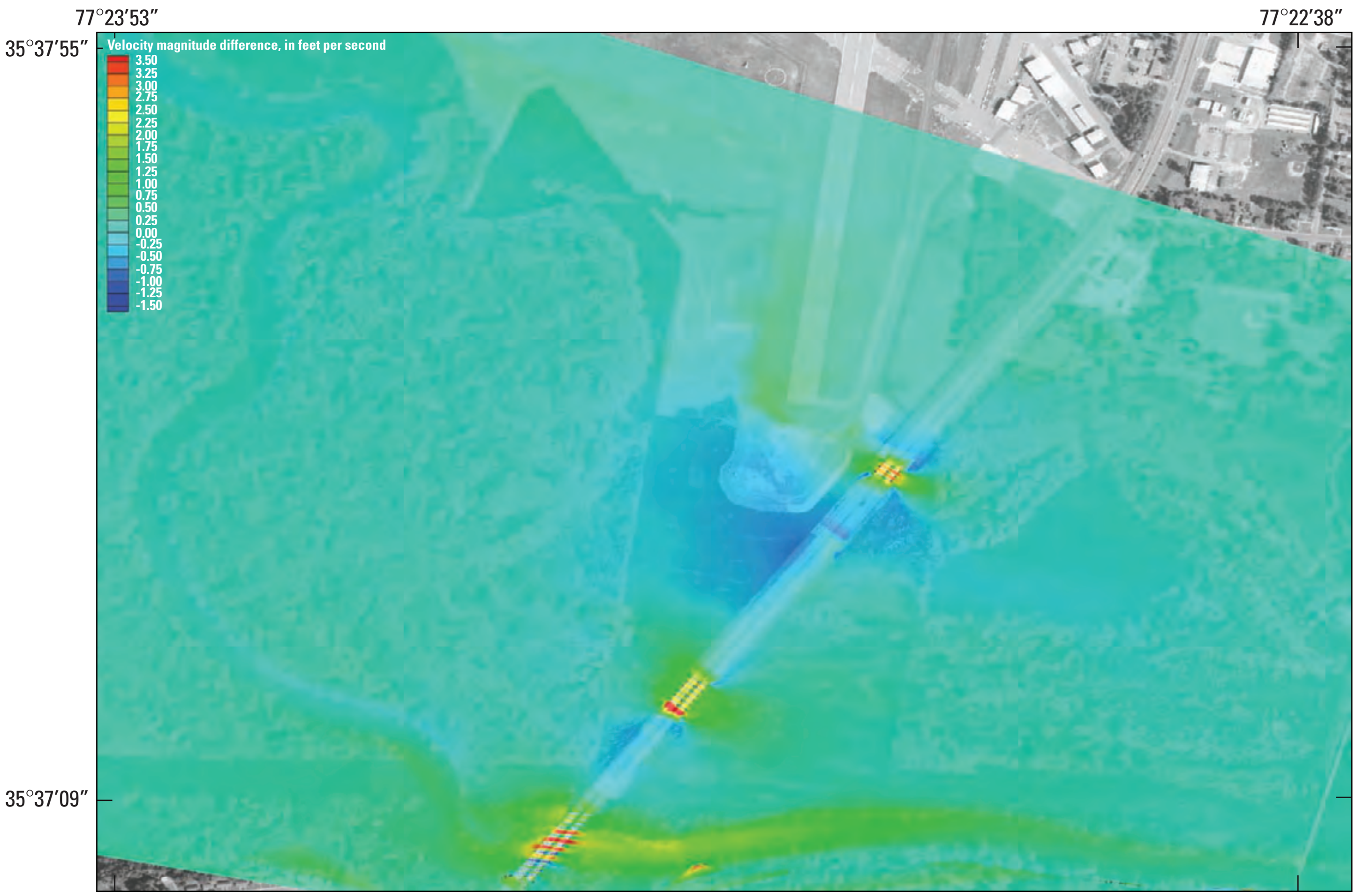


**Figure A1.** Difference map of the simulated velocity magnitudes for the existing bridge and pre-roadway model scenarios for the estimated 10-year return-period flow at U.S. 13 over the Tar River at Greenville, North Carolina. (Pre-roadway model results are subtracted from the existing model results.)

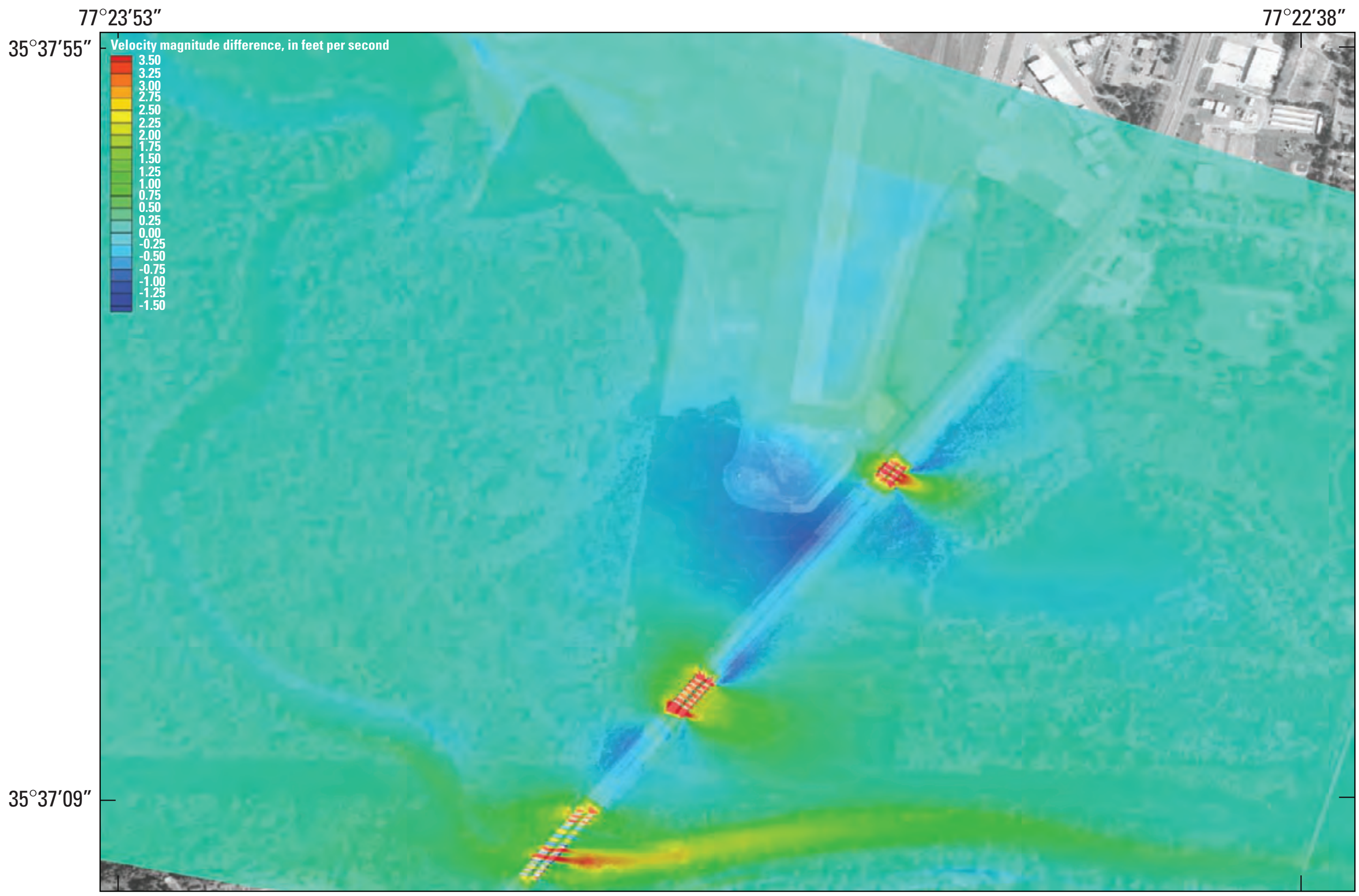


**Figure A2.** Difference map of the simulated velocity magnitudes for the existing bridge and pre-roadway model scenarios for the estimated 50-year return-period flow at U.S. 13 over the Tar River at Greenville, North Carolina. (Pre-roadway model results are subtracted from the existing model results.)

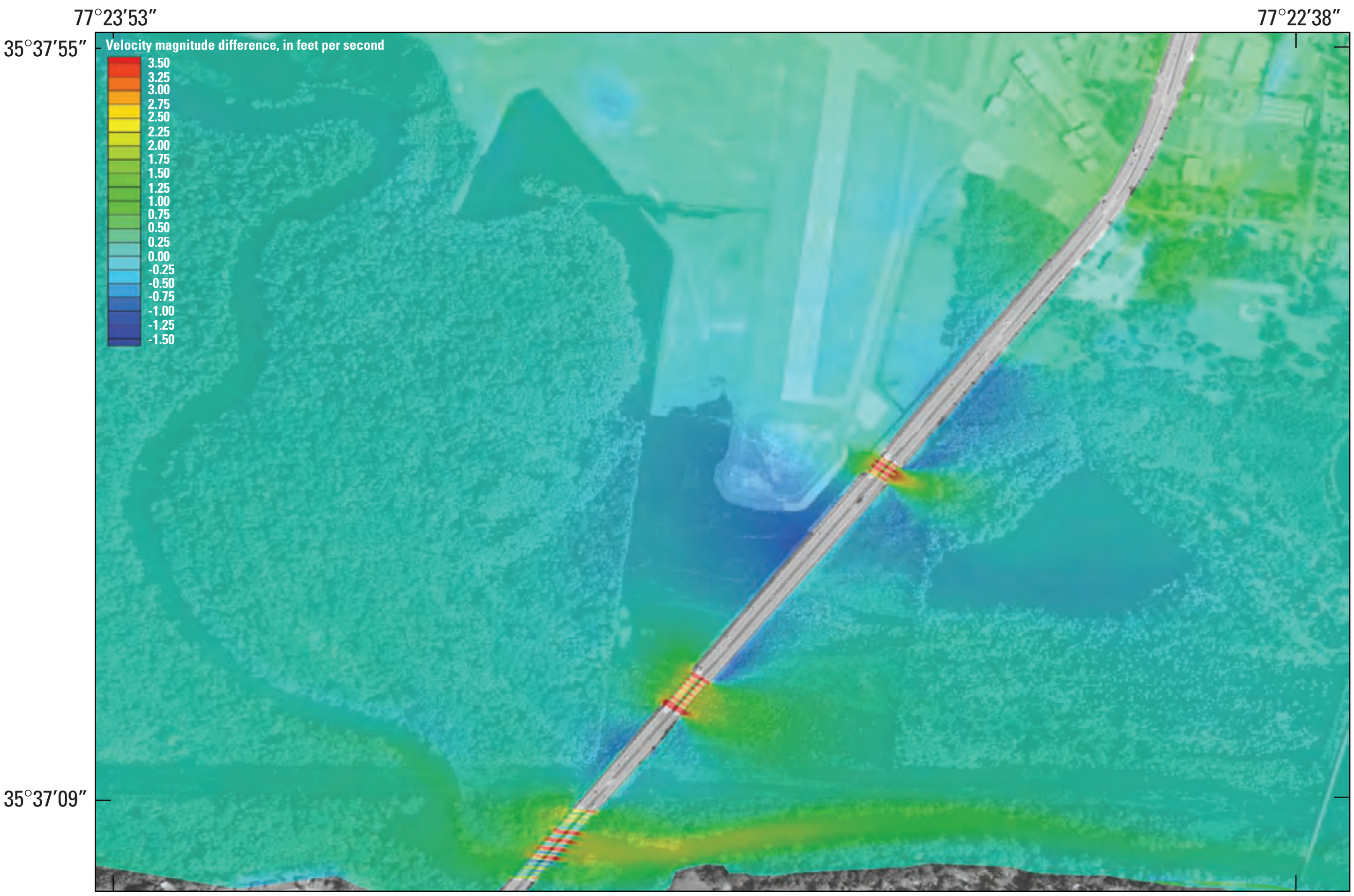


**Figure A3.** Difference map of the simulated velocity magnitudes for the existing bridge and pre-roadway model scenarios for the estimated 100-year return-period flow at U.S. 13 over the Tar River at Greenville, North Carolina. (Pre-roadway model results are subtracted from the existing model results.)

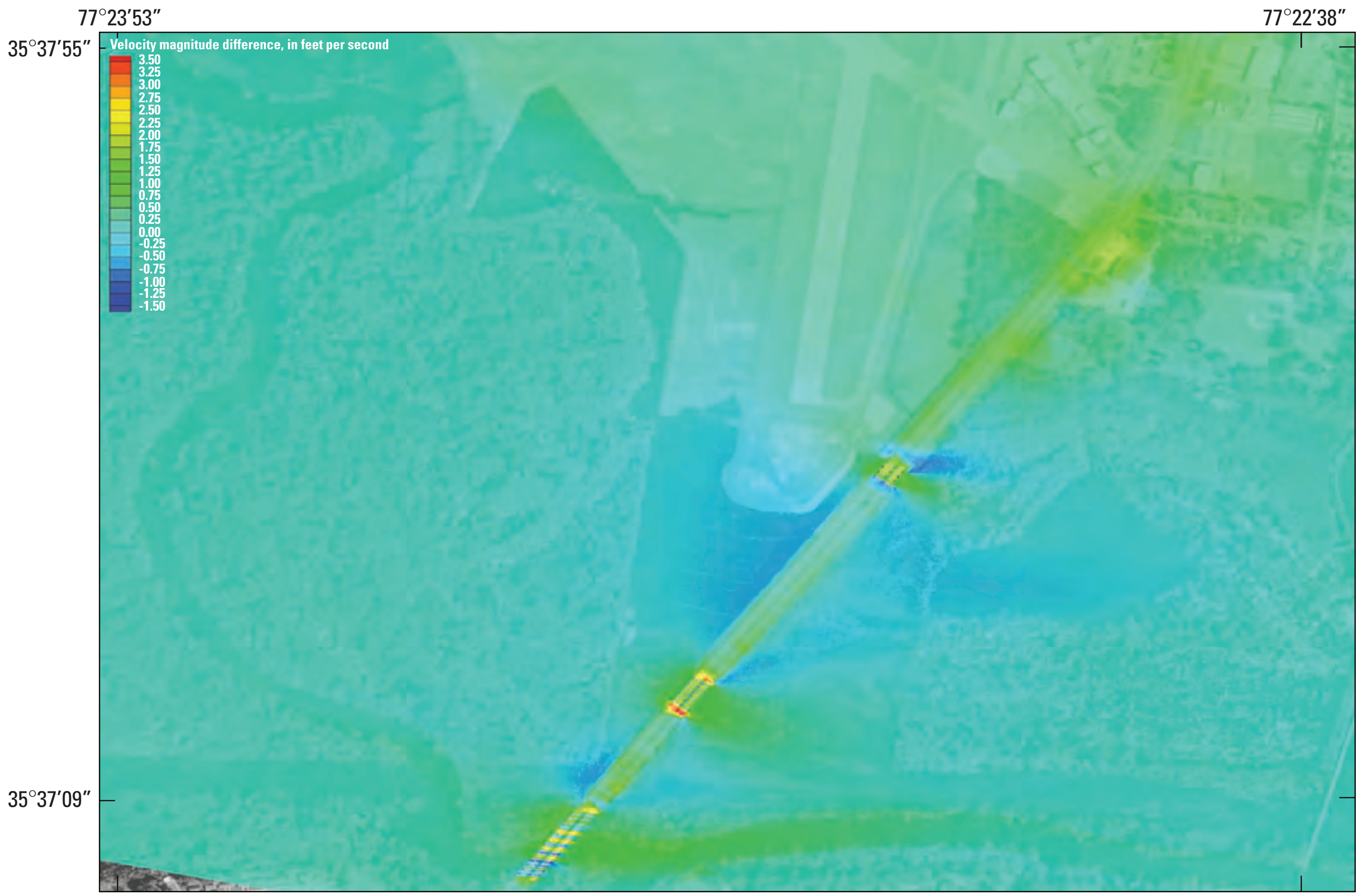


**Figure A4.** Difference map of the simulated velocity magnitudes for the existing bridge and pre-roadway model scenarios for the estimated 500-year return-period flow at U.S. 13 over the Tar River at Greenville, North Carolina. (Pre-roadway model results are subtracted from the existing model results.)

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