COMPARISON OF THE THEORY, APPLICATION, AND RESULTS OF ONE- AND

TWO- DIMENSIONAL FLOW MODELS

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COMPARISON OF THE THEORY, APPLICATION, AND RESULTS OF ONE- AND TWO- DIMENSIONAL FLOW MODELS

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COMPARISON OF THE THEORY, APPLICATION, AND RESULTS OF ONE- AND

TWO- DIMENSIONAL FLOW MODELS

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Kathryn Green Lee, daughter of James R. Green and Janice (Barrentine) Green, was born on March 14, 1981, in Brewton, Alabama. She graduated from Flomaton High School in 1999. She then graduated suma cum laude from Jefferson Davis Community College with an Associates of Science degree, in May 2001. She entered Auburn University in September 2001 and graduated suma cum laude with a Bachelor of Science degree in Civil Engineering in August, 2004. She also completed the cooperative education program, working under Hydrologists at the U.S. Geological Survey, where she is currently employed. She married Benjamen Paul Lee, son of Timothy Paul Lee and Elizabeth (Stewart) Lee, on August 28, 2004. She then entered Graduate School, Auburn University, in September 2004.

THESIS ABSTRACT

COMPARISON OF THE THEORY, APPLICATION AND RESULTS OF ONE- AND TWO- DIMENSIONAL FLOW MODELS

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The accurate simulation of flooding is crucial in the design of safe, cost-effective hydraulic structures. Hydraulic engineers are faced with several difficult decisions that will determine the accuracy of any modeling scenario. The foremost decision is the selection of the hydraulic model. Once the model has been selected the input parameters must be chosen, the model executed, and the results interpreted.

This study is a comparison of two flow models to determine their applicability to specific river reaches. The two models selected for comparison were Hydraulic Engineering Center's River Analysis System and Finite Element Surface Water Modeling System. The respective one- and two-dimensional models were used in conjunction with calibration data for flood flow simulation.

Two river reaches with varying basin characteristics were modeled. The roughness values required to simulate the high-water profiles were less for the two-dimensional model than for the one-dimensional model. Comparison of the two reaches indicated that the roughness values for one- and two- dimensional flow are not considerably different for basins with very flat slopes. It was also determined, for two-dimensional flow, a reasonable range of values; the base kinematic eddy viscosity has little effect on the resulting water-surface profile.

The high- water profiles predicted by the one- and two-dimensional models were examined and the hydraulic properties within the reach were investigated. The results showed that assumptions of one-dimensional flow are valid for a standard reach and a skewed roadway crossing up to approximately thirty degrees. The flow distribution and correlating velocities were compared to measured values, for the second river reach. It was found that the flow distribution for both models matched the measured value within three percent. The velocity profiles created within the two-dimensional model were found to overall provide a closer match to the calibration data.

ACKNOWLEDGEMENTS

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Style manual used: Guide to Preparation and Submission of Theses and Dissertations 2005.

Computer Software used: Surface-water Modeling System (SMS) 8.0 by Boss International, Inc and Brigham Young University; HEC-RAS River Analysis System by U.S. Army Corps of Engineers Hydrologic Engineering Center; Microsoft Word 2003, Excel 2003, WordPad; Adobe Illustrator; Arc Info by ESRI; TDS Survey Link; Terramodel by Trimble Navigation Limited; Terrain Navigator Pro by Maptech.

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I. INTRODUCTION

For many years engineers and scientist have made idealized assumptions to aid in the analysis of complex problems. An example of this idealization is hydraulic modeling of river reaches and its application to the design and evaluation of hydraulic structures. Many situations can be simulated with one-dimensional flow models. In more complex situations two- and even three- dimensional models are needed to reasonably depict the hydraulics of a reach. As modeling technology has advanced, hydraulic engineers are faced with the problem of model selection. It has been observed, in practice, that a two-dimensional model is not merited unless there are unusual circumstances. A few of these circumstances include severe skew of the road crossing, multiple structures, and large rivers. Two-dimensional models are also useful when lateral variations in water surface and flow distribution are significant.

This study, through the modeling of specific river reaches, is intended to provide insight on the applicability of the one-dimensional model. It also serves to provide comparisons of the necessary input, limitations, expected results, and one- and two-dimensional modeling approaches.

The construction, execution, and results of one- and two-dimensional models were investigated to provide insight into their differences and applicable scenarios. It is known that every reach has its own characteristics that affect flood flow simulation. This uniqueness is something that a modeler must learn to adapt to. Two reaches were chosen to provide optimal points of investigation. Each reach has valuable calibration data, from field measurement and observation, which will be used to compare the results of the one- and two- dimensional models. In order to properly compare one- and two- dimensional flow analyses, there has to be a standard for comparison. Unless there is a stage-discharge relationship (gaging station) or documented flood there are few opportunities for this. In the case of these study sites both were available.

The process of calibration and validation is an important step in surface-water modeling where so many factors are unknown. The practice of calibration is the adjustment of input parameters, within reason, until the results compare closely with a measured value. Once a model has been calibrated it can be validated with a separate flooding event. If the results of model simulation compare to the second flooding scenario without major adjustments to input variables, the model is considered valid over the range of flood flows.

The results of the two reaches chosen for this study were compared based on the calibration data. The primary reach investigated was Fivemile Creek. In order to investigate topics not explored in the Fivemile Creek study a second reach was examined. The second study site was on the Sucarnoochee River. It provided the opportunity to examine a roadway with multiple bridges that crossed the floodplain at an angle (skew) and the effects this has on the computed velocity profiles, and flow distribution.

The fundamental issue with the comparison of one- and two-dimensional models is the adjustment of input parameters. These adjustments, over a reasonable range of values, based on site topography, land cover variability, limited flood flow and stage information can make significant changes in model predictions. Table 1 outlines some of the input parameters that must be determined for both models. One of the major differences in the one- and two-dimensional models is the representation of energy losses and how they are calculated based on the input parameters. The results of this study is intended to provide insight on the variability of the input parameters and interpretation of results within surface-water modeling and guidance for determining when a more complex model may be needed.

One-Dimensional Input Parameters	**Two-Dimensional Input Parameters**
Manning's Roughness Coefficient	Manning's Roughness Coefficient
--	Base Kinematic Eddy Viscosity
Location of Control (Friction Slope)	Location of Control (Tailwater Elevation)
Ground Slope	Grid Adjustments
Peak Flow Assumptions	Location of Inflow (Sources, Nodestrings)

Table 1 Variable parameters within one- and two- dimensional flow models.

II. LITERATURE REVIEW

Currently quantitative comparison of one- and two- dimensional flow models is limited. The majority of surface-water modelers base their judgment on experience and their own analyses. Engineers are often faced with the dilemma of improved accuracy versus time constraints. The cost-to-benefits ratio often dictates the type of model used in every day practice.

The few published articles that provide insight into these comparisons are inconclusive or very site specific. “Development of a Methodology for incorporating FESWMS-2DH Results” (Parr and Zou 2000), provided a detailed comparison of various models. The University of Kansas investigators presented the methodology and results for a multiple bridge crossing near Neodesha, Kansas. The crossing consists of 3 bridges, one main and two relief structures. The site was previously investigated to determine the resulting backwater using Water Surface Profiler (WSPRO) [a one-dimensional step-backwater model used for computing water-surface profiles (Sherman, 1976)]. These computations resulted in 66% of the flow in the main channel bridge and 16% and 18% in the relief structures, for the 50-year flood. Using the same tailwater and roughness coefficients documented for the WSPRO computations a two-dimensional model was constructed and the computational program FESWMS-2DH was used. The results indicated the main channel bridge carried 69% of the flow and the relief structures carried

8% and 23 %. The conclusions were that the use of the two-dimensional model, in the design analysis, suggested no need for one or both of the relief structures. This raises several questions. Was the difference in the calculated flow distribution great enough to merit the use of a two-dimensional model? The mere 16% in the relief structure, indicated by the WSPRO analysis, was a good indicator that the relief structure only carries a small portion of the total flow. This specific investigation does show merit in the use of a two-dimensional model for multiple bridge crossings; however the only results presented are the flow distributions. Typically when bridges are designed, based on the hydraulics of the crossing, the primary indicators of a good bridge design, for a state highway crossing, are the 50-year mean velocity and the 100-year backwater. The numerical criteria of these values vary from state to state and are based on geographic, geological, and physiological characteristics of the area. This information was not presented in the report. Another point worth mentioning is the use of one-dimensional roughness coefficients in a two-dimension model. This assumption is presumed to present some error in the computations of the water-surface profile.

When comparing the results of two models such as these, field calibration data is a necessity. The models could be constructed and executed, and the one-dimensional model said not to be valid when the deviance from the two-dimensional model is significant. The actual value may be somewhere between the estimated values. Without the calibration data one would assume the one-dimensional model failed when in actuality it may have provided results that are just as close to the real value as the two-dimensional model. Without calibration data it is hard to draw a conclusion on the improved accuracy of the two-dimensional model.

III. STATEMENT OF RESEARCH OBJECTIVES

Due to the lack of documented results on the comparison of one- and two-dimensional flow models, this study is intended to compare the results of a typical site with no unusual characteristics and a site that would be considered a candidate for two-dimensional analysis. The models were constructed and executed based on field data and site inspection. The variation of the input data and the resulting hydraulic data were documented. The models were calibrated through the use of a documented flood event and gaging station data. Based on this information, conclusions can be drawn on the assumption that the cost to benefits ratio of a one-dimensional model surpasses that of a two-dimensional model. The effects of skewed crossings on flow distribution were also investigated. The results also provide a base line to use in the transition of picking roughness values for a two-dimensional model.

IV. DESCRIPTION OF THE PRIMARY STUDY REACH

The area selected for primary observation is the city of Tarrant, Alabama. Tarrant is located in north central Alabama, approximately 1.5 miles northeast of Birmingham, in Jefferson County, Alabama. A location map of the area can be viewed in Figure 1. The population of Tarrant is approximately 8,000. The city's area consists of 6.4 square miles with 0.6% being surface water. The average elevation to mean sea level is 546 feet. Tarrant is a primarily industrial town and is located in the Fivemile Creek basin. Currently the city planning is based on Federal Emergency Management Agency's (FEMA) flood insurance study. Anthropogenic changes in the Fivemile Creek basin have significantly altered the hydrologic and hydraulic conditions of the basin. On March 10, 2000 and May 7, 2003 the flood stage as recorded by the U.S. Geological Survey's stream gage (02457000) exceeded FEMA's 100-year flood stage significantly (2.9 feet and 4.7 feet, respectively). Both of these floods caused a considerable amount of damage in the Tarrant City community. The revision of the one-dimensional study with FEMA mandated Hydraulic Engineering Center's River Analysis System (HEC-RAS) would suffice the needs of Tarrant, however since this basin has recently experienced two major floods the availability of data made this site a prime candidate for the comparison of a one- and two- dimension model.

Area of Study

The reach of the stream evaluated extends from about 300 feet upstream of Lawson Road to just below the L&N Railroad (about 5,000 feet upstream, of U.S. Highway 31). The total reach length of the study is approximately 20,000 feet. The average slope of the basin, in the study reach, is 18.5 ft/mi. The stream flows in a southwesterly direction and has an average top width of 85 feet with a minimum and maximum value of 50 and 130 feet, respectively. The average flood plain width was 1,000 feet and varied between 200 and 2,000 feet. The land cover of the reach is characterized by grassy fields and some wooded areas with moderate vegetative growth. The reach extends through some areas of residential and industrial land use. These areas typically have minimal or maintained vegetative growth and areas of ineffective flow. The ineffective flow areas, for this study reach, consisted primarily of warehouses, homes, and large equipment lots.

Flood History

This reach was selected because of the availability of data. It has experienced two major floods in 2001 and 2003, and over a period of time has had two stream flow gages. Located at the upper end of the reach is the Fivemile Creek at Lawson Road gage (USGS gage 02456980). This gage was active until April 2001. Based on high-water marks a peak was calculated indirectly for the 2003 flood. Shortly after the 2003 flood the gage was reactivated. The other gage is located at the Ketona Lakes.

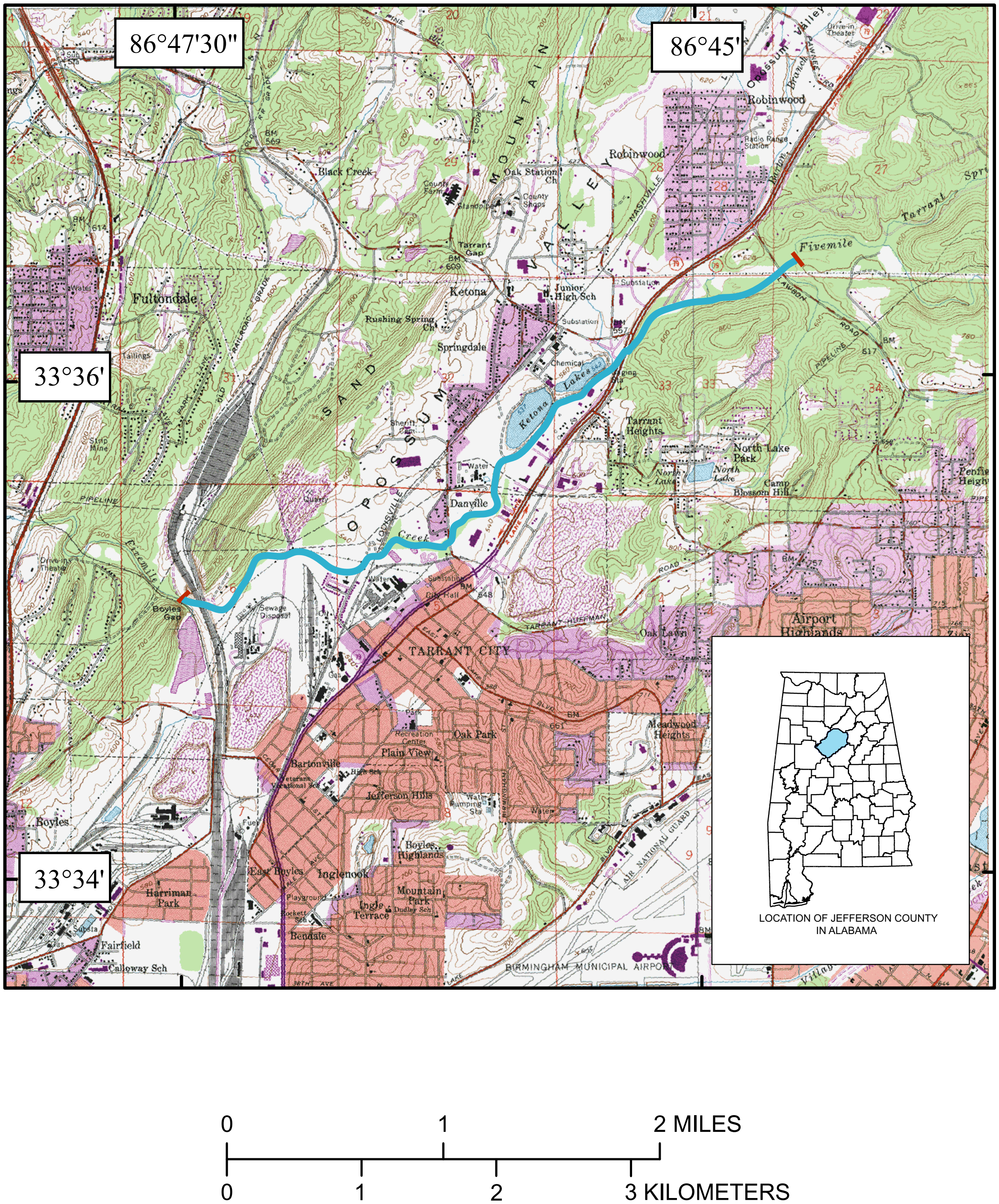

Figure 1. Fivemile Creek study reach, Tarrant, Jefferson County, Alabama.

V. DATA USED

The objective of this project will be accomplished through these steps: field data collection surveys, land-use (impervious cover) determinations, hydrologic determinations, and hydraulic modeling.

Survey Data

In order to accurately represent the geometry of the reach the elevation was defined using an electronic total station. This was accomplished by surveying eleven flood plain cross sections and the geometry of all significant drainage structures (with adjacent roadways). The reach consisted of six hydraulic structures, one culvert and five bridges. In efforts to calibrate the hydraulic model, eleven high-water marks, from the 2003 flood, were also surveyed. The river stations were computed working upstream with the furthermost downstream section being zero. Each section was also given an alphabetical identifier. The furthermost downstream section was labeled (A). The alphabetical identifiers and the river stations can be viewed in Table 2. Land cover (roughness) characteristics for the reach were assessed from field investigations of the site. Manning's roughness coefficient was selected to reflect current conditions and the conditions that existed during the 2003 flood. Photographs of the cross sections and the surrounding areas can be viewed in the Appendix in Figures 29 through 41.

Alphabetical Section Identifier	River Station (ft)
A	0
B	1,077
C	3,280
D	5,206
E	6,863
F	8,692
G	11,357
H	13,168
I	14,986
J	16,798
K	18,068

Table 2. Cross sections locations for the Fivemile Creek study reach.

Drainage Area

There were three areas that were determined to have significant changes in drainage area. These locations can be seen in Figure 2 and Table 3. The drainage areas for gaged locations are published by USGS and other areas can be delineated using a contour map or previously delineated drainage area maps. The drainage area was delineated (Figure 3) for the downstream most cross section. This area is known as Boyle's Gap.

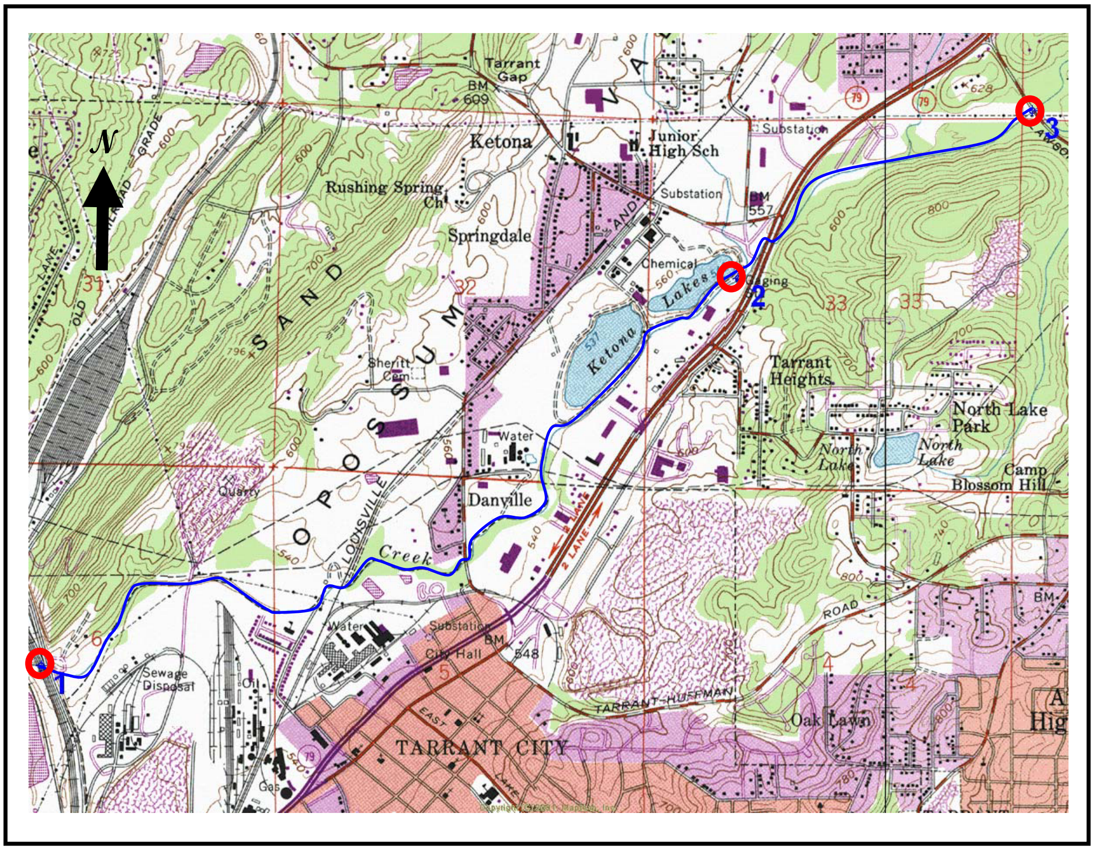

Figure 2. Location of significant changes in drainage area in the Fivemile Creek study reach.

Identifier	State Plane Coordinates NAD 1927	Location	Drainage Area (sq. miles)
1	1304781 N, 716295 E	Boyles Gap	28.4
2	1310471 N, 726288 E	Ketona Gage	23.9
3	1310535 N, 726340 E	Lawson Road Gage	18.6

Table 3. Geographic location of significant changes in drainage area in the Fivemile Creek study reach.

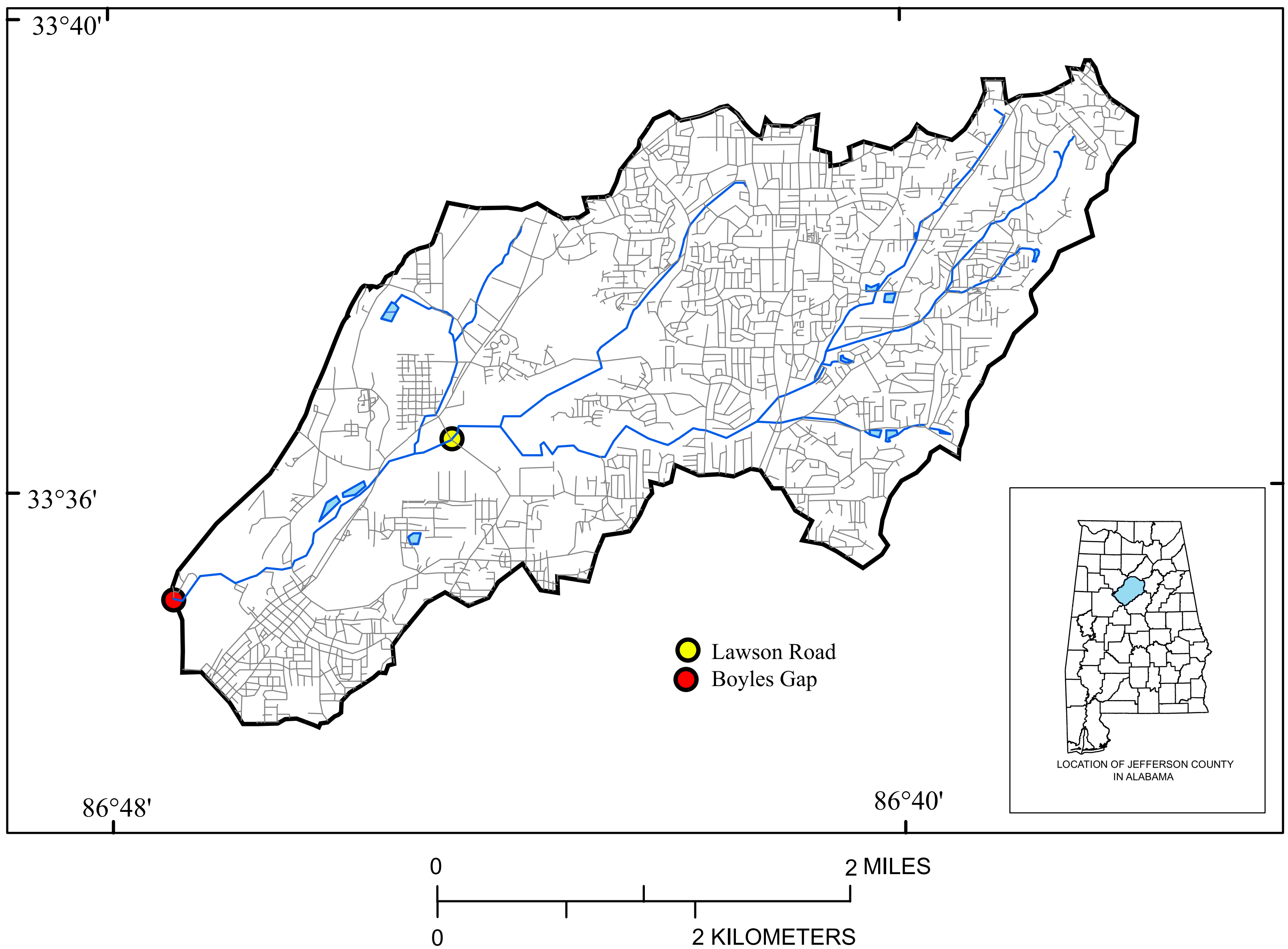

Figure 3. Fivemile Creek basin and associated development.

Land-use Determination

Due to industrial and residential growth in the Fivemile Creek basin the hydrology has drastically changed. As described in later chapters, factors used to determine the peak flow are drainage area and impervious area. In order to determine the peak flows, that reflect current conditions, the impervious area was computed. Land-use and impervious cover for the basin were calculated using the most recent aerial photographs available for the reach. These photographs were supplemented with surveys and field reconnaissance in the newer sections of development. Impervious area is described by Stamper as, "The impervious cover of a basin is that part of the total area that is covered by either buildings or pavement that are impenetrable by infiltration from rainfall. The percentage of impervious cover is an indication of the degree of development or urbanization of a basin". There are several methods of determining the percentage of impervious cover.

Aerial photography from 2004 was made available for the entire drainage basin. The areas of development, determined from aerial photography and field reconnaissance, were tabulated. The resulting areas were weighted based on the type of land use and an additional five percent was added to allow for future growth. The weighting factors and their descriptions can be seen in the Appendix in Table 14. The resulting value of percent impervious used in the hydrologic model was 25%. Upon inspection of Figure 3 it can be observed that the basin has little room for future growth.

As mentioned earlier, the most recent floods greatly exceeded the 100-year flood profile developed by FEMA. This is due to the growth experienced in the basin. In order to understand the magnitude of the modifications the basin has experienced the percent impervious was calculated for 1992. This date was chosen due to the availability of data. The National Map publishes the National Land Cover Data Set in Geographic Information System (GIS) form. The land cover data is categorized in codes that correspond to a land use type. The land use types were digitized using aerial photography and field reconnaissance. The National Land Cover Data Set was clipped based on the drainage area and the resulting shape was transformed into a binary map (Figure 4) to isolate the land cover categories of interest.

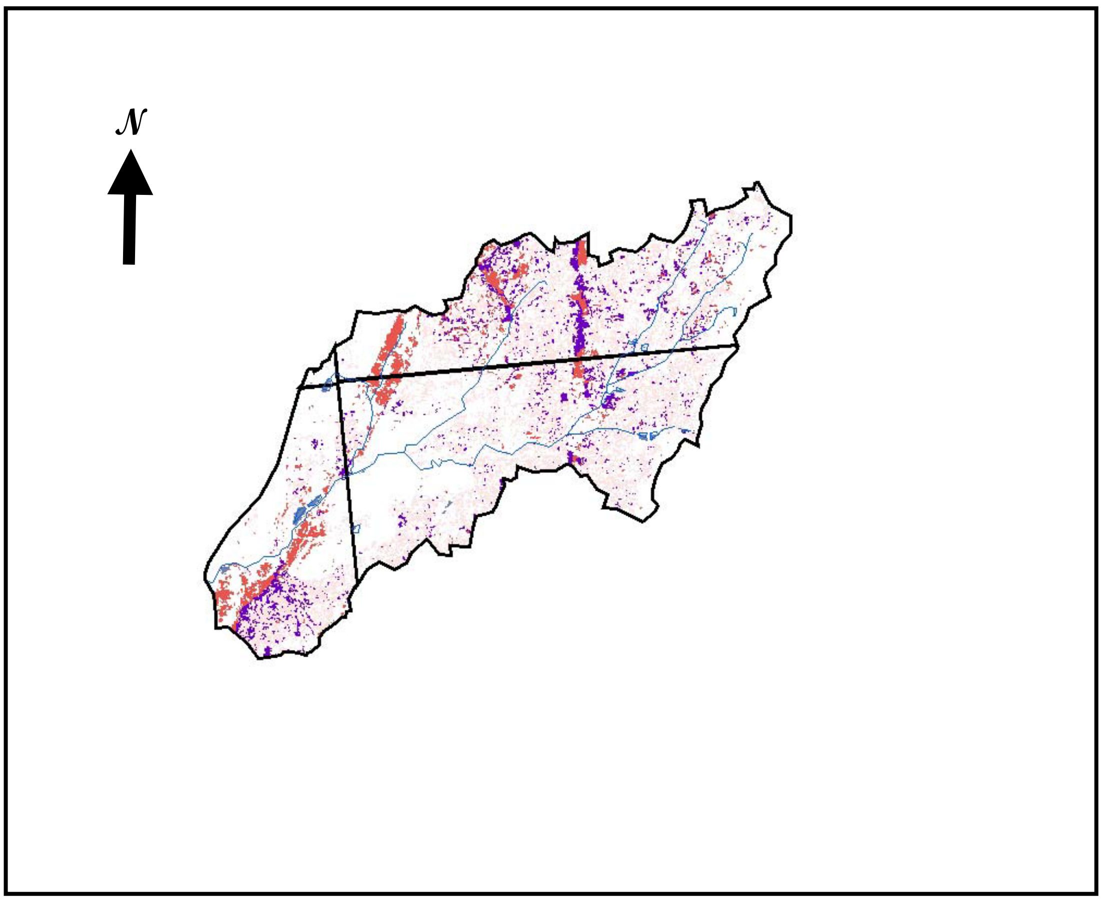

Figure 4. Binary map of urbanization for the Fivemile Creek basin.

The area of the remaining grid cells was tabulated, and the results were used to calculate the percent urbanization. The resulting percent impervious calculated was 12%. The results of the calculations, without a growth factor, show that in 1992 the basin would have been rated as 12% urban whereas current conditions show the basin being 20% urban. In a matter of 12 years the percent urbanization has almost doubled.

Hydrologic Determinations

Hydrologic conditions were analyzed using the USGS urban regression equations and procedures outlined in "Magnitude and Frequency of Floods in Alabama" by J.B. Atkins. The equations were developed in a previous study, Olin and Bingham (1982), and recounted in Atkins 1996. The methodology is recapitulated by Atkins as, "The urban equations were derived by multiple regression analyses of peak flows obtained from synthetic flow generated with calibrated rainfall-runoff model and basin characteristics for 23 urban stations in Alabama". The equations were developed for the 2-, 5-, 10-, 25-, 50-, and100-year recurrence intervals and can be seen in Table 4. Recurrence interval is defined as the "average interval of time between exceedance of the indicated flood magnitude" (Stamper 1975). This can be explained as a there being a 1 in 100 chance that the 100-year recurrence interval stage will occur in any given year. As with most equations, these are not without limitations. The equations in Table 4 should only be applied to basins with a drainage area of 0.16 to 83.5 square miles with greater than five percent urbanization.

In May 07, 2003, the city of Tarrant experienced a major flood that incited further inspection of the flood profiles computed and published by FEMA. In order to define the upper end of the stage-discharge relation, for the flood gage at Lawson Road, an indirect measurement was computed by the USGS for the 2003 flood, using the contracted opening method. The results of the indirect measurement can be seen in the Appendix in Figure 44. As seen in Table 5, the computed total flow is 14,100 ft^3/s. In order to transfer the computed peak to other areas of interest, the methods outlined in Atkins 1996 were applied. The peak at a gaged site can be transferred to an ungaged site on the same creek.

Recurrence Interval (years)	**Regression Equation for Streams in Urban Areas**	**Standard Error Of Estimate (percent)**
2	$Q(u) = 150A^{0.70}IA^{0.36}$	26
5	$Q(u) = 210A^{0.70}IA^{0.39}$	24
10	$Q(u) = 266A^{0.69}IA^{0.39}$	24
25	$Q(u) = 337A^{0.69}IA^{0.39}$	24
50	$Q(u) = 396A^{0.69}IA^{0.38}$	24
100	$Q(u) = 444A^{0.69}IA^{0.39}$	25

Table 4. Flood Frequency Relations for Urban Streams in Alabama (Olin and Bingham, 1982). [Q (u), flood flow in cubic feet per second; A, drainage area in square miles; IA, impervious area in percent]

Location	Drainage Area (sq. miles)	May 07, 2003 Flood (ft^3/s)
Boyles Gap	28.4	18,800
Ketona Gage	23.9	16,700
Lawson Road Gage	18.6	14,100

Table 5. Peak flow at various locations for the May 07, 2003 flood on Fivemile Creek.

Using the drainage areas, impervious area, and the equations (Table 4) the flood flows (Table 6) representative of the current land use conditions were computed for selected recurrence intervals. This was done to estimate the recurrence interval of the 2003 flood. Based on the estimated current hydrologic conditions the computed peaks for the May 07, 2003 flood rank between a 200- and 500-year flood event.

Location	Drainage Area (sq. miles) 25% Impervious	10-year (ft^3/s)	50-year (ft^3/s)	100-year (ft^3/s)	500-year (ft^3/s)
Boyles Gap	28.4	9,390	13,500	15,700	20,700
At Ketona Gage	23.9	8,340	12,000	13,900	18,300
At Lawson Road Gage	18.6	7,020	10,100	11,700	15,400

Table 6. Peak flow at various locations reflective of the current conditions within the Fivemile Creek study reach..

VI. ONE-DIMENSIONAL ANALYSIS

History

In order to understand the applicability of the models chosen for this reach, it is important to look at the historical development of one- and two- dimensional models. Initially any hydraulic computations made were computed by hand. After World War II hand calculations continued as a routine process for hydraulic engineers. The first automated process was released in the early 1960s. In 1966, Hydrologic Engineering Center (HEC), a division of the Institute of Water Resources (IWR), U.S. Army Corps of Engineers released a FORTRAN based program titled “Backwater, Any Cross Section”. This program was later revised and released in 1968 with the title of HEC-2(Haestad Methods).

The program, written by D.G. Anderson and W.L. Anderson, served as the basis for most processes of one-dimensional modeling (Shearman 1976). Through use and observations the model was improved. With refinements, and additions E431 “Step-Backwater and Floodway Analysis” was created. This model was developed by the USGS in 1976 and written by James O. Shearman. The major improvements over the previous model included, complete analysis of flow through bridges, ability to compute flow over weirs, and the special feature of analyzing the effects of encroachments (hydraulic structures) on existing conditions (Shearman 1976).

Along the same time HEC-2 (1968) was revised. HEC-2 was written by Bill S. Eichert. HEC-2 is the second in a series of developments made my HEC. The first was HEC-1, flood hydrograph program. The HEC-2 version dated 1976 featured improvements such as encroachment analysis, summarized output tables, and five alternatives for calculating friction losses.

Although the two models, E431 and HEC-2 were superior at the time, as technology advanced and needed improvements were noticed, the models were updated. Different versions were released as minor changes were made. Major advancements led to the creation of two new models.

The policy of Federal Highway Administration (FHWA) is to consider the effects of encroachment alternatives. Through inspection of existing models FHWA found that while each model had positive attributes likewise there were limitations. FHWA employed USGS to create a model that was more suitable to the design needs. In response USGS created "A Computer Model for Water Surface Profile Computations" known as WSPRO, also designated as HY-7 in the FHWA hydraulics computer program series. The enhancements included but are not limited to, improved bridge analysis and input format, the addition of the analysis of combine flow (bridge and weir), multiple bridge analysis, and selective output. The primary development was the applicability of bridge design using the risk analysis supposition. However, the model was still applicable for non-design circumstances (Shearman 1976).

In response to the USGS enhancements, HEC made efforts to adjust accordingly with their "Next Generation" (NexGen) of hydrologic software. A part of the NexGen projects was the development of "River Analysis System" (HEC-RAS). HEC-RAS was released in July of 1995. Subsequently HEC has released eight versions. The latest version is 3.1 and was released in September of 2001. HEC-RAS was developed by Gary W. Brunner. The major computer science advancement associated with this model is graphical user interface (gui). This interface allows the user to have visual representation of the input and output data which facilitates error recognition and allows the user to visually evaluate the applicability of the results. It has also been documented that HEC-RAS surpasses HEC-2 in hydraulic analysis capability which is demonstrated in the subsequent section on the theory of one-dimensional modeling (Brunner 2002).

<u>Theoretical Basis</u>

The theory investigated herein is that of the model applied for this study, HEC-RAS (Bruner 2002). Profiles are computed from one cross section to the next by application of the Energy Equation (Equation 1). The process of application is the standard step method.

$$Y_2 + Z_2 + \alpha_2 \left(\frac{V_2^{\ 2}}{2g} \right) = Y_1 + Z_1 + \alpha_1 \left(\frac{V_1^{\ 2}}{2g} \right) + h_e \qquad \text{Equation 1}$$

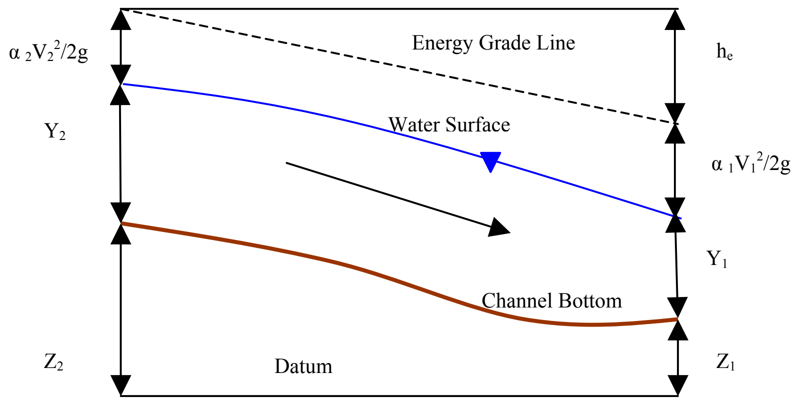

Figure 5. Schematic used in the application of the Energy Equation.

The term (h_e) in the Energy Equation is defined as the head loss from one section to the next. There are two primary losses considered, friction loss and expansion or contraction losses. The value of h_e can be determined from Equation 2.

$$h_e = (L)(S_f) + C\left[\left(\alpha_2\left(\frac{V_2^{\,2}}{2g}\right) - \alpha_1\left(\frac{V_1^{\,2}}{2g}\right)\right)\right] \qquad \text{Equation 2}$$

The discharge weighted reach length can be calculated from the following equation:

$$L = \left(\frac{L_L Q_L + L_c Q_c + L_R Q_R}{QL + Q_c + Q_R}\right) \qquad \text{Equation 3}$$

It is shown in Figure 6 where the user enters the corresponding flow lengths for the right and left overbanks and the main channel.

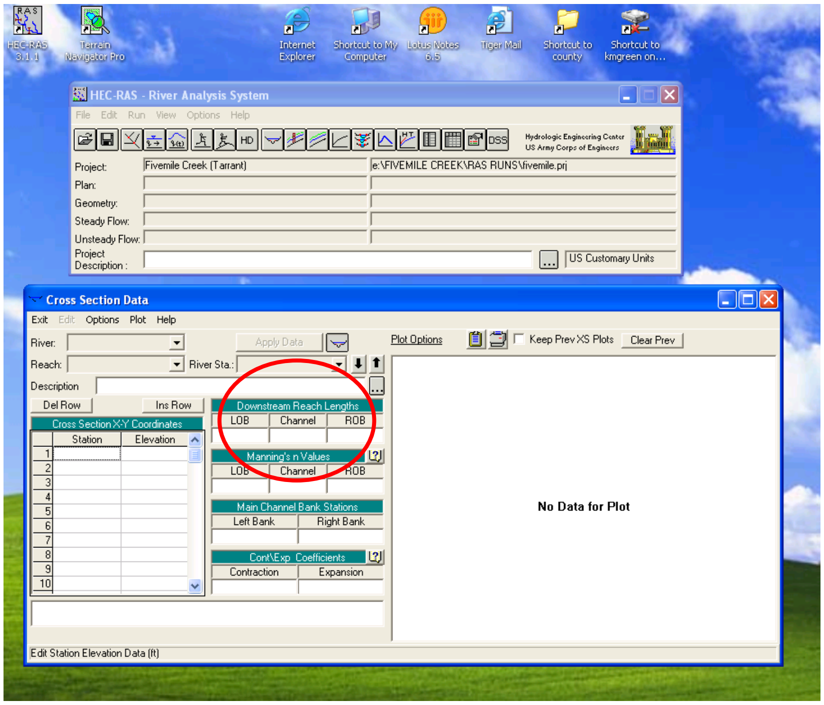

Figure 6. HEC-RAS input deck.

Cross Section Subdivision for Conveyance Calculations

Conveyance, a measure of the carrying capacity of a channel section, is an important property when computing water-surface profiles. Conveyance can be used to determine the flow distribution between the overbanks and the channel. This distribution is an important determination in the calculations of local scour. In order for HEC-RAS to determine the total conveyance and the velocity coefficient for a cross section the flow has to be divided into velocity tubes. HEC-RAS computes a conveyance for each subsection with a different Manning's roughness coefficient. The subsection values are summed to give a total conveyance. The conveyance (K) is computed for each subdivision break point using Equation 4.

If $Q = (K)(S)^{\frac{1}{2}}$ and $Q = \frac{1.486}{n}(A)\left(R^{\frac{2}{3}}\right)\left(S^{\frac{1}{2}}\right)$ Then

$$K = \frac{1.486}{n}(A)\left(R^{2/3}\right) = 1.486\left(A^{5/3}\right)\left(\frac{1}{P}\right)^{2/3} \qquad \text{Equation 4}$$

This is the default method used by HEC-RAS, but sometime there is a need to compute the conveyance using other methods. In the event that one is trying to duplicate a study using HEC-2 a different method is employed. HEC-2 was designed to compute a total conveyance at every geometric break point and then sum them to get the respective left, right and channel conveyance. These methodologies were further investigated to see the difference in the resulting numbers. Shown below is how the two methods differ mathematically.

$$K = \frac{1.486}{n}(A)\left(R^{2/3}\right)$$

$$R = \frac{A}{P}$$

$$K = \frac{1.486}{n}(A)\left(\frac{A}{P}\right)^{2/3}$$

The difference in the wetted perimeter will outrank the difference in the computation of the areas. Therefore, the greatest conveyance produced will be the sum of conveyance at each individual ground point (HEC-2 Method). Higher conveyance corresponds to a lower water-surface elevation. The results of examining the equations are further supported by actual computations. The conveyance methodology used by

HEC-RAS, for a water-surface elevation of 91.12 feet, produced a total conveyance of 296,152 ft^3/s. Using the HEC-2 method for the same water-surface elevation produced a total conveyance of 308,498 ft^3/s. There is a 4% error in the difference of these solutions.

Composite Manning's Roughness Coefficient for the Main Channel

HEC-RAS is programmed to check the applicability of a subdivided channel. If the side slope is steeper than 0.02 then a composite (n) value will be computed. If the input (Figure 7) is specified as having individual parts each with their own roughness they will be combined to a composite roughness using the following formula.

$$n_c = \left[\frac{\sum_{i=1}^{N} P_i (n_i)^{1.5}}{P} \right]^{2/3} \qquad \text{Equation 5}$$

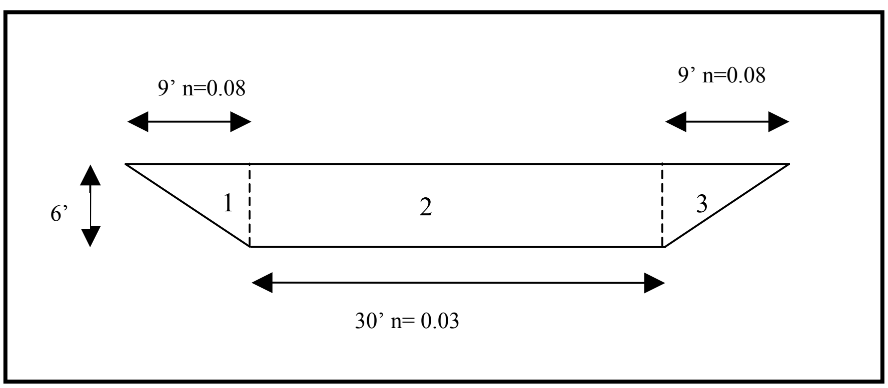

Figure 7. Typical channel sketch.

$$P_1 = P_3 = \left(6^2 + 9^2\right)^{1/2} = 10.8$$

$$P_2 = 30$$

$$n_c = \left[\frac{(10.8)(0.08)^{1.5} + (30)(0.03)^{1.5} + (10.8)(0.08)^{1.5}}{Q_1 + Q_2 + Q_3} \right]^{2/3} = 0.05383$$

Evaluation of the Mean Kinetic Energy Head

The mean kinetic energy head is based on the discharge weighted average of the velocity head of the subdivisions. The subdivisions are defined as the left and right overbank and the main channel. The mean kinetic energy head can be computed from the following equation:

$$\alpha\left(\frac{V^2{}_{avg}}{2g}\right) = \left(\frac{Q_1\left(\frac{V_1{}^2}{2g}\right) + Q_2\left(\frac{V_2{}^2}{2g}\right) + Q_3\left(\frac{V_3{}^2}{2g}\right)}{Q_1 + Q_2 + Q_3} \right) \qquad \text{Equation 6}$$

The velocity correction coefficient can be isolated and the following equation acquired:

$$\alpha = \left(\frac{Q_1V_1{}^2 + Q_2V_2{}^2 + Q_3V_3{}^2}{(Q_{total})(V^2{}_{avg})} \right) \qquad \text{Equation 7}$$

The equation can also be written in terms of conveyance

$$\alpha = \frac{\left(A^2{}_t\right)\left(\frac{K^2{}_1}{A^2{}_1} + \frac{K^2{}_2}{A^2{}_2} + \frac{K^2{}_3}{A^2{}_3}\right)}{K^3{}_t} \qquad \text{Equation 8}$$

Friction losses are computed from the product of the length and the friction slope. The friction slope can be obtained from Manning's equation.

$$S_f = \left(\frac{Q}{K}\right)^2$$ Equation 9

HEC-RAS uses a representative average friction slope for the reach. The standard default method is the average conveyance method, but this can be changed based on user input.

$$S_{avg,f} = \left(\frac{Q_1 + Q_2 + Q_3}{K_1 + K_2 + K_3}\right)^2$$ Equation 10

This method is used to compute the friction slope between cross-sections. This is not the beginning friction slope that is used to converge on the normal depth for subcritical flow.

Contraction and Expansion Loss Evaluation

The program determines if the channel is contracting or expanding based on the velocity. If the velocity head increases downstream it is assumed that contraction is occurring. If the velocity head is decreasing downstream then it is assumed that expansion is occurring. Based on these assumptions the loss due to contraction or expansion is computed with the following equation.

$$h_{ce} = C\left|\alpha_1\left(\frac{V_1^{\,2}}{2g}\right) - \alpha_2\left(\frac{V_2^{\,2}}{2g}\right)\right|$$ Equation 11

Computation Procedure

The initial consideration when performing water surface profile analysis by hand or with a computer aided model is the location of the control. The majority of the streams in Alabama have flat slopes and are assumed to be flowing at subcritical depths. If this is the case, the control is a downstream cross section. This section is often assumed to be far away enough from any structures, so that the flow is at normal depth. It is possible that the crossing of interest is affected by backwater from a downstream structure or headwater from another stream. In some reaches the stream may be flowing at supercritical depth and the control is located upstream.

Once the control has been determined, the model goes through a series of steps to compute the water-surface elevation at the next cross section using the standard step method. A water-surface elevation is assumed at the section upstream of the control (subcritical flow) and the total conveyance, velocity head, and the energy head losses are computed based on this assumption. All of these values are then used in the Energy Equation (Equation 1). If both sides of the Energy Equation agree the water-surface elevation assumed is correct. If the values are not the same the model continues to solve this iterative process. The program will proceed through 20 iterations. If the solutions have not converged the minimum error water surface elevation or the critical water surface elevation will be assumed. This usually is an indicator that there is insufficient cross section data. Either there are not enough sections, they are too far apart or coded wrong. There is also the possibility that this could be the result of improper flow regime. The model could be trying to calculate a subcritical depth for a section that is actually supercritical. This can be checked by looking at the Froude number. The Froude number is calculated for the main channel and the entire section. The break point for the differentiation between subcritical and supercritical flow, for HEC-RAS, is a Froude number of 0.94. This is chosen instead of 1.0 due to the inaccuracy in the calculation of the Froude number for irregular channels.

Critical Depth Determination

There model calculates critical depth to ensure the boundary condition entered by the user is correct. Critical depth can be defined as the point of minimum specific energy possible at a cross section. The specific energy diagram (Figure 8) below shows this point graphically, for a specified flow rate, Q.

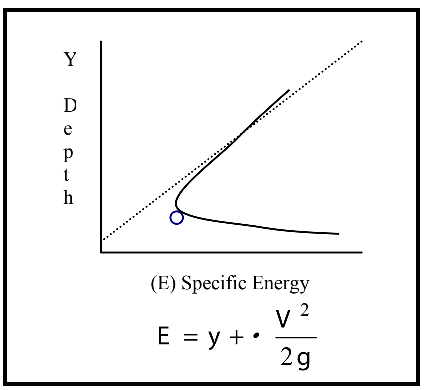

Figure 8. Specific energy diagram.

The specific energy is $E = y + \frac{V^2}{2g}$. The critical depth is determined by assuming depths and calculating the corresponding specific energy until a minimum value is found. In some cross sections there may be multiple points of minimum specific energy. This is often the result of a break in the total energy curve. This can occur when the cross section has very wide flat overbanks, levees or ineffective flow areas. If this is the case, the user should inspect the result of the critical depth. An example of a local minimum due to the overtopping of a levee is shown in Figure 9.

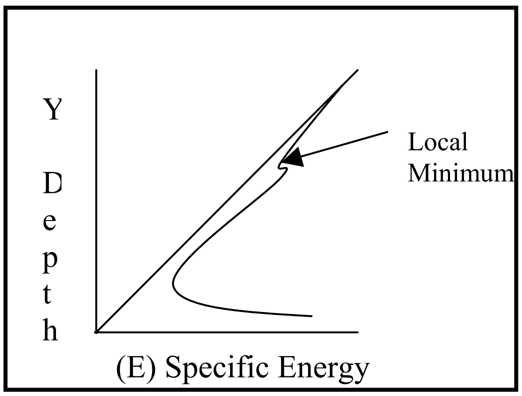

Figure 9. Specific energy curve with a local minimum critical depth.

Application of the Momentum Equation

In previous sections the process for computing water-surface elevations for gradually varied flow has been outlined. These procedures are not applicable to rapidly varied flow. Anytime the profile passes through critical depth the energy equation is not valid. There are several situations where the flow could pass from supercritical to subcritical or subcritical to supercritical. These instances include but are not limited to; significant changes in streambed slope, stream junctions, weirs, and bridges. In cases of hydraulic jumps, low flow at bridges and stream junctions HEC-RAS uses the momentum equation.

$$\left(\frac{Q_2B_2}{gA_2}\right)+A_2Y_{ave,2}+\left(A_1+A_2\right)\left(\frac{(L)S_o}{2}\right)-\left(A_1+A_2\right)\left(\frac{(L)S_{ave,f}}{2}\right)=\left(\frac{Q_1B_1}{gA_1}\right)+A_1Y_{ave,1}$$

Equation 12

This equation assumes that the discharge is different at each section. If the reach being analyzed is very small the external force of friction and the force due to the weight of the water is small and can be neglected.

Steady Flow Program Limitations

In order to determine if a one-dimensional or two-dimensional model is needed it is useful to look at the limitations of the one dimensional model. The fundamental limitation is that the flow is considered to be one-dimensional. This means that the velocity components in directions other than the flow are negligible. HEC-RAS assumes that the total energy head is the same for all points in the section. The second limitation is that the flow is considered steady state (not time dependant). HEC-RAS will handle unsteady flow, but a different set of equations are used and a hydrograph is needed. Otherwise the peak flow is assumed to be the flow of highest velocity and is used to represent the worst case scenario. The third limitation is that the flow must be gradually varied flow except in the location of bridges, weirs, and culverts. In cases such as these the momentum equation or empirical equations are used. The forth and last assumption is that the channels have slopes less than 0.1. Under this condition the pressure variation at a cross section is approximately hydrostatic. The pressure head (p) is measured by the vertical distance and the water depth (y) is measured by the perpendicular depth, as shown in Figure 10. When the slope exceeds 0.1 the values of (p) and (y) vary significantly.

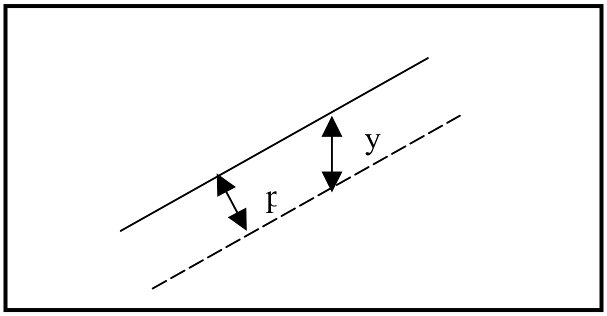

Figure 10. Illustration of pressure head and depth.

One-dimensional models provide competent, efficient, and useful solutions to many problems. However, there are certain circumstances when the need for a two-dimensional model is suggested. A few of these circumstances are:

Skewed Crossing

Large Rivers with Incised Channels

Multiple Bridges on a wide Floodplain

Superelevated Flow

Variables

A_1 ,A_2, A_3 = flow area in subdivisions 1, 2, and 3

A_t = total flow area of cross section

A= flow area for subdivision

α_1, α_2 = velocity correction coefficients

B = momentum correction coefficient

C= contraction or expansion coefficient

E = specific energy

g = acceleration due to gravity

h_{ce} = energy head loss due to contraction or expansion

h_e = energy head loss from one section to the next

K_1, K_3, K_3, = flow conveyance for subdivisions 1, 2, and 3

K_t = total conveyance of cross section

K = conveyance for subdivision

L= discharge weighted reach length

$L_{L,}$ L_C, L_R = reach lengths for flow in the left overbank, main channel and right overbank

n = Manning's roughness coefficient

n_c = composite roughness coefficient

n_i = coefficient of roughness of subdivision (i)

N = number of parts the main channel is divided into

P = wetted perimeter of entire main channel

P_i = wetted perimeter of subdivision (i)

Q_L , Q_C, Q_R = Arithmetic average of flow between sections for the left overbank, main channel, and right overbank

Q_1, Q_2, Q_3 = flow in the subdivisions 1, 2, and 3

Q_{total} = total flow of cross section

R = hydraulic radius for subdivisions (area/wetted perimeter)

S_f = representative friction slope between two cross sections

$S_{avg,\,f}$ = average conveyance friction slope

S_o = slope of the channel, based on mean bed elevations

V_1, V_2, V_3 = average velocities in subdivisions 1, 2, and 3

$\left(\frac{V^2}{2g}\right)$ = velocity head

$\alpha\left(\frac{V_{avg}{}^2}{2g}\right)$ = mean kinetic energy head

Y_1, Y_2 = depth of water at cross sections

Y_{ave} = depth from the water-surface to the centroid of the area

Z_1, Z_2 = elevation of the main channel inverts

Applications

The process of creating a HEC-RAS model and the adjustments made to emulate a known flood were documented for comparison with the two-dimensional model.

The initial step in HEC-RAS is the creation of a river reach. This can be done on top of a topographic map or by freehand. However, it is important that the reach be drawn from upstream to downstream. Once this is done the cross sections can be entered. When a cross section is created the user is prompted for the river station. A list of the section identifiers and river stations can be seen in the Table 2. The corresponding geographic locations of these cross sections can be seen in the Appendix in Figures 42 and 43. The other data input areas are the downstream reach lengths, Manning's roughness coefficient, and the subdivision break points. The downstream reach length is the distance to the section immediately downstream. In the case of the downstream most section, the river station and the downstream reach length is zero. The Manning's roughness coefficient was selected through field reconnaissance and aerial photography. This is one of the most debatable areas in surface water modeling. It is also one of the major points of comparison for this study reach. In order to provide a proper representation of the numerical value of Manning's roughness coefficient outside sources were consulted.

Manning's roughness coefficient can be determined several ways. Typically, inexperienced hydraulic engineers use tables, equations, and roughness verification studies to choose roughness values. Once the engineer is well versed in the area of Manning's roughness coefficients they are more likely to rely on their experience and personal judgment. One of the best ways to develop the art of roughness coefficient selection is through indirect calculations of known floods, calibrating output data to observed water-surface profiles, and training under an experienced engineer. This gives the engineer a sense of the streams typical to their area. Most engineers would agree that the selection of over bank roughness is easier to determine. The area that is more subjective is the channel roughness. This is why it is a good idea to do a sensitivity analysis in areas where there isn't any calibration data. This allows the engineer to determine how significant the channel roughness is in the profile calculations. Typically, if the channel is small and insignificant in capacity the channel roughness will not play a noteworthy role in the profile calculations. It is also a good idea to inspect the 2-year flood stage. Typically the stage of a 2-year flood event should be one to two feet deep in the floodplain.

The channel roughness value is difficult to determine because it is dependant on so many factors. These factors and how they affect roughness are outlined by Chow (Chow 1959). An understanding of these factors should lead to a better estimate of channel roughness. However, it should be made clear that calculated roughness coefficients are not applicable for all stages and discharges. This should be taken in to consideration and will be discussed in latter sections.

The majority of the factors influencing channel roughness are interdependent. One of the first factors to consider is surface roughness. This primarily focuses on the shape and grain size distribution of the material covering the streambed and side slopes. The material within the wetted perimeter acts as a flow retardant. Typically the larger grains retard the flow more and result in a larger roughness coefficient. The converse is true for smaller grained particles.

The other aspect of flow impedance is vegetation. Vegetation may project from the top of banks into the channel or grow along the side slopes. Not only does this impede the flow but it also reduces available flow area. The density and amount of foliage depends on seasonal conditions. During summer months it should be expected that the vegetation would be greater than winter months. Another important aspect of the influence of vegetation is the depth of flow. When the depth is shallow the flow will be hindered by vegetation more than at deep depths. It should also be considered whether the flow could push the vegetation over or up root it. This should be well thought-out and all aspects of the site considered. Streams with steep slopes have greater velocity and are more likely to make vegetation lay down. This would result in lower roughness coefficients. Typically it can be assumed that the roughness is greater for shallower depths. A study at the University of Illinois determined that trees 6-8 inches in diameter, with pruned branches, located on the side slopes of the channel, effect flow less than small bushy growths (Chow 1959). When in doubt of the vegetative conditions of a site, a good rule of thumb is to select the roughness coefficients based on conditions reflective of a typical spring flood. Conservative roughness coefficient selection is not always the upper range of values. When good velocity estimates are needed it would be conservative to choose values on the lower range of possible roughness coefficients.

The next area of concern is channel irregularity. This refers to sand bars, ridges, bends, and depressions. Anything that causes drastic changes in channel bed or changes in cross section size and shape warrants an increase in the roughness coefficient.

The channel alignment should be examined. If the reach has smooth curves with a large radius a lower roughness coefficient would be selected in comparison with sharp curves with severe meanders. Streams that are sinuous warrant an increase in roughness coefficient. Chow states that natural streams, all things being equal, a meandering stream may increase the roughness coefficients as much as 30% over a non-meandering stream. In connection with vegetation and obstructions to flow, sinuous streams should also be investigated for attack on embankments.

Another contributing factor is silting and scouring. Both of these processes can have an effect on the area and wetted perimeter of the channel. Scour can make a uniform channel irregular. Sandy or gravel beds will erode more uniformly where as clay will not. Typically there is not a significant increase in roughness for scour as long as it progresses uniformly. There is also a roughness change due to the material that is suspended during these processes. The suspended material consumes energy and causes head loss resulting in an increase in the apparent roughness values. Most of the time, the effects of suspended material is negligible. Silting may transform an irregular channel and decrease the roughness, depending on the material deposited. The addition of sand bars and sand waves will also increase the roughness.

Another technique used to determine roughness estimates is the consultation of a roughness verification study. Publications such as these are good reference tools, but extreme care should be exercised in selection of the publication. Manning's roughness coefficient is dependant upon geomorphology as well as physiographic location. The principles that are used for roughness coefficient selection in western boulder-strewn streams with steep slopes are not the same as southern flat slopes with heavily vegetated streams. One publication that makes an effort to cover a wide variety of geographic locations is, "Roughness Characteristic of Natural Channels" (1987), by Harry H. Barnes, Jr. This study examines the hydraulic conditions of 50 different stable streams. In order to be considered for the study the stream must had to have a bank full flood, peak flow measured by current meter methods or a well-defined stage-discharge relationship, good high-water marks, and a uniform reach near a stream gage. Bed material for these sites was determined through sampling methods. Once the data was processed Manning's equation and the Energy equation was applied. In these computations the value of α, velocity head coefficient, was assumed to be 1.0. The resulting roughness coefficients were published with pictures of the reach and cross sectional plots. Of the 50 sites, 6 were located in Georgia. The roughness coefficients for these sites ranged from 0.04 to 0.075. Although Georgia streams are similar to Alabama streams, this would be no substitute for methods determined by experience.

The third method of roughness coefficient selection is to consult a table of typical values for given conditions. This is a good place for a beginner to start but should not be relied upon solely. Due to the varying geographic and physiographic factors as mentioned above. It should also be noted that the table is based on sites that are primarily less than 100 feet in top width. When dealing with wider channels a lower roughness coefficient should be used.

The next step in creating a HEC-RAS project is to enter the hydraulic structures. The Fivemile Creek study reach has six hydraulic structures in the study reach. The bounding cross sections, for each structure, were chosen and the river station of the bridge was entered.

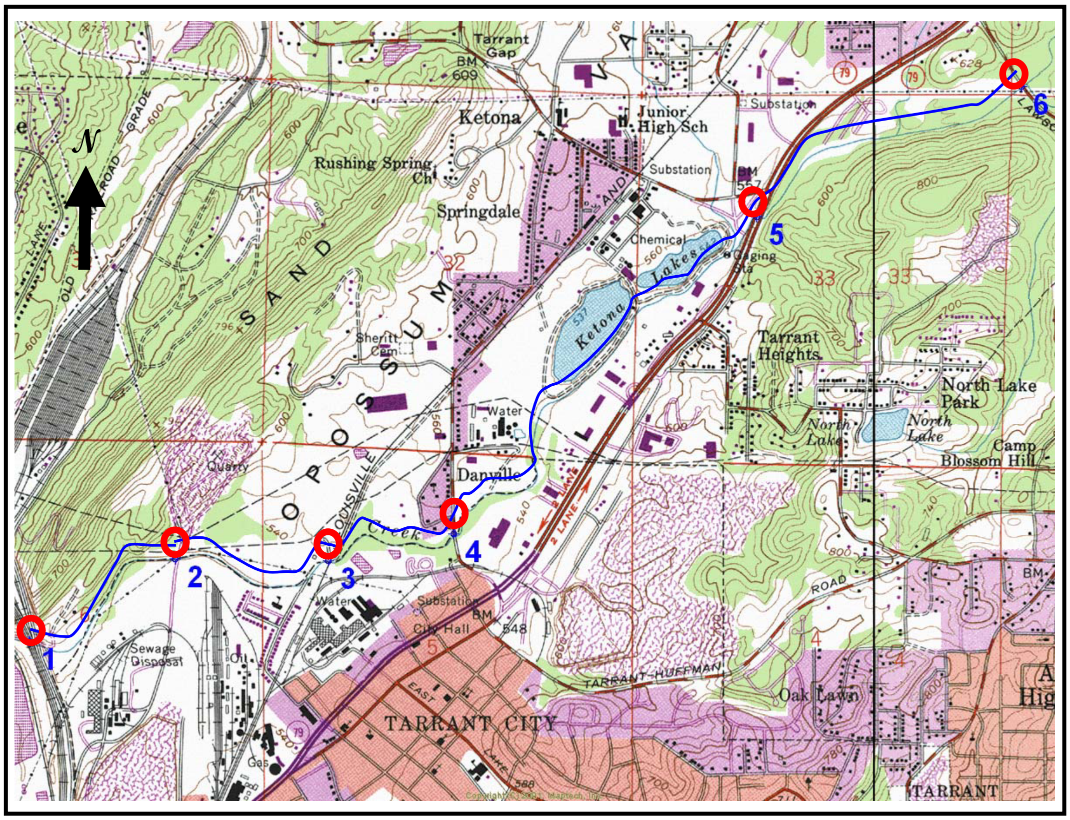

Figure 11. Location of hydraulic structures within the Fivemile Creek study reach.

Identifier	State Plane Coordinates NAD 1927	Structure
1	1304781 N, 716295 E	Culvert
2	1305990 N, 718376 E	Power Company Bridge
3	1305981 N, 720579 E	Rail Road Bridge
4	1306383 N, 722378 E	Springdale Road Bridge
5	1311015 N, 726712 E	State Highway 79
6	1312930 N, 730506 E	Lawson Road

Table 7.   Geographic location of hydraulic structures in the Fivemile Creek study reach.

After the bridges are entered the ineffective flow areas are set. The ineffective flow areas are based on the beginning and ending stations of the bridge. These are areas where flow is not allowed to pass. These points are also set on the bounding cross sections. The points of ineffective flow are based on the location of the cross section and the contraction and expansion coefficients. For this project, the values of expansion and contraction on the bounding cross sections were 0.5 and 0.3, respectively. The ineffective flow areas were set to the corresponding beginning and ending stations of the bridge, because the bounding cross sections were within a few feet of the upstream and downstream face of the structures.

Once the geometric data has been entered and saved, the boundary conditions are entered. The boundary conditions for this project include the peak flow at various locations and the slope used to calculate normal depth at Section A. The hydrology for the 2003 flood was computed and transferred to different locations based on the drainage areas and transfer equations from Atkins 1996. For this reach peak flows were entered at river stations 18068 (section K), 14986 (section I), and 5206 (section E). These locations were chosen to account for the additional inflow brought to the system by Barton Branch and the increase in drainage area.

The second boundary condition entered was the slope. The slope was calculated by plotting a profile of the high water marks and fitting a linear line to the profile (Figure 12). The value computed for the slope of the high water marks is 0.0032 feet per feet. For comparison sake, the same method was used to compute the streambed slope. The result of this was 0.0035 feet per feet.

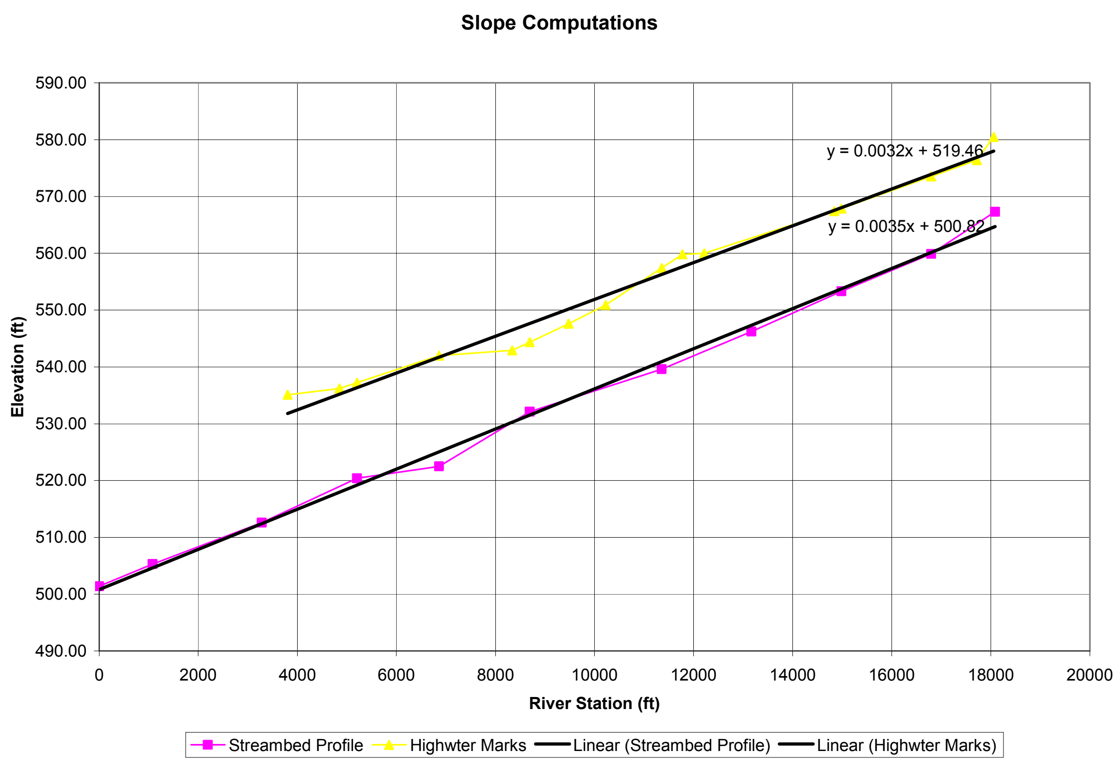

Figure 12. Slope computations for the Fivemile Creek basin.

Calibration

Subsequent to entering and checking the data, the computational component of the model was used. After the iterations were complete the warnings were checked and noted. The output was inspected and it showed that the resulting water-surface profile was low in some areas. The methods used to calibrate the water-surface profile were the addition of interpolated cross sections, changes in cross sectional geometry, and variations of the assigned Manning's roughness coefficient. In order to have closer agreement between the simulated profile and the actual profile, additional cross sections were added. These sections were developed using the "interpolate between cross sections" function in HEC-RAS. After the sections were generated they were checked for geometric accuracy. Roughness coefficients were assigned to these sections based on aerial photography and field observations. The original input data was also modified to account for areas of ineffective flow. Roughness values were also slightly adjusted to simulate the 2003 flood profiles.

Results

The final Manning's roughness coefficients used to simulate the 2003 flood can be seen in Figure 22. This figure represents the roughness coefficients for the entire reach. Pictures of the channel and floodplain in the vicinity of the cross sections can be viewed in the Appendix in Figures 29 through 41. The resulting profiles generated in HEC-RAS compared closely (within 0.25 feet) to the high-water profile of the 2003 flood.

The coefficient of discharge and weir coefficient were based on the default values, for these computations. The use of the default values is common practice but does not always provide accurate results. The flow through the Lawson Road bridge was known for the 2003 flood. This provided an excellent opportunity to compare the flow distribution computed with a known distribution.

The default weir coefficient and coefficient of discharge, in HEC-RAS, is 2.6 and 0.8, respectively. With these values and the model calibrated to the 2003 high-water profile the resulting flow distribution was 11800 ft^3/s (84%) bridge flow and 2300 ft^3/s (16%) weir flow. The indirect calculations (Appendix Figure 44) showed the bridge carried 8760 ft^3/s (62%) and the remaining 5320 (38%) ft^3/s was weir flow. This indicates that the default values resulted in 35 % error in the amount of flow through the bridge. The default values indicate that the bridge is more efficient than it actually is. The default values were adjusted to reflect actual conditions. The coefficient of discharge was lowered to 0.58. This value was tabulated based on the bridge submergence ratio. The ratio was calculated using the upstream stage and the low cord of the bridge. The weir coefficient was also adjusted to 3.4. With these values the computations resulted in 9020 ft^3/s (64%) in bridge flow and 5080 ft^3/s (36%) in weir flow. The calculated percent error reduced to only 3 %, this is a much more acceptable value. This also slightly adjusted the water-surface profile (Table 8). The resulting profile was calibrated within 0.25 feet of the 2003 flood profile. A graphical plot of the measured versus computed water-surface profile can be seen in the Appendix in Figure 45.

River Station	OBSERVED	HEC-RAS	Difference
5,206	537.22	537.25	0.03
6,863	542	542.1	0.1
8,692	544.37	544.13	0.24
11,357	557.4	557.54	0.14
13,315	565.84	565.93	0.09
14,986	567.88	568.09	0.21
16,798	573.53	573.78	0.25
17,618	576.4	576.61	0.21
18,068	580.5	580.7	0.2

Table 8. Resulting profiles using the hydraulic model HEC-RAS for the May 07, 2003 flood on Fivemile Creek..

VII. TWO- DIMESNIONAL ANALYSIS

History

The two-dimensional model selected for the study reach was, Finite Element Surface-Water Modeling System for Two-Dimensional Flow in the Horizontal Plane (FESWMS-2DH) contained within Surface water Modeling System (SMS). Initially the software of choice was Finite-Element Surface-Water Modeling System for Two-Dimensional Flow in the Horizontal Plane (FESWMS), (Froehlich, 1989). It is made up of a modular set of computer programs that aid in the creation and execution of the two-dimensional flow computations. The modules are; input data preparation (DINMOD), flow model (FLOMOD), output analysis (ANOMOD), and the graphics conversion module (HPPLOT). The major computational engine of this model was FLOMOD. This computer software package was created through cooperation of the U.S. Geological Survey (USGS) and the Federal Highway Administration (FHWA). As advancements in technology were made the model was updated to provide user friendly options. The new model was called Finite Element Model Interface (FEMI) (R.R. McDonald, U.S. Geological Survey, 1999). It is comprised of automated grid generator (GRIDGEN), output visualization tool (MODVIS), and flow model (FLOMOD). Currently the model has been updated to provide the latest graphical user interfaces (gui) to minimize the time spent on the construction and execution of the model. The current model is contained within Surface Water Modeling System (SMS). SMS was created by Brigham Young University. The program is comprehensive and enables the programmer to use several models one of them being the FESWMS-2DH.

Theoretical Basis

The partial differential equations that govern two-dimensional flow are derived from the three-dimensional equation (Froehlich 1989). The governing equations are solved using the Galerkin finite element method. In order to apply the finite element method the boundaries of the reach are set and the reach being modeled is divided into elements. These elements are triangular or quadrangular in shape. Each individual element is outlined by a series of nodes. Triangular elements are outline by placing nodes at the vertices and mid-side points. In the case of the quadrangles they consist of 9 nodes, having one in the center. The dependent values of these elements are determined using a set of interpolation functions, also called shape functions.

Depth-Averaged Momentum Equations

The depth average surface-water flow equations are derived by integrating the three-dimensional form of the conservation of mass and momentum equation. In this derivation the vertical velocities and accelerations are considered to be negligible. Hence the consideration of analysis of flow in two-dimensions.

X-Direction:

$$\frac{\partial}{\partial t}(HU)+\frac{\partial}{\partial x}(\beta_{uu}HUU)+\frac{\partial}{\partial y}(\beta_{uv}HUV)+g(H)\frac{\partial}{\partial x}(z_b)+\frac{g}{2}\frac{\partial}{\partial x}(H^2)-\Omega HV\ldots$$

$$+\frac{1}{\rho}\left[\tau_x{}^b-\tau_x{}^s-\frac{\partial}{\partial x}(H\tau_{xx})-\frac{\partial}{\partial y}(H\tau_{xy})\right]=0 \qquad \text{Equation 13}$$

Y-Direction:

$$\frac{\partial}{\partial t}(HV)+\frac{\partial}{\partial x}(\beta_{vu}HVU)+\frac{\partial}{\partial y}(\beta_{vv}HVV)+g(H)\frac{\partial}{\partial y}(z_b)+\frac{g}{2}\frac{\partial}{\partial y}(H^2)-\Omega HU\ldots$$

$$+\frac{1}{\rho}\left[\tau_y{}^b-\tau_y{}^s-\frac{\partial}{\partial x}(H\tau_{yx})-\frac{\partial}{\partial y}(H\tau_{yy})\right]=0 \qquad \text{Equation 14}$$

This is a very complicated equation and therefore each piece will be broken out separately and the variables explained. The figure below denotes the orientations used. The variable (H) is a function of position in the x and y directions. This is what allows the model to emulate superelevated flow. One-dimensional models only allow depth variations in the direction(x) of flow. The partial differentiation of (z_b) would result in the slope in the direction of flow (x) and in the perpendicular direction(y). These values are determined through the interpolation of user entered nodal data.

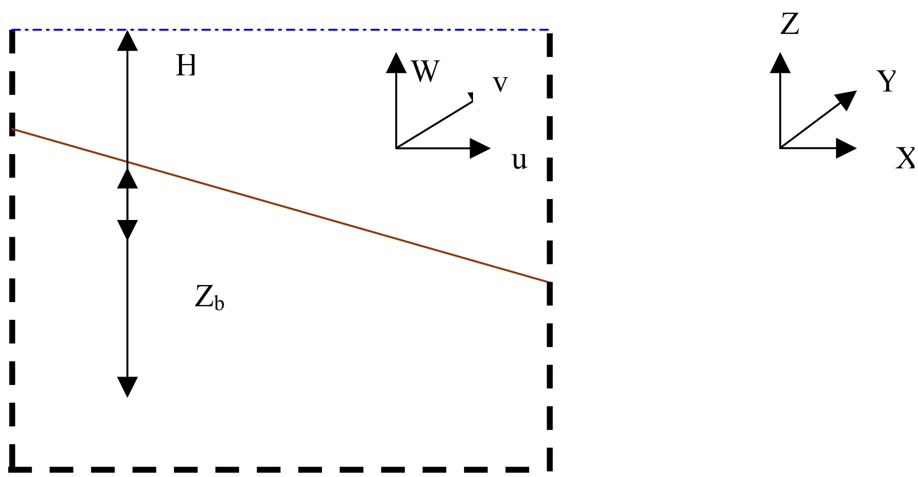

Figure 13. Control volume schematic.

Inside the control volume the velocities are represented by their point velocity nomenclature u and v. The orientation inside the control volume is referenced to the velocity. The point velocity (v) and the depth averaged velocity (V) are in the y direction, and the point velocity (u) and the depth averaged velocity (U) are the x direction. These combine to represent the velocity in the horizontal direction.

The capital letters U and V indicate the depth-averaged values of the functions that describe the velocity profiles. An illustration of these profiles can be seen in the Figure 14. In order to obtain the depth average of the velocity, the function, that describes the shape of the velocity, is integrated over the depth of the flow. This is done by setting the boundaries to: z_b+h, z_b.

$$U = \left(\frac{1}{H}\right)\int_{z_b}^{z_b+H} u(dz) \qquad \text{Equation 15}$$

$$V = \left(\frac{1}{H}\right)\int_{z_b}^{z_b+H} v(dz)$$ Equation 16

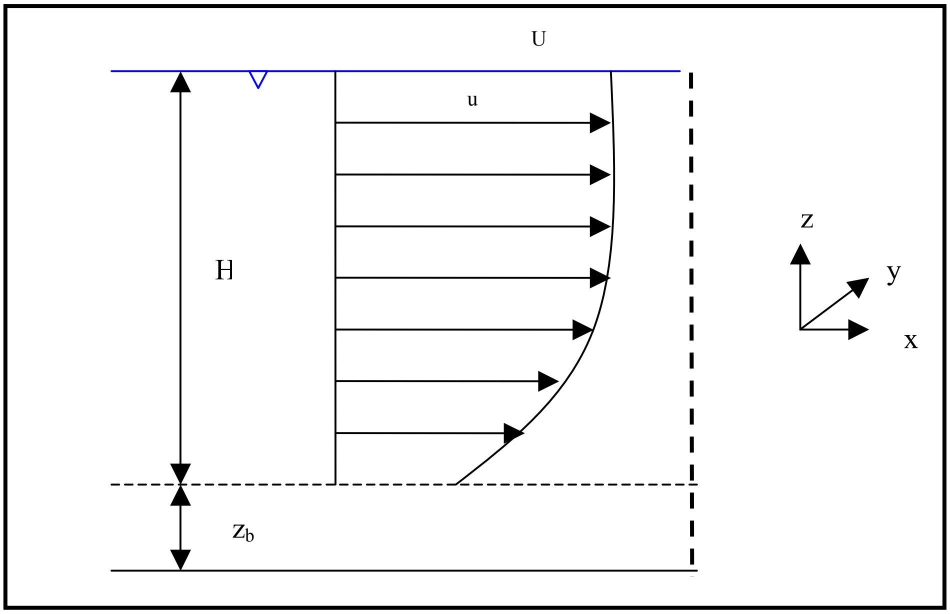

Figure 14. Velocity profile.

Momentum Correction Factor

The momentum equation results in a several correction factors. The first correction factor examined is the momentum correction factor (β). These correction factors account for vertical variation of the velocity in the x and y directions. It should be noted that $\beta_{uv} = \beta_{vu}$. In most cases these terms are assumed to be equaled to unity and a uniform vertical velocity is assumed. Based on this assumption, the variables can be removed from the original equations.

Coriolis Effect

The second correction factor accounts for the Coriolis effect. The Coriolis effect accounts for the force affecting flow due to the Earth's rotation. The variable used to define the Coriolis effect is (Ω).

$$\Omega = 2(\omega)\sin(\Phi) \qquad \text{Equation 17}$$

The variable (Φ) is measured from the equator and returns a positive value for the northern hemisphere and a negative value for the southern hemisphere. This effect is minimal, for reaches that have a large width to depth ratio. An example of that would be river or flood plain flow. The default value used in FESWMS-2DH is based on the assumption that the variation of (Ω) is small. To test this theory, the Coriolis factor was computed based on the average values of latitude for Auburn, Mobile, and Fort Payne.

City	Latitude	$\sin(\varphi)$	$\Omega = 2\omega*\sin(\varphi)$
Auburn	32° 36' 34"	0.5389	0.0045
Mobile	30° 41' 38"	0.5105	0.0043
Fort Payne	34° 28' 38"	0.5661	0.0047

Table 9. Coriolis correction factors for various geographic locations.

The results (Table 9) show that the values differ only slightly. This is due to the small magnitude of the angular velocity of Earth (ω). A constant value of 0.0045, for the angular velocity of the Earth, is assumed within FESWMS-2DH.

Bottom Shear Stress

There are three types of stress considered; bottom shear, surface shear, and stress caused by turbulence. Bottom shear stress, is defined by the variables (τ^b_x) and (τ^b_y). These variables represent bottom shear stress in the x and y directions, as previously defined. These variables are a function of the depth averaged velocity (U and V), density, bed slope in the x and y directions and the variable (c_f). In most cases the density is assumed to be a constant value. The variable (c_f) is known as the bed friction coefficient and can be computed by the two formulas shown below.

$$\tau_x{}^b = \rho(c_f)U\left(U^2+V^2\right)^{1/2}\left[1+\left(\frac{\partial z_b}{\partial x}\right)^2+\left(\frac{\partial z_b}{\partial y}\right)^2\right]^{1/2} \quad \text{Equation 18}$$

$$\tau_y{}^b = \rho(c_f)V\left(U^2+V^2\right)^{1/2}\left[1+\left(\frac{\partial z_b}{\partial x}\right)^2+\left(\frac{\partial z_b}{\partial y}\right)^2\right]^{1/2} \quad \text{Equation 19}$$

$$c_f = \left(\frac{g}{C^2}\right) \quad \text{Equation 20}$$

$$c_f = \left(\frac{gn^2}{\varphi H^{1/3}}\right) \quad \text{Equation 21}$$

It should be noted that FESWMS-2DH allows the roughness to be varied with depth. This is a handy tool when the area being modeled has drastic changes in surface roughness with depth. This is also an option contained within HEC-RAS and WSPRO. However the Cheesy coefficient can not be specified as a function of depth.

Surface Shear Stress

The second type of stress to be considered is surface shear stress. This component is created by the wind. Several independent studies have measured and investigated the values used to compute this stress. Based on these findings the default values, within FESWMS-2DH, are set and do not have to be adjusted by the user.

Stress Caused by Turbulence

The final variable considered represents the stress caused by turbulence. The Bossiness's eddy viscosity concept is used to determine the depth-averaged stress caused by turbulence. The stress is assumed to be proportional to the gradients of the depth-averaged velocities. The stress caused by turbulence is represented by the variables; τ_{xx}, τ_{xy} ,τ_{yx}, τ_{yy}.

$$\tau_{xx} = \rho \hat{\upsilon}_{xx}\left(\frac{\partial U}{\partial x} + \frac{\partial U}{\partial x}\right) \qquad \text{Equation 22}$$

$$\tau_{xy} = \tau_{yx} = \rho \hat{\upsilon}_{xy}\left(\frac{\partial U}{\partial y} + \frac{\partial U}{\partial x}\right) \qquad \text{Equation 23}$$

$$\tau_{yy} = \rho \hat{\upsilon}_{yy}\left(\frac{\partial V}{\partial y} + \frac{\partial V}{\partial y}\right) \qquad \text{Equation 24}$$

The variables $\hat{v}_{xx}, \hat{v}_{xy}, \hat{v}_{yx}, and\, \hat{v}_{yy}$ represent the depth-averaged kinematic eddy viscosity. FESWMS-2DH assumes the depth-averaged kinematic eddy viscosity is isotropic. Based on this assumption, there is no variation directionally ($\hat{v}_{xx}, \hat{v}_{xy}, \hat{v}_{yx}, \hat{v}_{yy} = \hat{v}$). The eddy viscosity can be computed using Equation 25.

$$\hat{v} = \hat{v}_o + c_\mu (U_*)(H) \qquad \text{Equation 25}$$

The eddy viscosity if related to the eddy diffusivity for heat or mass transfer by the following equation.

$$\Gamma' = \frac{\hat{v}}{\sigma_t} \qquad \text{Equation 26}$$

We are interested in steady flow based on a peak, for these purposes. This allows the first term, ∂/∂t (HU) to be eliminated. This along with the simplifications previously outlined the governing equations in the x- and y- directions reduce to the following equations.

X-Direction:

$$\frac{\partial}{\partial x}\left(HU^2\right) + \frac{\partial}{\partial y}(HUV) + g(H)\frac{\partial}{\partial x}\left(z_b\right) + \frac{g}{2}\frac{\partial}{\partial x}(H^2) - 0.0045HV \ldots$$

$$+\frac{1}{\rho}\left[\tau_x{}^b - \tau_x{}^s - \frac{\partial}{\partial x}\left(H\tau_{xx}\right) - \frac{\partial}{\partial y}\left(H\tau_{xy}\right)\right] = 0 \qquad \text{Equation 13}$$

Y-Direction:

$$\frac{\partial}{\partial x}\left(HVU\right)+\frac{\partial}{\partial y}(HVV)+g\left(H\right)\frac{\partial}{\partial y}\left(z_b\right)+\frac{g}{2}\frac{\partial}{\partial y}(H^2)-0.0045HU\ldots$$

$$+\frac{1}{\rho}\left[\tau_y{}^b-\tau_y{}^s-\frac{\partial}{\partial x}\left(H\tau_{yx}\right)-\frac{\partial}{\partial y}\left(H\tau_{yy}\right)\right]=0 \qquad \text{Equation 14}$$

Depth-Averaged Continuity of Mass

The other governing equation is the continuity of mass. The conservation of mass equation is shown below.

$$\frac{\partial H}{\partial t} + \frac{\partial (HU)}{\partial x} + \frac{\partial (HV)}{\partial y} = 0 \qquad \text{Equation 27}$$

As years have passed and computers have advanced through the use of graphical user interfaces the every day application of flow models has been greatly simplified. These advancements have made it easier to model complex situations. However, through the examination of the one- and two-dimensional equations, it is evident that an answer is not a competent answer without the proper knowledge of basic hydraulic principles.

Variables

c_f = bed friction coefficient

c_μ = dimensionless coefficient, in natural channels 0.6

C = the Chezy discharge coefficient

g = acceleration rate due to gravity 32.2 ft/s^2 or 9.81 m/s^2

H= Water depth

n = Manning's roughness coefficient

ρ = density of water

τ_x^b, τ_y^b = bottom shear stresses acting in the x,y directions respectively

τ_{xx}, τ_{xy}, τ_{yx}, τ_{yy} = shear stress caused by turbulence, where for example τ_{yx} acts in the x direction on a plane that is perpendicular to the y -direction

$\acute{\Gamma}$ = eddy diffusivity for heat or mass transfer

u =horizontal velocity in the x-direction (point velocity) along the vertical coordinate

U_* = bed shear velocity

U = depth-averaged velocity in the horizontal direction

v = horizontal velocity in the y-direction (point velocity) along the vertical coordinate

V= depth-averaged velocity in the vertical direction

$\hat{v}_{xx}, \hat{v}_{xy}, \hat{v}_{yx}, \hat{v}_{yy} = \hat{v}$ = directional values of the depth-averaged kinematic eddy viscosity

$\hat{v}_o$ = base kinematic eddy viscosity

ω = angular velocity of the rotating Earth (7.27 x 10^{-5} rad/s or 0.00417 degrees/second)

z_b = Bed elevation

Ω = Coriolis Parameter

Φ = mean angle of the latitude being modeled.

φ = 2.208 (U.S. Customary) and 1.0 (S.I.)

$\sigma_{t\,=}$ empirical constant, Prandt number (for diffusion of heat), Schmidt number (for diffusion of mass)

Applications

The Finite Element Surface-Water Modeling System for Two-Dimensional Flow in the Horizontal Plane (FESWMS-2DH) is one of many computational programs within the Surface Water Modeling System (SMS) package. As previously described it uses three partial differential equations to represent to conservation of mass and momentum (Froehlich 1989).

The simulation of flood flows on Fivemile Creek was represented with a finite-element grid. This grid was made up of 5,603 elements and 12,597 nodes (Figure 16). Of the 5,603 elements 5,094 were triangular in shape primarily representing the overbank areas. The remaining 509 elements represent the channel and weirs.

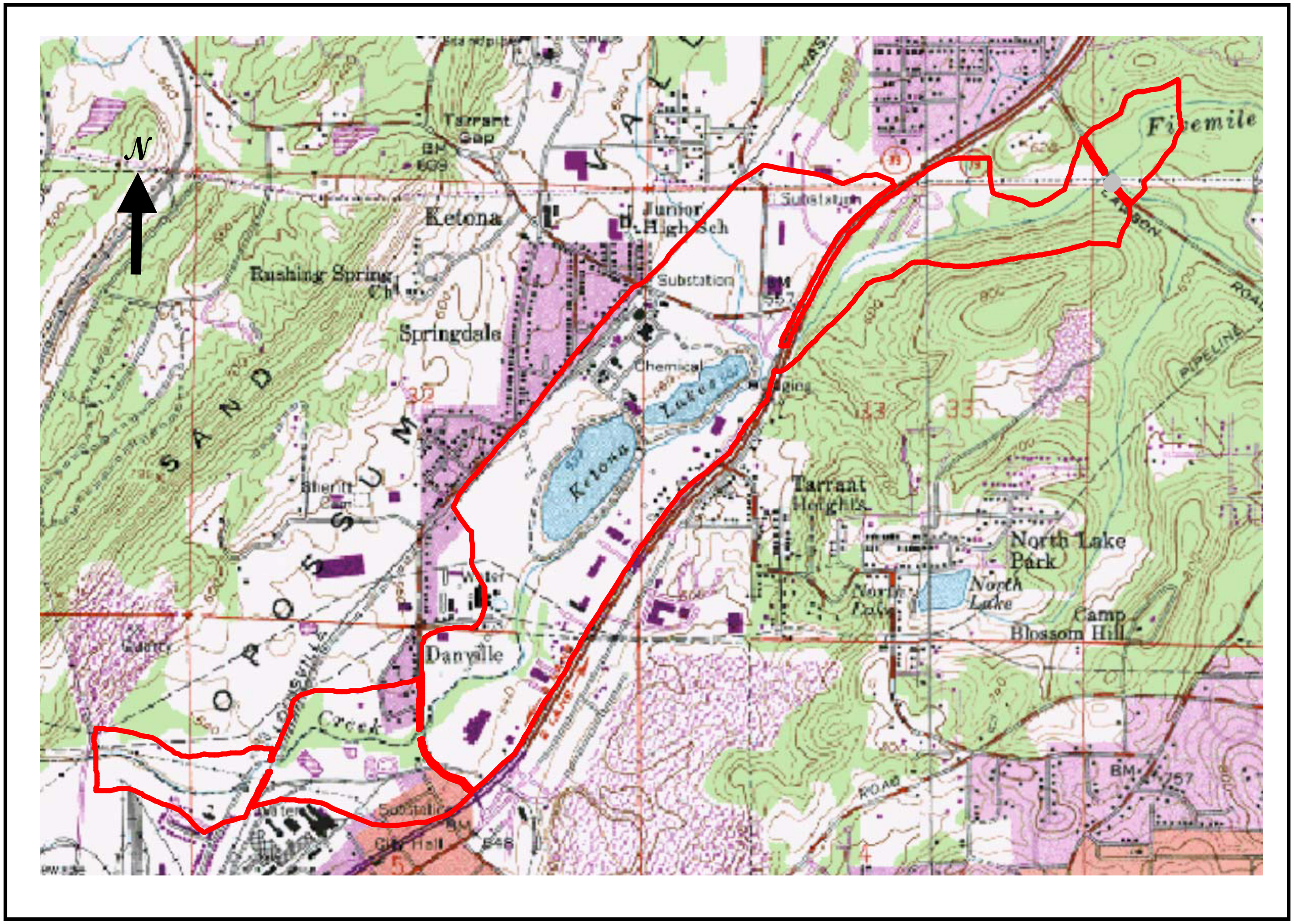

Figure 15. Study reach for two-dimensional flood flow simulations on Fivemile Creek.

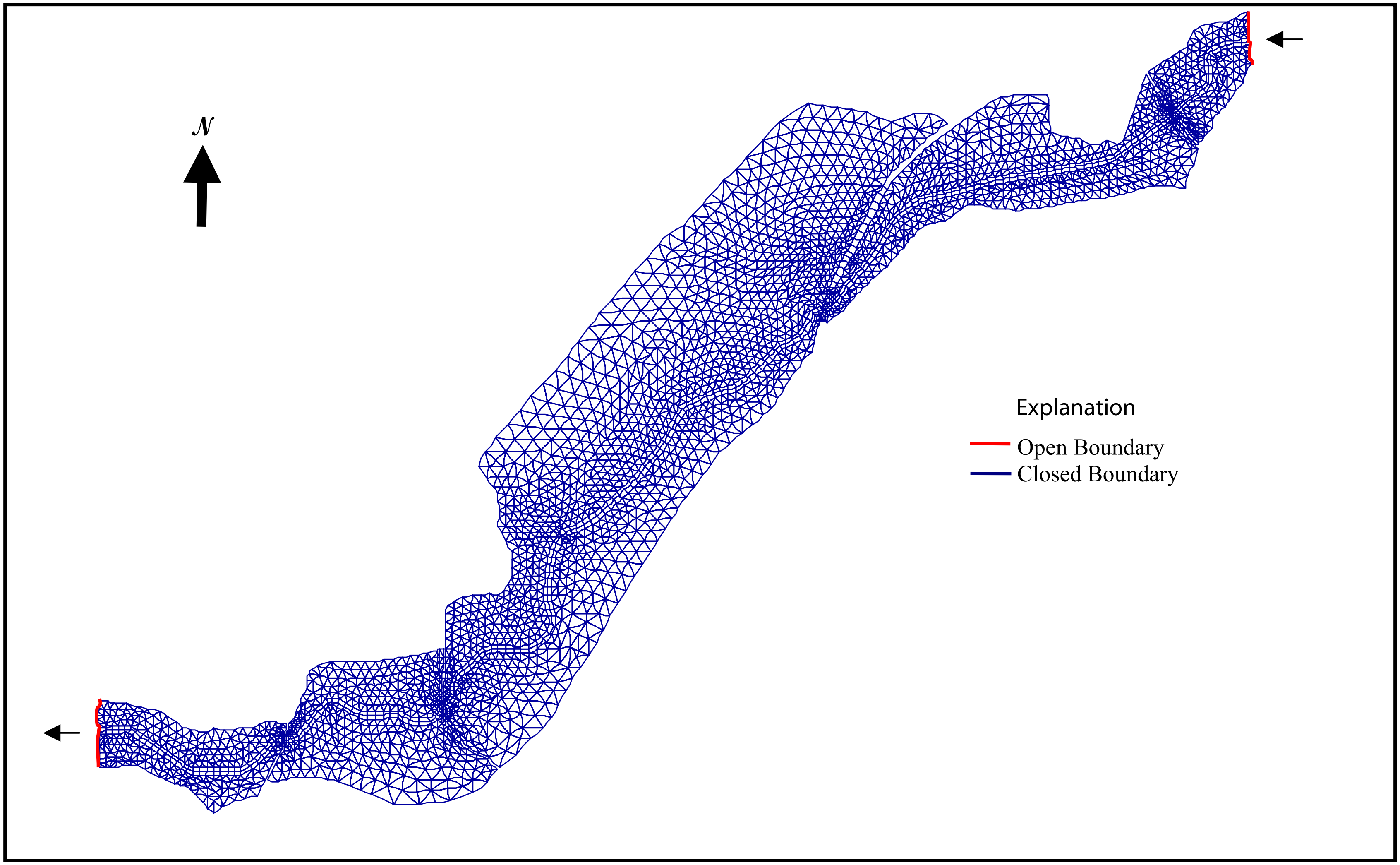

Figure 16. Finite-element grid used in flood flow simulations for the May 07, 2003 flood on Fivemile Creek.

Prior to the construction of the grid the boundary conditions must be determined. The boundary condition will be either closed or open. A closed boundary is one that does not allow flow to cross it. Closed boundary conditions usually exist at high ground, which is known to contain the flood waters, or places where the flow is obstructed. An open boundary represents areas where the flow is introduced and where it exits. The Fivemile Creek study reach is bound by a closed boundary determined from topographic maps and field surveys. The upstream and downstream boundaries of the creek were designated as open boundaries (Figure 15).

The initial step in constructing a grid network is to determine what type of image the grid will be built on. The most common is the use of a topographic map. Once the image is selected it needs to be geo-referenced in SMS.

When modeling a large reach it is helpful to use the automated grid generator. This is done through the use of feature arcs. Before drawing the feature arcs it is always a good idea to do some preparatory work. As discussed in latter sections, the channel can be made of one or more elements. Prior to construction the user should determine what geometric shape will best represent the channel in the grid network. All roadways should be inspected and their widths noted. The user should also ascertain how dense the resulting grid should be. To provided insight on this matter the friction and bed slopes should be inspected. In the case of this reach, the slope of the water surface profile, for the 2003 flood, is 0.0032 and the streambed slope is 0.0035 or 18 feet per mile. Based on this a dense grid is more desirable. Most guidelines suggest that FESWMS-2DH should not be used on a reach that has a slope greater than 20 to 25 feet per mile. In retrospect if the user decides the grid should have been denser there is an option to refine the grid.

This option will break each element into four elements. This can be done for the whole grid or select elements.

Once the necessary decisions are made the overbank can be outlined with feature arcs. The channel elements were determined to be 80 feet top width, for this project. To represent this, two feature arcs were drawn on either side of the channel 80 feet apart. Each feature arc is made up of nodes connected by line segments. The distance between the node points will determine how dense the resulting grid will be. Depending on the topography sections of the overbank are defined by connecting feature arcs to form a polygon. An illustration of this can be seen in Figure 17.

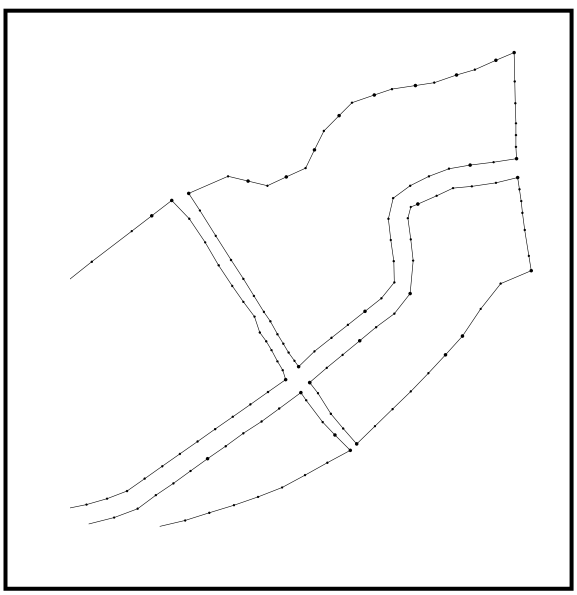

Figure 17. Illustration of mesh generation through the use of feature arcs.

All of the feature arcs forming the polygon are selected and the 'Feature Objects' → 'Build Polygons' command is executed. The polygon can be selected and double clicked on. This will bring up the window where the elements are generated. The mesh type chosen is up to the user, but it has been found that the paving method results in fewer errors and works computationally just as well. This can be viewed in Figure 18.

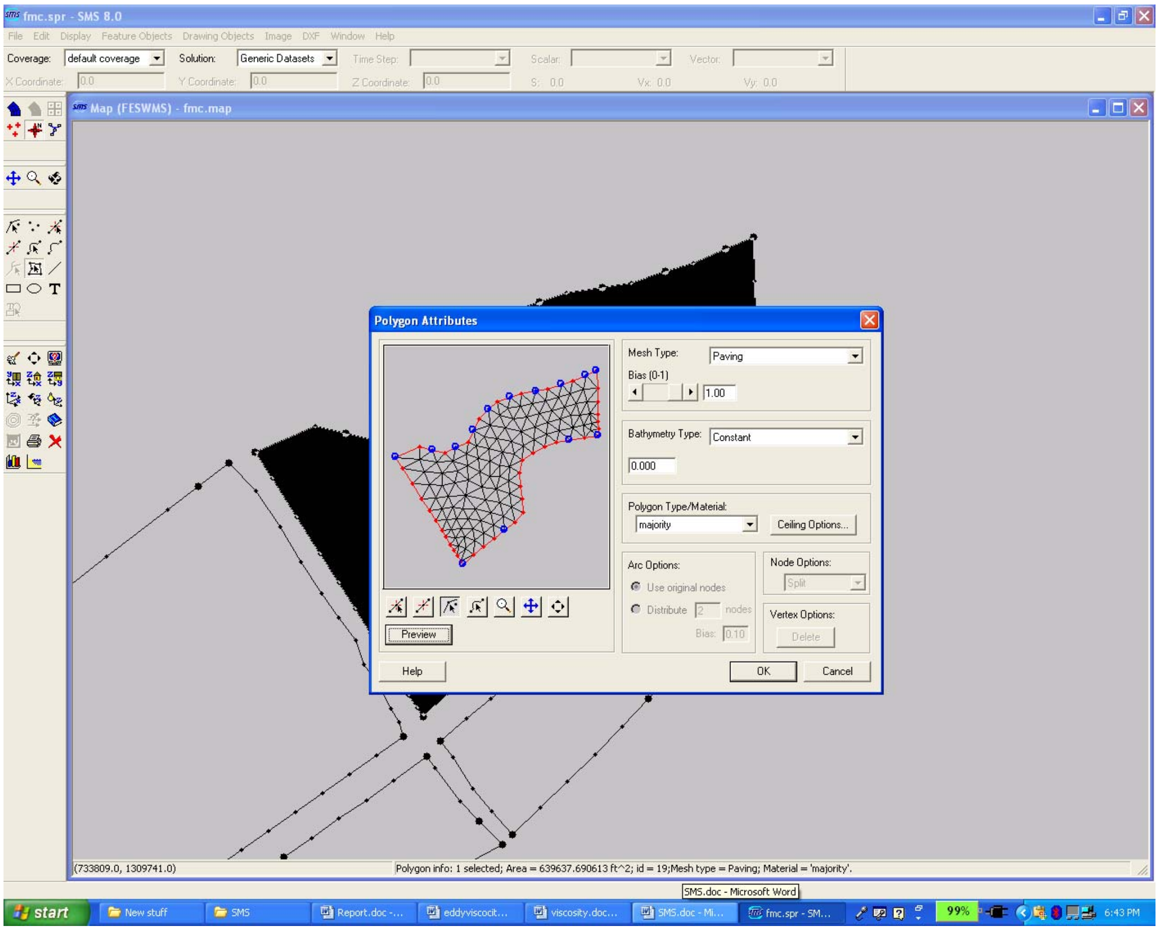

Figure 18. Mesh generation tools in SMS.

An important concept when inspecting the resulting grid is the density in relation to the channel. The grid should be denser near the channel and sparse away from the channel. The channel elements should be constructed by hand. It is preferable to stay consistent with the shape (rectangular or triangular) of element used in the channel. This is sometimes unavoidable and results in strange geometry within the channel.

The final elements to be constructed are the hydraulic structures. It is very important to make the grid dense around bridges. The area representing the bridge should be two elements deep. The bridge definition should be dense enough that the model does not have a problem converging on a solution.

In order to improve the quality of the solution there are guidelines used to judge the quality of elements within the mesh. The user can set the specifications or leave them at the default values. In this model the specifications were left at the default values. The minimum and maximum exterior angles should not exceed 29 and 120 degrees, respectively. The maximum slope computed across an element should not exceed 0.1. The model also checks the size of elements relative to adjacent elements. Element should not have an area change greater than 0.5 or 2.0. This makes it difficult to have a dense bridge opening with larger elements surrounding it. This is why it is suggested that the mesh be more refined around hydraulic openings and adjacent roadways. The model also checks for concave quadrilaterals and ambiguous gradients.

Once the grid is built the elevation of the nodes must be defined. This can be done through numerous ways. If the user has well defined Digital Elevation Maps (DEM) at their disposal the amount of time is greatly reduced. However, DEMs are rarely as distinct as field surveys. In the case of this model the same field survey used in the one-dimensional model was imported and the ground points were interpolated.

It is important to consider the elevation of the points imported. The thalwegs should be negated when importing ground points. The interpolation method is not capable of interpolating the channel definition. If the thalwegs are imported the elevation of the overbank areas will be skewed because of the low value of the thalweg. Another concern is the definition of obscure geometric break points. High or low areas that are not continuous from one cross section to the next should also be omitted. An example of this is a section H. The right overbank had a large mound of dirt that did not continue all the way to section G. These points were not imported; rather the area was defined by hand. Once the interpolation is complete the user should always check the validity of the points interpolated.

The typical one-dimensional method of defining geometry is not applicable for two-dimensional models. A channel within a flood plain cross section is usually represented by station and elevation. This allows the user to show every geometric break within the channel. One draw back of FESWMS-2DH is the channel is represented by one or more elements. The elements are bent to form a simple shape of the channel. Smaller sites are typically represented with one element, a rectangle. These rectangles are bent to correspond to a two-dimensional triangle. The mid side nodes are assigned an elevation to represent the thalweg of the channel. The figure below shows how the rectangles are transformed into a two-dimensional triangle. The red dots represent the midside nodes.

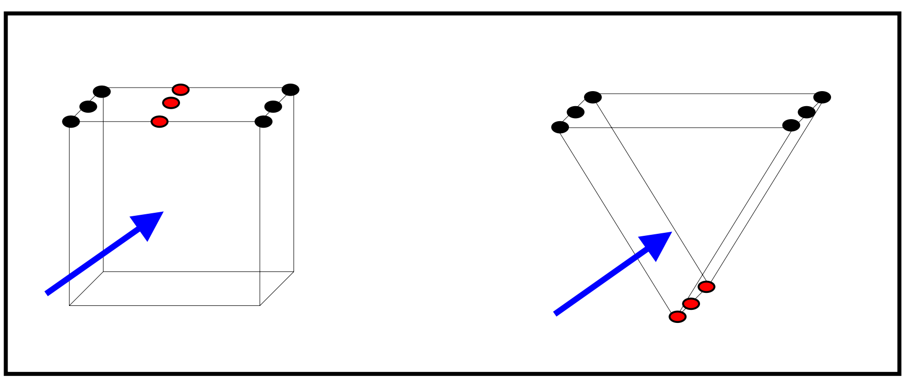

Figure 19. Illustration of the bending of elements to represent triangular channel geometry.

The shape that was determined to best suit this reach was a trapezoid. Because the creek was relatively small in comparison to most river reaches the channel was constructed of two rectangles bent at their joining node and mid side nodes. The figure shown below is a graphical representation of the channel shape modeled in this reach.

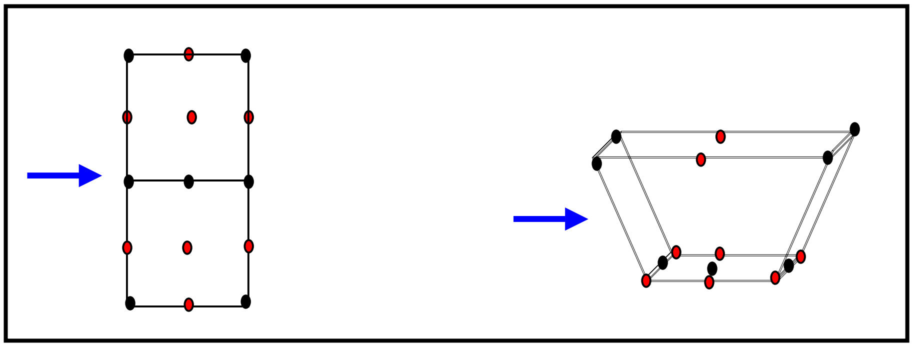

Figure 20. Illustration of the bending of elements to represent trapezoidal channel geometry.

This shape worked geometrically, but it was found that during the course of the modeling the midside nodes would change back to the average values in certain areas. This did not compromise the solution; however it took added time to maintain the desired shape. The elevations of the top of banks and thalwegs were entered by hand. It is important to do this after the ground points are imported and interpolated between. This was one of the major time investments.

Once all of the nodes have been programmed with the appropriate geometry the entire reach should be inspected for any nodes that were overlooked or were assigned the wrong elevation. This can be done by inspecting the contour plot (Figure 21) of the mesh. The elevations ranged from 509 to 588 feet, for this project.

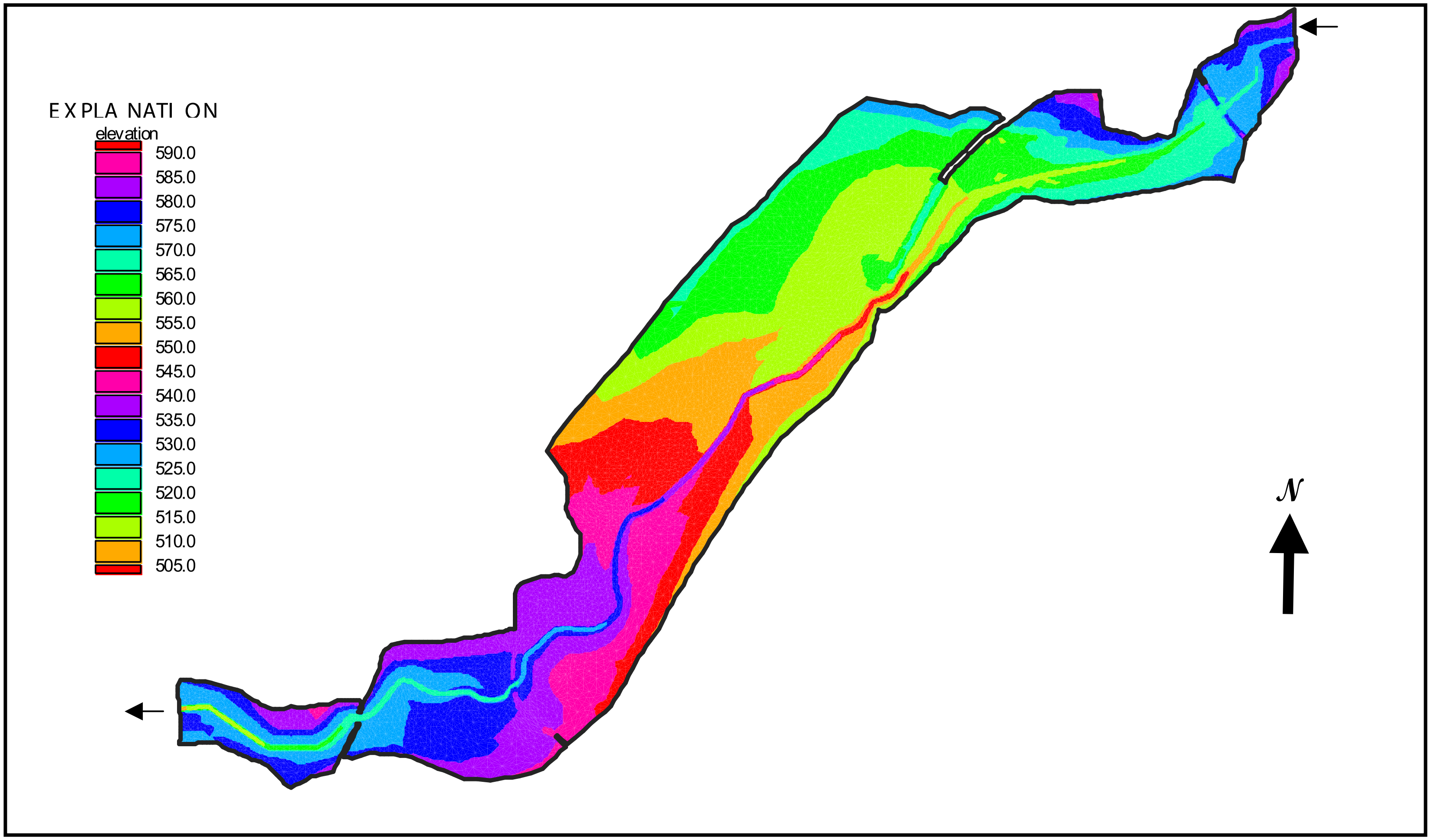

Figure 21. Land-surface elevations for the Fivemile Creek study reach.

Once the elevations have been interpolated or entered for each node the next logical step is to assign the material properties. The areas that need to be set are the Manning's roughness coefficient, base kinematic eddy viscosity, and eddy diffusivity.

The selection of Manning's roughness can be aided by the references Chow (1959), or Barnes (1987). However, these calculations are based on the assumption of one-dimensional flow. The depth averaged flow equations as outlined in the theory directly account for turbulence and horizontal variations in velocity. This indicates that if the same roughness values were used in a 2-dimensional model as a 1-dimensional model, these factors would be accounted for twice and the resulting water-surface elevation would be higher than it should be.

For the initial simulation for the 2003 flood, the same roughness values were used as the calibrated roughness values from HEC-RAS. This was done in order to determine the variation in one- and two dimensional roughness values. The roughness values were then lowered incrementally to simulate the 2003 flood event. It should be noted that it is easier for the model to transition from lower to higher values of roughness than vice versa. When calibrating to a known flood event it is useful to have a base line of how much to reduce one- dimensional roughness values. As stated in the discussion of HEC-RAS, when calibration data is not available, the 2-year flood event is inspected. Smaller floods are not easily modeled using FESWMS-2HD, so it helps to know the range of roughness coefficient reduction.

One of the other input material properties is the base kinematic eddy viscosity. The base kinematic eddy viscosity is based on the Boussinesq principle. Eddy viscosity is typically referred to as internal friction. It is created when laminar flow becomes turbulent as it passes over surface irregularities. It is also described as the turbulent transfer of momentum due to internal fluid friction. The eddy diffusivity is typically entered as 0.8.

The model can be error free and have accurate geometry but without the proper boundary conditions the results are not justified. The upstream boundary condition is the introduction of flow, for this reach. If the grid represents a relatively short reach there is little judgment involved. However if the reach is long and the drainage area varies significantly throughout the reach, the user should consider where the additional flow will be introduced. There are several methods of doing this.

The Fivemile Creek reach is approximately 20,000 feet long and as previously mentioned experiences notable changes in drainage area and peak flow. The first change in attributing area is the addition of Barton Branch. Barton Branch is introduced to the system above the Highway 79 crossing. Because this is an external tributary it could be represented as an inflow on the exterior of the mesh. If the flow of Barton Branch is introduced in a relatively high area it is possible the elements will dry up and the flow will not be introduced to the system. The other change in drainage area and peak flow is just above the railroad crossing. This area was selected to represent a portion of the reach where the drainage area has increased and the flow at the upstream boundary is no longer valid. Because this is simply a case of increased drainage area it would be easier to introduce the additional flow as a source. Both areas were represented as a source, for this reach.

The upstream most inflow was represented with a nodestring. A value of 14,100 (ft^3/s) was entered for the 2003 flood, for this nodestring. This is the discharge computed by indirect methods, at Lawson Road, by the USGS. The peak flow computed above Highway 79 was 16,700 ft^3/s. This is a net gain of 2,600 ft^3/s. This addition was introduced as a source.

Once all of the inflow boundaries were entered the tailwater was entered by defining a nodestring at the downstream boundary of the gird. In order to properly calibrate the model the tailwater should be known. The most popular way of calculating the tailwater is the use of a one-dimensional model to obtain a water-surface elevation at the downstream boundary of the grid. The two-dimensional model started at section C and extended through section K. The one-dimensional model started at cross section A and extended through cross section K. Once the one-dimensional model was calibrated, the computed water-surface elevation at section C was used as the tailwater elevation, for the two-dimensional model. If there was a documented high-water mark a section C this elevation would have been used instead. The computed tailwater elevation for this reach was 533.9 feet. For the initial run or "cold start", every element within the mesh must be wet. In order to achieve this, the initial tailwater used was 590.0 feet. This elevation was spun down to the boundary condition of 533.8. The process of spinning down variables will be discussed in latter sections.

Prior to running FESWMS-2DH it is important to make all necessary changes to the grid. Elements can not be created or deleted during the spin down process. This is why it is important to inspect all aspects of the grid. The creation of the grid is an art that is improved with practice. The user should look at the grid and visualize the water flowing through the elements.

Once the grid is determined to be satisfactory visually and computationally the nodes should be renumbered. This is done by selecting the upstream nodestring and selecting 'Renumber' from the 'Nodestring' menu. There are two methods of completing this task and the band width method should be chosen.

Before FESWMS-2DH can be executed the parameters must be set. The run type should be specified as hydrodynamic and the solution types as steady state. The bottom stresses were calculated using Manning's Equation, for this reach. The other input variable that is adjustable, depending on the reach modeled, is the slip condition. Either a slip, no-slip, or semi-slip condition is specified for the closed boundary. The tangential stresses are assumed to be zero, for the slip condition. Under these circumstances the velocity at the boundary nodes are required to satisfy zero flow crossing the boundary. The no-slip condition assumes the velocity is set to zero. This automatically satisfies the condition of zero flow crossing the closed boundary. Semi-slip is a combination of these conditions (Froehlich 1989). The slip condition was chosen, for this reach

One other area that should be set is the convergence parameters. The 'Unit Flow Convergence' and the 'Water Depth Convergence' were set to a value of 0.1. Based on this, the model will not converge on a solution until the maximum change is equal to or less than 0.1. The 'Element Wetting and Drying Tolerance' was also set to 0.25 feet. This is the minimum depth used to turn an element off.

FESWMS-2HD is very sensitive to drastic changes and therefore it has to go through a process called spinning down. This is where the initial values of the tailwater and the base kinematic eddy viscosity are given a large value and stepped down incrementally to the actual value. The same process is used for weir flow and bridge girders (pressure flow). The solver within the FESWMS-2HD module computes the water surface elevation and velocity for every node. This is done through an iterative process. In order to do this the solver has to have an initial water surface elevation. The solver assumes a constant water surface across the boundary and a velocity of zero. The first computation or cold start is based on the initial condition and boundary conditions files. The tailwater was set at 590 feet and the base kinematic eddy viscosity was set at 150 ft^2/s, for the cold start, for the Fivemile Creek study reach.

Once a cold start has been successfully executed the next step is a "hot start". The hot start uses the same iterative process based on the computed results of the previous run. For each successive run the variable being spun down is decreased until the target value is reached.

The first parameter that needs to be spun down is the tailwater. The tailwater is initially set high to make sure all elements are wet. During the spinning down process the tailwater is incrementally decreased. Once the tailwater is at the desired elevation the base kinematic eddy viscosity should be spun down. The typical starting value is 150 ft^2/s and is spun down in increments of 25 to 50. The target value is in the range of 10 ft^2/s to 25 ft^2/s depending on the flow energy and depth in the channel.

The next area of concern is weir flow. If the upstream stage of the hydraulic structures is less than the crest of the weir this is not a concern and the grid interface with the weir can be left as a closed boundary. The two-dimensional governing equations are depth averaged and the velocity in the vertical direction is assumed to be negligible. This is not a valid assumption for weir flow. Weirs and similar structures can have a significant velocity in the vertical direction. To account for flow over weirs, within the two-dimensional model, one-dimensional methods are used. This is done the through the use of an empirical equation.

The initial construction of the Fivemile Creek grid was based on the concept of weir segments. The model would not converge on a solution when the weir segments were lowered. Upon consultation of other modelers, it was found that this is a common problem. Various methods were employed to overcome this problem, but none resulted in a converged solution. Consequently, alternative methods were used. The model was updated to represent the weir as obstruction flow. This is an acceptable practice, but is not without flaw. This method of modeling weir flow assumes negligible velocity in the vertical direction. It should also be noted that obstruction flow is not as efficient as weir flow. This will cause the model to indicate a lower value of weir flow than direct weir methods.

The final step in the spin down process is the lowering of the ceiling values (pressure flow) of the structures. This is done in small increments due to the instability that it causes. This also was a source of error introduced into the model. The ceiling values of the two structures on the upper end of the reach were not successfully set to the actual value. If the ceiling elevations were successfully lowered the water-surface would have been slightly higher upstream of the structure. The lower ceiling elevation would also affect the coefficient of discharge and resulted in less flow going through the bridge.

Calibration

In order to provide a wide range of comparison FESWMS-2DH was executed for several different scenarios. The grid network was analyzed with the roughness coefficients from the one-dimensional study and reduced roughness values. Each method provided insight in proper way of representing the reach with two-dimensional flow assumptions.

The first simulation was used to compare directly to the one-dimensional results. The one-dimensional Manning's roughness coefficients were used in the two-dimensional model (Figure 22). The boundary of the roughness values were estimated from aerial photography. The results of this simulation gave a water-surface profile that deviated from the 2003 flood profile as much as 1.9 feet and as little as 0.4 feet. As expected the resulting water surface profile was higher than the one computed with the one- dimensional model.

Section	Observed (ft)	HEC-RAS (ft)	FESWMS-2DH Average (ft)	(ft)
D	537.22	537.25	537.65	0.4
E	542	542.1	541.3	0.8
F	544.37	544.13	546	1.9
G	557.4	557.5	556.4	1.1
I	567.88	568.09	568.9	0.8
J	573.53	573.8	574.5	0.7
K	580.5	580.55	581.55	1.0

Table 10. Resulting water-surface profile using one-dimensional Manning's roughness coefficients in the hydraulic model FESWMS-2DH for the Fivemile Creek study reach.

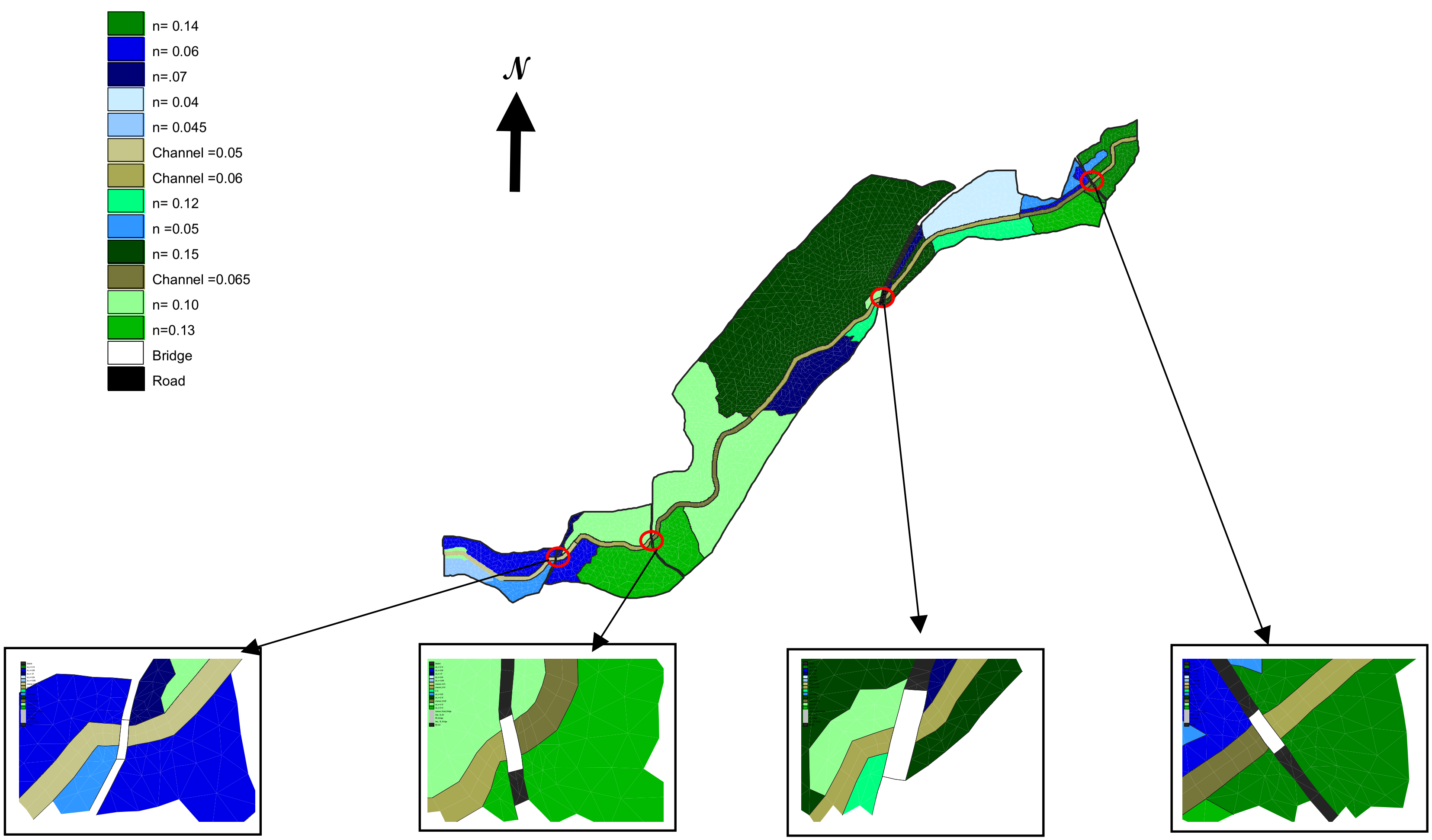

Figure 22. Initial Manning's roughness coefficients for the Fivemile Creek study reach

Results

Attempts were made to transition to lesser values of roughness in increments of 5 %, from the initial simulation. This was a difficult and unsuccessful process. This is evidence that if the user plans to calibrate the profile to a known flood their initial simulation should be with reduced roughness values. The question that most commonly arises is how much to decrease the roughness.

In order to calibrate the model to the 2003 flood profile a new simulation was created with the initial roughness values being decreased. The percent reduction was chosen from sensitivity analysis. The base reduction factor chosen was 20% for the overbanks and 5% for the channel roughness values. Preceding chapters outlined how variable Manning's roughness coefficient is. The methods used on this reach are not applicable in all situations, because every reach is unique and has its own characteristics. However, if enough studies are conducted and documented, relationships between the characteristics of the reach and the percent reduction can be developed and a based line reduction factor determined.

The decrease in roughness values alone did not ensure that the profiles accurately emulated the 2003 flood profiles. The majority of the cross sections depicted a water surface elevation that was within 0.8 feet of the known 2003 profile. The geometry was then inspected. Similar to the one-dimensional model areas of ineffective flow were added were large buildings are located. This was done by disabling elements to represent the buildings. This improved the accuracy of the profile. The profile (Table 11) created by the two-dimensional model was successfully calibrated within 0.65 feet of the known water surface profile.

Section	Observed (ft)	FESWMS-2DH Average (ft)	Difference (ft)
D	537.22	537.15	0.07
E	542.00	541.9	0.10
F	544.37	544.85	0.48
G	557.40	557.2	0.20
I	567.88	568.4	0.52
J	573.53	573.75	0.22
K	580.50	581.15	0.65

Table 11. Resulting water-surface profile with calibrated Manning's roughness coefficients for the hydraulic model FESWMS-2DH.

An important aspect of the two-dimensional model is the ability to represent superelevated flow. Superelevation is characterized by a higher water surface elevation in the outer bank of a bend and a lower water-surface elevation on the inner bank. It is created by the centrifugal force acting on the flow at the bend that causes the water to stack up in the outside of the bend, this also results in irregular velocity profiles in the area of the bend which corresponds to values of the coefficients α and β not being equal to unity (Chow1959).

One- dimensional flow models simulate the water-surface profile at a constant elevation for the entire cross-section. However, for short radius bends with a significant size channel or supercritical conditions the effects of superelevation should not be neglected. This is also a condition that should be considered when locating and documenting high-water marks. If the high-water mark is located in a bend a mark should also be located on the other bank. This is a good idea for one- and two-dimensional calibration. The two marks can be averaged, for one-dimensional calibration. The accuracy of superelevated results can be examined, for two-dimensional calibration. The maximum superelevation observed for the Fivemile Creek simulation was 1.5 feet.

Another area of inspection is flow distribution and velocity profiles. Larger flooding events often incite combination flow. Under these circumstances the model must separate the flow into bridge flow and weir flow. In some cases the model must determine the flow separation between multiple bridges and weir flow. Once the model has determined the flow distribution the resulting velocities are available. This is a major concern in the design of hydraulic structures. This reach was considered a prime candidate for the comparison of one- and two-dimensional results, because of a known flow distribution at Lawson Road. However, due to inconsistencies in the two-dimensional model it was determined that an accurate conclusion could not be drawn. The results of the two-dimensional model are considered to have sources of error near the Lawson Road crossing. The use of obstruction flow in the place of weir flow and difficulty in simulating pressure flow added error to the resulting flow distribution and velocity profile.

The one-dimensional model did not accurately depict the flow distribution between weir and bridge flow, at Lawson Road. From this it can be seen that the use of a calibrated water-surface elevation does not always produce an accurate flow distribution. Due to the lack of accurate results this could not be investigated for the two-dimensional model.

Other studies were researched to provide insight into the matter. One particular article "Use of Velocity Data to Calibrate and Validate Two- Dimensional Hydrodynamic Models", by Wagner investigated this topic with the use of RMA2. RMA2 is a finite element hydrodynamic numerical model developed by the U.S. Army Corps of Engineers. Wagner found that, "Calibrated models that accurately matched water-surface elevations did not necessarily guarantee an adequate match of the measured flow field" (Wagner 2002). The study consisted of a 40 mile reach. The reach had 40 cross sections with calibration and validation data for low and high flow. Each cross section was calibrated to the water-surface elevation. The velocity fields were then inspected and found to be in close agreement for low flow, but deviated from the observed values for high flow. It was found that in order to calibrate the model to the measured field a slip condition was used and a variance of roughness coefficients within the channel. This further supports the conclusion that a calibrated model does not ensure that all of the hydraulic properties will be calibrated.

VIII. COMPARISON OF ONE- AND TWO- DIMENSIONAL MODELS

The one-dimensional model is a highly efficient way of representing hydraulic conditions for flood studies and the design of hydraulic structures. However, the one-dimensional model is not suitable for all cases. It is based on several assumptions that do not always accurately represent the water-surface profile or velocity distribution. This includes reaches that have a high degree of sinuosity. The one-dimensional model assumes a constant water- surface elevation and is not equipped to simulate superelevated flow. One-dimensional models are not adequate to handle complex crossings with multiple openings, especially more than four or crossings having extreme skew.

Some of the difficulties incurred in the development and execution of the two-dimensional model include ground elevation data, Manning's roughness coefficient selection, and construction of the grid. Often accurate ground elevations are not available for the entire reach and interpolation introduces error. Most surface-water modelers use one-dimensional models on a day to day basis and develop their roughness coefficient selection skills based on the use of these models. These skills are not conducive to all reaches modeled with two-dimensional flow equations. The reduction of one-dimensional roughness values for two-dimensional flow varies based on the basin's characteristics. It was seen that for this reach the reduction of roughness values varied from 5% to 20%. These values are not applicable for all reaches but can serve as a base line reduction for a basin with similar characteristics. More research should be done on this topic to provide modelers with a sense of acceptable reduction factors when calibration data is not available. The construction of the grid is also time-consuming.

From examination of the results it can be seen that the maximum divergence of the water-surface elevation was 0.25 feet for the one-dimensional model and 0.65 feet for the two-dimensional model. The one-dimensional model was easier to calibrate and resulted in a closer match to the calibration data.

The theory, methodology and results of the one- and two- dimensional models have been compared for the given reach. Upon inspection of these topics it was found that for a standard reach, such as the one described, the benefits of the one- dimensional model far out weigh those of the two-dimensional model.

IX. SECONDARY STUDY

Based on these results of the Fivemile Creek study it was determined that for a standard reach the one-dimensional analysis is sufficiently accurate and more cost effective than the two- dimensional analysis. It was also concluded that more research should be done to provide correlations between roughness coefficient reductions and reach characteristics. The one area that is still gray is the performance of one- and two-dimensional analyses on velocity and flow distribution. In order to provide further investigation on these topics a second reach was investigated.

The second reach was selected to provide as many points of investigation as possible. The primary function of the reach was the investigation of flow distribution and velocity profiles. However, it also provided insight on multiple bridges on a skewed crossing and n roughness coefficient reduction for a basin with characteristics different from the Fivemile Creek reach.

Description of the Reach

The second reach selected for flood flow simulation was the U.S. Highway 11 crossing of Sucarnoochee River at Livingston, Alabama (Figure 23). The drainage area of this site was determined to be 607 square miles and the average streambed slope is

0004. This site is a prime candidate for a calibrated flood study. The USGS has operated a stream flow gage at this site since 1938. The gage has a stable stage-discharge relationship and an excellent high flow measurement. The measurement was taken during the1979 flood. The bridges have recently been replaced; however the models were based on the conditions reflective of the 1979 flood. At the time of the flood the hydraulic structures consisted of a 850 foot long main channel bridge and 165 foot long relief.

Hydrology

The measurement made in 1979 indicated that the total flow at the time of the measurement was 57,207 ft^3/s. Of this amount 48,700 ft^3/s (85%) passed through the main channel structure and 8,458 ft^3/s (15%) passed through the relief structure. In order to relate the measured value to a recurrence interval the flood frequency relation was developed for the gage for the 10-, 25-, 50-, 100-, 200-, and 500- year recurrence intervals. The flood peak discharges (Table 12) were computed using the rural regression equations for hydrologic area 4 and gaging station data collected for the site. Inspection of the flood frequency indicates that the measured event is approximately a 200-year recurrence interval flood.

Recurrence Interval (years)	Discharge (ft^3/s) Station Weighted	Gage Water-Surface Elevation (ft)
10	19,100	116.7
25	28,000	118.6
50	36,200	120.1
100	45,900	121.4
200	57,400	122.8
500	75,800	124.5

Table 12. Stage-discharge relationship for select recurrence intervals for the Sucarnoochee River reach

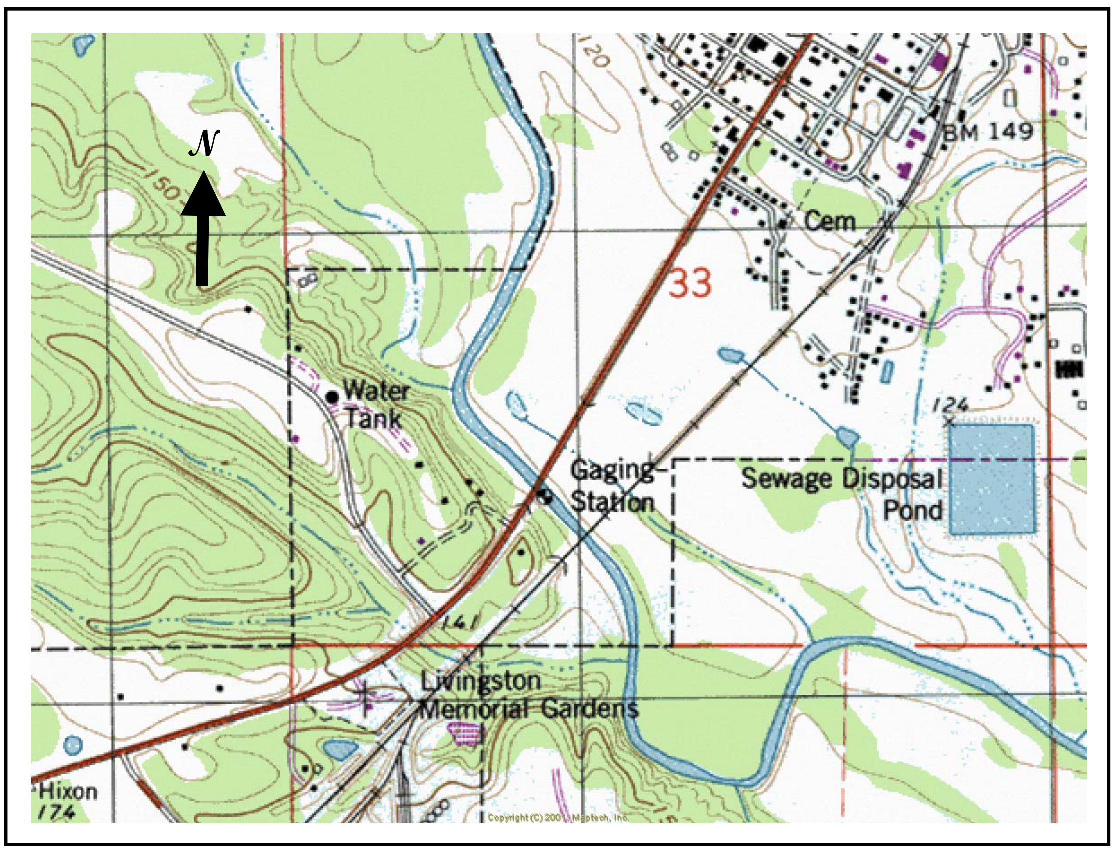

Figure 23. Location map of the Sucarnoochee River reach.

Hydraulics

The corresponding stage on the downstream side of the bridge is 122.8 feet. The lowest ceiling elevation for both bridges is 123.2 feet and the minimum weir elevation is 124.7 feet. This indicates that the reach will not be further complicated by pressure flow or weir flow. Examination of the location map shows that that approximately 550 feet downstream of the U.S. 11 crossing is Southern Railway. The Southern Railway crossing has two hydraulic structures that convey the flow. The headwater elevation for these bridges will serves as the downstream control for both models.

One-Dimensional Results

The one-dimensional model was constructed in the same manner as the Fivemile Creek study. The roughness values were selected through a site investigation and slightly adjusted to match the gage rating. The boundary condition was entered as a known water-surface elevation. The geometry was based on cross sections at the bridge and upstream and downstream of U.S. Highway 11.

Two-Dimensional Results

The two dimensional reach was approximately 3,500 feet long and consisted of 6,502 elements and 13,694 nodes. The grid was constructed and then refined using tools within SMS. The upstream extent of the reach extends just above the zone of contraction and the downstream extent if just upstream of the railroad crossing. The same default parameters described for the Fivemile Creek were used. The initial simulation was based on the roughness coefficients used in the calibration of the one-dimensional model. The roughness coefficients were then decreased incrementally and the changes in water-surface elevation were noted. Cross sections upstream and downstream of US 11 were imported and the elevations of the nodes were set through linear interpolation.

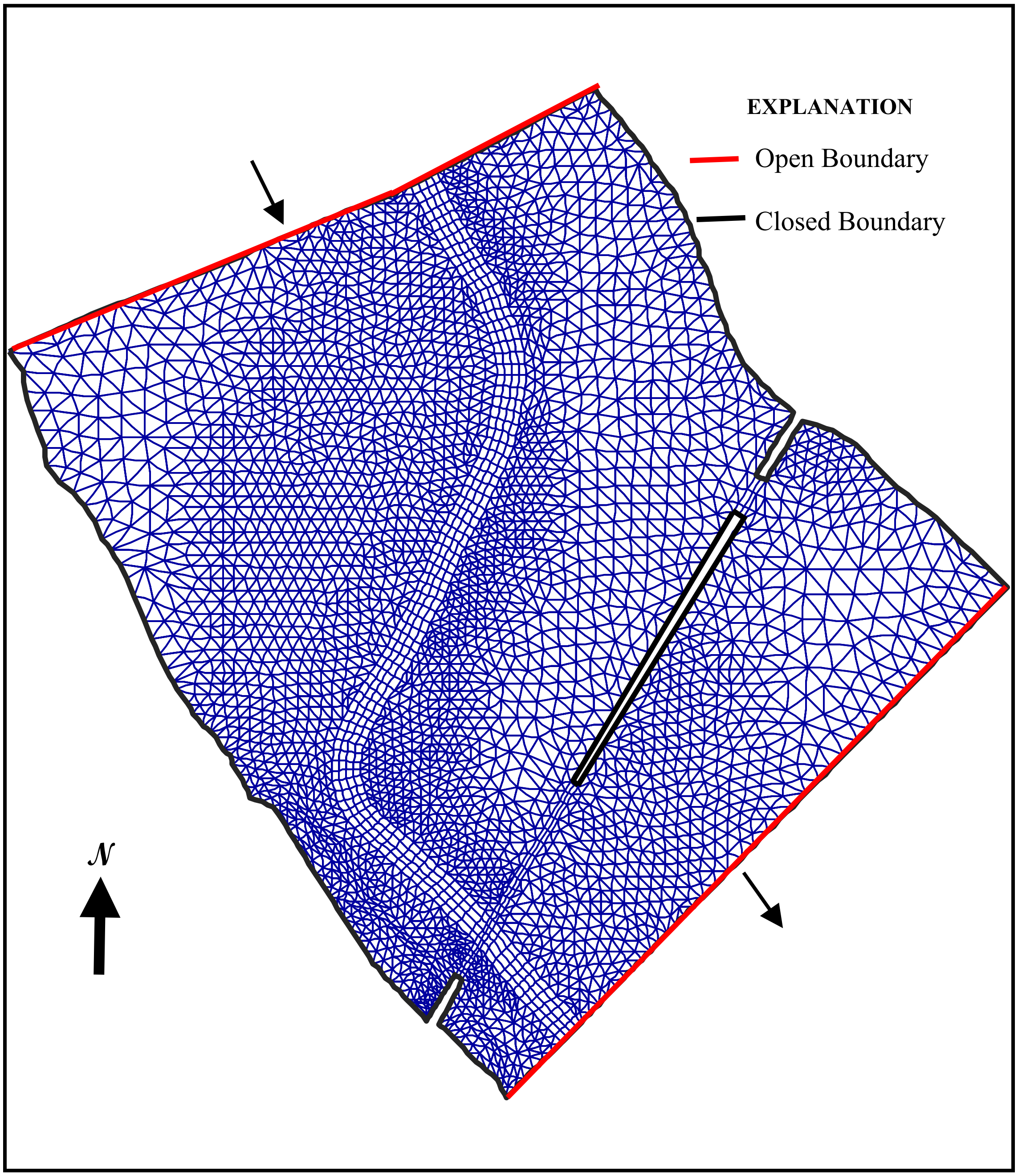

Figure 24. Finite-element grid used in flood flow simulations for the 1979 flood on the Sucarnoochee River.

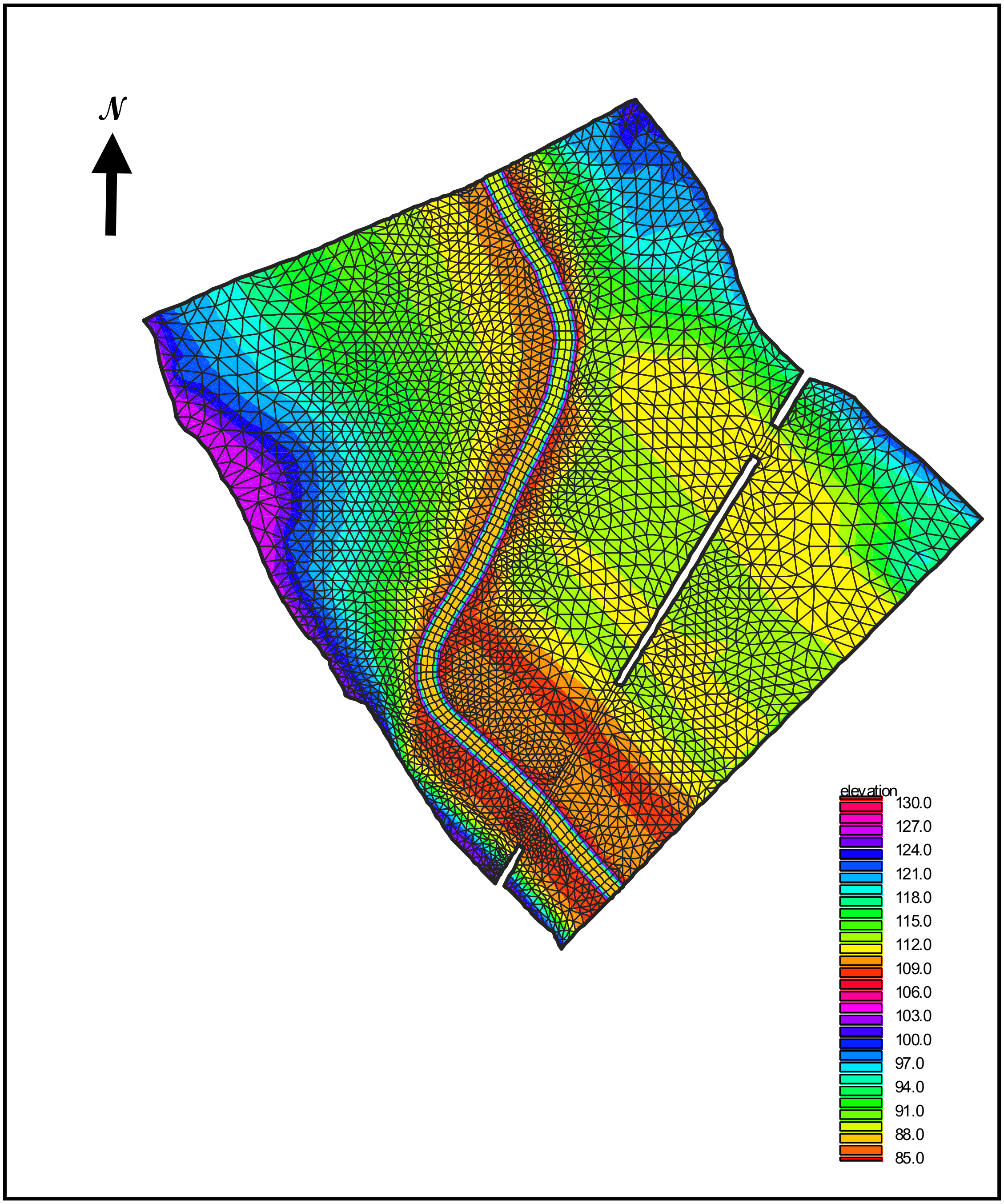

Figure 25. Land surface elevations for the Sucarnoochee River study reach

X. COMPARISON OF SECONDARY STUDY RESULTS

Flow Distribution and Velocity

It is a rarity to have high-water marks, flow distribution, and average velocity profiles. High-water marks are often used to indirectly calculate discharge for areas that do have a gage, has a stage only gage, or a gage that was destroyed by the flood. In some instances high-water marks are located solely for calibration data. Because of this it is rare to have high-water marks, flow distribution and velocities.

Since the water-surface elevation is the most common calibration data both models were constructed and calibrated to the known water-surface elevation at the downstream side of the bridge. The resulting flow distribution and velocities were then compared with the measured values. The measured discharge indicated 85 % of the flow passed through the main channel bridge and 15% through the relief. The one-dimensional model showed 88% of the flow in the main channel bridge and 12% in the relief. The two-dimensional model indicated 82% of the flow in the main channel and 18% in the relief. Both models were off from the measured value by 3%. Without the use of the calibration data the comparison of the results show that the flow distribution between the two models deviated by 6%. This would lead one to conclude that the one-dimensional model was off by 6% when in actuality it was only off by 3%. The results indicate that both models provide an accurate representation of the flow distribution. The measured velocities are an average tube velocity. This is also true for the velocities provided by the one-dimensional model. The two-dimensional model provides a depth-average point velocity. The maximum velocities for the nodes and midside nodes were documented from FESWMS-2DH. The corresponding tube velocity from the measurement and the HEC-RAS results were compared to this value. The results indicated that for both bridges FESWMS-2DH provided a closer estimated than HEC-RAS. However, in some areas both models deviated from the actual value significantly. To illustrate the output format of FESWMS-2DH and show the resulting velocity vectors Figures 26 and 27 are shown

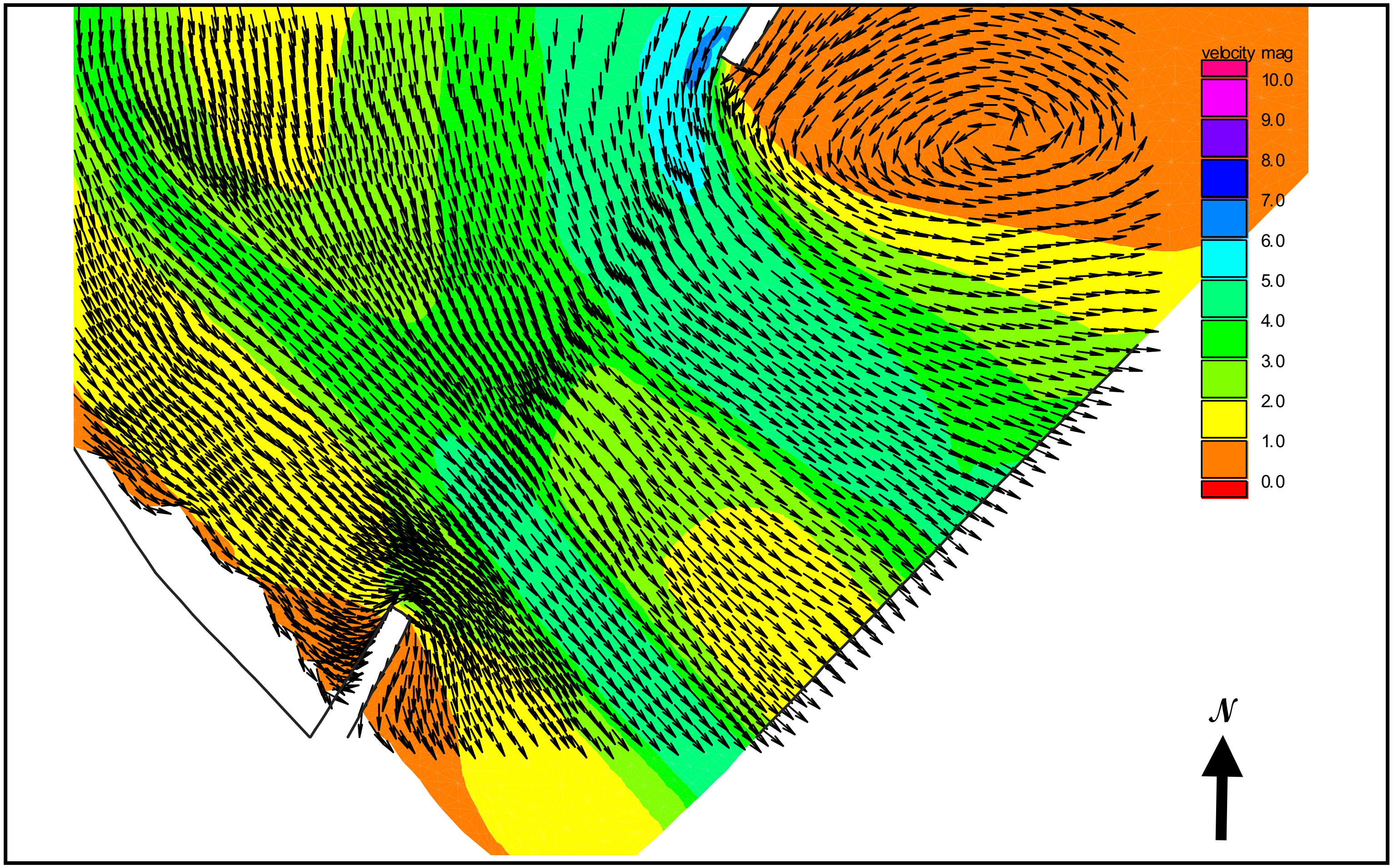

Figure 26. Computed velocity vectors for the main structure on the Sucarnoochee River for the April 1979 flood.

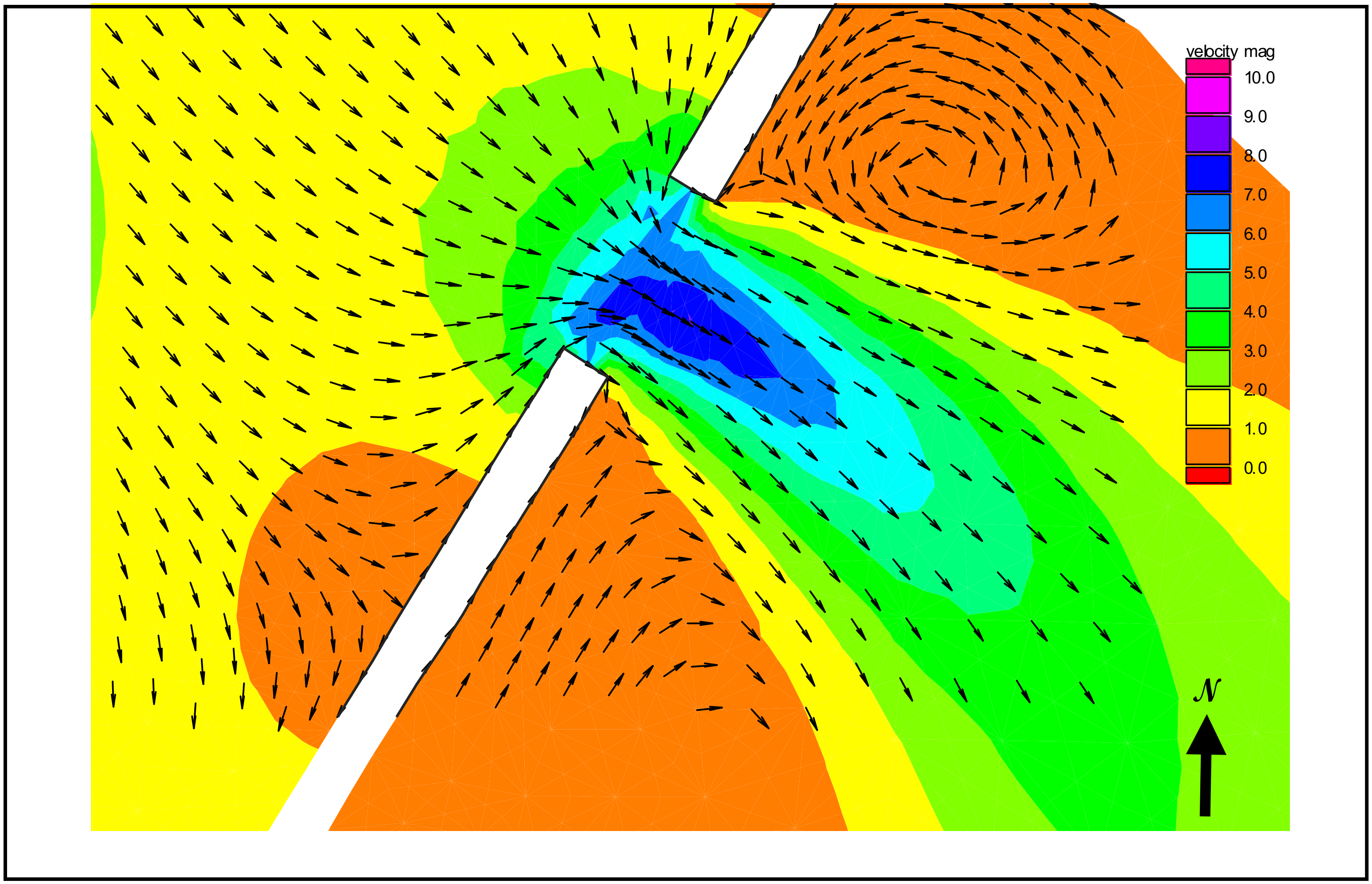

Figure 27. Computed velocity vectors for the relief structure on the Sucarnoochee River for the April 1979 flood.

Skewed Crossing

One of the advantages of this reach is the measured angle of attack. Inspection of the channel and road does not indicate there is a significant attack angle. However, the roadway crosses the flood plain at a slight skew. The discharge measurement recorded the skew varying from 12 to 25 degrees. This is an interesting topic in the comparison of one- and two- dimensional flow models. Skew is one of the major complexities that necessitates a two-dimensional model. It is also an area where many engineers question their judgment.

When the roadway crossing is skewed relative to the flood plain and multiple structures are present the modeling techniques are subjected to engineering judgment. Based on the flow path, for a skewed crossing one structure will be further downstream than the other. The typical one-dimensional models use conveyance and available flow area to determine a stagnation point between the structures. One-dimensional models can not divide flow between multiple structures on the same floodplain with different river stations. Typically, to account for the difference in river stations between the two structures, a river station at the half way point is selected and the openings are projected to this location. This is thought to provide a fairly accurate flow distribution, for minimal skew. The exact threshold of where this assumption becomes unrealistic is unknown. This is further complicated by another factor. The distribution is not only affected by the skew but also the spacing between the structures. This can be referenced to the flood plain width. This particular reach has a maximum skew value of 25 degrees for the 200-year event. The average flood plain width is approximately 2,900 feet, and the spacing between the structures is 1,200 feet. This is spacing to width ratio of 0.4. Based on the results of the flow distribution it can be concluded that an attack angle of 25 degree and spacing to width ratio of 0.4 is not great enough to warrant the use of a two-dimensional model.

Previous studies, not discussed in this publication, indicated that the flow distribution greatly varied for the one- and two- dimensional models for an average skew of 45 degrees. The study was not calibrated therefore the true distribution is unknown and no conclusion can be drawn as to how inaccurate the one-dimensional model was.

Based on these findings it can be assumed that the threshold is between 25 and 45 degrees. A safe assumption is that a roadway crossing of 30 degrees or greater necessitates a two-dimensional model for accurate flow distribution. The spacing to width ratio deserves further investigation.

Manning's Roughness Coefficient Reduction

Previous chapters discuss the variance between one- and two-dimensional roughness coefficients. This difference is primarily due to the different methods of accounting for bed shear stress and turbulence. These variables are directly included in one-dimensional roughness coefficients. A common way of selecting the appropriate roughness coefficient is through a roughness verification study or references such as Chow (1959). These calculated values directly account for the effects of turbulence and bottom shear stresses. The depth-averaged flow equations directly account for bottom shear stresses, surface shear stresses, and stresses caused by turbulence. Manning's roughness coefficient is used to determine the bottom shear stresses. The bottom shear stress is a function of the depth averaged velocity, density, bed slope in the x and y directions and the variable (c_f). The value of (c_f) is a function of Manning's roughness or the Chezy coefficient. The variables used to compute (c_f) can be seen in Equations 20 and 21.

Froehlich documents this in the users' manual "….[A]ssumed one-dimensional flow, implicitly accounts for the effects of turbulence and deviation from uniform velocity in a cross section. Because the depth-averaged flow equations directly account for horizontal variations of velocity and the effects of turbulence, values of (c_f) computed using coefficients based on one-dimensional flow assumptions may be slightly greater than necessary"(Froehlich 1989).

Based this it can be concluded that the reduction of the roughness coefficient will depend on the slope, velocity, and roughness value. Equation 21 shows that lower values of roughness will produce lower values for (c_f). This would indicate that within the same basin lower values of roughness would need to be reduced by a smaller factor. This was observed within the Fivemile Creek study reach. The roughness values for the channel were lower than the average floodplain roughness. To simulate the May 2003 flood the channel roughness values were reduced by 5% and the floodplain values were reduced by an average of 20%. These values are interconnected. To determine the actual reduction in channel values low flow or bank full calibration data would be needed. Since this is not available, the other two areas of interest, slope and velocity were further investigated.

Sucarnoochee River was chosen for the insight it would provide in this matter. The previous study, Fivemile Creek, had a bed slope of 0.0035. The variance in roughness for this reach ranged from 5 to 20 percent. Similar analyses were performed for Sucarnoochee River. The approximate bed slope was calculated to be 0.0004. The percent reduction used to calibrate the reach to the water surface elevation at the bridge was 5%. This provides insight into the change of roughness coefficient reduction with slope.

Sensitivity analysis was done to further document the changes in the water-surface elevation with the associated reduction in roughness values. The roughness coefficients were reduced in increments of five percent and the change in water-surface elevation noted in four locations. The four locations were chosen at the main and relief bridges, the upstream boundary and the mid point of the reach. The water-surface was taken at the left and right edges of water and averaged, for each location. The maximum change was seen at the upstream boundary and the minimum change was seen at the relief structure. Table 13 shows the change in water-surface based on the value documented prior to the reduction of roughness.

Location	Water-Surface change for 5% Reduction (ft)	Water-Surface change for 10% Reduction (ft)	Water-Surface change for 20% Reduction (ft)
Relief Bridge	0.006	0.011	0.0170
Upstream Boundary	0.115	0.229	0.447

Table 13. Correlation between water-surface elevation and Manning's roughness coefficients for the Sucarnoochee River reach.

Inspection of the table indicates that the changes of water-surface elevation in the bridge are not noteworthy. This is a great indicator that for a flat slope the effects due to turbulence are negligible and the one-dimensional roughness values would suffice.

However, it should also be noted that many factors are considered when determining roughness and how it effects the water-surface elevation. These factors should not be over looked and the decision should not be based on slope alone.

The other area of interest in the two-dimensional, depth-averaged flow equations is turbulence. The changes in water-surface elevation were noted as the base kinematic eddy viscosity was adjusted. It was found that large values of base kinematic eddy viscosity had an effect on the upstream stage. Variations between reasonable values had little effect on the stage of the reach. It should be noted that turbulence is a function of the eddy viscosity and velocity gradients. The variance of the base kinematic eddy viscosity indicated that the influences on the water-surface elevation are less than that of Manning's roughness coefficient. This confirms that the variance of roughness should be used in calibration and the base kinematic eddy viscosity should be held constant at a reasonable value.

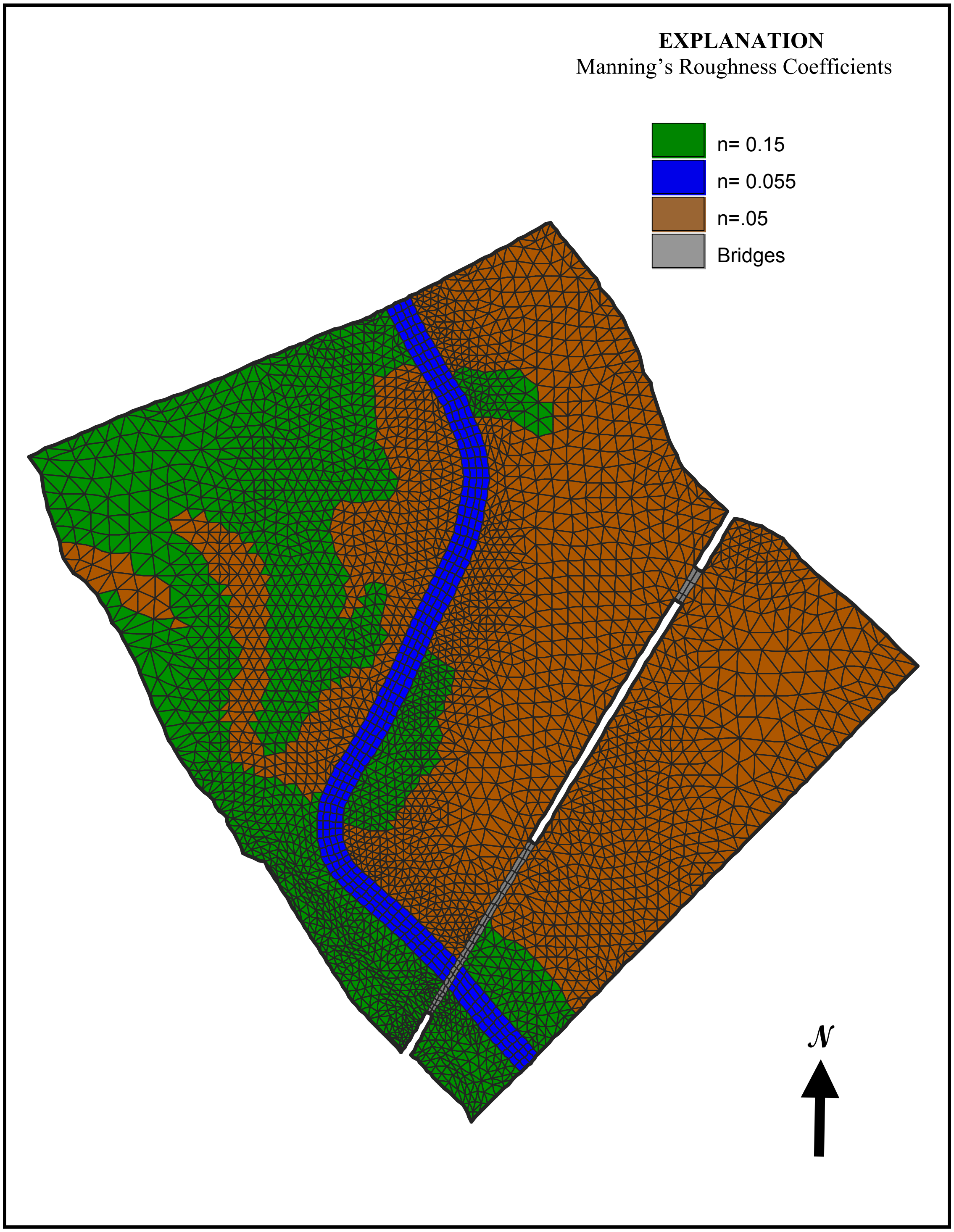

Figure 28. Manning's roughness coefficients used for the flood flow simulation of Sucarnoochee River.

XI. CONCLUSION AND ENDING REMARKS

The development, theory, and results of a one-dimensional step-backwater model and a two-dimensional finite-element surface-water model were examined. The creation, execution and steps taken toward calibration were documented. The theory as documented by the respective users' manuals was explained and elaborated upon where necessary. The results of the models were also examined for effects of superelevated flow, variance in Manning's roughness coefficient and base kinematic eddy viscosity, flow distribution, and velocity profiles.

The first reach examined was Fivemile Creek. The results showed that for a standard reach the cost to benefits ratio of a one-dimensional model out weighed that of a two-dimensional model. The process of construction, execution, and calibration for the two-dimensional model was time consuming and did not yield results closer to the measured values. It was also determined for a reach with similar characteristics the average roughness reduction is approximately 20% to transition between a calibrated one- and two-dimensional models.

The second reach, Sucarnoochee River, was examined for comparison to the first reach and the inspection of flow distribution and velocity profiles. This reach was chosen because it has a much flatter slope than the Fivemile Creek reach. The average roughness reduction determined for this reach was 5%. Inspection of the two-dimensional, depth-averaged equations showed the effects of turbulence and bottom shear stresses are a function of the slope, base kinematic eddy viscosity, and the roughness value. The calibration data indicated that reaches with a flatter slope should be reduced by a smaller percentage when transitioning from one-dimensional to two-dimensional flow. The effects of varying the base kinematic eddy viscosity were also inspected. The results indicated that for values of base kinematic eddy viscosity between 20 ft^2/s and 40 ft^2/s the water-surface elevation was not greatly affected. It was also determined that for values of base kinematic eddy viscosity greater than 40 ft^2/s the upstream stage was affected the most. This is the primary reason the Manning's roughness coefficient is adjusted to calibrate to a known water-surface elevation instead of the base kinematic eddy viscosity.

Since it was determined that one-dimensional models are sufficient for standard reaches the effects of a non-standard reach were examined. The general restrictions on a one- dimensional model are skewed roadway crossings, large rivers with incised channels, four or more bridges on a crossing, and superelevated flow. Of these stipulations the one that is vague is skewed crossings. The one-dimensional model can be adjusted to account for skewed crossings; however there is a point where this no longer provides accurate results. The variables to be considered are the angel of attack (skew), and the bridge spacing to flood plain width ratio. Sucarnoochee River was chosen to provide insight on this subject as well. A measurement of the 1979 flood, on Sucarnoochee River, indicated that the angle of attack for the 200-year flood event ranged from 15 to 20 degrees. The spacing to width ratio for this reach was 0.4. It was determined from inspection of the flow distribution between the main and relief structures that these two values are not great enough to merit the use of a two-dimensional model. This in conjunction with previous studies, not documented, led the author to the conclusion that multiple structures that cross the flood plain at an average angle greater then 30 degrees should be analyzed with a two-dimensional model.

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APPENDIX


Figure 29. Section A, outlet of the box culvert at the L and N Railroad.


Figure 30. Section B, looking downstream.

Figure 31. Alabama Power Company Bridge, looking upstream.


Figure 32. Section C, looking upstream.


Figure 33. Section D, looking downstream.


Figure 34. Railroad Bridge, looking upstream.


Figure 35. Section E, looking downstream.

Figure 36. Section F, looking west.

Figure 37. Section H, looking downstream.

Figure 38. Highway 79 Bridge, looking upstream.

Figure 39. Section I, looking downstream.

Figure 40. Section J, looking upstream.

Figure 41. Section K, looking downstream.

Type of Development	Description	Impervious Cover %
Urban Core	Central Business District	88.0
Arterial Commercial	Commercial Strip Development	68.8
Office- Shop	Regional Shopping	48.9
Urban Node	Neighborhood Shopping Areas	47.1
Industrial- High Density	Heavy Intercity Industry	56.5
Industrial -Med Density	Light Industry	44.0
Industrial -Low Density	Modern Industry Park	23.8
Residential- High Density	Concentrated Multi-Family Units	34.0
Residential-Med Density	Density of 3-6 units per acre	21.6
Residential-Low Density	Density 1-2 units per acre	15.0
Exurban	Density of less than 1 unit per acre	1.3
		100

Table 14. Weighting factors used to determine degree of urbanization.

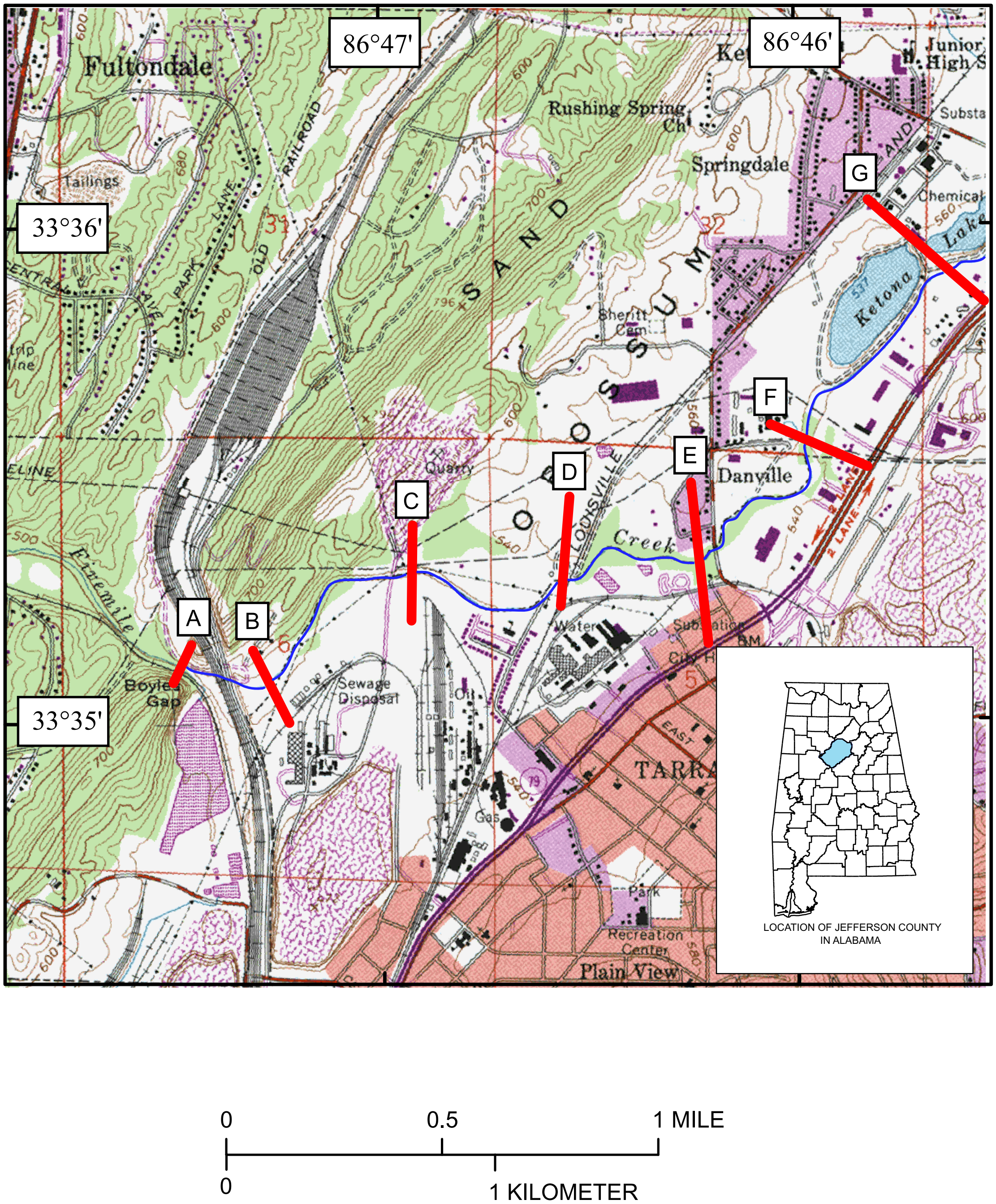

Figure 42. Fivemile Creek study reach approximate location of cross sections A through G.

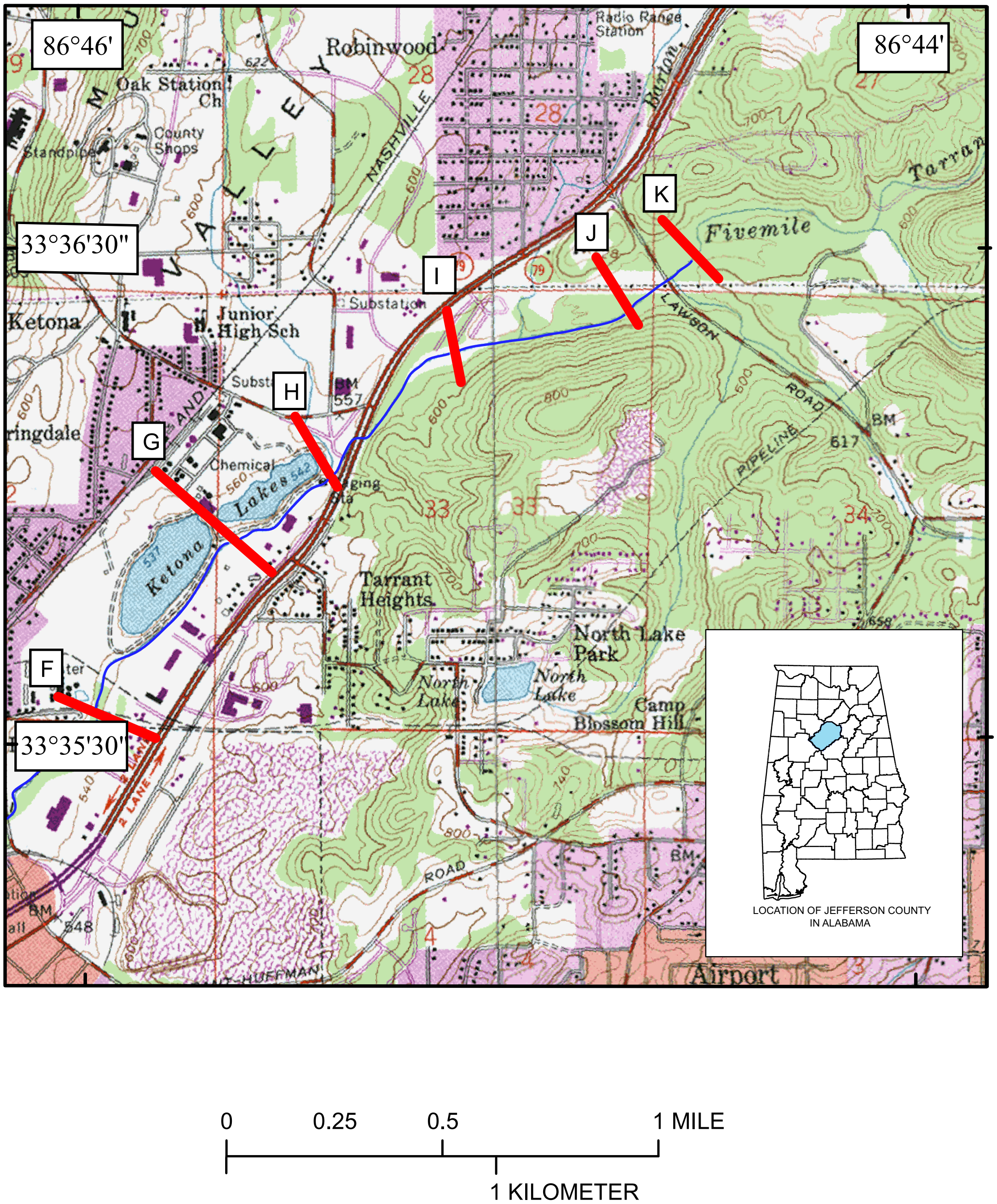

Figure 43. Fivemile Creek study reach approximate location of cross sections F through K.

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                              LAWRD.OP
        COMPUTATION OF DISCHARGE FOR CONTRACTED OPENING
Contracted opening measurement of: FIVEMILE CREEK AT LAWSON ROAD
Gage height:   11.50 feet

Discharge:  14078 cubic feet per second
Drainage area:  18.6 MI SQ

Unit discharge: 470.8 feet
-------------------------------------------------------------------------------------
-
AVE WATER-SURAFCE ELEVS
   US      DS   DELTA H   C   ALPHA 1    K1       K3      A1    A3     L    LW
-------------------------------------------------------------------------------------
-
 15.59  11.50   4.09   0.83   2.04     235352   79803    3683   823  300   40
------------------------------------------------------------
8.02 X C X A3 X (DELTAH^.5) = 8.02 X .83 X 823 X 2.021138 =11072.6
------------------------------------------------------------
ALPHA1 X (C^2) X (A3/A1)^2 = 2.04 X .6889 X 4.993399E-02 =    0.070
------------------------------------------------------------
2g X (C^2) X ((A3/K3)^2) = 64.4 X .6889 X 1.063558E-04 =     0.005
------------------------------------------------------------
(L + (Lw X (K3/K1)=  0 X 0 X .3390794 =   141.724
------------------------------------------------------------
Q=  11073 /  1.264 =   8758
-----------------------------------------
DISCHARGE THRU BRIDGE=   8758 FT3/S
COMPUTED FLOW OVER ROAD=   5320 FT3/S
TOTAL COMPUTED DISCHARGE=  14078 FT3/S
RUNOFF =     757 FT3/S PER MI SQ
PERCENT FLOW OVER ROAD= 3.7791
VEL AT SECTION 3= 10.64 FT/SEC
VEL AT SECTION 1=  2.38 FT/SEC
----------------------------------------------------------------
                         JOB DONE
□

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Figure 44. Results of the Indirect Calculations at Lawson Road by USGS.

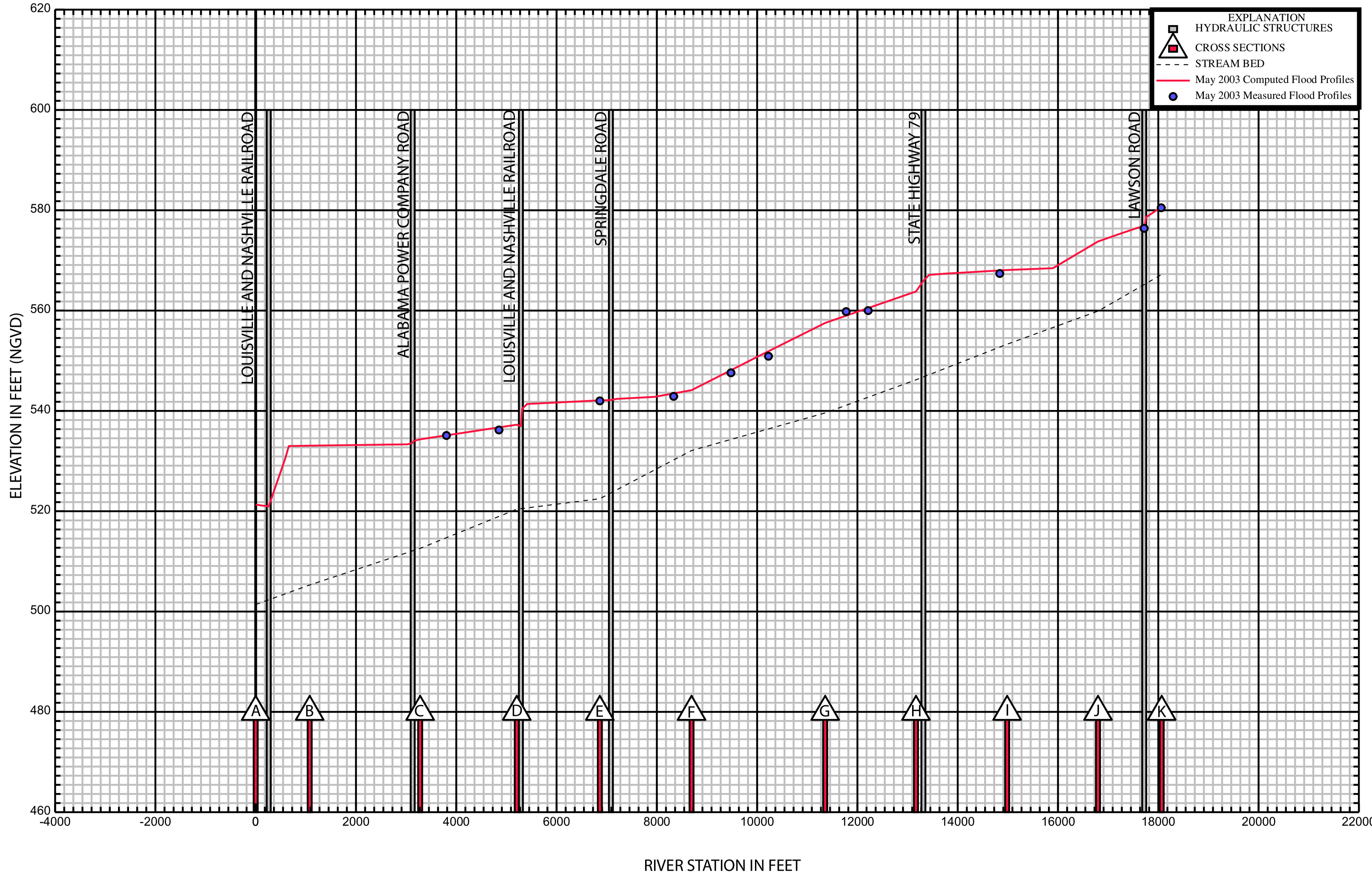

FIGURE 45. Comparison of computed and actual flood profiles for the May 2003 flood.