

Investigating Instabilities with HEC-RAS Unsteady Flow Modeling for
Regulated Rivers at Low Flow Stages

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ABSTRACT

Dams are used throughout the United States for generation of electricity and flood control. The Tennessee River Valley's regulated river system extends through Tennessee and parts of Kentucky, Mississippi, Alabama, Georgia, North Carolina, South Carolina, and Virginia. Of the dams in this area, 49 are managed by the Tennessee Valley Authority (TVA). The TVA forecasts and monitors the activities related to these dams and others in the area on an around-the-clock basis to ensure maximum generation potential is attained and to prevent flooding of the surrounding cities. To best forecast hydrology for the regulated river system, the TVA chose to upgrade the forecasting center to include HEC-RAS models for high and low flow simulations. These simulations can better guide TVA engineers to the best route when releasing water from the dam in the case of an emergency flooding situation or on a daily basis. Building a HEC-RAS model for the TVA reservoir system is a large undertaking and model stability issues arise from different possible causes. In this paper, the best methods of stabilizing modeled reaches on a regulated river system during low flows were compiled in logic flow charts, which can be of general assistance to modelers when stabilizing HEC-RAS simulations for regulated river systems. The flow charts were created and explained, with multiple options for stability described and analyzed throughout the creation process. They were based on results where stability issues were regularly due to cross sectional spacing and the addition of base flow through the upstream boundary and lateral inflows. Other potential methods were also considered for applicability in a model such as this, including the addition of pilot channels and increasing the theta weighting factor. Using the flow charts created, HEC-RAS modelers should gain a better understanding of stability issues in a river system and what causes these problems.

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CHAPTER I

INTRODUCTION AND GENERAL INFORMATION

Introduction

Large regulated river systems need improved hydraulic models in order to better simulate river conditions at different points in time. Water engineers and managers have the option of using many different simulation models to determine factors such as flow, velocity, and inundated area of a stream during a flood. Some of the available models are two-dimensional, such as TELEMAC-2D or River2D, while others are one-dimensional, for example, HEC-RAS. The Hydrologic Engineering Center's River Analysis System, or HEC-RAS, is a widely-used one-dimensional method for studying stream reaches. HEC-RAS is used within multiple water management groups throughout the United States for dam failure analyses within regulated watersheds. These analyses are then used to determine the effects that a failing dam can have on nearby cities and their populations, and can allow for ample time to warn authorities if a dam break scenario seems likely from overtopping due to a large flood. HEC-RAS also allows river forecasters to use recorded data to model river reaches for daily flow approximations.

As mentioned previously, HEC-RAS can be used for a variety of types of river simulation projects, and for steady, unsteady, and mixed flow regimes. In the past, the program has been used for dam failure analyses, flood mapping, completing flood frequency studies, and to simulate everyday flows through a reach. Studying typical daily flow (low flow) conditions in a river allows a user to better understand typical elevations and flow patterns within a stream. HEC-RAS also has the capability to be used with other software and programs, including the HEC Data Storage System (HEC-DSS) and GIS applications, among others.

Hicks and Peacock (2005) analyzed the suitability of using HEC-RAS for a flood forecast and determined it to be appropriate in this application. In the case of a dam failure, modelers demonstrate the flows that could be witnessed if the dam failed. Much documentation is available about dam failure, but these scenarios are very different from low flow situations due to the massive amounts of water involved. Thus, low flow scenarios and dam failure to have very different objectives, parameters, and results.

This study is based on the HEC-RAS modeling system applied to a large regulated river system in the upper Tennessee Valley. Research has been completed to determine which of the numerous options of one- and two-dimensional models are best for river evaluations. Based on multiple studies, HEC-RAS was found to be the best for this project due to the fact that it is a free software, is easy to navigate even for users with minimal modeling experience, and can run quickly for real-time forecasting (Castellarin et al. 2009). One major application of HEC-RAS being used currently is for the Ohio River. The majority of the Ohio River has been modeled using HEC-RAS through a collaboration of the Ohio River Forecast Center, U.S. National Weather Service, U.S. Army Corp of Engineers, Great Lakes and Ohio River Division, Water Management Division. Not much information is offered in the documentation available online about the model; however, the Tennessee Valley Authority (TVA) is creating a similar model that is in progress now. The TVA's large river model will include dam break scenarios as well as typical daily flow analyses, hereby referred to as low flow simulations. The HEC-RAS system allows modelers to promptly simulate flows over long periods of time. In this study, approximately fifteen years of historical data were used for calibration and stabilization of the models. The models were created to be used in a river forecasting center that uses real-time analyses of the river system to make decisions about the amount of water held in the reservoirs on a 6-hour basis, and to determine the elevations at which a river reach can range for different magnitudes of storms.

The HEC-RAS model proves unreliable in some low flow conditions, particularly those reaches that are unsteady and associated with steep slopes. Multiple modelers have come across similar issues to those found in this study when stabilizing HEC-RAS models. All models use numerical analysis, which can lead to inaccuracies and instability in simulations when incorrect input data is used. Gary Brunner, senior hydraulic engineer for the Hydraulic Engineering Center, created an assistance document for stability issues when modeling unsteady flow in HEC-RAS (2014). The document lists twelve factors for a RAS modeler to check in the case of a failing model. The list includes cross section spacing, choice of time step, theta weighting factor, low flow conditions, and steepness in reaches, among others. The primary aspects causing issues are explained in Brunner's document. For example, wide cross sectional spacing causes the simulation to have dampening of the flood wave, an incorrect time step selection can cause

diffusion and instability, low flows can cause pool-riffle sequences, and high slopes can cause the model to experience increased velocities and large depth changes (Brunner 2014). The aforementioned common issues are considered the most important due to the fact that many of the concerns are applicable to the model being undertaken for this study due to the slope, width and other reach variability throughout the river system. Due to the large amount of potential issues that can cause instability in a model, a methodology is needed for the most effective way to go about stabilizing a failing model.

The objective of this study is to investigate potential causes for instabilities with HEC-RAS unsteady flow modeling in regulated rivers at low flow stage. In addition, an outcome of this study will document protocol guidance for successful flow modeling of different types of river reaches (large, less complex geometry, with shallower slopes to small, more steep and complex geometry) for daily use in forecasting scenarios within the TVA reservoir system. Guidance will be in the form of flow charts leading a modeler from one step to the next as it walks the user through setting up a stable model. The flow charts will be dependent upon outputs given by HEC-RAS models that are crashing or running incorrectly.

CHAPTER II

LITERATURE REVIEW

The HEC-RAS model was created using standard water engineering concepts. River system models have been created using HEC-RAS and other programs to estimate and understand occurrences throughout channels. HEC-RAS has a user's manual (2010) describing the equations used for each step of the results process. Understanding the procedure used for calculating the results, which include water surface levels, velocities, and flows at each river mile, is important when discovering the instabilities in a model. This manual was studied carefully during the creation of stable unsteady models for this study.

HEC-RAS as a Modeling Tool Selection

HEC-RAS is a very widely used modeling program for river managers and researchers for learning about reaches. The flow routing methods use the principles of continuity and momentum, which most water engineers are familiar with (User's Manual 2010). As mentioned previously, HEC-RAS is a free software download and models one-dimensional flow. Although there are two-dimensional models available, HEC-RAS continues to be widely used due to its accuracy in modeling natural streams and due to its negligible cost (Castellarin et al. 2009).

HEC-RAS was compared with two other models in Horritt and Bates (2002). The two other models in the comparison were LISFLOOD-FP and TELEMAC-2D, which include two-dimensional modeling capabilities. Overall, the authors found that HEC-RAS was the overall best modeling system between the three options. HEC-RAS and TELEMAC-2D were also found to best map the flooded areas.

Current Uses

HEC-RAS has been used for many types of analyses to date, including dam break scenarios, floodplain delineation, flood forecasting, and flow routing. Dam break scenarios are arguably the most important analyses HEC-RAS can be used for, as they can predict and prevent major disasters from occurring. Floodplain delineation is useful to approximate water levels and extent in a flooding situation. Floodplain delineations are often used when deciding upon land use in areas and for determining the areas in which there is water in a drainage basin and how to

distribute water throughout the basin (Yang et al. 2006). Flood forecasting is used to determine the elevations at which a river reach can range for different magnitudes of storms.

As mentioned previously, the Ohio River has a large HEC-RAS model currently being used for simulating and forecasting scenarios along it. Many other HEC-RAS models have been created to run floodplain delineation, dam break, and forecasting simulations, among others. Notable other projects using HEC-RAS include multiple case studies mentioned below. Knebl et al. (2005) developed a framework for regional scale flood modeling based on an event in the San Antonio River Basin which also integrates multiple other programs. Tahmasbinejad et al. (2012) created a rainfall-runoff model of the Karun River using HEC-RAS and other programs to provide a tool for hydrological predictions of flooding on a local scale. Drake et al. (2010) used HEC-RAS for a temperature model to estimate groundwater contributions and understand the proficiencies for temperature in HEC-RAS to approximate groundwater discharge. As is visible from the different types of studies available through HEC-RAS, it is a widely used tool for hydrologic modeling, applicable to many different areas of study.

Complications with Model Stability

The most important information found when researching common model stability issues is compiled in this section. These issues were the basis for the research completed in this HEC-RAS project in which model calibration protocols were assessed.

Geometry

When utilizing hydraulic models, some users have experienced that incorrect topography of rivers can create small errors that overall create troublesome model instabilities and inaccuracies (Pappenberger et al. 2004). Others have noted that as more complexity is added into a model, the parameters will have a smaller effect on the simulation, which overall creates a more accurate model (Fewtrell et al. 2011). In Thomas and Williams (2007), it is noted that over-extended cross sections can cause model instabilities due to critical depth calculations.

Spacing was a topic Brunner (2014) mentioned in his article about HEC-RAS model stability. Brunner's documentation notes the instability that can be caused by cross sectional spacing that is too wide or too close. He explains that when cross sections are too far apart, numerical diffusion can become an issue, while cross sections that are too close can result in

over-steepened flood waves. The equations he mentions for use in cross sectional spacing are Dr. Fread's (1988 and 1993) and P.G. Samuels's (1989). These equations are for maximum spacing and are displayed in Equations 1.1 and 1.2.

$$\text{Fread: } \Delta x \leq cT_r/20 \quad (1.1)$$

$$\text{Samuels: } \Delta x \leq 0.20D/S_o \quad (1.2)$$

Where Δx is cross sectional spacing in feet, T_r is the time of the rise of the main flood wave in seconds, c is the wave speed of the flood wave in feet per second, D is average bankfull depth of the channel in feet, and S_o is average bed slope. A concern arose with these equations that they are specified in this document to be used for dam break analyses, and the low flow model differs enough that the equations may not apply. Other authors have also researched the optimal cross sectional spacing for one-dimensional models. One article notes that the cross sectional spacing equations best for model accuracy are those by P.G. Samuels, and the conclusion was drawn that too many cross sections can lend greater inaccuracies to the model (Castellarin et al. 2009). The equations by P.G. Samuels, however, were tested in gentle-sloped rivers, which could lead to misguidance in steep river reaches. Further, the authors note that real-time forecasting of floods can be achieved simply with a small number of cross sections.

Another option to improve cross sectional spacing is to use the more automated tools available in HEC-RAS. The HEC-RAS User's Manual (2010) states that the best cross sectional interpolation option for most reaches is the "Within a Reach" tool, which interpolates cross sections at a specified distance throughout the reach. Another option available allows the modeler to choose an area between two cross sections to add as many cross sections in that reach that they feel is necessary. Brunner (2014) notes that in unusually steep reaches, interpolation of cross sections may be needed for better model results. Although none of the reaches in this study is considered "steep," the interpolated cross sections were an option considered as part of the cross-sectional addition model stability method.

Unsteady Flow

An implicit finite difference scheme referred to as the box scheme is used for the unsteady flow determination in HEC-RAS (Hydraulic Reference Manual 2010).

The User's Manual (2010) specifies multiple aspects of the flow calculations involved when determining the results of the run that could cause the model to be unstable. The manual states

“In practice, other factors may also contribute to the non-stability of the solution scheme. These factors include dramatic changes in channel cross-sectional properties, abrupt changes in channel slope, characteristics of the flood wave itself, and complex hydraulic structures.”

The HEC-RAS model uses the Saint-Venant equations for flows, which are the best choice for dam break scenarios (Ackerman and Brunner 2005). These equations are based on differential equations. Since the simulation being completed is a low flow model, the Saint-Venant equations have the potential to cause some inaccuracy.

Roughness

Flow-resistance formulas have been visited by multiple researchers studying hydraulics. Roughness in steep reaches is very different than the roughness in flat reaches due to the velocity of flow and slope of the channel. Aberle and Smart (2003) determined that there was no flow-resistance formula for mountainous stream mean flow velocity. They studied what the most accurate methods would be to determine this value. The authors found that the shear bed stress concept, which is dependent on uniform flow, applies only to flatter reaches. The roughness in steeper and mountainous areas has been calculated using the log-law and power law resistance equations in the past, and the authors determined that these methods may not be accurate. They found a better estimation process for the roughness parameter to be using the standard deviation of the bed elevations used in their study.

An analysis of the flow resistance notes multiple options for equations that can be used. The article includes Hey's (1979) formula for gravel beds, Bathurst's (1978, 1985, 1986) equation which related roughness to relative submergence and his later equation that is best for average to high roughness values, Thompson and Campbell's (1979) formula, which was created for a boulder-bed spillway, and the formula created by Griffith which is the most applicable to steep reaches (Aguirre-Pe and Fuentes 1990). All of the equations are dependent on the type of

reach and bed material, which further displays the uncertainty in roughness parameters and the instability that could be caused in using only one equation in hydraulic modeling.

Manning's roughness coefficient is primarily used in HEC-RAS as the friction factor affecting stream reaches. The HEC-RAS Manual features guidance on picking the most accurate value for Manning's n ; however, the value is always estimated and open to much uncertainty due to the lack of a method to find the exact value of the parameter. In Ferguson (2010) the validity of the Manning equation for use in predictions in models is analyzed. Ferguson argues that since the roughness decreases as flow increases, the equation typically underestimates flow resistance. He concludes that the Manning equation is valid for deep, flat rivers, but could lead to large inaccuracies in shallow-flowing rivers with gravel or boulders.

Manning's roughness coefficients that are too low in the HEC-RAS model can cause many issues that will lead to instability. Some of these problems include shallow water depth, faster velocities, and supercritical flows (Brunner 2014). These issues are especially a concern in steep reaches, as the roughness values are commonly underestimated here. Brunner suggests using Dr. Robert Jarrett's equation for high gradient streams (Eq. 1.6).

$$n = 0.39 S_f^{0.38} R^{-0.16} \quad (1.6)$$

Where R is the hydraulic radius in feet and S_f is the slope of the energy grade line.

The roughness coefficient was determined to have small effects on the flow hydrograph results when varied by ± 0.005 (Hicks and Peacock 2005). Although roughness was not used for stabilization in this study, it is a helpful parameter, as it was used for calibration of models using observed and simulated flows.

Low Flow

Hydrological models normally are not fit to simulate or predict low flow situations (Staudinger et al. 2011). Low flow simulations in this instance refer to daily flows or flows that are lower than typical daily, rather than peak flow and flooding situations. Most models are built more specifically for peak flow simulations, since those are of the most concern to modelers. The authors of the literature studied suggest use of different models for different seasons and flow levels. Winter low flows and summer low flows are typically of different magnitudes, which can create model inaccuracies (Staudinger et al. 2011).

Pool and riffle sequences and shallow depths can cause low flow conditions to have significant model instability (Brunner 2014). To fix the instability, Brunner proposes to add base flow or create a pilot channel. Pilot channels give cross sections greater depth without extending the area of flow. The HEC-RAS User's Manual (2010) suggests a one foot width be used for the pilot channel. The small amount of extra base flow can be used to combat the instabilities that occur when a reach dries out due to steepness and lack of water in the area. For steep streams with low flows, Brunner mentions attempting to increase roughness coefficients, base flow, or running the model as a mixed flow regime for more model stability.

Time Step

Choosing the appropriate time step is dependent on the capability to determine the hydrograph shape, although in some models, authors have found the time step to have small effects on the overall results (Hicks and Peacock 2005). Brunner (2014) acknowledges methods for time step selection. Time steps that are too large can cause model instability, while a time step that is too small will create a long computation time. Brunner suggests the Courant condition as a guideline for time step selection, as shown in Equations 1.3 and 1.4.

$$C_r = V_w (\Delta t / \Delta x) \leq 1.0 \quad (1.3)$$

$$\Delta t \leq \Delta x / V_w \quad (1.4)$$

Where C_r is the Courant number, V_w is flood wave velocity, Δt is computational time step, and Δx is distance between cross sections. Further, Brunner notes that for medium or large rivers, the Courant condition may yield results that are not accurate, and a larger time step could be used in these cases. The equation given for medium to large rivers is Equation 1.5.

$$\Delta t \leq T_r / 20 \quad (1.5)$$

Where T_r is time of rise of the flood wave.

Theta Weighting Factor

When theta weighting factor for numerical analysis stability is equal to one, the RAS model is fully implicit and highly stable (Hicks and Peacock 2005). A weighting factor of 0.6 to 1.0 is suggested, but 1.0 is typically used due to the higher level of stability it can produce (Hicks and Peacock 2005). Other author had similar conclusions about the theta weighting factor, stating that using a value of 1.0 is the best option for the parameter (Pappenberger et al. 2005).

Brunner (2014) recognizes the theta weighting factor as another medium through which instability could be found. He notes that a theta value of 1.0 will create the most stability, while a theta value of 0.6 will provide the higher accuracy. Brunner suggests using a value of 1.0 to stabilize the model and then lowering the value of theta until it is as close to 0.6 as possible while keeping a stable model.

Steepness in Streams

For steep streams, Brunner (2014) mentions attempting to increase roughness coefficients, base flow, or running the model as a mixed flow regime for more model stability. Mixed flow regimes incorporate super- and subcritical flows to allow the model to choose which regime it needs to run in. According to the stream classification analysis provided by Rosgen (1994), a very steep stream is above 10 percent slope, a steep stream is 4 to 10 percent, and gentle gradient streams have a slope lower than 2 percent. Sturm (2010) defines a steeply sloped reach as one that is approximately five to six percent using gradually varied flow equations. Above this slope, the momentum balance assumptions are no longer satisfied.

Output Files

The HEC-RAS User's Manual (2010) includes a section specific to troubleshooting. One of the key troubleshooting processes suggested is the use of the Errors, Warnings, and Notes Summary that is available after running the model. The summary is a detailed set of tables that can be viewed per time step or for the maximum water surface elevation. The manual describes some common errors received in the summary tables in detail and states that most of the notes should be self-explanatory. Often, cross sectional spacing or other geometry issues are a factor mentioned in the errors and warnings. The manual also refers to the log output file, which is used for debugging and can be set to give thorough information about certain dates or the entire simulation time. It is often used to hone in on an issue that is occurring at a specific date of time, to better understand where the instability is stemming from. Jensen (2003) created a document explaining the details of the HEC-RAS model output when running an unsteady flow model. He noted that the log file can become very large in extensive models if it is used for a long simulation time. A file of a magnitude of this size has the potential to slow the simulation significantly. To combat this occurrence, Jensen suggests creating a restart file, which allows the

user to start the simulation closer to where the issue occurred or where the model crashed. This could be a helpful tool when attempting to stabilize a model.

Output Tables

Profile output tables are created for each time step in a simulation when the post processor is used. There are many options for analysis that can be viewed in the tables, including the typical variables of total flow and water surface elevation, but other options include the Froude number, velocity, and energy grade line, among others. The times accessible for viewing in this table are all of the simulated times, along with the maximum water surface elevation, and each cross section has an output for each time step. These detailed tables can be used for many different types of studies, as they allow a user to observe any of the parameters associated with the location within the reach, and the time when it occurred.

Profile Plot

The profile plot is also created from the post processor. It is a visual representation of the reach, and when the simulation runs, it can help a user pinpoint errors that may be occurring. The profile plot is convenient because a user can view what happens in the reach through a video of each time step. Large jumps in water surface elevation or changes in flow regime can signify areas or times that should be viewed more closely and potentially edited before the next simulation is run.

Acceptable Water Surface Elevation Error

Water surface elevation errors are viewable from the computation messages that are created as HEC-RAS is running its simulation. The errors are the first aspects to be analyzed when running a simulation to determine if the model seems to be running well. When running a model, the user must decide what the acceptable water surface elevation should be for their specific project. For the projects created in the making of this paper, the water surface elevation error to signify areas that needed to be viewed more closely was determined to be anything above a value of 0.020 feet. This magnitude of error is the default water surface calculation tolerance from the HEC-RAS User's Manual (2010).

Other Modeling Concerns

Many modeling errors can be caused from lateral structures, bridges, culverts and breach characteristics (Brunner 2014). These issues will not be analyzed because for the model in question, these structures have been removed. Bridges are not input as structures; instead they are input as cross sections upstream and downstream of the area. This surely causes inaccuracies in the model, but the overall goal of modeling the low flow conditions can still be met for low flow conditions with the structures removed.

CHAPTER III

METHODS AND MODELING

Methods

This study was conducted using data from the TVA’s river basin. Figure 1 is an image of the Tennessee River Valley. This visual is scaled for accuracy. According to the United States Geological Survey website, this area is considered a HUC-2, and the Hydrologic Unit Code that applies to the Tennessee River Valley is Region 06, referred to as “Tennessee” (<http://water.usgs.gov/GIS/regions.html>). The HUC area is shown in Figure 2. For the Tennessee Valley Authority’s model, all of the reaches were separated between each dam to be stabilized and calibrated separately. This allowed for more accuracy. Four reaches were analyzed and stabilized prior to this study. These include the reach between Guntersville and Nickajack Dams, the reach that begins at Chickamauga Dam and ends at Apalachia, Ocoee, and Watts Bar Dams, Chilhowee Dam to Fontana, and Douglas Dam to Nolichucky Dam. After some consideration, Guntersville and Chickamauga reaches were determined to be the best overall representations of the Tennessee Valley’s river reaches, allowing this study to focus primarily on them. A summary table of important information about these two reaches is shown in Table 1.

Table 1: Information about Modeled Reaches

Downstream Boundary	Upstream Boundary	Latitude	Longitude	Section Length (miles)	Elevation	Reservoir Volume (acre-ft)
Guntersville	Nickajack	34.424	-86.392	75	595	1,018,000
Chickamauga	Watts Bar Apalachia Ocoee 1	35.104	-85.229	60 82 75	682.5	622,500

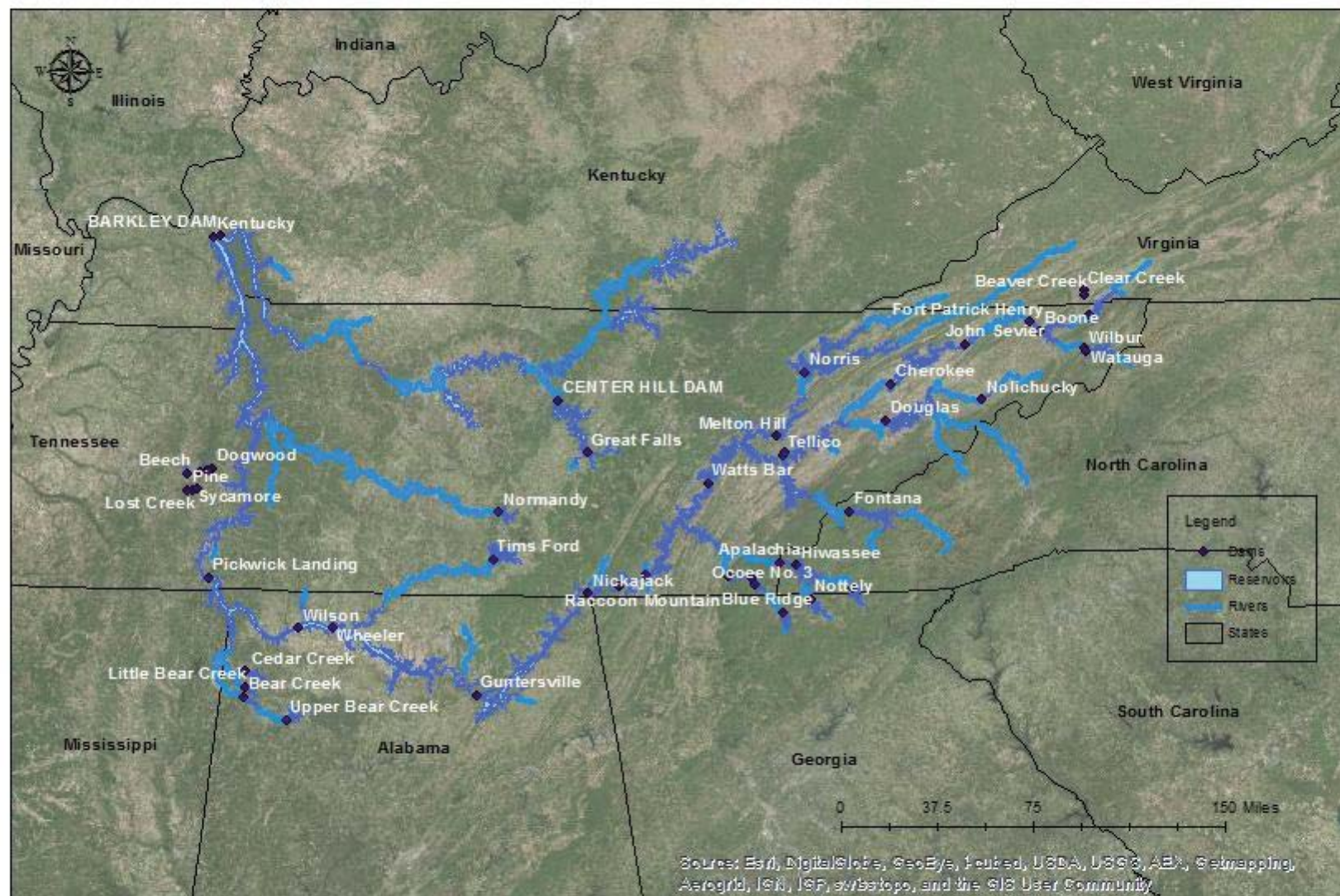


Figure 1: Image of the Tennessee River Valley System

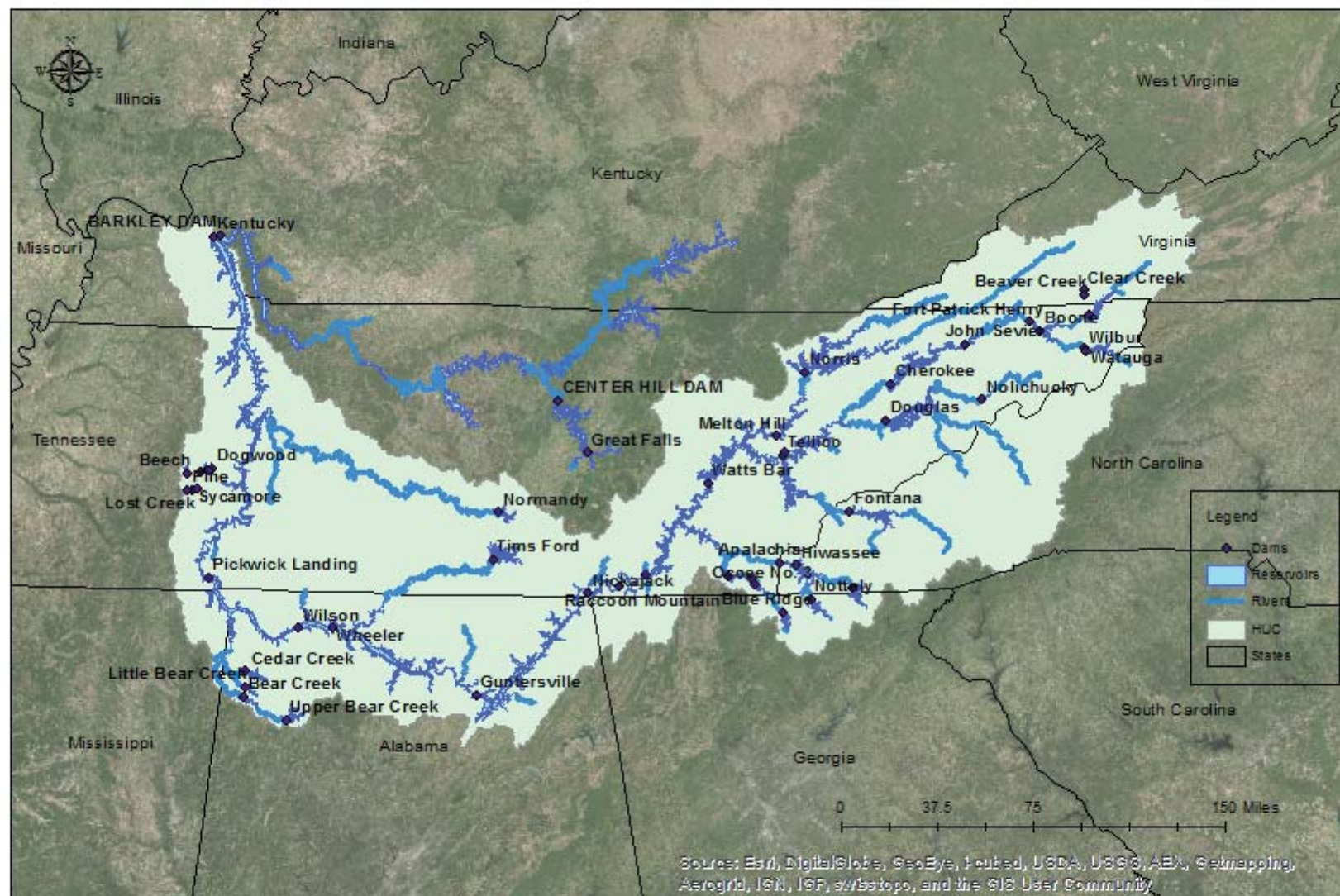


Figure 2: Image of the Tennessee River Valley HUC

The primary reason behind the need for a methodical approach for stabilizing failing models such as this is that companies managing multiple dams must consider the best method when managing dams. The best technique is one that decreases the need for major decision-making in times of crisis. These crises can include situations in which lives and land are at stake. To have a general methodology for what needs to be completed at the dams in times like these, the HEC-RAS model can be used for a simulated dam failure analysis. Low flow conditions are important for these companies also, as they allow for a more automated approach to daily activities and simulations useful for day-to-day observation comparisons.

The basic study design approach was first to calibrate and obtain a stable low-flow model for the two study reaches. This effort is described below. Following model calibration for each study reach, model simulations were conducted on the variables listed below to assess which lends greater model stability. A control simulation using none of the stability methods was also completed for final comparison.

- Minimum flow at upstream boundary
- Lateral inflow at two locations in the reach
- Pilot channel
- Samuels's cross sectional spacing
- 100-foot spacing of cross sections

Final Stabilized Reaches

Guntersville

Of all of the reaches stabilized and calibrated, the Guntersville to Nickajack Dam reach was the most simple to run. The reach spans from Guntersville Dam at Tennessee River Mile 349.0 to Nickajack Dam at Tennessee River Mile 424.7. The reach has no inflows from tributary streams. It has a low slope and is a wide reach overall. These characteristics allow easy stabilization for a modeler due to the amount of water that is always present in the channel. Minimal additions had to be made to the Guntersville model to make the model run once it was set up appropriately.

The majority of the stabilized reach's cross sections added into the model were added solely for use as flow boundary conditions or observed flow data. The cross sections added

include Tennessee River miles 424.68, 424.67, 418.096, and 385.507. All of the cross sections listed were used as boundary conditions. Tennessee River mile 424.67 was added for necessary supplementary flow to ensure the reach had an adequate amount of water, while the other boundary conditions were linked to observed data through the reach. The extra flow was in the form of lateral inflow and had a magnitude of 1,000 cubic feet per second, or cfs. It was removed approximately 23 miles downstream at river mile 401.57, where the model was able to maintain stabilization without the added flow. A minimum flow of 500 cfs was also used at Nickajack Dam in the boundary condition, as Nickajack experienced flows low enough that the model could not run stably without a minimum dam flow.

Once the flow data, plan and geometry were applied in the model, the model ran and had acceptably low errors. Using the stage and flow hydrographs, primarily the Computed-Observed Stage versus Flow graph, comparisons were made between the computed and observed data. Roughness coefficients were then altered with multipliers that ranged around one to slightly increase or decrease the stage to make the computed data more closely match the observed. The stage could then more accurately match historical data and therefore be a better estimate of overall flows.

The maximum water surface elevation error for the Guntersville reach was 0.179 feet. The water surface elevation errors received in the output files after running the HEC-RAS models were determined to be a good predictor of the fit of the model to the observed reach data available. Most of the errors received were below 0.100 feet for this model. This level of error was considered high in the study; however, it was deemed acceptable for this reach due to the fact that it is so large and 0.100 feet is minimal in the low flow analysis, since extreme situations, such as a dam break or flood during low flow conditions, are not likely. Images of the final additions made to the model are included in the Appendix, Figures 12 through 17.

Chickamauga

The Chickamauga reach had many stability issues due to the fact that it is very complex and because of the multiple stretches of rivers that had to be considered. It runs from Chickamauga Dam at Tennessee River mile 471.0 to Watts Bar Dam at Tennessee River mile 529.9. In the middle of the Tennessee River stretch, the reach stretches over to Apalachia and Ocoee 1 Dams, at Hiwassee River mile 66.0 and Ocoee River mile 11.9, respectively. The area

that extends toward Apalachia is the steepest part of the overall reach and since it and the Ocoee 1 Dam reaches are heavily reliant on the flows received from the dams for the first few miles downstream, multiple stability issues were received throughout these areas.

The Chickamauga model required the most additions to original recorded data. Besides the minimum flow addition of 600 cfs at Apalachia Dam, 500 cfs was added at Ocoee and 500 cfs was added at Watts Bar. Some lateral inflows were used to increase base flow, including 500 cfs at Hiwassee river miles 64 and 57.45. The Ocoee River reach also needed a lateral inflow to run stably. This was 1000 cfs in magnitude, placed at Ocoee River mile 11.8770, as close to the dam as was possible. The 2,000 cfs of added inflows were removed from the reach halfway down the Hiwassee River section between the junction and the cross section where the Hiwassee meets the Tennessee River, at Hiwassee River mile 19.36. The cross sections used for these inflows were either silt ranges provided by the TVA or HEC-RAS interpolated cross sections, as described further below.

Also essential to the stability of the Chickamauga model were the additional interpolated cross sections for extra stability in steeper areas of the reach. Samuels's recommended cross sectional spacing was analyzed for each river in this reach. More details about these calculations are discussed in the Modeling section of this paper. The level of spacing suggested by Samuels's calculation, however, was satisfied by the cross sections already in place. After running a few simulations, problem cross sections were pinpointed, which allowed cross sections to be placed at Hiwassee River miles 63.0559, 62.1050, 59.8217, 58.5830, 56.6034, 55.9733, 55.2850, and 51.9635 for stability, along river miles 65.9882 and 18.2018 for a boundary condition and observed data, respectively. Tennessee River mile 529.888 was also added for a boundary condition. Ocoee River miles 11.6029 and 4.466746 were pinpointed and added for stability, and 11.8780 was added as a boundary condition.

The reach required the addition of pilot channels to maintain a small amount of flow at all times. Pilot channels are described in detail in the HEC-RAS User's Manual (2010). Pilot channels were used on the Hiwassee and Ocoee Rivers. The Tennessee River did not require the addition of pilot channels. Images of the final additions made to the model are included in the Appendix, Figures 18 through 31.

Due to the observed data available, the Chickamauga reach was able to be calibrated to ensure the most accurate results in the model simulation. The maximum water surface elevation error was .062 feet for the reach and most errors ranged around .040 feet.

After the stabilization of the models was complete, the need for a methodology to follow for similar reaches was fully realized. When stabilizing these reaches, a guess-and-check method was used. The guess-and-check method was determined to be significantly slower than developing a procedure to monitor changes. When creating the methodology, each of the reaches was considered for accuracy. The simulated reaches matched the observed flow data available closely, shown in the stage hydrographs in Figures 3 and 4. The Chickamauga reach is one with a high complexity, while Gunter'sville has little complexity. This allows for a broader analysis of differently sloped reaches' stabilization.

To systemize the method for creating a stable unsteady low flow model in HEC-RAS, two flow charts were built to instruct a modeler. The charts display which options would be the best to alter when attempting to fix a model, depending on the output generated. To create these charts, which are shown in the Results section of this paper, the following modeling and method analysis was completed. The reaches originally stabilized vary in geometry, flows, and slope, creating an opportunity for different types of errors to occur in each reach. Throughout the reaches, though, errors related to low flow conditions, time step, and cross sectional spacing were dominant.

Site Selection

After all of the models were stabilized using the methods mentioned, the Chickamauga model was then fragmented to make comparisons about the output data compared with the water surface elevation errors received. This analysis was completed for the Chickamauga model because it had a level of complexity that could describe different types of reaches throughout the Tennessee Valley. The Apalachia reach (Hiwassee River, Nottely-Ocoee) of the Chickamauga model represents an area with greater steepness that should be considered when stabilizing. The stretch along the Tennessee River from Watts Bar Dam to Chickamauga Dam is wide and flat, and should not be victim to many stability issues. The Ocoee River and Hiwassee River Ocoee-

Tennessee reaches have lower slopes but different types of issues regarding stability are found on the two reaches. The average slope for each reach is available in Table 2.

Table 2: Average Slope of Reaches

	Reach			
	Tennessee River (Watts Bar Dam to Chickamauga)	Hiwassee River, Ocoee-Tennessee	Ocoee River, BRH-Hiwassee	Hiwassee River, Nottely-Ocoee
Slope (ft/ft)	0.00010	0.00021	0.00058	0.00317

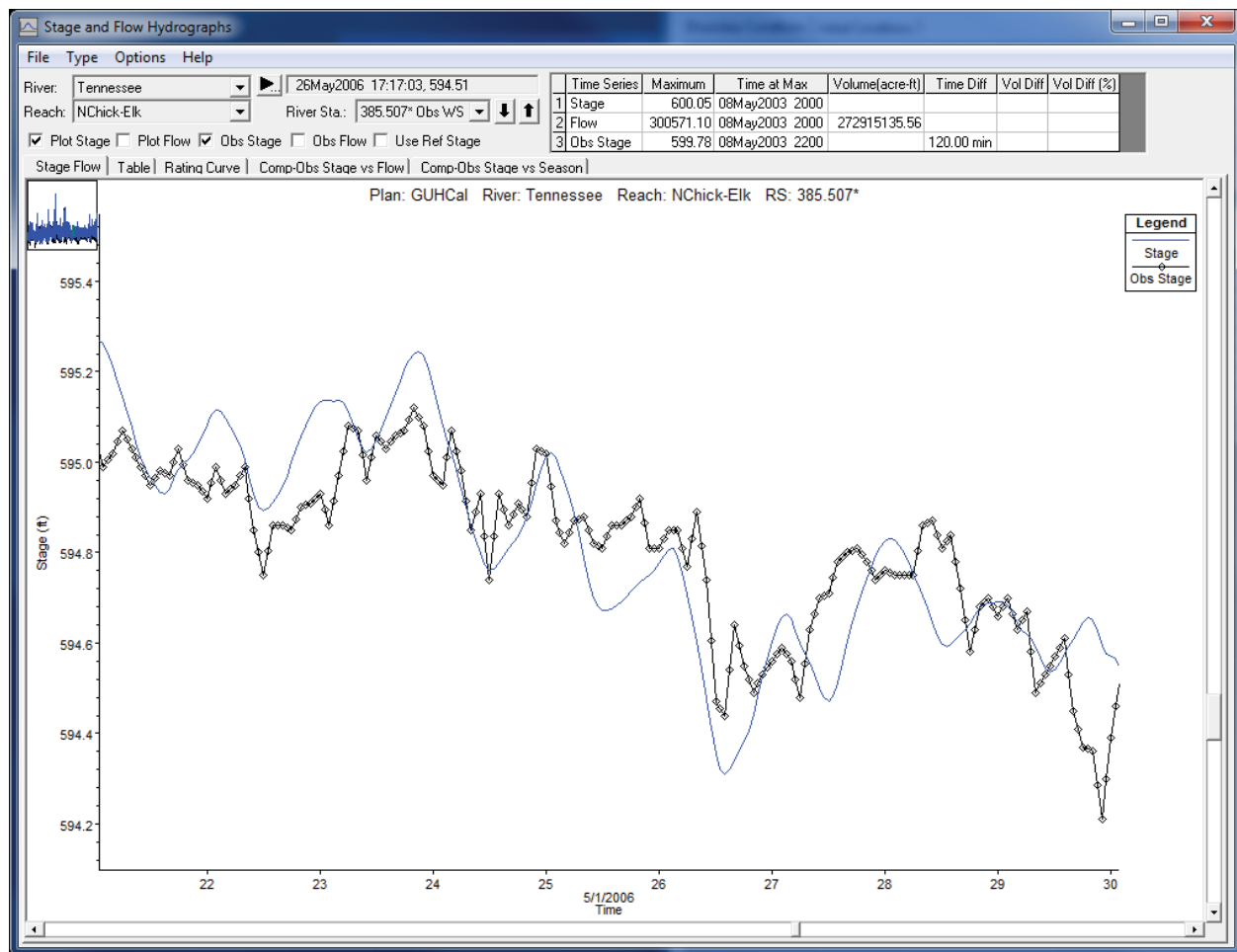


Figure 3: Final Gunterville Model Observed and Simulated Stages on Tennessee River Mile 385.507 at Scottsboro Gage

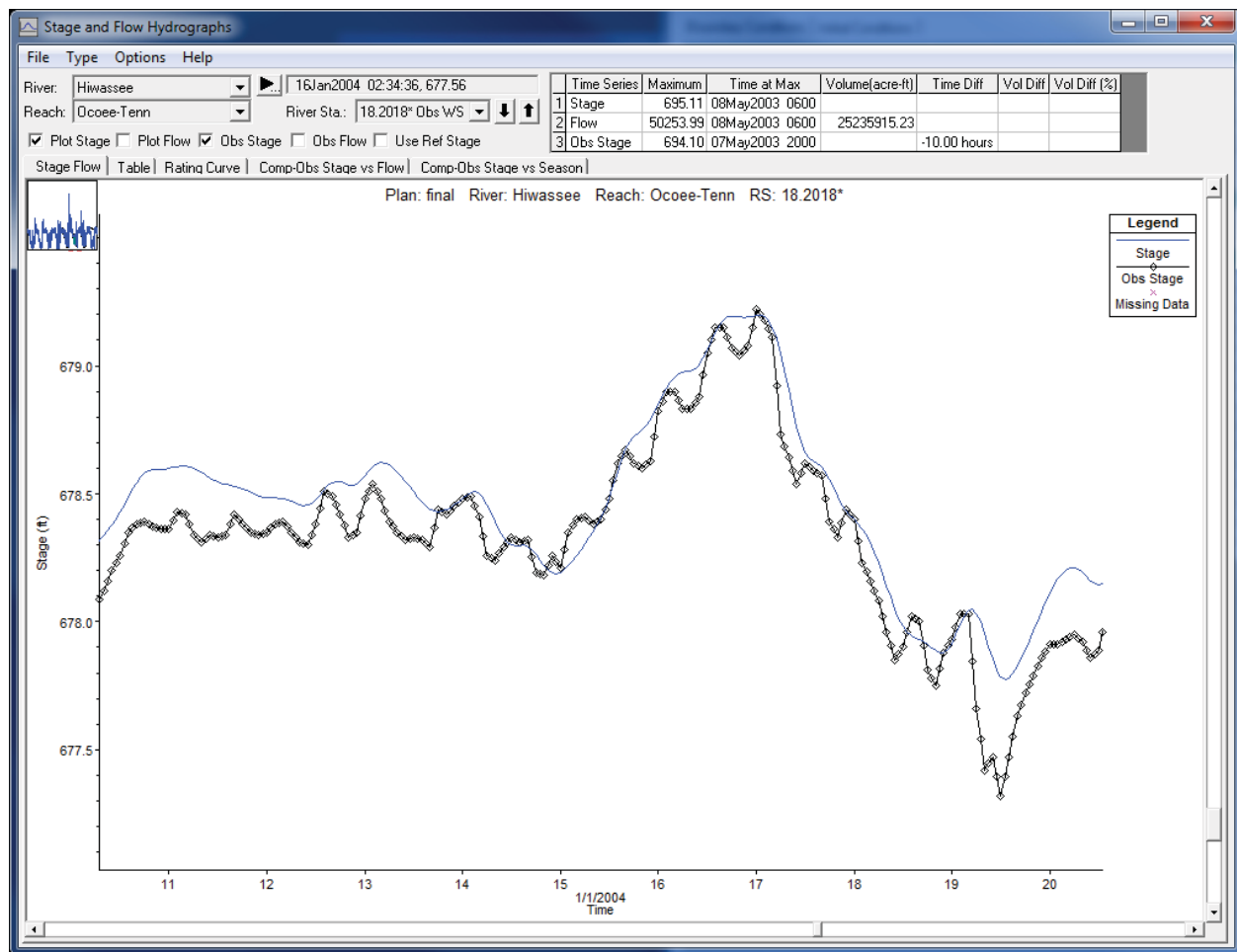


Figure 4: Final Chickamauga Model Observed and Simulated Stages on Hiwassee River Mile 18.2018 at Charleston Gage

Chickamauga Fragmentation and Method Validation

When creating the models for comparison, the final model was taken back to its original form. Only surveyed data were used in the geometry file, with no other additions. The boundary conditions were all set as observed data from the TVA's records. No pilot channels, interpolated cross sections, or minimum flows were used when creating the fragmented model. The fragmented models had to be set up slightly differently than the larger models were, however. Some specifics on each reach's boundary conditions are detailed below. Images of the original reach boundary conditions are available in the Appendix.

Hiwassee River, Nottely-Ocoee Reach

The upstream boundary condition on the Hiwassee River, Nottely-Ocoee reach was the Apalachia Dam flow. The downstream boundary used was a normal depth, which was set at the downstream end of the Ocoee-Tennessee reach to lessen the effects on the Nottely-Ocoee reach. The normal depth used was an average of the bed slope, which was determined to be an accurate representation of the energy grade line. The larger models previously discussed used stage as the downstream boundary conditions. The difference in the downstream boundary conditions used is due to the fact that the fragmented models do not typically flow directly into a dam at the downstream boundary. Because of this, the normal depth was a more appropriate choice. For consistency, all downstream conditions on the fragmented models were normal depth.

Ocoee River, BRH-Hiwassee Reach

The upstream boundary condition on the Ocoee River, BRH-Hiwassee reach was the Ocoee 1 Dam flow. The downstream boundary used was a normal depth, which was set at the downstream end of the Ocoee-Tennessee reach to lessen the effects on the BRH-Hiwassee reach.

Hiwassee River, Ocoee-Tennessee Reach

The upstream boundary condition on the Hiwassee River, Ocoee-Tennessee reach was a flow hydrograph at the upstream cross section of this reach taken from the completed and stabilized Chickamauga Dam model. The downstream boundary used was the normal depth near Chickamauga Dam.

Tennessee River (Chickamauga) Reach

The upstream boundary condition on the Tennessee River reach was the Watts Bar Dam flow hydrograph. The downstream boundary used was the normal depth near Chickamauga Dam, which was determined to be the best option for accuracy.

Tennessee River (Guntersville) Reach

The Tennessee River and Hiwassee River, Ocoee-Tennessee River reaches had enough stability that they did not create any water surface elevation errors with which to compare. After the realization was made that the Tennessee River (Chickamauga) reach was stable in the original run with no stability procedures used, the Guntersville reach was fragmented and analyzed for validation of each model stability technique used as in the Chickamauga reaches, since it provided errors from which conclusions could be drawn. The Guntersville reach had a slope of .00010, which is identical to the slope on the Watts Bar to Chickamauga reach of the Tennessee River. The upstream boundary condition used was the flow hydrograph at Nickajack Dam, and the downstream boundary condition was the normal depth near Guntersville Dam. The data from this reach was compared with the data provided for Chickamauga, and is visible in Table 17 in the Appendix. The model in which nothing was added is also shown on the table, as a control to better analyze the results. The images from the additions to the Guntersville reach are also available in the Appendix, Figures 50 through 56.

Stability Methods Used for Analysis

To create the flow charts, it was determined that stability methods must be singled out to ensure the results received were not affected by any other parameter. Once the model was fragmented, the five methods of stability for analysis were decided on. The five chosen as seemingly most effective were the addition of minimum flow at the upstream boundary condition, the addition of lateral inflow that acts as base flow at two locations in the reach, the addition of a pilot channel, the addition of interpolated cross sections matching Samuels's suggested spacing, and the addition of interpolated cross sections every 100 feet throughout the channel being tested. For comparison, the original model with none of these additions was also run. Tables 13 through 16 describing and quantifying the additions made to each model can be

found in the Appendix. Images of the additions made in the HEC-RAS models are available in the Appendix, Figures 32 through 49.

Time Step

For the reaches stabilized throughout this document, the time step interval selected was set to one hour. This time step was determined to be best for the models in question due to the necessity for the final stabilized models to run simulations quickly for real-time every day forecasting, the magnitude of the overall model, and the following analysis of time step.

The time step is a definite factor affecting stability of a model. It is a fraction of the wave time speed, which allowed the time step an accurate analysis of flood waves as they moved through the system. The time step was not used as a focus in this analysis due to the suitability of the one hour time step for the TVA's model. Some comparison was completed, however, on the time step and its effect on the errors and simulation times.

Tables 3 and 4 show the maximum water surface elevation error and the hours of simulation as it is related to the chosen time step, respectively. This analysis was completed only on the highest and lowest-sloped reaches in the study.

Table 3: Maximum Water Surface Elevation Error and Time Step

Maximum Water Surface Elevation Error Received Before Instability			
Reach	Time Step Chosen		
	5-minute	1-hour	6-hour
Hiwassee Nottely-Ocoee	0.170	0.035	N/A ¹
Tennessee River (Guntersville)	N/A ²	0.076	0.097

¹Did not list any errors before becoming unstable

²Model did not have errors on the reach

Table 4: Hours of Simulation Completed and Time Step

Hours of Simulation Run Before Instability			
Reach	Time Step Chosen		
	5-minute	1-hour	6-hour
Hiwassee Nottely-Ocoee	199.5 hours	414 hours	198 hours
Tennessee River (Guntersville)	N/A ¹	N/A ¹	N/A ¹

¹ Model did not become unstable

The tables show the Hiwassee Nottely-Ocoee reach varying in hours of simulation and error as the time step is changed. An overall analysis of the results for this reach suggests that the best option when modeling the Nottely-Ocoee reach is the 1-hour time step. The Guntersville reach, however, listed no errors when it was run at a 5-minute time interval, and displayed an overall increase in error as the time step was lowered.

As was visible from Figure 1, the Tennessee River Valley spans a large area and yields a very large overall model to be run in these everyday simulations. The 1-hour time step chosen was deemed applicable for all simulations in this study primarily because a lower time step such as the 5-minute would take an immense amount of time to run the simulation and cannot meet the time demands of the TVA's model, considering the importance of quickly run simulations in crisis situations.

Minimum Flow

When creating a minimum flow at the upstream boundary condition in each reach, Brunner's (2014) suggested peak flow ratio was used. The peak flow was attained from the flow hydrograph associated with the upstream boundary condition. This was determined to be a very important addition to make to a reach, due to the fact that the upstream boundary condition in these models was often regulated dam. It was realized that regulated dams could have a quantity of flow that was zero at times due to the need to hold water in the reach or to keep the elevation at a constant level between two dams. The records used for this study have zero flows when the dams are not generating. For cases such as this, minimum dam flow conditions were key, as the model would otherwise often "run dry" due to the lack of flow at the upstream boundary.

Although it was not good practice to commonly use this method due to the amount of time it would take to accurately calculate the volume of water actually being added in, it did help model stability significantly in several instances.

Lateral Inflow at Two Locations

The lateral inflow at two locations stability method was similar to the minimum flow. Using the peak flow ratio, half of that amount of water was added into the model as a lateral inflow at a point near the upstream condition and then the other half of the quantity of water suggested was added at a cross section halfway down the reach.

Pilot Channel

The pilot channel was created in each model for comparison according to the suggestions made by Brunner (2014) discussed in the Literature Review. A roughness coefficient had to be specified for each cross section, and a value of .035 was used for these reaches, as the quantity of water in the reach should allow that roughness value to be realistic enough to give reliable results. An analysis similar to the time step analysis was completed on the pilot channel parameters of width and roughness coefficient as follows in Tables 5 and 6.

Table 5: Maximum Water Surface Elevation Error and Manning Roughness in Pilot Channel

Maximum Water Surface Elevation Error Received Before Instability			
Reach	Manning Roughness Selected		
	0.0175	0.035	0.0525
Hiwassee Nottely-Ocoee	0.082	0.050	0.069
Tennessee River (Guntersville)	0.076	0.076	0.076

¹Unstable before it could compute errors

Table 6: Hours of Simulation and Manning Roughness in Pilot Channel

Hours of Simulation Run Before Instability			
Reach	Manning Roughness Selected		
	0.0175	0.035	0.0525
Hiwassee Nottely-Ocoee	611 hours	431 hours	244 hours
Tennessee River (Guntersville)	N/A ¹	N/A ¹	N/A ¹

¹Model did not become unstable

The tables show that the roughness coefficient caused a difference for the Nottely-Ocoee reach in the maximum water surface elevation error if the roughness value was lower or higher than the channel-appropriate 0.035. The Guntersville reach had no differences from the pilot channel addition.

Next, the width of the channel was analyzed using the same technique. Tables 7 and 8 display these results.

Table 7: Maximum Water Surface Elevation Error and Width of Pilot Channel

Maximum Water Surface Elevation Error Received Before Instability		
Reach	Pilot Channel Width Selected	
	1-foot	50-foot
Hiwassee Nottely-Ocoee	0.050	N/A ¹
Tennessee River (Guntersville)	0.076	0.076

¹Unstable before it could compute errors

Table 8: Hours of Simulation and Width of Pilot Channel

Hours of Simulation Run Before Instability		
Reach	Pilot Channel Width Selected	
	1-foot	50-foot
Hiwassee Nottely-Ocoee	431 hours	0 hours
Tennessee River (Guntersville)	N/A ¹	N/A ¹

¹Model did not become unstable

The tables show that the pilot channel width increase caused a difference for the Nottely-Ocoee reach and caused the model to immediately become unstable. The Guntersville reach had no differences from the pilot channel changes. These results and the ones from the roughness value in pilot channels support the use of 1-foot width in the channel and the 0.035 roughness value.

Samuels's Cross Sectional Spacing

Because other authors researched the equation by Samuels and found it to be an accurate representation for gently sloping river reaches, the Samuels equation was used when analyzing the model stability method of cross sectional additions. The bankfull depth used for the calculation was the slimmest bank-to-bank distance in the reach due to the fact that it yielded the most conservative overall spacing. The slope of the bed used is the average bed slope throughout the reach. The cross sectional spacing suggested by the equation was used to interpolate cross sections within a reach to meet the spacing recommended. The bankfull depths and Samuels's maximum spacing calculations for some reaches are included in the Appendix. The reason the other reaches analyzed are not included in the table will be discussed later in the document.

One Hundred-Foot Cross Sectional Spacing

The HEC-RAS User's Manual (2010) notes appropriate cross sectional spacing for specific stream reaches. It states that streams flowing at high velocities may require cross sectional spacing of 100 feet or less. The 100-foot spacing was analyzed as a minimum spacing for this study. The addition of this many cross sections, however, greatly slowed the simulations

and caused multiple simulations to run out of memory before completion. Some areas did not permit this cross sectional spacing to be used due to interpolation issues between two cross sections. In these areas, the cross sections on either end of the interpolation area were so vastly different that the interpolation could not be completed. These instances were limited, allowing an overall idea of stability to still be attained on the reaches.

Modeling and Output Analysis Files

The geometry data used in this model was provided largely by the TVA, in the form of bathymetric survey data, which are also known as silt ranges (TVA EAP 2014). Some gaps were found in the data when assembling, and those were augmented by Riverside Technology, Inc. using external data sources (TVA EAP 2014). The inflows within the watersheds were also analyzed by Riverside, and are included in the models as lateral inflows and uniform lateral inflows. Lateral inflows are flows that enter the model as a boundary condition at a specific cross section, whereas uniform lateral inflows are applied to the reach spanning multiple cross sections. The company completed an extensive basin-wide calibration analysis of the hydrologic models to predict the inflows into each piece of the system (TVA Basin Calibration 2014). The watershed basins used for the inflow analysis are shown in Figure 5. The calibration analysis also yielded specific roughness coefficients deemed accurate for the reaches which were used for this study.

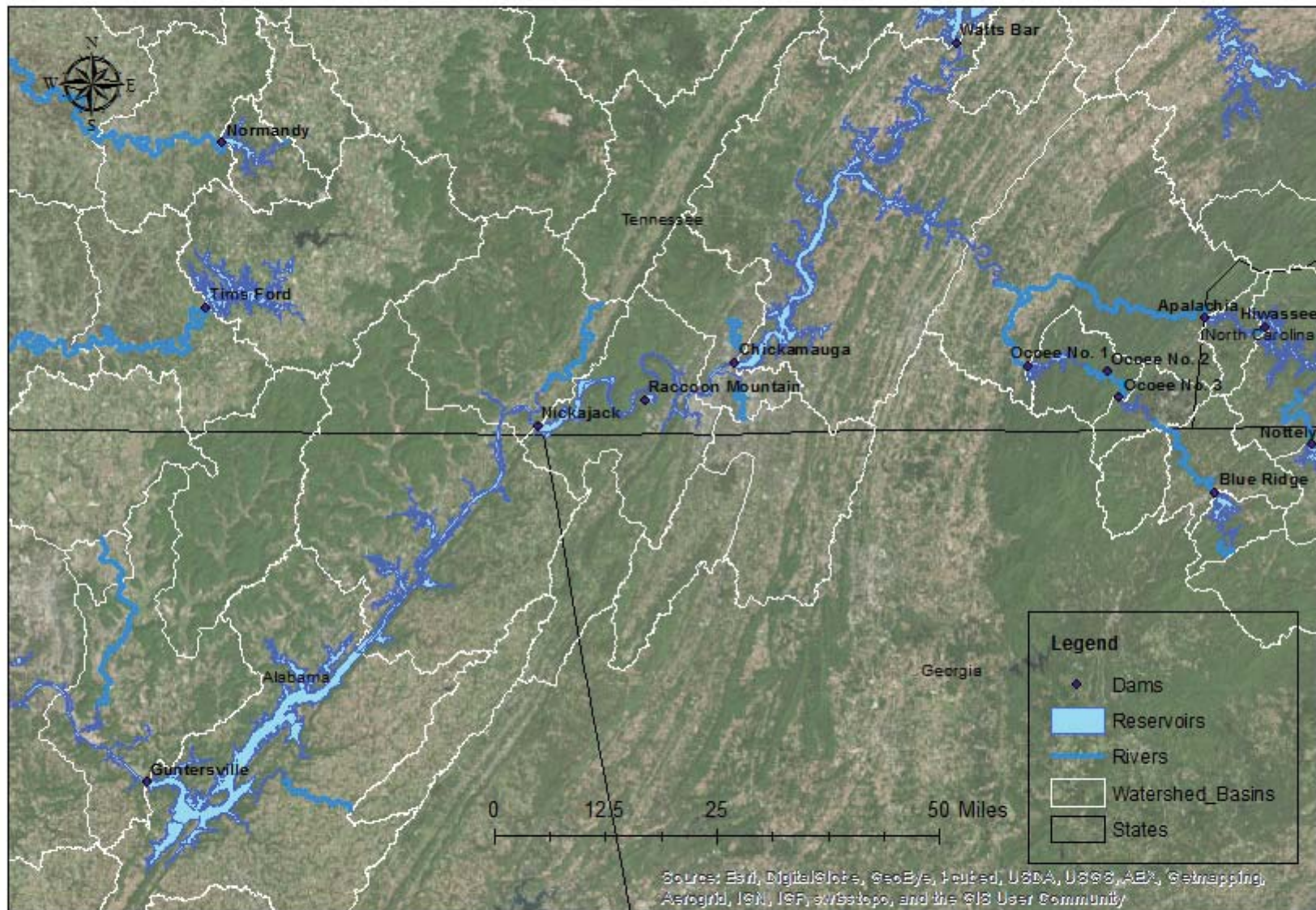


Figure 5: Watershed Areas Used for Inflow Analysis

The compilation of this data was used when creating the HEC-RAS unsteady low flow models of the reaches. These models required a trial-and-error process to discover what changes to the model could resolve the issues causing instabilities. Most of the concerns appeared to have stemmed from issues discussed in the Literature Review section of this paper, ranging from cross-sectional spacing to the necessary addition of base flow. Details on each reach's necessary stability changes will be discussed further in the Results section. The methods of undertaking any concerning aspects of the inputs and achieving a stable model are detailed as follows.

Typically, the first run gave a listing of setup errors to be fixed before the model could create a simulation. These were generally easily corrected with small changes and by ensuring the data and files were input properly in each area. Issues frequently listed included boundary conditions being set up incorrectly, simulation times that did not match the data being used, or flows that were overlooked when entering other data. After amending the file errors, stability issues began to arise.

The overall stability of the model increased by changing theta to 1.0, as discussed previously (Hicks and Peacock 2005). Again, this was noted to decrease the accuracy of the model, but for the initial runs it was used (Hicks and Peacock 2005). After further studying the theta weighting factor, however, it was determined that the changes to the overall stability in the model dependent on the theta weighting factor were negligible, which suggested that numerical stability had been attained in the models and the errors were likely stemming from other issues in the model. The theta weighting factor is also related to the time step, allowing for additional confidence in the time steps chosen for the reaches.

The models were analyzed using each stability method previously mentioned. The reaches were simulated separately for each method, and each output showed the amount of error that was at the river station at a particular simulated date and time. The computational messages, summary of errors, warnings, and notes, log file, and output tables were heavily relied upon when deciding upon the best methods to use for stability in creation of the flow charts. More details about these files are given below. The HEC-RAS model itself could not be attached to this document due to the sensitive nature of the data, as it is used on a daily basis by the TVA in multiple aspects of the company.

Computational Messages

The computational messages shown after running the model include the water surface elevation errors and the date that the maximum error was exceeded in the model. The quantity and magnitude of errors received can be a valuable representation of the truthfulness of results. The time when the simulation crashed was another telling factor of the usefulness of methods to increase stability. The time range for the simulation was shortened for these comparisons to July 2, 1998 to August 31, 2000, as a representation of the beginning of the data sequence to ignore the effects of end data that may have been flawed and to prevent files from becoming too large. Errors below 0.020 feet were typically accepted for this model due to the fact that it is multi-faceted. After the simulations were developed over the entire time period, the model listed the overall maximum water surface elevation error reached when running the simulation, and that value was analyzed also.

Summary of Errors, Warnings, and Notes

As mentioned to in the Troubleshooting section of the HEC-RAS User's Manual (2010), the Summary of Errors, Warnings, and Notes can be analyzed at each cross section for each time step. The first day of noticeable instability (explained in detail in the Results section) was studied for comparison with the maximum water surface elevation summary.

Log File

The log file was a very large file in most cases in the analyses. When it seemed that the file would be too large to allow the simulation to run completely, the log file was specified to output only the time step in which the first error noted was above 0.020 feet. When the model was able to run fully, without specifying dates for the log output, the modeler had the log file run for the entire simulation to view multiple aspects that could be causing errors in the reaches.

Output Tables

When analyzing the output tables, it was determined that the most important parameters for determining the hour of model instability were the Froude number and total flow in the channel.

Froude Number

The Froude number is considered to be the “most important governing dimensionless parameter” in open channel flow and is defined as the relationship of the inertial and gravity forces (Sturm 2010). The equation for Froude number is as follows:

$$F = \frac{V}{(gD)^{0.5}} \quad (4.1)$$

Where V is mean velocity, D is depth, and g is the acceleration due to gravity (Sturm 2010). The Froude numbers were compared with the magnitude of the water surface elevation errors to determine whether or not the Froude number can predict the level of the error.

Total Flow

The total flow output was used in conjunction with the Froude number to determine when the model stopped giving accurate results due to instability.

CHAPTER IV

RESULTS AND DISCUSSION

Results

Each piece of the Chickamauga model was run with the aforementioned stability parameters in place. The way in which each stability method was used allowed for the maximum amount of stability to be acquired per method. For example, the maximum allowable value for the minimum flow value on the upstream boundary condition was Brunner's (2014) peak flow relationship, and applying that amount of flow to the model permitted the model maximum stability from that specific parameter, without being undesirably affected by any other factor. The other systems in which the stability methods were maximized for stability were mentioned previously. After the models were run with a stability method in place, the outputs were used for analysis.

Sensitivity Analysis of Methods Used for Stability

Defining the Point of Instability Using the Computational Methods and Output Tables

Determining the time step when the model was unstable was an important aspect when analyzing the stability methods. There were many different options when attempting to find this time step.

In the computational messages, HEC-RAS will note when the model has become unstable, and at which cross section it occurred. When viewing the computational methods, however, instability is often visible before the model notes it. Commonly, the time steps noted on the computation messages are associated with large water surface elevation errors (above 0.020 feet). Some errors above .020 feet were also received prior to this point; however, the first unstable time step was still determined to be most accurately represented by a Froude number, water surface elevation error, and flow investigation. Most simulations reached a time step in which they began listing errors at every time step on the computational messages screen, which can be another indication toward significant instability. The computational messages were saved as an Excel file for further instability analysis.

The Froude number was determined to be a necessary parameter to note for analysis of when flow regimes changed abruptly, which could then be analyzed to see if the change in flow regime was appropriate in the model, or if it changed significantly due to instabilities occurring. Often, the Froude number would reach a very high magnitude, even reaching the hundreds, once the model reached instability. This Froude number was commonly obtained on the same day as or on the day after a negative total flow was attained. Since the model in question should not be victim to large negative flows or extreme Froude numbers, the date associated with these parameters was considered the first unstable day, and only the Froude number attained prior to that day was analyzed in the water surface elevation error-Froude number analysis briefly mentioned above. An example of the Excel files used when determining the first unstable day is shown in Figure 6. The highlighted line denotes the first unstable time step. Note that the Froude Number is in the 4,000s on the unstable time step and the flow is large in quantity and negative.

Lateral Inflow 2 Locations Ocoee Comp Messages.xlsx - Microsoft Excel

	A	B	C	D	E	F	G	H	I	J	K
1											
2	Geometric Preprocessor Version 4.1.0 Jan 2010										
3											
4	Finished Processing Geometry										
5											
6	Performing Unsteady Flow Simulation Version 4.1.0 Jan 2010										
7											
8	Maximum iterations of 40 at:		RS	WSEL	ERROR	Q Total	Min Ch El	WS El	Crit WS	Vel Chl	Froude
9	01JUL1998 01:00:00 Ocoee	BRH-Hiwassee	11.8790*	719.31	0.834	0.39	719.29	719.31		0.06	0.07
10	02JUL1998 02:00:00 Ocoee	BRH-Hiwassee	11.15	712.36	0.023	-1144.12	712.3	712.36	714.37	-4821.42	4861.32
11	02JUL1998 04:00:00 Ocoee	BRH-Hiwassee	11.15	712.4	0.037	-283.27	712.3	712.4	713.35	-454.64	360.11
12	02JUL1998 08:00:00 Ocoee	BRH-Hiwassee	11.8790*	719.31	0.79	0.36	719.29	719.31		0.05	0.06
13	02JUL1998 09:00:00 Ocoee	BRH-Hiwassee	11.8790*	719.31	0.787	0.37	719.29	719.31		0.06	0.07
14	02JUL1998 11:00:00 Ocoee	BRH-Hiwassee	11.8790*	719.31	0.059	1.56	719.29	719.31		0.23	0.28
15	02JUL1998 12:00:00 Ocoee	BRH-Hiwassee	11.15	716.75	0.696	141.22	712.3	716.75		0.32	0.03
16	02JUL1998 13:00:00 Ocoee	BRH-Hiwassee	11.15	712.38	0.028	-1101.26	712.3	712.38	714.33	-2980.79	2690.58
17	02JUL1998 22:00:00 Ocoee	BRH-Hiwassee	9.87	712.56	0.062	362.97	710.5	712.56		1.17	0.15
18	02JUL1998 23:00:00 Ocoee	BRH-Hiwassee	11.88	778.11	0.024	0	719.3	778.11		0	0
19	03JUL1998 02:00:00 Ocoee	BRH-Hiwassee	11.8790*	780.84	0.033	645.46	719.29	780.84		0.02	0
20	03JUL1998 03:00:00 Ocoee	BRH-Hiwassee	8.2	705.07	0.065						
21	03JUL1998 04:00:00 Ocoee	BRH-Hiwassee	11.15	712.5	0.068	-315.49	712.3	712.5	713.42	-123.9	69.02
22	03JUL1998 05:00:00 Ocoee	BRH-Hiwassee	9.87	710.52	0.022						
23	03JUL1998 12:00:00 Ocoee	BRH-Hiwassee	11.8790*	719.31	0.804	0.37	719.29	719.31		0.05	0.07
24	03JUL1998 13:00:00 Ocoee	BRH-Hiwassee	11.8790*	719.31	0.797	0.38	719.29	719.31		0.06	0.07
25	03JUL1998 14:00:00 Ocoee	BRH-Hiwassee	11.15	712.36	0.024	-1247.73	712.3	712.36	714.46	-4715.66	4627.02
26	03JUL1998 19:00:00 Ocoee	BRH-Hiwassee	11.15	712.36	0.02	123.94	712.3	712.36	713.03	637.56	675.7
27	03JUL1998 22:00:00 Ocoee	BRH-Hiwassee	8.75	706.19	0.027						
28	03JUL1998 23:00:00 Ocoee	BRH-Hiwassee	9.35	707.77	0.027						
29	03JUL1998 01:00:00 Ocoee	BRH-Hiwassee	8.75	704.72	0.044						
30	04JUL1998 03:00:00 Ocoee	BRH-Hiwassee	11.8790*	719.31	0.335	0.37	719.29	719.31		0.06	0.07
31	04JUL1998 04:00:00 Ocoee	BRH-Hiwassee	11.8790*	719.31	0.768	0.36	719.29	719.31		0.05	0.06
32	04JUL1998 05:00:00 Ocoee	BRH-Hiwassee	11.88	724.4	0.029						

Computation Messages Max WS Sheet3

Figure 6: Ocoee BRH-Hiwassee Reach Computational Messages and Output File Analysis (Lateral Inflow at 2 Locations)

Summary Table

Commonly for all of the models and methods of stability, the Summary Table recommended the addition of cross sections due to energy losses greater than one foot, a conveyance ratio of less than .7 or greater than 1.4, or a change in velocity head of more than 0.5 feet. The conveyance ratio is calculated as the upstream conveyance divided by the downstream conveyance, as explained within the Summary Table. The HEC-RAS Hydraulic Reference Manual (2010) notes the way in which conveyance is calculated more in detail. It states that the default method when calculating the conveyance is to calculate it only at roughness-value break points. Since the stability methods were analyzed separately, it was thought that these occurrences would be alleviated when the 100-foot cross sectional spacing was used. This was not the case, however, and the 100-foot spacing yielded the same result.

The Ocoee BRH-Hiwassee reach 100-foot cross sectional spacing's Summary Table for the first day that the model was determined to have gone unstable is visible in Figure 7. It was desired to have outputs for the 100-foot spacing model's maximum water surface summary table and for the same model's first unstable day. This would allow the modeler to determine what the issues were right at the time of instability. The Ocoee BRH-Hiwassee 100-foot cross sectional spacing model, however, was so unstable that it crashed on the first time step and did not give an output for anything except the maximum water surface shown in Figure 7.

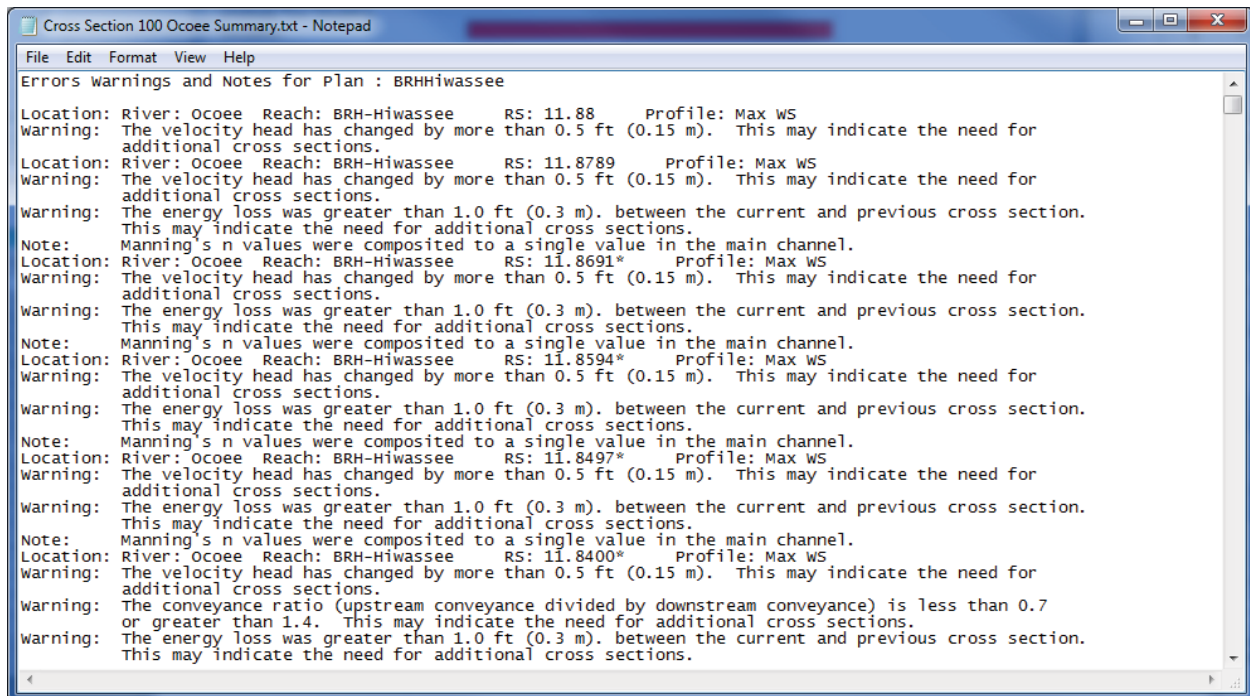


Figure 7: Ocoee BRH-Hiwassee Reach Maximum Water Surface Summary Table (100-Foot Cross Sectional Spacing)

Log File

The Log file was viewed to find computational issues. Some Log files noted that there were significant changes in flow within a reach, which can signify discrepancies between user-entered initial flow values and the flow in a reach at the beginning of the simulation. After ensuring these parameters were correct, the next suggestion in the Log file was analyzed. The next suggestion was to turn on the flow optimization option. After this option was used, however, the model had the same issues, likely due to the massive amounts of instabilities throughout the model. The Log file was determined therefore to be a helpful when checking conditions during specific time steps (which is the suggested use in the HEC-RAS User's Manual (2010)), but not as helpful when attempting to determine the overall stability lent to the model by specific stability methods. An image of the Log file created from the Hiwassee Nottely-Ocoee reach pilot channel stability method is included in Figure 8.

Pilot Channel Nottely Log.txt - Notepad											
File Edit Format View Help											
Iter	River	Station	Elev	DZ	River	Station	Q	DQ			
3	Hiwassee	48.67	753.00	0.12065	Hiwassee	49.7	1291	71			
Solving for Time window = -200.00000 to -199.00000 Hours											
Iter	River	Station	Elev	DZ	River	Station	Q	DQ			
4	Hiwassee	48.67	752.97	0.03196	Hiwassee	50.51	1344	19			
Solving for Time window = -200.00000 to -199.00000 Hours											
Iter	River	Station	Elev	DZ	River	Station	Q	DQ			
5	Hiwassee	48.67	752.97	0.00746	Hiwassee	50.51	1341	4			
Hiwassee Nottely-Ocoee											
Riv. Sta.	Q Diff.	Q Backwater	Q Timestep		Z Diff.	Z Backwater	Z Timestep				
65.99	-10.00	1100.00	1110.00		-0.94	1168.65	1169.59				
65.9882*	-8.92	1100.00	1108.92		-0.94	1168.65	1169.59				
65.4	-1054.73	1100.00	2154.73		-0.82	1165.38	1166.20				
64.7	-842.67	1100.00	1942.67		-0.61	1158.64	1159.24				
64	-590.84	1100.00	1690.84		-0.59	1151.92	1152.51				
63.5	-518.77	1100.00	1618.77		-0.23	1146.72	1146.95				
62.5	-608.05	1100.00	1708.05		0.61	1132.78	1132.17				
61.7	-670.19	1100.00	1770.19		-0.39	1105.82	1106.20				
61	-574.84	1100.00	1674.84		-0.20	1081.71	1081.91				
60.4	-478.18	1100.00	1578.18		-0.40	1061.50	1061.90				
59.2	-369.18	1100.00	1469.18		-0.13	1027.58	1027.71				
57.45	-287.27	1100.00	1387.27		-0.81	930.85	931.67				
56.3	-243.93	1100.00	1343.93		-0.71	905.34	906.05				
55.6	-216.85	1100.00	1316.85		-1.12	871.03	872.15				
54.96	-180.98	1100.00	1280.98		-0.57	859.94	860.51				
3.75	-356.13	1100.00	1456.13		0.03	670.55	670.52				
1.73	-119.21	1100.00	1219.21		-0.15	649.56	649.70				
These differences may, for some data sets, cause instability in the model. They are often caused by discrepancies between the values entered for initial conditions and the values on the boundary hydrographs. For instance, for the upstream end of a reach the flow for time = 0.0 on the hydrograph may not match the flow in the initial condition. A difference at an internal cross section may be caused by a lateral diversion (e.g. lateral weir), that is not represented in the initial condition flows. For some data sets (such as those with lateral weirs), turning Flow Optimization on may help.											
COMPUTED STAGES AND DISCHARGES AT T = -199.0000 HOURS - 7/ 1/1998 AT TIME 0:00:00											
Hiwassee Nottely-Ocoee											
Riv. Station	Z	Q	V	Riv. Station	Z	Q	V	Riv. Station	Z	Q	V
65.99	1169.59	1110.00	0.65								
1.73	649.70	1219.21	1.76								
*** Start of New Time Step ***											
Solving for Time window = -199.00000 to -198.00000 Hours											

Figure 8: Hiwassee Nottely-Ocoee Reach Log File (Pilot Channel)

Froude Analysis

The Froude number analysis included a study of the correlation between the water surface elevation error and Froude number to determine if they were somehow interlinked with the stability methods used. This examination was later determined to be futile, however, as the correlations were too reliant on only a small number of data points to draw a true conclusion between the parameters. The small quantity of data points referred to was due to the massive amounts of instability in the models causing the models to become unstable and no longer produce accurate results. An example of the Froude number and error parameter comparison is shown in Figure 9. It is noted that there are not Froude number analyses for all of the stability methods on each reach. This is due to the fact that the model either crashed too quickly or was too large to compute the output tables needed for the analyses.

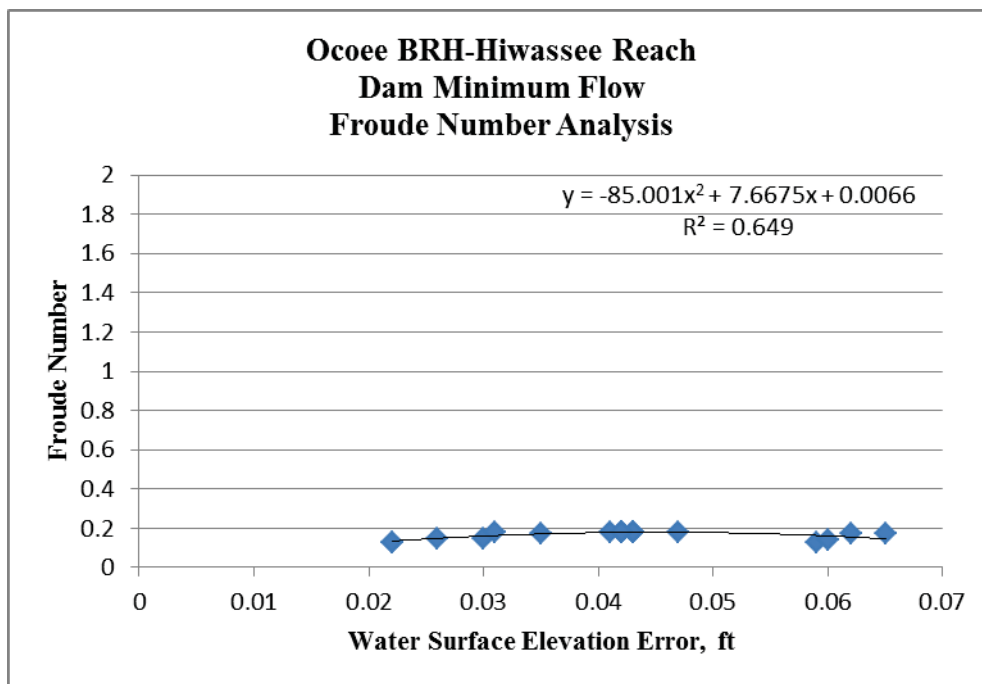


Figure 9: Ocoee BRH-Hiwassee Reach Froude Number Analysis (Minimum Flow)

Manning's Roughness Coefficient

A roughness coefficient analysis was completed to also determine the sensitivity of stability related to it. The results from this analysis are shown in Tables 9 and 10.

Table 9: Maximum Water Surface Elevation Error and Manning Roughness of Models

Maximum Water Surface Elevation Error Received Before Instability			
Reach	Manning Roughness Changes Made		
	50%	0%	150%
Hiwassee Nottely-Ocoee	N/A ¹	0.035	0.344
Tennessee River (Guntersville)	0.045	0.076	0.092

¹Unstable before it could compute errors

Table 10: Hours of Simulation and Manning Roughness of Models

Hours of Simulation Run Before Instability			
Reach	Manning Roughness Changes Made		
	50%	0%	150%
Hiwassee Nottely-Ocoee	0 hours	414 hours	605 hours
Tennessee River (Guntersville)	N/A ¹	N/A ¹	N/A ¹

¹Model did not become unstable

The roughness values used in the models, as discussed previously, had been compiled and calibrated and were considered accurate when used in the models. Increasing the parameter by 50 percent caused an increased water surface elevation error in both of the reaches but allowed a

longer simulation time for the Nottely-Ocoee reach. The halved roughness values caused the Nottely-Ocoee reach model to fail immediately, but decreased the errors in the Guntersville reach. The halved and increased values are not likely accurate, however. For this study's models, the original values were used, as they were deemed best and most appropriate for the reaches.

Maximum Water Surface Elevation and Time of Simulation

Since the Summary Tables typically suggested adding more cross sections even when the 100-foot spacing was used and the Log file often suggested the use of the flow optimization tool and checking initial boundary conditions, the final flow charts were created using the results compiled primarily from the output table and computational message analysis. These were the same tools used when discussing the first time step of instability. A combination of the time of simulation before instability and maximum water surface received, as well as the overall magnitude of the errors received prior to instability, was analyzed to create the flow charts.

From Tables 11 and 12, which were created using the time of simulation and error data, the most stable reaches are easily noticeable. While the Hiwassee Nottely-Ocoee and Ocoee BRH-Hiwassee reaches became unstable at a time step, the other models did not. From the first unstable time step noted in Table 12, the BRH-Hiwassee reach became unstable much quicker than the Nottely-Ocoee reach, and often listed very high errors or was unstable before it could list any errors. Because of this, the BRH-Hiwassee reach was deemed the most unstable and the Nottely-Ocoee reach was the second-most unstable, with Tennessee River (Guntersville) reach as the third. Again, the Tennessee (Chickamauga) and Ocoee-Tennessee reaches were too stable to draw conclusions about the effects on instability from the methods used and therefore were not listed in this analysis.

Grouping each stability method allowed better comparison between the techniques and the stability lent through the use of them. Occasionally, one reach failed quickly while another attained reliable results. In those cases, the more reliable results were considered more highly for the decision on highest stability given to the model. Since the models had a similar slope gradient and overall reach, the results attained were able to be combined for the most accurate overall flow chart to be created for reaches similar to the BRH-Hiwassee or Nottely-Ocoee.

On the Ocoee BRH-Hiwassee reach, the minimum flow addition impressively increased the stability, to the extent at which the model was able to run for the full simulation time without crashing. The error on this reach was above the 0.020 foot guideline, but was the lowest error received from the methods of addition on the BRH-Hiwassee reach. On the Nottely-Ocoee reach, the minimum flow addition allowed for the longest simulation time, but it also yielded the highest level of error. Due to the massive change in instability created by the addition of minimum flow at the upstream boundary condition in the very unstable BRH-Hiwassee reach and the fact that the method created the longest simulation time to the Nottely-Ocoee reach, this method was deemed the most useful stability method for reaches similar to the ones used in this study.

On the Guntersville reach, the minimum flow allowed for a higher level of error; however, because the model was stable without any additions and the reach is the least similar to the other reaches, it tended to have different stability method results than the other two reaches. The Guntersville reach was therefore analyzed separately following the Nottely-Ocoee and BRH-Hiwassee reaches due to its differences.

The 100-foot cross sectional spacing on the Nottely-Ocoee reach allowed the model to run stably. It gave an error higher than that of the original model, but the original model crashed quickly. The BRH-Hiwassee reach model became unstable before errors could be listed, and crashed prior to the original model's unstable time step. Although the 100-foot spacing seemed to be more effective than other methods at increasing stability, the 100-foot spacing is placed on the flow charts as one of the last options to choose. The logic backing this decision is the amount of time and space used when 100-foot spacing is applied to a model. The quantity of the cross sections often causes significantly longer simulation times, and sometimes causes the model to crash. The option suggested instead of adding these 100-foot spaced cross sections to the entire reach is to apply the spacing only to specific areas struggling with stability. This option allows the shorter simulation time that is sought and can increase the stability in the areas of concern.

On the Nottely-Ocoee reach, the lateral inflow allowed for the lowest error and ran to a few time steps after the model in which nothing was added ran to. Using the lateral inflow method for the BRH-Hiwassee reach still created a model that crashed quickly, with a high error, since the model only reached the second time step before becoming unstable. The other stability

methods produced similar results on this reach, however, which caused the focus for this stability technique to be on the Nottely-Ocoee reach. Since the Nottely-Ocoee reach ran through a few more time steps than the original model with an error close to that of the original model, the Nottely-Ocoee reach results were taken into consideration rather than the BRH-Hiwassee reach, and it was decided that lateral inflow was the second-best method of attaining stability in the model.

From a pilot channel addition, the Nottely-Ocoee reach became unstable slightly after the nothing added original model, and the error was higher than that given in the nothing added model. On the BRH-Hiwassee reach, the pilot channel caused the model to fail immediately, giving no error for comparisons. The Samuels's spacing method also did not improve either model's stability in terms of error or time of simulation. On both reaches, the spacing was wide enough that using the Samuels's spacing method made no changes from the original model. The high levels of instability using both of these methods yielded the result that these two methods should be used in conjunction with other methods such as the minimum flow or lateral inflow conditions for the best results. It was determined that these two stability options should be placed on the same level in the flow chart, as neither of them lent enough stability to stabilize the models alone. The 100-foot spacing and lowering the time step are suggested last in the chart, as the 100-foot spacing can greatly slow the model, mentioned previously, and changing the time step allows areas of concern to be pinpointed, but should ultimately be returned to the appropriate time step interval for the reach. The final flow chart for the Nottely-Ocoee reach is displayed in Figure 10.

The Guntersville reach showed the lowest overall error when using the lateral inflow method. Since the reach did not become unstable at any point during the use of any of the methods, this was determined to be the most useful overall stabilization technique to lower the errors to an acceptable level. The 100-foot cross sectional spacing method was the second-most useful method, as it created an error only slightly higher than the lateral inflow. The pilot channel, Samuels's cross sectional spacing, and minimum flow methods for this reach overall created no change from the original model in which no additions were made, which suggests that these three methods are best used in conjunction with one of the methods listed above. For example, the Samuels's spacing method would be most effective if the 100-foot spacing was

used also or if the lateral inflow was combined with the method. The final flow chart created from these results is shown in Figure 11.

Table 11: Maximum Water Surface Elevation Error Received Before Instability

Maximum Water Surface Elevation Error Received Before Instability						
Reach	Method					
	Nothing Added	Minimum Flow	Lateral Inflow (2 Locations)	Pilot Channel	Samuels's Spacing	100-Foot Spacing
Ocoee BRH-Hiwassee	0.028	0.065	0.834	N/A ¹	0.028	N/A ¹
Hiwassee Nottely-Ocoee	0.035	0.149	0.028	0.050	0.035	0.086
Hiwassee Ocoee-TN	N/A ³	N/A ³	N/A ³	N/A ³	N/A ³	N/A ³
Tennessee River (Chickamauga)	N/A ³	N/A ³	N/A ³	N/A ³	N/A ³	N/A ³
Tennessee River (Guntersville)	0.076	0.076	0.057	0.076	0.076	0.059

¹Unstable before it could compute errors

²Unstable on first time step on which errors were listed

³Model did not have errors on the reach

Table 12: Time Step that Model Became Unstable (From Froude and Flow Analysis)

Hours of Simulation Run Until Model Became Unstable (From Froude and Flow Analysis)						
Reach	Method					
	Nothing Added	Minimum Flow	Lateral Inflow (2 Locations)	Pilot Channel	Samuels's Spacing	100-Foot Spacing
Ocoee BRH-Hiwassee	7 hours	N/A ¹	3 hours	1 hour	7 hours	1 hour
Hiwassee Nottely-Ocoee	414 hours	461 hours	417 hours	431 hours	414 hours	N/A ¹
Hiwassee Ocoee-TN	N/A ¹	N/A ¹	N/A ¹	N/A ¹	N/A ¹	N/A ¹
Tennessee River (Chickamauga)	N/A ¹	N/A ¹	N/A ¹	N/A ¹	N/A ¹	N/A ¹
Tennessee River (Guntersville)	N/A ¹	N/A ¹	N/A ¹	N/A ¹	N/A ¹	N/A ¹

¹Model did not become unstable

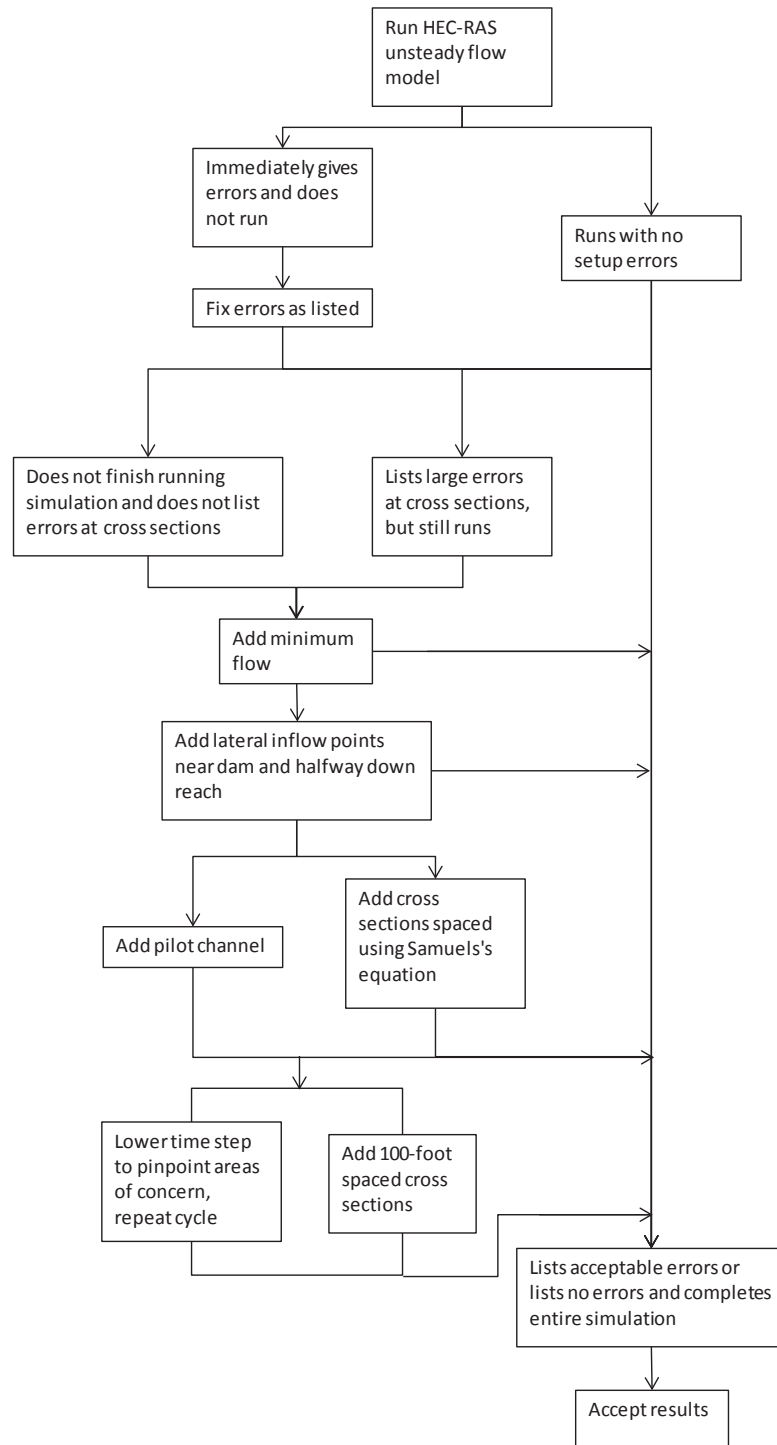


Figure 10: Stabilization Flow Chart for Reaches Similar to Hiwassee River Nottely-Ocoee Reach and Ocoee River, BRH-Hiwassee Reach

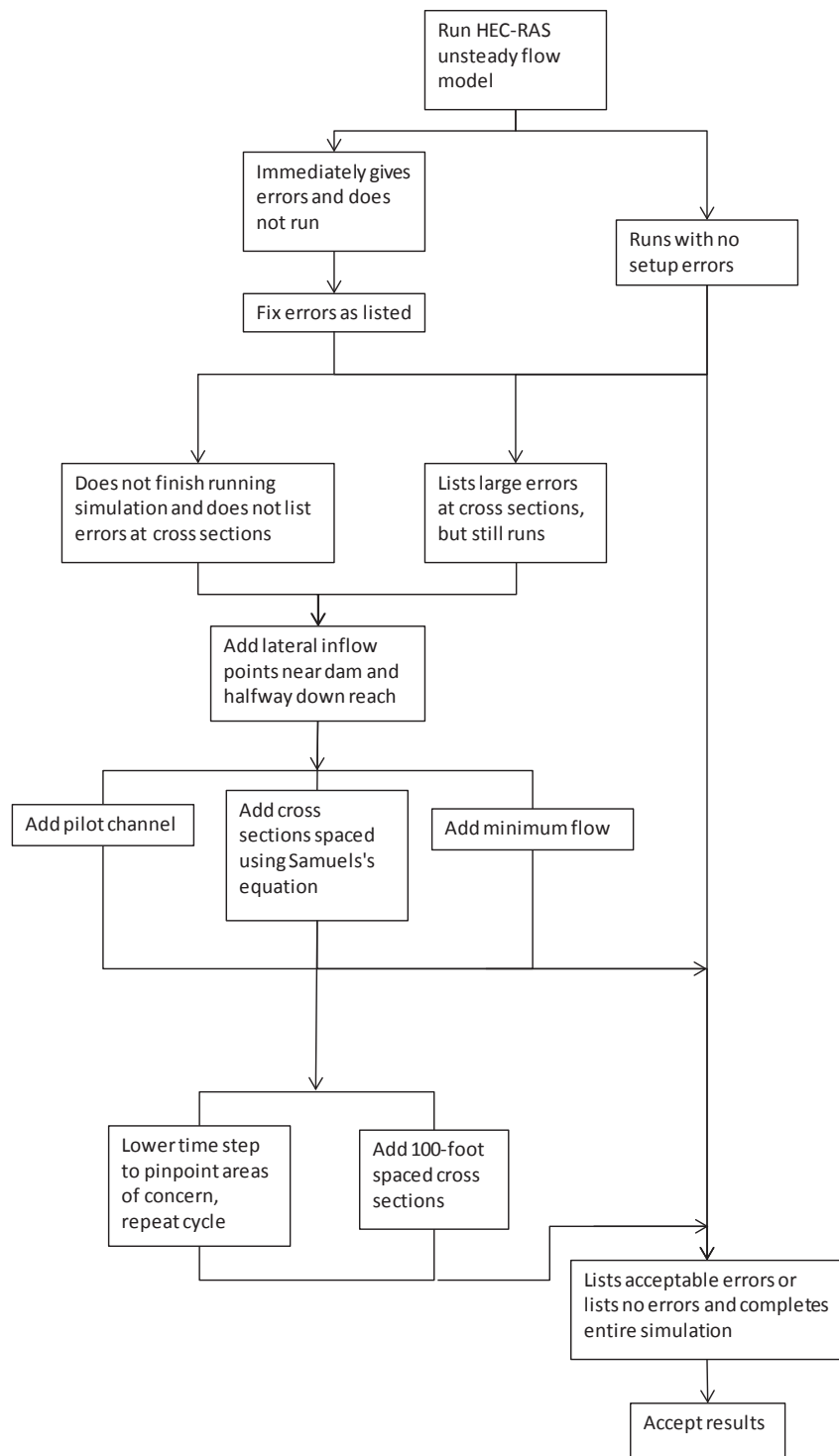


Figure 11: Stabilization Flow Chart for Reaches Similar to Tennessee River (Guntersville) Reach

Discussion

After creating the flow charts, each method was analyzed to ensure the results received were genuine, as explained below.

For the chart created from the Nottely-Ocoee and BRH-Hiwassee reaches, the reason that minimum flows allow the highest stability was discussed briefly in the Modeling section of this paper. The primary issue in the areas where minimum flow at a boundary condition is most effective is discussed more in depth as follows.

The primary difference in the stability method importance between the Nottely-Ocoee, BRH-Hiwassee chart and the Guntersville one is the fact that the minimum flow is in very different locations in the charts. The reason behind the variance in position is due to the stability of the Guntersville reach and the quantity of water present in the channel at all times. Since the Guntersville reach is on the Tennessee River, the amount of water in the area is much higher in magnitude than in the Apalachia and Ocoee Dam areas. The Apalachia and Ocoee water surface areas are 1,070 and 1,930 acres, respectively, while Guntersville's water surface area is 67,900 acres (TVA.gov 2014). The Apalachia and Ocoee reaches are also steeper, creating the higher velocities discussed previously and moving water more quickly, causing instability. When Apalachia Dam or Ocoee Dam experiences a zero flow boundary condition, the water has a greater effect on the area because the quantity of water available before moves away from the dam and down the reach much more quickly, causing HEC-RAS to believe the reach has "run dry." This has a much lesser effect, therefore, on the Guntersville reach, and does not change stability much. Although dam minimum flows caused the biggest stability difference in the Nottely-Ocoee and BRH-Hiwassee models, lateral inflows were shown to be helpful for these and the Guntersville model, too.

Commonly, lateral inflow was more helpful once the model was stable enough to yield results, particularly those with high levels of water surface elevation error. Since the amount of water added during a minimum inflow analysis was difficult to distinguish and the reaches in question are significantly shorter than the Chickamauga reach itself, the extra water was not removed from the models for the lateral inflow and minimum dam flow analyses. This also provides more consistency of the effectiveness of the lateral inflow and minimum dam methods. Most likely, the water that was added into these areas would be removed in a downstream reach

or a significant distance from the area in question, so it should still allow accurate results in this case. In the larger finalized models, however, the lateral inflow and minimum flow was accommodated to attain the most accurate results.

The lateral inflow addition method can provide enough water to prevent the water surface from going to critical depth when inappropriate. Critical depth through a reach can be an indicator of issues in data or cross sections. Critical depth is the flow depth where energy is at a minimum for a specific discharge (Furniss et al. 2006). When the flow goes below the critical depth, it is said to be subcritical (Furniss et al. 2006). Subcritical depth was acceptable in the models for these reaches in some instances. The issue began when the water surface elevation went quickly from supercritical to subcritical, easily noticeable from the Profile Plot in HEC-RAS. Watching the video allowed the user to swiftly check for critical depth in unsuitable areas. As mentioned previously, the Froude number was another indication of the type of flow in a reach; however, the critical depth is best used with the profile plot.

The lateral inflow method on the Guntersville reach was very effective at lowering flows because the flow was dispersed more than it would have been using a minimum flow at the upstream boundary condition. When the minimum flow became zero or very low, the lateral inflow assisted the area further down the channel from having no flow. Splitting the flow allowed more stability as the zero flow time step affected the channel and decreased the overall error by ensuring an amount of water was constantly in each area of the channel.

Additional cross sections in the Samuels's spacing were often not enough to stabilize a model alone. Using Samuels's equation, some reaches were already suited to the cross sections suggested. The 100-foot spacing, though, was more effective for both methods, as it changed the model geometry more significantly and allowed more stability by creating smaller differences in flow from cross section to cross section, lower conveyance ratios, and lower changes in velocity.

Pilot channels were the final option analyzed to mend a channel having issues with stability; however, pilot channels alone were not typically enough to stabilize the model. As discussed previously, pilot channels add a particular depth of water to the selected cross sections. This depth can add additional base flow to aid in the stabilization process.

CHAPTER V

CONCLUSIONS AND RECOMMENDATIONS

To create the flow charts shown in Figures 10 and 11, the results from this study were analyzed and summarized. The flow charts are applicable to river reaches with unsteady flow and dams that are regulated. Modelers should only make use of the chart if working on this type of reach, as a free-flowing river reach could yield much different results.

As discussed previously, different magnitudes of slopes in rivers were analyzed to compile this chart. The two higher gradient stream results – the Nottely-Ocoee and BRH-Hiwassee river reaches – generally agreed with the order in which a modeler should approach stability in a model. The lower gradient streams – the Tennessee River at Chickamauga and Guntersville and the Ocoee-Tennessee reaches – were stabilized after all inputs were entered correctly and run with Guntersville providing errors above the suggested value. Although two of the lower gradient streams did not allow for an analysis of the model stability options, these reaches support a valuable point involving the flow charts. The discharge through these areas is different than those on the Nottely-Ocoee and BRH-Hiwassee reaches because they have a significant amount of water the majority of the time. When Watts Bar Dam was not generating, the pool elevation was still high enough that the HEC-RAS model registered that there was water in the reach. In the Ocoee-Tennessee reach, the water inflowing from Apalachia and Ocoee 1 Dams was consistently at a level high enough to allow the reach water for HEC-RAS to continue running. At the Ocoee 1 and Apalachia Dams, however, the slope is higher, the quantity of water in the area is lower, and the reaches are slightly longer than that from Watts Bar downstream to Chickamauga. These reaches, in HEC-RAS, are more likely to “run out of water” because they are not on the wide Tennessee River, which allows steadier, slower flows, a stage high enough for the HEC-RAS model to register that water remains in the reach at times of zero generation, and more overall model stability. This further proves the difference between importance in the minimum flow addition to the models. It also suggests the significance of lateral inflows when analyzing model stability. Without any water in the reaches at certain times, the HEC-RAS model cannot run a truthful simulation.

The flow charts that are the contribution from this document are important to river modeling due to the fact that they include methodologies leading a modeler of a regulated river system through the stabilization process for low flow modeling. Low flow modeling can be difficult due to the little amount of water sometimes flowing through the reaches and the instability issues the low flows can cause in the HEC-RAS modeling system. There are many aspects to consider when attempting to stabilize a failing model, and the flow charts created can assist a modeler in attaining more accurate results more quickly.

The best use of the flow charts are achieved after researching and understanding the inputs in a model and gaining some ability to recognize where issues may be stemming from. The flow charts aid with the understanding of the instabilities through identifying where the biggest errors could be coming from and noting how to fix them. When using the flow charts, a modeler should be sure to check flows and errors to make sure the answers from the output seem reasonable and are not over- or understating the results due to the addition of inflows or other model stability practices. When viewing flows to ensure about authenticity of the results, selection of a few time steps in a row to compare flows from cross section to cross section and analyze if the results indicate realistic magnitudes through each area is an option. When observed data is available, it should be used to weigh against the simulated data post-additions.

LIST OF REFERENCES

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APPENDIX

Data Tables

Table 13: Specific Additions Made- Minimum Flow at Upstream Boundary Condition

Minimum Flow at Upstream Boundary Condition	
Area	Quantity of Added Flow, cfs
BRH-Hiwassee	750
Nottely-Ocoee	300

Table 14: Specific Additions Made- Lateral Inflow at 2 Locations

Lateral Inflow at 2 Locations		
Area	Cross Section	Quantity of Added Flow, cfs
BRH-Hiwassee	Ocoee River Mile 11.15	325 cfs
	Ocoee River Mile 6.6	325 cfs
Nottely-Ocoee	Hiwassee River Mile 65.4	150 cfs
	Hiwassee River Mile 50.51	150 cfs

Table 15: Specific Additions Made- Pilot Channel Addition

Pilot Channel		
Area	Cross Section	Pilot Channel Lower Elevation, ft
BRH-Hiwassee	Ocoee River Mile 11.8790	715
	Ocoee River Mile 0.05	677
Nottely-Ocoee	Hiwassee River Mile 65.99	1160
	Hiwassee River Mile 55.6	800
	Hiwassee River Mile 44.12	675
	Hiwassee River Mile 36.1	670

*1 foot width, .035 roughness value used

Table 16: Specific Additions Made- Samuels's Equation

Samuels's Equation		
Area	Bankfull Depth, ft	Recommended Spacing, ft
BRH-Hiwassee	166	57,241
Nottely-Ocoee	175	11,041

Table 17: Specific Additions Made- Tennessee River (Guntersville) Reach

Guntersville		
Stability Method Used	Method Addition Description	
Minimum Flow	Quantity of Added Flow, cfs	
	14,400	
Lateral Inflow at 2 Locations	Cross Section	Quantity of Added Flow, cfs
	Tennessee River Mile 420.49	7,000
	Tennessee River Mile 378.44	7,000
Pilot Channel ¹	Cross Section	Pilot Channel Lower Elevation, ft
	Tennessee River Mile 424.69	570
	Tennessee River Mile 349.01	525
Samuels's	Bankfull Depth, ft	Recommended Spacing, ft
	1,325	2,650,000

¹1 foot width, .035 roughness value used

Figures

Final Model Guntersville Figures

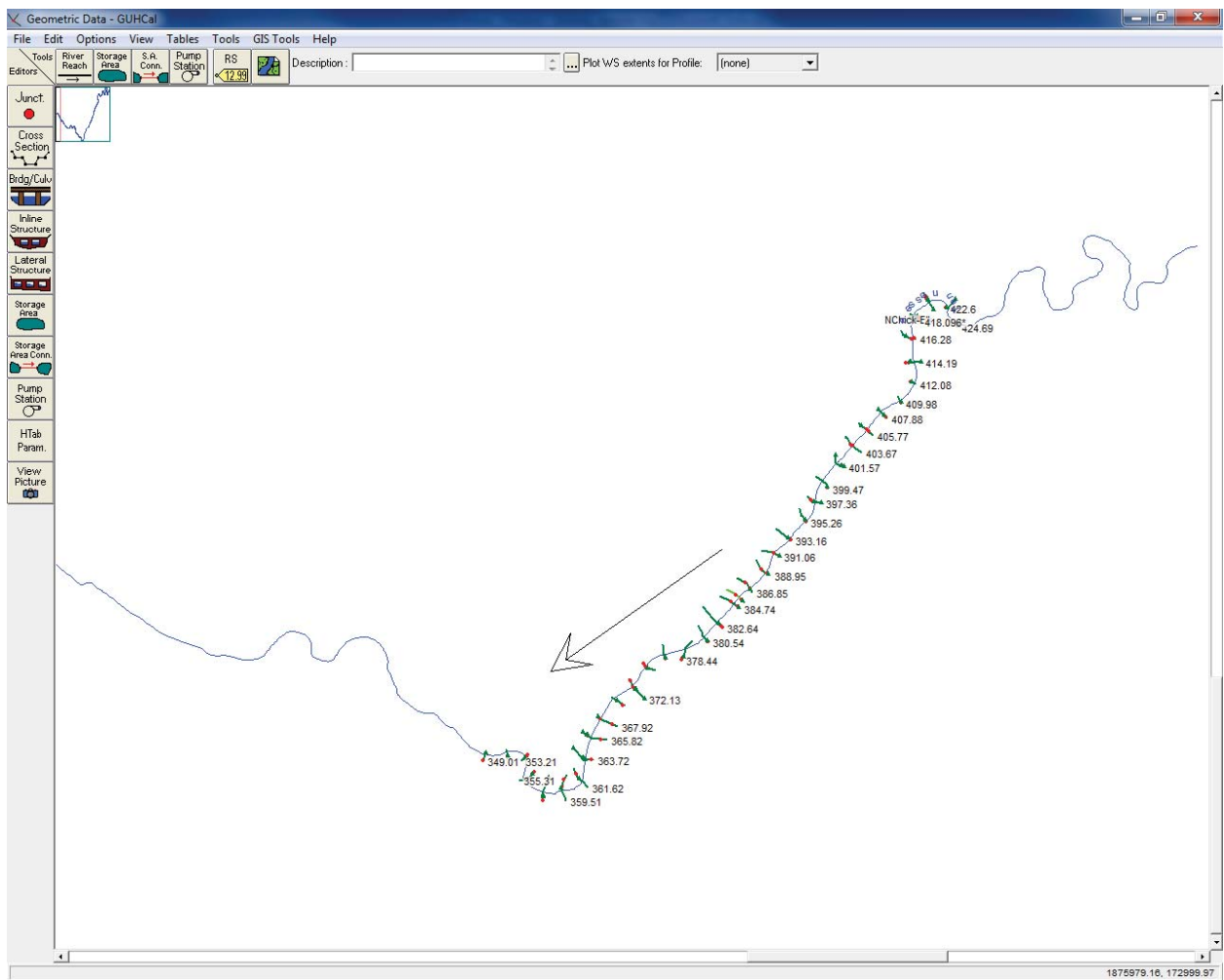


Figure 12: Final Model Guntersville Cross Sectional Layout

Unsteady Flow Data - GUHCal

File Options Help

Boundary Conditions Initial Conditions Apply Data

Boundary Condition Types

Stage Hydrograph	Flow Hydrograph	Stage/Flow Hydr.	Rating Curve
Normal Depth	Lateral Inflow Hydr.	Uniform Lateral Inflow	Groundwater Interflow
T.S. Gate Openings	Elev Controlled Gates	Navigation Dams	IB Stage/Flow
Rules			

Add Boundary Condition Location

Add RS ... Add Storage Area ... Add SA Connection ... Add Pump Station ...

Select Location in table then select Boundary Condition Type

	River	Reach	RS	Boundary Condition
1	Tennessee	NChick-Elk	424.69	Flow Hydrograph
2	Tennessee	NChick-Elk	424.68*	Lateral Inflow Hydr.
3	Tennessee	NChick-Elk	424.67	Lateral Inflow Hydr.
4	Tennessee	NChick-Elk	422.6	Uniform Lateral Inflow
5	Tennessee	NChick-Elk	401.57	Lateral Inflow Hydr.
6	Tennessee	NChick-Elk	391.06	Uniform Lateral Inflow
7	Tennessee	NChick-Elk	363.72	Lateral Inflow Hydr.
8	Tennessee	NChick-Elk	349.01	Stage Hydrograph

Observed DSS data set

Figure 13: Final Model Guntersville Boundary Conditions

055 Set Locations and Paths for Observed Data in DSS

River: Tennessee Delete row from table

Reach: NChick-Elk River Sta.: 424.69 Add selected location to table

	River	Reach	RS	Dn Dist	DSS File	Part A	Part B	Part C	Part D	Pa
1	Tennessee	NChick-Elk	424.69		z:\Work\Flood Ri	NICKAJACK	HT	ELEVATION	01JAN1976	1HOUR
2	Tennessee	NChick-Elk	418.096*		z:\Work\Flood Ri		TENN@SOUTH_PITTSB	ELEVATION	01OCT1998	1HOUR
3	Tennessee	NChick-Elk	385.507*		z:\Work\Flood Ri		TENN_NR_SCOTTSBORO	ELEVATION	01DEC1985	1HOUR
4	Tennessee	NChick-Elk	349.01		z:\Work\Flood Ri	GUNTERSVILLE	29	FLOW	01JAN1976	1HOUR

DSS File: z:\Work\Flood Risk\2014 HEC-RAS Operational Mode\NRAS Update Catalog

	Part A	Part B	Part C	Part D	Part E	Part F
Filter						
1	APALACHIA	HT	ELEVATION	01SEP2003	1HOUR	OBSERVED
2	APALACHIA	HT	ELEVATION	01OCT2003	1HOUR	OBSERVED
3	APALACHIA	HT	ELEVATION	01NOV2003	1HOUR	OBSERVED
4	APALACHIA	HT	ELEVATION	01DEC2003	1HOUR	OBSERVED
5	APALACHIA	HT	ELEVATION	01JAN2004	1HOUR	OBSERVED
6	APALACHIA	HT	ELEVATION	01FEB2004	1HOUR	OBSERVED
7	APALACHIA	HT	ELEVATION	01MAR2004	1HOUR	OBSERVED
8	APALACHIA	HT	ELEVATION	01APR2004	1HOUR	OBSERVED

Select DSS Pathname

Plot Selected Pathname

OK Cancel

Figure 14: Final Model Guntersville Observed Data

Lateral Inflow Hydrograph

River: Tennessee Reach: NChick-Elk RS: 424.67

☐ Read from DSS before simulation Select DSS file and Path

File:

Path:

☒ Enter Table Data time interval: 1 Year

Select/Enter the Data's Starting Time Reference

☒ Use Simulation Time: Date: 01OCT1998 Time:

☐ Fixed Start Time: Date: Time:

No. Ordinates Interpolate Missing Values Del Row Ins Row

Hydrograph Data			
	Date	Simulation Time	Lateral Inflow
		(hours)	(cfs)
1	30Sep1998 2400	00:00	1000.
2	30Sep1999 2400	8760:00	1000.
3	30Sep2000 2400	17520:00	1000.
4	30Sep2001 2400	26280:00	1000.
5	30Sep2002 2400	35040:00	1000.
6	30Sep2003 2400	43824:00	1000.
7	30Sep2004 2400	52584:00	1000.
8	30Sep2005 2400	61344:00	1000.
9	30Sep2006 2400	70104:00	1000.
10	30Sep2007 2400	78888:00	1000.
11	30Sep2008 2400	87648:00	1000.
12	30Sep2009 2400	96408:00	1000.
13	30Sep2010 2400	105168:00	1000.
14	30Sep2011 2400	113952:00	1000.
15	30Sep2012 2400	122712:00	1000.
16	30Sep2013 2400	131472:00	1000.
17	30Sep2014 2400	140232:00	1000.
18	30Sep2015 2400	149016:00	1000.

Time Step Adjustment Options ("Critical" boundary conditions)

☐ Monitor this hydrograph for adjustments to computational time step

Max Change in Flow (without changing time step):

Min Flow: Multiplier:

Plot Data OK Cancel

Figure 15: Lateral Inflow Addition for Final Guntersville Model at Tennessee River Mile 424.67

Lateral Inflow Hydrograph

River: Tennessee Reach: NChick-Elk RS: 401.57

☐ Read from DSS before simulation Select DSS file and Path

File:

Path:

☒ Enter Table Data time interval: 1 Year

Select/Enter the Data's Starting Time Reference

☒ Use Simulation Time: Date: 01OCT1998 Time:

☐ Fixed Start Time: Date: Time:

No. Ordinates Interpolate Missing Values Del Row Ins Row

Hydrograph Data			
	Date	Simulation Time	Lateral Inflow
		(hours)	(cfs)
1	30Sep1998 2400	00:00	-1000.
2	30Sep1999 2400	8760:00	-1000.
3	30Sep2000 2400	17520:00	-1000.
4	30Sep2001 2400	26280:00	-1000.
5	30Sep2002 2400	35040:00	-1000.
6	30Sep2003 2400	43824:00	-1000.
7	30Sep2004 2400	52584:00	-1000.
8	30Sep2005 2400	61344:00	-1000.
9	30Sep2006 2400	70104:00	-1000.
10	30Sep2007 2400	78888:00	-1000.
11	30Sep2008 2400	87648:00	-1000.
12	30Sep2009 2400	96408:00	-1000.
13	30Sep2010 2400	105168:00	-1000.
14	30Sep2011 2400	113952:00	-1000.
15	30Sep2012 2400	122712:00	-1000.
16	30Sep2013 2400	131472:00	-1000.
17	30Sep2014 2400	140232:00	-1000.
18	30Sep2015 2400	149016:00	-1000.

Time Step Adjustment Options ("Critical" boundary conditions)

☐ Monitor this hydrograph for adjustments to computational time step

Max Change in Flow (without changing time step):

Min Flow: Multiplier:

Plot Data OK Cancel

Figure 16: Lateral Inflow Removal for Final Guntersville Model at Tennessee River Mile 401.57

Flow Hydrograph

River: Tennessee Reach: NChick-Elk RS: 424.69

☒ Read from DSS before simulation Select DSS file and Path

File: z:\Work\Flood Risk\2014 HEC-RAS Operational Model\RAS\Observed
 Path: /NICKAJACK/28/FLOW/01JAN1976/1HOUR/OBSERVED_DISCHARGE

☐ Enter Table Data time interval: 1 Hour

Select/Enter the Data's Starting Time Reference

☒ Use Simulation Time: Date: 01OCT1998 Time:

☐ Fixed Start Time: Date: Time:

No. Ordinates Interpolate Missing Values Del Row Ins Row

Hydrograph Data			
	Date	Simulation Time	Flow
		(hours)	(cfs)
1	30Sep1998 2400	00:00	
2	01Oct1998 0100	01:00	
3	01Oct1998 0200	02:00	
4	01Oct1998 0300	03:00	
5	01Oct1998 0400	04:00	
6	01Oct1998 0500	05:00	
7	01Oct1998 0600	06:00	
8	01Oct1998 0700	07:00	
9	01Oct1998 0800	08:00	
10	01Oct1998 0900	09:00	
11	01Oct1998 1000	10:00	
12	01Oct1998 1100	11:00	
13	01Oct1998 1200	12:00	
14	01Oct1998 1300	13:00	
15	01Oct1998 1400	14:00	
16	01Oct1998 1500	15:00	
17	01Oct1998 1600	16:00	
18	01Oct1998 1700	17:00	

Time Step Adjustment Options ("Critical" boundary conditions)

☐ Monitor this hydrograph for adjustments to computational time step

Max Change in Flow (without changing time step):

Min Flow: 500 Multiplier:

Plot Data OK Cancel

Figure 17: Minimum Flow Addition for Final Guntersville Model at Nickajack Dam

Final Model Chickamauga Figures

Flow Hydrograph

River: Hiwassee Reach: Nottely-Ocoee RS: 65.99

☒ Read from DSS before simulation Select DSS file and Path

File: z:\Work\Flood Risk\2014 HEC-RAS Operational Model\RAS\Observe
 Path: /APALACHIA/22/FLOW/01JAN1976/1HOUR/OBSERVED_DISCHA

☐ Enter Table Data time interval: 1 Hour

Select/Enter the Data's Starting Time Reference

☒ Use Simulation Time: Date: 02JUL1998 Time:
☐ Fixed Start Time: Date: Time:

No. Ordinates Interpolate Missing Values Del Row Ins Row

Hydrograph Data			
	Date	Simulation Time	Flow
		(hours)	(cfs)
1	01Jul1998 2400	00:00	
2	02Jul1998 0100	01:00	
3	02Jul1998 0200	02:00	
4	02Jul1998 0300	03:00	
5	02Jul1998 0400	04:00	
6	02Jul1998 0500	05:00	
7	02Jul1998 0600	06:00	
8	02Jul1998 0700	07:00	
9	02Jul1998 0800	08:00	
10	02Jul1998 0900	09:00	
11	02Jul1998 1000	10:00	
12	02Jul1998 1100	11:00	
13	02Jul1998 1200	12:00	
14	02Jul1998 1300	13:00	
15	02Jul1998 1400	14:00	
16	02Jul1998 1500	15:00	
17	02Jul1998 1600	16:00	
18	02Jul1998 1700	17:00	
19	02Jul1998 1800	18:00	

Time Step Adjustment Options ("Critical" boundary conditions)

☐ Monitor this hydrograph for adjustments to computational time step

Max Change in Flow (without changing time step):

Min Flow: 600 Multiplier:

Plot Data OK Cancel

Figure 18: Minimum Flow Addition for Final Chickamauga Model at Apalachia Dam

Flow Hydrograph

River: Ocoee Reach: BRH-Hiwassee RS: 11.88

☒ Read from DSS before simulation Select DSS file and Path

File: z:\Work\Flood Risk\2014 HEC-RAS Operational Model\RAS\Observed
 Path: /OCOEEEN01/26/FLOW/01JAN1976/1HOUR/OBSERVED_DISCHARGE

☐ Enter Table Data time interval: 1 Hour

Select/Enter the Data's Starting Time Reference

☒ Use Simulation Time: Date: 02JUL1998 Time:

☐ Fixed Start Time: Date: Time:

No. Ordinates Interpolate Missing Values Del Row Ins Row

Hydrograph Data			
	Date	Simulation Time	Flow
		(hours)	(cfs)
1	01Jul1998 2400	00:00	
2	02Jul1998 0100	01:00	
3	02Jul1998 0200	02:00	
4	02Jul1998 0300	03:00	
5	02Jul1998 0400	04:00	
6	02Jul1998 0500	05:00	
7	02Jul1998 0600	06:00	
8	02Jul1998 0700	07:00	
9	02Jul1998 0800	08:00	
10	02Jul1998 0900	09:00	
11	02Jul1998 1000	10:00	
12	02Jul1998 1100	11:00	
13	02Jul1998 1200	12:00	
14	02Jul1998 1300	13:00	
15	02Jul1998 1400	14:00	
16	02Jul1998 1500	15:00	
17	02Jul1998 1600	16:00	
18	02Jul1998 1700	17:00	
19	02Jul1998 1800	18:00	

Time Step Adjustment Options ("Critical" boundary conditions)

☐ Monitor this hydrograph for adjustments to computational time step

Max Change in Flow (without changing time step):

Min Flow: 500 Multiplier:

Plot Data OK Cancel

Figure 19: Minimum Flow Addition for Final Chickamauga Model at Ocoee 1 Dam

Flow Hydrograph

River: Tennessee Reach: Clinch-Hiwassee RS: 529.89

☒ Read from DSS before simulation Select DSS file and Path

File: z:\Work\Flood Risk\2014 HEC-RAS Operational Model\RAS\Observed
 Path: /WATTSBAR/18/FLOW/01JAN1976/1HOUR/OBSERVED_DISCHARGE

☐ Enter Table Data time interval: 1 Hour

Select/Enter the Data's Starting Time Reference

☒ Use Simulation Time: Date: 02JUL1998 Time:

☐ Fixed Start Time: Date: Time:

No. Ordinates Interpolate Missing Values Del Row Ins Row

Hydrograph Data			
	Date	Simulation Time	Flow
		(hours)	(cfs)
1	01Jul1998 2400	00:00	
2	02Jul1998 0100	01:00	
3	02Jul1998 0200	02:00	
4	02Jul1998 0300	03:00	
5	02Jul1998 0400	04:00	
6	02Jul1998 0500	05:00	
7	02Jul1998 0600	06:00	
8	02Jul1998 0700	07:00	
9	02Jul1998 0800	08:00	
10	02Jul1998 0900	09:00	
11	02Jul1998 1000	10:00	
12	02Jul1998 1100	11:00	
13	02Jul1998 1200	12:00	
14	02Jul1998 1300	13:00	
15	02Jul1998 1400	14:00	
16	02Jul1998 1500	15:00	
17	02Jul1998 1600	16:00	
18	02Jul1998 1700	17:00	
19	02Jul1998 1800	18:00	

Time Step Adjustment Options ("Critical" boundary conditions)

☐ Monitor this hydrograph for adjustments to computational time step

Max Change in Flow (without changing time step):

Min Flow: 500 Multiplier:

Plot Data OK Cancel

Figure 20: Minimum Flow Addition for Final Chickamauga Model at Watts Bar Dam

Lateral Inflow Hydrograph

River: Hiwassee Reach: Nottely-Ocoee RS: 64

☐ Read from DSS before simulation Select DSS file and Path

File:

Path:

☒ Enter Table Data time interval: 1 Year

Select/Enter the Data's Starting Time Reference

☒ Use Simulation Time: Date: 02JUL1998 Time:

☐ Fixed Start Time: Date: Time:

No. Ordinates Interpolate Missing Values Del Row Ins Row

Hydrograph Data			
	Date	Simulation Time	Lateral Inflow
		(hours)	(cfs)
1	01Jul1998 2400	00:00	500.
2	01Jul1999 2400	8760:00	500.
3	01Jul2000 2400	17520:00	500.
4	01Jul2001 2400	26280:00	500.
5	01Jul2002 2400	35040:00	500.
6	01Jul2003 2400	43824:00	500.
7	01Jul2004 2400	52584:00	500.
8	01Jul2005 2400	61344:00	500.
9	01Jul2006 2400	70104:00	500.
10	01Jul2007 2400	78888:00	500.
11	01Jul2008 2400	87648:00	500.
12	01Jul2009 2400	96408:00	500.
13	01Jul2010 2400	105168:00	500.
14	01Jul2011 2400	113952:00	500.
15	01Jul2012 2400	122712:00	500.
16	01Jul2013 2400	131472:00	500.
17	01Jul2014 2400	140232:00	500.
18	01Jul2015 2400	149016:00	500.
19	01Jul2016 2400	157776:00	500.

Time Step Adjustment Options ("Critical" boundary conditions)

☐ Monitor this hydrograph for adjustments to computational time step

Max Change in Flow (without changing time step):

Min Flow: Multiplier:

Plot Data OK Cancel

Figure 21: Lateral Inflow Addition for Final Chickamauga Model at Hiwassee River Mile 64

Lateral Inflow Hydrograph

River: Hiwassee Reach: Nottely-Ocoee RS: 57.45

☐ Read from DSS before simulation Select DSS file and Path

File:

Path:

☒ Enter Table Data time interval: 1 Year

Select/Enter the Data's Starting Time Reference

☒ Use Simulation Time: Date: 02JUL1998 Time:

☐ Fixed Start Time: Date: Time:

No. Ordinates Interpolate Missing Values Del Row Ins Row

Hydrograph Data			
	Date	Simulation Time	Lateral Inflow
		(hours)	(cfs)
1	01Jul1998 2400	00:00	500.
2	01Jul1999 2400	8760:00	500.
3	01Jul2000 2400	17520:00	500.
4	01Jul2001 2400	26280:00	500.
5	01Jul2002 2400	35040:00	500.
6	01Jul2003 2400	43824:00	500.
7	01Jul2004 2400	52584:00	500.
8	01Jul2005 2400	61344:00	500.
9	01Jul2006 2400	70104:00	500.
10	01Jul2007 2400	78888:00	500.
11	01Jul2008 2400	87648:00	500.
12	01Jul2009 2400	96408:00	500.
13	01Jul2010 2400	105168:00	500.
14	01Jul2011 2400	113952:00	500.
15	01Jul2012 2400	122712:00	500.
16	01Jul2013 2400	131472:00	500.
17	01Jul2014 2400	140232:00	500.
18	01Jul2015 2400	149016:00	500.
19	01Jul2016 2400	157776:00	500.

Time Step Adjustment Options ("Critical" boundary conditions)

☐ Monitor this hydrograph for adjustments to computational time step

Max Change in Flow (without changing time step):

Min Flow: Multiplier:

Plot Data OK Cancel

Figure 22: Lateral Inflow Addition for Final Chickamauga Model at Hiwassee River Mile 57.45

Lateral Inflow Hydrograph

River: Ocoee Reach: BRH-Hiwassee RS: 11.8770

☐ Read from DSS before simulation Select DSS file and Path

File:

Path:

☒ Enter Table Data time interval: 1 Year

Select/Enter the Data's Starting Time Reference

☒ Use Simulation Time: Date: 02JUL1998 Time:

☐ Fixed Start Time: Date: Time:

No. Ordinates Interpolate Missing Values Del Row Ins Row

Hydrograph Data			
	Date	Simulation Time	Lateral Inflow
		(hours)	(cfs)
1	01Jul1998 2400	00:00	1000.
2	01Jul1999 2400	8760:00	1000.
3	01Jul2000 2400	17520:00	1000.
4	01Jul2001 2400	26280:00	1000.
5	01Jul2002 2400	35040:00	1000.
6	01Jul2003 2400	43824:00	1000.
7	01Jul2004 2400	52584:00	1000.
8	01Jul2005 2400	61344:00	1000.
9	01Jul2006 2400	70104:00	1000.
10	01Jul2007 2400	78888:00	1000.
11	01Jul2008 2400	87648:00	1000.
12	01Jul2009 2400	96408:00	1000.
13	01Jul2010 2400	105168:00	1000.
14	01Jul2011 2400	113952:00	1000.
15	01Jul2012 2400	122712:00	1000.
16	01Jul2013 2400	131472:00	1000.
17	01Jul2014 2400	140232:00	1000.
18	01Jul2015 2400	149016:00	1000.
19	01Jul2016 2400	157776:00	1000.

Time Step Adjustment Options ("Critical" boundary conditions)

☐ Monitor this hydrograph for adjustments to computational time step

Max Change in Flow (without changing time step):

Min Flow: Multiplier:

Plot Data OK Cancel

Figure 23: Lateral Inflow Addition for Final Chickamauga Model at Ocoee River Mile 11.8770

Lateral Inflow Hydrograph

River: Hiwassee Reach: Ocoee-Tenn RS: 19.36

☐ Read from DSS before simulation Select DSS file and Path

File:

Path:

☒ Enter Table Data time interval: 1 Year

Select/Enter the Data's Starting Time Reference

☒ Use Simulation Time: Date: 02JUL1998 Time:

☐ Fixed Start Time: Date: Time:

No. Ordinates Interpolate Missing Values Del Row Ins Row

Hydrograph Data			
	Date	Simulation Time	Lateral Inflow
		(hours)	(cfs)
1	01Jul1998 2400	00:00	-2000.
2	01Jul1999 2400	8760:00	-2000.
3	01Jul2000 2400	17520:00	-2000.
4	01Jul2001 2400	26280:00	-2000.
5	01Jul2002 2400	35040:00	-2000.
6	01Jul2003 2400	43824:00	-2000.
7	01Jul2004 2400	52584:00	-2000.
8	01Jul2005 2400	61344:00	-2000.
9	01Jul2006 2400	70104:00	-2000.
10	01Jul2007 2400	78888:00	-2000.
11	01Jul2008 2400	87648:00	-2000.
12	01Jul2009 2400	96408:00	-2000.
13	01Jul2010 2400	105168:00	-2000.
14	01Jul2011 2400	113952:00	-2000.
15	01Jul2012 2400	122712:00	-2000.
16	01Jul2013 2400	131472:00	-2000.
17	01Jul2014 2400	140232:00	-2000.
18	01Jul2015 2400	149016:00	-2000.
19	01Jul2016 2400	157776:00	-2000.

Time Step Adjustment Options ("Critical" boundary conditions)

☐ Monitor this hydrograph for adjustments to computational time step

Max Change in Flow (without changing time step):

Min Flow: Multiplier:

Plot Data OK Cancel

Figure 24: Lateral Inflow Removal for Final Chickamauga Model at Hiwassee River Mile 19.36

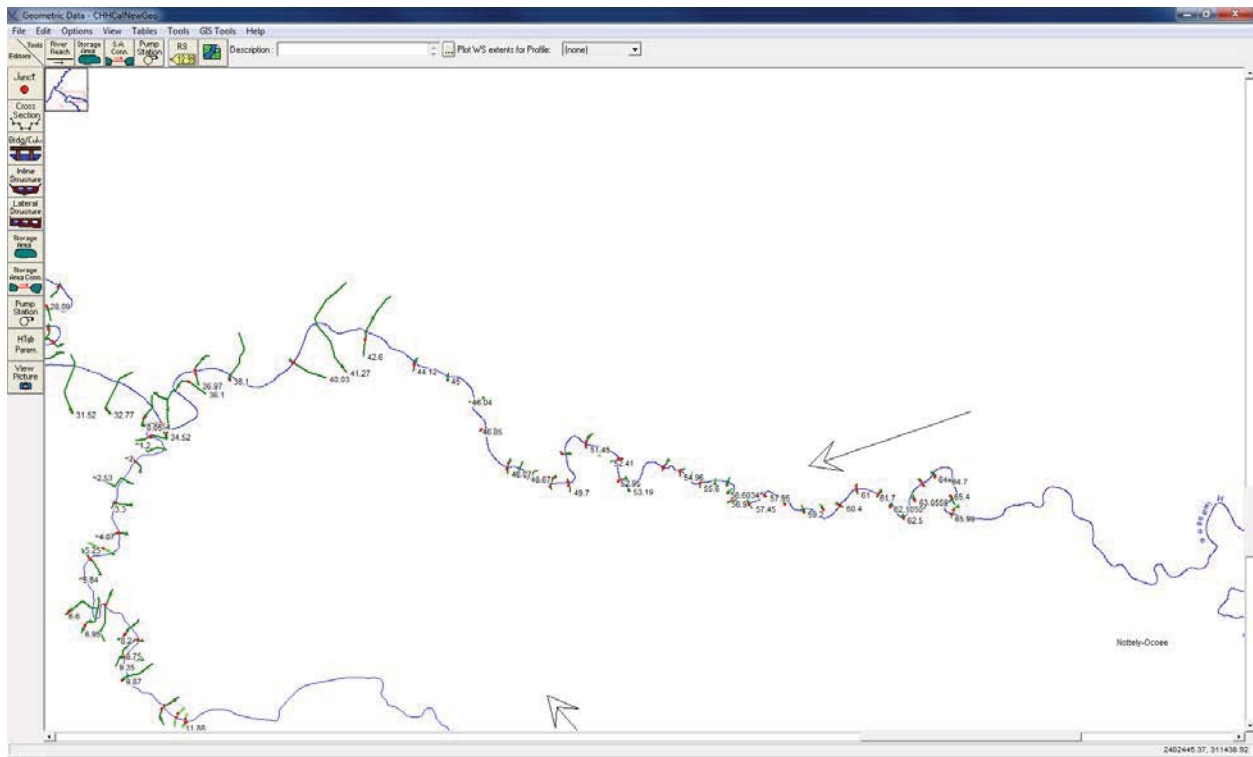


Figure 25: Final Chickamauga Model Hiwassee River Nottely-Ocoee Reach Cross Sectional Additions

Unsteady Flow Data - CHHCal

File Options Help

Boundary Conditions Initial Conditions Apply Data

Boundary Condition Types

Stage Hydrograph Flow Hydrograph Stage/Flow Hydr. Rating Curve

Normal Depth Lateral Inflow Hydr. Uniform Lateral Inflow Groundwater Interflow

T.S. Gate Openings Elev Controlled Gates Navigation Dams IB Stage/Flow

Rules

Add Boundary Condition Location

Add RS ... Add Storage Area ... Add SA Connection ... Add Pump Station ...

Select Location in table then select Boundary Condition Type

	River	Reach	RS	Boundary Condition
1	Hiwassee	Nottely-Ocoee	65.99	Flow Hydrograph
2	Hiwassee	Nottely-Ocoee	65.9882*	Uniform Lateral Inflow
3	Hiwassee	Nottely-Ocoee	64	Lateral Inflow Hydr.
4	Hiwassee	Nottely-Ocoee	57.45	Lateral Inflow Hydr.
5	Hiwassee	Ocoee-Tenn	19.36	Lateral Inflow Hydr.
6	Hiwassee	Ocoee-Tenn	18.9	Uniform Lateral Inflow
7	Ocoee	BRH-Hiwassee	11.88	Flow Hydrograph
8	Ocoee	BRH-Hiwassee	11.8780*	Uniform Lateral Inflow
9	Ocoee	BRH-Hiwassee	11.8770	Lateral Inflow Hydr.
10	Tennessee	Clinch-Hiwassee	529.89	Flow Hydrograph
11	Tennessee	Clinch-Hiwassee	529.888*	Uniform Lateral Inflow
12	Tennessee	Clinch-Hiwassee	525.69	Lateral Inflow Hydr.
13	Tennessee	Hiwassee-Dallas	499.495	Uniform Lateral Inflow
14	Tennessee	Hiwassee-Dallas	471.01	Stage Hydrograph

Observed DSS data set

Figure 26: Final Model Chickamauga Boundary Conditions

055 Set Locations and Paths for Observed Data in DSS

River:

Reach: River Sta.:

	River	Reach	RS	Dn Dist	DSS File	Part A	Part B	Part C	
1	Hiwassee	Nottely-Ocoee	65.99		z:\Work\Flood Ris	APALACHIA	HT	ELEVATION	01SEP
2	Hiwassee	Ocoee-Tenn	18.2018*		z:\Work\Flood Ris		HIWASSEE@CHARLESTON	ELEVATION	01JUL
3	Ocoee	BRH-Hiwassee	11.88		z:\Work\Flood Ris	OCDEEND1	HT	ELEVATION	01JUL
4	Tennessee	Clinch-Hiwassee	529.89		z:\Work\Flood Ris	WATTSBAR	HT	ELEVATION	01JAN
5	Tennessee	Hiwassee-Dallas	471.01		z:\Work\Flood Ris	CHICKAMAUGA	27	FLOW	01JAN

DSS File:

	Part A	Part B	Part C	Part D	Part E	Part F
Filter						
1	APALACHIA	HT	ELEVATION	01SEP2003	1HOUR	OBSERVED
2	APALACHIA	HT	ELEVATION	01OCT2003	1HOUR	OBSERVED
3	APALACHIA	HT	ELEVATION	01NOV2003	1HOUR	OBSERVED
4	APALACHIA	HT	ELEVATION	01DEC2003	1HOUR	OBSERVED
5	APALACHIA	HT	ELEVATION	01JAN2004	1HOUR	OBSERVED
6	APALACHIA	HT	ELEVATION	01FEB2004	1HOUR	OBSERVED
7	APALACHIA	HT	ELEVATION	01MAR2004	1HOUR	OBSERVED
8	APALACHIA	HT	ELEVATION	01APR2004	1HOUR	OBSERVED

Select DSS Pathname

Plot Selected Pathname

Figure 27: Final Model Chickamauga Observed Data

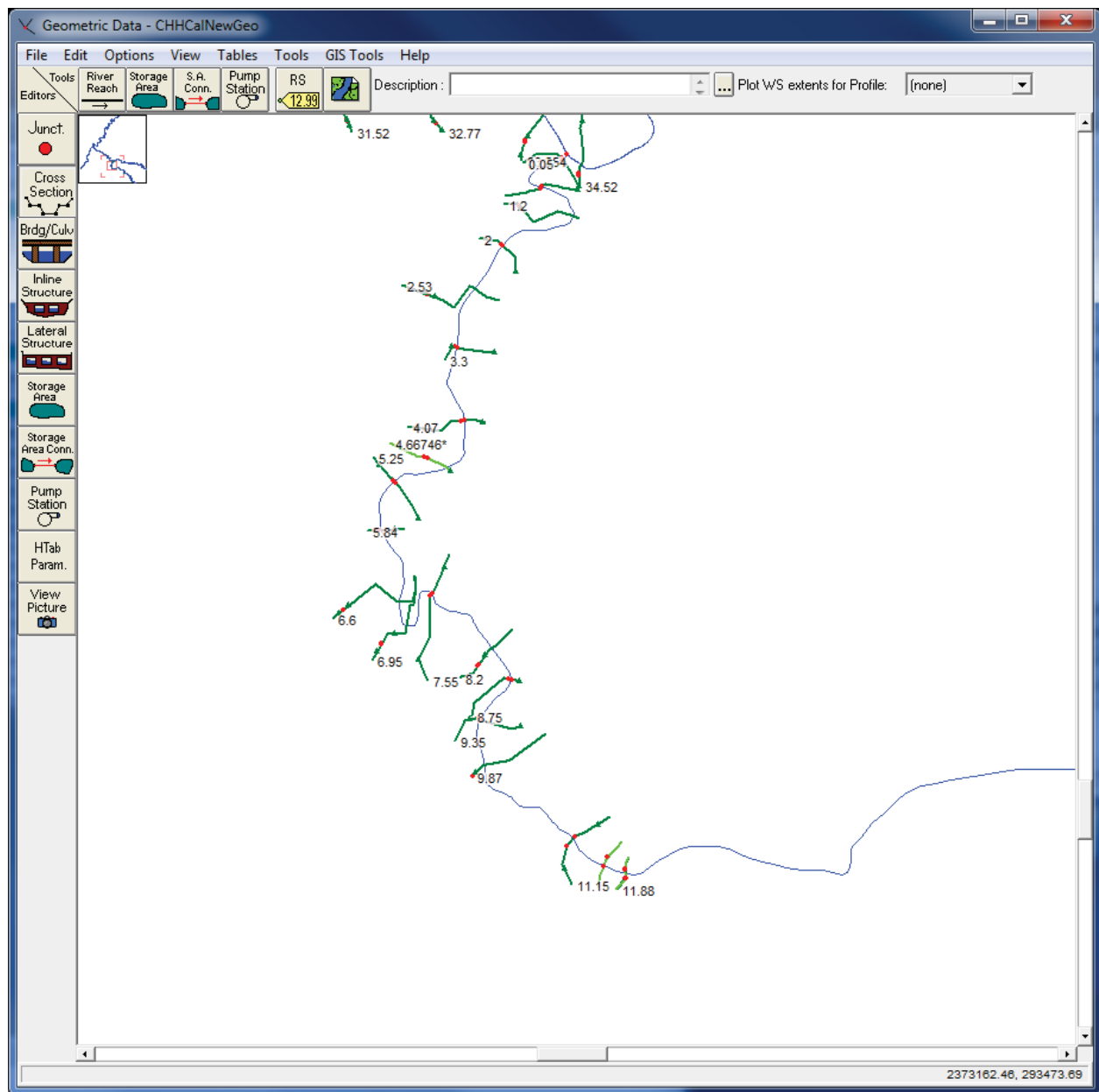


Figure 28: Final Chickamauga Model Ocoee River BRH-Hiwassee Reach Cross Sectional Additions

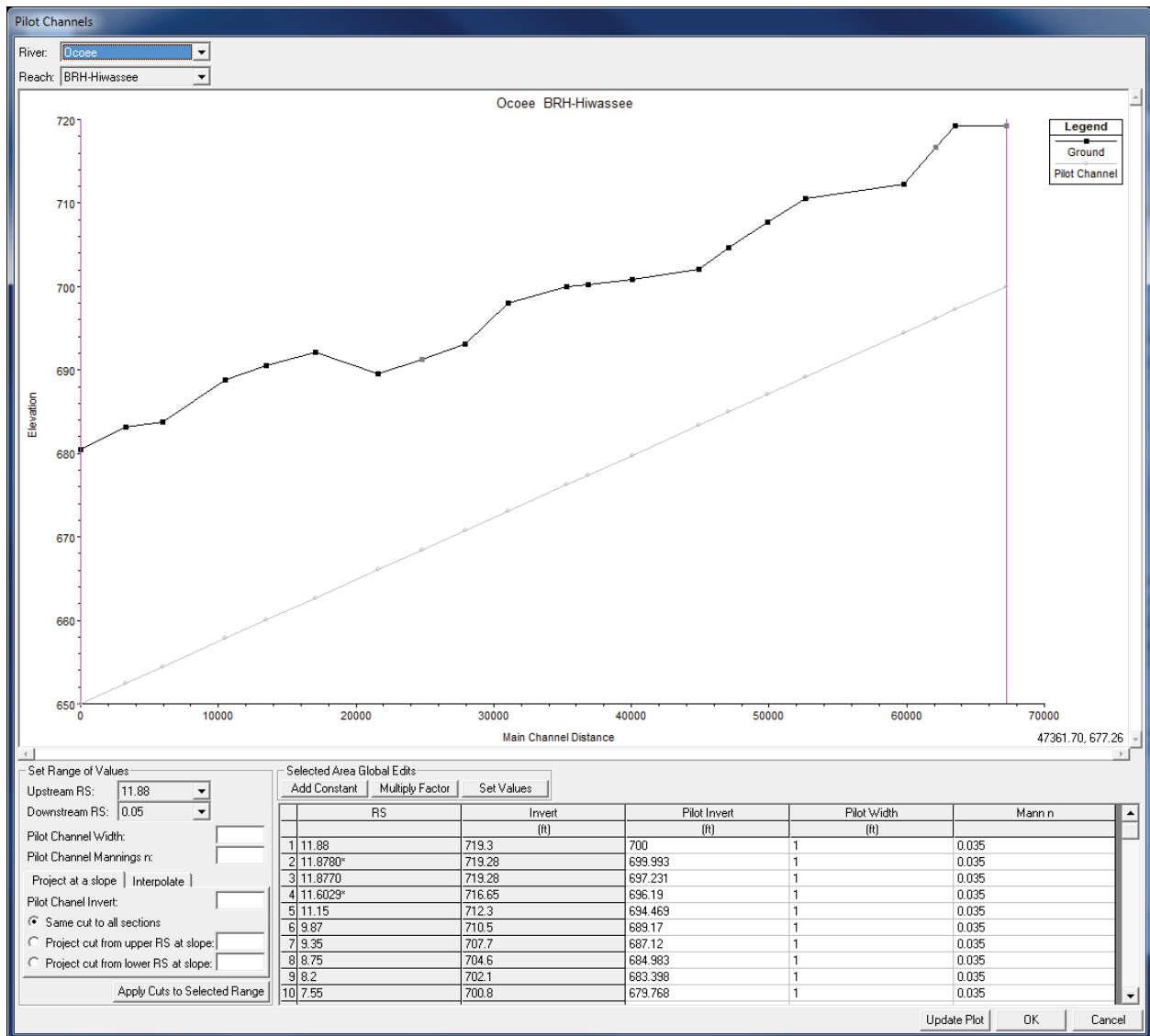


Figure 29: Final Chickamauga Model Ocoee River BRH-Hiwassee Reach Pilot Channel Addition

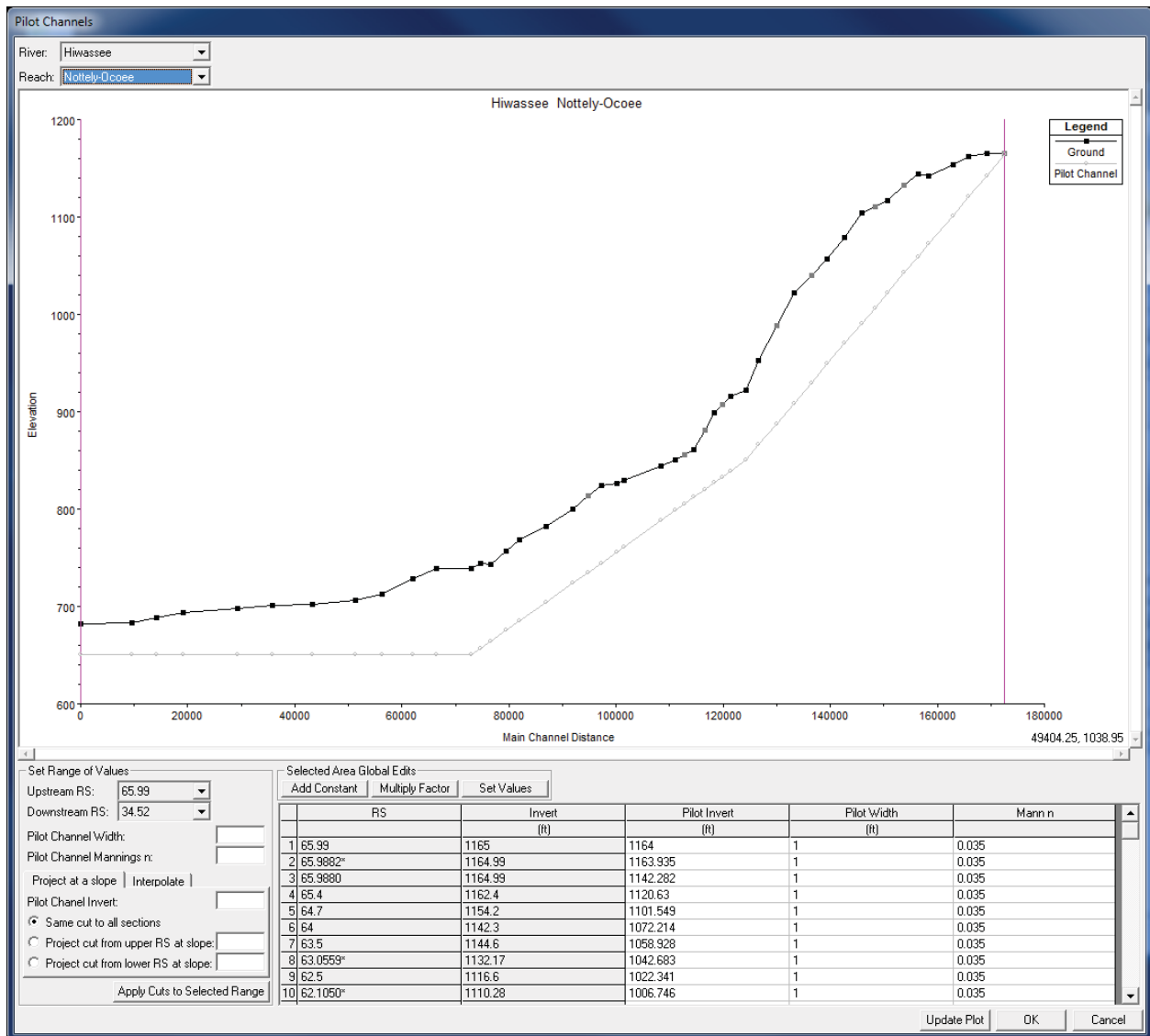


Figure 30: Final Chickamauga Model Hiwassee River Nottely-Ocoee Reach Pilot Channel Addition

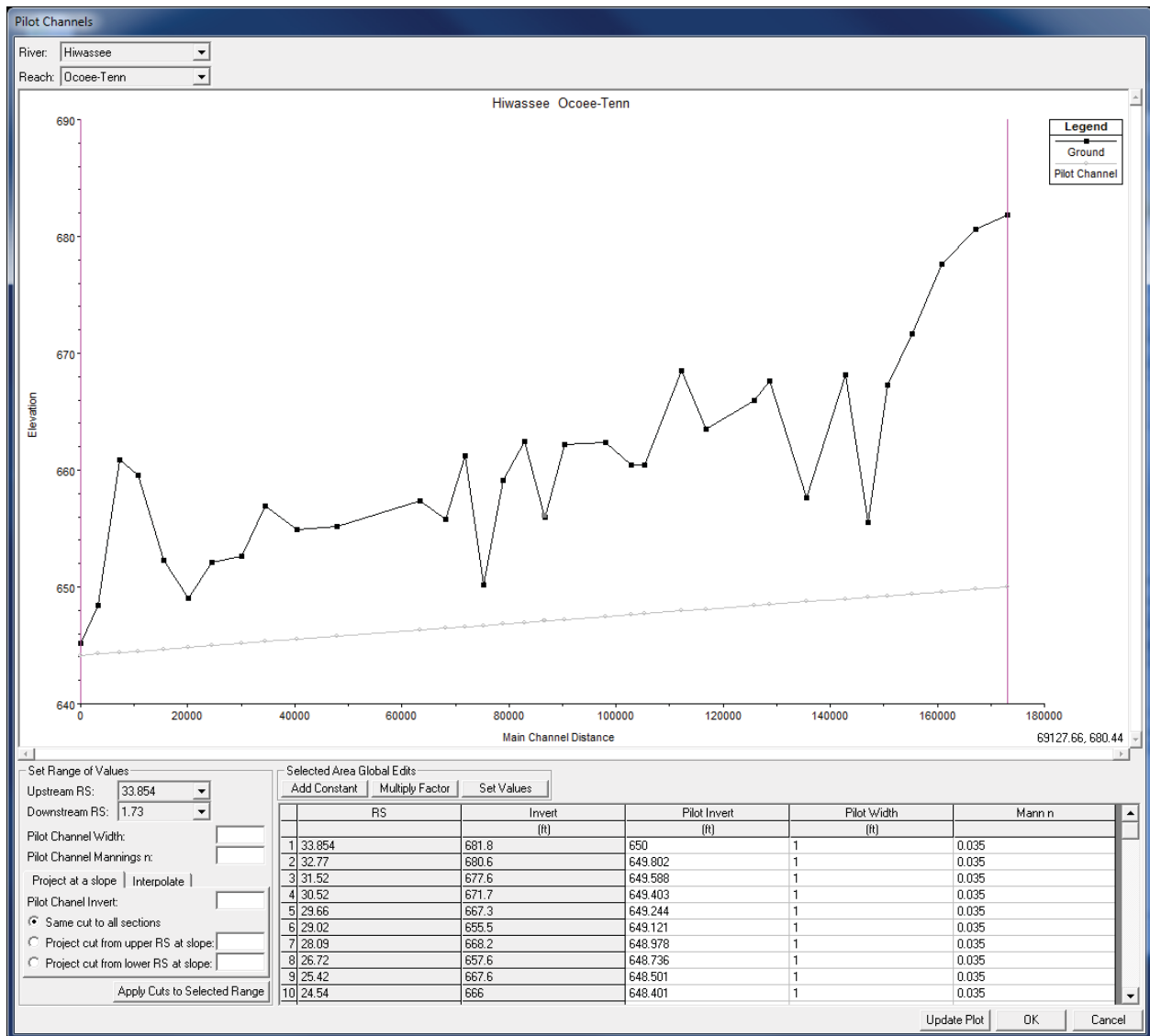


Figure 31: Final Chickamauga Model Hiwassee River Ocoee-Tennessee Reach Pilot Channel Addition

Stability Method Model Figures

Ocoee BRH-Hiwassee Reach

Unsteady Flow Data - CHHOcoeeBRHHiwassee

File Options Help

Boundary Conditions | Initial Conditions | Apply Data

Boundary Condition Types

Stage Hydrograph	Flow Hydrograph	Stage/Flow Hydr.	Rating Curve
Normal Depth	Lateral Inflow Hydr.	Uniform Lateral Inflow	Groundwater Interflow
T.S. Gate Openings	Elev Controlled Gates	Navigation Dams	IB Stage/Flow
Rules			

Add Boundary Condition Location

Add RS ... Add Storage Area ... Add SA Connection ... Add Pump Station ...

Select Location in table then select Boundary Condition Type

	River	Reach	RS	Boundary Condition
1	Hiwassee	Nottely-Ocoee	36.1	Flow Hydrograph
2	Hiwassee	Ocoee-Tenn	18.9	Uniform Lateral Inflow
3	Hiwassee	Ocoee-Tenn	1.73	Normal Depth
4	Ocoee	BRH-Hiwassee	11.88	Flow Hydrograph
5	Ocoee	BRH-Hiwassee	11.8789	Uniform Lateral Inflow

Observed DSS data set

Figure 32: Boundary Conditions for the Original BRH-Hiwassee Model

Flow Hydrograph

River: Ocoee Reach: BRH-Hiwassee RS: 11.88

☒ Read from DSS before simulation Select DSS file and Path

File: c:\Users\jksharke\Pictures\Thesis test files\ObservedDamDischarges
 Path: /OCOEEEN01/26/FLOW/01JAN1976/1HOUR/OBSERVED_DISCHA

☐ Enter Table Data time interval: 1 Hour

Select/Enter the Data's Starting Time Reference

☒ Use Simulation Time: Date: 02JUL1998 Time:
☐ Fixed Start Time: Date: Time:

No. Ordinates Interpolate Missing Values Del Row Ins Row

Hydrograph Data			
	Date	Simulation Time	Flow
		(hours)	(cfs)
1	01Jul1998 2400	00:00	
2	02Jul1998 0100	01:00	
3	02Jul1998 0200	02:00	
4	02Jul1998 0300	03:00	
5	02Jul1998 0400	04:00	
6	02Jul1998 0500	05:00	
7	02Jul1998 0600	06:00	
8	02Jul1998 0700	07:00	
9	02Jul1998 0800	08:00	
10	02Jul1998 0900	09:00	
11	02Jul1998 1000	10:00	
12	02Jul1998 1100	11:00	
13	02Jul1998 1200	12:00	
14	02Jul1998 1300	13:00	
15	02Jul1998 1400	14:00	
16	02Jul1998 1500	15:00	
17	02Jul1998 1600	16:00	
18	02Jul1998 1700	17:00	
19	02Jul1998 1800	18:00	

Time Step Adjustment Options ("Critical" boundary conditions)

☐ Monitor this hydrograph for adjustments to computational time step

Max Change in Flow (without changing time step):

Min Flow: 750 Multiplier:

Plot Data OK Cancel

Figure 33: Minimum Flow at Ocoee 1 Dam for Stability Method Analysis

Lateral Inflow Hydrograph

River: Ocoee Reach: BRH-Hiwassee RS: 11.15

☐ Read from DSS before simulation Select DSS file and Path

File:

Path:

☒ Enter Table Data time interval: 1 Year

Select/Enter the Data's Starting Time Reference

☒ Use Simulation Time: Date: 02JUL1998 Time:

☐ Fixed Start Time: Date: Time:

No. Ordinates Interpolate Missing Values Del Row Ins Row

Hydrograph Data			
	Date	Simulation Time	Lateral Inflow
		(hours)	(cfs)
1	01Jul1998 2400	00:00	325.
2	01Jul1999 2400	8760:00	325.
3	01Jul2000 2400	17520:00	325.
4	01Jul2001 2400	26280:00	325.
5	01Jul2002 2400	35040:00	325.
6	01Jul2003 2400	43824:00	325.
7	01Jul2004 2400	52584:00	325.
8	01Jul2005 2400	61344:00	325.
9	01Jul2006 2400	70104:00	325.
10	01Jul2007 2400	78888:00	325.
11	01Jul2008 2400	87648:00	325.
12	01Jul2009 2400	96408:00	325.
13	01Jul2010 2400	105168:00	325.
14	01Jul2011 2400	113952:00	325.
15	01Jul2012 2400	122712:00	325.
16	01Jul2013 2400	131472:00	325.
17	01Jul2014 2400	140232:00	325.
18	01Jul2015 2400	149016:00	325.
19	01Jul2016 2400	157776:00	325.

Time Step Adjustment Options ("Critical" boundary conditions)

☐ Monitor this hydrograph for adjustments to computational time step

Max Change in Flow (without changing time step):

Min Flow: Multiplier:

Plot Data OK Cancel

Figure 34: Lateral Inflow at Ocoee River Mile 11.15 for Stability Method Analysis

Lateral Inflow Hydrograph

River: Ocoee Reach: BRH-Hiwassee RS: 6.6

☐ Read from DSS before simulation Select DSS file and Path

File:

Path:

☒ Enter Table Data time interval: 1 Year

Select/Enter the Data's Starting Time Reference

☒ Use Simulation Time: Date: 02JUL1998 Time:

☐ Fixed Start Time: Date: Time:

No. Ordinates Interpolate Missing Values Del Row Ins Row

Hydrograph Data			
	Date	Simulation Time	Lateral Inflow
		(hours)	(cfs)
1	01Jul1998 2400	00:00	325.
2	01Jul1999 2400	8760:00	325.
3	01Jul2000 2400	17520:00	325.
4	01Jul2001 2400	26280:00	325.
5	01Jul2002 2400	35040:00	325.
6	01Jul2003 2400	43824:00	325.
7	01Jul2004 2400	52584:00	325.
8	01Jul2005 2400	61344:00	325.
9	01Jul2006 2400	70104:00	325.
10	01Jul2007 2400	78888:00	325.
11	01Jul2008 2400	87648:00	325.
12	01Jul2009 2400	96408:00	325.
13	01Jul2010 2400	105168:00	325.
14	01Jul2011 2400	113952:00	325.
15	01Jul2012 2400	122712:00	325.
16	01Jul2013 2400	131472:00	325.
17	01Jul2014 2400	140232:00	325.
18	01Jul2015 2400	149016:00	325.
19	01Jul2016 2400	157776:00	325.

Time Step Adjustment Options ("Critical" boundary conditions)

☐ Monitor this hydrograph for adjustments to computational time step

Max Change in Flow (without changing time step):

Min Flow: Multiplier:

Plot Data OK Cancel

Figure 35: Lateral Inflow at Ocoee River Mile 6.6 for Stability Method Analysis

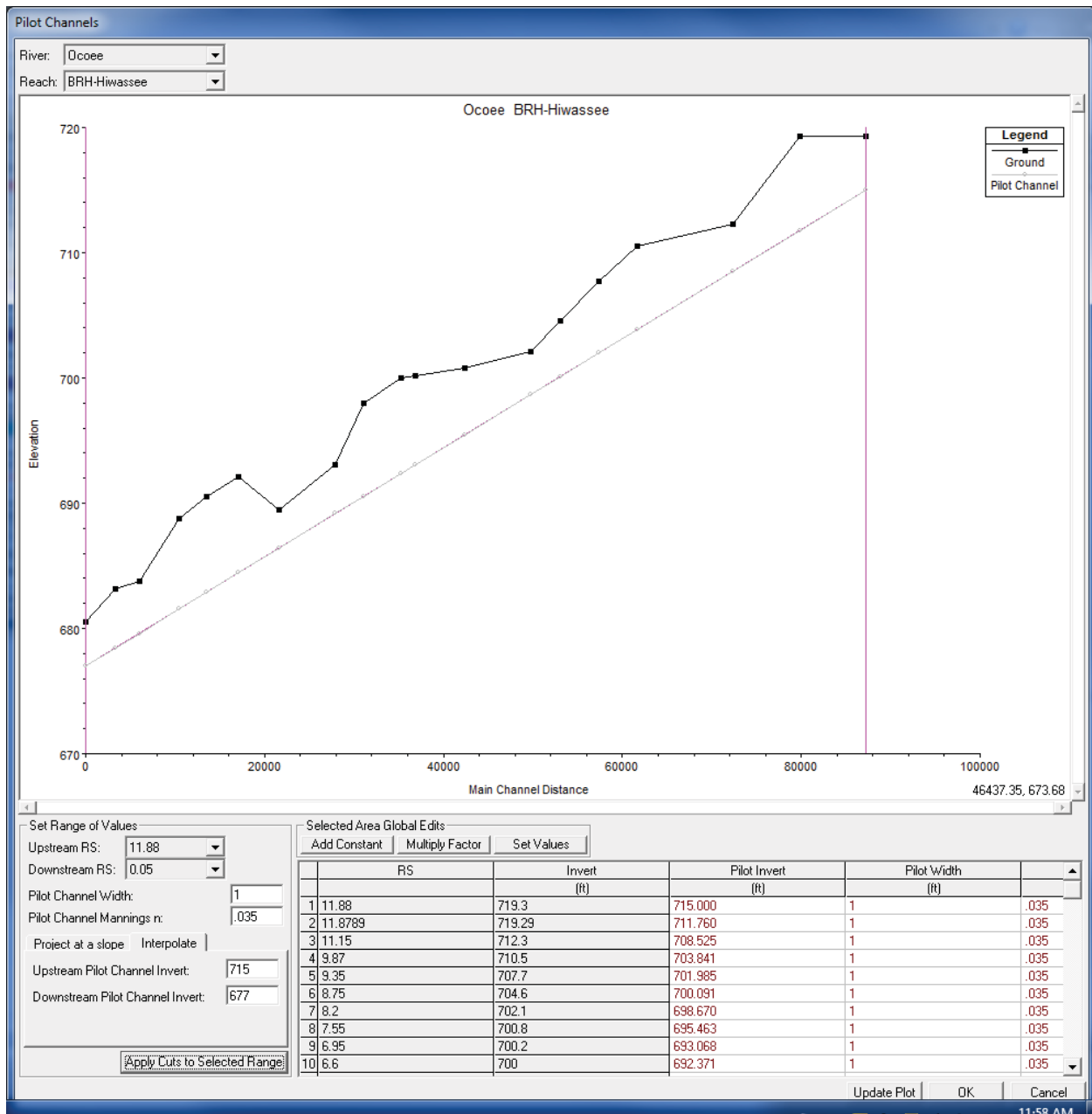


Figure 36: Pilot Channel on Ocoee River Reach for Stability Method Analysis

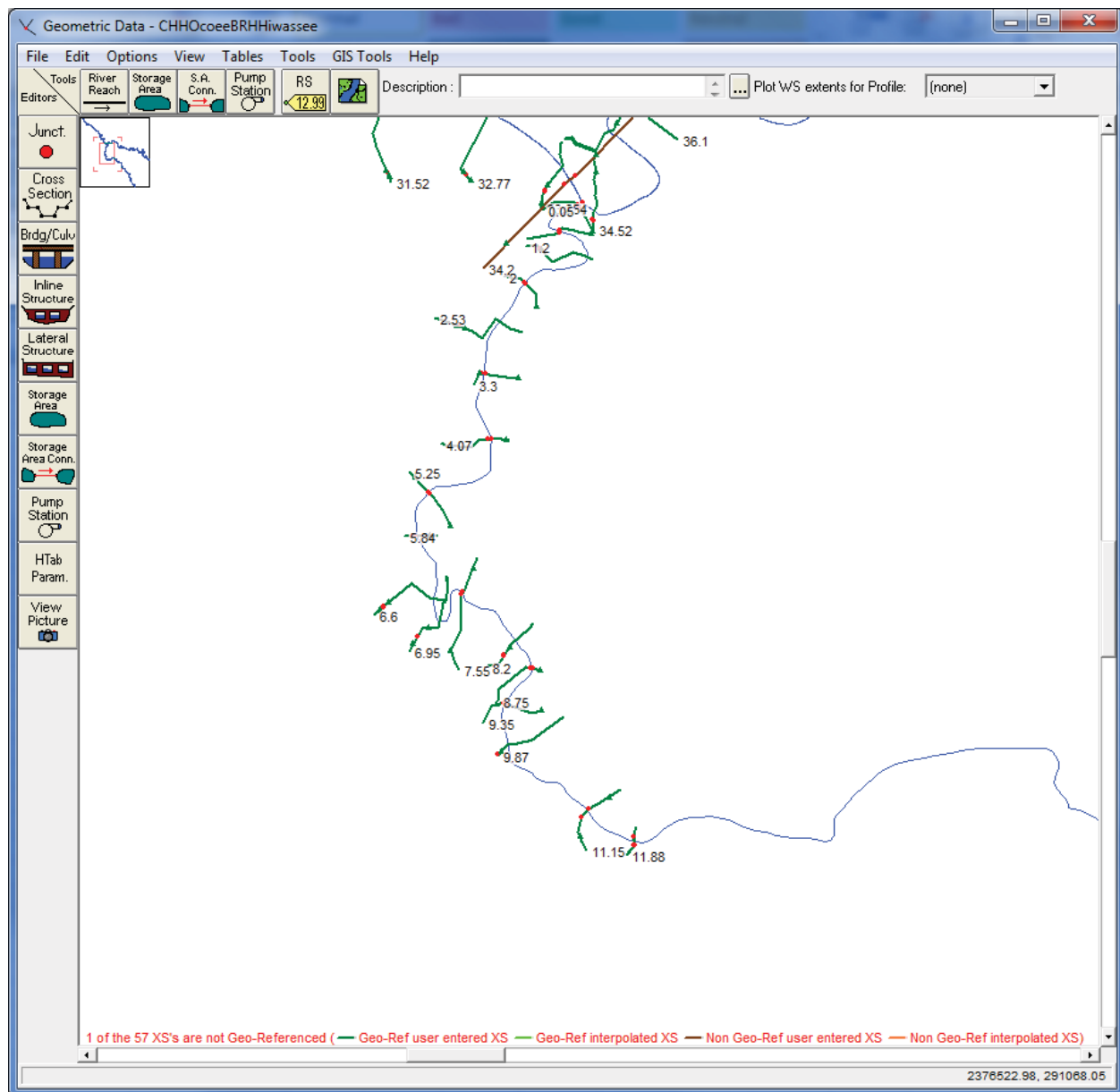


Figure 37: Samuels's Cross Sectional Spacing on Ocoee River Reach for Stability Method Analysis

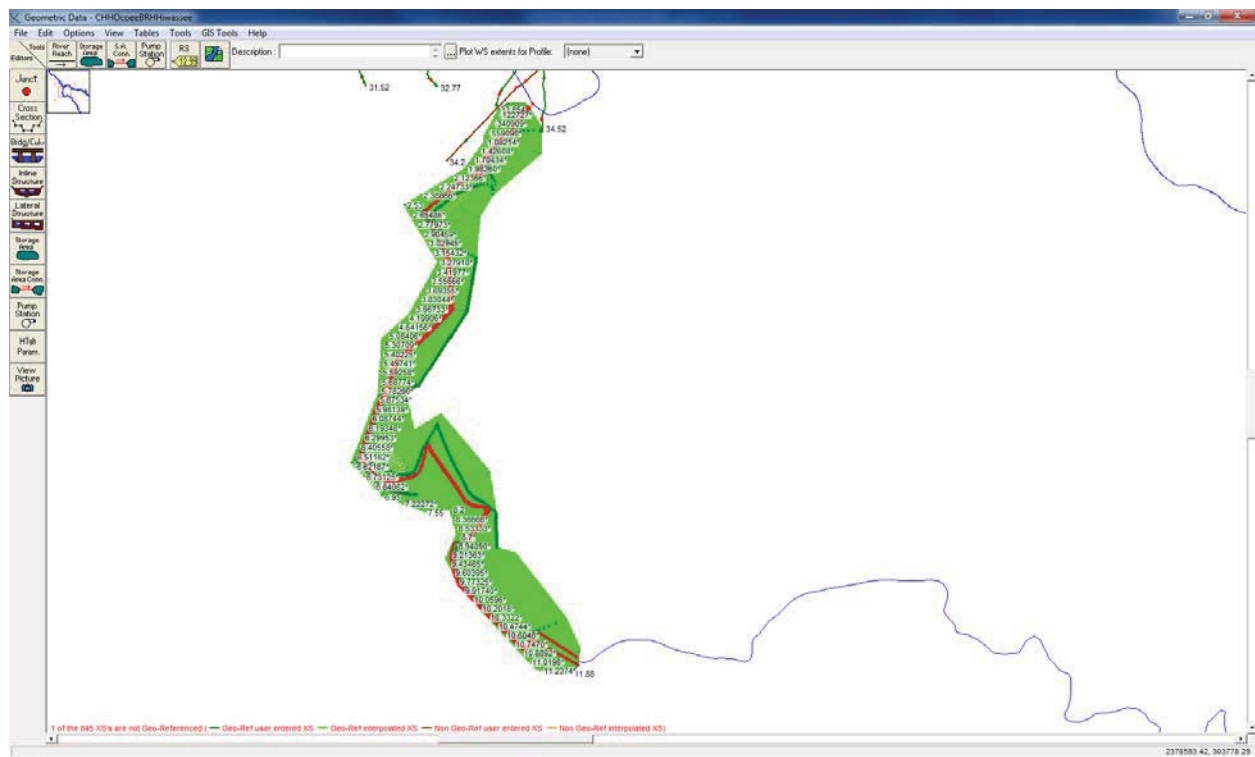


Figure 38:100-Foot Cross Sectional Spacing on Ocoee River Reach for Stability Method Analysis

Hiwassee River, Nottely-Ocoee Reach

Unsteady Flow Data - CHHCalNottelyOcoee

File Options Help

Boundary Conditions | Initial Conditions | Apply Data

Boundary Condition Types

Stage Hydrograph	Flow Hydrograph	Stage/Flow Hydr.	Rating Curve
Normal Depth	Lateral Inflow Hydr.	Uniform Lateral Inflow	Groundwater Interflow
T.S. Gate Openings	Elev Controlled Gates	Navigation Dams	IB Stage/Flow
Rules 			

Add Boundary Condition Location

Add RS ... Add Storage Area ... Add SA Connection ... Add Pump Station ...

Select Location in table then select Boundary Condition Type

	River	Reach	RS	Boundary Condition
1	Hiwassee	Nottely-Ocoee	65.99	Flow Hydrograph
2	Hiwassee	Nottely-Ocoee	65.9881	Lateral Inflow Hydr.
3	Hiwassee	Nottely-Ocoee	1.73	Normal Depth

Observed DSS data set

Figure 39: Boundary Conditions for the Original Nottely-Ocoee Model

Flow Hydrograph

River: Hiwassee Reach: Nottely-Ocoee RS: 65.99

☒ Read from DSS before simulation Select DSS file and Path

File: c:\Users\jksharke\Pictures\Thesis test files\ObservedDamDischarges
 Path: /APALACHIA/22/FLOW/01JAN1976/1HOUR/OBSERVED_DISCHA

☐ Enter Table Data time interval: 1 Hour

Select/Enter the Data's Starting Time Reference

☒ Use Simulation Time: Date: 02JUL1998 Time:
☐ Fixed Start Time: Date: Time:

No. Ordinates Interpolate Missing Values Del Row Ins Row

Hydrograph Data			
	Date	Simulation Time	Flow
		(hours)	(cfs)
1	01Jul1998 2400	00:00	
2	02Jul1998 0100	01:00	
3	02Jul1998 0200	02:00	
4	02Jul1998 0300	03:00	
5	02Jul1998 0400	04:00	
6	02Jul1998 0500	05:00	
7	02Jul1998 0600	06:00	
8	02Jul1998 0700	07:00	
9	02Jul1998 0800	08:00	
10	02Jul1998 0900	09:00	
11	02Jul1998 1000	10:00	
12	02Jul1998 1100	11:00	
13	02Jul1998 1200	12:00	
14	02Jul1998 1300	13:00	
15	02Jul1998 1400	14:00	
16	02Jul1998 1500	15:00	
17	02Jul1998 1600	16:00	
18	02Jul1998 1700	17:00	
19	02Jul1998 1800	18:00	

Time Step Adjustment Options ("Critical" boundary conditions)

☐ Monitor this hydrograph for adjustments to computational time step

Max Change in Flow (without changing time step):

Min Flow: 300 Multiplier:

Plot Data OK Cancel

Figure 40: Minimum Flow at Apalachia Dam for Stability Method Analysis

Lateral Inflow Hydrograph

River: Hiwassee Reach: Nottely-Ocoee RS: 65.4

☐ Read from DSS before simulation Select DSS file and Path

File:

Path:

☒ Enter Table Data time interval: 1 Year

Select/Enter the Data's Starting Time Reference

☒ Use Simulation Time: Date: 02JUL1998 Time:

☐ Fixed Start Time: Date: Time:

No. Ordinates Interpolate Missing Values Del Row Ins Row

Hydrograph Data			
	Date	Simulation Time	Lateral Inflow
		(hours)	(cfs)
1	01Jul1998 2400	00:00	150
2	01Jul1999 2400	8760:00	150.00
3	01Jul2000 2400	17520:00	150.00
4	01Jul2001 2400	26280:00	150.00
5	01Jul2002 2400	35040:00	150.00
6	01Jul2003 2400	43824:00	150.00
7	01Jul2004 2400	52584:00	150.00
8	01Jul2005 2400	61344:00	150.00
9	01Jul2006 2400	70104:00	150.00
10	01Jul2007 2400	78888:00	150.00
11	01Jul2008 2400	87648:00	150.00
12	01Jul2009 2400	96408:00	150.00
13	01Jul2010 2400	105168:00	150.00
14	01Jul2011 2400	113952:00	150.00
15	01Jul2012 2400	122712:00	150.00
16	01Jul2013 2400	131472:00	150.00
17	01Jul2014 2400	140232:00	150.00
18	01Jul2015 2400	149016:00	150.00
19	01Jul2016 2400	157776:00	150.00

Time Step Adjustment Options ("Critical" boundary conditions)

☐ Monitor this hydrograph for adjustments to computational time step

Max Change in Flow (without changing time step):

Min Flow: Multiplier:

Plot Data OK Cancel

Figure 41: Lateral Inflow at Hiwassee River Mile 65.4 for Stability Method Analysis

Lateral Inflow Hydrograph

River: Hiwassee Reach: Nottely-Ocoee RS: 50.51

☐ Read from DSS before simulation Select DSS file and Path

File:

Path:

☒ Enter Table Data time interval: 1 Year

Select/Enter the Data's Starting Time Reference

☒ Use Simulation Time: Date: 02JUL1998 Time:

☐ Fixed Start Time: Date: Time:

No. Ordinates Interpolate Missing Values Del Row Ins Row

Hydrograph Data			
	Date	Simulation Time	Lateral Inflow
		(hours)	(cfs)
1	01Jul1998 2400	00:00	150
2	01Jul1999 2400	8760:00	150.00
3	01Jul2000 2400	17520:00	150.00
4	01Jul2001 2400	26280:00	150.00
5	01Jul2002 2400	35040:00	150.00
6	01Jul2003 2400	43824:00	150.00
7	01Jul2004 2400	52584:00	150.00
8	01Jul2005 2400	61344:00	150.00
9	01Jul2006 2400	70104:00	150.00
10	01Jul2007 2400	78888:00	150.00
11	01Jul2008 2400	87648:00	150.00
12	01Jul2009 2400	96408:00	150.00
13	01Jul2010 2400	105168:00	150.00
14	01Jul2011 2400	113952:00	150.00
15	01Jul2012 2400	122712:00	150.00
16	01Jul2013 2400	131472:00	150.00
17	01Jul2014 2400	140232:00	150.00
18	01Jul2015 2400	149016:00	150.00
19	01Jul2016 2400	157776:00	150.00

Time Step Adjustment Options ("Critical" boundary conditions)

☐ Monitor this hydrograph for adjustments to computational time step

Max Change in Flow (without changing time step):

Min Flow: Multiplier:

Plot Data OK Cancel

Figure 42: Lateral Inflow at Hiwassee River Mile 50.51 for Stability Method Analysis

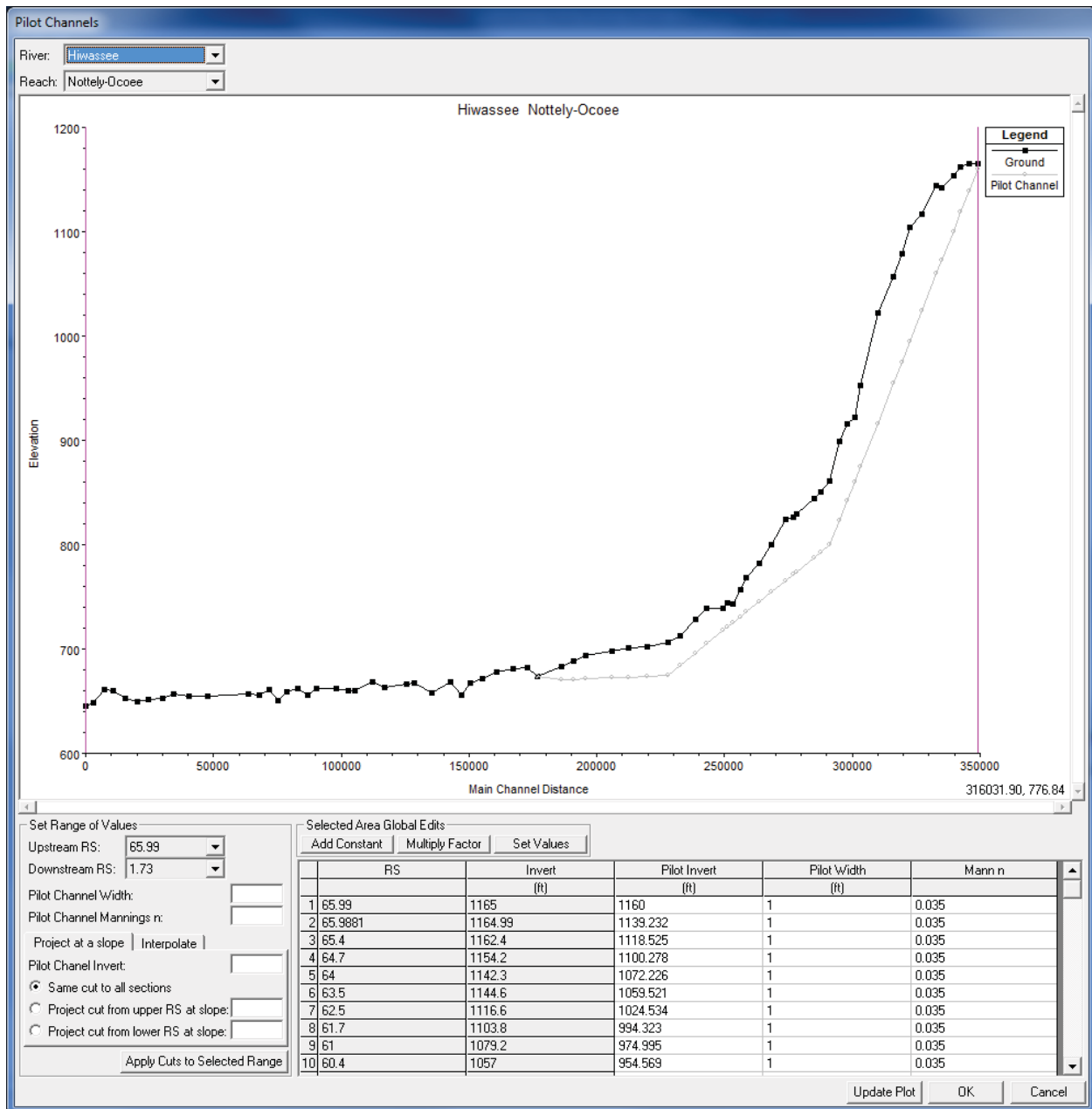


Figure 43: Pilot Channel on Hiwassee River Reach for Stability Method Analysis

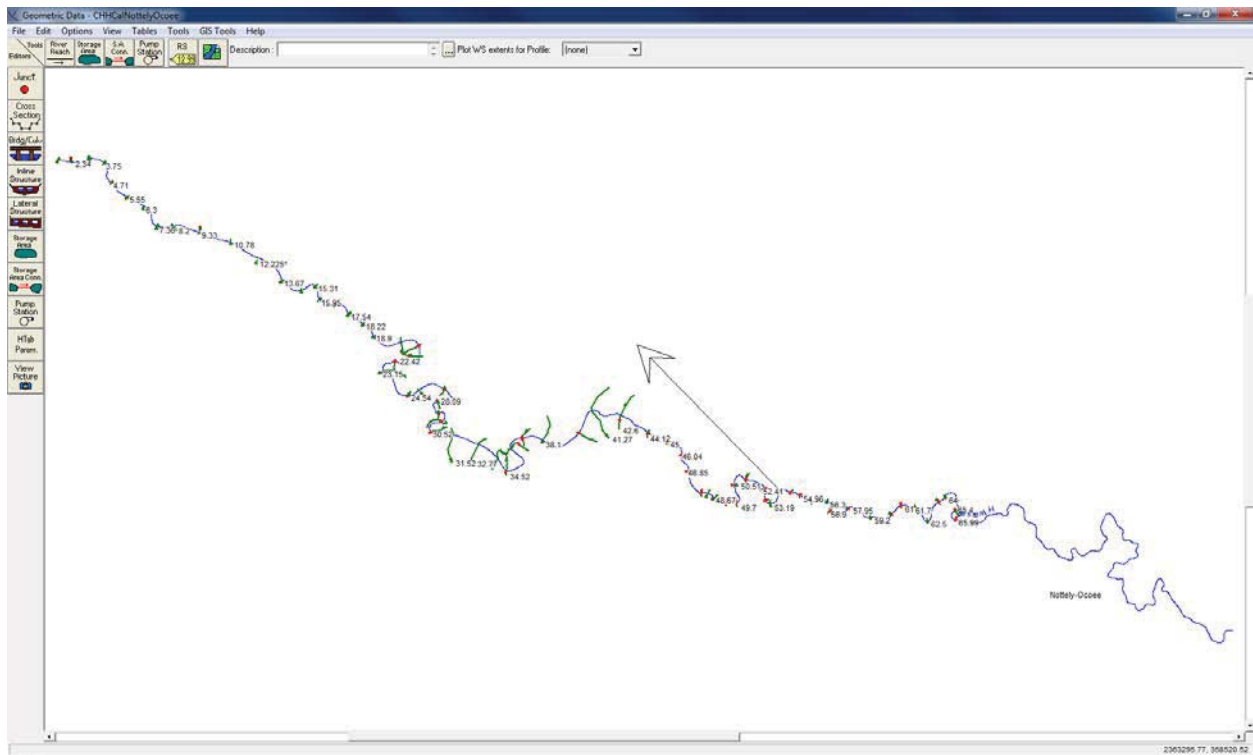


Figure 44: Samuels's Cross Sectional Spacing on Hiwassee River Reach for Stability Method Analysis

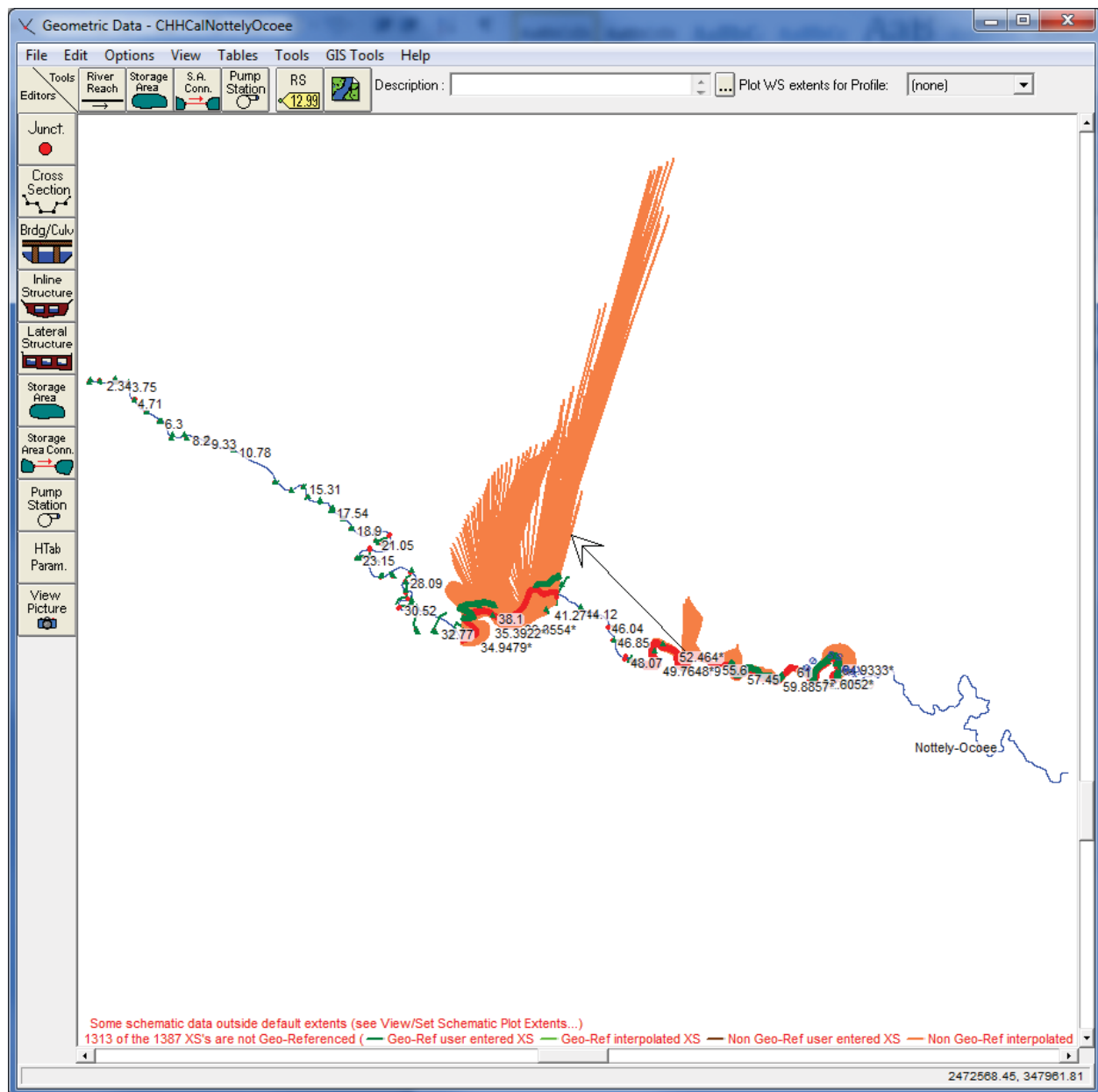


Figure 45:100-Foot Cross Sectional Spacing on Hiwassee River Reach for Stability Method Analysis

Tennessee River (Chickamauga) Reach

Unsteady Flow Data - CHHTennesseeRiver

File Options Help

Boundary Conditions | Initial Conditions | Apply Data

Boundary Condition Types

Stage Hydrograph	Flow Hydrograph	Stage/Flow Hydr.	Rating Curve
Normal Depth	Lateral Inflow Hydr.	Uniform Lateral Inflow	Groundwater Interflow
T.S. Gate Openings	Elev Controlled Gates	Navigation Dams	IB Stage/Flow
Rules			

Add Boundary Condition Location

Add RS ... Add Storage Area ... Add SA Connection ... Add Pump Station ...

Select Location in table then select Boundary Condition Type

	River	Reach	RS	Boundary Condition
1	Tennessee	Clinch-Hiwassee	529.89	Flow Hydrograph
2	Tennessee	Clinch-Hiwassee	529.888*	Uniform Lateral Inflow
3	Tennessee	Clinch-Hiwassee	525.69	Lateral Inflow Hydr.
4	Tennessee	Clinch-Hiwassee	499.495	Uniform Lateral Inflow
5	Tennessee	Clinch-Hiwassee	471.01	Normal Depth

Observed DSS data set

Figure 46: Boundary Conditions for the Original Tennessee (Chickamauga) Model

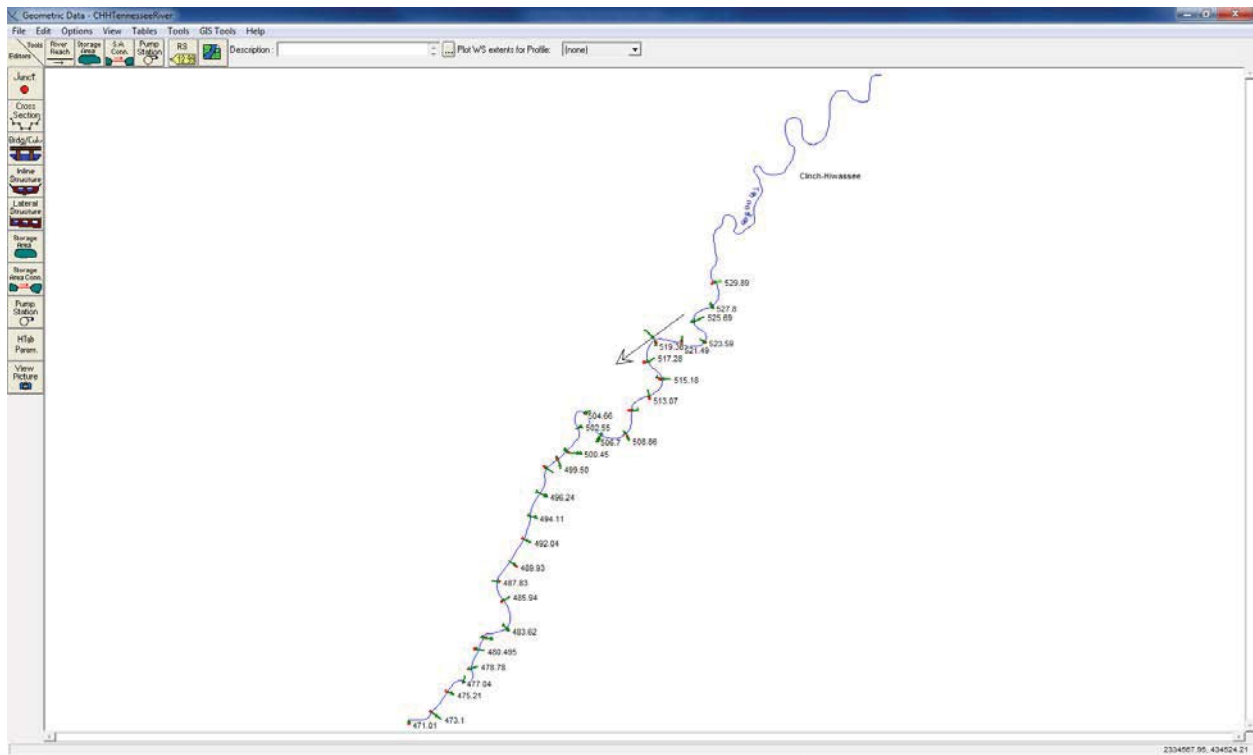


Figure 47: Cross Sections for the Original Tennessee (Chickamauga) Model

Ocoee- Tennessee Reach

Unsteady Flow Data - CHHOcoeeTN

File Options Help

Boundary Conditions | Initial Conditions | Apply Data

Boundary Condition Types

Stage Hydrograph	Flow Hydrograph	Stage/Flow Hydr.	Rating Curve
Normal Depth	Lateral Inflow Hydr.	Uniform Lateral Inflow	Groundwater Interflow
T.S. Gate Openings	Elev. Controlled Gates	Navigation Dams	IB Stage/Flow
Rules			

Add Boundary Condition Location

Add RS ... Add Storage Area ... Add SA Connection ... Add Pump Station ...

Select Location in table then select Boundary Condition Type

	River	Reach	RS	Boundary Condition
1	Hiwassee	Ocoee-Tenn	33.854	Flow Hydrograph
2	Hiwassee	Ocoee-Tenn	18.9	Uniform Lateral Inflow
3	Tennessee	Clinch-Hiwassee	529.89	Flow Hydrograph
4	Tennessee	Hiwassee-Dallas	471.01	Normal Depth

Observed DSS data set

Figure 48: Boundary Conditions for the Original Ocoee-Tennessee Model

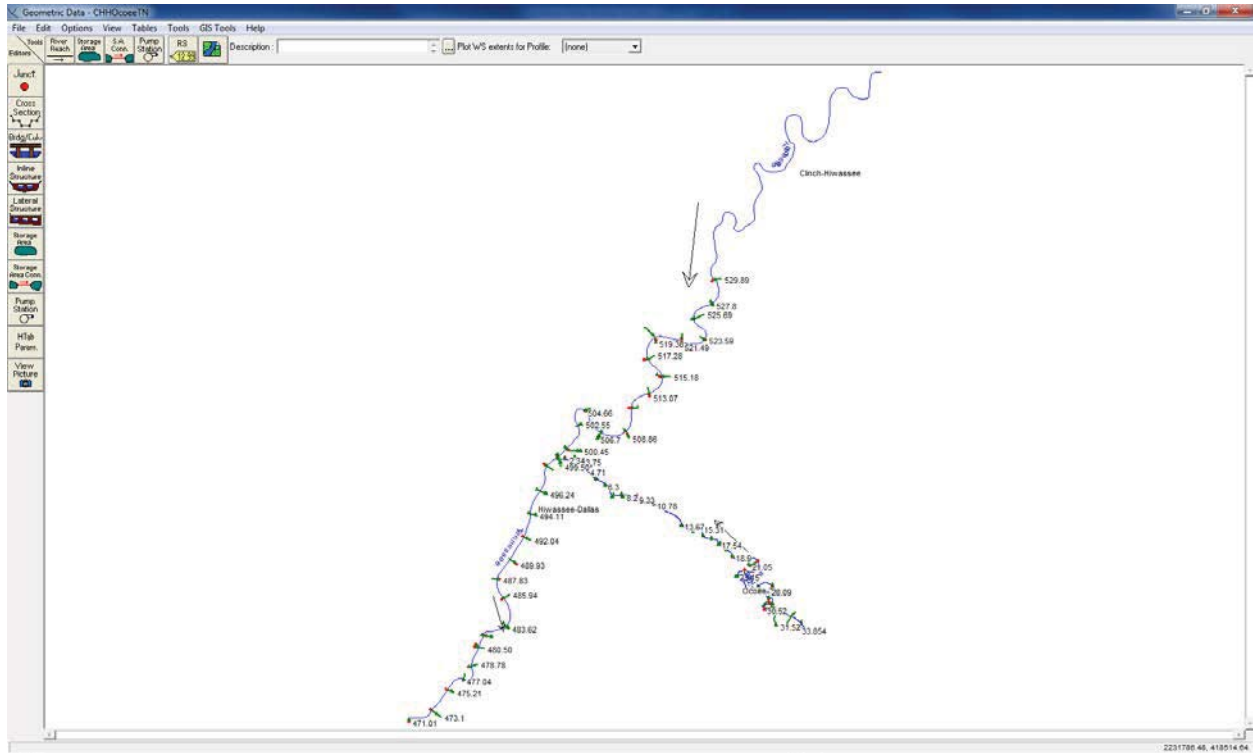


Figure 49: Cross Sections for the Original Ocoee-Tennessee Model

Tennessee River (Guntersville) Reach

Unsteady Flow Data - GUHThesis

File Options Help

Boundary Conditions Initial Conditions Apply Data

Boundary Condition Types

Stage Hydrograph	Flow Hydrograph	Stage/Flow Hydr.	Rating Curve
Normal Depth	Lateral Inflow Hydr.	Uniform Lateral Inflow	Groundwater Interflow
T.S. Gate Openings	Elev Controlled Gates	Navigation Dams	IB Stage/Flow
Rules			

Add Boundary Condition Location

Add RS ... Add Storage Area ... Add SA Connection ... Add Pump Station ...

Select Location in table then select Boundary Condition Type

	River	Reach	RS	Boundary Condition
1	Tennessee	NChick-Elk	424.69	Flow Hydrograph
2	Tennessee	NChick-Elk	424.67	Lateral Inflow Hydr.
3	Tennessee	NChick-Elk	422.6	Uniform Lateral Inflow
4	Tennessee	NChick-Elk	391.06	Uniform Lateral Inflow
5	Tennessee	NChick-Elk	363.72	Lateral Inflow Hydr.
6	Tennessee	NChick-Elk	349.01	Normal Depth

Observed DSS data set

Figure 50: Boundary Conditions for the Original Tennessee (Guntersville) Model

Flow Hydrograph

River: Tennessee Reach: NChick-Elk RS: 424.69

☒ Read from DSS before simulation Select DSS file and Path

File: c:\Users\jksharke\Pictures\Thesis test files\ObservedDamDischarges
 Path: /NICKAJACK/28/FLOW/01JAN1976/1HOUR/OBSERVED_DISCHA

☐ Enter Table Data time interval: 1 Hour

Select/Enter the Data's Starting Time Reference

☒ Use Simulation Time: Date: 02JUL1998 Time:
☐ Fixed Start Time: Date: Time:

No. Ordinates Interpolate Missing Values Del Row Ins Row

Hydrograph Data			
	Date	Simulation Time	Flow
		(hours)	(cfs)
1	01Jul1998 2400	00:00	
2	02Jul1998 0100	01:00	
3	02Jul1998 0200	02:00	
4	02Jul1998 0300	03:00	
5	02Jul1998 0400	04:00	
6	02Jul1998 0500	05:00	
7	02Jul1998 0600	06:00	
8	02Jul1998 0700	07:00	
9	02Jul1998 0800	08:00	
10	02Jul1998 0900	09:00	
11	02Jul1998 1000	10:00	
12	02Jul1998 1100	11:00	
13	02Jul1998 1200	12:00	
14	02Jul1998 1300	13:00	
15	02Jul1998 1400	14:00	
16	02Jul1998 1500	15:00	
17	02Jul1998 1600	16:00	
18	02Jul1998 1700	17:00	
19	02Jul1998 1800	18:00	

Time Step Adjustment Options ("Critical" boundary conditions)

☐ Monitor this hydrograph for adjustments to computational time step

Max Change in Flow (without changing time step):

Min Flow: 14000 Multiplier:

Plot Data OK Cancel

Figure 51: Minimum Flow at Nickajack Dam for Stability Method Analysis

Lateral Inflow Hydrograph

River: Tennessee Reach: NChick-Elk RS: 420.49

☐ Read from DSS before simulation Select DSS file and Path

File:

Path:

☒ Enter Table Data time interval: 1 Year

Select/Enter the Data's Starting Time Reference

☒ Use Simulation Time: Date: 02JUL1998 Time:

☐ Fixed Start Time: Date: Time:

No. Ordinates Interpolate Missing Values Del Row Ins Row

Hydrograph Data			
	Date	Simulation Time	Lateral Inflow
		(hours)	(cfs)
1	01Jul1998 2400	00:00	7000
2	01Jul1999 2400	8760:00	7000.00
3	01Jul2000 2400	17520:00	7000.00
4	01Jul2001 2400	26280:00	7000.00
5	01Jul2002 2400	35040:00	7000.00
6	01Jul2003 2400	43824:00	7000.00
7	01Jul2004 2400	52584:00	7000.00
8	01Jul2005 2400	61344:00	7000.00
9	01Jul2006 2400	70104:00	7000.00
10	01Jul2007 2400	78888:00	7000.00
11	01Jul2008 2400	87648:00	7000.00
12	01Jul2009 2400	96408:00	7000.00
13	01Jul2010 2400	105168:00	7000.00
14	01Jul2011 2400	113952:00	7000.00
15	01Jul2012 2400	122712:00	7000.00
16	01Jul2013 2400	131472:00	7000.00
17	01Jul2014 2400	140232:00	7000.00
18	01Jul2015 2400	149016:00	7000.00
19	01Jul2016 2400	157776:00	7000.00

Time Step Adjustment Options ("Critical" boundary conditions)

☐ Monitor this hydrograph for adjustments to computational time step

Max Change in Flow (without changing time step):

Min Flow: Multiplier:

Plot Data OK Cancel

Figure 52: Lateral Inflow at Tennessee River Mile 420.49 for Stability Method Analysis

Lateral Inflow Hydrograph

River: Tennessee Reach: NChick-Elk RS: 378.44

☐ Read from DSS before simulation Select DSS file and Path

File:

Path:

☒ Enter Table Data time interval: Year

Select/Enter the Data's Starting Time Reference

☒ Use Simulation Time: Date: 02JUL1998 Time:

☐ Fixed Start Time: Date: Time:

No. Ordinates Interpolate Missing Values Del Row Ins Row

Hydrograph Data			
	Date	Simulation Time	Lateral Inflow
		(hours)	(cfs)
1	01Jul1998 2400	00:00	7000
2	01Jul1999 2400	8760:00	7000.00
3	01Jul2000 2400	17520:00	7000.00
4	01Jul2001 2400	26280:00	7000.00
5	01Jul2002 2400	35040:00	7000.00
6	01Jul2003 2400	43824:00	7000.00
7	01Jul2004 2400	52584:00	7000.00
8	01Jul2005 2400	61344:00	7000.00
9	01Jul2006 2400	70104:00	7000.00
10	01Jul2007 2400	78888:00	7000.00
11	01Jul2008 2400	87648:00	7000.00
12	01Jul2009 2400	96408:00	7000.00
13	01Jul2010 2400	105168:00	7000.00
14	01Jul2011 2400	113952:00	7000.00
15	01Jul2012 2400	122712:00	7000.00
16	01Jul2013 2400	131472:00	7000.00
17	01Jul2014 2400	140232:00	7000.00
18	01Jul2015 2400	149016:00	7000.00
19	01Jul2016 2400	157776:00	7000.00

Time Step Adjustment Options ("Critical" boundary conditions)

☐ Monitor this hydrograph for adjustments to computational time step

Max Change in Flow (without changing time step):

Min Flow: Multiplier:

Plot Data OK Cancel

Figure 53: Lateral Inflow at Tennessee River Mile 378.44 for Stability Method Analysis

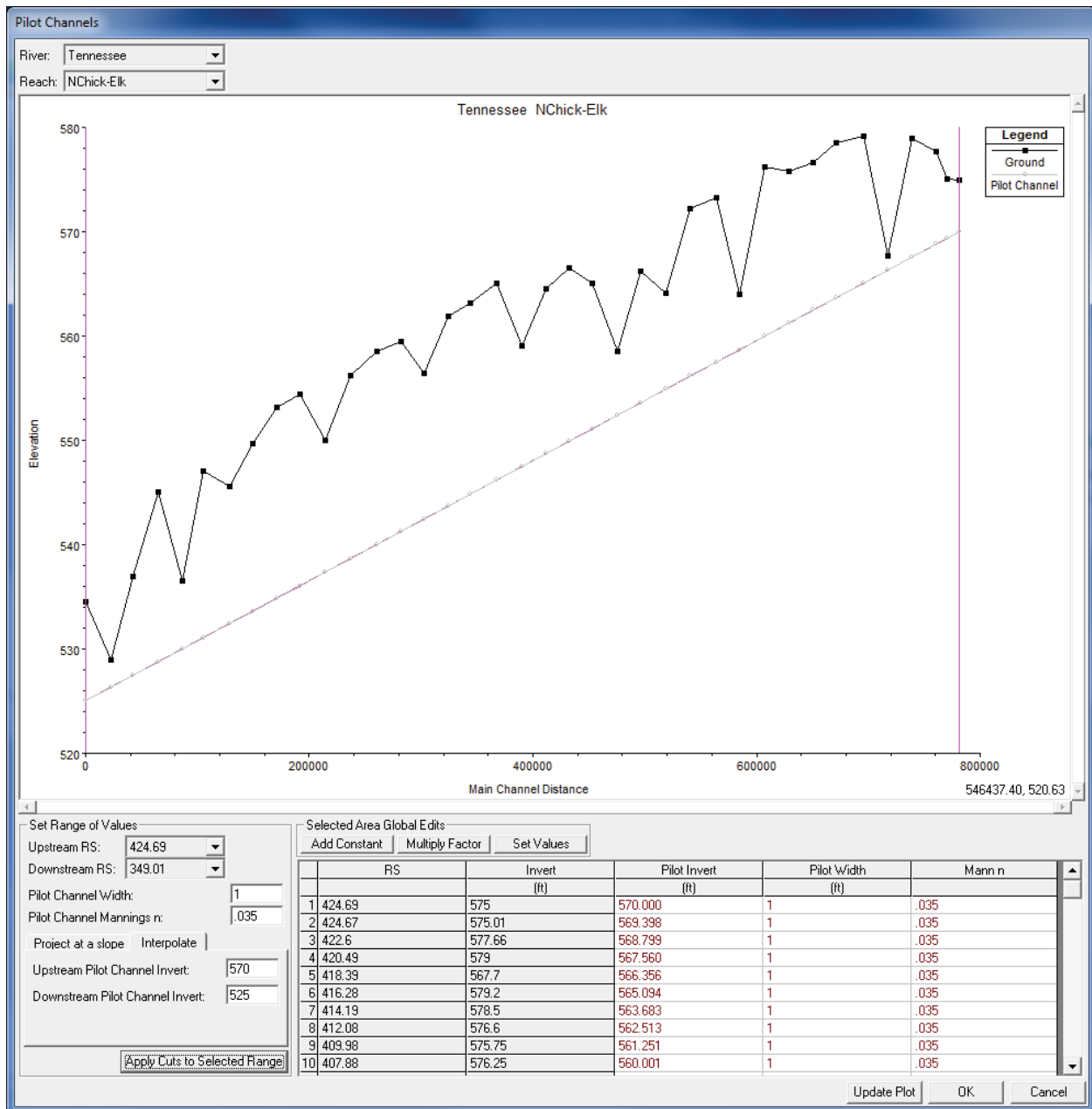


Figure 54: Pilot Channel on Tennessee River (Guntersville) Reach for Stability Method Analysis

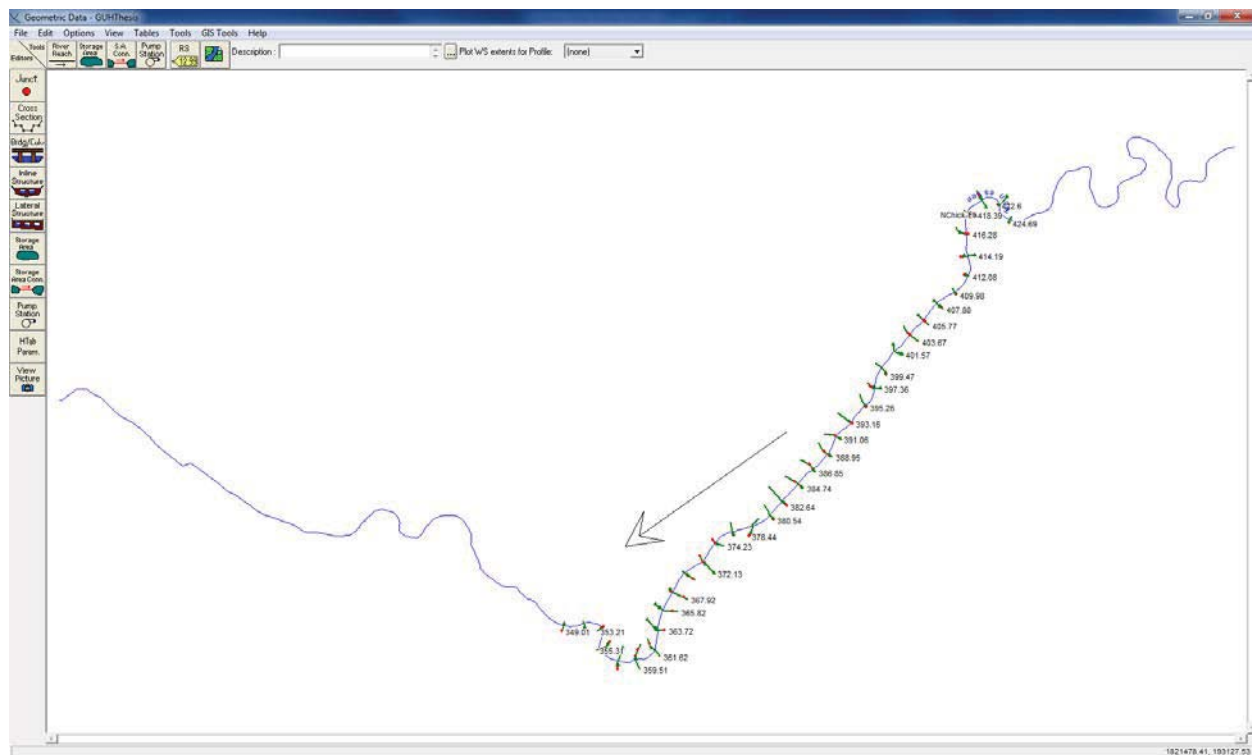


Figure 55: Samuels's Cross Sectional Spacing on Tennessee River (Guntersville) Reach for Stability Method Analysis

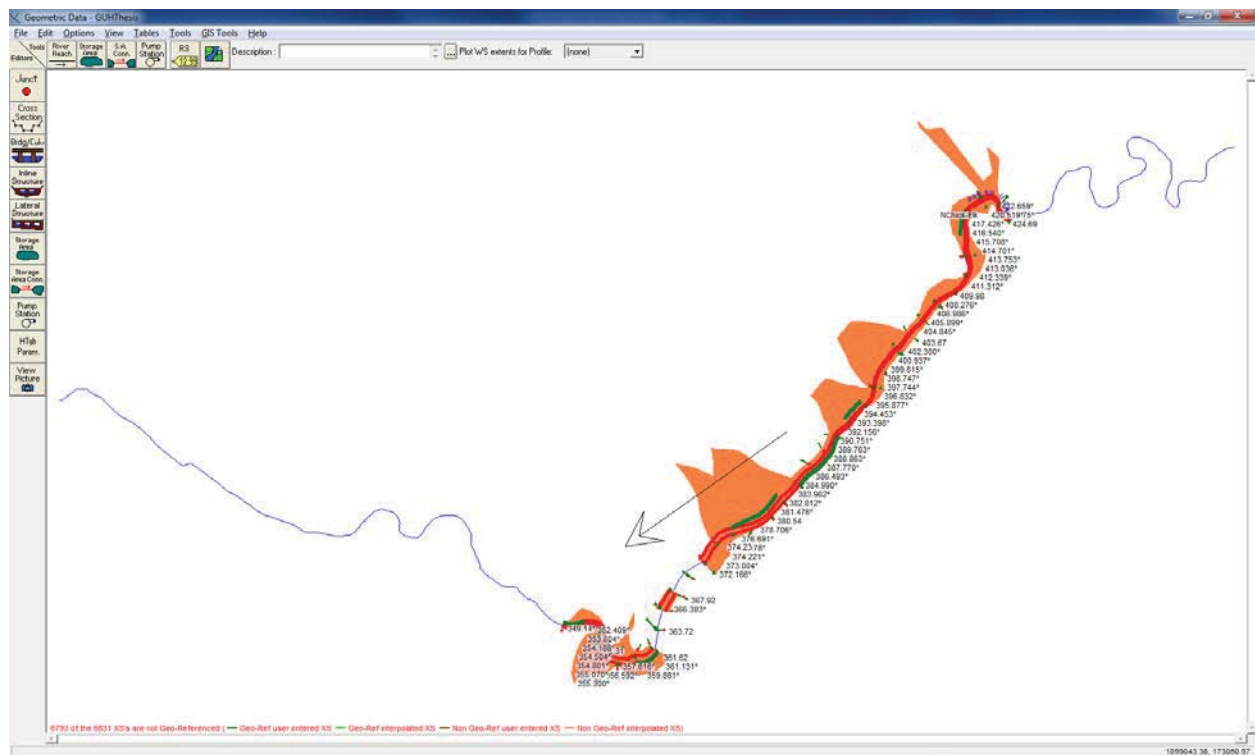


Figure 56: 100-Foot Cross Sectional Spacing on Tennessee River (Guntersville) Reach for Stability Method Analysis

Vita

Jennifer Kay Sharkey was born in Indianapolis, Indiana and moved to Murfreesboro, Tennessee soon after. After concluding her studies at Riverdale High School in 2009, Jennifer entered The University of Tennessee in Knoxville, Tennessee. She began an internship with the Tennessee Valley Authority in 2012, working in the River Management division. She received a Bachelor of Science with a major in Civil Engineering in December 2013. For the next year, she continued her internship at the Tennessee Valley Authority, expanding her understanding of the Tennessee River Valley and HEC-RAS unsteady flow modeling, while simultaneously pursuing her master's degree from the University of Tennessee in Water Resources Engineering.