

Optimization/Simulation Model for Determining Real-Time Optimal Operation of
River-Reservoirs Systems during Flooding Conditions

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A Dissertation Presented in Partial Fulfillment
of the Requirements for the Degree
Doctor of Philosophy

Approved June 2015 by the
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August 2015

ABSTRACT

A model is presented for real-time, river-reservoir operation systems. It epitomizes forward-thinking and efficient approaches to reservoir operations during flooding events. The optimization/simulation includes five major components. The components are a mix of hydrologic and hydraulic modeling, short-term rainfall forecasting, and optimization and reservoir operation models. The optimization/simulation model is designed for ultimate accessibility and efficiency. The optimization model uses the meta-heuristic approach, which has the capability to simultaneously search for multiple optimal solutions. The dynamics of the river are simulated by applying an unsteady flow-routing method. The rainfall-runoff simulation uses the National Weather Service NexRad gridded rainfall data, since it provides critical information regarding real storm events. The short-term rainfall-forecasting model utilizes a stochastic method. The reservoir-operation is simulated by a mass-balance approach. The optimization/simulation model offers more possible optimal solutions by using the Genetic Algorithm approach as opposed to traditional gradient methods that can only compute one optimal solution at a time. The optimization/simulation was developed for the 2010 flood event that occurred in the Cumberland River basin in Nashville, Tennessee. It revealed that the reservoir upstream of Nashville was more contained and that an optimal gate release schedule could have significantly decreased the floodwater levels in downtown Nashville. The model is for demonstrative purposes only but is perfectly suitable for real-world application.

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CHAPTER ONE - INTRODUCTION

1.1 Real-Time Flood Forecasting

Throughout human history, flooding has caused the most devastating and costly natural disasters on the planet. The impacts of floods go far beyond the cost and fatalities. Devastating impacts such as family and community disruptions, dislocation, and permanent injuries, often have long-term societal and socioeconomic implications. In the past, tremendous efforts have been exhausted to mitigate flood hazards. One of the most important aspects of minimizing the impacts of flooding is the proper operation of flood control systems. In order to operate flood control systems, the ability of forecast flooding is essential. Flood forecasting in its current application is used to estimate phases in future flooding. “Flood Forecasting” refers to the determination of the flow rates and water surface levels at various points within a river system as a result of using both observed and simulated inflow hydrographs.

Real-time flood forecasting is an essential component of flood warning, since proper flood warnings issued by federal, state, or local agencies heavily depend on the reliable forecast time-profiles of channel flow and stage levels of water at various locations. Application of real-time flood forecasting combines the use of real-time and forecasted precipitation and streamflow data in hydrologic and hydraulic simulation models to forecast flow rates and stages in rivers for periods ranging from hours to days in advance (Mays and Tung, 1992). Depending on the location, size, and topography of the watersheds, complicated flood forecasting systems will also need to account for the

effects of flood plains, washlands, flood defenses, snowmelt, flood control gate operations, etc.

Real-time flood forecasting is used throughout the United States. The National Weather Service (NWS) prepares its flood forecasts in collaboration with agencies such as US Geological Survey (USGS), US Army Corps of Engineers (USACE), US Bureau of Reclamation, Natural Resource Conservation Service, National Park Service, and many state and local emergency agencies across the country (NWS, 2011a).

Flood forecasting is used to provide warnings for residents to evacuate areas threatened by floods and to assist water management personnel in operating flood-control structures, such as reservoir gates and gated spillways in dams. In flood forecasting, the forecast variables are the water levels of rivers, lakes, and reservoirs. The goal of flood forecasts is to determine the water levels, which result from flash floods, seasonal floods, dam breaks, and storm surges on estuaries and coastal areas with combined river and sea flooding. The forecasting period could range from a short period to a long period. A short forecasting period could be hours, whereas a long forecasting period could be weeks. Flood forecasting includes the steps of (Mays and Tung, 1992):

1. Obtaining real-time precipitation and stream flow data and forecasted precipitation;
2. Use of hydrologic and hydraulic models to simulate rainfall-runoff and stream flow by utilizing both the real-time and forecasted data;
3. Make forecasts of flood flowrates and water levels for either a short-period or a long-period depending on the needs of the environment and

size of the watershed.

A river-reservoir system is depicted in Figure 1.1. As seen in the figure, the flows upstream of the dam are entered into the reservoir. The time series of the incoming flows are depicted in the upstream inflow hydrograph. When it is necessary, such as creating flood storage in the reservoir, water is released from the reservoir. The time series of such reservoir releases is represented in a reservoir release hydrograph. The system can be applied in a forecasted scenario. For example, a forecasted streamflow from upstream rivers is entered into the reservoir through simulations. This forecasted information can be used to determine the actions necessary for operation of the reservoir gates ahead of the real storm event.

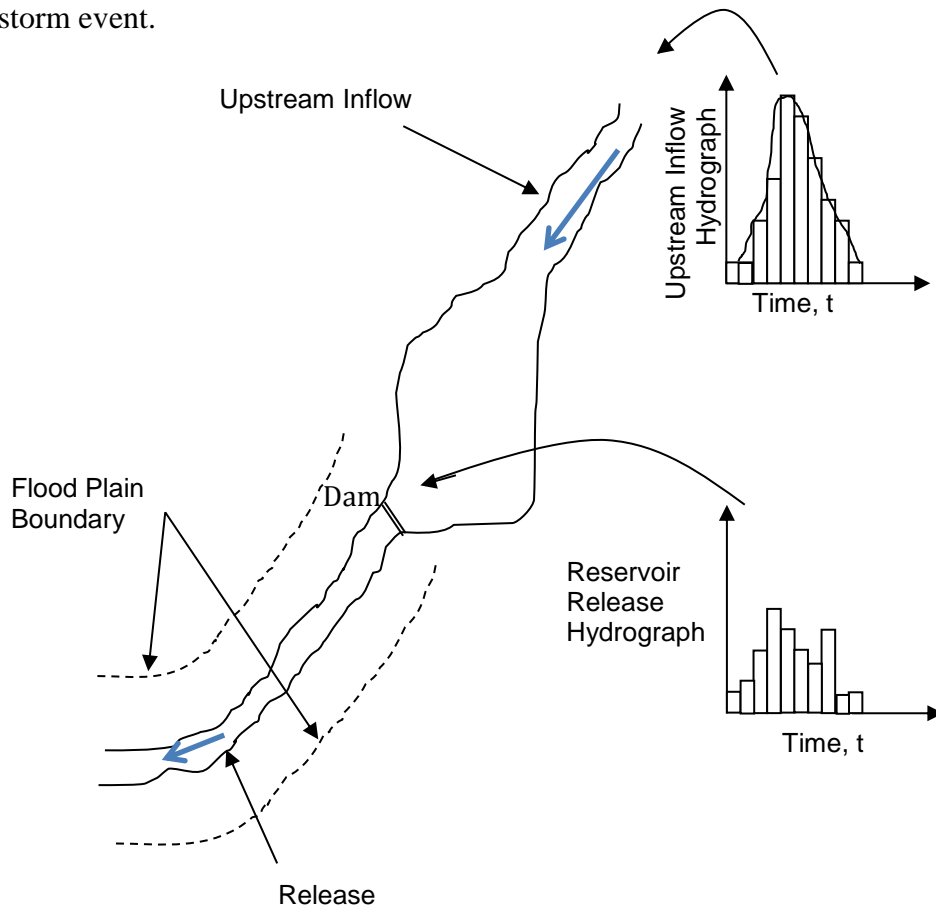


Figure 1.1: Schematic of a River-Reservoir System

In practice, unsteady flow simulation models are one of the approaches for streamflow forecasting. For a given set of operation policies, an unsteady flow (one-dimensional) simulation model can be used to simulate the flow rates, water surface elevations, and velocities at various locations for specified time steps. The basic equations that describe the unsteady flow (propagation of a wave) in an open channel are the Saint-Venant equations represented by continuity and momentum equations (Chow et al, 1988):

- Continuity Equation

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} - q = 0 \quad (1.1)$$

- Momentum Equation

$$\frac{\partial Q}{\partial t} + \frac{\partial(\beta Q^2/A)}{\partial x} + gA \left(\frac{\partial h}{\partial x} + S_f + S_e \right) - \beta q v_x = 0 \quad (1.2)$$

where

x is the longitudinal distance along the channel;

t is the time;

Q is the flow rate;

q is the lateral inflow;

β is the momentum correction factor;

A is the cross-sectional area of flow;

V_x is the velocity of lateral inflow in x-direction;

h is the water surface elevation in the channel;

S_f is the slope of the energy grade line;

S_e is the large-scale eddy loss slope for contraction/expansion, and

g is the acceleration of gravity.

Different types of unsteady flow models used in practice are presented in Chapter 5.

One of the most important criterion in flood forecasting is the *lead time*, which is the interval of time between the issuing of a forecast and the expected arrival of the forecasted event (Mays and Tung, 1992). Both time and location are important in flood forecasting. For example, a relatively short lead time for a short river reach may become a long lead time for locations much further downstream. Consider the scenario depicted in Figure 1.2 (Mays and Tung, 1992). There are three urban areas: A, B, and C; with a major rainfall in the upper region of the watershed. A short lead time is required for urban area A, with a longer time for urban area B, where urban area C has the longest lead time. Due to the time for the flood to travel down the river, a longer lead time is needed. The flood hydrographs at urban areas A, B, and C are shown in Figure 1.3, respectively.

In this example, the lead time for urban area A is very short but the lead time for urban area C is relatively longer. Moreover, the beginning of the flood hydrograph at urban area C occurs approximately at the same time the rainfall ends. This example also shows that, in order to forecast for a flood hydrograph at urban area A, precipitation forecasts are required, whereas for urban area C, the precipitation will be observed throughout the rainfall event in order to forecast properly. Often, several precipitation

forecasts are needed during the flood event. As shown in Figure 1.3, urban area A needs four rainfall forecasts, where urban area C requires one.

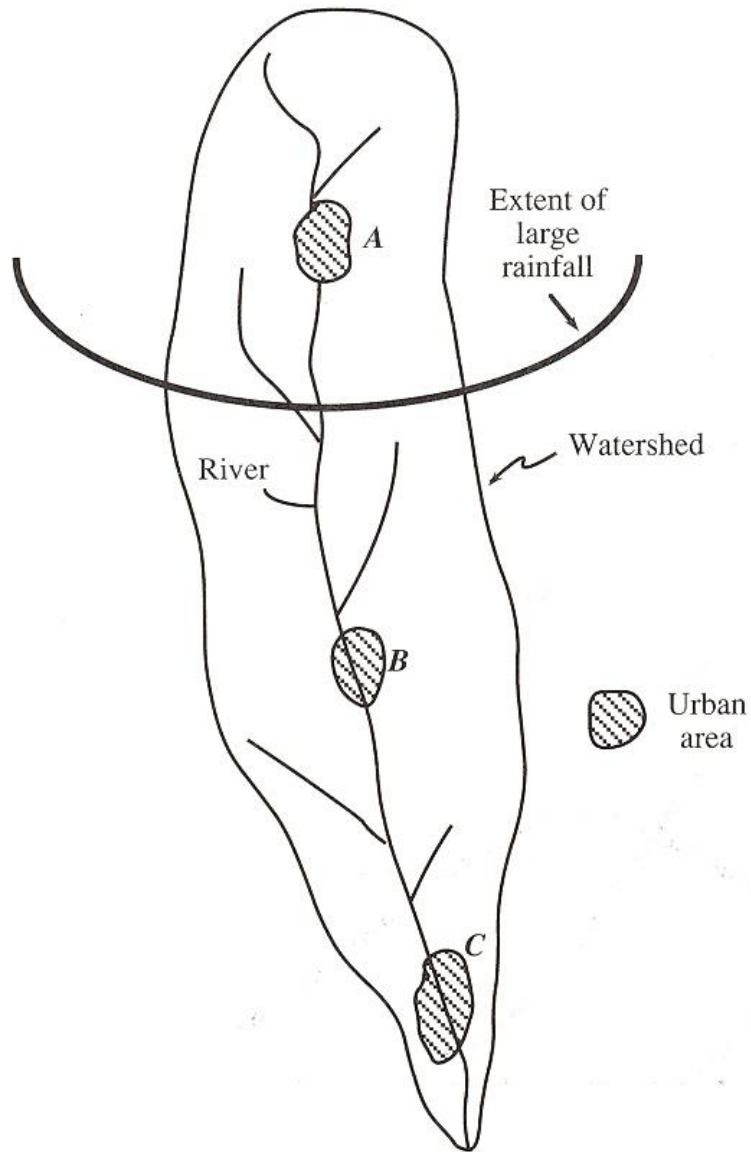


Figure 1.2: Effect of Lead Time (*Source: Mays and Tung, 1992*)

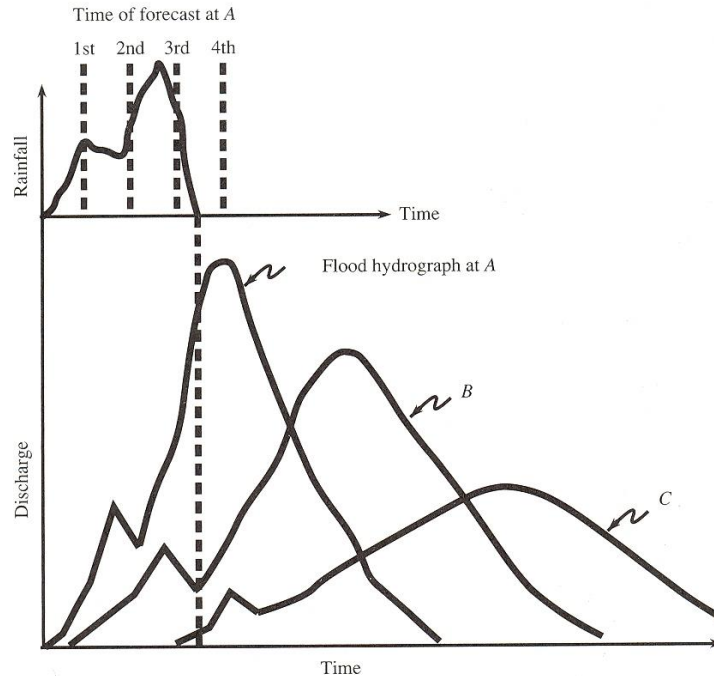


Figure 1.3: Flood Hydrograph at Downstream Location in a Watershed (Mays and Tung, 1992)

As of 2008, the NWS River Forecast Center would either use operational hydraulic models, the Dynamic Wave Operational Model (DWOPER) or the Flood Wave Dynamic Model (FLDWAV) in 29 separate river systems, covering 5500 river miles throughout the United States (NWS, 2011a). In 2007, a team of hydrologists from the NWS reviewed several well-known unsteady hydraulic models to identify methods to improve hydraulic modeling capabilities for NWS operational forecasting. NWS recommends including the United States Army Corps of Engineers' (USACE) Hydrologic Engineering Center-River Analysis System (HEC-RAS) in the operational environment. By testing FLDWAV and HEC-RAS on identical data sets, the NWS developed methods for transitioning FLDWAV models to the HEC-RAS (Reed, 2010 and Moreda, 2010). This is because the NWS believes that HEC-RAS offers more information of hydraulic structures, better documentation and training, the option to

illustrate more detailed cross-sections, and user-friendlier graphical user interfaces (GUI). Thus, NWS managers decided to replace FLDWAV and DWOPER with HEC-RAS. The transitioning process has been underway since 2009 (Reed, 2010 and Moreda, 2010).

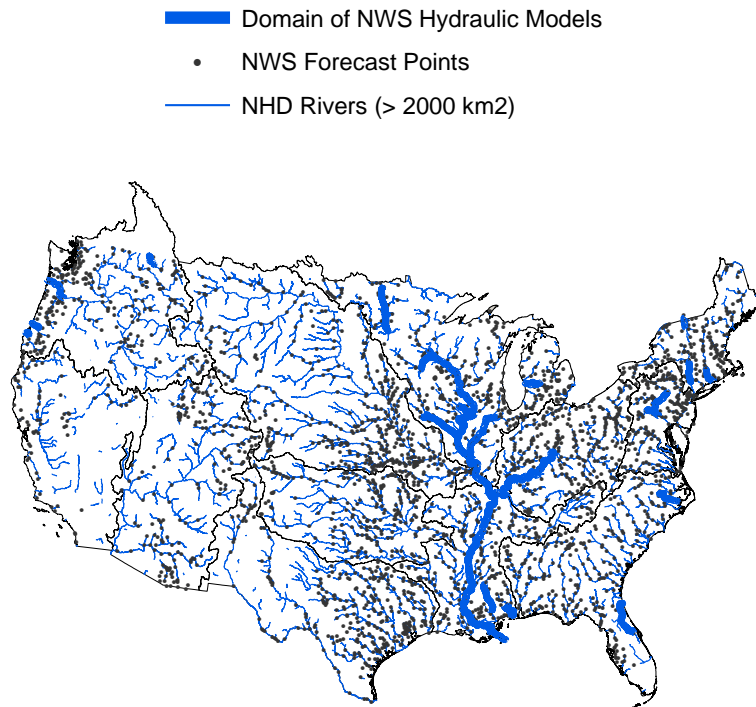


Figure 1.4: Domain of NWS Hydraulics Models (*Source: NWS, 2011a*)

Figure 1.4 illustrates the operational hydraulic models domain and the forecast point of the NWS River Forecast Center, as of 2008 (NWS, 2011a). Figure 1.4 also illustrates the National Hydrography Dataset (NHD) Rivers of the USGS throughout the United States. In this figure, river segments that have more than 2,000 km² (773 sq. mi.) are shown. The NHD is a digital vector dataset used by Geographic Information System (GIS). The NHD contains data that typically relate to lakes, ponds, streams, rivers, canals, dams, and stream gages. The NHD domain does not use unsteady hydraulic models for streamflow forecasting. Stage forecasts on these rivers are produced using

hydrologic routing methods, such as Tatum, SSARR, Lag-K, Layered Coefficient, Muskingum, and rating curves (NWS, 2011a).

Most NWS hydraulic models have been applied to rivers in the lowest slope regime (NWS, 2011a). The average slopes of rivers in the Continental U.S. (CONUS) are shown in Figure 1.5. However, there are many rivers throughout the country where implementation of hydraulic models should be considered. There are high potentials to improve hydrologic routing models in the low slope regime (green lines on Figure 1.5), as well as in the medium slope regime (orange lines on Figure 1.5) where at the least the diffusion wave approaches are recommended.

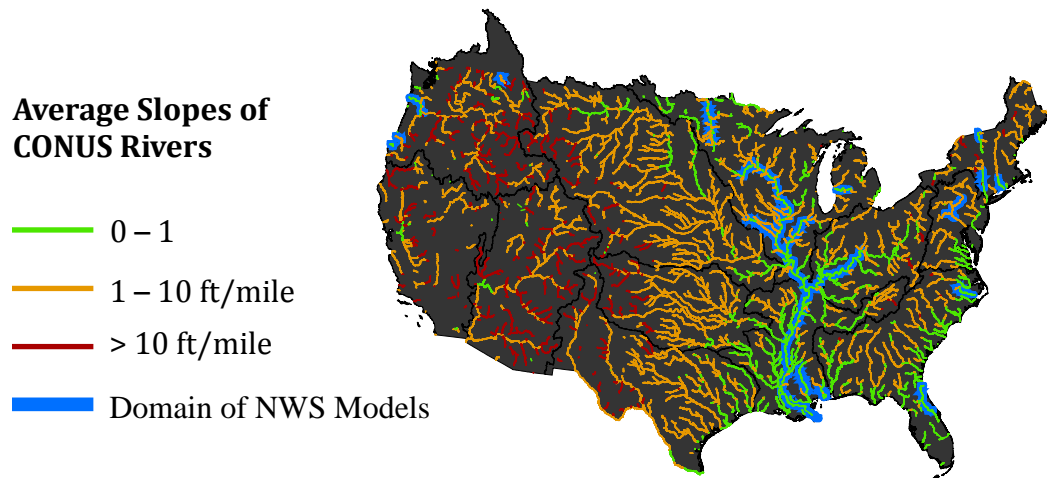


Figure 1.5: Average Slopes of Continental US Rivers >773 mi² Drainage Area
(Source: Reed, 2010)

Table 1.1 shows the approximate length of streams that are modeled hydraulically by the NWS and the length of streams that are rule-of-thumb candidates for diffusion and dynamic wave modeling. The 5500 miles hydraulically modeled rivers are only about

21% of the rivers with average slopes less than 1 foot per mile, and only 6% of the rivers with average slopes less than 10 feet per mile (Reed, 2010).

Table 1.1: Length of Stream in Different Categories from Figure 1.5 (*Source:* Reed, 2010)

Category	Length (miles)	Rule-of-Thumb Model Type
Length covered by NWS hydraulic models	5500	
Slope \leq 1 ft/mile	26236	Dynamic
1 ft/mile < Slope \leq 10 ft/mile	71063	Diffusion
Slope > 10 ft/mile	17116	Kinematic

Both slope and flood-rising rate influence the dynamic loop size strongly when applying rule-of-thumb type models. Thus the need for a more common use of hydraulic models has been suggested by the NWS. The information in Figure 1.5 and Table 1.1 suggest that there are potential benefits of a more widespread implementation of hydraulic models. Hydraulic unsteady flow models have not been implemented more widely for flood forecasting in the country for many reasons, such as: 1) there have not been adequate studies to convince forecasting agencies to invest in hydraulic unsteady modeling; 2) unsteady hydraulic models can be more difficult to apply in forecasting compare to hydrologic models because of the far more complex mathematical theory and numerical modeling; and 3) forecasters have developed techniques to modify hydrologic routing parameters in real-time to compensate for simulation inaccuracies. However,

advances in computing power, and improved GUI tools are making the implantation easier.

1.2 Real-Time Reservoir Operation

Reservoir operation for flood control is a complicated problem that involves a number of conflicting objectives. This includes the amount of water releases from reservoirs prior to the arrival of flood waters, the storage and water level in the reservoir during flood events, and ensuring reservoir gate releases during flood events will not heavily damage downstream areas. If a river basin consists of a system of reservoirs, the problem becomes even more complex, as each of the decisions made for one reservoir would have great impacts on the rest of the reservoirs in the system and also the flood conditions in the entire basin. Typically, decision makers of flood control reservoir operations use fixed reservoir rule curves and stage-discharge relationships to determine the reservoir releases based on the immediate reservoir stages. These fixed reservoir rules are based typically on past flood records. However, when facing an extreme precipitation event, traditional methods such as using reservoir stage-discharge relationships are not sufficient to achieve flood control objectives since most of these reservoir operation rules are not backed up by extreme flooding scenarios.

An optimization/simulation model should be used to help make real-time operation decisions (gate operations) for a river-reservoir system during flooding conditions by incorporating a real-time precipitation and stream flow data and forecasted rainfall throughout the system. The model should consists of five components, which are: 1) a rainfall forecast model, 2) a hydrologic rain-runoff model, 3) an unsteady flow routing model for the reservoir system, 4) a spillway-gate operation model for each of the

dams in the system, and 5) an optimization model for determining reservoir gate/spillway operation. An important part for the completeness of these components is a real-time operation model that predicts the results of a given operation policy for forecasted flood hydrographs. Figure 1.6 illustrates a real example. As one observes from the graph, precipitation events occurred in the month of March 1967 at Kanawha Falls, West Virginia (USACE, 1983). Real rainfall data were recorded from March 11th through March 19th. On March 19th, precipitation forecasts were made for the next several hours, which are represented by dashed line running vertical through the graph. In Figure 1.6 (a), the precipitation forecasts were made in the morning of March 19th, resulting in the ability to make forecasts of flood hydrographs. Similar phenomenon is seen in Figure 1.6 (b), where the precipitation and flood hydrograph forecasts were made in the evening on the same day.

The real-time reservoir operation problem involves the operation of a reservoir system by making decisions about reservoir releases as information becomes available, with relatively short time intervals, ranging from several minutes to several hours. Real-time operation of multi-reservoir systems involves many considerations, such as hydrologic, hydraulic, operational, technical, and institutional considerations. This will enable engineers in the field to make critical decisions about releases from the reservoirs in order to control floodwaters. For an operation to be efficient, a monitoring system is essential to provide the operator of the reservoir with the flows and water levels at various locations in the river system. These include upstream flow conditions, tributaries, reservoir levels, and precipitation data for the watersheds of which output (rainfall and runoff) are not gaged. Flood forecasting in general, and real-time flood

forecasting in particular, have always been an important problem in hydrologic engineering, especially when flood-control reservoir operations are involved.

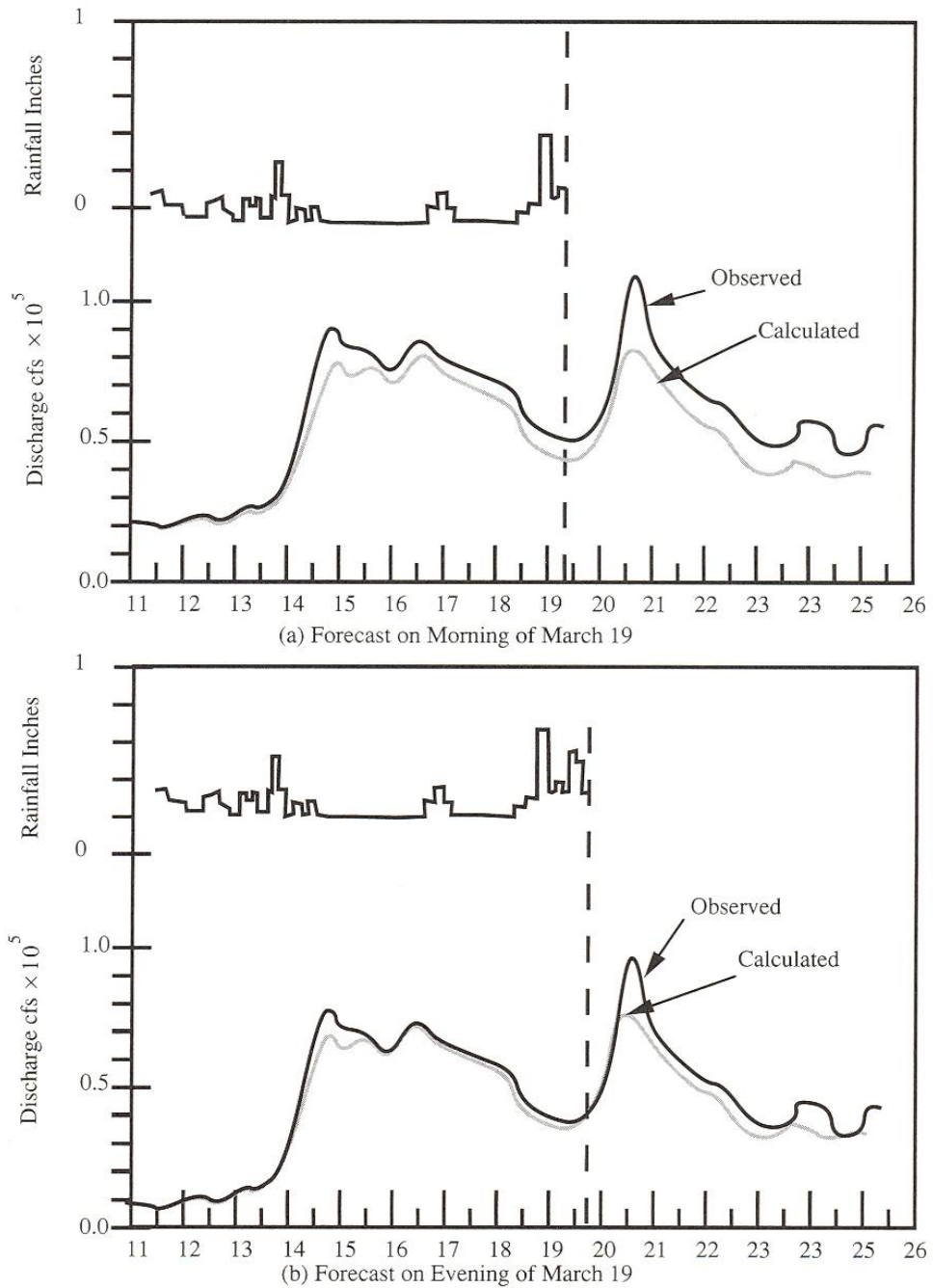


Figure 1.6: Observed and Forecasted Hydrographs at Kanawha Falls, Resulting from a Forecast of the March 1967 Flood Event (*Sources: U.S. Army Corps of Engineers, 1983*)

The forecasting problem can be viewed as a system with inputs and outputs. The inputs of the system are inflow hydrographs at the upstream end of the river system and runoff from rainfall in other catchments converging to the system. The outputs of the system are flow rates and/or water levels at points of interest in the river system (Mays and Tung, 1992).

1.3 The Need for an Optimization/Simulation Model for Determining Real-Time Optimal Operation of River-Reservoir Systems during Flooding Conditions

The value of a real-time flood-management model is shown by a real flood event on September 1952 in the Highland Lake System of the Lower Colorado River Basin (LCRB). The LCRB is illustrated in Figure 1.7. The 42,000 square miles LCRB extends across the Texas down to the Gulf Coast. Major tributaries of the Colorado River near and in the area of the Highland Lake System are the Concho River, Pecan Bayou River, San Saba River, Llano River, and the Pedernales River. All of these tributaries enter the Colorado River upstream at Lake Travis. The Highland Lake System consists of six reservoirs and dams, which are Lake Buchanan (Buchanan Dam), Lake Inks (Inks Dam), Lake LBJ (Wirtz Dam), Lake Marble Falls (Starke Dam), Lake Travis (Mansfield Dam), and Lake Austin (Tom Miller Dam) (Mays, 1991).

A large reservoir system, such as the Highland Lake System, is considered to be integrated in the operation of multiple facilities for multiple objectives such as, flood control, water supply, and recreation. Major flood problems occur more often when water is released from the reservoirs due to the development on the flood plains of the Highland Lake System. For example, Lake Travis is designed to provide 780,000 acre-

feet of flood storage, combined with a target release of 90,000 ft³ per second, which provides flood protection to the city of Austin and many downstream areas. However, a series of development encroachment on the flood plain that is downstream of Lake Travis has reduced the safe releases (non-flooding condition) to less than 30,000 ft³ per second—too low to result in any flow over the Mansfield Dam, uncontrolled overflow spillway (USACE, 1979). Flood control operation of the Highland Lake System is further complicated by Lake Buchanan and Lake Travis, because they are the only two lakes designed to store substantial floodwaters (Mays, 1991). In fact in 1952, there was a flood event that overwhelmed the system and which is a good case to examine here to highlight the issue.

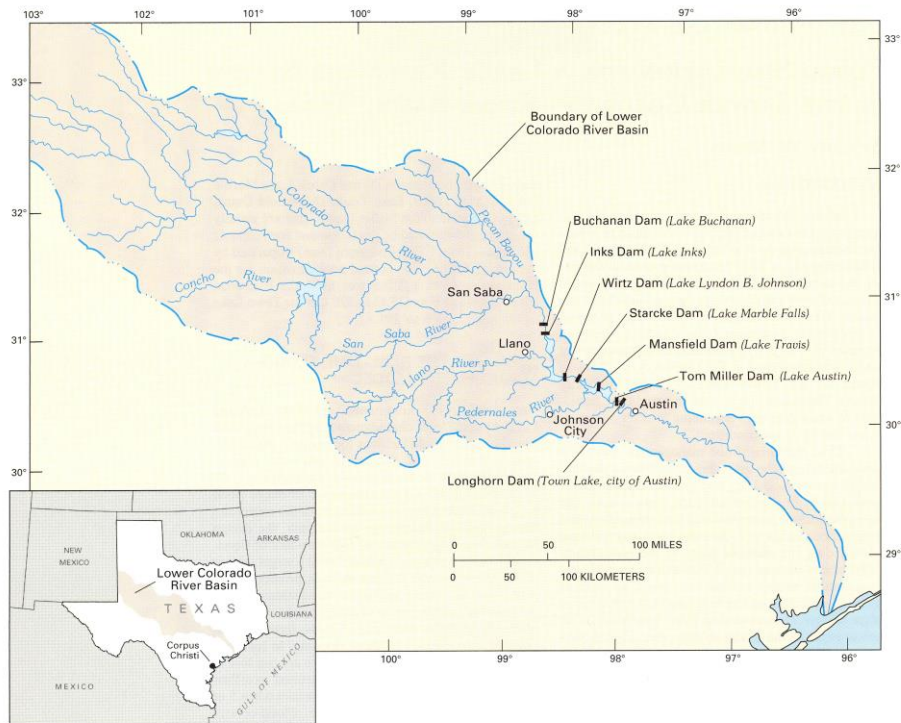


Figure 1.7: Reservoir and Dams of the Highland Lake System in the Lower Colorado River Basin, TX (Sources: Mays, 1991)

In September 1952, the flood event in the Highland Lake System exceeded all known previous floods in the region at many points in the basins of the San Saba, Llano, and Pedernales Rivers (Mays, 1991). In the beginning of September 1952, the Highland Lake System had storage of 374,000 acre-feet, only 30% of the conservation storage. From September 9th through the 11th, 2 to 26 inches of rain fell on an area of 100 miles by 250 miles in the basins. On September 9, Lake Travis had an estimated peak inflow of 840,000 ft³ per second and the water level rose 56 feet in less than 24 hours. During the flood event, five persons lost their lives, 71 homes were destroyed, and 453 homes were damaged. In 1979, the U.S. Army Corps of Engineers estimated that the peak flow of the 1952 flood event in Austin would have been 803,000 ft³ per second if Lake Travis had not had the capacity to store most of the floodwaters. Figures 1.8 illustrates the severity of the flood event at Lake Travis (Unver, 1987).

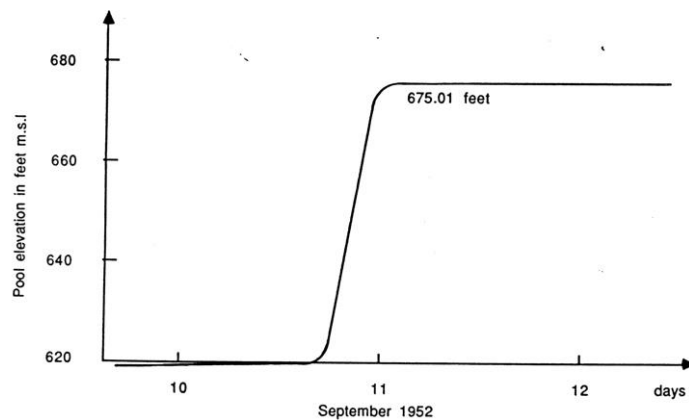


Figure 1.8 (a): Lake Travis Water Elevation (*Source*: Unver, 1987)

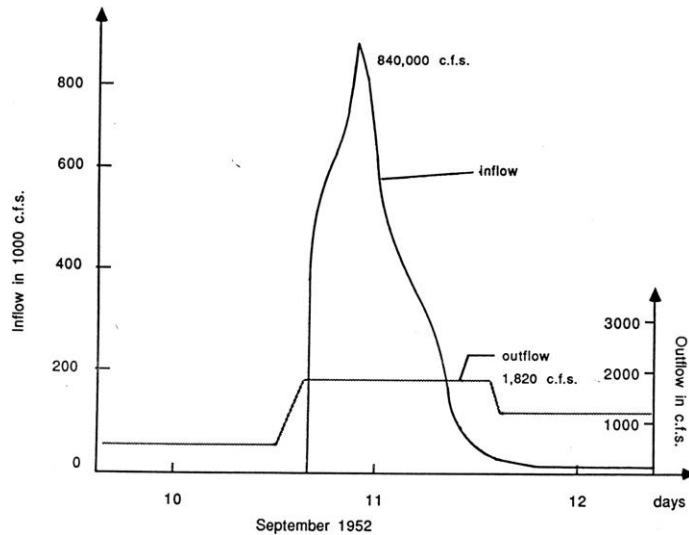


Figure 1.8 (b): Inflow and Outflow from Lake Travis (*Source: Unver, 1987*)

In the 1950s, flood managers clearly did not have the modern technology for flood simulations let alone real-time flood forecasting. However, there are still today many large river-reservoir systems throughout the United States and around the world that do not have real-time reservoir operation strategies during flooding conditions. The application of real-time operations of river-reservoir systems is still not widely adapted and remains in its infancy in many other places. Unfortunately, the antiquated processes in different regions in the world has resulted unnecessary flooding events and loss of life which could have been easily prevented. The Lower Colorado River Authority eventually adapted a real-time flood management model in the 1980s, which was developed at the University of Texas at Austin (Unver 1987, Unver et al, 1987, Unver and Mays 1990, and Mays 1991). A detailed description of the LCRB flood management model is presented in Chapter 3. The following section describes the May, 2010 flood event in the Cumberland River System in Tennessee in which Nashville suffered 2 billion dollars in damage. The operation of the Old Hickory dam upstream of Nashville nearly

resulted in the failure of the dam. This example by the USACE clearly illustrates the need for the type of model proposed in this research.

1.4 Cumberland River System – 2010 Flood Event

1.4.1 Cumberland River Basin

This research focuses on both the development and application of a new optimization-simulation model for the real-time operation of river-reservoir system. The application will be applied to the Cumberland River Basin during the May 2010 flood event. The Cumberland River Basin lies entirely within the states of Kentucky and Tennessee and has a total area of 17,914 square miles, of which 10,695 square miles (60%) are in the state of Tennessee. The topography of the basin varies from rugged mountains in the eastern upstream portion to rolling low-plateaus in the western, or downstream sector. Elevations range from 4,150 feet above mean sea level (msl) in the Cumberland Mountain to 302 feet in the pool at the mouth of the river (USACE, 2010c).

The U.S. Corps of Engineers Nashville District maintains and operates five projects on the Cumberland River main stream and five projects on its tributaries. The projects in the mainstream are Cordell Hull, Barkley, Cheatham, Old Hickory, and Wolf Creek. Congress authorizes only Barkley and Wolf Creek for flood risk management. Congress authorizes Barkley, Cheatham, Old Hickory, and Cordell Hill for the purposes of hydropower and commercial navigation. The five Corps of Engineers tributary projects, which are Dale Hollow, Center Hill, Martin's Fork, Laurel, and J. Percy Priest are congressionally authorized for flood risk management (USACE, 2010c, 2012).

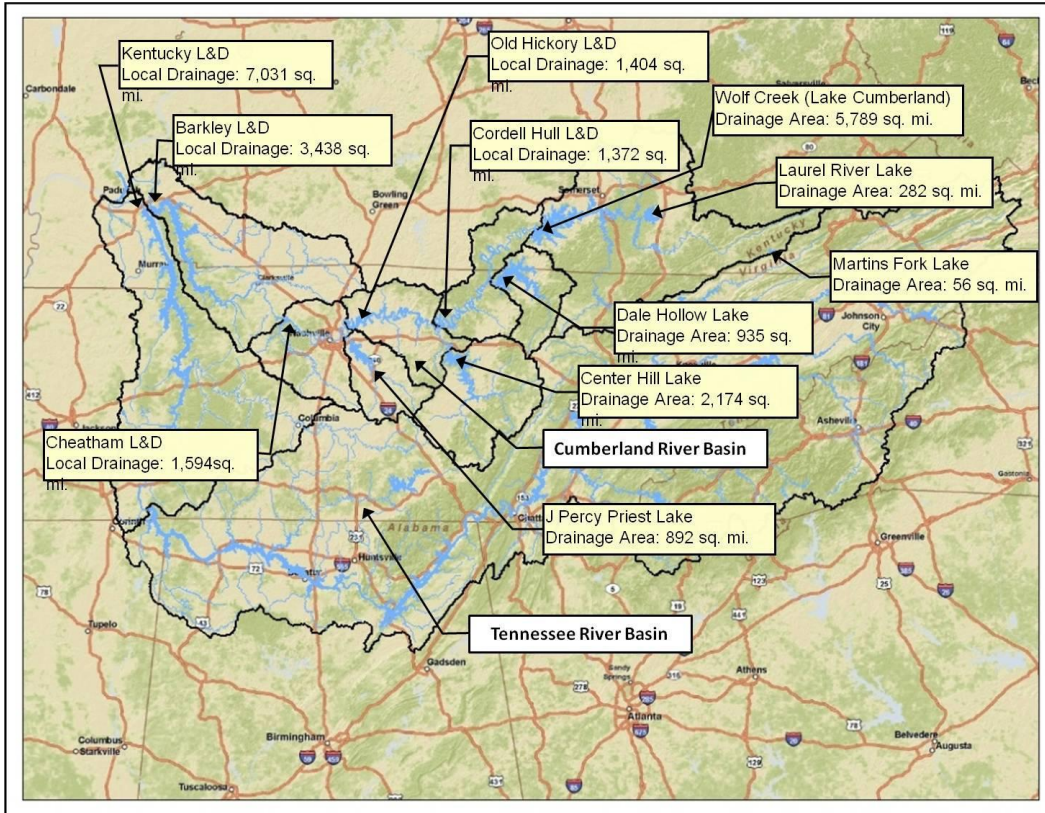


Figure 1.9: The Cumberland River Basin (Source: USACE, 2010c)



Figure 1.10: Existing Projects on the Cumberland River (Source: USACE, 2010c)

In-depth descriptions on these existing reservoir projects and their roles are presented in Chapter 2. Figure 1.9 and 1.10 illustrates the general map of the Cumberland River Basin with its surrounding areas, and the existing projects.

1.4.2 May 2010 Flood Event

In May 2010, portions of the Cumberland River Basin experienced a 36-hour rainfall that produced record flooding (USACE 2010c, 2012). Officials estimated the two-day storm to be far greater than a 1,000-year rain event. Catastrophic flooding occurred in greater Nashville, western Kentucky, and central Tennessee on May 1st to 4th, 2010. The event began with heavy rain on Saturday, May 1st. There were numerous flash floods and rivers quickly exceeded their banks. A second period of heavy rain occurred over much the same area on Sunday May 2nd, resulting in a repeat of flash flooding and escalated river flooding to record flood levels (USACE, 2010c, 2012). During Sunday afternoon and evening hours, a critical period, the NWS and USACE did not communicate effectively regarding updated releases from USACE reservoirs. This lack of mutual understanding and critical information exchange of each other's operations led to inaccurate river crest forecasts on the Cumberland River. The USACE personnel were completely engaged in critical operations to prevent damage to structures or dams along the Cumberland River as the flooding intensified. However, with incorrect or untimely information from the USACE about their own operations, as well as miscommunications and ineffective information exchanges between the USACE and NWS, NWS forecast crests were quickly exceeded on Sunday, May 2, when the river stage in Nashville rose rapidly. Throughout that weekend into the following Monday morning, the Cumberland River at Nashville gage rose more than 33 feet, cresting at 52 feet on Monday night and Tuesday May 3rd. This crest stage was approximately 4 feet higher than any other previous peak, the highest of which was 48 feet in 1975 prior to this event, and 10 feet higher than the original forecast issued Sunday morning. Consequently, record

discharges were set from USACE projects in the Cumberland River Basin, including those at Cordell Hull, Old Hickory (just upstream of Nashville), Cheatham, and Barkley. Sadly, 26 people lost their lives due to flooding, and property damage estimates in Greater Nashville alone were over \$2 billion (USACE, 2010c, 2012). Figure 1.11 illustrates the severity of the flood event that showed that the flood stage was well above the major flood stage at the Nashville gage. A detailed description of the event and reservoirs operation is presented in Chapter 2.

1.4.3 Operation of the Old Hickory Dam during the Flood Event

The primary control location for the release from the Old Hickory Dam is Nashville, Tennessee, which is about 25 miles downstream of the dam (USACE, 2010c). Flow propagate through Nashville is directly affected by the releases from the Old Hickory Dam and the J. Percy Priest Dam as illustrated in Figure 1.10. J. Percy Priest is a flood control structure so it has a greater capacity than the Old Hickory. However, the J. Percy Priest Dam is on the Stone River, which is a tributary river to the main Cumberland River. The Old Hickory, which is on the Cumberland River main stream, is not a flood control dam. The Old Hickory project does not have any flood control storage capability. It does, however, have a small amount of space dedicated to flood storage. The Old Hickory is permitted to have pre-flood drawdown prior to the arrival of the flood waters. The Old Hickory Dam has certain guidelines for operation during a storm event (USACE, 2010c, 2012). For instance, according to the Old Hickory Water Control Manual, the six gates must be opened uniformly as soon as the headwater rises above 447 ft, as shown in Table 1.2 (USACE, 1998).

Table 1.2: Old Hickory Dam Spillway Releases for various Headwater Levels (USACE, 1998)

Headwater Elevation (feet)	Minimum Gate Opening (feet)	Minimum Spillway Discharge (cfs)
445	0	0
446	0	0
447	0	0
448	1	7500
449	2	14880
450	3	22440

According to the control manual, as flooding progresses, the Old Hickory discharges are increased and flow rate in Nashville are allowed to reach control levels before any storage is used. Once the control flows are reached, the J. Percy Priest discharge is then reduced to maintain the control flow in Nashville. If the Nashville control flow cannot be maintained, then flood storage of the Old Hickory is utilized (USACE, 1998).

Prior to the May 2010 storm event, the U.S. Corps of Engineers did not conduct any substantial pre-flood drawdown from the Old Hickory. As seen in Figure 1.11, there were not any significant releases from the Old Hickory Dam until afternoon on May 1, even though the storm had started in the early morning hours that day. The dam releases increased exponentially on May 2, as the storm entered the second major wave of the event. Towards the end of the storm event, the Old Hickory had the flow rate of nearly 200,000 ft³ per second, and the reservoir levels were about a foot away from overtopping the dam, which could have resulted in a complete failure of the Old Hickory (USACE, 2010c, 2012). Throughout the event, there was no application of real-time strategies for

the reservoir's management, nor was there any flood forecasting information being utilized.

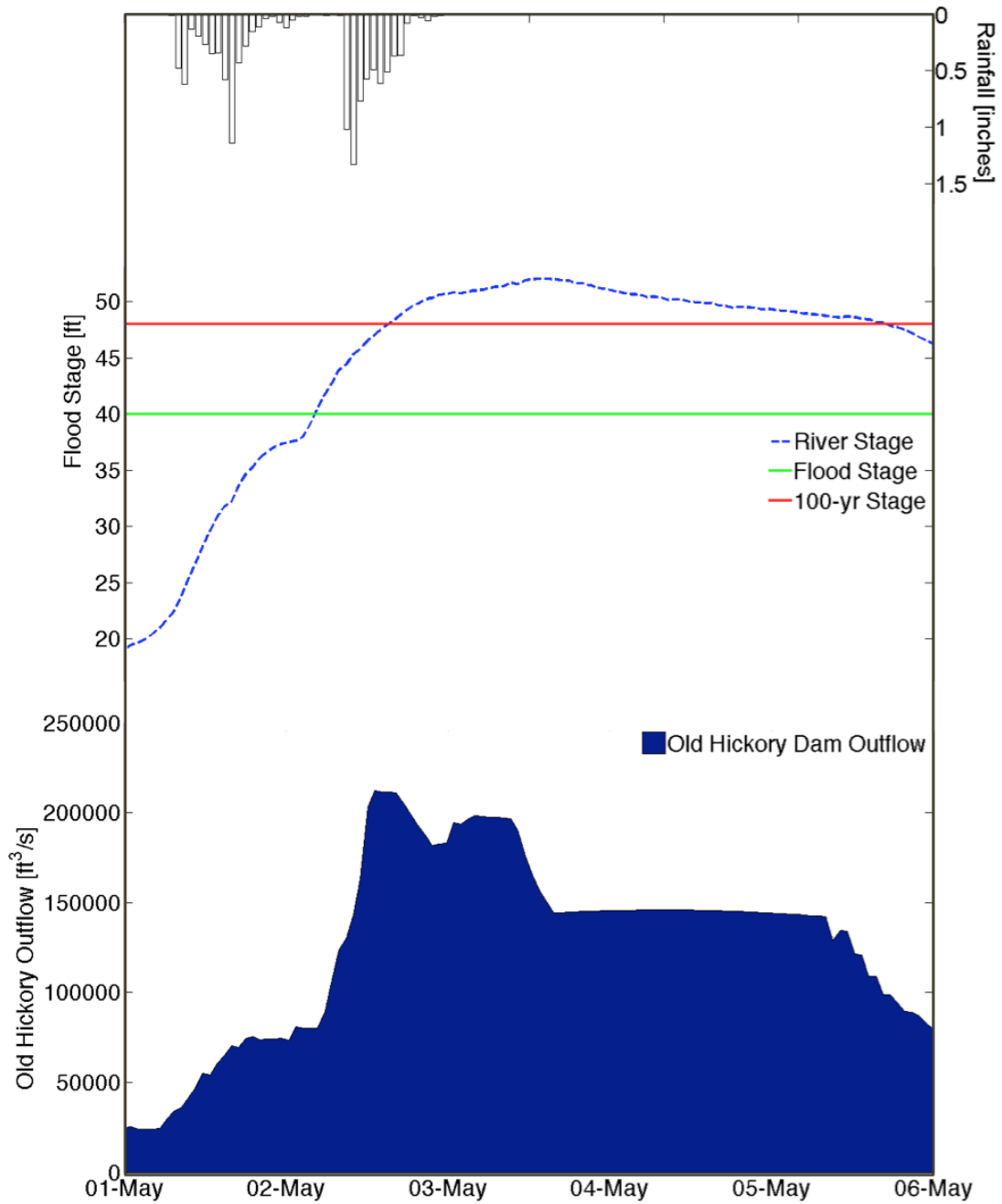


Figure 1.11: River Stages and Flowrate at the Nashville Gage (*Source: USACE, 2010c*)

1.5 Research Objectives

The objective of this research is to develop an overall methodology for determining reservoir release schedules, which are implemented prior to, during, and immediately following an extreme flood event in real time. By doing so, the floodwater flows and flood elevations are kept under the desired target levels. The problem is formulated as a real-time optimal control problem in which reservoir gate openings represent the decision variables. Figure 1.12 illustrates the basic steps of the optimization/simulation model algorithm. First, the model requires real-time rainfall data (NEXRAD data) to start the rainfall-runoff simulation. Once the watershed hydrographs are obtained, they are entered into an unsteady flow simulation as inputs. Once the floodwaters are routed to the locations where the reservoirs are located, the flow data enters into an optimization model to compute the real-time operation decisions (gate operations) of reservoirs. The model would then generate the initial reservoir gate operation. Once the optimization model determines the solutions, the values of the flowrate at the time of the reservoir gate releases, are reentered into the unsteady flow model to simulate the flow of further downstream locations. When the floodwater enters the target location (i.e. Nashville), the model determines whether or not the objective is met. For example, in determining the successful completion of the objective, the model calculates whether or not the water levels are controlled and under the desirable level. If they are not, the model returns to the gate optimization process to determine an improved reservoir operation. If the objective is met, the model will repeat the overall optimization/simulation process for the next forecast period (over the next Δt), if the forecasted rainfall data is available. The rainfall forecasting is based on the known

rainfall up to current simulation time, t . The optimization/simulation model continues until the very last simulation period, at which time, the model stops.

Figure 1.13 shows the components of the real-time river reservoir system operation model. The model will be applied to both the simple hypothetical example (as illustrated in Figure 1.14) and the 2010 Cumberland River flooding event in Nashville, Tennessee. The optimization/simulation model for real time reservoir operation model consists of several major components. The first component is the hydrologic model HEC-HMS, which simulates precipitation-runoff processes of watershed systems (USACE, 2000a and 2010b). The second component is the hydraulic unsteady flow model HEC-RAS, which allows users to perform one-dimensional, unsteady flow computation (USARC, 2010a). The third component of the methodology is the reservoir operation model for gate operation. The fourth component is the optimization model for searching the optimal decision variables. The last component is the short-term rainfall-forecasting model. The NWS gridded rainfall values and/or rainfall values from a gage network are used as the actual rainfall data until it is time to resume reservoir operations. In addition, the model will measure real-time flood elevations in a river-reservoir system. Subsequently, a methodology of projecting short-term rainfall must be developed in the immediate minutes and hours after the reservoir operations resumed.

An optimization procedure based upon a genetic algorithm (GA) optimizer interfaces the other component of the model to determine actual gate operations during the real-time operation of the reservoir systems.

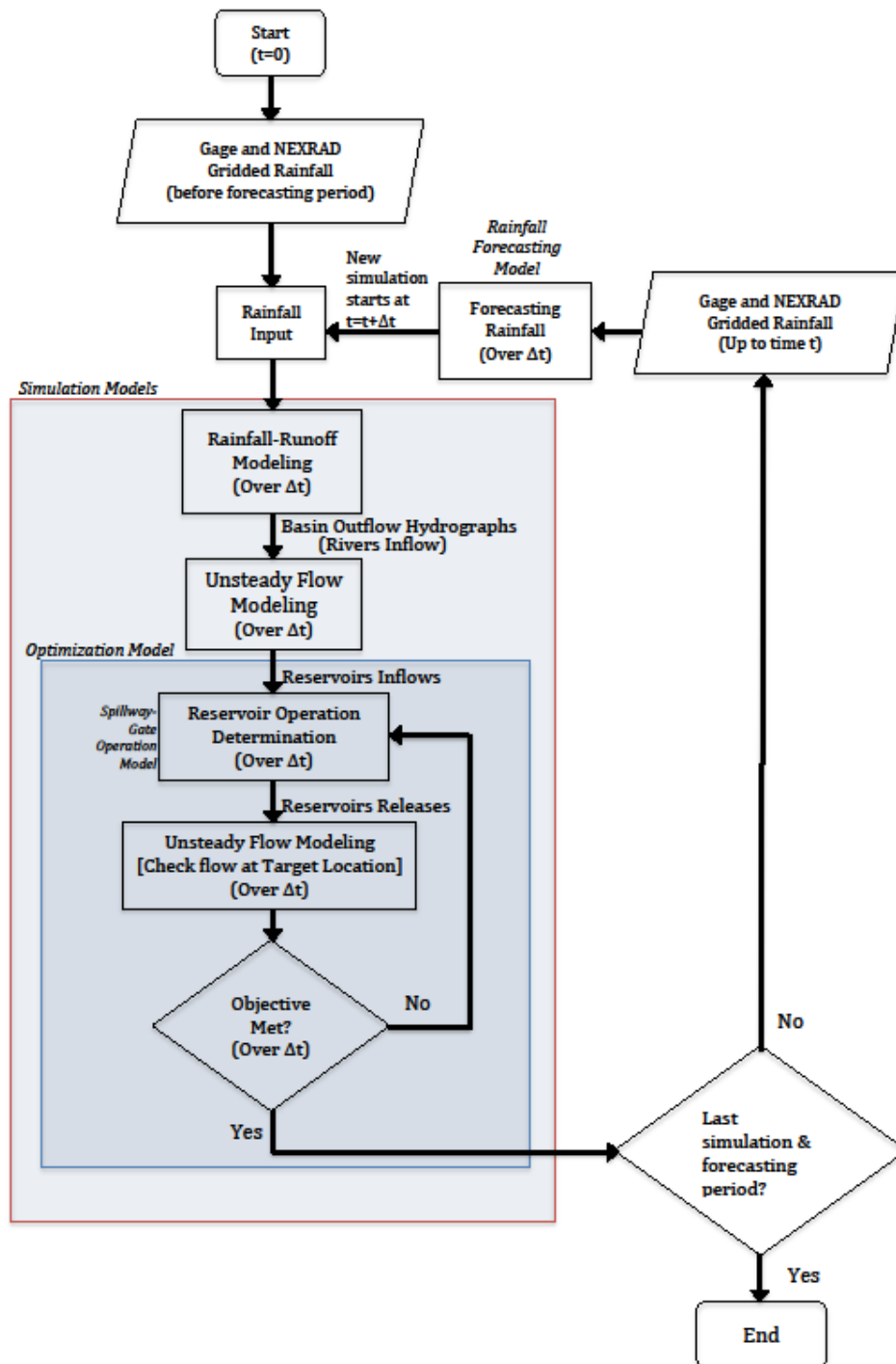


Figure 1.12: Basic Steps of the Optimization/Simulation Model

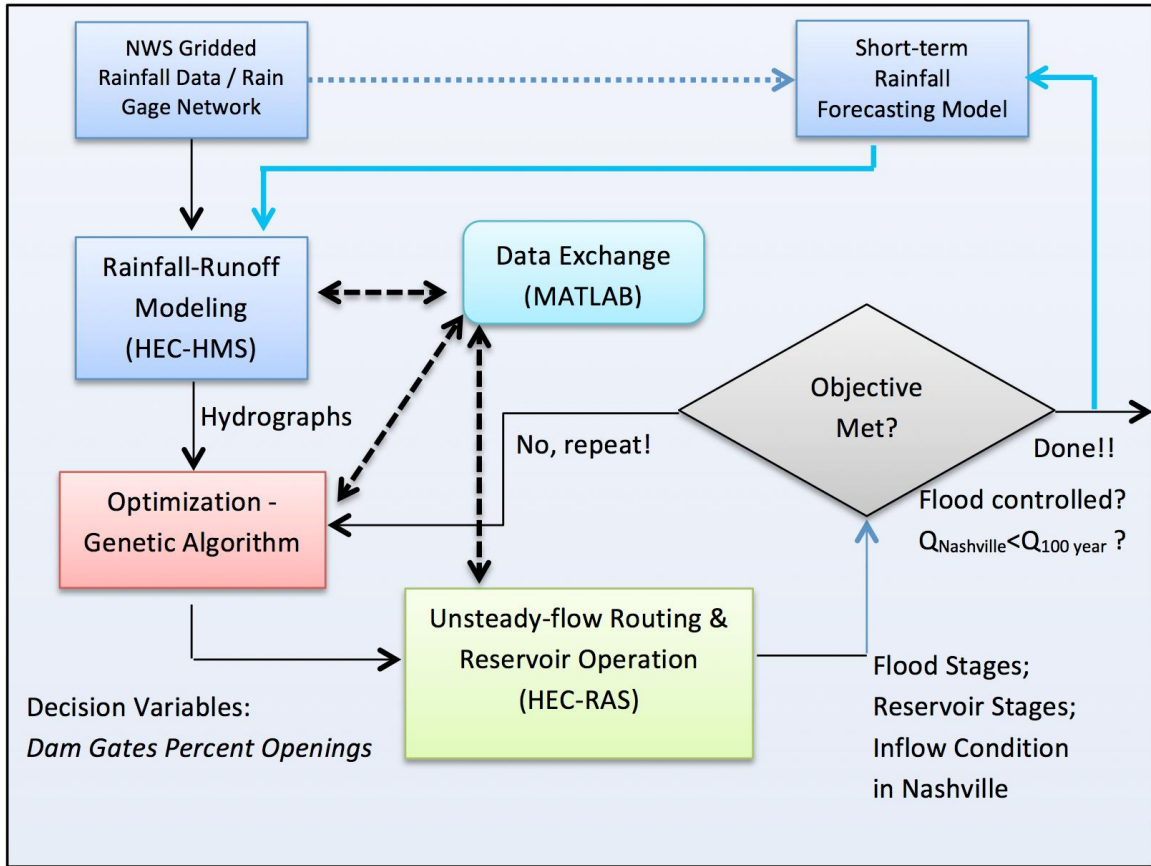


Figure 1.13: Interconnection of Components

Figure 1.13 offers a brief description of the interfacing of the components of the real-time river reservoir system operation model are now explained with much greater detail provided in later chapters. The National Weather Service gridded rainfall data (NEXRAD) is used to run simulations of the watershed rainfall-runoff model, HEC-HMS, and then the hydrographs are used as inputs of the optimization model to determine the gate openings and releases of the reservoirs in a river reservoir system. Once the sets of feasible (or optimal) solutions (i.e. gate openings or reservoirs releases) are determined, the decision variables are entered into the unsteady flow routing model HEC-RAS to simulate the floods in the river-reservoir system.

The optimization method used in this research is the genetic algorithm (GA). The genetic algorithm does not require a well-defined function unlike some classical methods like the simplex and Lagrangian gradient methods, which do. First developed in the 1970s, the genetic algorithm is a model which mimics Charles Darwin's Theory of Evolution by Natural Selection. Genetic algorithm generally consists of three operators: selection, crossover, and mutation. Genetic algorithm is used here because of the advantages over traditional optimization algorithms, like its ability to deal with complex optimization problems and parallelism (Holland 1975, Goldberg 1989, Mitchell 1996, and Deb 2001). A detailed description of the genetic algorithm and how it is incorporated in the real-time optimization/simulation model is presented in Chapter 7 and in the Appendix.

In this research, the main objective of the methodology is to control the flood flows and flood elevations at various locations of a river-reservoir system. One example might be to keep the flowrates and flood elevations below the 100-year level. If the objective is not met, the genetic algorithm optimization would repeat its process to determine the reservoir's release until the objective is achieved. Once the objective is achieved, the model moves to the next iteration. At that time, the short-term projected rainfall is used to run the precipitation-runoff model to determine the reservoirs operation for the next forecasting period. The real rainfall data is then used to compute the actual watershed runoff, reservoir stages, release of reservoir gates, and the unsteady flows. The process repeats and continues until the objective is met and all constraints are satisfied for the entire simulations period. The reason for the model to start simulation days before the storm events is that, it can determine which actions are necessary for the

reservoir to take in order to prepare for the floodwaters for the coming days. A detailed description of the real-time reservoir operation model is presented in Chapter 6 and Chapter 7.

1.6 Research Phases

1.6.1 Phases of Model Development

- Studied literature reviews on real-time forecasting, rainfall-runoff models, and unsteady flow models.
- For my research, I based my model upon and using the HEC-HMS and HEC-RAS, Genetic Algorithm Solver, M.S. Excel, and MATLAB version R2014b. I also used a free open software Pulover's Macro Creator version 4.1.0, so that a data exchange system can be programmed to interface data among the other components of the modeling system.
- Determined the programming languages needed to perform the interfacing of the various model components including the rainfall projection software, HEC-HMS, HEC-RAS, the NEXRAD rainfall data, and the genetic algorithm for the optimization routine.
- Searched for the best way in which the genetic algorithm can be used in selecting gate operations of the various reservoirs.
- Developed a model to forecast short-term future rainfall for hours in advance of a known rainfall.
- Performed extensive testing of all the model components.

- Created a simplified hypothetical model (see Figure 1.14), illustrated below, after the model components had been tested.

This research addresses the importance of using real-time and forecasted data for real-time flood control operation of a river-reservoir system. It is also important to first demonstrate the methodology using a simple hypothetical scenario. A simple two basin – two reservoir model was developed. Figure 1.14 illustrates the schematic of the simple application. Using designed storms and projected rainfall, the model is able to determine the reservoir releases such that the objective of flowrate control is achieved. This simple application is presented in Chapter 8.

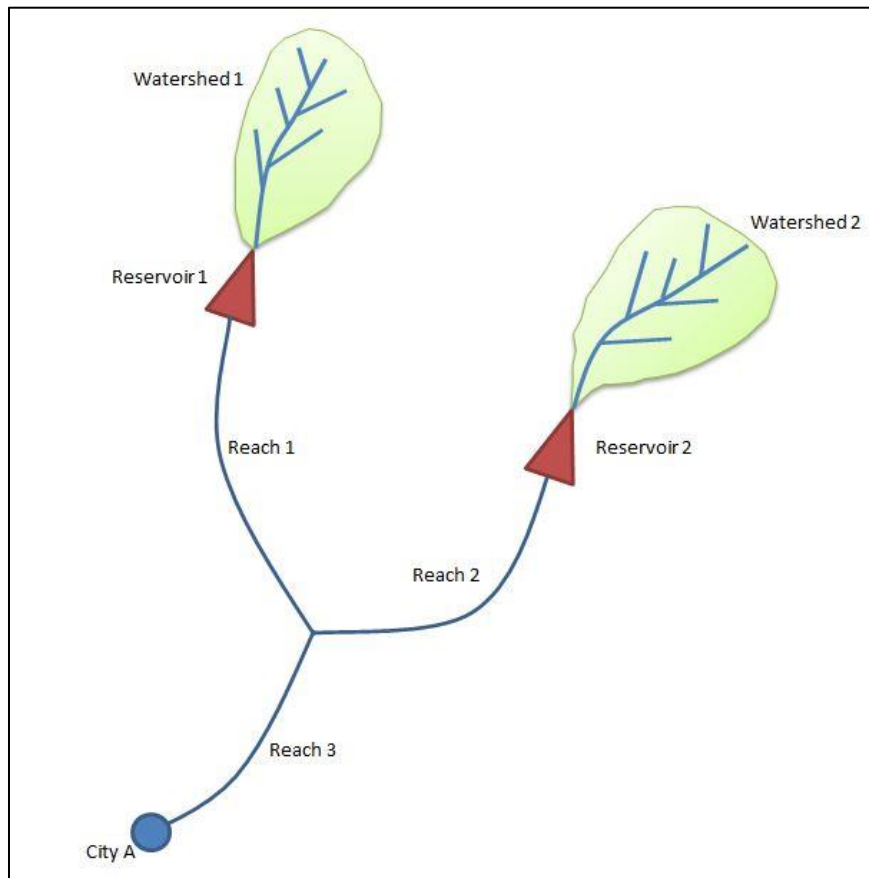


Figure 1.14: Schematic of the Simple Model

1.6.2 Phases of Model Application

A demonstration of the model performed using the data from May 2010 flood event on the Cumberland River system. The U.S. Army Corps of Engineers developed the HEC-HMS and HEC-RAS models for the 2010 flood event.

- The model was applied to a portion of the Cumberland River system that includes the Cordell Hull Dam, J. Percy Priest Dam, and the Old Hickory Dam (see Figure 1.15 below). These are the three dams that have the most impact on the Cumberland River upstream of Nashville, Tennessee.
- I performed a detailed study of the Old Hickory dam operations during the 2010 flood event while considering the actual operation led by the U.S. Army Corps of Engineers, and the operation rules established years prior.
- The results of the study prompted a optimization/simulation model of the 2010 Flood event in order to test alternative operation methods.
- The simulation provided an improved operation method, which would have prevented much of the damage sustained in Nashville.
- For future work, the model will be expanded to the entire Cumberland River System. This phase of the research is to apply the methodology to all the reservoirs in the Cumberland River Basin for multi-purpose scenarios.

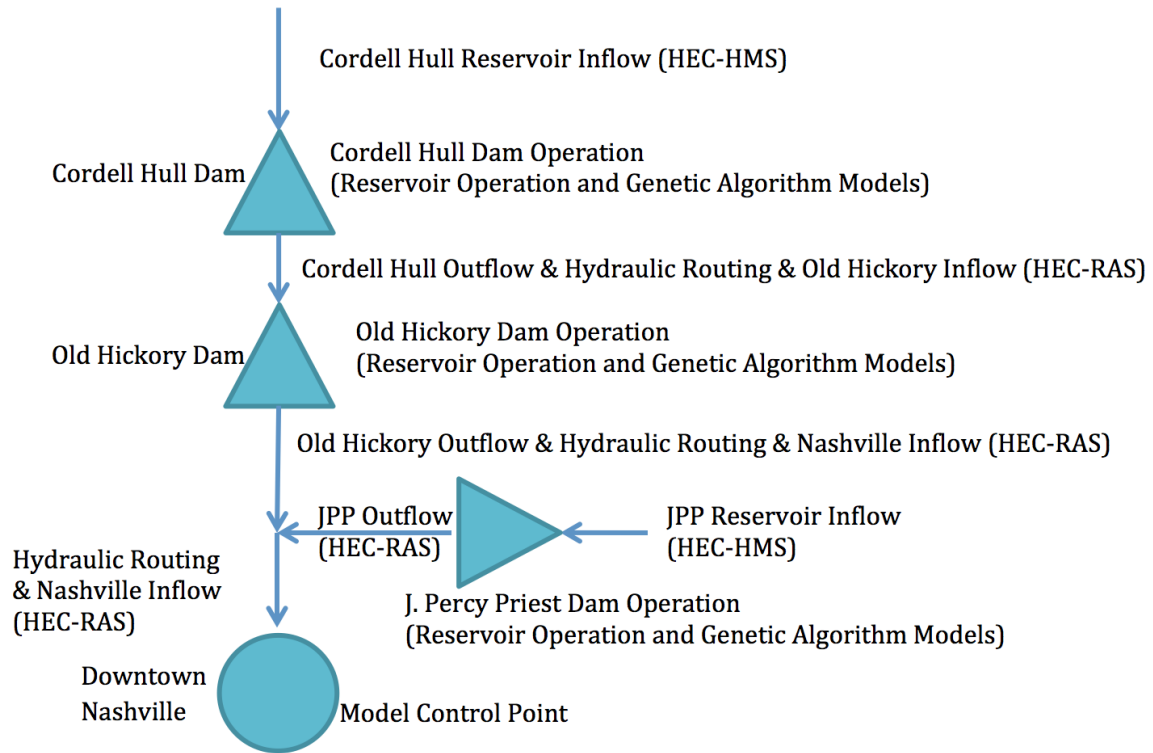


Figure 1.15: Reservoirs on the Cumberland River near Nashville

1.7 Relevance of Research

During the May 2010 flood event in Cumberland River Basin, the operations of the reservoirs in the basin were not performed in real-time. Independent from other reservoirs, the release from each reservoir was determined using the traditional fixed reservoir rule. Real-time flood forecast information provided by the NWS was not fully utilized by the USACE, resulting in record flooding in Nashville, and record river stages in the Cumberland River Basin. Much of the catastrophic flooding could have been prevented if a systematic reservoir operation had been performed using the model proposed in this research. The reservoir system operations should be determined using an optimization/simulation approach. This is because any of the decisions made at a

particular location could have major impacts on the rest of the reservoirs and rivers in the system. Utilizing an optimization/simulation approach, the entire reservoir system in the basin was taken into account, and the decision process no longer considered just an individual reservoir, rather the effect reservoirs had on the entire system. The simulation-optimization framework uses real-time information to simulate forecasted stream flows and river stages to generate an optimal operation for all the reservoirs in the system. If the simulation-optimization framework had been applied in real-time during the 2010 flood event in the Cumberland River Basin, much of the catastrophic flood damage would have been reduced, or even prevented. Real-time operation policies that are created using the simulation-optimization framework can be applied to other reservoir systems. Thus this approach is extremely relevant for real world applications.

1.8 Organization of the Proposal

Chapter 1 of this dissertation, I provided the background and overall scope of the research I conducted. I offered brief analyses on real-time flood forecasting, real-time reservoir operations, the 2010 Cumberland River Basin flood event, and the research method. Chapter 2 examines the May 2010 flood event in greater detail. I attempted to summarize the intensity of the storms, the damage they caused, and the responses by the U.S. Army Corps of Engineers, USGS, NWS, and the Tennessee Valley Authority (TVA). Chapter 3 is a survey of current real-time operation models, such as the NWS models, and the model being used in the Lower Colorado River Authority. Chapter 4 presented some of the most currently used precipitation-runoff models in practice today. Chapter 5 is a survey of widely used unsteady flow routing models, such as the NWS

Flood Wave Dynamic Model (FLDWAV) and HEC-RAS by the U.S. Army Corps of Engineers. Chapter 6 presents the reservoir operation model of this research. Chapter 7 presents the mathematical background of the optimization/simulation model. This chapter is my analysis of the problem statement, mathematical formulations, and the solution procedure (i.e. the computation flowchart). Chapter 8 shows the application and results of the simple model, as shown in Figure 1.14. Chapter 9 shows the real-time reservoir operation model on the Cumberland River Basin flood event. The model included the J. Percy Priest Dam, the Cordell Hull Dam, and the Old Hickory Dam in operation, since they impacted the flood conditions in Nashville the most. A detailed discussion of the results is presented. Chapter 10 presents the concluding remarks and suggestions for future work of this research project.

CHAPTER TWO – CUMBERLAND RIVER BASIN & THE MAY 2010 FLOOD EVENT

2.1 Basin Location and Characteristics

The Ohio River starts at the confluence of the Monongahela and Allegheny Rivers in Pittsburgh, Pennsylvania. The Ohio River flows along the borders of states like Indiana, Illinois, Kentucky, West Virginia, and Ohio to its confluence with the Mississippi River at Cairo, Illinois. Figure 2.1 illustrates the Ohio River Watershed. The Ohio River the largest tributary, by volume, to the Mississippi River, and contributes 60% on average of the flow in the Mississippi River at Cairo. The Ohio River is 981 miles long and has a total drainage area of about 204,000 mi² converging parts of 15 states. The Cumberland River enters the Ohio River 58 miles upstream of its junction with the Mississippi River as illustrated on Figure 2.1 (USACE, 2010c and 2012).

The Cumberland River is the second largest tributary of the Ohio River. From that point the 694 miles long river flows southwest toward Nashville, Tennessee; then flows toward northwest into western Kentucky. The Cumberland River Basin lies entirely within the states of Kentucky and Tennessee and has a total area of 17,914 square miles, of which 10,695 square miles (60%) are in the state of Tennessee. The topography of the basin varies from rugged mountains in the eastern upstream portion to rolling low-plateaus in western, or downstream, sector. Elevations range from 4,150 feet above mean sea level (msl) in the Cumberland Mountain to 302 feet in the pool at the mouth of the river (USACE, 2010c and 2012). Figure 1.9 from the last chapter illustrates the general map of the Cumberland River Basin and surrounding areas.

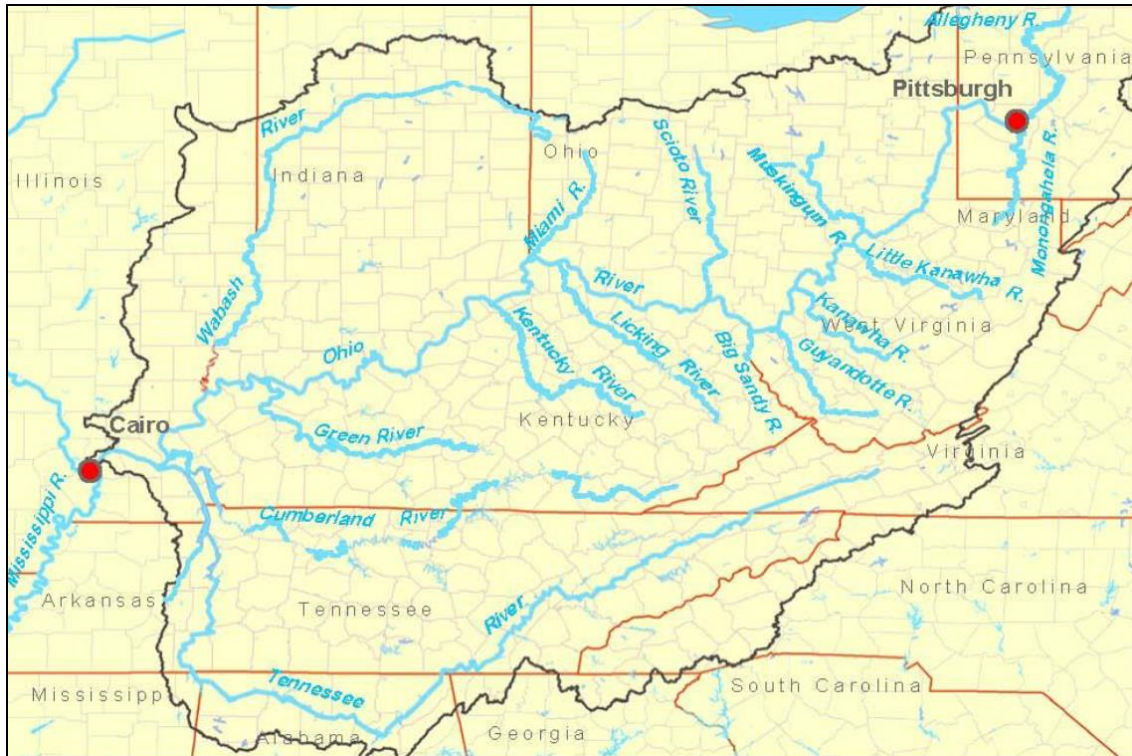


Figure 2.1: Ohio River Basin (Source: USACE, 2010c)

2.2 Existing Reservoirs in the Cumberland River Basin

The U.S. Corps of Engineers Nashville District maintains and operates five projects on the Cumberland River main stream, and five projects on its tributaries. The projects in the mainstream are Cordell Hull, Barkley, Cheatham, Old Hickory, and Wolf Creek. Congress authorizes only Barkley and Wolf Creek for flood risk management. Congress authorizes Barkley, Cheatham, Old Hickory, and Cordell Hill for the purposes of hydropower and commercial navigation. The five Corps of Engineers tributary projects, Dale Hollow, Center Hill, Martin’s Fork, Laurel, and J. Percy Priest are congressionally authorized for flood risk management (USACE, 2010c and 2012).

Figure 2.2, also Figure 1.10 from the previous chapter illustrate the existing U.S. Corps of Engineers projects in the Cumberland River Basin, and Table 2.1 shows the summary of current purposes of these congressionally authorized projects.

As illustrated in Table 2.1, the current water resources system for control of the Cumberland River and its tributaries comprises of ten dams, five on the main stem, and the other five are on the tributaries. Nearly all of them produce hydropower, with the exception of the Martin's Fork Dam. Four of the projects have navigation locks and six do not. All of the projects enhance water supply of the Cumberland River Basin, however the U.S. Congress for water supply purposes specifically authorizes none. All projects contribute to improve water quality, but the Martin's Dam is only project that is specifically authorized for water quality improvement. The entire Corps' projects in the Cumberland provide recreation, fish, and wildlife enhancement. Despite the fact that potential floods affect all dams, only six dams are authorized for flood control purpose. The storage reservoirs of Wolf Creek, Dale Hollow, Center Hill, and J. Percy projects in essence provide the controls of flood on the Cumberland River between Wolf Creek and Barkley Dams. These dams account for 71% of the flood storage volume in Cumberland River Basin. They also control runoff from 55% of the total basin drainage area and 77% of the drainage area upstream of Nashville, Tennessee. Lake Cumberland behind the Wolf Creek Dam has the greatest flood control capacity in the Cumberland River Basin. Lake Cumberland has 42% of the basin's flood storage and 58% of the capacity upstream of Nashville. It also controls runoff from 33% of the Cumberland drainage area (USACE, 2010c and 2012).

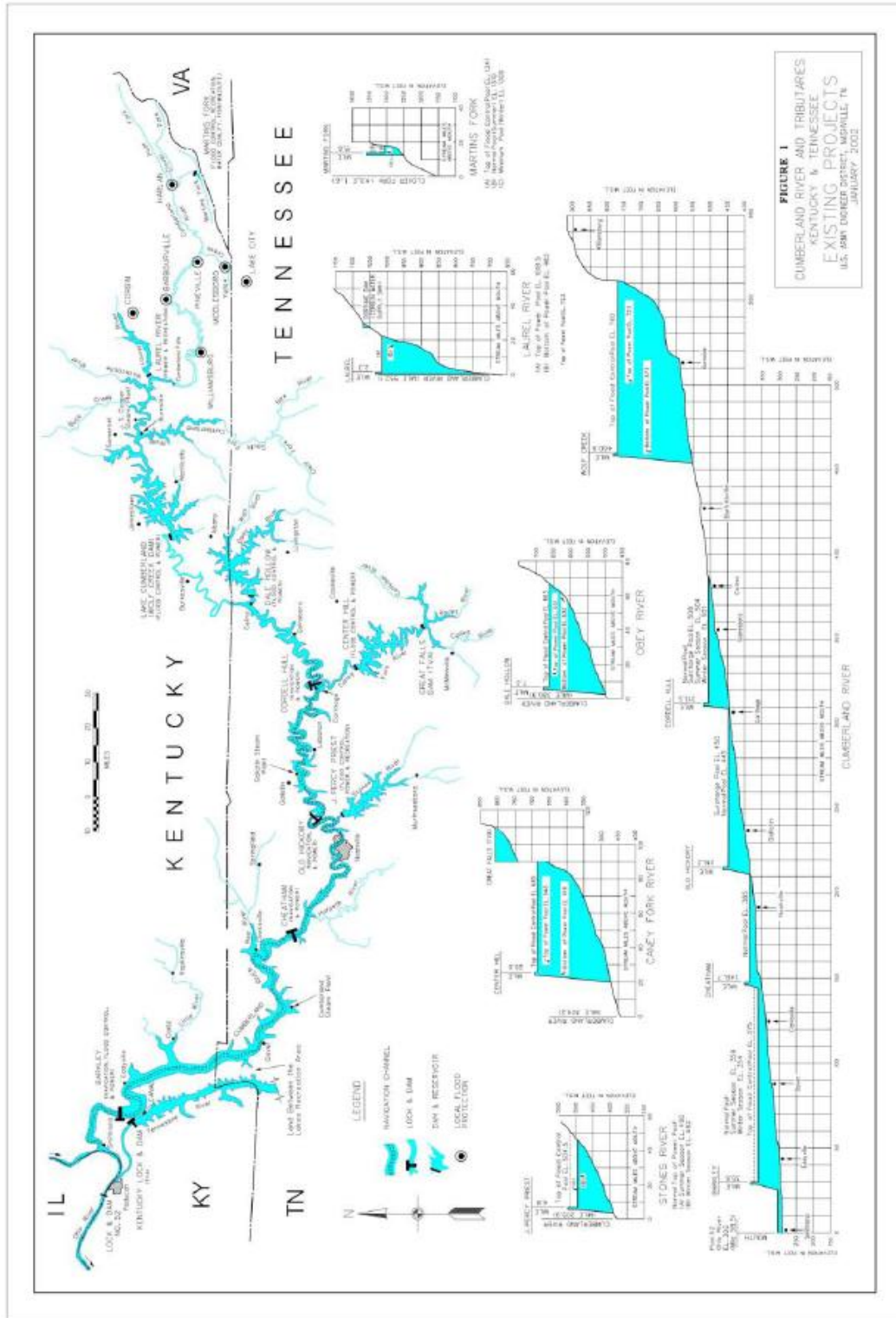


Figure 2.2: U.S. Army Corps of Engineers' Projects in the Cumberland River Basin (Source: USACE, 2010)

Table 2.1: Currently Congressionally Authorized Projects Purposes (*Source: USACE, 2010*)

Project	Flood Risk Management	Commercial Navigation	Hydropower	Recreation	Water Quality	Fish & Wildlife
Mainstem Projects						
Wolf Creek Dam	X		X	X	X	X
Cordell Hull Lock & Dam		X	X	X	X	X
Old Hickory Lock & Dam		X	X	X	X	X
Cheatham Lock & Dam		X	X	X	X	X
Barkley Lock & Dam	X	X	X	X	X	X
Tributary Projects						
Martin's Fork Dam	X			X	X	X
Laurel Dam			X	X	X	X
Dale Hollow Dam	X		X	X	X	X
Center Hill Dam	X		X	X	X	X
J. Percy Priest Dam	X		X	X	X	X

Lake Cumberland has storage allocated to retain over 6.5 inches of runoff within its flood control pool elevation, and during the early flood season in spring time, storage space is occasionally available within the power pool to store an additional 3.5 inches of runoff. The key location that the Wolf Creek Dam controls is Celina, Tennessee, located along the Cumberland River, 80 miles downstream. Celina locates about 108 miles northeast of Tennessee. Dale Hollow Lake contains about 7% of the basin flood storage capacity. Similar to the Wolf Creek Dam, Dale Hollow Dam mainly controls flood at Celina, Tennessee. Center Hill Lake contains 15% of the Cumberland River Basin's flood storage capacity. The main control point for floods by Center Hill Dam is Carthage, Tennessee. Carthage is about 55 miles east of Nashville. J. Percy Priest Reservoir contains about 7% of the basin flood storage capacity. The primary location the J. Percy Priest Dam controls is Nashville, Tennessee; the dam also controls 7% of the drainage area upstream of Nashville. Martin's Fork reservoir has flood storage of only 0.4% of the basin flood control storage, thus its effect of controlling flood is negligible. The three

mainstream projects, Cordell Hull, Old Hickory, and Cheatham, provide no flood control purpose due to their limited storage capacity. The three projects are exclusively designed for navigation and hydropower generation. The permanent impoundment of the water within the river valley decreases the natural capacity of the channel to store flood water. Thus, it is necessary to operate these reservoirs in a way to mitigate the loss of natural valley storage in the reservoir areas during floods. Barkley Dam is the most downstream project in the system. It controls runoff from 98% of the drainage for the Cumberland River Basin, and it also has 28% of the basin flood control storage. The primary areas receiving flood protection from the project are outside the Cumberland River Basin (USACE, 2010c and 2012).

The existing projects in the Cumberland River Basin provide a very high degree of flood control capability to mitigate major damage along the main stem of the Cumberland River between Wolf Creek dam and Nashville. The storage capacity of the reservoirs reserved for flood water amounts to about 7 inches of runoff from the drainage areas for each of the four major upstream reservoir projects; the system should be sufficient enough for flood protection during normal rainy seasons. During major flooding events, storage projects may reduce the outflow to zero to minimize the flow at key control points: Celina, Carthage, Nashville, and Clarksville (USACE, 2010c and 2012).

Nonetheless, uncontrolled inflows below projects may result in flows, which significantly exceed damage levels, mainly on the lower parts of the river. Reservoirs continue to store incoming upstream floodwaters during the course of a major flood event until streamflow recedes at the control locations, after which the water stored in the reservoirs is gradually released until the flood control storage has been evacuated and the pool

levels have been lowered to their normal non-flood operation levels. On the lower Cumberland River, uncontrolled tributary inflows during flood events are such that the effectiveness of reservoir control is rather less than in the upper portions of the river. For instants, early portions of a flood may exceed flood levels before upstream discharge reductions become more effective in the lower river. However, during an extreme flooding event, for example, a 500-year flood event or more, the traditional reservoir operation rules during flooding condition may not be sufficient and effective for flood control purposes. A new philosophy and approaches for flood control are therefore necessary to response potential future extreme precipitation and flooding events, which the occasion of these extreme events are evidently become more frequent and more intense primarily due to climate change.

2.3 Rainfall and Flood Event in May 2010

A catastrophic flooding event occurred across western and middle parts of Tennessee, also western central parts of Kentucky from May 1st to May 4th, 2010. Flood damage was estimated more than two billion dollars and 26 flood-related fatalities. This event was the worst flooding ever occurred in and around Greater Nashville (NWS, 2011b).

2.3.1 Antecedent Condition

In most cases, extended period of rainfall increases soil moisture and river stream flows, therefore increasing the potential for runoff. Conditions like such typically precede major or sometimes extreme large-scale flood events. Drier than normal conditions were observed in Tennessee and Kentucky from February through late April in

2010; however, showers and thunderstorms moving through the region from April 24th to April 28th 2010 did bring widespread rainfall (NWS, 2011b). Figure 2.3 to Figure 2.7 are the high-resolution precipitation image illustrating the movement of the showers and thunderstorms from April 24th to April 28th. Total rainfall received in the projects in the Cumberland River Basin, prior and after the May 2010 storm event is summarized in Table 2.2 for months of March through June in 2010. With appropriate conversion factor, runoff values in inches are calculated from monthly net effective runoff volume divided by that drainage area. The information from Table 2.2 shows the runoff from these storms did not cause flooding but did increase antecedent conditions to normal levels immediately preceding the May 2010 flood event, in other words, the total rainfall values were close to historical averages. The analysis in Table 2.2 shows the previous rain event restored the area to normal condition, the antecedent conditions were irrelevant due to the massive amount of rainfall which followed on May 1st and 2nd.

Table 2.2: Cumberland River Basin Project Drainage Basin Rainfall/Runoff Values
(Source: USACE, 2012)

Drainage Basin	<u>Rainfall (in.)</u>			<u>Runoff (in.)</u>		
	Observed	Normal	Difference	Observed	Normal	Difference
<u>Barkley L&D</u>						
March	3.46	4.96	-1.5	2.02	3.66	-1.64
April	4.94	4.27	0.67	1.23	2.57	-1.34
May	10.11	4.97	5.14	4.21	2.27	1.94
June	4.18	4.14	0.04	0.7	1.26	-0.56
<u>Cheatham L&D</u>						
March	3.87	5.3	-1.43	1.76	3.55	-1.79
April	4.23	4.19	0.04	1.23	2.67	-1.44
May	15.25	5.21	10.04	3.5	2.1	1.4
June	3.08	4.19	-1.11	0.42	1.02	-0.6

Table 2.2 Continued

Drainage Basin	<u>Rainfall</u> (in.)	<u>Runoff</u> (in.)	Drainage Basin	<u>Rainfall</u> (in.)	<u>Runoff</u> (in.)	Drainage Basin
<u>J. Percy Priest Dam</u>						
March	3.2	5.57	-2.37	2.1	3.65	-1.55
April	2.08	4.18	-2.1	0.55	2.2	-1.65
May	11.43	5.16	6.27	7.43	2.08	5.35
June	4.02	4.29	-0.27	0.21	0.98	-0.77
<u>Old Hickory L&D</u>						
March	3.11	5.35	-2.24	1.67	3.4	-1.73
April	3.42	4.1	-0.68	1.26	2.55	-1.29
May	12.86	5.17	7.69	5.14	2.01	3.13
June	3.35	4.3	-0.95	0.41	0.98	-0.57
<u>Center Hill Dam</u>						
March	3.6	5.94	-2.34	2.21	3.77	-1.56
April	2.12	4.41	-2.29	1.2	2.74	-1.54
May	8.64	5.28	3.36	3.61	2.16	1.45
June	3.74	4.45	-0.71	0.66	1.02	-0.36
<u>Cordell Hull L&D</u>						
March	3.05	5.2	-2.15	1.52	3.44	-1.92
April	3.22	4.01	-0.79	1.33	2.62	-1.29
May	11.4	5.07	6.33	4.73	2.01	2.72
June	4.64	4.43	0.21	0.39	1.07	-0.68
<u>Dale Hollow Dam</u>						
March	2.46	5.25	-2.79	1.67	3.56	-1.89
April	2.69	4.23	-1.54	1.17	2.65	-1.48
May	9.34	5.22	4.12	5.09	2.01	3.08
June	4.34	4.54	-0.2	0.38	0.9	-0.52
<u>Wolf Creek Dam</u>						
March	2.37	4.85	-2.48	1.37	3.46	-2.09
April	3.05	4.04	-0.99	1.38	2.65	-1.27
May	7.11	5.1	2.01	4.25	2	2.25
June	4.29	4.47	-0.18	0.26	1.07	-0.81

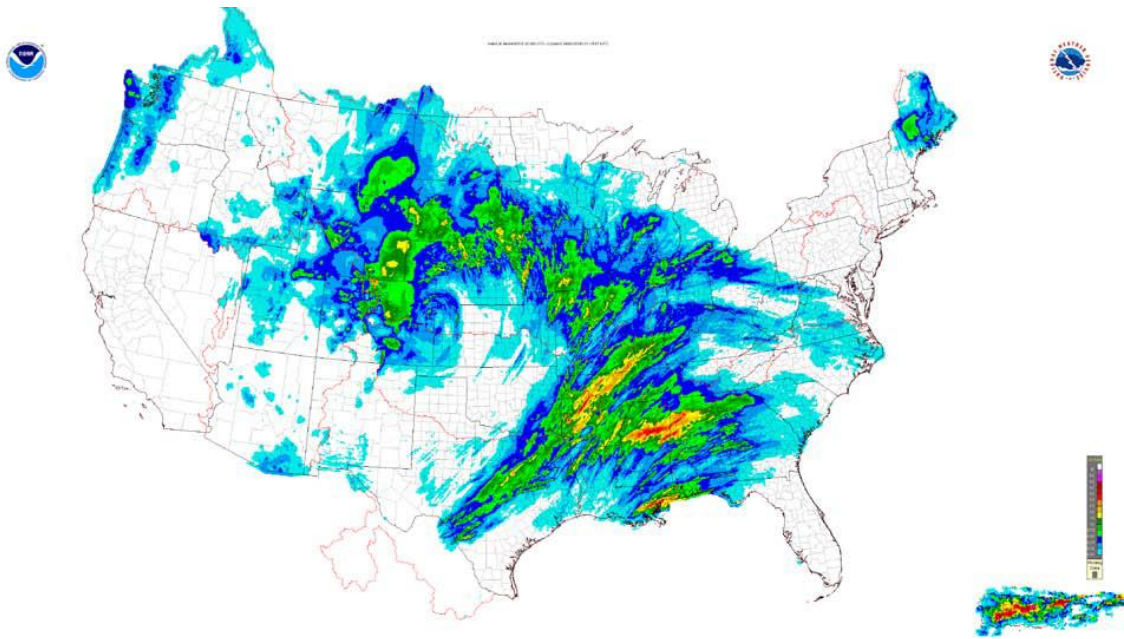


Figure 2.3: Composite High Resolution Precipitation Image at April 24 2010 12:00 UTC
(Source: USACE, 2012)

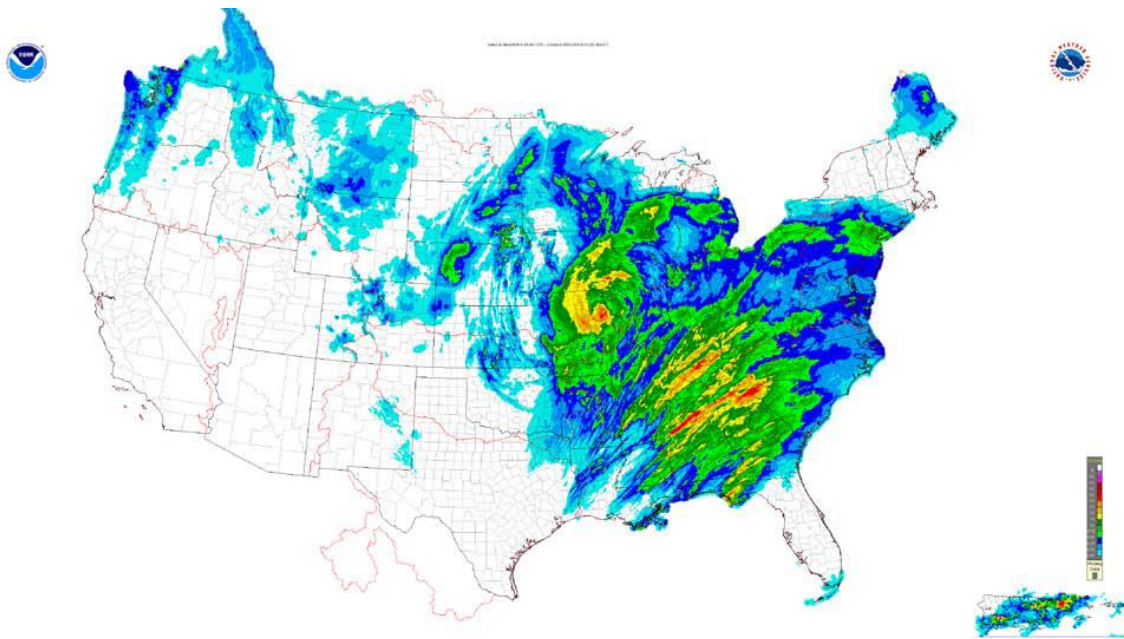


Figure 2.4: Composite High Resolution Precipitation Image at April 25 2010 12:00 UTC
(Source: USACE, 2012)

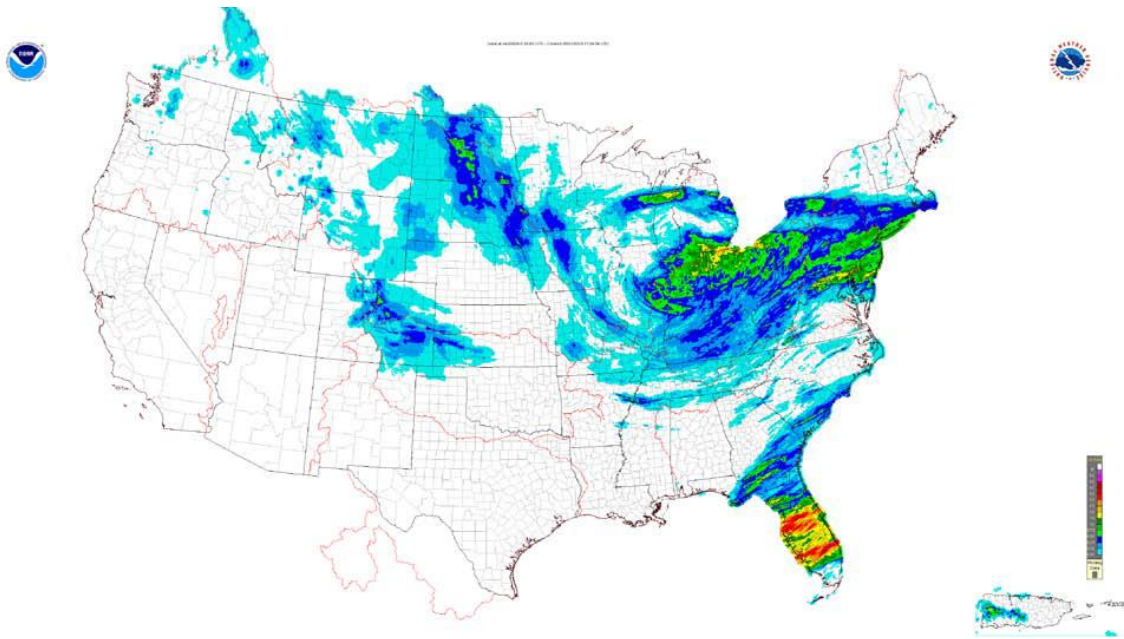


Figure 2.5: Composite High Resolution Precipitation Image at April 26 2010 12:00 UTC
 (Source: USACE, 2012)

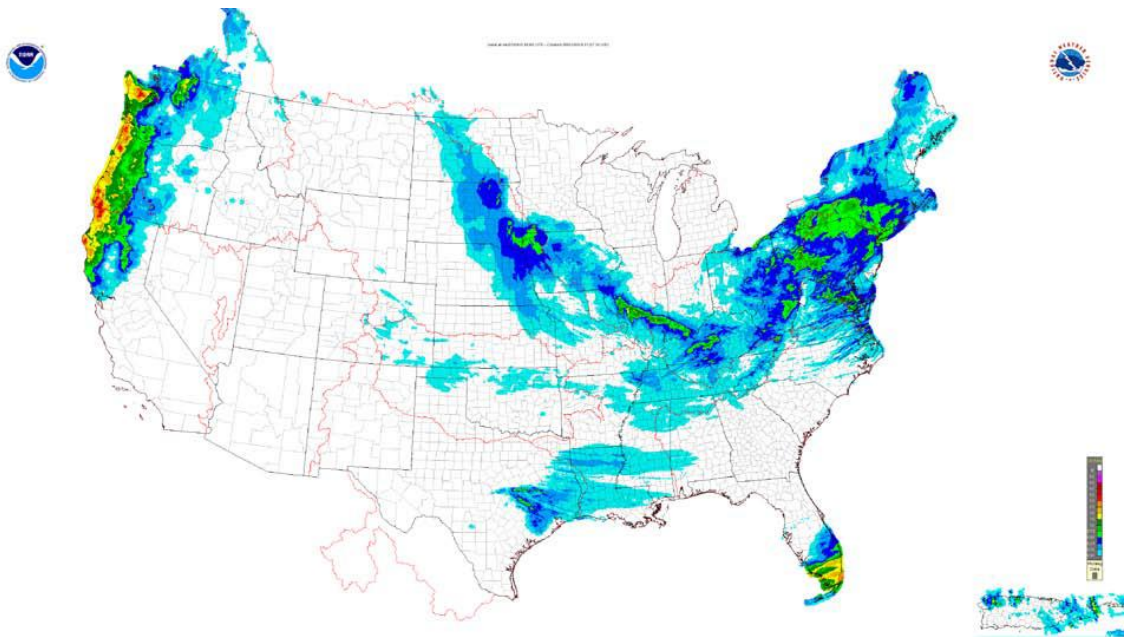


Figure 2.6: Composite High Resolution Precipitation Image at April 27 2010 12:00 UTC
 (Source: USACE, 2012)

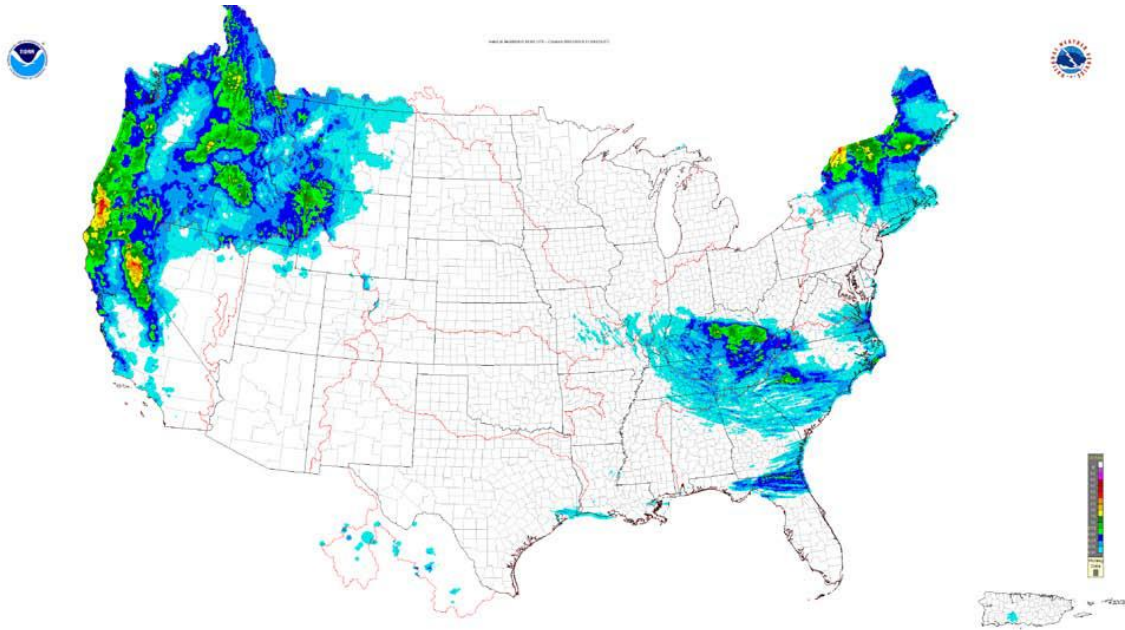


Figure 2.7: Composite High Resolution Precipitation Image at April 28 2010 12:00 UTC
 (Source: USACE, 2012)

2.3.2 Meteorological Condition

Weather disturbances in the mid-levels atmosphere contributed to trigger storms that produced heavy rainfall over the mid-Mississippi and Lower Ohio Valley region (NWS, 2011b). This rare convergence of conditions favorable for a prolonged and powerful rainfall event over the central Continental U.S. caused the May 2010 historic precipitation and flooding across Tennessee and Kentucky. Primary factors contribute to the record rainfall event are: (1) unseasonably strong late-spring storm system; (2) stationary upper-air pattern; (3) persistent tropical moisture deed; and (4) the time of the impulse moving through the jet stream. On April 30th, a very intense storm system moved into the central parts of the United States. The deep system which was unseasonably, maintain a central pressure as low as 988 millibars. The jet stream moved

from central Mexico north through the Mississippi Valley and into eastern Canada. The configuration caused an extreme favorable for upper-air condition for widespread heavy storm and severe thunderstorms over the mid-Mississippi, Tennessee and Cumberland River Basin on May 1st, 2010. A stationary front, jet stream orientation and moisture supply provided for a second round of heavy rain and intense thunderstorm activities on May 2nd, 2010. Figure 2.8 shows the weather disturbances in the mid-levels of the atmosphere helped trigger storms that produced heavy rainfall and intense thunderstorms on May 1st and 2nd (NWS, 2011b).

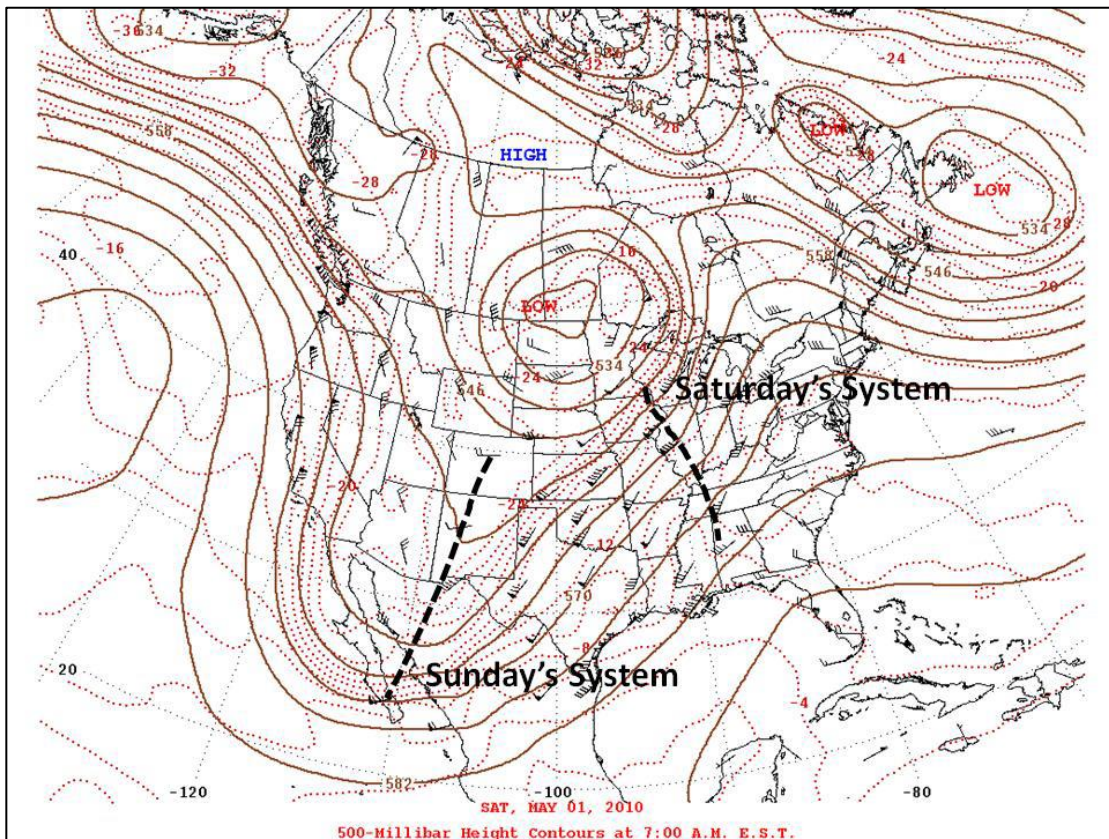


Figure 2.8: Upper Air Chart Showing Flow and Disturbances at Approx. 18000 ft. AGL, May 1st, 7:00 a.m. (Source: NWS, 2011b)

In the lower levels of the atmosphere, a 75 miles per hour jet was the main source of transporting moisture into the region, this phenomenon is illustrated in Figure 2.9. The orientation of the jet streams, positioned roughly south to northeastward, was perpendicular to the surface front, west to northeastward, stopping it from progressing eastward and allowed for an endless supply of tropical moisture across the Gulf of Mexico into the Mississippi Valley (NWS, 2011b).

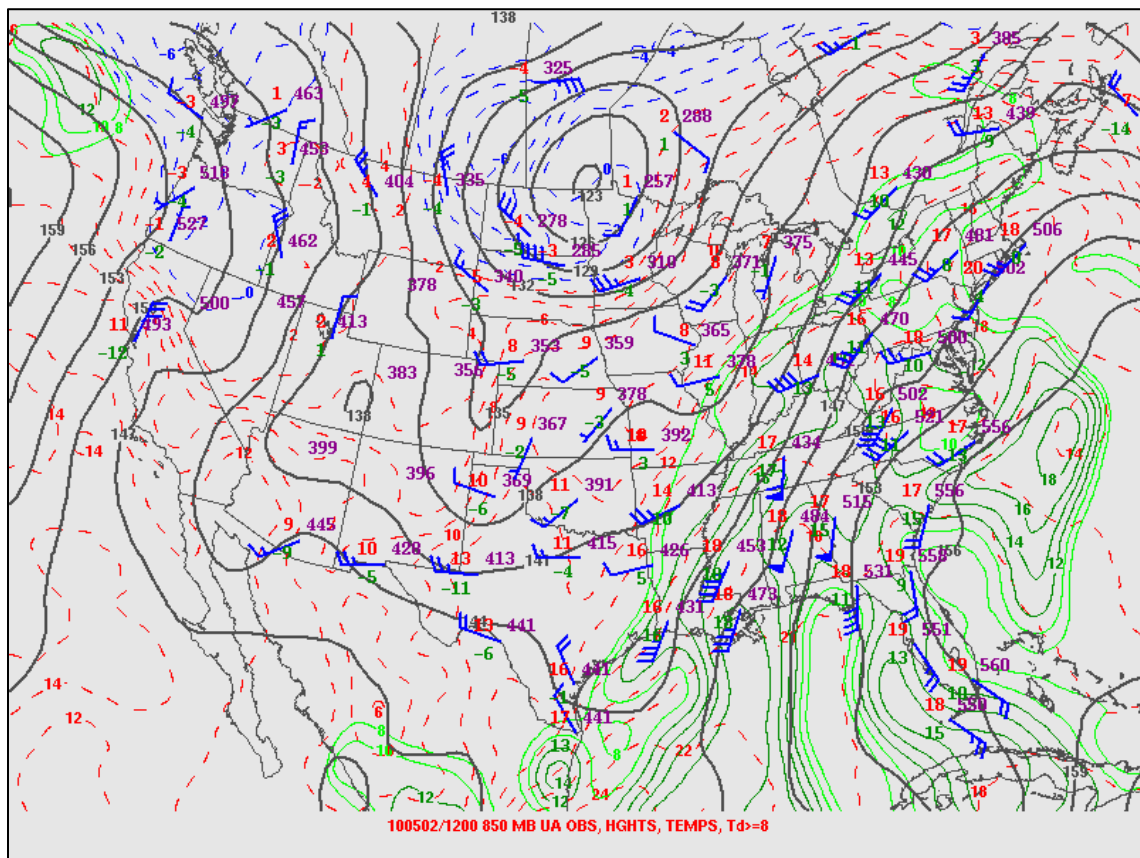


Figure 2.9: Lower Levels Atmosphere Showing Moisture Transport (green lines) at Approx. 5000 ft. AGL, May 1st, 7:00 a.m. (Source: NWS, 2011b)

These elements combined to produce two episodes of heavy intense rainfall across Kentucky, and western and Middle Tennessee. Between 10 to 20 inches of rain fell within 36 hours on May 1st and 2nd, causing catastrophic flooding events. The heaviest

rains fell primary on unregulated portions of the Cumberland River Basin, downstream of the reservoirs containing sufficient flood control storage to help contain the event's runoff and mitigated flood damages (NWS, 2011b). Figure 2.10 and 2.11 illustrate the total spatial precipitation data in the Cumberland River Basin on May 1st and May 2nd, 2010; and Figure 2.12 shows the total rainfall received over the two days (USACE 2012).

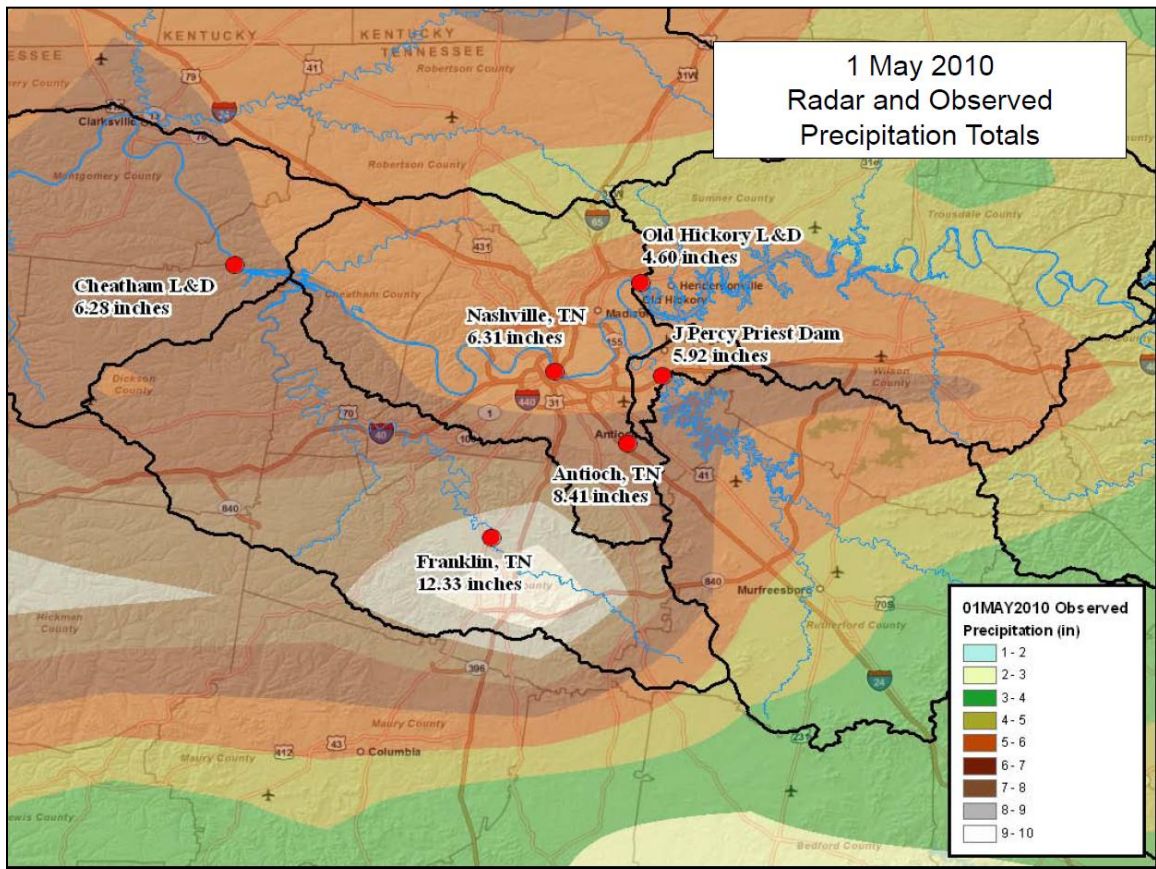


Figure 2.10: Total Precipitation Data in the Cumberland River Basin on May 1st, 2010
(Source: USACE, 2012)

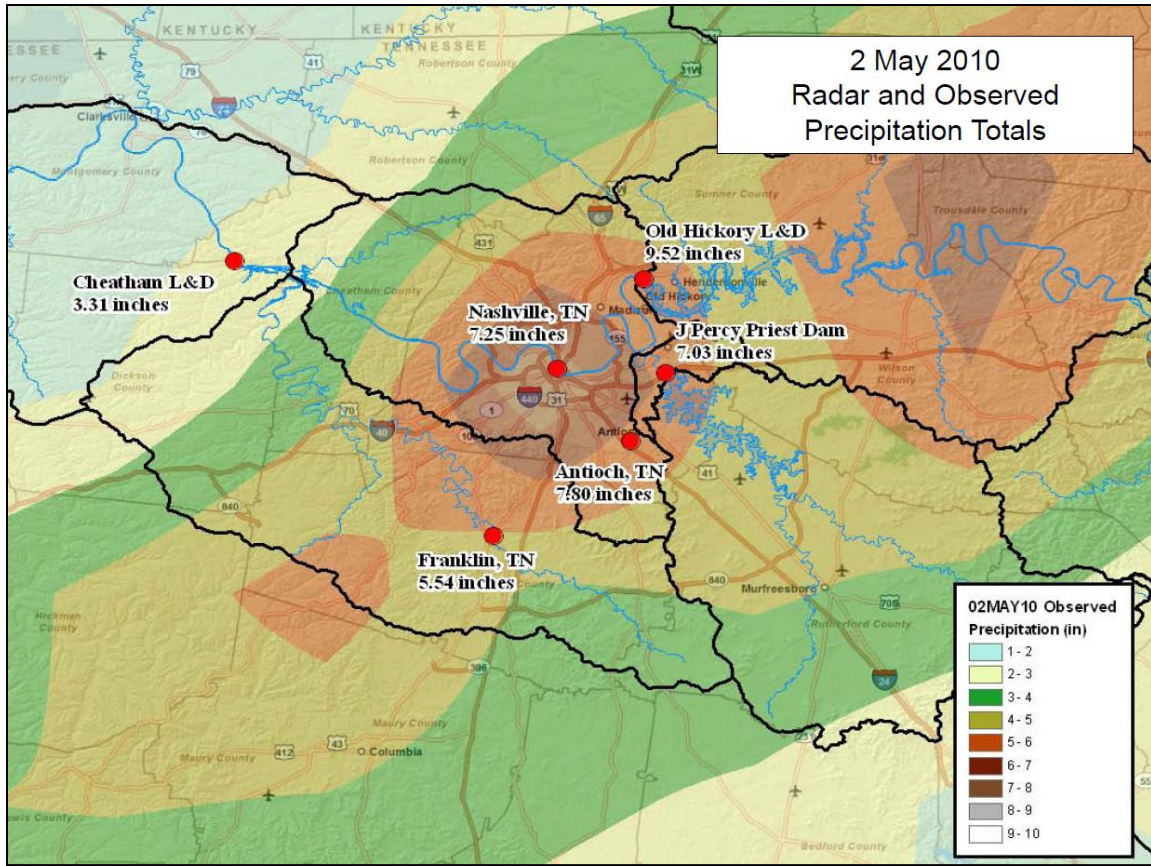


Figure 2.11: Total Precipitation Data in the Cumberland River Basin on May 2nd, 2010
(Source: USACE, 2012)

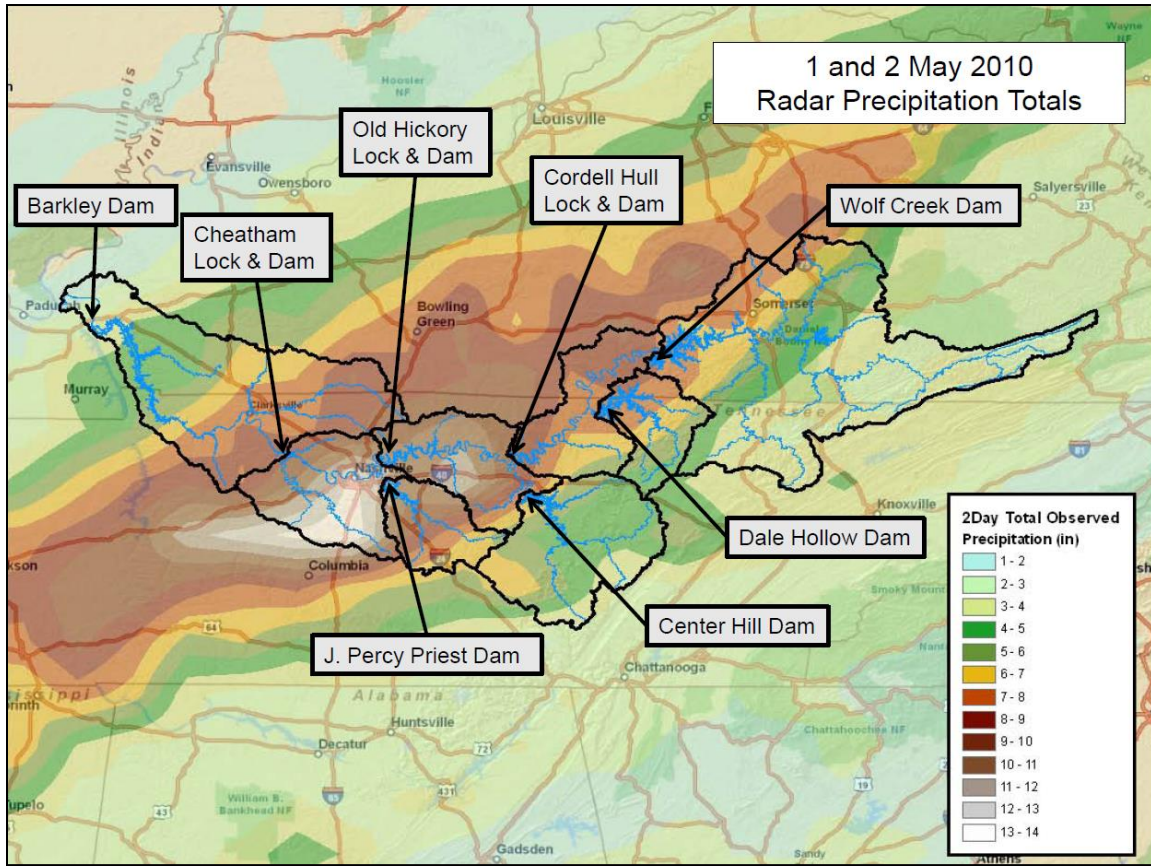


Figure 2.12: Total Precipitation Data in the Cumberland River Basin over May 1st and 2nd, 2010 (Source: USACE, 2012)

Hourly and accumulative rainfall data at the Nashville International Airport are shown in Figure 2.13. In Nashville, over 13 inches of rain was recorded during a 36-hour period; 6.23 inches on May 1st, the 3rd highest 24-hour total ever on record, and 7.25 inches on May 2nd, which exceeded the previous 24-hours rainfall record of 6.60 inches set in September, 1979 (NWS, 2011b). The highest weekend rainfall total was reported by NWS Cooperative Observer in Camden, Tennessee at 19.41 inches. Figure 2.13 also depicts the resultant river level rise (the brown curve) on the Cumberland River at Nashville, Tennessee. As seen in the figure, the flood crest at 53.86 feet was well above the major flooding stage of 45 feet. The record rain event also set water level and

discharge records on numerous tributaries and at several main stem locations across the Cumberland River Basin. Table 2.3 summarizes the significant river crests across the Cumberland River Basin.

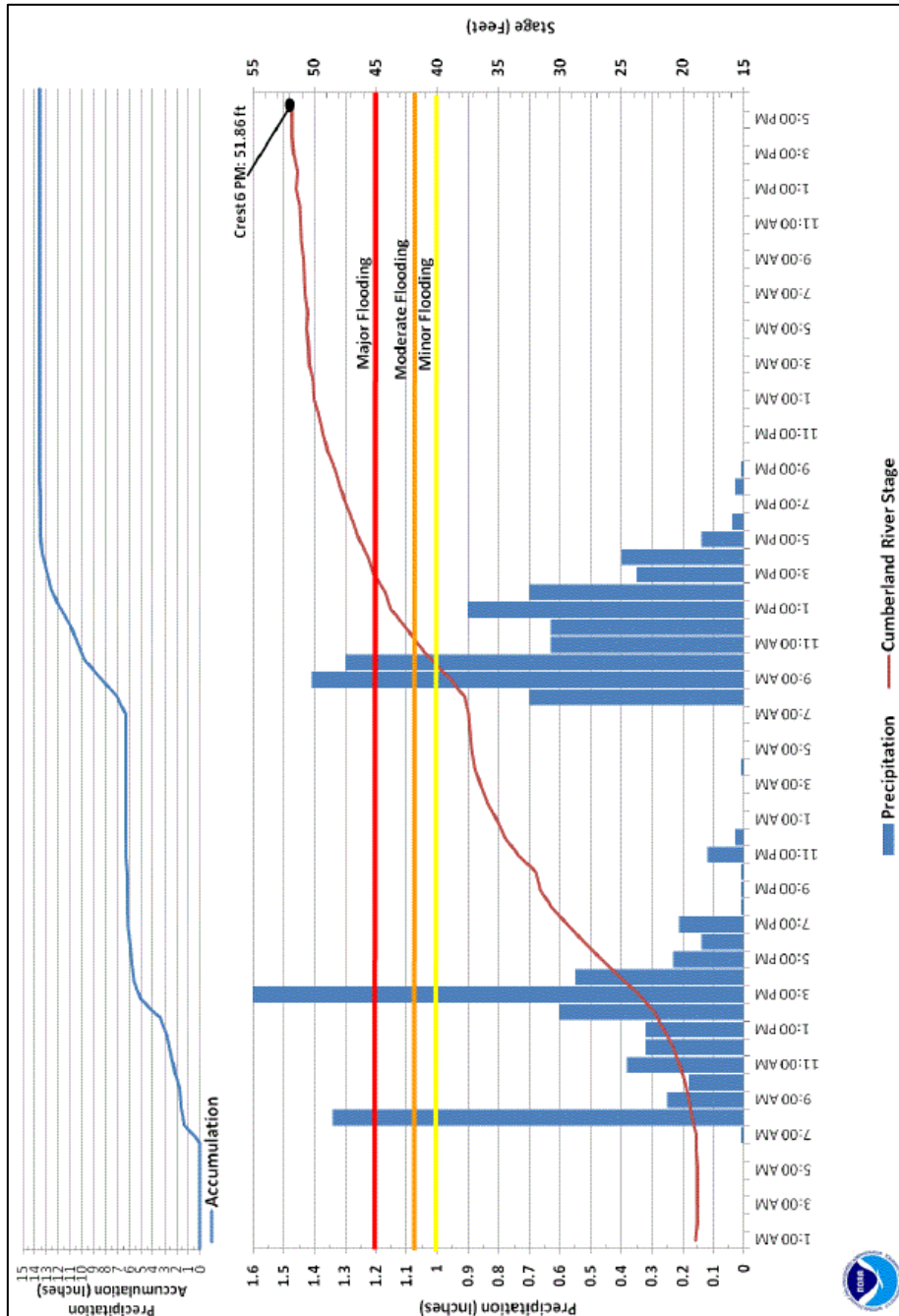


Figure 2.13: Hourly and Accumulative Rainfall at Nashville International Airport from 12:00 a.m., May 1st to 12:00 a.m., May 3rd (Source: NWS: 2011b)

Table 2.3: Record of Flood Levels Set During the May 1-2, 2010 Flood Event (*Source: USACE, 2010*)

Location	Flood Crest	Old Record	Date of Old Record	Flood Stage	Estimated Flow Frequency
Cumberland River at <i>Clarksville</i>	62.58 ft	57.1 ft	14-Mar-75	46 ft	270 year
Cumberland River at <i>Nashville</i>	51.86 ft	47.6 ft	15-Mar-75	40 ft	300 year

Table 2.4 summarizes some of the rainfall totals across the region over the 2-day record-flooding event. The gages selected are a part o a larger network administrated by the U.S. Army Corps of Engineers Nashville District and Tennessee Valley Authority.

Table2.4: Rainfall Total from May 1st to May 3rd, 2010 (*Source: USACE, 2010*)

Gage Location	Total Rainfall [in.]
<u>Cumberland River Basin</u>	
Clarksville, TN	9.22
Elkton, KY	9.4
Springfield, TN	10.38
Franklin, TN	17.87
Antioch, TN	16.22
J. Percy Priest Dam, Nashville, TN	12.96
Lascassas, TN	9.33
Murfreesboro, TN	9.76
Old Hickory Dam, Hendersonville, TN	11.88
Spring Creek near Lebanon, TN	9.51
Statesville, TN	9.58
Bethpage, TN	12.11
Cordell Hull Dam, Carthage, TN	9.15
Liberty, KY	10.587

Figure 2.14 and 2.15 illustrate the base condition of the Cumberland River level and the peak stage inundation and the severity during the May 2010 flood event.



Figure 2.14: Nashville Area during Base Condition (Source: USACE, 2012)



Figure 2.15: Nashville Area during Peak Stage Condition (Source: USACE, 2012)

During a critical period May 2nd Sunday afternoon and evening, the NWS and USACE did not communicate effectively regarding the updated reservoir releases from USACE projects (USACE 2010c, 2012 and NWS 2011b). This lack of critical exchange of information and mutual understanding of each agency’s operations led to inaccurate river stage forecasts on the Cumberland River. USACE personnel were completely involved in critical operations to prevent damage to structures or dam failures along the Cumberland River as the flooding condition worsen. With untimely and incorrect data from the USACE about their reservoir operations, as well as miscommunications and ineffective exchanges of information between the two Federal agencies, NWS crests

forecast on the Cumberland River were quickly exceeded on Sunday when the river stage at Nashville, TN, rose rapidly through moderate and major flood levels as seen on Figure 2.13 (USACE 2010c and 2012). The next section, the actions and reservoir operations of the USACE during the flood event is described in detail.

2.4 Action Taken by the U.S. Army Corps of Engineers during the Event

Typically during normal flooding events, the Corps uses water control manuals for guidance for each flood risk management project. These water control manuals provide instructions on how best to regulate levels of water at the project, therefore minimizing downstream flooding. Water control manuals are based on the dynamics of the entire watershed; these dynamics include uncontrolled tributary drainage areas downstream, reservoir storage capacity and the time distribution and volume of inflows from upstream drainage areas (USACE 1990 and 1998). Due to the magnitude of the May 2010 flooding event, the environment of which the Corps operated was far beyond the scope of the guidance instructed in the water control manuals for each project. With proper decision-making, the projects are capable of operating outside the manuals' scope; however, the water control manuals did not cover the full range of the reservoirs' capability during extreme events. During the event, the reservoir storage capacities were not fully utilized at Wolf Creek, Dale Hollow, and Center Hill Dams due to the fact the intense rainfall was concentrated in the downstream drainage areas in the Cumberland River Basin rather than upstream (USACE 2010c and 2012). Figure 2.16 reveals the radar and observed precipitation totals for the May 1st and 2nd 2010 flood event, also the locations of the rainfall in relation to controlled and uncontrolled drainage basins of

Cumberland River, respectively. As seen on the figure, the storage capacities in those projects which are purposeful for flood control such as Wolf Creek, Dale Hollow, and Center Hill Dams (see Table 2.1) were not fully used; whereas, dams that are not designed to have flood control purposes such as Cordell Hull Lock and Dam, and Old Hickory Lock and Dam were nearly overtopped by unusually extreme flood water volume during the event; which both have significantly less total storage as compare to Wolf Creek, Dale Hollow, and Center Hill Dams (USACE 2010c and 2012).

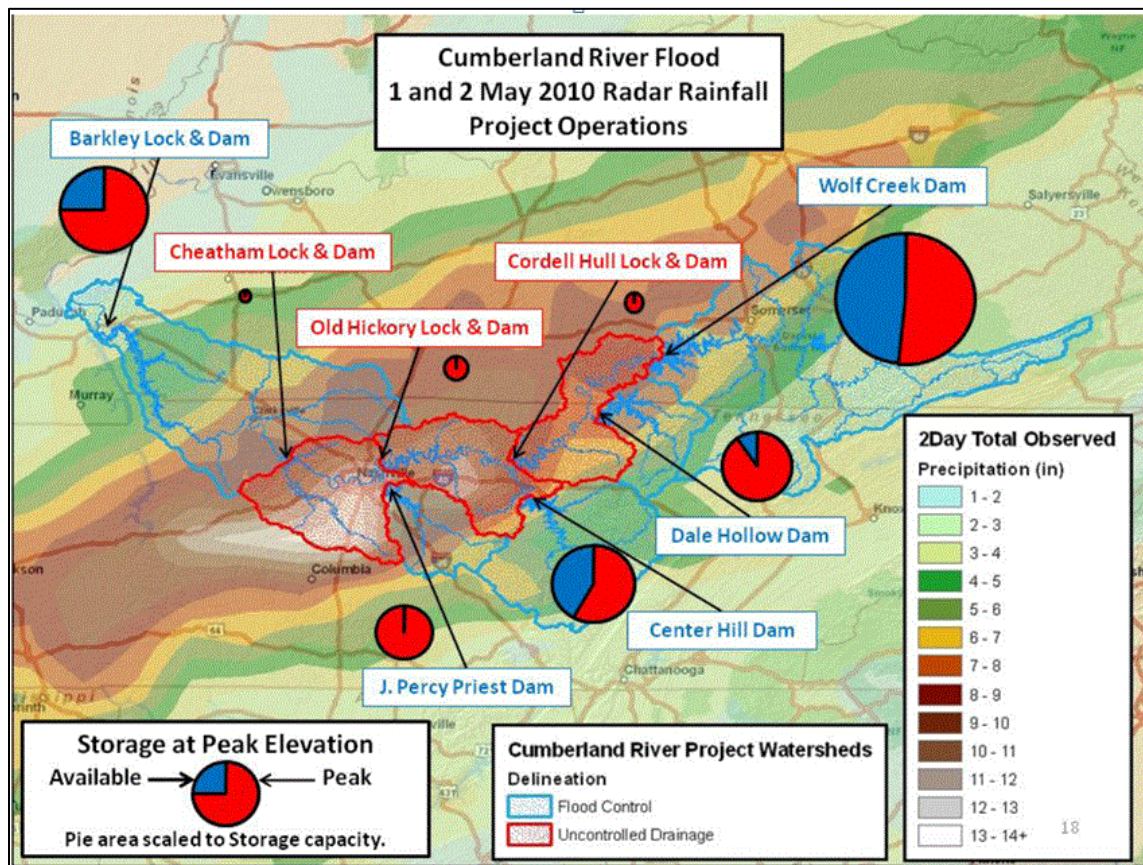


Figure 2.16: Cumberland River Basin Projects, Controlled and Uncontrolled Drainage Areas: May 1st and 2nd, 2010 (Source: UASCE, 2010c)

During the May 2010 flooding event, at J. Percy Priest Dam, located upstream of Nashville, the spillway gates were nearly overtopped (USACE 2010c and 2012). The flood storage capacity was exceeded and required operation of those spillway gates to avoid overtopping and potentially catastrophic failure of the gates. Cheatham Lock and Dam, a Cumberland River navigation project located downstream of Nashville were overtopped. Spillway-gate operations were necessary at the navigation projects of Cordell Hull and Old Hickory to prevent failure of critical structure and losing control of water leases. J. Percy Priest Dam operated in a fashion to decrease the impacts of releases from the project the flood crest moved down the Cumberland River, which resulted in the lake level exceeding the top of spillway gate elevation of 504.5 ft. Barkley Lock and Dam had a historical maximum discharge of 303,200 ft³/s. During the flood event, the project was visually inspected twice a day. Old Hickory Lock and Dam experience a tremendous water load coming within 6.6 inches from complete dam failure. A maximum historical discharge of 212,260 ft³/s along with a historical maximum headwater elevation of 451.45 feet was set during this event (USACE 2010c and 2012). If the dam were overtopped at Old Hickory, the spillway gate would have been inoperable, resulting uncontrolled flow and increased downstream damage impact. Figure 2.17 illustrates a brief summary of the operations at Old Hickory and J. Percy Priest.

During the event, the spillway gate operation at Cordell Hull changed as often as every 30 minutes; and on Monday May 3rd, 2010, it experienced a new pool elevation of 508.33 feet and a recorded discharge of 130,100 ft³/s. The recorded pool elevation at Cordell Hull was only 2 inches from overtopping the lock gate. If water had reached the

point of overtopping the dam at Cordell Hull, it would have resulted in extreme large flows downstream in the Cumberland River. Cheatham Lock and Dam experienced the most impact, which experienced a maximum historical discharge of 240,000 ft³/s along with a maximum historical headwater elevation of 404.15 feet (USACE 2010c and 2012).

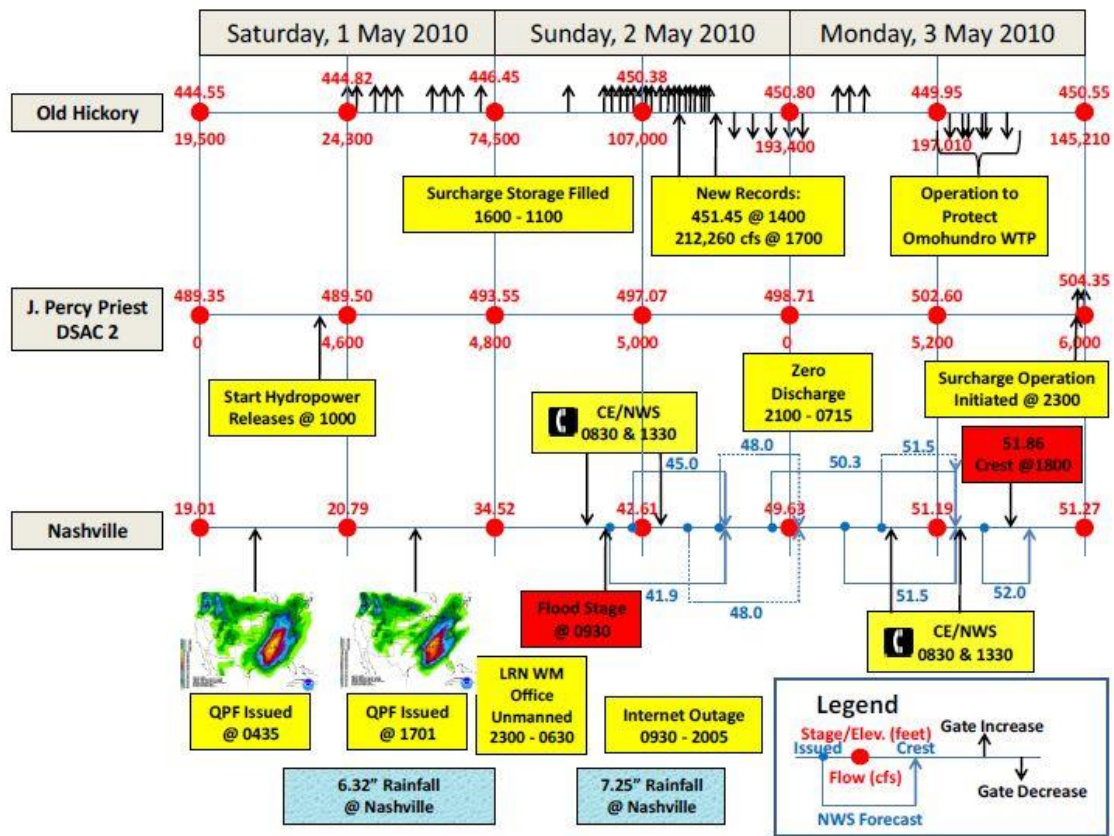


Figure 2.17: Old Hickory, J. Percy Priest, and Nashville Gage (Source: UASCE, 2010)

The Army Corps of Engineers projects in the Cumberland River Basin use traditional reservoir operation method of headwater-discharge relationship (USACE 1990 and 1998). Many of the projects, including some in the main stream of the Cumberland River, the operation policies do not extend to the full range such as when extreme events.

The ability to sustain operation of the Cumberland River Basin reservoir system under extreme rain and flooding events is highly questionable. The water control manuals of the projects were last updated in 1998 and these updates were mostly updates of the original water control manuals. The magnitude, duration, and location of the rainfall during this May 2010 event were such that flood stages along the Cumberland River were elevated to new record levels. The information in the control manuals at the time did not cover the full range of operations required to respond to this particular record rainfall event. For example, the spillway rating curve for Old Hickory did not extend to the full range of required gate openings.

As a normal operation procedure, each day the Corps of Engineers provides the NWS a morning report that includes the reservoir release data and forecast for reservoir with in the Great Lakes and Ohio River Division (LRD) (USACE 2010c and 2012). The NWS applies the information to account for the operation of the USACE projects in its hydrological forecasts. However, there were no direct communications between the USACE Nashville District (LRN) and the NWS regarding the forecast discharges on Saturday, May 1st, 2010 (USACE 2010c and 2012). On Sunday, between conference calls of the two agencies, additional releases from the projects occurred and this information was not provided to the NWS except during the scheduled conference calls. The conditions at Cordell Hull, Old Hickory, and Cheatham were so dynamic that discharge information relayed during the calls quickly became outdated. LRN had discussed conditions at the navigation projects to portray the serious nature of the flooding observed at those projects, and not with the understanding that the NWS Ohio River Forecast Center (OHRFC) was applying the discharge information in their

hydraulic models. As a result, LRN WM did not recognize the need to update that information as it rapidly changed throughout the afternoon and evening on Sunday, May 2. Once that expectation was realized, LRN Water management (WM) readily shared updated spillway release information with NWS OHRFC (USACE 2010c and 2012).

Before the May 2010 flood event, the NWS had produced 3-Day Quantitative Precipitation Forecast (QPF) as its usual practice (USACE 2010c and 2012); the USACE Nashville District had the forecast information days before the flooding event, but did not act early or nor made any operations decisions in the Cumberland River Basin. Figure 2.18 illustrated increased 3-day rainfall total up to 7 inches in central Tennessee. However, the USACE did not utilize the information NWS 3-day QPF which was available before the actual event. It is fairly clear that little if any of the decision making process concerning the operation of the reservoirs used by the U.S. Army Corps of Engineers was based upon the forecast modeling performed by the National Weather Service.

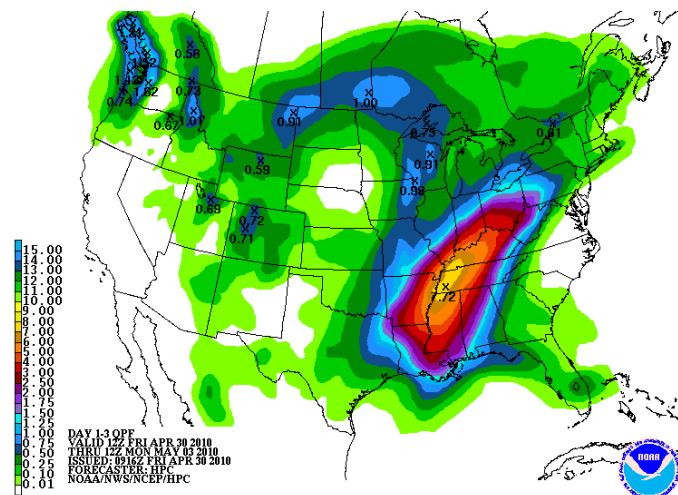


Figure 2.18: NWS QPF Published on April 30, 2010 (Source: UASCE, 2010)

2.5 Damage and Effects of the Flood Events

The May 2010 flood event established the new flood record for much of the middle Tennessee. Figure 2.19 shows the aftermath on Cumberland River near downtown Nashville.



Figure 2.19: Flooding along First Avenue on the Cumberland River near Downtown Nashville (*Source: USACE, 2012*)

The immediate concern was issues regarding the quality of municipal water supplies. It was reported that 42 water supply systems were adversely affected. Ten of these systems were completely off line with several being out of service for two weeks or more. City of Nashville lost the usage of one of the primary water treatment plants; another water treatment plant was nearly inundated, which would have affected the water supply ability to nearly 750,000 people. Numerous water line breaks also occurred due to

exposed and damaged water lines. An estimated 70 wastewater treatment facilities in Tennessee were damaged by flooding, while about 20 of them were severely damaged and required to close for few weeks. Although water and wastewater contamination was of immediate concern to public health, drift and debris that were carried by floodwater often create additional damage to the flooding areas, such as clogging the important waterways and drainage. 52-county region was affected by the flooding. The flooding within the Cumberland River Basin impacted thousands of homes and businesses. An estimated of \$2 billion dollars in property damage were experienced as a result of this flood event. Tragically, the flood of May 2010 resulted in the deaths of 26 individuals in West and Middle Tennessee and western and central Kentucky, 18 of which occurred within the USACE Nashville District boundaries (USACE, 2010c and 2012).

2.6 The Lack of Real-Time Operation Strategies in the Cumberland River Basin

The primary control location for the release from the Old Hickory Dam is Nashville, Tennessee, which is about 25 miles downstream of the dam (USACE, 1998). Flow propagate through Nashville is directly affected by the releases from the Old Hickory Dam and the J. Percy Priest Dam as illustrated in Figure 2.2. J. Percy Priest is a flood control structure so it has a greater capacity than the Old Hickory. However, the J. Percy Priest is a tributary river (the Stone River) to the main Cumberland River, the Old Hickory that is on the Cumberland River main stream, is not a flood control dam. The Old Hickory project does not have any flood control storage capability. It does, however, have a small amount of space dedicated to flood storage. The Old Hickory is permitted to have pre-flood drawdown prior to the arrival of the flood waters (USACE, 1998). The Old Hickory Dam has certain guidelines for operation during a storm event. For instance,

the gates (six in total) must be opened uniformly as the headwater rises about elevation of 447 feet as shown in Table 2.5. As flood progresses, the Old Hickory discharges are increased and Nashville flows are allowed to reach control levels before any storage is used. Once the control flows is reached, J. Percy Priest discharge are then reduced to maintain the control flow at Nashville. If the Nashville control flow cannot be maintained, then flood storage of the Old Hickory is utilized. The increase in maximum combined spillway releases from the Old Hickory and the J. Percy Priest is limited to 5000 ft³/s per hour. The maximum combined decrease in spillway discharges from Old Hickory and J. Percy Priest is limited to 10,000 ft³/s per hour (USACE, 1998).

Prior to the May 2010 storm event, the projects in the Cumberland River Basin managed by the U.S. Army Corps of Engineers used traditional method, the *headwater-discharge relationship*, for their reservoir operations. The decisions of releases are based on the pool elevation of control points at the time. As of May 2010, the flood regulation at the Old Hickory Dam was based on the decades old USACE Water Control Manuel (USACE, 2010c). The managers at the dam were to follow the Flood Regulation instruction during flooding condition. According to the USACE Water Control Manuel (USACE, 1998), the flood operations of the run-of-river Old Hickory Dam on the Cumberland River, 25 miles upstream of Nashville, are based on the peak stage and rate of rise at the control location Nashville. The reservoir operators then use the rating table to determine the spillway gate openings at the Old Hickory Dam as illustrated in Table 2.5.

Table 2.5: Spillway Releases for various Headwater Levels (*Source: USACE, 1998*)

Headwater Elevation (feet)	Minimum Gate Opening (feet)	Minimum Spillway Discharge (cfs)
445	0	0
446	0	0
447	0	0
448	1	7500
449	2	14880
450	3	22440

During the May 2010 Flood event, USACE personals were sent to the reservoirs and flood sites to observed flood stages (USACE, 2010c), and reservoir decisions were made based on observations at the time, but not based on pre-flood forecasting. As illustrated in Figure 2.17, during the midday on May 2nd, the pool elevation of the Old Hickory Dam reached above 450 feet, and nearly a foot over by the end of the day, which was above its maximum flood surcharge storage pool of 450 feet. Although the Old Hickory Dam does not primarily provide flood control service, with adequate real-time operation strategies of the entire river-reservoir system, the pool elevation of the Old Hickory Dam should have been below the maximum flood surcharge storage pool of 450 feet. Had the real-time optimal operation of river-reservoir system, as described briefly in Chapter 1 and in detailed in later chapters, been adopted, the flood damage during the May 2010 might have been minimized. By employing the real-time optimal operation of river-reservoir system, the operating decisions are made for the entire reservoir systems simultaneously based on rainfall-runoff forecasting, operational hydrologic and hydraulic model simulations, and optimization model. The entire reservoir system operation decisions could have been made hours, or even days before the real storm arrive.

2.7 The Impacts of Old Hickory Dam

The USACE operation at the Old Hickory Dam can be further analyzed. The Old Hickory Dam is the dam immediately upstream of Nashville. Figure 2.20 shows the gate opening over the five-day span from the start of May 1st to the end of May 5th. Figure 2.21 shows the reservoir discharge over the same five-day span.

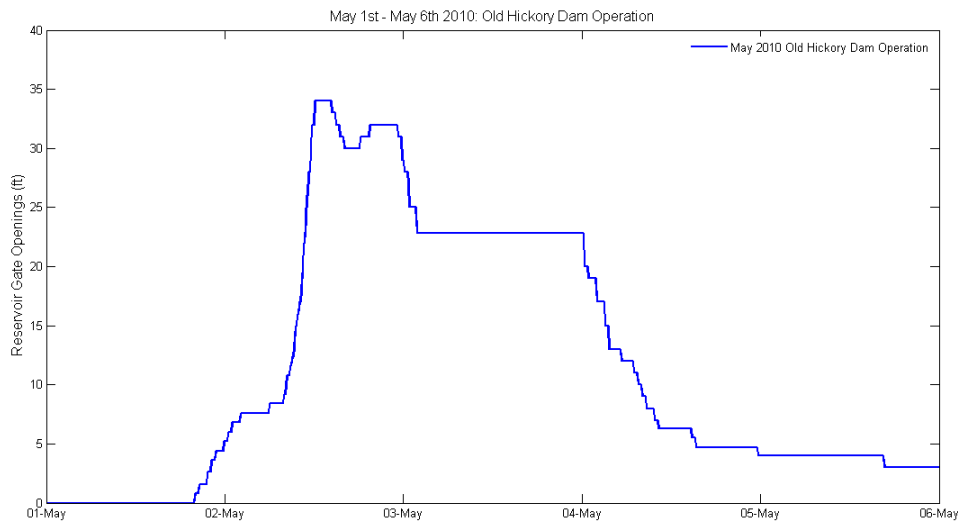


Figure 2.20: The Gate Openings at the Old Hickory Dam during the May 2010 Storm Event (USACE, 2010c)

As seen in the figure, the USACE did not start operating the dam well after the storm has started (see Figure 1.11 and Figure 2.13). It was not until later on May 1st the USACE started to release water from the reservoir gates. The late response at the Old Hickory to the storm was one of the main reasons why Nashville was flooded. The USACE needed to release quickly thus the gates were open rapidly in May 2nd, causing huge flow coming out of the dam as seen in Figure 2.21. Figure 2.22 shows the flow comparison of the Old Hickory Dam outflow and downtown Nashville. There was a strong correlation between the two flow time series, and it was evident that the huge flow

from the Old Hickory Dam was the cause of the flooding at Nashville. The 100-year flood stage at Nashville is 48 feet; the flood stage at Nashville was greater than the 100-year flow for the majority of the time span between May 2nd and May 4th as seen in Figure 2.23.

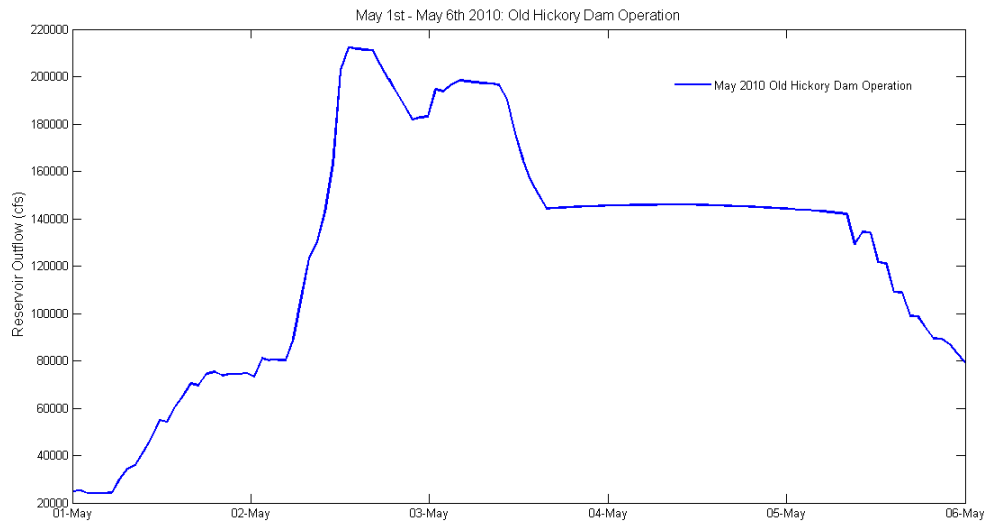


Figure 2.21: Reservoir Outflow at the Old Hickory Dam during the May 2010 Storm Event (USACE, 2010c)

In fact, not only the Old Hickory Dam operation during the May 2010 storm event was flawed, the existence of the dam was also problematic under flooding conditions. According to the study and simulation conducted by civil engineering professor Dr. Larry W. Mays at Arizona State University as seen in Figure 2.23, with the dam in place and the operation used by the USACE caused a 2.2 ft. increase in maximum water surface elevation for the May 2010 storm event.

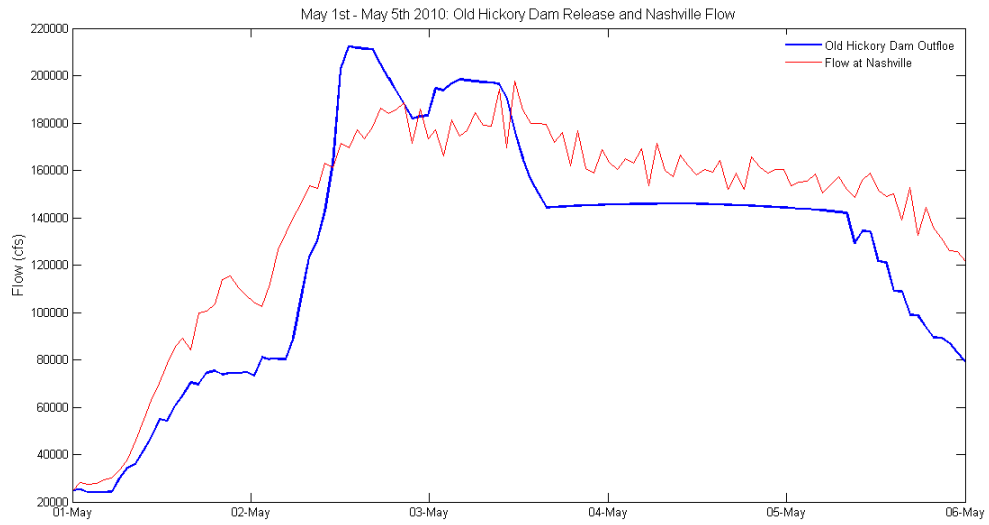


Figure 2.22: Reservoir Outflow at the Old Hickory Dam and Flow at Nashville during the May 2010 Storm Event

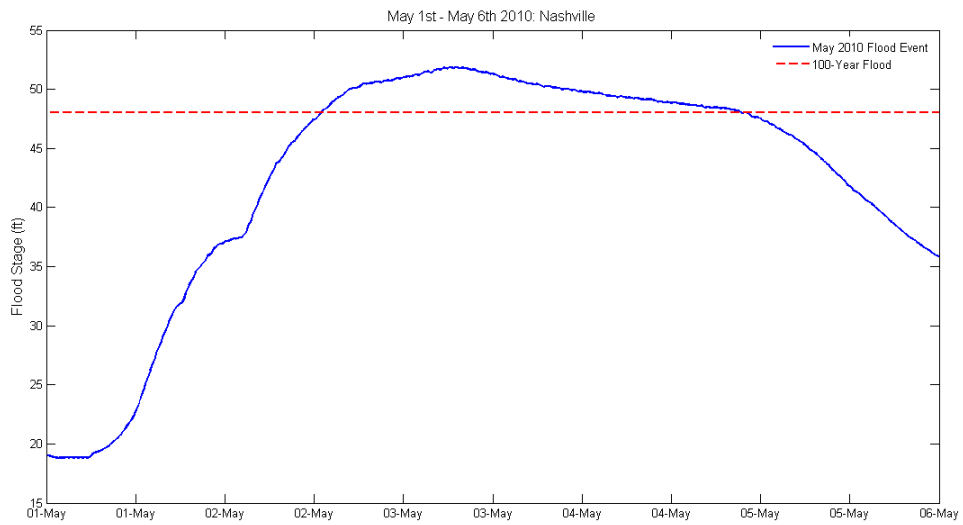


Figure 2.23: Flood Stage Condition at Nashville during the May 2010 Storm Event

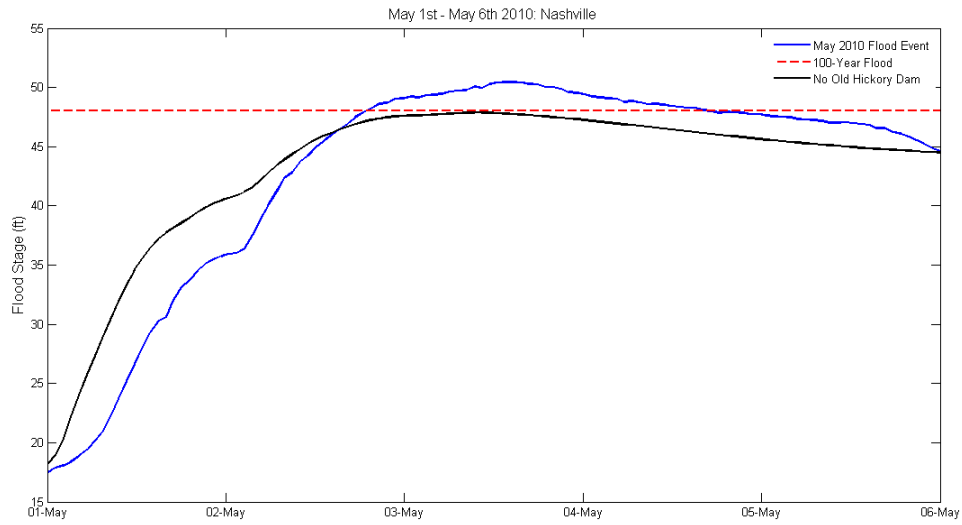


Figure 2.24: Flow Comparison (with and without Old Hickory Dam) at Nashville during the May 2010 Storm Event

CHAPTER THREE – STATE OF THE ART OF REALTIME FORECASTING

3.1 National Weather Service

3.1.1 Weather Prediction Center (WPC)

The Weather Prediction Center (WPC) is one of the nine centers under National Centers for Environmental Prediction (NCEP), which is part of the National Weather Service (NWS) (WPC, 2014). The WPC serves as a center for quantitative precipitation (QPF), medium range forecasting, typically three to eight days, and the interpretation of numerical weather prediction models. The QPF depicts the amount of liquid precipitation expected to fall in a given period of time. The WPC issues storm information on storm systems bringing significant rainfall to portions of the United States. The WPC also forecasts precipitation amount for the Contiguous United States (CONUS) for systems expected to make impact over the next seven days. The WPC-QPF prepares and issues forecasts of quantitative of precipitation accumulation, heavy rain, heavy snow, and highlights areas with the possible for flash flooding, with forecasts effective over the following five days (WPC, 2014). These data are sent to the NWS Weather Forecast Offices (WFOs) and are available on the web for the general public. One station of the National Environmental Satellite Data and Information Service (NESDIS) is co-located with the WPC-QPF station, which together form the National Precipitation Prediction Unit (NPPU). NESDIS meteorologists prepare rainfall estimation and the current trends based on satellite data, and this information is used by the Day 1 QPF forecasters to help create individual 6-hourly forecasts that cover the next 12 hours. With access to radar data, satellite estimates, and NCEP model forecast data as well as current weather observations and WPC evaluations, the forecasters have the latest

data for use in real-time operational forecasting models preparation of short-range precipitation forecasts. To produce QPFs, the WPC meteorologists analyze the current condition of the atmosphere. Then they use numerical model to forecast pressure systems, fronts, jet stream intensity, etc., to form a conceptual model of how the storm (or weather) will evolve. The WPC forecasters would make consecutive runs of the forecasting model to obtain the trend analysis of the model QPFs (WPC, 2014). Figure 3.1 illustrates an example of a Day 1 QPF on May 31st, 2013.

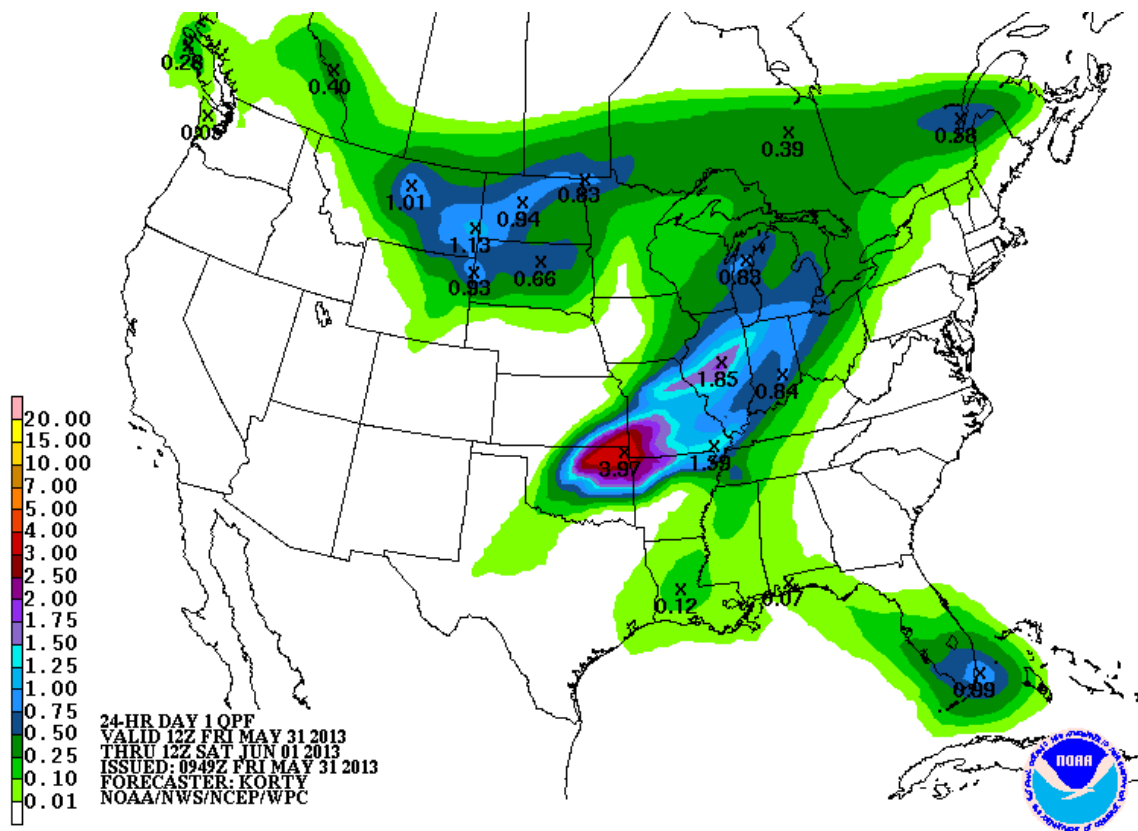


Figure 3.1: Example of a Quantitative Precipitation Forecast (WPC, 2014)

3.1.2 Advanced Hydrologic Prediction Service (AHPS)

The Advanced Hydrologic Prediction Service (AHPS), under the National Weather Service, is a web-based suite of accurate and data-rich forecast information (NWS, 2002). The AHPS produces the magnitude and uncertainty of occurrence of floods or droughts, from hours to days and months, in advance. The AHPS uses sophisticated computer models and large amount of data from a variety of sources such as super computers, automated gauges, geostationary satellites, Doppler radars, weather observation stations, and the computer and communications system, called the Advances Weather Interactive Processing System (AWIPS). The NWS provides hydrologic forecasts for almost 4,000 locations across the CONUS (NWS, 2002).

The current group of AHPS products covers forecasting periods from hours to months. It also includes information about the chances of flood or drought. The information, such as the flood forecast level to which a river will rise and when it is likely to reach its peak or crest, is shown through hydrographs. Other information includes but not limited to (NWS, 2002):

- the probability of a river exceeding minor, moderate, or major flooding,
- the probability of a river exceeding certain level, volume, and flow of water at specific points on the river during 90 day periods, and
- a map of areas surrounding the forecast point that provides information about major roads, railways, landmarks, etc. likely to be flooded, the levels of past floods, etc.

3.1.3 River Forecasting System (NWSRFS)

The National Weather Service River Forecasting System (NWSRFS) comprises programs and techniques for developing river forecasts (NWS, 2005). The NWSRFS is not a single model but rather a framework containing hydrologic/hydraulic algorithms to model a basin for river, flash flood and water resources forecasting. The NWSRFS contains three major systems which are utilized to set up and use hydrologic and hydraulic models in river forecasting. The three components include (NWS, 2005): (1) the Calibration System, (2) the Operational Forecast System (OFS), and (3) the Ensemble Streamflow Prediction System (ESP). Each system is interrelated and can be used with different models to produce a river forecast. Figure 3.2 shows the major components of the NWSRFS.

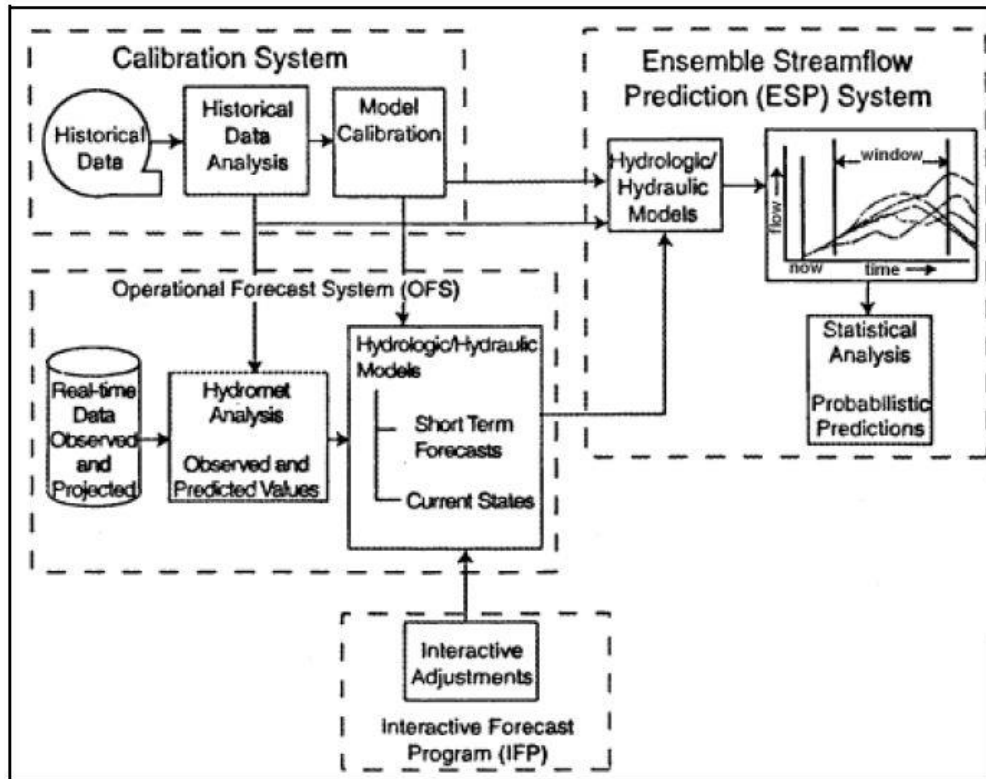


Figure 3.2: Major Components of the NWS River Forecast System (NWS, 2005)

The components of the NWSRFS have the following primary functions:

Operation Forecast System

- generate short-term river and flood forecasts using calibrated model parameters
- maintain model state variables

Calibration System

- use historical data to generate time series
- determine model parameters

Ensemble Streamflow Prediction System

- generate probabilistic forecasts extending weeks or months into the future using current model states, calibrated model parameters, and historical time series.

Hydrologic operations in the NWSRFS are organized into Table 3.1 to specify the physics of water movement for any sub-basin (NWS, 2005):

Table 3.1: Hydrologic Operations in the NWS River Forecast System (NWS, 2005)

<u>Types of Operations</u>	<u>Methods</u>
Snowmelt models	HYDRO-17 Snow Model
Rainfall-Runoff models	Sacramento Soil Moisture Accounting NWS RFC Antecedent Precipitation Index Model Xinjiang Soil Moisture Accounting
Temporal runoff	Unit Hydrograph
Channel losses and gains	Simplified Loss/Gain Method, Consumptive Use
Routing models	Lag and K; Muskingum; Layered Coefficient; Tatum Dynamic wave routing models (DWOPER/FLDWAV)
Baseflow simulation	base flow simulation model
Reservoir regulation	Single, independently controlled reservoir under various modes of operation Multiple reservoirs operated jointly
Adjustment procedures	Simplified flow adjustment and blend
Stage/discharge conversion	Single valued rating curve with log or hydraulic extensions and loop ratings
Time Series Computation	Computation of mean discharge; Weight time series

The National Weather Service River Forecast Centers (RFCs) uses the NWSRFS to make short-term forecasts (one day to a week in advance) in river flows and floods and long-term probabilistic river outlook (one week to months in advance) in support of water supply management and flood mitigation. The RFCs use the NWSRFS to generate the followings (NWS, 2005):

- flood forecast
- general river forecasts used for navigation, recreation and other purposes
- reservoir inflow forecast
- snowmelt flood forecast
- flash flood guidance

The NWSRFS has been in operation for over thirty years and is continuously refined and improved (NWS, 2005).

3.1.4 Community Hydrologic Prediction System (CHPS)

In the past thirty years, NWS hydrologists have used the NWSRFS as the essential infrastructure for their hydrologic operations. NWSRFS is remarkable that it has met most of the NWS needs for a long time. With increasing operational needs and rising support costs, the NWSRFS will be retired and replaced by the Community Hydrologic Prediction System (CHPS). CHPS has been developed by the NWS in collaboration with Deltares (formerly known as Delft Hydraulics) in the Netherlands. The Delft-Flood Early Warning System (FEWS) serves as the infrastructure for CHPS with NWS hydrologic models and United States Army Corps of Engineers (USACE) hydraulic models providing the forecasting core. Figure 3.3 illustrates the core idea of the relationship between CHPS and FEWS (NWS, 2010):

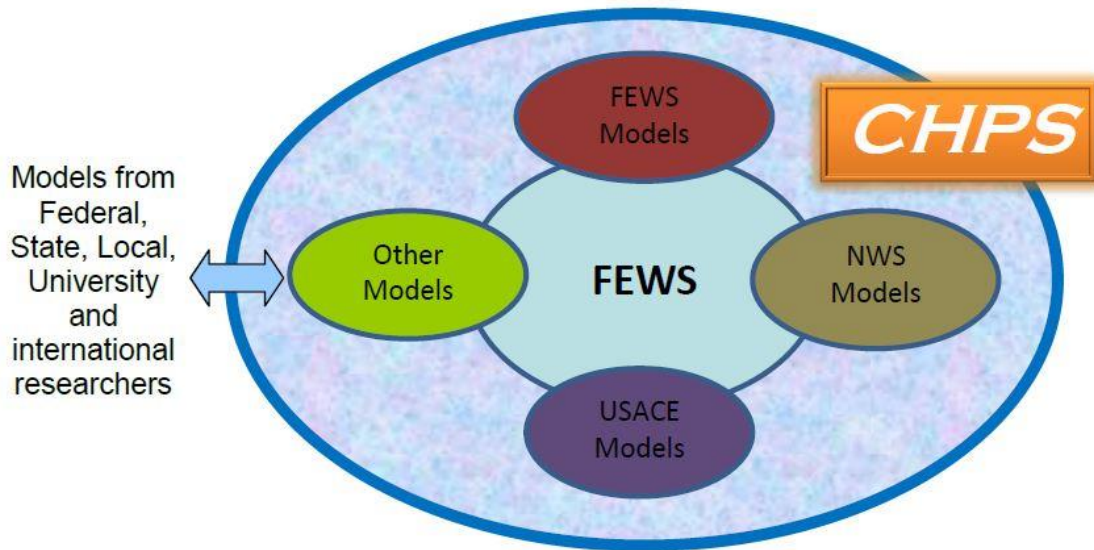


Figure 3.3: Relationship between CHPS and FEWS (NWS, 2010)

CHPS is both a system and a concept. The community concept of CHIP indicates a desire on the part of National Oceanic and Atmospheric Administration (NOAA) to reach out to broader hydrologic community. CHPS is also an open forecasting system designed to be modular in nature, and built upon standard software packages, modern protocols, and open data modeling standards. CHPS uses the FEWS as the core of its infrastructure combined with NWS and U.S. Army Corps of Engineers hydrologic and hydraulic models. FEWS provides data import, storage, display, and some basic hydrologic calculations. The current CHPS includes the same models that are currently used in NWSRFS, with the exception of the hydraulic routing models. The NWS models includes: the Anderson Snow model (the Snow 17 model); the Sacramento Soil Moisture and Continuous Antecedent Precipitation Index Runoff Model; a Unit Hydrograph model; Lag and K, Tatum, Layered Coefficient, and Muskingum routings; and NWS developed glacial melt model; and NWS Rain/Snow Elevation Model; and NWS channel

baseflow and losses models. The NWS DWOPER and FLDWAV unsteady flow routing models will not be ported in CHPS. The U.S. Army Corps of Engineers HEC-RAS will be used for the unsteady hydraulic routing by the NWSRFCs in their operational forecasting environment for the first time (NWS, 2010).

3.2 Lower Colorado River Authority

Since the late 1980s, the Highland Lake System under the Lower Colorado River Authority (LCRA) has adopted a mathematical model, developed by the University of Texas at Austin for the reservoirs and dams management (Mays, 1991). The model uses current and anticipated river discharge, rainfall data, and reservoir characteristics to simulate and demonstrate the potential for flooding in specific communities under various scenarios of reservoir operation in real time and through graphic displays. The real-time flood management model consists of two components: 1) a real-time flood control module, and 2) a data-management module. Figure 3.4 illustrated the basic structure of the real-time flood management model.

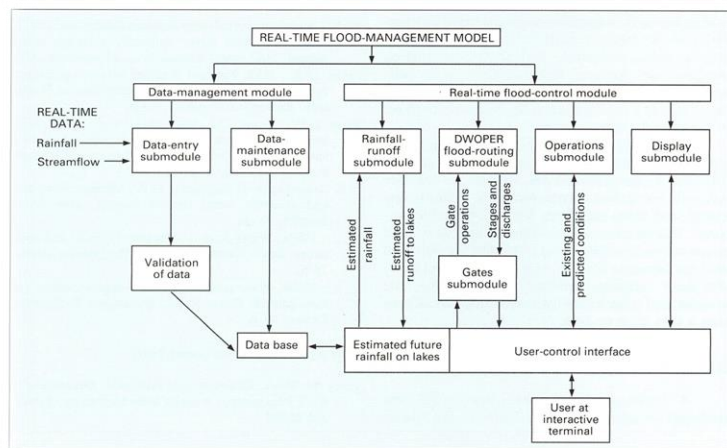


Figure 3.4: Structure of the LCRA Highland Lake System Real-Time flood Management Model (Mays, 1991)

The real-time flood control module contains the following submodules:

1. Rainfall-runoff submodule – rainfall-runoff model developed by the University of Texas at Austin for ungauged drainage area;
2. Unsteady flow routing submodule – NWS Dynamic Wave Operational Model
3. Gate and Operation submodule – a computer program developed by the University of Texas at Austin to determine gate-operation information for the unsteady flow model;
4. Display submodule – graphical display software developed by the University of Texas at Austin

The data-management submodule was developed by the LCRA for maintaining and validating data. The data-management module consists of two types of data: 1) real-time data, which is dynamic, and 2) stored data, which are stored in database and are fixed. Real-time data are rainfall collected at gaging stations, streamflow collected at automated stations, headwater and tailwater elevations at each dam, information on which rivers and reservoirs are to be simulated in flood routing, and current reservoir operations. Stored data are drainage-area information, hydrologic-parameter estimates for the rainfall-runoff submodule, unsteady flow model data that describe the physical system and include river cross-section information on roughness and other characteristics, and characteristics of reservoir spillway structures (Mays, 1991).

The development of this model represents a logical step in the evolution of flood-forecasting and flood management models that can be used in a real-time mode for multiple reservoir operation. The combination of the rainfall-runoff models and the

hydraulic-routing models in the Highland Lake System has been a step forward in real-time operational forecasting models development. The integration of these models for real-time flood management using real-time data along with simulated future rainfall, river-stage, and operational controls is a further step in the evolution of real-time operational forecasting models for large river-reservoir systems (Mays, 1991).

3.3 Flood Forecasting and Warning Service in Italy

In Italy, the Civil Protection Authority, created under the National Law 225/92 (Todini, et.al., 2005), is responsible for forecasting and mitigating risks and acts together with the central and local governments and the principle forces. The regional Civil Authorities in charge of managing flood emergencies, while a number of “Functional Centers” were created for issuing real time flood forecasting and warnings to the Civil Protection Authorities. Thus, the implementation of the law varies from one region from another. The following subsection presents an example of a river system in Italy that is under the administration of a regional Civil Authority.

3.3.1 The Upper Po River flood Forecasting System

In the Upper Po river basin, the Civil Protection Authority developed flood emergency plans in stages: Survey, Warning, Alarm, and Emergency. Emergency services are initiated by flood forecast, and then the flood control policies are carried out based on observing the evolution of the flood event. Risk is categorized by three levels: 1) normal situation, 2) low danger, and 3) high danger. The plans are carried out in the SSRN (Room for the Situation of Natural Risks), as the operational center dedicated to

managing the task. The SSRN is a 24-hour operation for survey and warning. The technical activities of the SSRN include:

- Hydro-meteorological survey by running computer systems and collecting and collating data from the survey network;
- Hydro-meteorological forecast which produce and disseminate forecasting and warnings, also carry out appropriate studies improvement for the system and the practice.

The information systems used by the SSRN are the following:

- Automatic network for hydro-meteorological monitoring;
- Meteorological radar;
- Automatic vertical profiler of the atmosphere;
- Meteorological forecasts on local and global scale;
- Numerical modeling for flood forecasting on the main river system.

Flood forecasting is conducted using the MIKE-FLOODWATCH system.

3.4 Flood Forecasting and Warning Service in the United Kingdom

The Environment Agency (EA) is a non-departmental organization, formed in 1996 and under administrated by the United Kingdom government's Department for Environment, Food and Rural Affairs (DEFRA), with the responsibilities relating to the protection and enhancement of the environment in England, such as: climate change, air quality, land quality, water quality, water resources, fishing, and river navigation (Todini, et.al., 2005). The EA is the primary authority for flood risk management operation. The EA is responsible for increasing public awareness of flood forecasting/warning, flood

risk, and has a general supervisory duty for flood control management. The EA administrates six regions in the United Kingdom: the Anglian Region, the Midlands Regions, the North West Region, the South West Region, the South East Region, and the Yorkshire & North East Region. The following subsection discusses the real-time flood forecast and operation in the Anglian Region, which is the largest of the six Environment Agency administrative regions.

3.4.1 The Anglian Flow Forecasting Modeling System (AFFMS)

The Environment Agency Anglian Region is responsible for flood forecasting and flood warning in the region (Todini, et.al., 2005). The Environment Agency Anglian Region cover an area of 10,502 square mile, and it is about twenty percent of England and Wales. The Anglian Region is the largest of the six Environment Agency administrative regions. The Anglian region has developed an internet-based comprehensive and fully operational, region-wide flow forecast modeling system, the Anglian Flow Forecasting Modeling System (AFFMS). The AFFMS has the following fundamental features (Todini, et.al., 2005):

- Highly accessible internet-based user interface that can used to view forecast data and conduct forecasts throughout the Anglian Region;
- Comprehensive geographic information system (GIS) user-interface for available geographical information;
- Easily understood display of forecast information designed for the general public;
- Comprehensive forecast databases with forecast analysis archive;

- A external data interface to allow visualization and application of a variety of data types from different sources;
- A generic modeling interface which allows application of different forecast modeling tools including the MIKE 11 system;
- User-defined scenarios to be evaluate alternative operation policies and uncertainty analysis.

3.5 Related Work on Real-Time Forecasting

3.5.1 Real-time River-Reservoir Optimization/Simulation Models

A modeling and methodology (Unver, 1987 and Unver and Mays, 1990) was developed for the real-time optimal flood operation of river-reservoir systems. The methodology was based on interfacing a nonlinear optimization model, which based upon the generalized reduced gradient approach, GRG2 (Lasdon, et al 1978 and Lasdon and Warren, 1978), with the U.S. NWS 1-D unsteady flood-routing simulation model, DWOPER (Fread, 1978). The model's objective function was based upon minimizing total damages of flood, which are functions of water surfaces elevations. The optimization model was formulated for the operation policy of multi-reservoir systems under flooding conditions to minimize the objective function, which defined by minimizing the total deviations from target level of water stages and/or discharges. The optimization model included hydraulic constraints and operational constraints. The optimization model (Unver, 1987 and Unver and Mays, 1990) for the operation of multi-reservoir systems under flooding conditions was formulated as follows:

1. Objective:

$$\text{Minimize } z = f(h, Q)$$

2. Constraints:

- a) Hydraulic constraints defined by the Saint-Venant equations for one-dimensional gradually varied unsteady flow and other relationships such as upstream, downstream, and internal boundary conditions and initial conditions that describe the flow in the different components of a river-reservoir system,

$$g(h, Q, r) = 0$$

- b) Bounds on discharges defined by minimum and maximum allowable reservoir releases and flow rate at specified locations,

$$\underline{Q} \leq Q \leq \bar{Q}$$

- c) Bounds on elevations defined by minimum and maximum allowable water surface elevations at specified locations (reservoir levels included),

$$\underline{h} \leq h \leq \bar{h}$$

- d) Physical and operational bounds on gate operations,

$$0 \leq \underline{r} \leq r \leq \bar{r} \leq 1$$

- e) Other constraints such as operating rules, targets, storages, storage capacities, etc.

$$W(r) \leq 0$$

The objective z is defined by minimizing the total flood damage or deviations from target levels, or water surface elevations in flood areas or spills from reservoirs or maximizing

storage in reservoirs. The variable h and Q are, respectively, the water surface elevation and the discharge at the computational points, and r is the gate setting. The objective function for minimizing the overall damage was formulated as the summation of the total damaged at each location. The mathematical expression for this objective function is:

$$\min z = \sum_i \sum_j c_i h_i^j ,$$

which $i \in I_c$ and $j \in T$, where z is the objective function value; i location index; I_c is the set that contains flood control locations; j is the time index; T is the time domain; c is the cost coefficient of flood damage. The real-time model was applied to the Highland Lake System including Lake Travis on the Lower Colorado River in Texas.

A newer model was developed (Ahmed and Mays, 2013), and it was also applied to the Highland Lake System. This newer model used a different unsteady one-dimension flow routing simulation model, the FEQ model (Franz and Melching, 1997 a and b), and a different optimization approach, a simulated annealing approach (Ahmed, 2006). The constraints of this model were the same as the Unver (1987) and Unver and Mays (1990) model. The objective of this model was also similar to the Unver and Mays model, which minimize the total damage or deviations from target levels of water surface elevations and/or discharges.

$$\min z = \sum_i \sum_j c_i h_i^j + c'_i Q_i^j ,$$

which $i \in I_c$ and/or I_c , $j \in T$. The indices are the same as the previous model described previously. The new term, c'_i is the cost coefficient as a function of discharge.

3.5.2 Reservoir Release Forecast Model (RRFM)

A Decision Support System for real time flood operation was developed for the Folsom Project on the American River above Sacramento, California (Bowles, et. al., 2004). It was a collaboration of multiple agencies. The Water Resources Development Act of 1999 specifically required the Secretary of Army, in cooperation with the Secretary of Interior, to update the Flood Management Plan for the Folsom Dam to reflect the operational capabilities created by a modification to increase outlet capacity, and by improved weather forecasts based on the Advanced Hydrologic Prediction System (AHPS) of the Nation Weather Service (NWS). The main objective of the project is to capture as much flood water as the existing infrastructure reasonably can achieve. Utah State University (USU) has developed the Decision Support System, the Folsom Reservoir Release Forecast Model (RRFM). The model provides a means of examining various tradeoffs associated with the timing of reservoir releases as they affect downstream flood management, lead time for evacuation in the event of releases that are expected, and dam safety. Since 1996, a group of multi-agencies has work with USU team to develop, test, and the implementation of the RRFM for real-time operation of the Folsom Project (Bowles, et. al., 2004). The working group includes: the U.S. Bureau of Reclamation (USBR), the U.S. Army Corps of Engineers (USACE), the Sacramento Area Flood Control Agency (SAFCA), the National Weather Service (NWS), the California Division of Water Resources (DWR), the American River Flood Control District, the U.S. Fish and Wildfire Service (USFWS), and the Hydrologic Research Center (HRC). The model consists of two modes: 1) the Deterministic and Uncertainty Options of the Operational (real time) Mode, and 2) the Batch and Interactive (pseudo real-time)

Options of the Planning (off-line) Mode. This basic structure of the RRFM is illustrated in Figure 3.5.

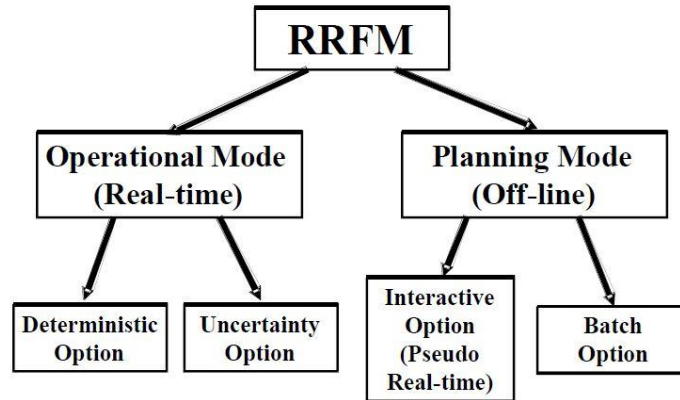


Figure 3.5: The Basic Structure of the Folsom Reservoir Release Forecasting Model (Bowles et al, 2004)

The RRFM can be used in an Operational (real time or on-line) Mode or a Planning (off-line) Mode (Bowles, et. al., 2004), as illustrated in Figure 3.5. Under the Deterministic Operational Mode, which is currently in use by the U.S. Bureau of Reclamation, the model captures various input variables, including inflow forecasts from the National Weather Service, the California Nevada River Forecast Center, and Sacramento Soil Moisture Accounting Model simulations. Through application of the flood control and emergency spillway release rules, the RRFM provides forecasts of release rates and timing, downstream river stages, and reservoir replenishment. The RRFM can be used during flood operations to simulate “what if” scenarios to explore other substitute operating approaches. Under the Uncertainty Operational Mode, the

RRFF can provide probabilistic approximations for the forecast variables based on propagating forecast uncertainties through reservoir operation.

In the Planning Mode, the model can simulate under either Batch or Interactive (pseudo real time) processing (Bowles, et. al., 2004). It generates a collective of inflow forecasts with forecast error structure that statistically resembles that observed historically. All output variables are presented in probabilistic form. This mode can be used to simulate historical events, design floods, or hypothetical inflow hydrographs for training, for assisting downstream emergency managers in developing practices for using RRFM probabilistic release forecasts or for developing and testing operating rule alterations, including possible pre-release strategies.

3.5.3 Rainfall Forecasting Using Artificial Neural Network

In the past couple decades, the use of Artificial Neural Network (ANN) for rainfall forecasting has gained significant attention due to the advancement of computational power. The development of the artificial neural network started in the 1940s, which was inspired by a desire to understand the human brain and emulate its functioning (Govindaraju, 2000a). Mathematically, an ANN is often regarded as a tool for universal approximation. The power to identify a relationship from given pattern make it probable for ANNs to solve complex large-scale problems such, pattern recognition, classification, nonlinear modeling, and control (Govindaraju, 2000a). An ANN is a massive parallel-distributed information processing system that has similar performance characteristics resembling biological neural networks of the human brain (Govindaraju, 2000a). An ANN is based on the following assumptions:

- Information processing occurs at many single elements called nodes;
- Signals are passed between nodes through connection links;
- Each link has an associated weight that represent its connection strength;
- Each node sometimes applies a nonlinear transformation called an activation function to its net input to determine its output signal.

One example of an ANN is a feed-forward network. This type of ANN is generally arranged in layers, starting from an input layer and ending at the final output layer. Figure 3.6 illustrates a configuration of a feed-forward three layer artificial neural network. Between the input and output layers, there can be hidden layers in the middle, with each layer have one or more nodes. Information enters from the input layer and exits on the output layer. The nodes are connected to the next layer, but not in the same layer. Thus, the output node in a layer only depends on the input it receives from the previous layers and the corresponding weights.

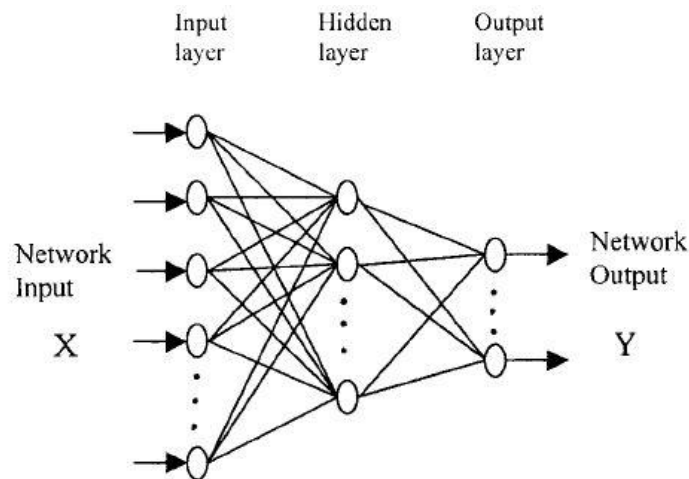


Figure 3.6: Configuration of Feed-Forward Three Layer Artificial Neural Network (Govindaraju, 2000a)

In most networks, the input layers receive the input variables and information for the problem. This consists of all quantities that can influence the output. Thus, the input layer is transparent and is a means of providing data to the network. The output layer consists of values predicted by the network and represents the output of the ANN model. The number of hidden layers and the number of nodes in each hidden layer are usually defined by a trial-and-error procedure. Govindaraju (2000b) described a number of studies which have used artificial neural networks to forecast rainfall over a short time domain. French, Krajewski, and Cuykendal (1992) developed the first ANN simulation, which hypothetically generated rainfall, and the storm data were used to calibrate and validate ANN models. In their studies, the ANN is quite capable of capturing the complex relationship associated with spatiotemporal evolution of rainfall inherent in a complex rainfall simulation model. Brath, Montanari, and Toth (2002) compared the ANN model with empirical predictors for real-time rainfall forecasting. Their study showed that, besides the adaptive training, the ANNs showed a remarkable improvement over the deterministic rainfall-runoff model in real-time rainfall forecasting. Nasseri, Asghari, and Abedini (2008) coupled an artificial neural network with a genetic algorithm optimization model for real-time forecasting. They used the genetic algorithm optimization model to train the ANN to produce optimized forecasted rainfall.

3.5.4 Previous Optimization/Simulation Models

Many optimization models have been reported in the literature over the years for reservoir operation including flood control as one of the purposes. These include: Windsor (1973), Can and Houck (1984), Marien (1984), Kelman, et al. (1989), Marien, et al. (1994), Chang and Chen (1998), Chuntian (1999), Needham et al. (2000), Cheng and

Chau (2001), Cheng and Chau (2004), Chang (2008), Fu (2008), Asadipoor and Samani (2010), Choudhury (2010), and Kumar et al. (2010). Unfortunately none of these models have the capabilities of the proposed model developed in this research.

A modeling methodology (Unver, 1987 and Unver and Mays, 1990) was developed for the real-time optimal flood operation of river-reservoir systems. The methodology was based on interfacing a nonlinear optimization model, which based upon the generalized reduced gradient approach, GRG2 (Lasdon, et al 1978 and Lasdon and Warren, 1978), with the U.S. NWS 1-D unsteady flood-routing simulation model, DWOPER (Fread, 1978). The model's objective function was based upon minimizing total damages of flood, which are functions of water surfaces elevations. The optimization model was formulated for the operation policy of multi-reservoir systems under flooding conditions to minimize the objective function, which is defined by minimizing the total deviations from target levels of water stages and/or discharges. The optimization model included hydraulic constraints and operational constraints. The real-time model was applied to the Highland Lake System including Lake Travis on the Lower Colorado River in Texas (Mays, 1991).

Other models that were developed to be used for real-time purposes include Hsu and Wei (2007) who developed a real-time operation model for determining the optimal releases during a typhoon. This model had the objective of minimizing the peak flow at downstream control points along with maximizing the reservoir storage at the end of the flood. A real-time model using optimization- simulation was developed by Wei and Hsu (2008) for determining reservoir releases at each time during a flood. This model

included two models a hydrological forecasting model and a reservoir operation model. Two flood-control operation strategies for a multi-reservoir system were formulated and solved using mixed-integer linear programming. Wei and Hsu (2009) developed optimal tree-based release rules for real-time flood control operations on a multipurpose multi reservoir system. Chang, et al. (2010) reported a real-time reservoir operation mode for flood control using artificial intelligent techniques. Malekmohammadi, et al. (2010) developed a real-time flood management model for river-reservoir systems based upon combining a reservoir operation based upon a genetic algorithm with a hydraulic flood routing simulation for routing reservoir releases in the downstream river. The operation model determines the hourly releases that minimize the flood damages in the downstream river. Bayat, et al. (2011) developed an optimization-simulation model short-term reservoir operation under flooding conditions. The problem was formulated as a combination of particle swarm optimization and a simulation model for river flood routing using both hydrologic and hydraulic flood routing methods. The purpose of the model was for minimizing flood damages in the downstream areas. Fallah-Mehdipour, et al. (2012) suggested the use of genetic programming (GP) to develop a reservoir operation policy simultaneously with inflow prediction. The method was to extract an operational policy simultaneously with inflow prediction helps the operator to make decision to determine how much water to release from the reservoir without employing a prediction model. Wang, et al (2013) developed a multi-objective optimization model in a multi-reservoir system during flood season using short-term Numerical Weather Predictions (NWP) outputs. The optimization model was coupled with the Water and Energy Budget-based Distributed Hydrological Model that was used to forecast the

reservoir inflows. The reservoir objective function was established by considering the reservoir, upstream, and downstream safety, as well as future water use. A methodology proposed by Chiang and Willems (2015) combines Genetic Algorithm (GA), with the Model Predictive Control (MPC) technique to develop and test a real-time flood control method for the 12-gated weirs in the Belgian case study of the river Demer. The model searches for better operation by minimizing a cost function while avoiding violation of the system constraints. Schwanenberg, et al (2015) developed a model for short-term reservoir operation by integrating several components, including hydrological model and data assimilation techniques for predicting stream flow, optimization model for decision making on the reservoir operation and the technical framework for integrating these components with data feeds from gauging networks, remote sensing data and meteorological weather predictions. The model used a multi-stage stochastic optimization approach.

3.6 U.S. Army Corps of Engineers

In the recent years, the USCAE has been developing real time reservoir operation software package called the Corps Water Management System (CWMS) (USACE, 2000b and 2002). CWMS links HEC-HMS, HEC-RAS, HEC-ResSim for reservoir operation, and HEC-FIA together. HEC-FIA is an economic damage program, which stands for Flood Impact Analysis. The basic goal of CWMS is to determine optimal reservoir operations that will minimize downstream flood damages. It uses hydrologic predictions, such as the quantitative precipitation forecasts (QPF), as input to the HMS models. CWMS software itself is USACE-specific software, which means that only USACE

personnel or contractors for the USACE can use the software. The USACE has been implementing CWMS in most of the watersheds throughout the US (USACE 2011b). There is a version of CWMS for the general public; it is called HEC-RTS (Real Time System). However, HEC-RTS is still in development and not yet released to the public (USACE, 2010d). The CWMS and HEC-RTS use penalty function and linear programming for the optimization approach (USACE, 2003 and 2011a), which is becoming outdated and has limited solution search capability; as compared to far more robust meta-heuristic approaches, for example, genetic algorithm and simulated annealing, to name a few.

3.6.1 Corps Water Management System (CWMS)

The Corps Water management System (CWMS) is an automated information system that supports the USACE water control management throughout the United States (USACE, 2002). The CWMS is used to obtain real-time data on watersheds parameters, develop hydrologic and hydraulic forecasts of project inflows and uncontrolled flows projects, determine project releases, and evaluate possible impacts of alternate release scenarios. Real-time data of the CWMS includes river stage, reservoir elevation, gage and spatial precipitation, quantitative precipitation forecasts (QPF) and other hydro-meteorological parameters. Hydrological response throughout a watershed area is derived using these input data, including short-term future reservoir inflows and uncontrolled downstream flows (USACE, 2002). The reservoir operation model flows are then determined to provide proposed releases to meet reservoir and downstream operation objectives. The total expected flows in the river system and profiles are computed, inundated areas mapped, and flood impacts analyzed. The CWMS is able to

evaluate any number of operation alternatives before a final forecast scenario and release decision are adopted (USACE, 2002).

CWMS suite includes precipitation input, hydrologic response modeling, reservoir operation modeling, steady and unsteady flow river profile analysis, inundated area determination, and analysis of flood impacts. Figure 3.7 shows the models as integrated in CWMS. The models that are implemented in the CWMS include HEC-HMS – precipitation runoff; HEC-ResSim – reservoir system; HEC-RAS – river profile analysis; HEC-FIA – flood impact (damage); and ArcInfo/ArcView – inundation boundary computation and GIS viewing (USACE, 2002). Figure 3.7 shows the models as integrated in CWMS.

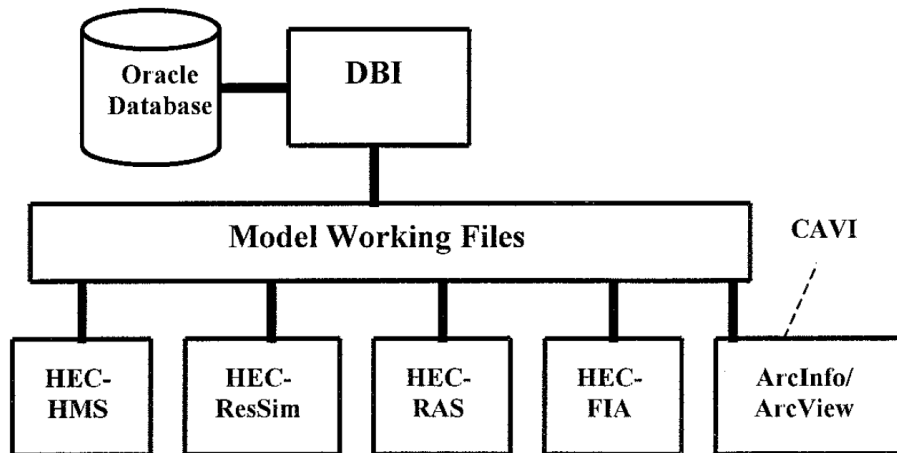


Figure 3.7: Corps Water Management System Models Integration Schematic (USACE, 2002)

3.6.2 HEC-RTS (Real-Time Simulation)

The Hydrologic Engineering Center's Real-Time Simulation (HEC-RTS) system is a publicly available version of CWMS's data visualization and modeling capabilities that runs on a single computer (USACE, 2010d).

HEC-RTS provides support for operational decision making by forecast simulation modeling using any combination of the USACE models. Rainfall-runoff modeling with HEC-HMS based on gaged or radar-based precipitation, Quantitative Precipitation Forecasts (QPF) and other future precipitation scenarios provides forecasts of uncontrolled flows into and reservoir downstream (USACE, 2010d). Simulation of reservoir operations with HEC-ResSim provides operational decision information for the manager. The river hydraulics program HEC-RAS computes river stages and water surface profiles for these scenarios. Inundation boundaries and depth maps of water in the flood plain can be generated from the HEC-RAS results using ArcInfo. The economic impacts of different flow alternatives are computed by using HEC-FIA (USACE, 2010d). The user-defined sequence of modeling software allows engineers to evaluate operational decisions for reservoirs and other control structures, and view and compare hydraulic and economic impacts for various alternative scenarios (USACE, 2010d). The data flow for the models in HEC-RTS is illustrated in Figure 3.8.

In summary very few models have been reported that have developed combined optimization/simulation approaches for the real-time operation of river-reservoir systems. The model by Unver and Mays (1990) was the first. A newer model was developed by Ahmed and Mays (2013), and it was also applied to the same river-reservoir system as

Unver and Mays (1990) used. This newer model used a different unsteady one-dimension flow routing simulation model, the FEQ model (Franz and Melching, 1997 a and b), and a different optimization approach, a simulated annealing approach (Ahmed, 2006). There have been various reservoir operation models based upon simulation/optimization for various purposes, e.g. for sediment control models have been developed by Carriaga and Mays (1995 a, b) and Nicklow and Mays (2000 and 2001).

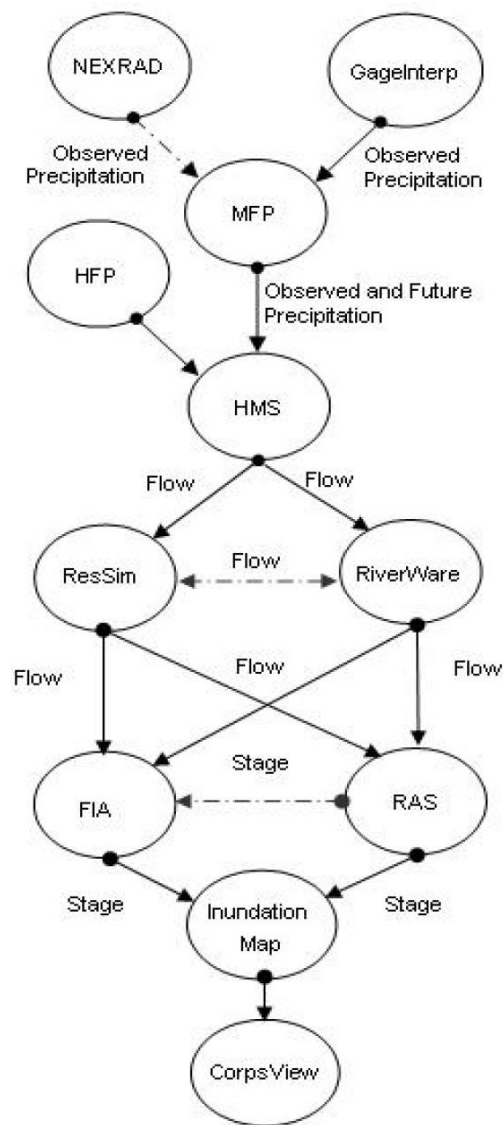


Figure 3.8: HEC-RTS Models Integration Schematic (USACE, 2010d)

The optimization/simulation model introduced herein uses more innovative and robust approaches including a hydrologic model (HEC-HMS), an 1-D unsteady flow model (HEC-RAS), a reservoir operation model, a genetic algorithm optimization model, and a rainfall forecasting model, all interfaced together for use as a real-time operation model. To our knowledge such a model has never been reported in the literature.

CHAPTER FOUR – RAINFALL-RUNOFF MODELS

4.1 KINEROS2

The Kinematic Runoff and Erosion Model, KINEROS2, is an event-oriented, physically based model describing the process of interception, infiltration, surface runoff, and erosion from small agricultural and urban watersheds (Woolhiser et al, 1990). A cascade of planes and channels represents the watershed; the partial differential equations describing overland flow, channel flow and erosion, and sediment transport are solved by finite difference techniques. Spatial variability of rainfall and infiltration, runoff, and erosion parameters can be accommodated. KINEROS may be used to determine the effects of various artificial features such as urban developments, small detention reservoirs, or lined channels on flood hydrographs and sediment yield. The solution solving procedure of the model is presented below (Woolhiser et al, 1990):

Initial Procedure

Divide Watershed into Plane
and Channel Elements



Interception

Interception Depth (I) is
Subtracted from Rainfall before
infiltration is calculated



Infiltration

Smith and Parlange Equation is used in KINEROS

$$f_c = K_s \frac{e^{F/B}}{e^{F/B} - 1}$$

where

f_c = infiltration capacity
 K_s = hydraulic conductivity
 F = Cumulative Infiltration
 $B = G\phi(S_{max} - S_i)$
 G = effective net capillary drive



Overland Flow (Hortonian)

$$h_{j+1}^{i+1} - h_{j+1}^i + h_j^{i+1} - h_j^i + \frac{2\Delta t}{\Delta x} \left\{ \theta_w \left[\alpha_{j+1}^{i+1} (h^m)_{j+1}^{i+1} - \alpha_j^{i+1} (h^m)_j^{i+1} \right] + (1 - \theta_w) \left[\alpha_{j+1}^i (h^m)_{j+1}^i - \alpha_j^i (h^m)_j^i \right] \right\} - \Delta t (\bar{q}_{j+1} + \bar{q}_j) = 0$$

Only unknown: $(h^m)_{j+1}^{i+1}$

Kinematic Wave equation is solved numerically by 4-pts implicit method by Newton's Method.



Channel Routing

$$A_{j+1}^{i+1} - A_{j+1}^i + A_j^{i+1} - A_j^i + \frac{2\Delta t}{\Delta x} \left\{ \theta_w \left[\frac{dQ}{dA} (A_{j+1}^{i+1} - A_j^{i+1}) \right] + (1 - \theta_w) \left[\frac{dQ}{dA} (A_{j+1}^i - A_j^i) \right] \right\} - 0.5\Delta t (q_{c,j+1}^{i+1} + q_{c,j}^{i+1} + q_{c,j+1}^i + q_{c,j}^i) = 0$$

Only unknown: A_{j+1}^{i+1}

Kinematic equations for channel are solved numerically by 4-pts implicit method (Newton's iterative technique).



Reservoir Routing

$$\frac{dV}{dt} = \frac{V_i - V_{t+\Delta t}}{\Delta t} = q_i - q_o = q_i - c_1 (h_r - h_z)^{C_2}$$

Reservoir is described by mass balance and outflow equations. The stage of reservoir is determined by *bisection method*.



Erosion and Sediment Transport

Mass balance equation is used to describe the sediment dynamics at any point along a surface flow path:

$$\frac{d}{dt}[AC_s] = \frac{d}{dx}[QC_s] - e(x,t) = q_s(x,t)$$

where

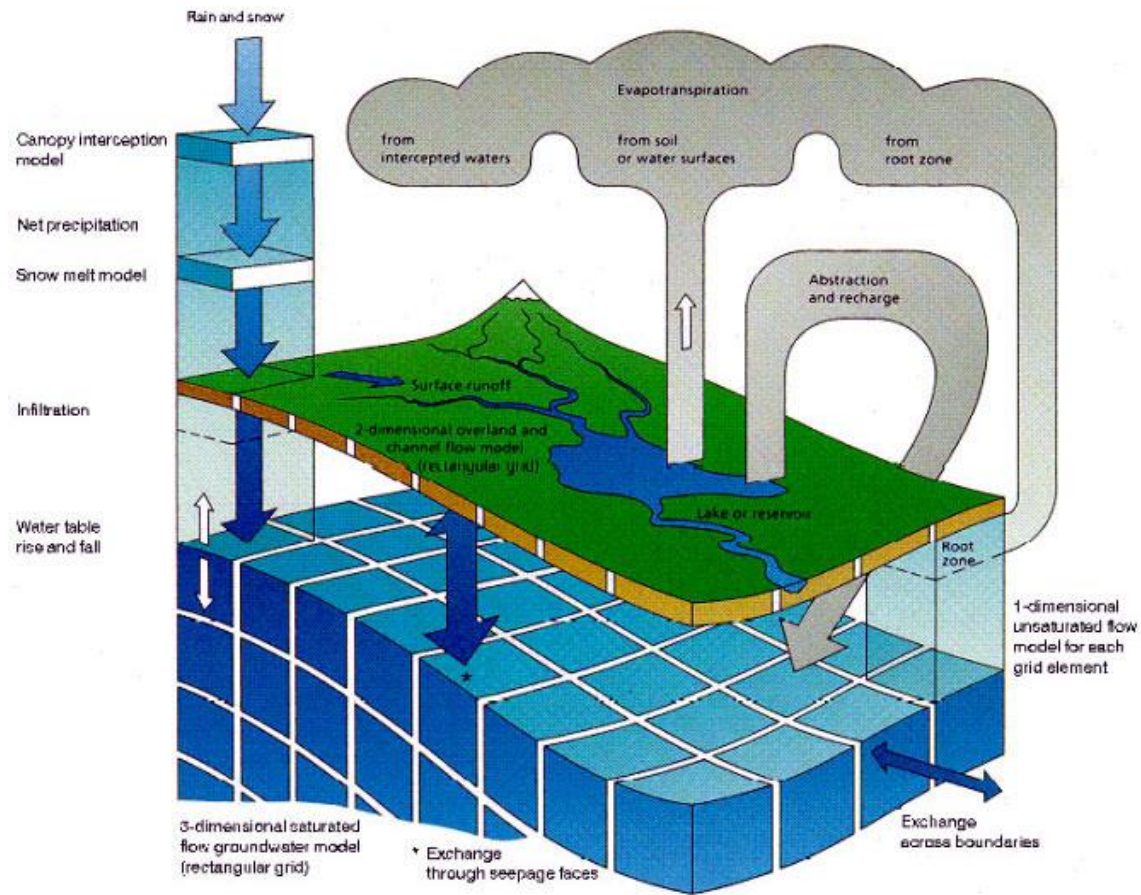
C_s = sediment concentration

q_s = rate of lateral sediment inflow for channels

e = rate of erosion of soil bed

4.2 MIKE SHE

The integrated hydrological modeling system MIKE SHE was first developed by Institute of Hydrology in the United Kingdom, Société Grenobloise d'Etudes et d'Applications Hydrauliques (SOGREAH) in France, and Danish Hydraulic Institute (DHI) in Denmark in 1977 (DHI, 2005). The model simulates water flow in the entire land based phase of the hydrological cycle from rainfall to river flow, via different flow processes such as overland flow, infiltration in soils, evapotranspiration from vegetation, and ground water flow. MIKE SHE can be characterized as a deterministic, physically based, distributed model. Figure 4.1 shows how a catchment is represented in an integrated fashion by the major processes and their interaction (DHI, 2005):



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Figure 4.1: Process-based Structure of the MIKE SHE Hydrological Modeling System
(Sources: DHI, 2005)

Applications of MIKE SHE include but not limited to integrated catchment hydrology, conjunctive use of surface water and subsurface water, irrigation and drought management, wetland management and restoration, environmental river flows, floodplain management, induced groundwater flood, climate and land use change, nutrient fate and management, and groundwater remediation (DHI, 2005).

4.3 HEC-HMS

The first component of the optimization/simulation model is the hydrologic model HEC-HMS. HEC-HMS is the U.S. Army Corps of Engineers' Hydrologic Modeling System (HMS) computer program developed by the Hydrologic Engineering Center (HEC). For rainfall-runoff simulation, HEC-HMS fundamentally offers the following components (USACE, 2000a and 2010b):

- 1) Specific precipitation input options: users can use observed hyetographs from real event as the rainfall input to the model, or users can use hypothetical precipitation which is frequency based;
- 2) Rainfall loss models: estimate the effective runoff volume, given the precipitation and watershed properties;
- 3) Direct runoff models: account for Hortonian overland flow, storage, and losses of energy as water runs off a watershed and into stream, channels;
- 4) Hydrologic routing models: account for energy flux and storage as water moves through stream network.

Figure 4.2 illustrates a watershed scale rainfall-runoff process represented by HEC-HMS.

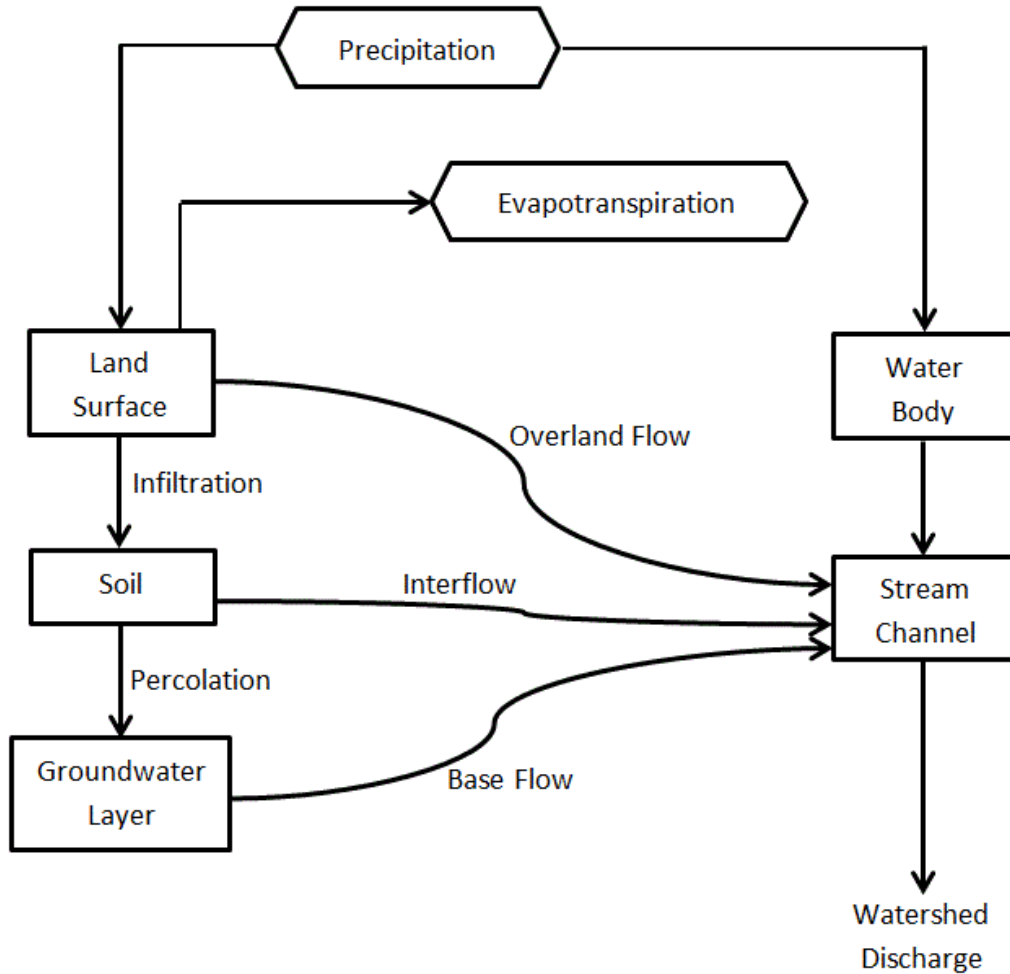


Figure 4.2: HEC-HMS Representation of Watershed Runoff (Sources: USACE, 2000a)

When modeling a storm event using HEC-HMS, precipitation falls on land surface, and water may pond. For continuous, non-event based simulation, evapotranspiration may be included in the model. Depending soil type, land surface type, antecedent moisture and other properties of the watershed, a portion of the water may infiltrate. This infiltrated water is stored temporarily in the soil layer. Although physically, some of the infiltrated water may rise to the surface again due to capillary action, HEC-HMS does not include this phenomenon. Instead, HEC-HMS accounts for

horizontal movement as interflow just beneath the surface, and the model also account for vertical percolation of water from soil layer to groundwater aquifer underneath the watershed. The interflow eventually moves into the basin stream channel. Water in the groundwater layer although moves very slowly, a portion of it eventually returns to the channel as base flow. Rainfall that does not pond or infiltrate moves by overland flow to a basin stream channel, and the total watershed outflow is the combination of overland flow, the rainfall that directly falls on water bodies in the watershed, and interflow and base flow (USACE, 2000a and 2010b). In the optimization/simulation model, the HEC-HMS serves as the first component to compute the watershed runoff after a storm event, with given input of an observed hyetograph or a designed storm. The watershed runoff data then becomes the input of the next component, the hydraulic unsteady flow model.

CHAPTER FIVE – UNSTEADY FLOW MODELS

5.1 NWS Models

5.1.1 DWOPER (Dynamic Wave Operational Model)

In the early 1970s, the NWS Hydrologic Research laboratory began to develop a dynamic wave routing method based on the implicit finite difference solution of the St. Venant equations. This model is known as DWOPER (Dynamic Wave Operational Model) (Fread, 1978). DWOPER routing is a dynamic wave flood routing model that routes an inflow hydrograph to a point downstream. It can be used on a single river or system of rivers where storage routing methods are inadequate due to the effects of backwater, tides and mild channel bottom slopes. The model is based on the complete one-dimensional St. Venant equations. A weighted four-point nonlinear implicit finite difference scheme is used to obtain solution to the St. Venant equations using a Newton-Raphson iterative technique.

DWOPER has a number of features (Fread, 1978) that make it applicable to a variety of natural river systems for real-time forecasting. It is designed to accommodate various boundary conditions and irregular cross-sections at unequal distances along a single multiple-reach or several such rivers having a dendritic configuration. It allows for roughness parameters to vary with location, stage or discharge. Temporally varying lateral inflow, wind effects, bridge effects, off-channel storage and weir-flow channel bifurcations to simulate levee overtopping are included among its features. Time steps are solely on desired accuracy since the implicit finite difference technique is not restricted to the very small time steps of explicit technique due to numerical stability considerations. This enables DWOPER to be very efficient as to computational time for

simulating slowly varying floods of several days duration. The mathematical basis for DWOPER is a finite difference solution of the conservation form of the one-dimensional equations flow consisting of the conservation of mass and momentum equations (Fread, 1978):

- Conservation of Mass Equation

$$\frac{\partial Q}{\partial x} + \frac{\partial(A + A_0)}{\partial t} - q = 0 \quad (5.1)$$

- Momentum Equation

$$\frac{\partial Q}{\partial t} + \frac{\partial(Q^2 / A)}{\partial x} + gA \left[\frac{\partial h}{\partial x} + S_f + S_e \right] - qv_x + W_f B = 0 \quad (5.2)$$

where

Q is the discharge;

A is the cross-sectional area;

A_0 is the off channel cross-sectional area where velocity is negligible;

h is the water surface elevation;

q is the lateral inflow or outflow;

x is the distance along the channel;

t is the time;

g is the acceleration of gravity;

v_x is the velocity of lateral inflow in x-direction;

W_f is the wind term;

B is the top width of the channel;

S_f is the slope of the energy grade line derived from the Manning's equation;

S_e is the large-scale eddy loss slope for contraction/expansion.

5.1.2 DAMBRK (Dam-Break Flood Forecasting Model)

Forecasting downstream flash floods due to dam failure is an application of flood routing that has gain considerable attention in recent decades. The most widely used dam-break model in late 1970s to early 1990s was the NWS DAMBRK (Dam-Break Flood Forecasting) model by Fread (1977,1978, 1980). This model consisted of three functional components: (1) temporal and geometric description of the dam breach; (2) computation of the breach outflow hydrograph; and (3) routing the breach outflow hydrograph downstream. In the DAMBRK model, the reservoir outflow consisted of both the breach outflow Q_b (board-crested weir flow) and spillway outflow Q_s :

$$Q = Q_b + Q_s \quad (5.3)$$

The break outflow can be computed by using the combination of the formulas for a board-crested rectangular weir, gradually enlarging as the breach widens, and a trapezoidal weir for the breach side slopes (Fread, 1980):

$$Q_s = 3.1B_w t_b C_v K_S \frac{(h - h_b)}{T} + 2.45z C_v K_S (h - h_b)^{2.5} \quad (5.4)$$

where

t_b is the time after dam breaching;

B_w is the width of the breach bottom;

C_v is the correction factor for the approaching velocity;

K_S is the submergence correction for the tail water effects on weir flow;

h is the reservoir water surface elevation;

h_b is the breach bottom elevation;

T is the failure time interval;

z is the side slope of the breach (trapezoidal shape assumed).

The spillway out flow can be computed using the following formula (Fread, 1980):

$$Q_s = C_s L_s (h - h_s)^{1.5} + \sqrt{2g} C_g A_g (h - h_s)^{0.5} + C_d L_d (h - h_d)^{0.5} + Q_t \quad (5.5)$$

where

C_s is the uncontrolled spillway discharge coefficient;

L_s is the of the uncontrolled spillway length;

h_s is the uncontrolled spillway crest elevation;

C_g is the gated spillway discharge coefficient;

A_g is the area of the gate opening;

h_g is the center-line elevation of the gated spillway;

C_d is the dam crest flow discharge coefficient;

L_d is the crest length;

h_d is the dam crest elevation;

Q_t is the constant outflow or leakage.

The DAMBRK model used hydrologic storage routing or the dynamic wave model to compute the reservoir outflow. The reservoir outflow hydrograph is then routed downstream using the full dynamic wave model (Fread, 1980), or simply the continuity and momentum equations, neglecting wind shear and lateral flow momentum:

$$\frac{\partial(K_c Q)}{\partial x_c} + \frac{\partial(K_l Q)}{\partial x_l} + \frac{\partial(K_r Q)}{\partial x_r} + \frac{\partial(A_c + A_l + A_r + A_0)}{\partial t} - q = 0 \quad (5.6)$$

and

$$\frac{\partial Q}{\partial t} + \frac{\partial(K_c^2 Q^2 / A_c)}{\partial x_c} + \frac{\partial(K_l^2 Q^2 / A_l)}{\partial x_l} + \frac{\partial(K_r^2 Q^2 / A_r)}{\partial x_r} + gA_c \left[\frac{\partial h}{\partial x_c} + S_{fc} + S_e \right] + gA_l \left[\frac{\partial h}{\partial x_l} + S_{fl} \right] + gA_r \left[\frac{\partial h}{\partial x_r} + S_{fr} \right] = 0 \quad (5.7)$$

The subscripts, l , r , and c , denoted in equations 5.5 and 5.6 are the left flood plain, the right flood plain, and the channel. The cross-section area of the flow is the sum of A_c , A_l , A_r , and A_o . The constants K_c , K_l , and K_r divide the total flow Q into channel flow, left flood plain flow, and the right flood plain flow, respectively, which $K_c = Q_c / Q$, $K_l = Q_l / Q$, and $K_r = Q_r / Q$. In the late 1980s and early 1990s, a newer computational hydraulic routing model, the NSW Flood Wave Routing Model (FLDWAV), eventually replaced the DAMBRK model.

5.1.3 FLDWAV

The NWS FLDWAV model (Flood Wave Routing Model), is a combination of DWOPER and DAMBRK, and adds significant modeling capabilities not available in either of the other model. FLDWAV is primarily based on the four-point implicit finite-difference numerical solution scheme of the expanded complete Saint-Venant equations of one-dimensional unsteady flow along with appropriate internal boundary equations representing downstream dams, ridges, weirs, waterfalls, and other man-made or natural flow controls. The expanded Saint-Venant equations, which govern the FLDWAV model (Fread, 1998) are:

- Conservation of Mass Equation

$$\frac{\partial Q}{\partial x} + \frac{\partial s_{co}(A + A_0)}{\partial t} - q = 0 \quad (5.8)$$

- Momentum Equation

$$\frac{\partial(s_m Q)}{\partial t} + \frac{\partial(\beta Q^2 / A)}{\partial x} + gA \left[\frac{\partial h}{\partial x} + S_f + S_e + S_i \right] - L + W_f B = 0 \quad (5.9)$$

where

Q is the discharge;

A is the cross-sectional area of flow;

A_0 is the inactive off-channel cross-sectional area;

h is the water surface elevation;

s_{co} and s_m are the sinuosity factors which vary with h ;

q is the lateral inflow or outflow per lineal distance;

x is the longitudinal distance along the river;

t is the time;

β is the momentum correction coefficient;

g is the acceleration of gravity;

L is the momentum effect of lateral flow;

W_f is the surface wind resistance;

B is the top width of the channel;

S_f is the slope of the energy grade line derived from the Manning's equation;

S_e is the contraction/expansion slope;

S_i is the additional friction slope associated with internal viscous dissipation of non-Newtonian fluids.

The FLDWAV model includes the following capabilities not found in DAMBRK: (1) the flood may occur in a system of interconnected rivers such as main-stem river and the tributaries; (2) levee-overtopping/crevasse flows into and through levee-protected floodplains that may be compartmentalized by dikes and elevated roadways; (3) automatic calibration of Manning's n values based on observed historical floods; (4) improved numerical stability; (5) menu-driven interactive data input; and (6) color graphics displays of model output.

The NWS FLDWAV model has been widely used by hydrologists/engineers for real-time flood forecasting of dam-break floods and/or natural floods, dam breach flood analysis of overtopping associated with the PMF flood, flood plain inundation mapping for contingency dam break flood planning, debris inundation mapping, and improvements of waterway design (Fread, 1998). In the late 2000s, NWS began the phases to replace the FLDWAV model with the USACE HEC-RAS model (Reed, 2010 and Moreda, 2010).

5.2 FEQ (the USGS Model)

The Full Equations (FEQ) model by the U.S. Geological Survey (USGS) for the simulation of one-dimensional unsteady flow in open channels and through control structures was first developed in 1976 (Franz and Melching 1997a and 1997b). The FEQ has been widely used and updated since its first development. A system of stream that is simulated by application of FEQ is subdivided into stream reaches, parts of the stream

system for which complete information on flow and depth are not required (classified as dummy branches), and level-pool reservoirs. These components are connected by special features, such as, hydraulic control structures, including junctions, bridges, culverts, dams, waterfalls, spillways, weirs, side weirs, and pumps. The principles of conservation of mass and conservation of momentum are used to calculate the flow and depth throughout the stream system given the known information of initial and boundary conditions. The FEQ is solved by an implicit finite-difference approximation at fixed points. The equations represented in the FEQ model are the integral form of the conservation of mass (continuity equation) and conservation of momentum (motion equation) (Franz and Melching, 1997a and 1997b):

- Conservation of Mass

$$\int_{x_1}^{x_2} [(A)_{t_2} - (A)_{t_1}] dx = \int_{t_1}^{t_2} [(uA)_{x_1} - (uA)_{x_2}] dt \quad (5.10)$$

- Conservation of Momentum

$$\begin{aligned} \int_{x_1}^{x_2} [(uA)_{t_2} - (uA)_{t_1}] dx &= \int_{t_1}^{t_2} [(u^2 A)_{x_1} - (u^2 A)_{x_2}] dt \\ + g \int_{t_1}^{t_2} [(I_1)_{x_1} - (I_2)_{x_2}] dt &+ g \int_{t_1}^{t_2} \int_{x_1}^{x_2} I_2 dx dt \\ + g \int_{t_1}^{t_2} \int_{x_1}^{x_2} A(S_0 - S_f) dx dt \end{aligned} \quad (5.11)$$

where

u is the velocity;

A is the cross-sectional area of flow;

x is the distance along the channel;

t is the time;

g is the acceleration due to gravity;

I_1 is the hydrostatic pressure exerted on the ends of the control-volume element;

I_2 is the component of pressure in the direction of the channel axis because of the non- prismatic channel wall;

S_0 is the bottom slope of the channel, positive with decline downstream;

S_f is the energy gradient.

The FEQ model solves the numerical solutions of the continuity and momentum equations by the finite-different four-point weighted implicit scheme.

5.3 MIKE 11 (the DHI Model)

The MIKE 11, developed by the Danish Hydraulic Institute (DHI), is an implicit finite model for one-dimensional unsteady flow computation (DHI, 2005). The model applied with the fully dynamic descriptions solves the vertically integrated equations of the conservation of mass and conservation of momentum, known as the Saint Venant equations, which are based on the following assumptions:

- i. Incompressible flow and homogeneous
- ii. Very mild channel bed slope
- iii. Wave lengths are large compared to water depth
- iv. Open channel flow regime is sub-critical

MIKE 11 uses the implicit 6-point Abbott scheme to solve the governing equations (DHI, 2009):

- Conservation of Mass Equation

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = q \quad (5.12)$$

- Momentum Equation

$$\frac{\partial Q}{\partial t} + \frac{\partial(\beta Q^2 / A)}{\partial x} + gA \frac{\partial h}{\partial x} + \frac{gQ|Q|}{C^2 AR} = 0 \quad (5.13)$$

where

Q is the discharge;

A is the flow area;

x is the distance along the channel;

t is the time;

g is the acceleration due to gravity;

q is the lateral flow;

h is the flow depth;

R is the hydraulic radius;

C is the Chezy resistance coefficient;

β is the momentum correction factor.

MIKE 11 has been designed to perform detailed modeling of rivers, including special condition of floodplains, road overtopping, culvert, gate openings, and weir flows. MIKE 11 is accepted by the U.S. Federal Emergency Management Agency (FEMA) for use in the National Flood Insurance Program (DHI, 2005).

5.4 HEC-RAS (the USACE Model)

The second major component of the optimization/simulation model is the hydraulic unsteady flow model HEC-RAS, which allows users to perform one-dimensional, unsteady flow computation. The physical laws that govern the unsteady flow of water in a stream are called the Saint-Venant equations represented by the continuity equation and the momentum equation. The *Saint-Venant equations* first developed by Barre de Saint-Venant in 1871, describes one-dimensional unsteady open channel flow (USACE, 2010a).

The elementary control volume for derives the continuity equation and the momentum equation is illustrated in Figure 5.1. The complete derivation of the *Saint-Venant equations* can be found in text *Applied Hydrology* (Chow, Maidment, and Mays, 1988).

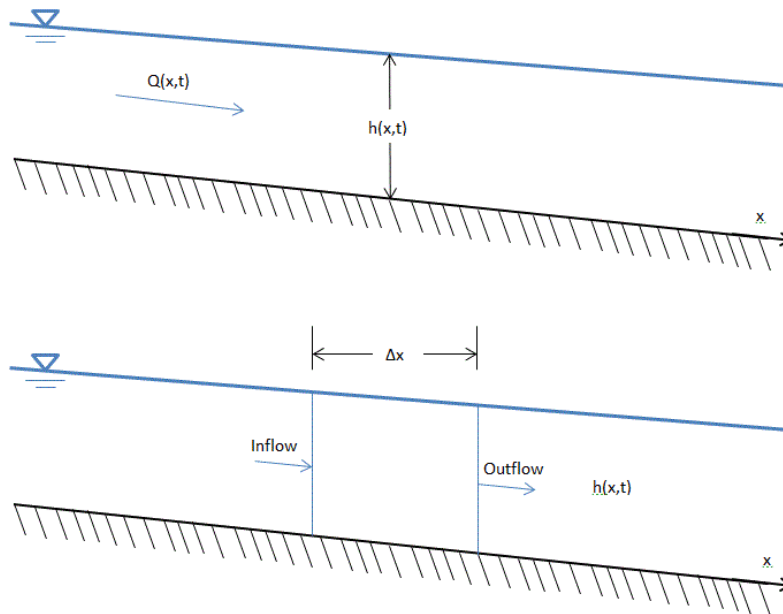


Figure 5.1: Elementary Control Volume for the Saint-Venant Equations
(Sources: USACE, 2010a)

The continuity equation describes the conservation of mass in a control volume. For one-dimension system, the continuity equation, with the addition of a storage term, S , can be written as (USACE, 2010a):

$$\frac{\partial A}{\partial t} + \frac{\partial S}{\partial t} + \frac{\partial Q}{\partial x} - q = 0 \quad (5.14)$$

where

x is the longitudinal distance along the channel;

t is the time;

Q is the flow rate;

q is the lateral inflow per unit distance;

S is the storage from non-conveying portions of cross section;

A is the cross-sectional area of flow;

The above equation can also be written for channel and floodplain:

$$\frac{\partial Q_c}{\partial x_c} + \frac{\partial A_c}{\partial t} = q_f \quad (5.15)$$

and

$$\frac{\partial Q_f}{\partial x_f} + \frac{\partial A_f}{\partial t} + \frac{\partial S}{\partial t} = q_c + q_l \quad (5.16)$$

where the subscripts c and f refer to channel and floodplain, respectively. q_l is the lateral inflow per unit length of floodplain; q_c and q_f are exchanges of water between the channel and the floodplain.

The momentum equation states that the momentum rate of change in the control volume is equal to the sum of the external forces acting on the control surface of the system. On form of the momentum equation can be written as:

$$\frac{\partial Q}{\partial t} + \frac{\partial(VQ)}{\partial x} + gA\left(\frac{\partial z}{\partial x} + S_f\right) = 0 \quad (5.17)$$

where

V is the velocity of lateral inflow in x-direction;

z is the water surface elevation in the channel;

S_f is the slope of the energy grade line;

g is the acceleration of gravity.

Similar to the continuity equation, the momentum equation can also be written for channel and floodplain:

$$\frac{\partial Q_c}{\partial t} + \frac{\partial(V_c Q_c)}{\partial x_c} + gA_c\left(\frac{\partial z}{\partial x_c} + S_{fc}\right) = M_f \quad (5.18)$$

and

$$\frac{\partial Q_f}{\partial t} + \frac{\partial(V_f Q_f)}{\partial x_f} + gA_f\left(\frac{\partial z}{\partial x_f} + S_{ff}\right) = M_c \quad (5.19)$$

where the M_c and M_f are the momentum fluxes per unit distance exchanged between channel and floodplain, respectively.

The continuity and momentum equations are nonlinear higher-order partial differential equations. The solution may be obtained analytically by using the characteristics equations. HEC-RAS solves the *Saint-Venant equations* numerically using the four-point implicit scheme for the finite different approximations, also known as the *box scheme* (USACE, 2010a). Figure 5.2 illustrates the solution cell on the space-time ($x-t$) plane used for numerical solution of the *Saint-Venant equations* by the finite-different method. Implicit finite-different methods advance the solution of the *Saint-Venant equations* from one time line to the next simultaneously for all points along the time line. Applying the Saint-Venant equations simultaneously to all known values on a time line generates a system of algebraic equations. Implicit methods were developed because of the limitation on the time-step size required for numerical stability of the explicit methods. The implicit finite-different scheme uses a weighted four-point method between adjacent lines at a point M, as illustrated in Figure 5.2. The general implicit finite different forms for time derivative, $\frac{\partial f}{\partial t}$, and spatial derivative, $\frac{\partial f}{\partial x}$, for a function, f , are:

$$\frac{\partial f}{\partial t} \approx \frac{\Delta f}{\Delta t} = \frac{f_i^{j+1} + f_{i+1}^{j+1} - f_i^j - f_{i+1}^j}{2\Delta t} \quad (5.20)$$

and

$$\frac{\partial f}{\partial x} \approx \frac{\Delta f}{\Delta x} = \theta \frac{f_{i+1}^{j+1} - f_i^{j+1}}{\Delta x} + (1-\theta) \frac{f_{i+1}^j - f_i^j}{\Delta x} \quad (5.21)$$

The average function value for f is calculated as:

$$f \approx \bar{f} = \theta \frac{f_i^{j+1} + f_{i+1}^{j+1}}{2} + (1-\theta) \frac{f_i^j - f_{i+1}^j}{2} \quad (5.22)$$

For the *box scheme*, θ value is set to be 0.5.

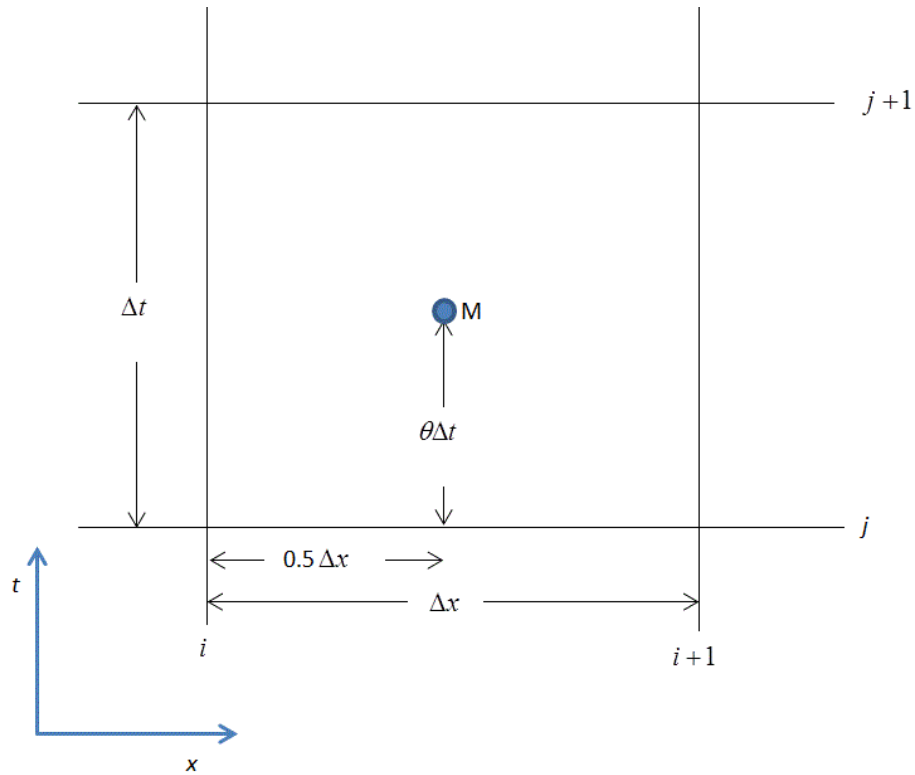


Figure 5.2: Solution Cell on the Space-Time (x - t) Plane used for Numerical Solution of the Saint-Venant Equations by the Four-Point Finite-Different Method (Chow, Maidment, and Mays, 1988)

The continuity equations for both channel and floodplain flow, as described in Equations (5.15) and (5.16), can now be approximated using the four-point implicit scheme for finite different method:

$$\frac{\Delta Q_c}{\Delta x_c} + \frac{\Delta A_c}{\Delta t} = \bar{q}_f \quad (5.23)$$

and

$$\frac{\Delta Q_f}{\Delta x_f} + \frac{\Delta A_f}{\Delta t} + \frac{\Delta S}{\Delta t} = \bar{q}_c + \bar{q}_l \quad (5.24)$$

Adding the above equations and rearranging yields:

$$\Delta Q + \frac{\Delta A_c}{\Delta t} \Delta x_c + \frac{\Delta A_f}{\Delta t} \Delta x_f + \frac{\Delta S}{\Delta t} \Delta x_f - \bar{Q}_l = 0 \quad (5.25)$$

where \bar{Q}_l is the average lateral inflow.

The momentum equations for both channel and floodplain flow can also be approximated using the four-point implicit finite difference method:

$$\frac{\Delta Q_c}{\Delta t} + \frac{\Delta(V_c Q_c)}{\Delta x_c} + g \bar{A}_c \left(\frac{\Delta z}{\Delta x_c} + \bar{S}_{fc} \right) = M_f \quad (5.26)$$

and

$$\frac{\Delta Q_f}{\Delta t} + \frac{\Delta(V_f Q_f)}{\Delta x_f} + g \bar{A}_f \left(\frac{\Delta z}{\Delta x_f} + \bar{S}_{ff} \right) = M_c \quad (5.27)$$

Note the momentum fluxes per unit length exchanged between channel and floodplain is the same for both ways, therefore, $\Delta x_c M_c = -\Delta x_f M_f$. Adding the Equations (5.26) and (5.27) together and rearranging yields:

$$\begin{aligned} & \frac{\Delta(Q_c \Delta x_c + Q_f \Delta x_f)}{\Delta t} + \Delta(V_c Q_c) + \Delta(V_f Q_f) + g(\bar{A}_c + \bar{A}_f) \Delta z \\ & + g \bar{A}_c \bar{S}_{fc} \Delta x_c + g \bar{A}_f \bar{S}_{ff} \Delta x_f = 0 \end{aligned} \quad (5.28)$$

The last two terms define the friction force from the banks acting on the fluid. Thus, the final form of the momentum equation is:

$$\frac{\Delta(Q_c \Delta x_c + Q_f \Delta x_f)}{\Delta t} + \Delta(\beta V Q) + g \bar{A} \Delta z + g \bar{A} \bar{S}_f \Delta x_e = 0 \quad (5.29)$$

where

$g \bar{A} \bar{S}_f \Delta x_e$ is the equivalent force ($g \bar{A} \bar{S}_f \Delta x_e = g \bar{A}_c \bar{S}_{fc} \Delta x_c + g \bar{A} \bar{S}_{ff} \Delta x_c$);

Δx_c is the equivalent flow path;

\bar{S}_f is the friction slope for the entire cross-section;

\bar{A} is the sum of \bar{A}_c and \bar{A}_f ;

β is the momentum correction factor

In the optimization/simulation model, HEC-RAS uses the watershed hydrograph generated from HEC-HMS as input, then apply the four-point implicit scheme for the finite different approximations to solve Equations (5.25) and (5.26) simultaneously for the flow rate and water surface elevation for a river system. The output of HEC-RAS, specifically the flow rate just upstream of the reservoir, would become the input of the third component of the optimization/simulation model, the spillway-gate operation model.

CHAPTER 6 –RESERVOIR OPERATION MODEL

6.1 Reservoir Operation Model Based on Mass Balance

The third component of the optimization/simulation model is the spillway-gate operation model. The spillway-gate operation model is based on the principle of conservation of mass for a control volume. The conservation of mass states that:

$$\left\{ \begin{array}{l} \text{The accumulation rate of} \\ \text{mass in the control volume} \end{array} \right\} + \left\{ \begin{array}{l} \text{Net outflow of mass} \\ \text{through the control volume} \end{array} \right\} = 0$$

In mathematical form, the conservation of mass for a control volume can be written as:

$$\frac{d}{dt} m_{CV} + \sum_{CS} \dot{m}_{out} - \sum_{CS} \dot{m}_{in} = 0 \quad (6.1)$$

where

$\frac{d}{dt} m_{CV}$ is the accumulation of mass in the control volume

$\sum_{CS} \dot{m}_{in}$ is the total inflow of mass through the control surface

$\sum_{CS} \dot{m}_{out}$ is the total outflow of mass through the control surface

Equation (6.1) can be modified to show the volume flow rate instead of the mass flow rate as shown below:

$$\frac{1}{\rho} \frac{d}{dt} m_{CV} + \frac{1}{\rho} \sum_{CS} \dot{m}_{out} - \frac{1}{\rho} \sum_{CS} \dot{m}_{in} = 0 \quad (6.2)$$

which simplifies to

$$\frac{d}{dt}V_{CV} + \sum_{CS} Q_{out} - \sum_{CS} Q_{in} = 0 \quad (6.3)$$

where

$\frac{d}{dt}V_{CV}$ is the volume changes in the control volume

$\sum_{CS} Q_{in}$ is the total volumetric inflow through the control surface

$\sum_{CS} Q_{out}$ is the total volumetric outflow through the control surface

Equation (6.3) is the basis of the spillway-gate operation model. Figure 6.1 shows the schematic of a reservoir with components of flows and reservoir storage

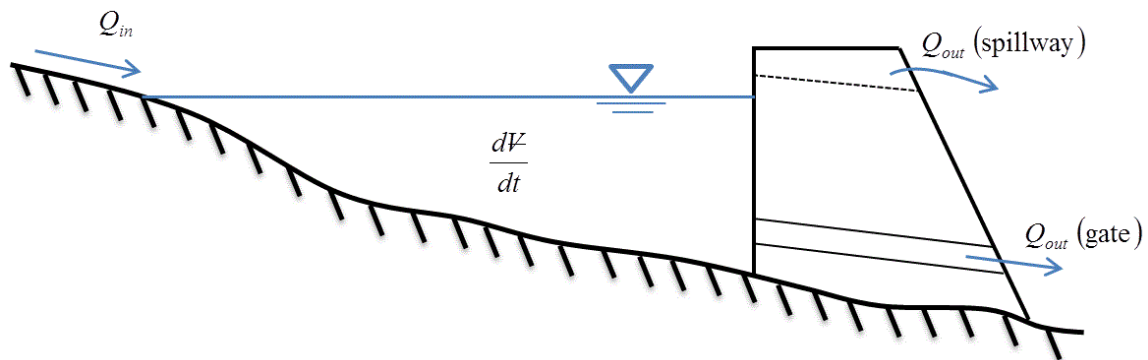


Figure 6.1: Reservoir Inflow, Outflow, and Storage

During flooding condition, the spillway-gate operation model is couple with an optimization model to determine the gate release from reservoir gates. The reason of

coupling with the optimization model is that, the optimization process can help determine the gate release schedule such that the flow rate or flood stage at the downstream target location are under desired values. An optimization procedure based upon a genetic algorithm (GA) optimizer interfaces the other component of the model to determine actual gate operations during the real-time operation of the reservoir systems. The optimization model is the next major component of the optimization/simulation model, which it's complete formulation is explained next.

CHAPTER 7 – MATHEMATICAL MODEL FORMULATION

7.1 Problem Statement

The theoretical general optimization/simulation problem for the releases of river-reservoir system under flooding conditions can be written as follows:

To determine the releases of reservoirs in a river-reservoir system which:

minimize the total flood damage,

subject to following constraints

- (a) hydrologic constraints such as precipitation-watershed runoff relationships solved by HEC-HMS,
- (b) unsteady flow equations and other relationships which describes the flow in various components in a river-reservoir system solved by HEC-RAS,
- (c) maximum and minimum allowable reservoir releases and flow rates at specified locations,
- (d) maximum and minimum allowable water surface elevations at specified location in the system, which includes reservoir surface elevations.
- (d) rules of operation, targets, storage capacities, limitations.

The simulation problem is described by the first and second constraint of the optimization/simulation model, for example, solution of the Saint-Venant equations and

other flow relationships that apply to other regulating structures for a given reservoir policy. The variables used in the optimization/simulation model are:

- water surface elevations at all computational points;
- discharges values at all computational points;
- reservoir releases at all reservoirs.

A mathematical statement of the optimization/simulation problem for the operation of river-reservoir system, based on the contents described above, can be stated as follows:

7.2 Objective Functions

$$Min Z = \sum_{i=1}^I \sum_{t=1}^T [C_i Q_{i,t}] \quad (7.1.a)$$

where $Q_{i,t}$ is the time-series flow rate vectors of control points, i , in the river-reservoir system. The objective is to minimize the flow rates at all control points i at all-time t . The coefficient C_i is the penalty coefficient at control point i . i and t are location and time indices, respectively. Alternatively, the objective function can be written as follow:

$$Min Z = Max[h_{i,t}] \quad (7.1.b)$$

The objective is to minimize the peak flow stage at all control points i at all-time t . By setting the objective function to minimize the peak flow stage can relax the upstream reservoir loads, as compared to first objective function that tend to fill the reservoirs.

7.3 Constraints

Generally, the constraints for the optimization/simulation model can be categorized to five main types:

a) Hydrologic constraints are defined by the rainfall-runoff relationships such as subbasins areas, rainfall losses due to canopy interceptions, depression storage and soil infiltration, effective rainfall transform methods, watershed runoff routing methods, internal boundary conditions and initial conditions that depict the rainfall-runoff process in different components of a watershed system,

$$h(P_{i,t}, H_{i,t}, Q_{i,t}) = 0 \quad (7.2)$$

where $P_{i,t}$ is the matrix of precipitation data in the system; $H_{i,t}$ is the rainfall losses matrix of the watershed system; $Q_{i,t}$ is the watershed and reaches discharge matrix of the system. All the hydrologic constraints are in matrix form because the problem has dimension of space, i , and time, t .

b) Hydraulic constraints are defined by the *Saint-Venant equations* for one-dimensional gradually varied unsteady open channel flow (see Equations 1.1 and 1.2), and other relationships such as upstream condition, downstream condition, internal boundary conditions and initial conditions that depict the flow in different components of a river-reservoir system,

$$g(h_{i,t}, Q_{i,t}) = 0 \quad (7.3)$$

where $h_{i,t}$ is the matrix of water surface elevations in the system; $Q_{i,t}$ is the discharge matrix of the system. All the hydraulic constraints are in matrix form is because of the problem has dimension of space, i , and time, t .

c) Bounds on discharges defined by the maximum and minimum allowable reservoir releases and flow rates at target location:

$$\underline{Q}_i \leq Q_{i,t} \leq \bar{Q}_i \quad (7.4)$$

The bars above and underneath the variable denote the upper limit and lower limit for that variable, respectively.

d) Water surface elevation bounds defined by the allowable the upper limit and lower limit at specified locations, including reservoir levels:

$$\underline{h}_i \leq h_{i,t} \leq \bar{h}_i \quad (7.5)$$

e) Other types of constraints such as rules of reservoir releases, target storages, reservoir storage capacities, etc., are also necessary to be included in the optimization/simulation model:

$$W(Q_{i,t}, h_{i,t}) \leq 0 \quad (7.6)$$

The constraints of the optimization/simulation model can be categorized into two types: i) *hydrologic-hydraulics constraints* (Eqs. 7.2-7.4), and ii) *operational constraints* (Eqs. 7.5-7.6).

7.4 Solution Approach of the Optimization/Simulation Model

This section illustrates the development of an optimization/simulation model for determining reservoir release schedules before, during, and after an extreme flood event in a real time fashion, which keeps the floodwater flows and flood elevations under desired target levels. The problem is formulated as a real time optimal control problem in which reservoir releases represent the decision variables. The known real-time rainfall input is used as the actual rainfall up to the time of decision-making. Also the model will generate short-term forecast precipitation and floods using real-time rainfall from a rainfall network of gages and measured real-time flood elevations in a river-reservoir system. A methodology of projecting future rainfall within the next few minutes to hours will be developed as feature of the methodology. Forecasted rainfall is used to run simulations of the watershed rainfall-runoff model, HEC-HMS, and then the hydrographs are used as inputs of the optimization model to determine the releases of the reservoirs in a river reservoir system.

Once the sets of feasible (or optimal) solutions (i.e. reservoirs releases) are determined, the decision variables are inputted into the unsteady flow routing model HEC-RAS to simulate the floods in the river-reservoir system. In this proposal, the main objective of the methodology is to control the flood flows and flood elevations at various locations of a river-reservoir system. One example might be to keep the flowrates or flood elevations at the control point below a certain target, for example, a 100-year flood level. If the objective were not met, the model would repeat its optimization process to determine the reservoir releases until the objective is achieved. Once this occurs, the

model moves to the next iteration, in which the projected rainfall is used to run the precipitation-runoff model (HEC-HMS) to determine the reservoirs operation for the next time period. The actual rainfall data is then used to compute the actual watershed runoff, reservoirs stages, releases of reservoirs, and the unsteady flows. The processes repeat and continue until the objective is met and all constraints are satisfied for the entire simulation period. The reason for the model could be used to start simulating prior to the storm events is that, the model can determine the necessary actions to ensure the flooding condition is minimized.

Figure 7.1 illustrates the basic steps of the optimization/simulation model algorithm. First, the model requires real-time rainfall data (i.e. NEXRAD data) to start the rainfall-runoff simulation; once the watershed hydrographs are obtained then they are entered into the unsteady flow simulation as inputs. Once the floodwaters are routed to the locations where the reservoirs are located, the flow data enters into an optimization model to compute the real-time operation decisions (releases) of reservoirs. The model would then generate the initial reservoir operation. Once the optimization model determines the decisions, the flows released from the reservoir are entered into again the unsteady flow model to simulate the flow to further downstream locations. When the floodwater enters the location of interest, or the target location, the model determines whether the objective is met, for example, are the water levels controlled at target location?

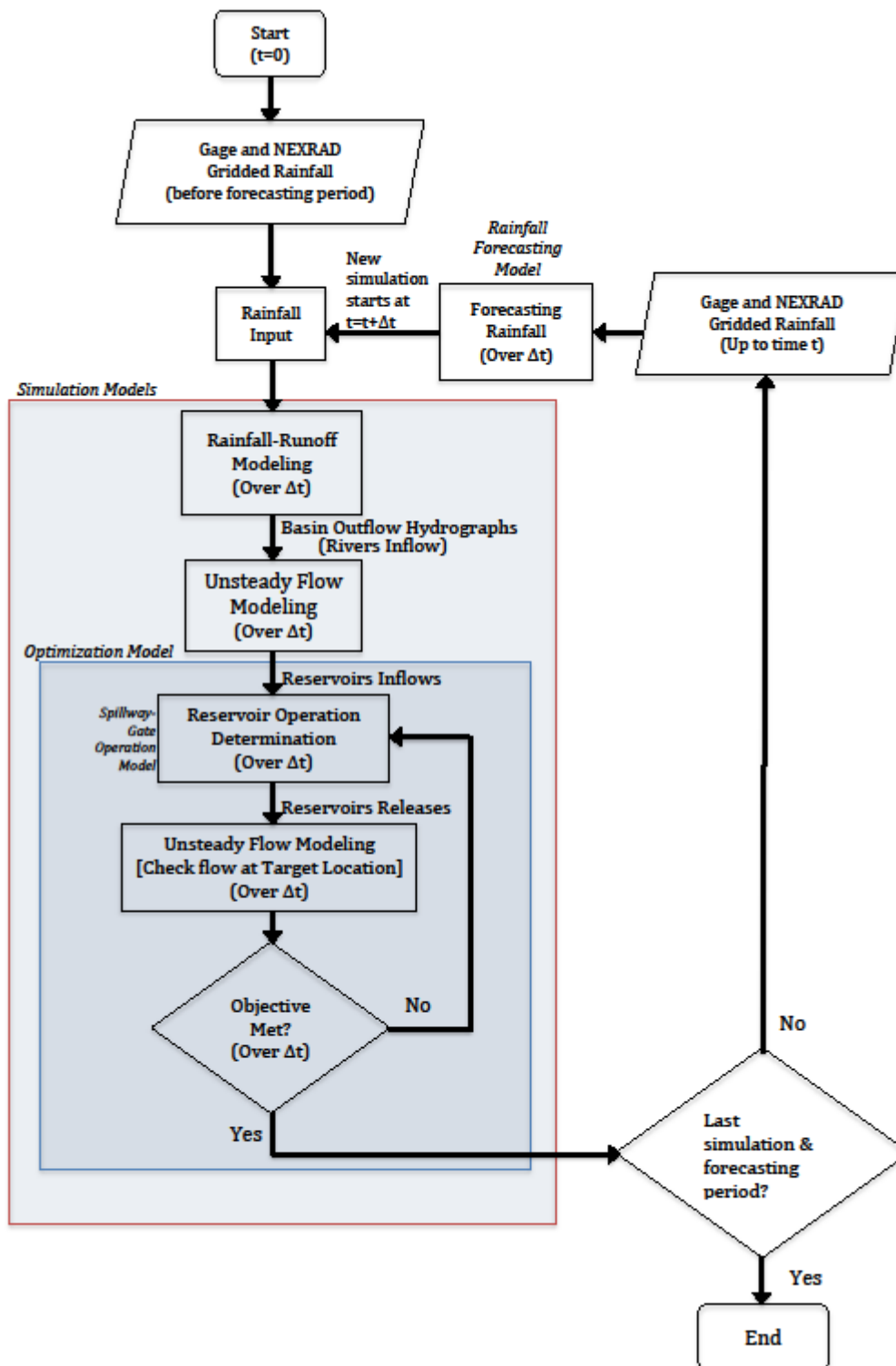


Figure 7.1: Basic Steps of the Optimization/Simulation Model

Are the water levels under the desirable level? If the answer is no, the model returns to the reservoir operation optimization process to determine an improved reservoir operation. When the objective is met, the model will repeat the overall optimization/simulation processes for the next simulation period (over the next Δt) if the period is not the last. Next, the model enters the process of projecting rainfall. The rainfall forecasting model generates forecast rainfall over the Δt , and the model uses the known rainfall up to current simulation time, t . Once the forecasted rainfall is generated, the model moves to the next iteration that returns to the simulation process starting with hydrologic simulation. The optimization/simulation model continues until the very last simulation period, and then the model stops. Figure 7.2 shows the components of the real-time river reservoir system operation model.

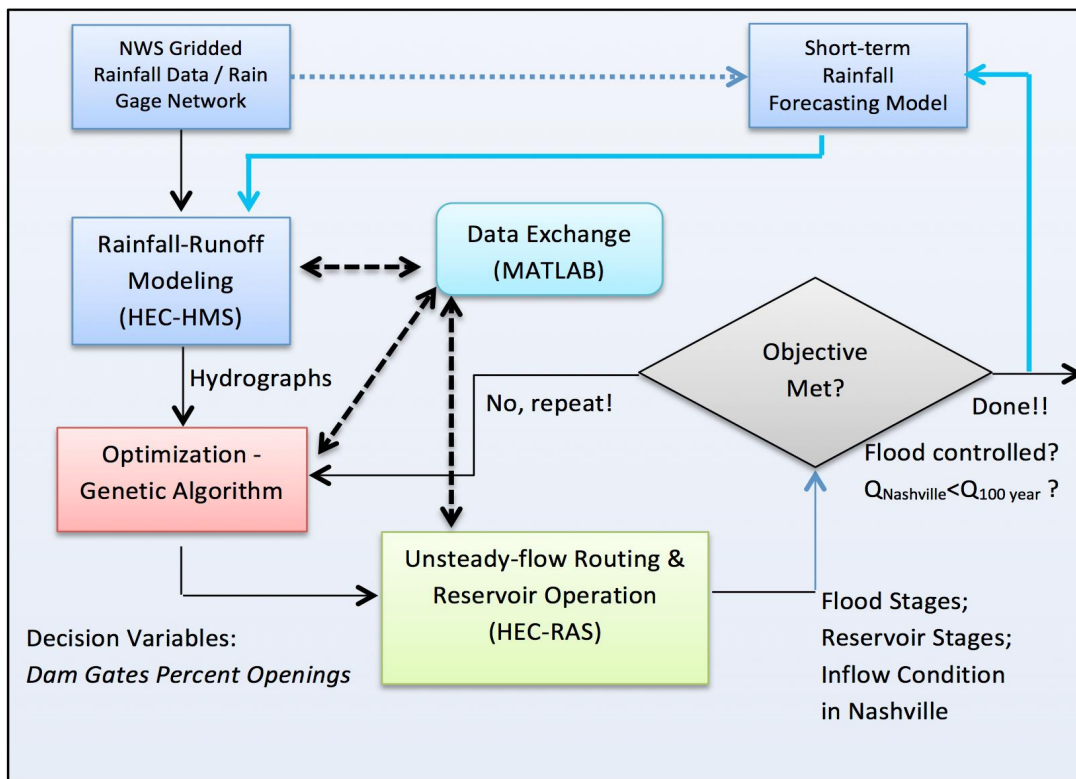


Figure 7.2: Interconnection of Components

The model developed herein uses MATLAB to communicate between HEC-HMS, the reservoir operation model, and HEC-RAS. Once the watershed hydrographs are computed by HEC-HMS, MATLAB receives the data and send the data to HEC-RAS as river input. The HEC-RAS would perform routing of flood water to the reservoir locations. Once the floodwaters data arrives at the reservoirs, the data then is sent to the reservoir operation model to determine the decision of the operations. The decision made here is based on the flowrate or flood stage at control points that are downstream of the reservoir. The “blue box” in Figure 7.1 shows the subroutine of the optimization process, and it is further explained in Figure 7.3.

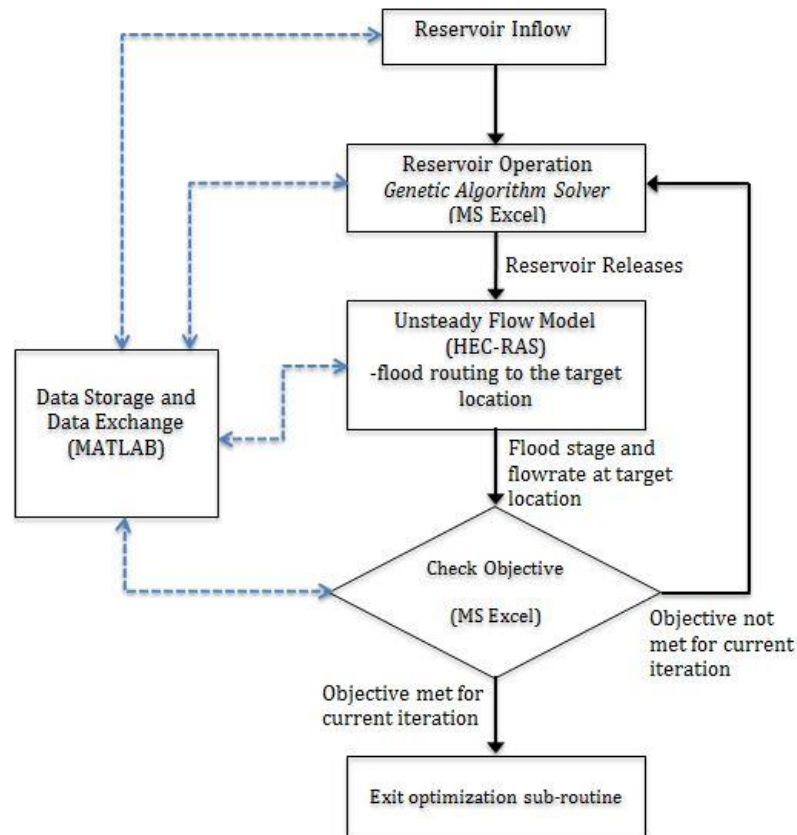


Figure 7.3: Optimization Sub-Routine (over Δt) Flowchart

Once the reservoir operation model has made a set of decisions for the releases, the reservoir releases data is then send back to HEC-RAS by MATLAB continuing to unsteady flow routing to control locations. The model now would determine whether the objective has been met. If the objective is met, then model exits the optimization subroutine, and start the next forecast simulation. If the objective is not met, the model would repeat the optimization subroutine as shown in Figure 7.3.

The next component of the optimization/simulation model is the reservoir operation model. The reservoir operation model is based on the principle of conservation of mass for a control volume. The conservation of mass states that:

$$\left\{ \begin{array}{l} \text{The accumulation rate of} \\ \text{mass in the control vloume} \end{array} \right\} + \left\{ \begin{array}{l} \text{Net outflow of mass} \\ \text{through the control surface} \end{array} \right\} = 0$$

In mathematical form, the conservation of mass of incompressible flow for a control volume can be simplified as volumetric flow form:

$$\frac{d}{dt} V_{CV} + \sum_{CS} Q_{out} - \sum_{CS} Q_{in} = 0 \quad (7.7)$$

where

$\frac{d}{dt} V_{CV}$ is the volume changes in the control volume

$\sum_{CS} Q_{in}$ is the total volumetric inflow through the control surface

$\sum_{CS} Q_{out}$ is the total volumetric outflow through the control surface

Equation 7.7 is the basis of the reservoir operation model.

During flooding condition, the reservoir operation model is coupled with an optimization model to determine the releases from reservoirs. The decision variables are the reservoir releases over Δt . The reason of coupling with the optimization model is that, the optimization process can help determine the proper reservoir release schedule such that the flow rate or flood stage at the downstream target location are under desired values. An optimization procedure based upon a genetic algorithm (GA) optimizer interfaces with the other component of the model to determine reservoir operations is discussed earlier and shown in Figure 7.3.

7.5 Optimization Model

The optimization method used in this study is genetic algorithm (GA). Unlike classical optimization search methods, for examples, like the simplex method and gradient-based methods, genetic algorithm does not necessary required a well-defined function. Genetic algorithm, developed in the 1970s, is a model or abstraction of biological evolution based on Charles Darwin's theory of natural selection. Even though there is no universal definition, genetic algorithm generally consists of three operators: 1) selection, 2) crossover (mating), and 3) mutation. Genetic algorithm is used here because of the advantages over traditional optimization algorithms, specifically in this research, the ability of dealing with complex optimization problems and parallelism (Holland 1975, Goldberg 1989, Mitchell 1996, Deb 2001, and Yang 2010). Nicklow et al (2010) discussed the use of GA in water resources engineering. The general procedure of genetic algorithm is illustrated in Figure 7.4. In the optimization/simulation model, the

population in GA is the reservoir releases over Δt ; fitness is the objective function, which is the flow rate and flood stage over Δt at the target location downstream of reservoirs.

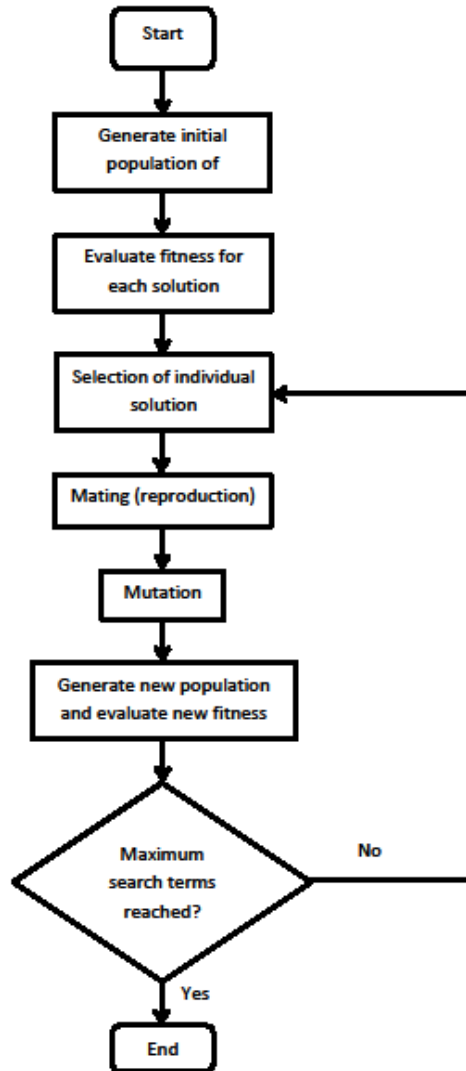


Figure 7.4: General Procedure of Genetic Algorithm

7.6 Rainfall Forecasting

7.6.1 General Approach

Rainfall forecasting is another major component in the optimization/simulation model. As an example discussed earlier in the Chapter One, observed precipitation were used for rainfall forecasting during the flood event at Kanawha Falls, West Virginia in March 1967. Forecasted precipitation is needed for flood forecasting since reservoir management personnel would have to make reservoir releases decision based upon the forecasted information prior to the actual rainfall event and floodwater arrive. In this study, a statistical regression analysis approach is used for the rainfall forecasting model. A *model* for forecasting observations in time period $t+\Delta t$ that can make at the end of the current time period t can be written as (Montgomery et al., 2012):

$$\hat{P}_{t+\Delta t} = \hat{\phi}P_t + (1-\hat{\phi})\hat{\beta}_0 + \hat{\beta}_1[(t+\Delta t)-\hat{\phi}t] \quad (7.8)$$

where

$\hat{P}_{t+\Delta t}$ is the vector of predicted rainfall values over time period $t+\Delta t$

P_t is the vector of known rainfall values at the end of the current time period, t

$t+\Delta t$ is the forecasting time period

t is the current time period

ϕ is the autocorrelation parameter, defined as $\phi = \frac{\sum_{t=2}^t e_t e_{t-1}}{\sum_{t=1}^t e_t^2}$

e_t is the vector of residuals from the prediction,

$\hat{\beta}_1, \hat{\beta}_2$ are the model parameters

The general procedure of rainfall forecasting model is illustrated in Figure 7.5 where P_t in this model is the rainfall data, and t is the time. First, the model obtains the actual rainfall up to current time, t . Then the rain data is entered into the step that the prediction model (Equation 7.8) is generated over $t+\Delta t$. Once the prediction model of time period $t+\Delta t$ is generated, the prediction model is used to make rainfall forecast over Δt . After obtaining the projected rainfall, this data will exit the rainfall forecasting sub-routine and is entered into the optimization/simulation model as illustrated in Figure 7.1. When the last simulation period, t , ends, the forecasting model repeats the process by obtaining the actual rainfall up to current time, t . A new prediction model will be generated for each simulation time step, Δt , therefore, each of the prediction models is unique for each forecasting period. The process repeats until the very last simulation period when forecasting is no longer needed.

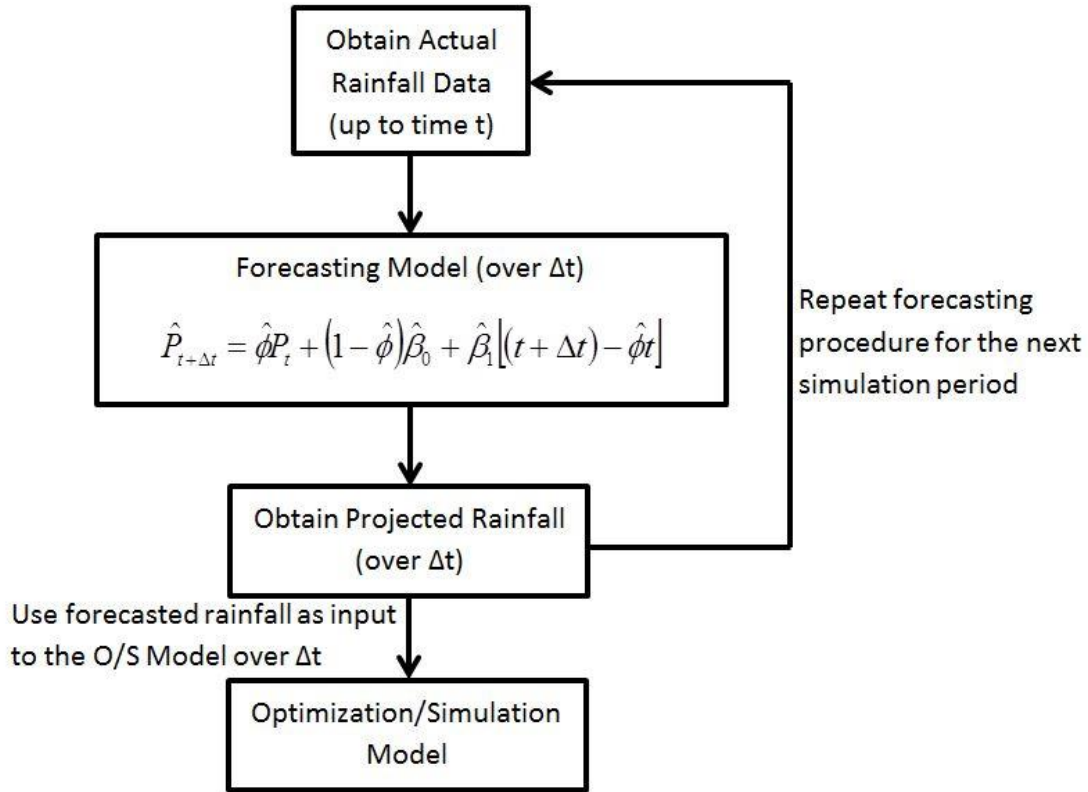


Figure 7.5: General Procedure of the Rainfall Forecasting Model

7.6.2 Comparison of Proposed Forecasting Models

Four time series forecasting models are suggested here for the optimization/simulation model, they are: autoregressive model (AR), autoregressive exogenous model (ARX), autoregressive moving average exogenous model (ARMAX), and the state-space estimation model (SSEST). These four specific models are proposed because of the convenience in the MATLAB built-in control environment. A generated hypothetical rainfall hyetograph (see Figure 7.6) is used for the comparison of the four time series forecasting models. The time span is set to be 72 hours, and the forecasting starts at $t = 7$ hour.

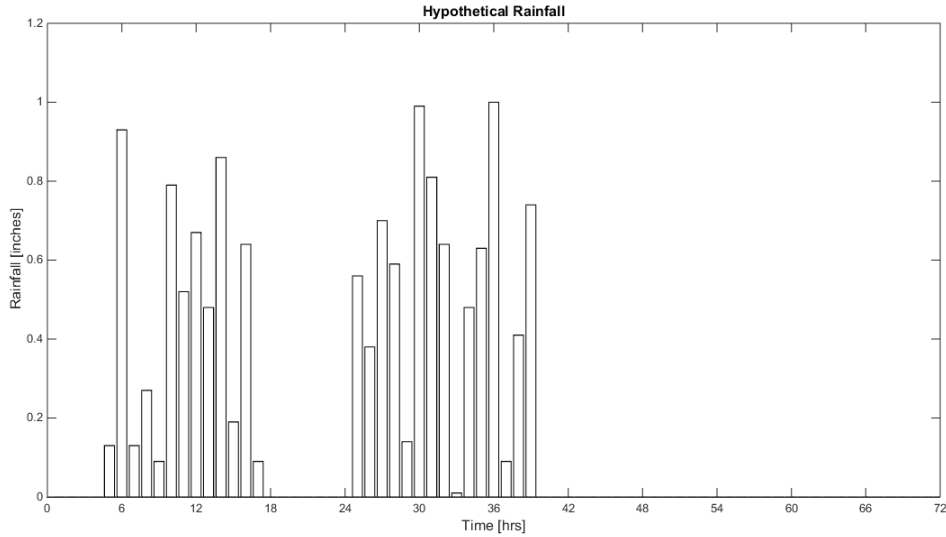


Figure 7.6: Hypothetical Rainfall

The autoregressive model (AR) is a stochastic process for time series that gives the output variable depends linearly on its own previous values (Diebold, 2006). The formula of an N^{th} order autoregressive polynomial model for time series of rainfall \hat{P}_t is presented as follows:

$$P_{t+\Delta t} + a_1 P_t + a_2 P_{t-\Delta t} + a_3 P_{t-2\Delta t} \dots + a_N P_{t-(N-1)\Delta t} = e_{t+\Delta t} \quad (7.9)$$

where

$P_{t+\Delta t}$ is the forecast rainfall values over time period $t+\Delta t$

P_t is the known rainfall values at the end of the current time period, t

a_N are the model parameters which depend on the time series pattern

$t + \Delta t$ is the time of current forecasting

t is the current time period

$e_{t+\Delta t}$ is the white noise from the forecast

The advantages of the autoregressive models are they are remarkably flexible in handling a wide range of different time series patterns. Figure 7.7 depicts the forecasting result based on the hypothetical rainfall hietograph. The algorithm follows the processes shown in flow chart on Figure 7.5.

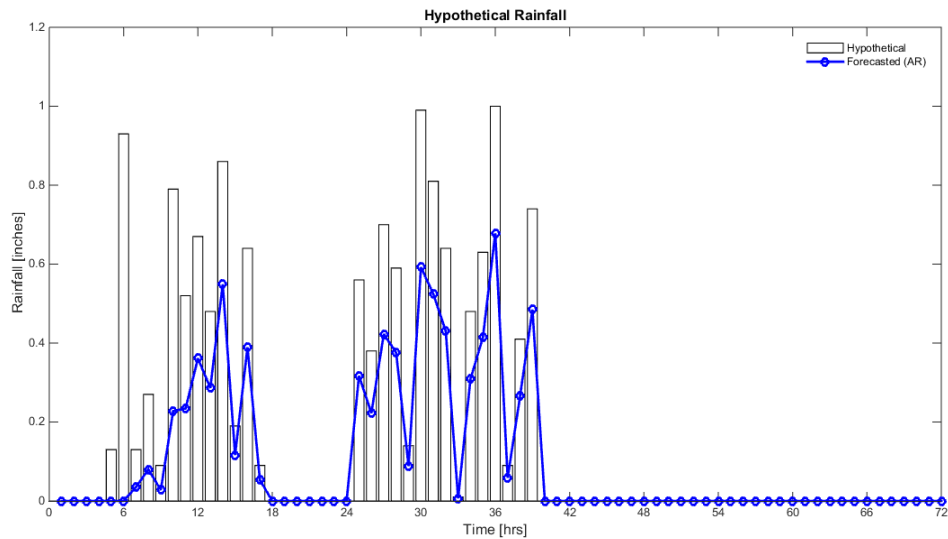


Figure 7.7: Forecasting Result of the AR Model

The autoregressive exogenous model (ARX) uses the same concept of the AR model which uses previous values which are linearly related, but incorporating exogenous variables which also depend on previous values (Diebold, 2006). The basic formulation of an ARX model is as follow:

$$P_{t+\Delta t} + a_1 P_t + a_2 P_{t-\Delta t} + a_3 P_{t-2\Delta t} \dots + a_N P_{t-(N-1)\Delta t} = b_1 u_t + b_2 u_{t-\Delta t} + b_3 u_{t-2\Delta t} \dots + b_N u_{t-(N-1)\Delta t} + e_{t+\Delta t} \tag{7.10}$$

where

$P_{t+\Delta t}$ is the forecast rainfall values over time period $t+\Delta t$

u_t is the exogenous values at the end of the current time period, t

a_N, b_N are the model parameters

$e_{t+\Delta t}$ is the white noise from the forecast

Since the rainfall is purely hypothetical, the exogenous variable used here is the cumulative rainfall up to time t . Figure 7.8 depicts the forecasting result based on the hypothetical rainfall hyetograph. The algorithm follows the processes shown in flow chart on Figure 7.5.

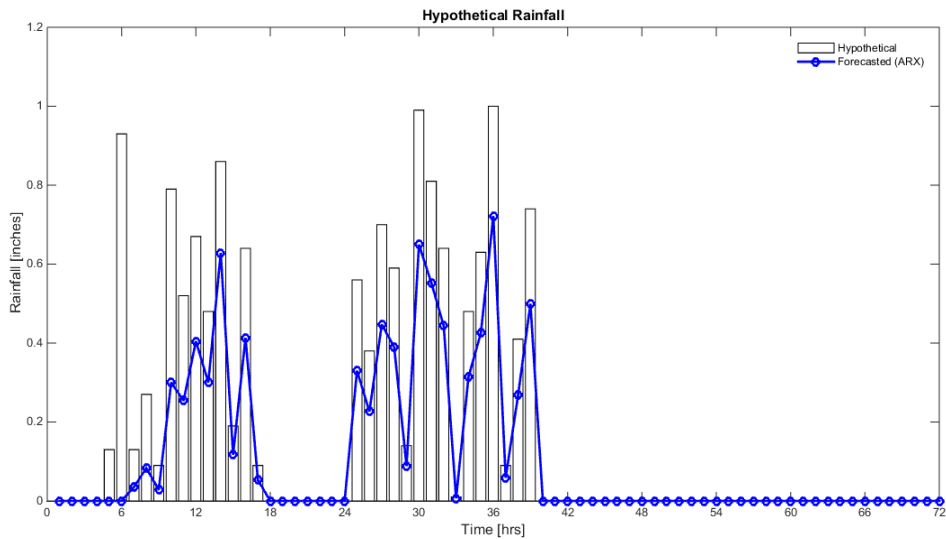


Figure 7.8: Forecasting Result of the ARX Model

The autoregressive moving average exogenous model (ARMAX) incorporates the autoregressive portion and exogenous variable, which are previously defined, and also the component of the moving average, or simply the past forecast error (Diebold, 2006).

The basic formulation of an ARMAX model is as follow:

$$\begin{aligned}
 P_{t+\Delta t} + a_1 P_t + a_2 P_{t-\Delta t} + a_3 P_{t-2\Delta t} \dots + a_N P_{t-(N-1)\Delta t} = \\
 b_1 u_t + b_2 u_{t-\Delta t} + b_3 u_{t-2\Delta t} \dots + b_N u_{t-(N-1)\Delta t} + \\
 c_1 e_t + c_2 e_{t-\Delta t} + c_3 e_{t-2\Delta t} \dots + c_N e_{t-(N-1)\Delta t} + e_{t+\Delta t}
 \end{aligned}
 \tag{7.11}$$

where

$P_{t+\Delta t}$ is the forecast rainfall values over time period $t+\Delta t$

u_t is the exogenous values at the end of the current time period, t

e_t is the forecast error at the time, t

a_N, b_N, c_N are the model parameters

$e_{t+\Delta t}$ is the white noise from the forecast at time $t+\Delta t$

Figure 7.9 depicts the forecasting result based on the hypothetical rainfall hyetograph.

The algorithm follows the processes shown in flow chart on Figure 7.5.

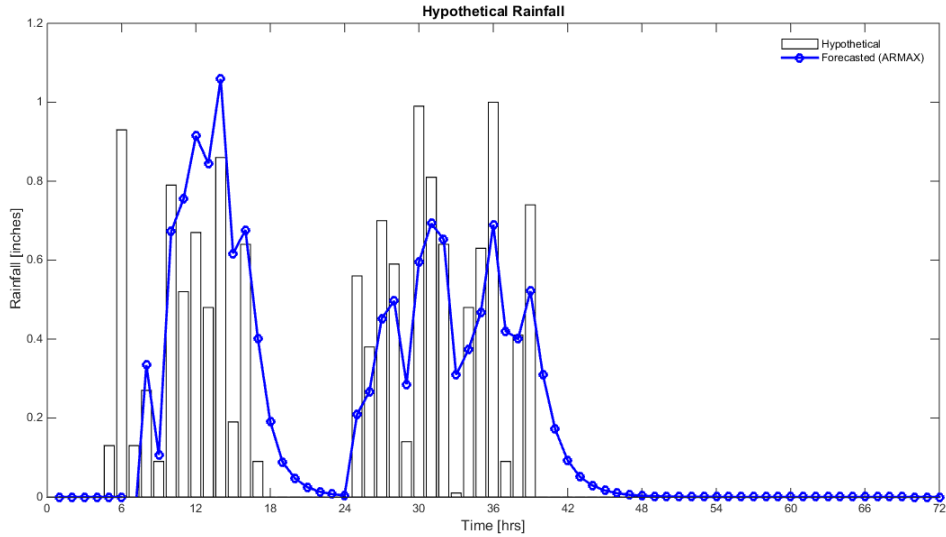


Figure 7.9: Forecasting Result of the ARMAX Model

The state-space estimation model (SSEST) is a mathematical model of a physical as a set of input, output, and state variables related by ordinary first-order differential equations (Ljung, 1999). The SSEST model is often used in system control engineering. The followings are the basic formulation of the SSEST model:

$$\begin{aligned} \frac{dx}{dt} &= Ax_t + Bu_t + Ke_t \\ P_t &= Cx_t + Du_t + e_t \end{aligned} \quad (7.12)$$

where

P_t is the model output variable (rainfall values)

u_t is the model input variable (time)

x_t is the model state variable (average)

e_t is the model disturbance (forecast error)

A, B, C, K are the model parameters

Figure 7.9 depicts the forecasting result based on the hypothetical rainfall hietograph.

The algorithm follows the processes shown in flow chart on Figure 7.5.

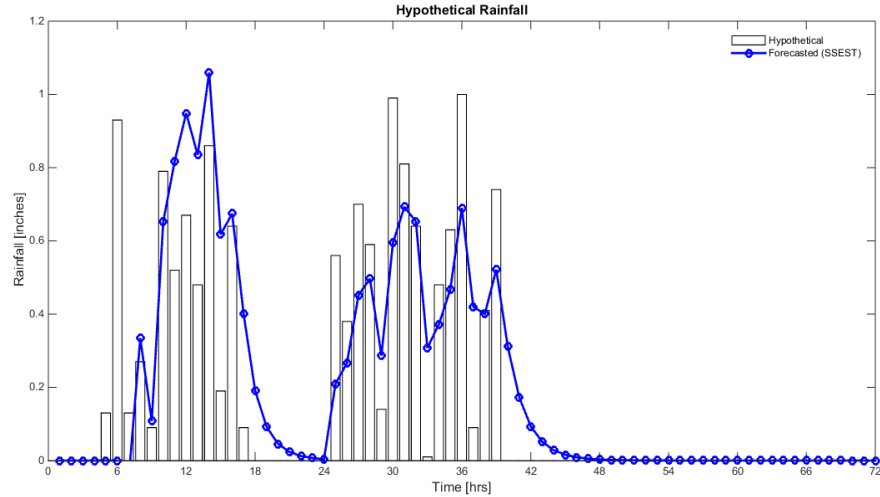


Figure 7.10: Forecasting Result of the SSEST Model

Three metrics are used for the comparison of the four models: (1) the cumulative forecasting error (CFE), (2) the root mean squared error (RMSE), and (3) the computational time per forecasting period (per iteration).

The cumulative forecasting error (CFE) calculates the percent difference between the actual cumulative rainfall and the forecast cumulative rainfall. The formulation is as follow:

$$CFE = \frac{\left(\sum_{\tau} P_t \right) - \left(\sum_{\tau} \hat{P}_t \right)}{\left(\sum_{\tau} P_t \right)}, \forall t \in \tau \quad (7.13)$$

where if CFE $\begin{cases} > 0, \text{ under forecast} \\ < 0, \text{ over forecast} \end{cases}$

P_t = hypothetical rainfall up to time t

\hat{P}_t = forecasted rainfall up to time t

τ = total forecasting period

The CFE is a way to measure the performance of the forecasting model in terms of “quantity”. Ideally, the small CFE is desirable since large deviation in cumulative forecast rainfall would create uncertainties in the rest of the optimization/simulation model. Based on the CFE formation, a negative value indicates over-forecasting in cumulative rainfall, a positive value indicates under-forecasting in cumulative rainfall.

The root mean square error (RMSE) calculates the sample standard deviation of the difference between the actual cumulative rainfall and the forecast cumulative rainfall.

The formulation is as follow:

$$\text{RMSE} = \sqrt{\frac{\sum_{t \in \tau} (P_t - \hat{P}_t)^2}{n}}, \quad \forall t \in \tau \quad (7.14)$$

where

P_t = hypothetical rainfall up to time t

\hat{P}_t = forecasted rainfall up to time t

τ = total forecasting period

The RMSE is a way to measure the performance of the forecasting model in terms of “quality”, and it is always greater than or equal to zero. Preferably, the small RMSE is desired since large RMSE indicates a large standard deviation difference between the

actual and forecast rainfall, thus resulting unwanted uncertainties in the rest of the optimization/simulation model.

The computation time is calculated by taking the total computation time divide by the total number of forecasting period. In a real-time decision making scenario, less computational time is desired, since many cases decision would need to be made in a short time fashion.

Table 7.1: Summary of the Forecasting Model

	RMSE [in]	CFE	Time per Iteration [s]
AR	0.1499	0.4148	0.0716
ARX	0.134	0.3768	0.1048
ARMAX	0.1544	-0.1	0.2507
SSEST	0.1561	-0.109	0.7763

The RMSE for all four models are not too far off from each other, therefore four models produce similar quality of forecasting. There are quite differences in the CFEs, however, when comparing all four models. The autoregressive (AR) model and the autoregressive exogenous (ARX) model both have large CFE values, which mean both methods are way under forecast cumulatively compare to the actual (hypothetical) rainfall. Whereas the autoregressive moving average exogenous (ARMAX) model and the state-space estimation (SSEST) model performed much better than the AR and ARX models. Both the ARMAX and the SSEST over forecast cumulatively, and only over produced around 10%. The last criterion of the comparison is the computational time. Both the AR and the ARX models took less time than the ARMAX and the SSEST models. However, due to the less quality produced by the AR and the ARX models, the

ARMAX and the SSEST models is then compared against each other. The ARMAX takes significantly less time than the SSEST in this hypothetical rainfall example, thus the ARMAX is the most desired method out of all four methods. The optimization/simulation model would use the ARMAX approach for its rainfall forecasting component due to its quality production and lesser computational time required. The rainfall forecasting component of the optimization/simulation model incorporate an updating procedure for projected rainfall in real time similar to the procedure such as presented by Madsen and Skotner (2005), as well as the discussion in Section 7.4.

CHAPTER 8 – EXAMPLE APPLICATION

8.1 Hypothetical Model

This proposal addresses the importance of using real-time and forecasted data for extreme flood events in the real-time flood control operation of a river-reservoir system. It is also important to demonstrate the methodology using a test hypothetical example. A simple *two watershed – two reservoir* model is developed. Figure 8.1 illustrates the schematic of the simple application and Table 8.1 shows the parameters used in this simple model.

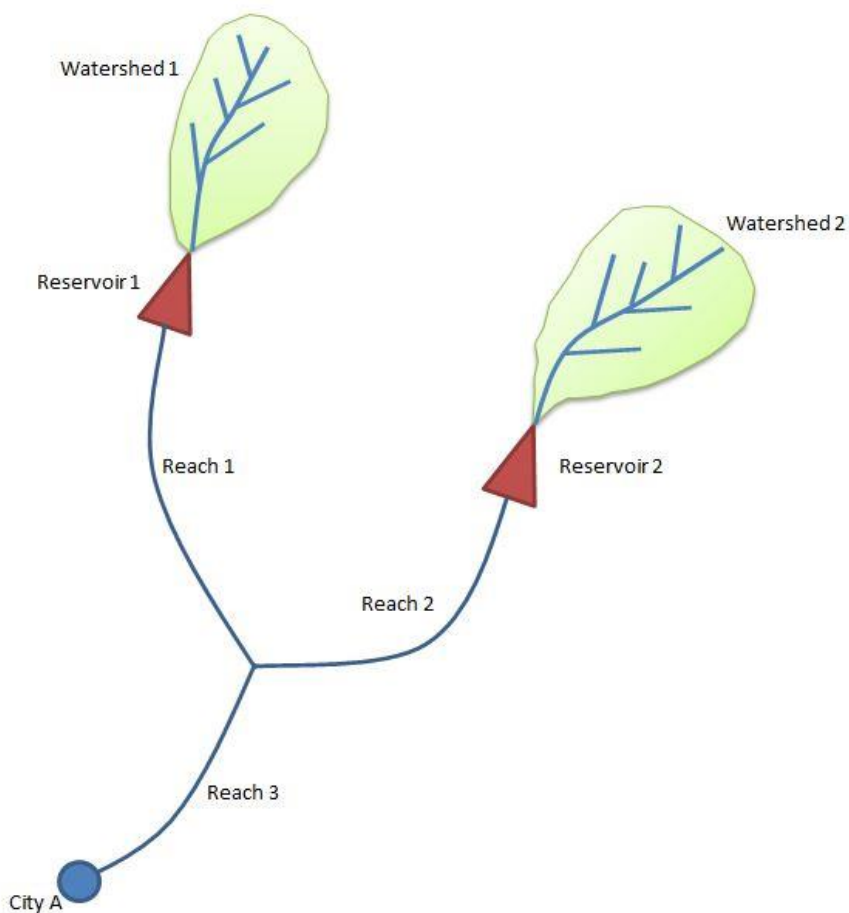


Figure 8.1: Schematic of the Test Model

Reservoir 1 and Reservoir 2 are downstream of Watershed 1 and Watershed 2, respectively. The runoffs of each watershed are directly discharged into their respective reservoirs. The releases from the Reservoir 1 and Reservoir 2 are then routed through Reach 1 and Reach 2, respectively. The reaches then merged into Reach 3 and eventually the floodwater reaches the control point, City A. The problem objective function is to minimize the total flows into City A with the penalty coefficient equals to one (Equation 7.1.a), while satisfying all hydraulic and operation constraints (Eqs. 7.2–7.6), as discussed previous section.

Table 8.1: Parameters of the Hypothetical Test Model

Parameter	Unit	Value
Area of Watershed 1	mi ²	58
Area of Watershed 2	mi ²	54
Length of Reach 1	miles	3
Length of Reach 2	miles	2
Length of Reach 3	miles	2
Reservoir 1 Maximum Flood Storage	ft ³	1.34×10^9
Reservoir 2 Maximum Flood Storage	ft ³	1.35×10^9
Reservoir 1 Inactive Storage	ft ³	4.45×10^8
Reservoir 2 Inactive Storage	ft ³	4.45×10^8
Reservoirs Rate of Discharge Limits	cfs/h	8000
Flood Stage at Control Point (City A)	ft	10

In the test simulation, each watershed is subjected to randomly generated storms over 72 hours; also storms arrive at the two watersheds at random time. Figure 8.2 depicts the hypothetical rainfall events of Watershed 1 and Watershed 2, respectively.

The total simulation time is 96 hours starting at t=0 hour, with the computation interval of 1 hour. The forecasting period is $\Delta t = 2$ hours, in other words, an iteration of

the optimization/simulation model shown in Figure 7.1 is 2 hours. The results, including the reservoir release schedules determined by the GA optimization, the forecasted rainfall, and flow rate at the control point (City A), are presented and discussed next.

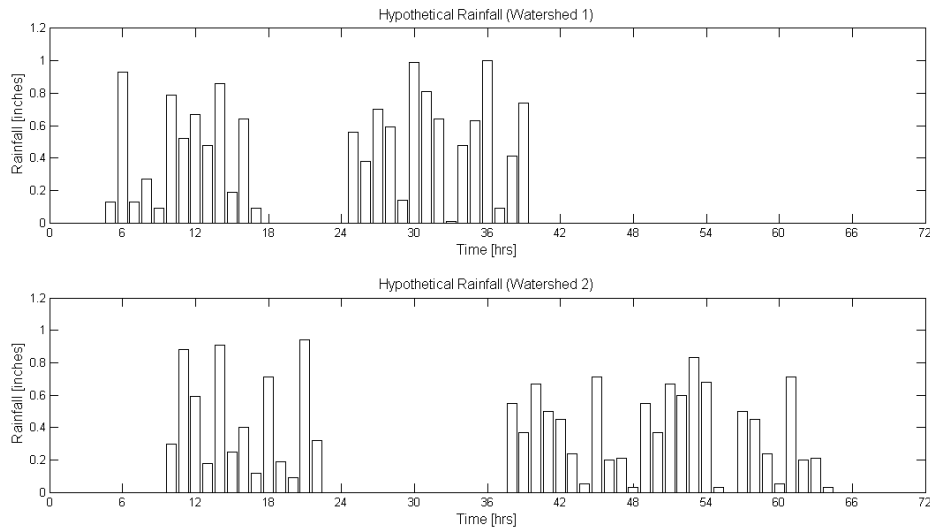


Figure 8.2: Hypothetical Rainfall Events

8.2 Model Results and Discussion

The optimization/simulation process started at $t=0$ hour. The forecasting process started at $t=7$ hour and $t=12$ hour for watershed 1 and watershed 2, respectively. The hypothetical and the complete forecasted rainfall time series for both watersheds are shown in Figure 8.3 and Figure 8.4. The rainfall forecasting model was not able to accurately predict the extreme peaks of the rainfall, thus the total forecasted rainfall volume are generally underestimated. However, the rainfall forecasting model was able to capture the general trend of the randomly generated hypothetical rainfall for both watersheds. To obtain a better forecasted estimation of rainfall, it is suggested that the forecasting interval should be decreased, such as $\Delta t = 1$ hour. The forecasted rainfall

values over Δt were used to compute the watershed outflow hydrographs, using the hydrological model HEC-HMS, over Δt for each watershed. The watershed outflow hydrographs for watershed 1 and watershed 2 are illustrated in Figure 8.3 and Figure 8.4. The outflow watershed hydrographs for watershed 1 and watershed 2 immediately become the inflow hydrographs for reservoir 1 and reservoir 2, respectively. In this hypothetical test model, the reservoirs are connected immediately downstream of their respective watersheds as shown in Figure 8.1. The optimization/simulation model entered into the next phase, which is the reservoir operation model coupled with the genetic algorithm solver in MS Excel.

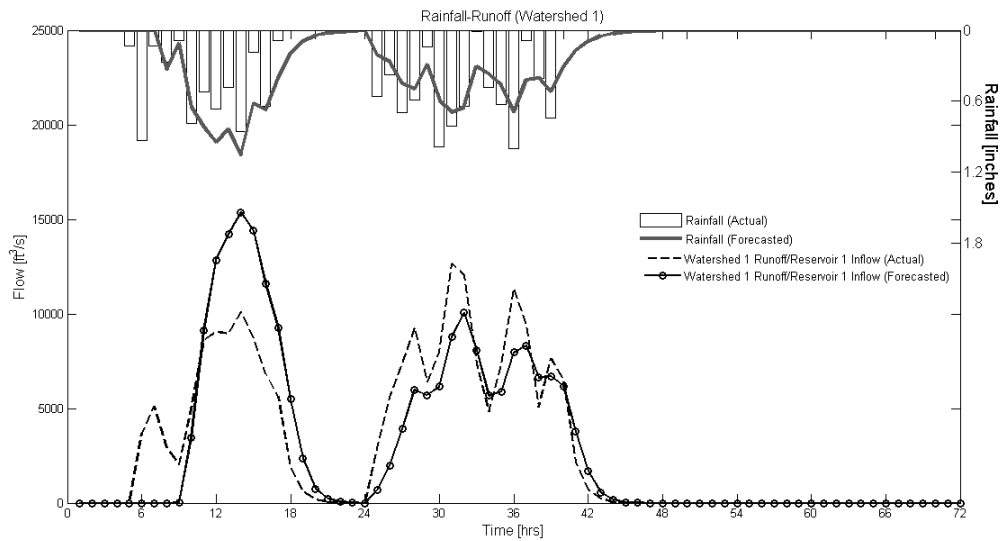


Figure 8.3: Hypothetical/Forecasted Rainfall Events and Runoff Hydrograph of Watershed 1

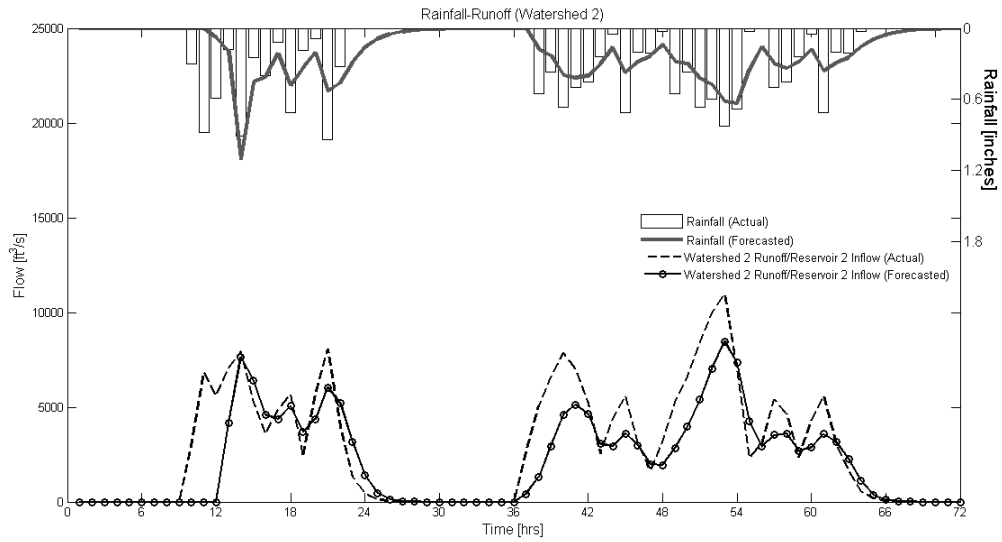


Figure 8.4: Hypothetical/Forecasted Rainfall Events and Runoff Hydrograph of Watershed 2

The reservoir operation model used the reservoir storage information at current time t and the forecasted inflow over Δt , to determine the reservoir releases over Δt . The reservoir operation model must keep the storage level above the inactive storage, and below the maximum flood storage, also the stage at control point City A must always be under the flood stage. The initial storages of both reservoirs were set to be 50% of the maximum flood storage. Table 8.1 shows the reservoir parameters for the hypothetical test model. The time series of releases from each dam are depicted in Figure 8.5.

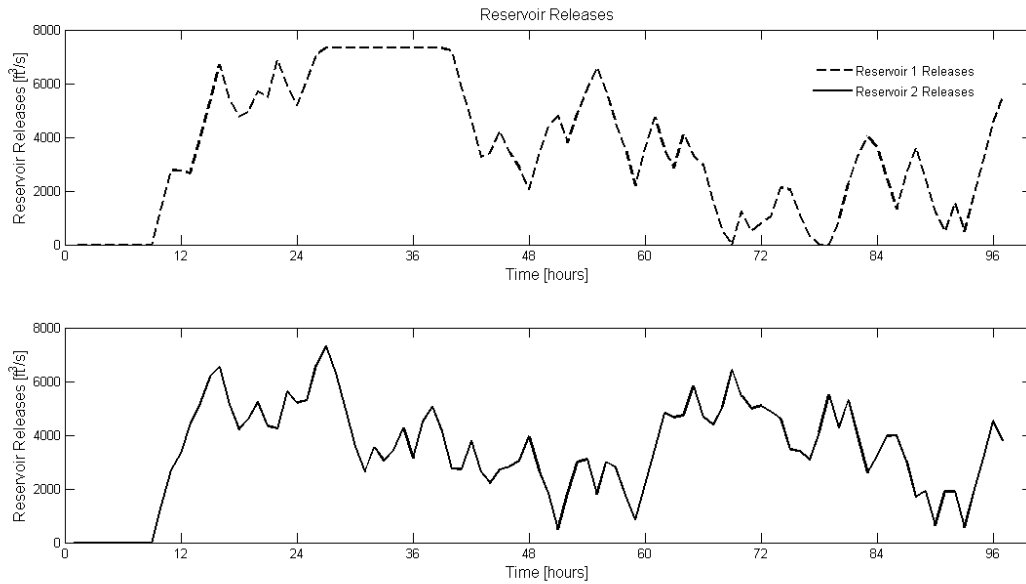


Figure 8.5: Releases from both Reservoirs

The storages of reservoir 1 and reservoir 2 are displayed in Figure 8.6. The reservoir operation model and the genetic algorithm solver were able to keep the reservoir storage of both reservoirs within the desirable range, which the storage for both reservoirs were kept above the inactive storage and below the maximum flood storage, thus preventing any potential dam failure. Moreover, the stages at the control point City A were under the flood stage at all time, which the results are discussed next. The outflow reservoir hydrographs were entered to the next component of the optimization/simulation model, the unsteady flow model using HEC-RAS.

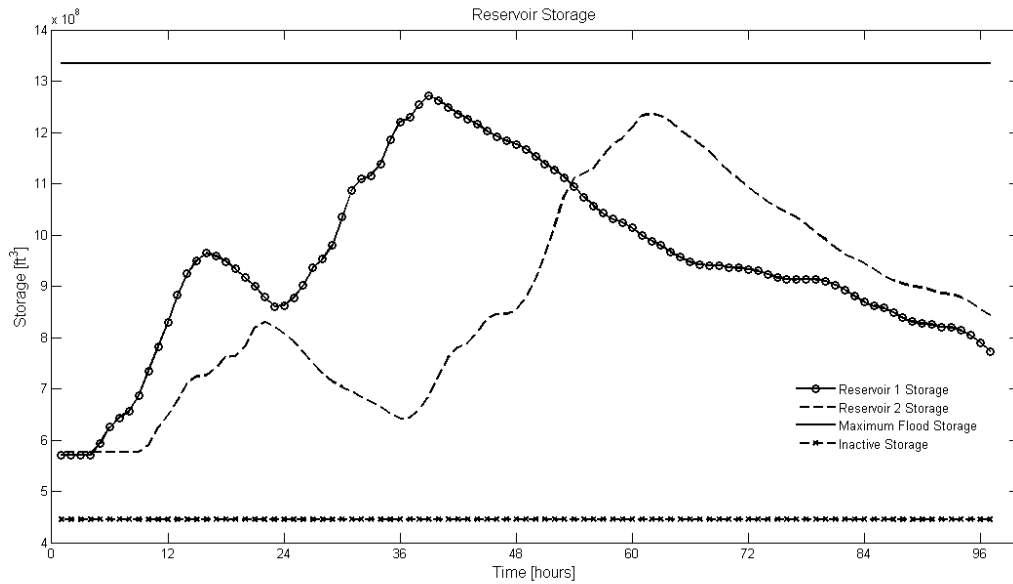


Figure 8.6: Simulated Storage of both Reservoirs

The stage conditions of the control point, City A, are illustrated in Figure 8.7. As seen on Figure 8.7, the simulated stages at the control point, depicted as solid line with circles, were also below the flood stage (solid line) of 10 ft. Figure 8.7 also shows the stage conditions of the control point (dash line) as if there were no reservoirs existed in the system. As shown in the figure, if there were no reservoirs in place and no operations, the flood stage were exceeded in several occasions, some as high as approximately 8 feet above the flood stage, consequently causing tremendous damages if City A was a populated residential area. The flow conditions of the control point, City A, are illustrated in Figure 8.8. Similar to Figure 8.7, Figure 8.8 depicts the flow conditions under both scenarios, which were with and without reservoirs and operations. Figure 8.8

shows that there were several huge flood peaks (dash line), which the maximum flood was as high as 32,227 ft³/s, if there were no reservoirs and operations in place.

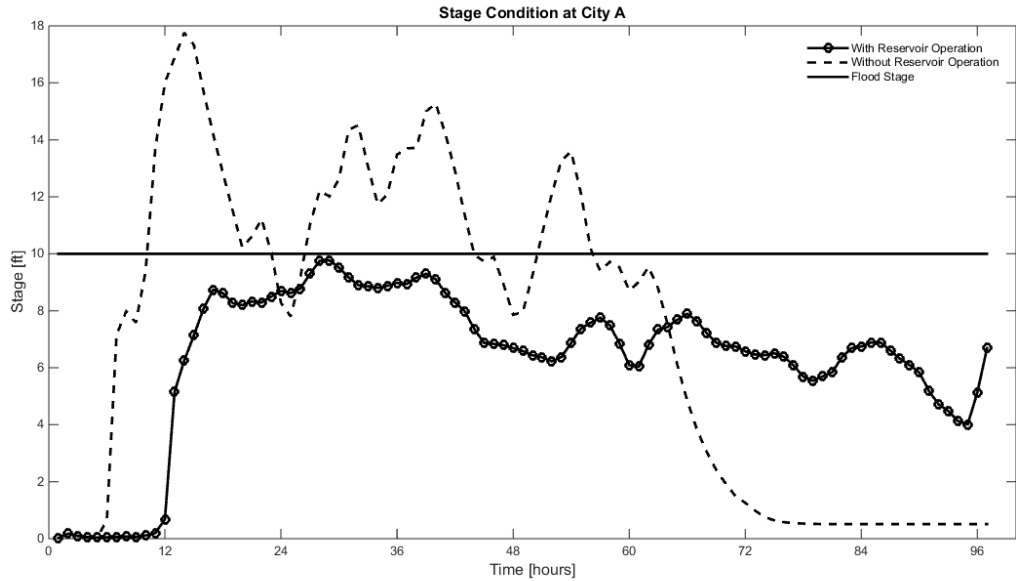


Figure 8.7: Stage Condition at Control Point – City A

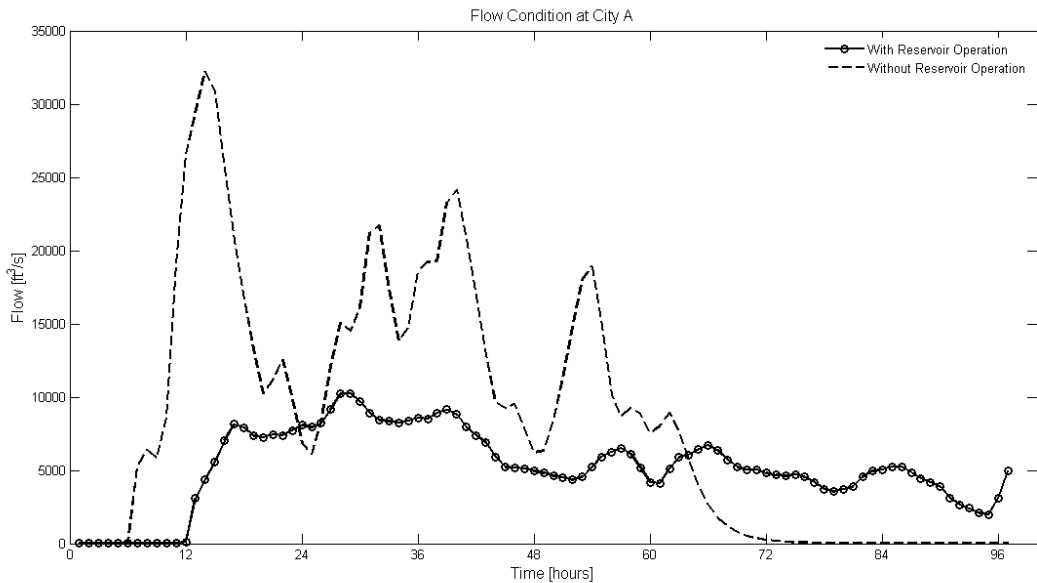


Figure 8.8: Flow Condition at Control Point – City A

The flow peaks were correlated with the stage peaks in Figure 8.7. When the simulation was incorporated with the river-reservoir simulation/operation model, the flow conditions at City A were well under control, which depicted as solid line with circles in Figure 8.8. The peak flow at City A was 10,240 ft³/s when the reservoir operations were applied, which did not cause flooding. The total volume of water passed through City A was 2.99×10^9 ft³ when there were no reservoirs and their operations. When the river-reservoir simulation/operation model was incorporated, the total volume of water passed through the control point was reduced 1.8×10^9 ft³ (a 39% reduction), since the reservoirs were able to store much of the floodwaters and make release decisions such that no flooding would occur.

The results of the simple hypothetical scenario shown here have demonstrated the importance and practical usefulness of the optimization/simulation model for real-time optimal of river-reservoir system for flood control purposes. The model can be applied to a much larger and a more complicated system. Besides for real-time decision making, the model can also be used to determine the optimal reservoir operations by generating possible extreme storms and determining optimal operation policies so that agencies would have a better understanding of the river-reservoir system.

CHAPTER 9 – APPLICATION ON THE CUMBERLAND RIVER BASIN

9.1 Rainfall-Runoff and Unsteady Flow Models

The optimization/simulation model described in Chapter 7 has been applied to a large river-reservoir system, the Cumberland River basin. The objective of the optimization/simulation model was to minimize the peak flood stage at Nashville (Equation 7.1.b), subsequently to keep the flood stage under the 100-year stage of 48 feet during the entire simulation period. The rules of reservoir operations were set in the model according to the water control manual by USACE as presented in Appendix L. The rainfall-runoff and the unsteady flow simulation used in the optimization/simulation model are the respectively the HEC-HMS and the HEC-RAS models. Figure 9.1 illustrates the model domain of the HEC-HMS and HEC-RAS on the Cumberland River Basin.

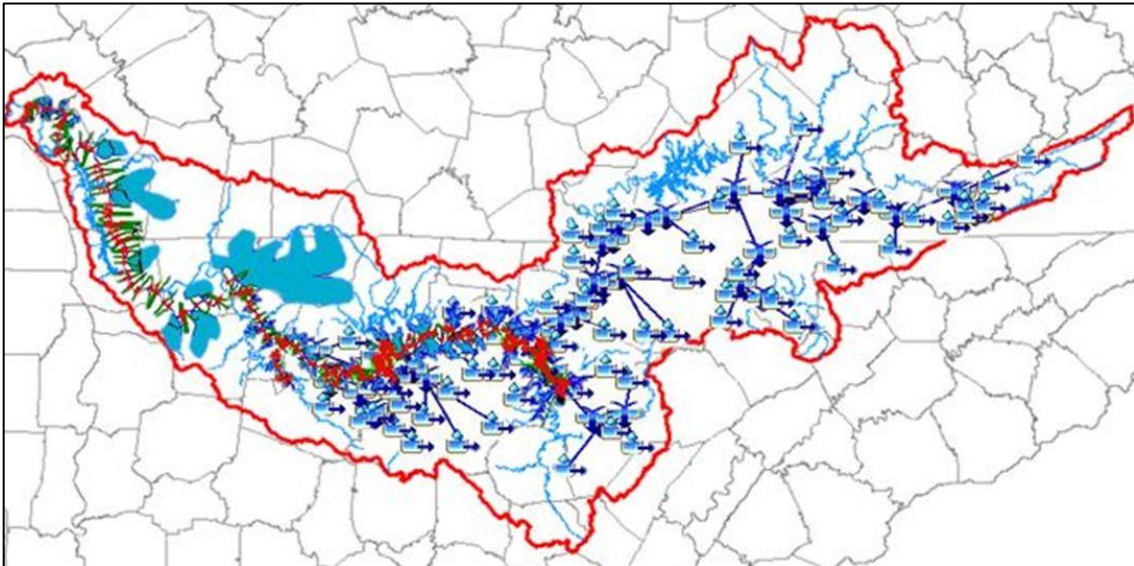


Figure 9.1: Model Domain of the HEC-HMS and HEC-RAS on the Cumberland River Basin

As described in Chapter 7, the rainfall data, both in real-time and forecasted, were the input of the HEC-HMS model. After the HEC-HMS performed hydrologic simulations, the outflow hydrographs from the model became the input of the hydraulic model, the HEC-RAS, for unsteady flow simulation.

The hydrologic model HEC-HMS covers approximately 14,160 mi² of the Cumberland River Basin, starting from the headwater of the basin, and ends at the Cheatham Dam, 32 miles downstream of Nashville, Tennessee. The HEC-HMS model consists of 69 subbasins and 66 reaches; basin areas range from 7 mi² to 1700 mi², with the average area of 205 mi². Table 9.1 shows hydrologic process methods used in the Cumberland River Basin HEC-HMS model:

Table 9.1: The Hydrologic Processes and Methods used within the HEC-HMS Application

Hydrologic Process	Method Used
Loss	Deficit Constant
Transform	Clark Unit Hydrograph
Base Flow	Bounded Recession
Channel Routing	Muskingum

The high resolution gridded rainfall forcing used in the hydrologic model HEC-HMS to simulate the May 2010 storm event is a product generated by Next-Generation Radar (NEXRAD). Figure 9.2 illustrates a time revolution (May 1st 10 a.m. to 1 p.m.) of the storm movement the NEXRAD gridded rainfall data during the May 2010 event.

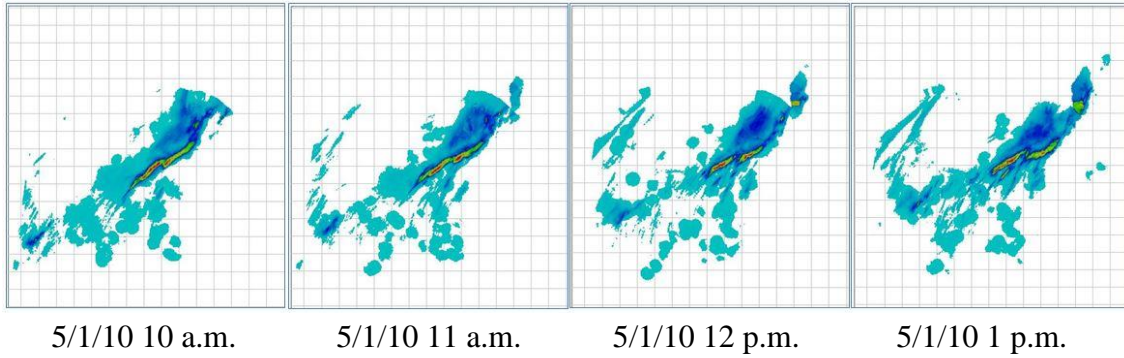


Figure 9.2: Sample Time Revolution of the May 2010 Storm Event from NEXRAD Gridded High Resolution Data

The HEC-HMS model has been calibrated and validated for the May 2010 storm event. Figure 9.3 shows the comparison of the simulated and the observed Dale Hollow reservoir inflow hydrographs during the May 2010 event. The root mean square error (RMSE) at the Dale Hollow reservoir for the HEC-HMS model is 6174 ft³/s, which is acceptable considering the magnitude of the storm event. Root mean square error is defined by

$$RMSE = \sqrt{\frac{\sum_{i=1}^N (Q_{observed,i} - Q_{simulated,i})^2}{N}} \quad (9.1)$$

where

$Q_{observed,i}$ is the i -th observed hydrograph ordinate

$Q_{simulated,i}$ is the i -th simulated hydrograph ordinate

N is the number of hydrograph ordinate for the model validation.

To further investigate the model validation, the quantitative measure of

performance or goodness of fit, *modeling efficiency* can be used. The modeling efficiency, described by Nash and Sutcliffe (1970) is based on the deviation variance. It is expressed as the following:

$$E = 1 - \frac{\sigma_e^2}{\sigma_o^2} \quad (9.2)$$

where

E = model efficiency

σ_e^2 = variance of the deviation between observation and simulation, and

σ_o^2 = variance of the observations.

The efficiency is similar to the statistical parameter *coefficient of determination*, R^2 . It has a value 1 for a perfect fit, which can be converted to percentage by multiplying by 100. The variance of deviation is defined as:

$$\sigma_e^2 = \frac{1}{N-1} \sum_{i=1}^N (Q_{observed,i} - Q_{simulated,i})^2 \quad (9.3)$$

The HEC-HMS Cumberland River Basin is well validated with a model efficiency of 0.853. In operational forecasting, having low RMSE and high modeling efficiency is necessary to build confidence in the model to produce reliable forecast information for critical decision making.

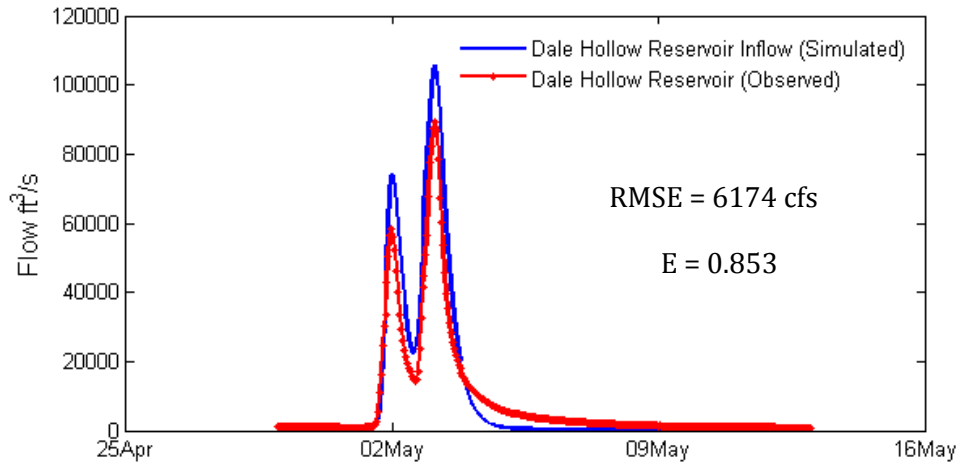


Figure 9.3: HEC-HMS Model Validation for the May 2010 Storm Event at Dale Hollow Dam

The HEC-RAS model for the Cumberland River application contains 801 computational nodes. These include 675 cross-sections, 1 bridge, 8 inline structures, and 117 lateral structures. The HEC-RAS model also has been calibrated and validated for the May 2010 storm event. Figure 9.5 shows the comparison of the simulated and the observed hydrographs at Nashville during the May 2010 event. The root mean square error (RMSE) at the Nashville for this HEC-RAS model is 14550 ft³/s, which is acceptable considering the magnitude of the storm event and the nature of unsteady flow simulation. Figure 9.6 illustrates the comparison of the simulated and the observed stage at Nashville during the May 2010 event. The root mean square error (RMSE) at the Nashville stage for this simulation is 1.777 ft. The model efficiency for the HEC-RAS model is 0.890, this implies that the unsteady flow model is well calibrated and validated, or in order words, eighty nine percent of the deviations are explained statistically.

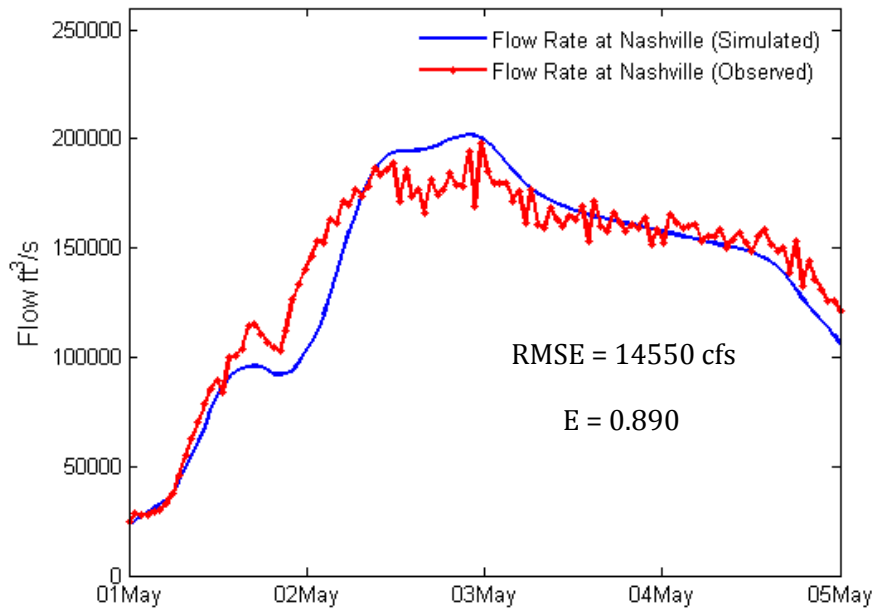


Figure 9.4: HEC-RAS Flow Validation for the May 2010 Storm Event in Nashville

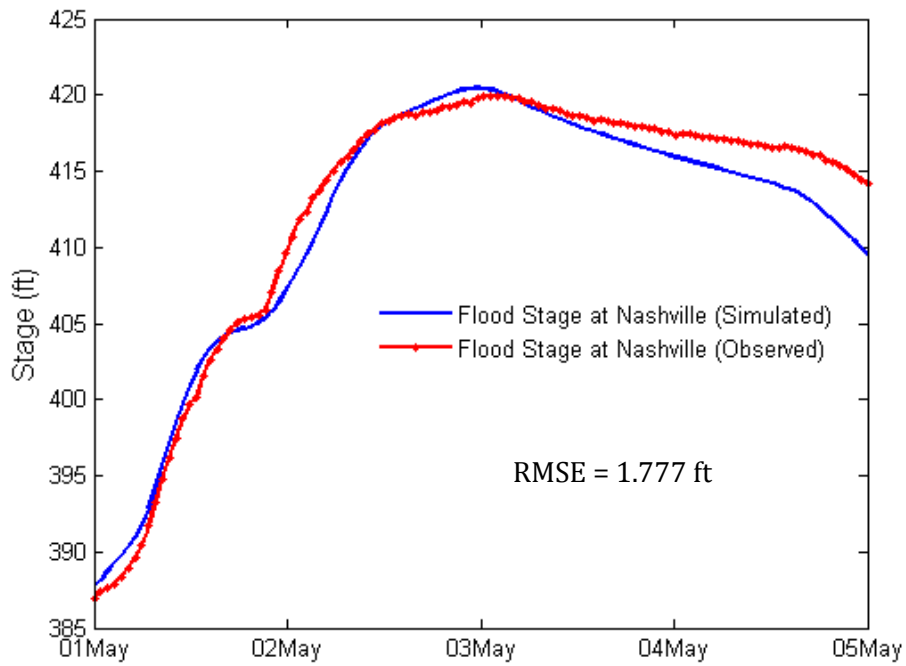


Figure 9.5: HEC-RAS Stage Validation for the May 2010 Storm Event in Nashville

9.2 Area-Weighted Rainfall Forecasting

Forecasting gridded rainfall is sometimes difficult and tedious. For this research, a simpler approach is applied to make the rainfall forecasting process efficient. An area-weighted rainfall forecasting was proposed. First, for each grid cell, rainfall up to current time step, t , was extracted to form a hyetograph (a time series) as illustrated in Figure 9.6.

For each subbasin, the weights of grid cells, w , that are overlaying the subbasin are calculated as illustrated in Figure 9.7. Then, a time series of rainfall up to current time, t , for the i -th subbasin can be determined by:

$$P_{i,t} = \sum_j w_{i,j} P_{j,t} \quad (9.4)$$

where

$P_{i,t}$ = time series of rainfall up to current time, t , for the i -th subbasin

$w_{i,j}$ = weight of the j -th grid overlaying the i -th subbasin

$P_{j,t}$ = time series of rainfall up to current time, t , for the j -th grid

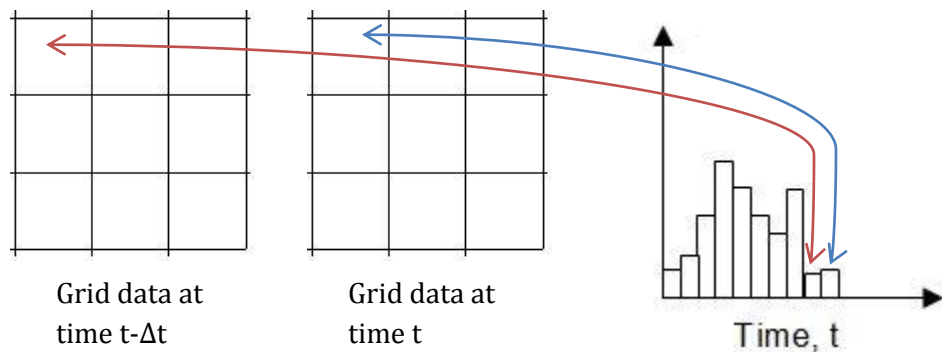


Figure 9.6: Hyetograph Generation for a Cell by Grid Data Extraction

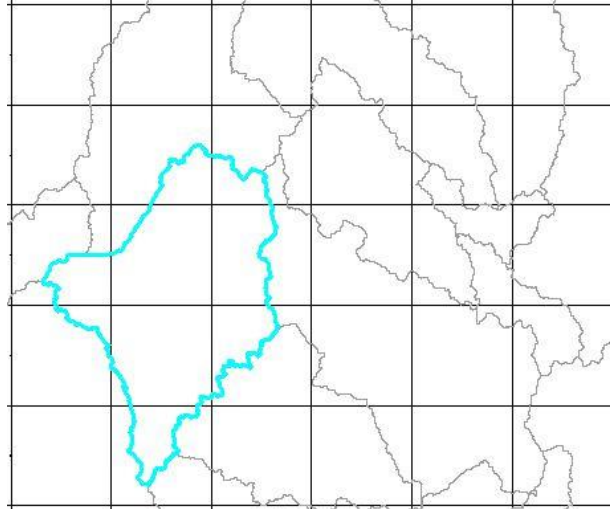


Figure 9.7: The Overlaying of Subbasins and Grid

Each $P_{i,t}$ is entered in to Equation 7.8 for rainfall forecasting. Figure 9.8 demonstrates the comparison of the actual rainfall data and the forecast rainfall in subbasins *ClearFkSaxton* and *BrownsCrFairgrounds*, respectively.

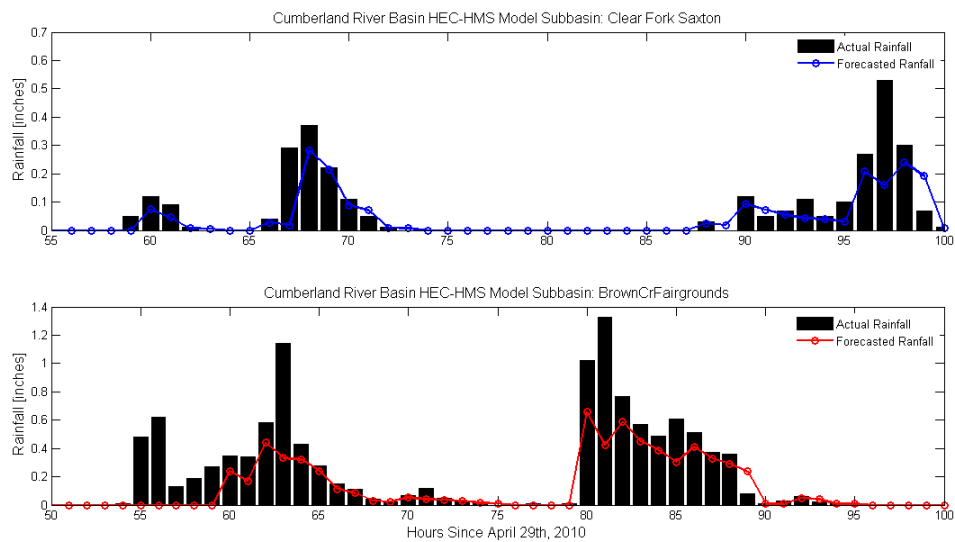


Figure 9.8: Comparison of the Actual and Forecast Rainfall for HEC-HMS Model Subbasins *ClearFkSaxton* and *BrownsCrFairgrounds*

The rainfall forecasting model was not able to accurately predict the extreme peaks of the rainfall, thus the total forecasted rainfall volume were generally underestimated. The rainfall forecasting model was able to capture the general trend of the rainfall for both subbasins. All rainfall-runoff forecast are presented in the Appendix.

9.3 Simulation Approach

Demonstration of the optimization/simulation model was performed using the May 2010 flood event on the Cumberland River system. The U.S. Army Corps of Engineers developed the HEC-HMS and HEC-RAS models for the 2010 flood event. The model was first be applied to a portion of the Cumberland River system that includes Old Hickory and Cordell Dams (see Figure 9.9 below), which are the two immediate dams on the Cumberland River upstream of Nashville, Tennessee.

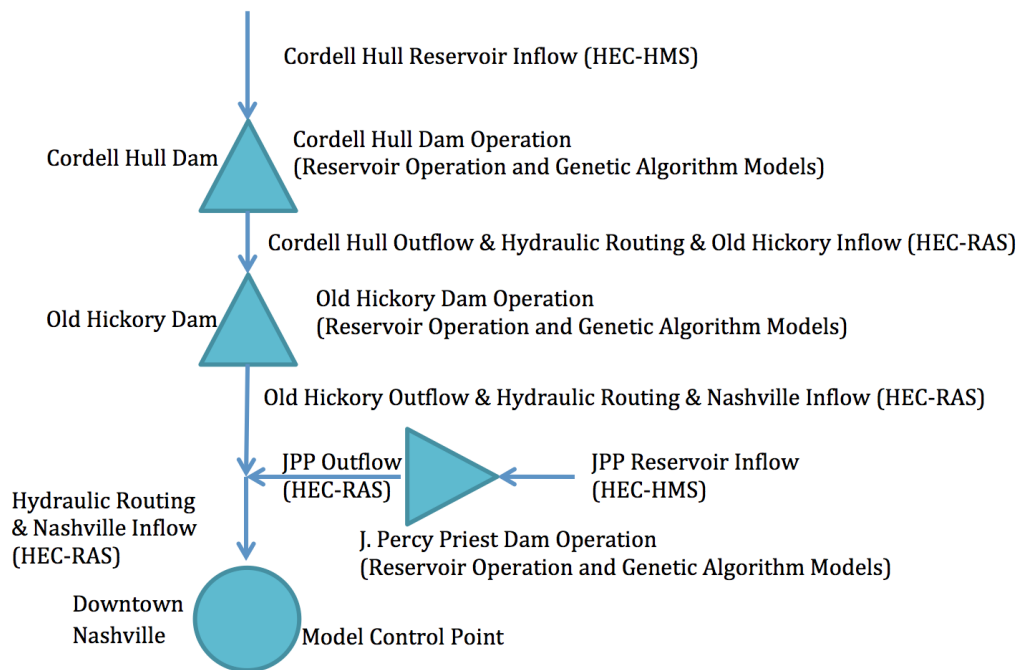


Figure 9.9: Basic Schematic of the Optimization/Simulation Model on the Cumberland River Basin

Simulation process as depicted in Figure 9.9 started with the hydrologic modeling using HEC-HMS, which simulated the Cumberland River Basin rainfall-runoff process. The HEC-HMS model covered all subbasins and reaches upstream to the Cordell Hull Reservoir, which the reservoir inlet node is the outlet node of the HEC-HMS model. Once the HEC-HMS model generated the Cordell Hull reservoir inflow hydrograph, the inflow hydrograph was entered into the Cordell Hull reservoir operation and optimization models for determining its operation. Once the gate release decisions were determined, the information became the input of the unsteady flow HEC-RAS model for downstream hydraulic routing. Once the flow information was reached at the Old Hickory Dam, the inflow hydrograph was entered into the Old Hickory reservoir operation and optimization models for its operation determination. Similar to the process at the Cordell Hull Dam, once the gate release decisions were determined at the Old Hickory Dam, the information became the input of the unsteady flow HEC-RAS model for downstream hydraulic routing. Similar procedures were applied to the J. Percy Priest Dam. The decision variables of the optimization/simulation model would be determined by the condition at the system control point, downtown Nashville. The processes were to repeat if the objective was not met, or the processes move to the next forecasting period if the objective was met, as discussed in Chapter 7 (see Figure 7.1). For the Cumberland River Basin application, the objective is to keep the peak flood levels at downtown Nashville under 100-year level throughout the May 2010 storm event. In other words, the optimization/simulation model would keep the flow rate at the model control point below Q_{100} at all time, while satisfying all the model constraints described in Chapter 7. As seen in Figure 9.10, which depicts the flood stage condition in Nashville during the May

2010, the 100-year flood stage (the red-dashed line) is 48 feet and the corresponding flow rate is $Q_{Nashville,100} = 172,000 \text{ ft}^3/s$.

The optimization/simulation would determine the optimal operation at the Cordell Hull Dam and Old Hickory by utilizing rainfall and flood forecasting, to keep the flood stage at Nashville under the red-dashed line. The result and detailed analysis is presented in the next section.

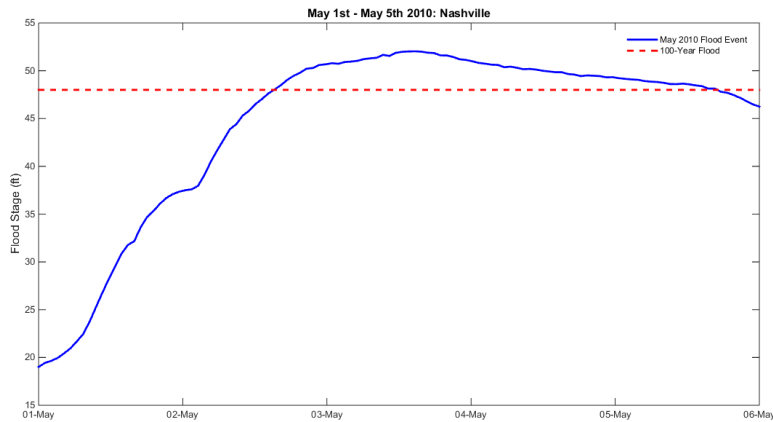


Figure 9.10: Flood Stage Condition at Nashville during the May 2010 Storm Event

9.4 Model Results and Discussion

The optimization/simulation model results for the May 2010 Storm Event in the Cumberland River Basin is presented here. First, the operation at the Old Hickory Dam is discussed here. The simulated optimal gate operation at the dam is compared to the actual gate operation by the USACE during the event. Second, the flood condition at

model control point, Downtown Nashville, is analyzed. Here, comparisons between the actual flood condition and the flood condition simulated by the optimization/simulation model are presented and discussed.

9.4.1 Operation at the Old Hickory Dam

The decision variables of the optimization/simulation model are the gate opening at the dams. The constraints at dam are incorporated in the optimization/simulation model. These constraints include: 1) reservoir stage-storage relationship, 2) gate openings-discharge relationship, 3) gate operation rules under flooding condition, 4) height rate of change of the gates per hour. Due to the severity of the May 2010 storm event at the Cumberland, which was a 1000-year event, the optimization/simulation model would determine the gate releases in hour-to-hour basis. In other words, for every forecasting period, which is the next two hours, the operation of the following two hours is determined. Figure 9.11 shows the gate operation determined by the optimization-simulation model, and Figure 9.12 shows the Old Hickory Dam releases determined by the optimization-simulation model.

During the actual event, the United States Army Corps of Engineers did not release until the late May 1st, even though the forecast rainfall information was available at the time. The mismanagement was one the main reasons that downtown Nashville was flooded. The optimization/simulation model used the rainfall forecasting model to project the precipitation and used the hydrologic model to project upstream watershed runoff, then used the unsteady flow model to make flood forecast. The optimization/simulation model recognized the forecasted information was available, and then made decision at the Old Hickory Dam well ahead of the arrival of the actual storm.

As seen in Figure 9.11, the optimization/simulation model determined that it was necessary to release water from the Old Hickory Dam by the forecast information. By the time the U.S. Army Corps of Engineers started to react to incoming storm and flood water, the Old Hickory was well prepared by the decision made in the optimization-simulation model.

To quantitatively compare the operation by the U.S. Army Corps of Engineers and the optimization-simulation model, Figure 9.13 illustrates the cumulative releases under two operations.

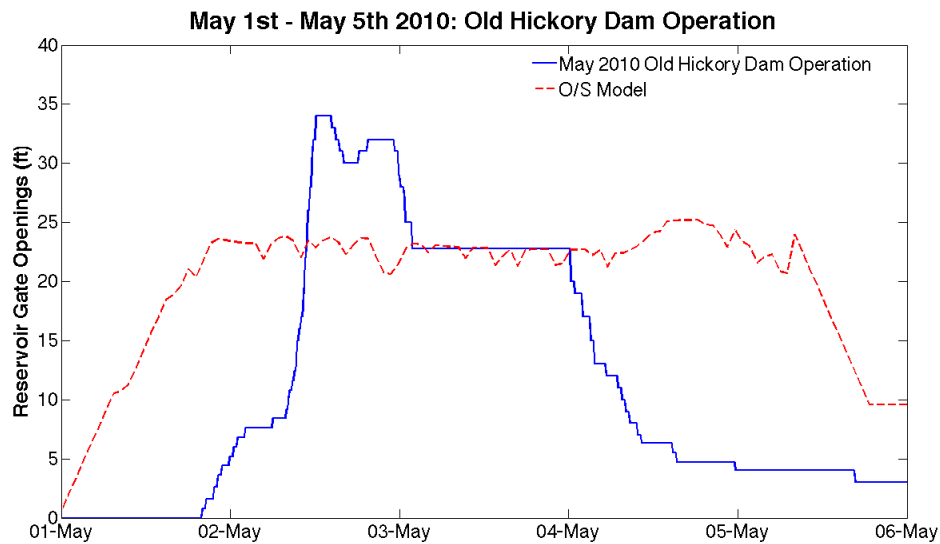


Figure 9.11: Optimal Operation at the Old Hickory Dam by the Optimization/Simulation Model

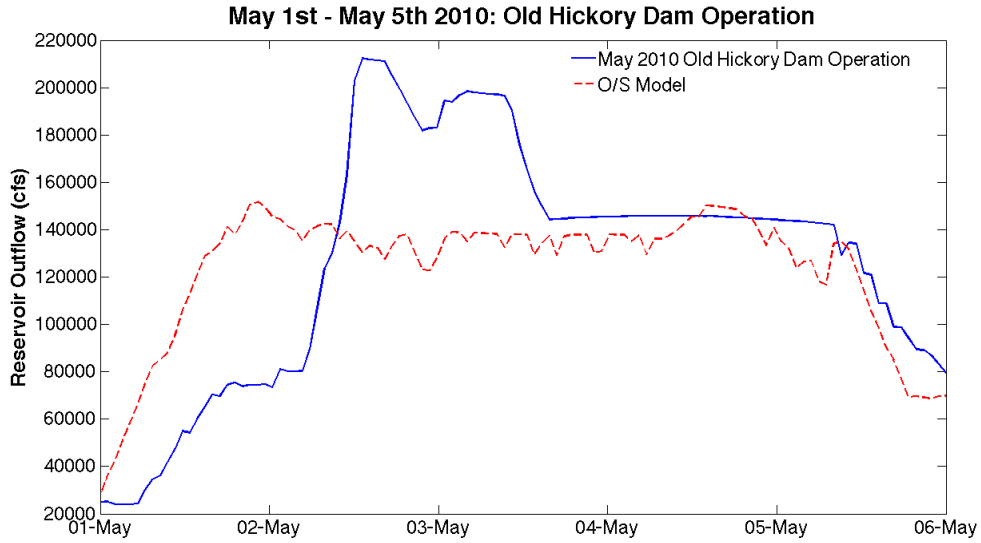


Figure 9.12: Old Hickory Dam Releases by the Optimization/Simulation Model

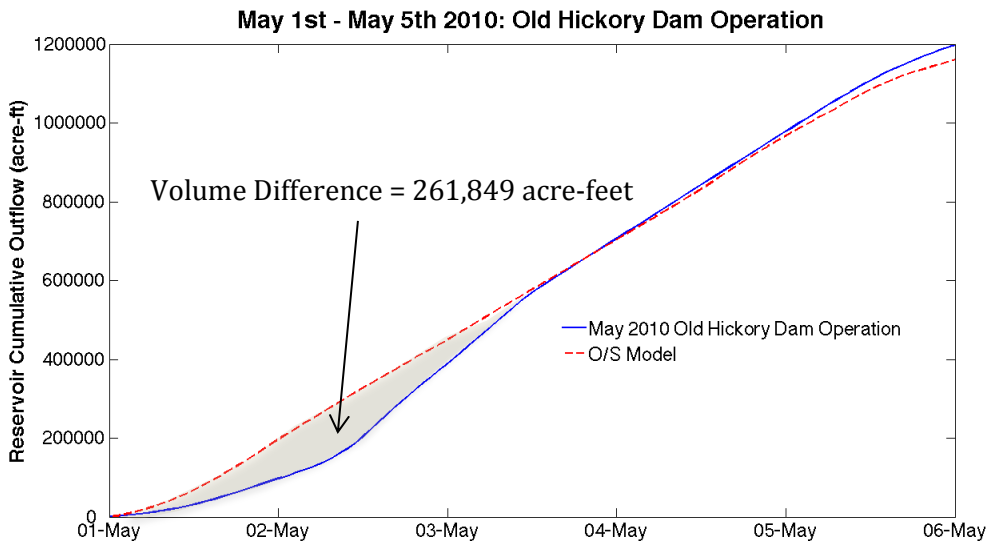


Figure 9.13: Cumulative Old Hickory Dam Releases by the Optimization/Simulation Model and the USACE

As seen in Figure 9.13, the total cumulative releases over the five-day period under both operations were relatively the same. Under the U.S. Army Corps of

Engineers operation, the total cumulative release from the Old Hickory Dam was 1,198,660 acre-ft; on the other hand, the total cumulative release from the Old Hickory Dam under the optimization/simulation model was 1,160,631 acre-ft. The optimization/simulation model was actually releasing less water in total as compared to under the U.S. Army Corps of Engineers operation. The reason of this was because the optimization/simulation made the decision to release water well in advance of the real storm, thus under this operation, less water was released. Due to the early response from the Old Hickory under the optimization/simulation model operation, the cumulative release was actually well above the operation under the U.S. Army Corps of Engineers operation until in the later of May 3rd, 2010, which the two cumulative releases were equal. Up to this point, the actual storm has ended (see Figure 1.11 and Figure 2.14); the cumulative release from the Old Hickory Dam under the optimization/simulation operation was 261,849 acre-ft more than the cumulative release by the U.S. Army Corps of Engineers.

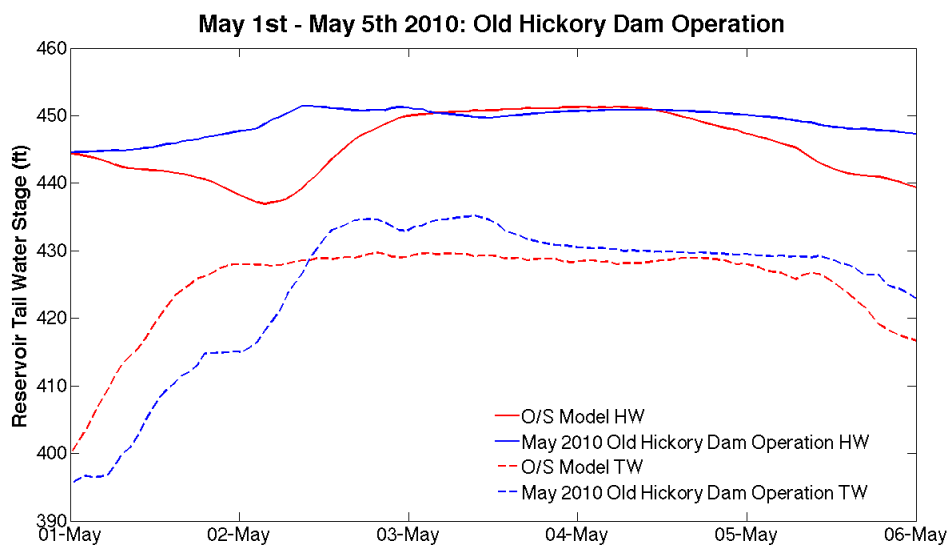


Figure 9.14: Old Hickory Dam HW and TW Stage Level by the Optimization/Simulation Model and the USACE

Figure 9.14 depicts the Old Hickory headwater and tailwater stage levels during the May 2010 storm event under both the U.S. Army Corps of Engineers and the optimization-simulation model operations. The tailwater stage is the water surface level immediate downstream of the dam. As seen from Figure 9.14, under the U.S. Army Corps of Engineers operation, there was a huge difference (approximately 40-foot different) in maximum and minimum tailwater stage between the start of May 1st and the end of May 3rd. Under the optimization/simulation model operation, the difference between the maximum and minimum tailwater stage was only less than 30-feet and well controlled, also the tailwater stage has never reached as high as the condition under the U.S. Army Corps of Engineers operation. Figure 9.14 also illustrated that, the O/S model decided to empty the reservoir early as compared to the U.S. Army Corps of Engineers' operation, this can be seen by the decrease of the headwater level.

9.4.2 Flood Condition at Nashville

The flow condition at downtown Nashville, being the model control point, is the most important factor of the optimization/simulation model. Figure 9.15 shows the flood stage at Nashville during the actual May 2010 storm event and the condition under when optimization/simulation operating rules were implemented.

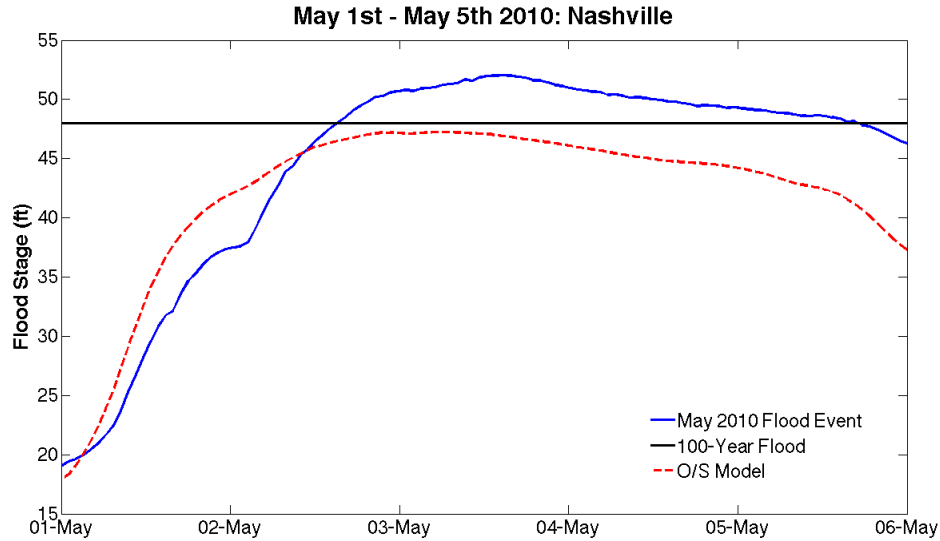


Figure 9.15: Flood Stage Condition (Simulated and Observed) at Nashville during the May 2010 Storm Event

The objective of the optimization/simulation model was to keep the flood stage at Nashville under the 100-year stage during the entire simulation period by minimizing the peak flow at Nashville. As seen in Figure 9.9, for the most of the simulation period, the flow stage is well under 100-year stage of 48 feet. The highest flood stage at Nashville under the optimization/simulation model operation was 47.26 feet, although this water surface level was close to the 100-year level, but would not have caused devastating flooding like the event in May 2010. Figure 9.16 shows the flow rate at downtown Nashville during the actual May 2010 storm event and the condition under when optimization/simulation operating rules were applied.

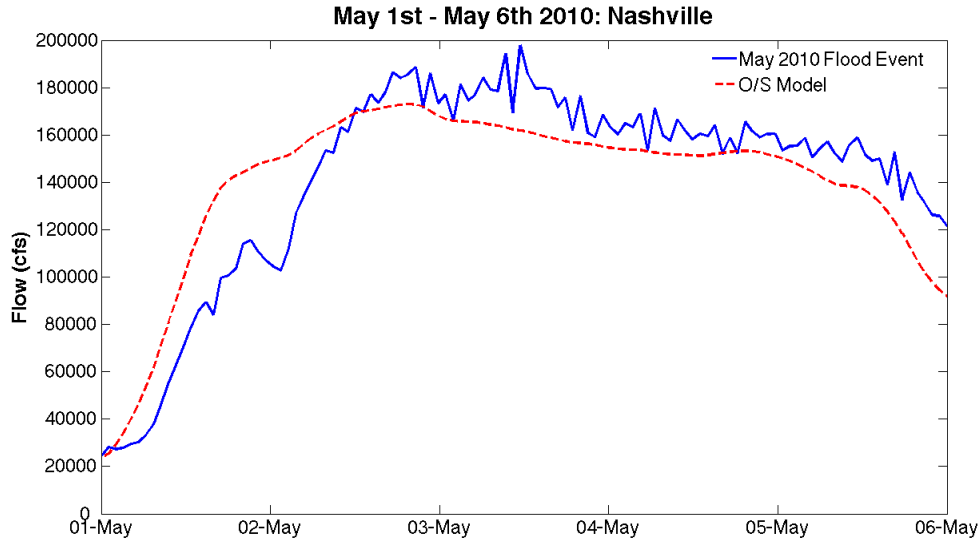


Figure 9.16: Floodwater Flow Rate (Simulated and Observed) at Nashville during the May 2010 Storm Event

The 100-year flow rate at the control point is $Q_{Nashville,100} = 172,000 \text{ ft}^3/s$, and the flow condition at Nashville under the optimization/simulation model was well under controlled. The maximum flow rate under the optimization/simulation model simulation was $Q_{Nashville,max} = 171,076 \text{ ft}^3/s$, and the for the majority of the simulation period, the flow rate was well managed and controlled. To quantitatively compare the impact on the flow condition at Nashville by the U.S. Army Corps of Engineers and the optimization-simulation model operations, Figure 9.17 illustrates the cumulative releases under two circumstances.

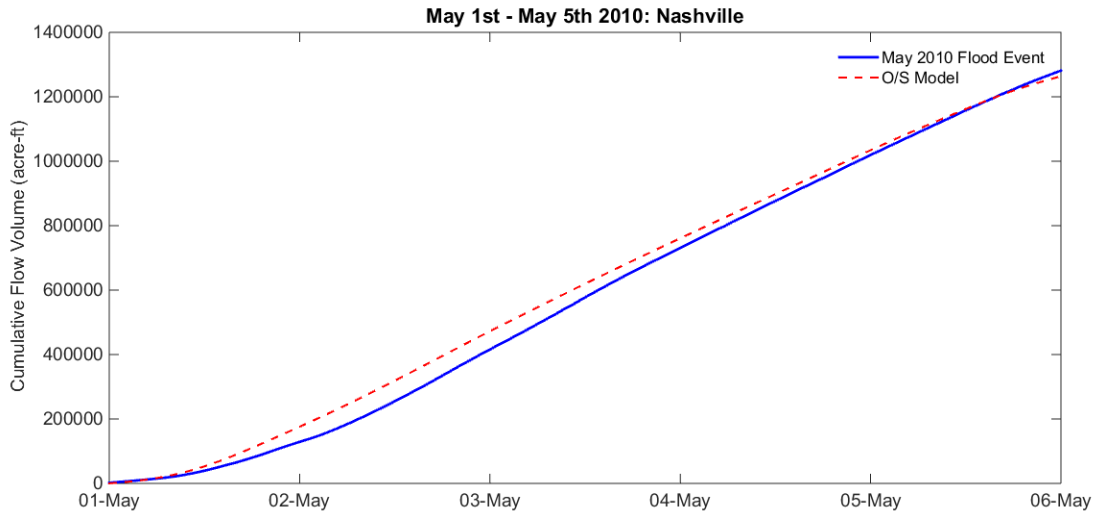


Figure 9.17: Cumulative Floodwater (Simulated and Observed) at Nashville during the May 2010 Storm Event

Under the optimization/simulation model scenario, the total floodwater volume passed through Nashville was less than the actual event over the five-day span. However, during the storm event between May 1st and May 5th, more flood water passed through Nashville under the model simulation than the actual event. This was due to the fact that, under the optimization/simulation model dam operation, the Old Hickory Dam started released water well ahead of the storm arrival. Despite the fact that there was more water passed through during the storm event (May 1st through May 5th), the optimization/simulation model was able to achieve the objective that keeping the flood stage and flow rate under 100-year level.

CHAPTER 10 – SUMMARY, CONCLUSION, RECOMMENDATIONS AND FUTURE WORK

10.1 Summary and Conclusion

In this research, an optimization/simulation model was created to determine optimal reservoir-release schedules during flood events. The optimization/simulation combines hydrologic and hydraulic models, a short-term rainfall-forecasting model, and genetic algorithm optimization and reservoir operation models. Combining each component into one larger model allowed for easily accessible and efficient operation. It was developed in response to the 2010 flood event that occurred in the Cumberland River basin in Nashville, Tennessee. This tragic event resulted in major structural and property damage and loss of life. The optimization/simulation model specifically addressed these issues from this tragic event.

As expected, the optimization/simulation model revealed more efficient and sustainable modes of short-term reservoir operations, which reaffirmed the problematic nature of the antiquated reservoir operational procedures. Specifically, it revealed that the Old Hickory dam, the reservoir upstream of Nashville, was more contained in the model simulation than the actual reservoir during the 2010 event. During the actual event, the massive rainfall nearly overtopped the Old Hickory dam. So, the model in this case, worked to create extra flood volume in the reservoir ahead of the flood event so that headwater would not approach the dam's overtopping level. This is because the model contained a built-in forecasting component that was able to project short-term rainfall. This forecasting system could then be theoretically used to run a subsequent flood simulation to determine the necessary steps to take in order to prevent floods. One

especially crucial step, for example, would be to determine an optimal gate release schedule. In the case of the Old Hickory dam, the gate was released only after the storm had accelerated. Moreover, the gate was released far too quickly rather than incrementally and at a slower pace in order to control the velocity of the floodwaters. As the model showed, had the Old Hickory dam imposed an optimal gate release schedule ahead of the actual storm, the floodwater levels in downtown Nashville could have been significantly reduced. The information gathered from the simulation model could be given to the appropriate agencies on the ground so that they may disseminate it in a way best suited for a particular community.

What this project has ultimately proven is how incredibly important continued research in this area is. As well-intentioned government and private organizations are in managing major flood and weather events, mistakes can be made, as seen in Nashville. In Nashville, for example, the forecast data from the NWS was not fully utilized by the U.S. Army Corps of Engineers, who was tasked with handling the locks and dams project in the Cumberland River basin. Unfortunately, this resulted in the deaths of 26 individuals and two billion dollars in property damage. The advantages of continued research in water-resource management are incalculable given the very real human toll associated with it. As devastating as the 2010 flood event was, real-time operation strategies for reservoirs around the globe are still not widely applied. Clearly, there is much to be done through research and development to ensure a more accessible, less complex application procedure.

10.2 Recommendations and Future Work

As I worked with my optimization/simulation model, I found that I needed to make modifications in order to better satisfy real-world demands. Specifically, more focus should be on using more efficient operation models because the traditional models that are widely used today have potentially dangerous implications on communities and the environment. Moreover, engineers, in large part, prefer efficiency to complexity since efficiency ensures a more sustainable platform from which to navigate the river systems. The example of the flood event in the Cumberland River outlined throughout this research proves this point and is why I have argued for the implementation of the simulation model, as it addresses many of these issues. Thus, I offer the following recommendations for future research. The first of these recommendations is to improve upon the existing model since it is clearly the next step in the progression towards a more efficient model. Secondly, there appears to be a real opportunity to expand the capabilities of the model to include more functional tasks. And lastly, I have recommended that the model be applied to more weather events. First, however, I discussed the ways in which we can improve upon the five different components that make up the current simulation model.

To be clear, these five components are actually five individual models that make up the larger simulation model I created for this research, and are listed as follows: a rainfall forecasting model, a rainfall-runoff model, an unsteady flow model, a reservoir operation model, and a genetic algorithm optimization model. Each of these components function independently of other components to serve their own purpose. However, for

the intention of this research, I integrated all five components so that they worked in concert with one another in order to establish a more realistic representation of flood events. But this task proved challenging because each component was unable to interface with other components as they worked with different logistical language. This challenge ultimately led me to consider alternative solutions. In particular, it is clear that a more efficient method of computer language exchange is needed to improve the overall performance of the optimization/simulation model. Of course, one way to create a more efficient method would be to eliminate the unused portions of the model and the software associated with it. This would help speed up the model's computational time and increase the model's efficiency, which would starkly improve upon each component's interfacing capabilities.

Indeed, there are software programs that have better interfacing capabilities than the ones I used in this research. My model interface and automation was designed using MATLAB version R2014b and Pulover's Macro Creator version 4.1.0., a free open software program. MATLAB is standard in academia and among researchers but are rarely used in the engineering industry. The software programs more commonly used in the engineering industry and the ones I recommend for model interfacing are M.S. Visual Basic, M.S. Visual Studio, and Python. These software programs, in particular, have better visual development environments and much stronger interfacing capabilities. Moreover, the database storage for interfacing can be developed using the far more superior programs, MySQL or T-SQL instead of MATLAB, which is the program used in this research. At this point, it is important to distinguish how each individual model or

component of the larger model functions in order to understand the ways in which we can improve upon them.

Specifically, the optimization/simulation model developed for this research uses the conventional hydrologic and hydraulics models, HEC-HMS and HEC-RAS, respectively. These are very popular in the private and public industries. However, the HEC-HMS is a lumped hydrologic model, which, of course, means that users do not get a higher resolution for computation. However, an example of a more desirable hydrologic model would be the MIKE-SHE model. The MIKE-SHE model is a distributed model, which provides a finer resolution for computation, and would likely be beneficial if the energy exchange between the atmosphere and the earth's surface was needed in the analysis. More relevant models, on the other hand, could also replace the hydraulic models used in this research.

In particular, a 2-D unsteady flow model, such as the Flow2D model, could be more advantageous than HEC-RAS, a 1-D model, especially when the terrain is complex or when a more detailed computation is needed. For example, a river that has numerous tributaries and constantly changing slopes would be considered a complex terrain that would need a more detailed computation than a 1-D model could provide. The next individual model/component within the larger optimization/simulation model I examined is the short-term, rainfall-forecasting model.

For this research, I used MATLAB once again for the short-term rainfall-forecasting component. Although MATLAB is an excellent software program with outstanding statistical and forecasting applications, there are much better software

programs, such as R and Python. R, in particular, is specifically designed for statistical and forecasting analysis, so it is better equipped to handle forecasting computations than MATLAB. Python, on the other hand, is similarly designed as MATLAB, except that Python performs faster with a lot more capabilities. The last two components of the optimization/simulation model are used by the same software program.

The last two components of the optimization/simulation model are the optimization model and the reservoir operation model. For each model, I used M.S. Excel because it has a built-in optimization evolutionary solver. This enabled me to determine the optimal reservoir release values as opposed to other software programs that are not as user friendly. In addition, M.S. Excel allowed me to use the mass balance approach in a spreadsheet, which makes the data easier to visualize. My second recommendation, noted earlier, is to expand the capabilities of the optimization/simulation model to include more functional tasks.

Currently, this optimization/simulation model only contains a flood-related objective. Obviously, a single objective model limits our abilities to apply it to other real-world scenarios, which can be as equally important or have just as many theoretical implications on communities and environments as flood events. Therefore, it stands to reason, that the model should be modified to include multi-operational objectives. Some examples of multi-operational objectives within reservoir settings, is its water supply, irrigation, hydroelectric generation, and conservation capabilities. In addition to these other objectives, engineers should strongly consider what function reservoirs could play in a given area if operated simultaneously to serve a specific purpose to which they are

assigned. For instance, one reservoir's objective could be to control flood flow while another reservoir in the same area could have the singular objective of generating electricity to the local community. The idea being that if multiple reservoirs could operate simultaneously with their own purpose assigned to them by the optimization/simulation model, any conflict that would have existed between reservoirs would be minimized. As engineers and others have observed on the ground in past incidences, when reservoirs function independently of other reservoirs, conflicts like the flood event we saw at the Cumberland River can occur. Thus, the desire and rationale among engineers, in particular, to modify current models to include multi-operational objectives, is critically important. In order to achieve these multi-operational objectives, I elected to use a heuristic method of optimization rather than the traditional gradient approach.

Specifically, I chose to use the Genetic Algorithm, a perfectly suitable heuristic method for my research. However, there are others that would have been as equally interesting to explore, like Simulated Annealing, Ant Colony Optimization, Bee Algorithm, Tabu Search, or the Particle Swarm Optimization methods, for example. More importantly, however, is the reason I elected to use a heuristic method over the gradient-based approach. In recent years, heuristic search methods have gained significant attention from researchers. This is because they often produce many nondominated, optimal solutions simultaneously in multi-objective optimization problems, as compared to traditional gradient methods, such as linear and nonlinear programming, which can only produce a locally converged optimal solution. Heuristic methods provide water resource managers many possible optimal solutions from which to

choose the best solution. Therefore, heuristic methods are more ideal prospective tools to interpret nonlinear and multi-objective data. Moreover, most heuristic methods can be linked to the optimization/simulation model. Lastly, I recommend that the optimization/simulation model be applied to more river reservoir systems throughout the country and even globally.

Indeed, the Cumberland River flood event was the example I chose to base my research on in part because it occurred most recently. But, clearly there are many more flood events I could have studied using my optimization/simulation model. In fact, engineers applied an older version of this model in the 1980s in response to the 1952 flood event in the lower Colorado River basin. This model, designed by engineers in the 1980s and still being applied today, has proven beneficial to the region for nearly 30 years. Obviously, this gives us a strong indication that we could go further in improving upon and modernizing the optimization/simulation model to achieve even greater benefits on a much larger scale. In reality, my optimization/simulation model, particularly if some of the proposed modifications were adapted, can and should be applied to large river systems around the globe that still do not use real-time optimization models. There are hardly any reasonable justifications to not follow the example of the Colorado River basin model or create a more efficient system of modeling that could help safeguard and sustain communities worldwide.

Simply put, while determining what kind of modifications could be adapted to better meet real-world demands, I offered three recommendations for modification that seemed to logically improve upon the existing optimization/simulation model. In addition to the aforementioned recommendations, there are two other very important

areas worth exploration using the optimization/simulation model. The first of which is the issue of water quality constraints and the second is addressing sedimentation and erosion control within reservoirs.

First, water quality constraints are regularly set and enforced by various management and regulation entities to ensure compliance with quality standards. I simply propose that the optimization/simulation model be equipped to take on additional functions in order to diminish the conflict between other reservoir functions and to make it easier for reservoir managers to comply with water quality standards.

The second area the model could be utilized for is sedimentation and erosion control. This is an important area because inadequate management of sedimentation and erosion can have fairly consequential impacts on the entire reservoir system. For example, sediment transport and erosion can cause reservoir silting that can lead to the reduction of hydropower generation, water supply, and discharge regulation. Furthermore, sediment transport and erosion can damage turbines and other hydraulic equipment. Of course, all these possibilities have costly implications. Thus, the optimization/simulation model can be used for minimizing sediment from entering the reservoir and to prevent deposition of sediment in the reservoir.

With all the recommendations offered in this research, it is imperative for researchers and engineer practitioners to work in concert with one another on their findings or recommendations rather than shield out constructive communication between them. For instance, most researchers focus on finding the best optimization algorithm for the reservoir problem even if it requires complex computations while reservoir operators

tend to look for the easiest practical solution. Indeed, it is probable that a more transparent working relationship between researchers and engineer practitioners on the ground could have prevented some of the damage sustained in the 2010 flood event in Nashville. Even today, the development of adequate real-time reservoir operation strategies during flood events is still in its infancy. Although some of these operation strategies are in the beginning phases, it should not prevent industry leaders and practitioners from communicating early on in the planning stages.

Lastly, I cannot think of a more important environmental issue that impacts our planet more than preserving the very resource that sustains all life on earth. Water is so fundamental to our way of being that it is absolutely paramount that educators not only teach students but also work with communities on ways of how *not* to take it for granted. The country is fortunate to have a relative abundance of water resources, but with the increasing occurrence of droughts and hotter temperatures, the manner in which we use water today will not be a luxury our children will have in the future. The research I have been involved in over the years and the research I would be so honored to continue as a researcher or engineer could profoundly impact our local and regional environment in such substantial ways that I could not imagine pursuing anything else. More importantly, I want to be able to replicate my passion for water sustainability within in my future students. Indeed, there is power in numbers and when we have large communities of aspiring engineers devoted to water sustainability, we begin to see meaningful policy changes in both public and private sectors and a conscientious shift in our fundamental relationship with water.

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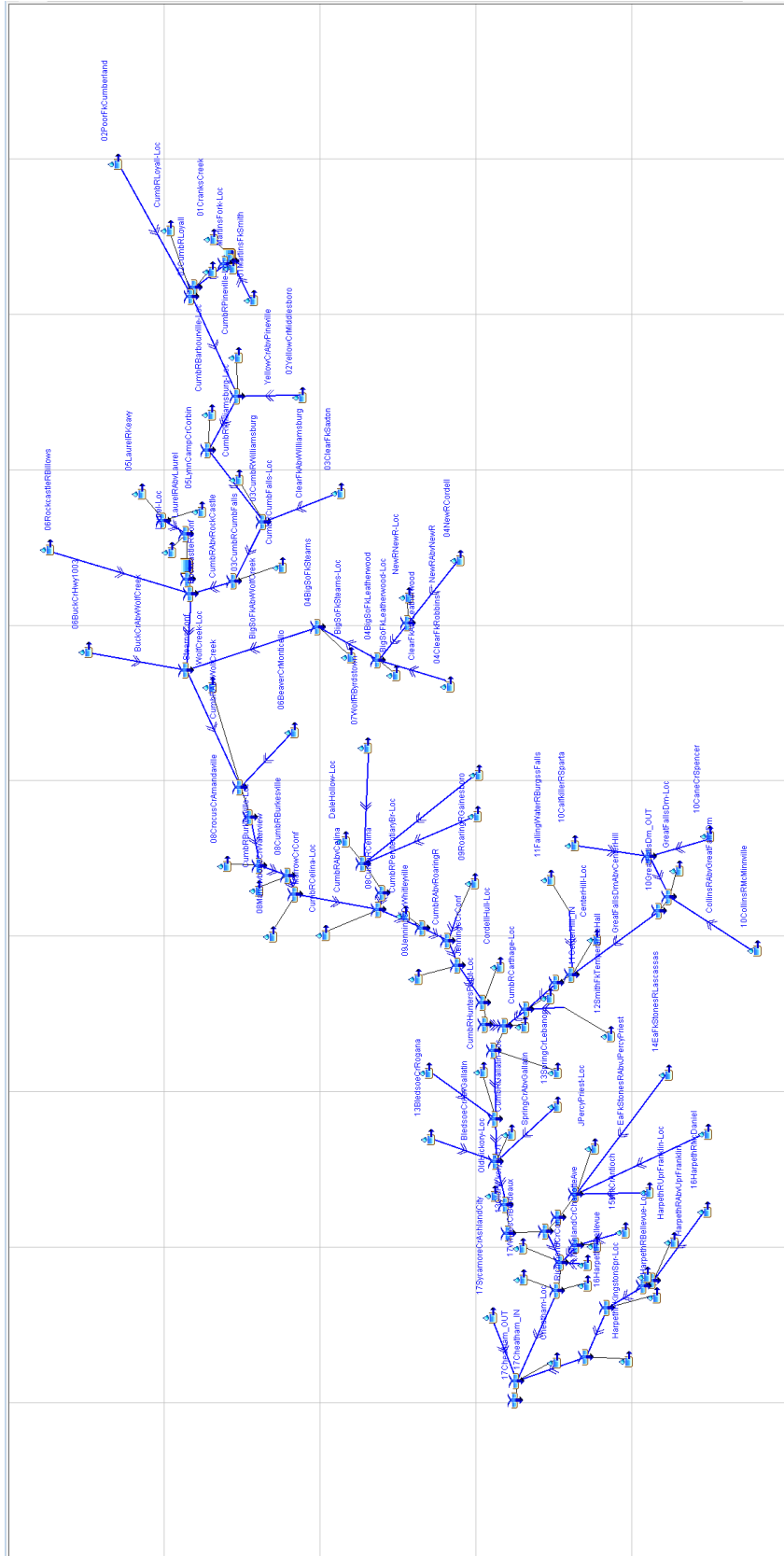
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APPENDIX A

HEC-HMS CUMBERLAND RIVER BASIN MODEL DOMAIN



APPENDIX B

HEC-HMS CUMBERLAND RIVER BASIN MODEL SUB-BASIN NAMES

Basin	Basin Names	Basin	Basin Names	Basin	Basin Names
1	CumbrLoyall-Loc	31	07WolfRByrdstown	61	17WhitesCrBordeaux
2	MartinsFkHarlan-Loc	32	CumbrCelina-Loc	62	17RichlandCrCharlotteAve
3	01CranksCreek	33	CumbrPenitentiaryBranch-Loc	63	HarpethRUpFranklin-Loc
4	01MartinsFkSmith	34	09RoaringRGainesboro	64	16HarpethRMcDaniel
5	MartinsFork-Loc	35	09JenningsCrWhitleyville	65	HarpethRLwrFranklin-Loc
6	02PoorFkCumberland	36	CordellHull-Loc	66	HarpethRBellevue-Loc
7	CumbrPineville-Loc	37	GreatFallsDm-Loc	67	HarpethRKingstonSpr-Loc
8	02YellowCrMiddle	38	10CollinsRMcMinnville	68	Cheatham-Loc
9	CumbrBarbourville-Loc	39	10CalfkillerRSparta	69	17SycamoreCrAshlandCity
10	03ClearFkSaxton	40	10CaneCrSpencer		
11	CumbrWilliamsburg-Loc	41	CenterHill-Loc		
12	CumbrCumbFalls-Loc	42	11FallingWaterRBurgessFalls		
13	06RockcastleBill	43	12SmithFkTemperanceHall		
14	05LaurelRKeavy	44	CaneyFkRStonewall-Loc		
15	05LynnCampCrCorkbin	45	CumbrCarthage-Loc		
16	Laurel-Loc	46	13RoundLickCrGrant		
17	04NewRCordell	47	CumbrHuntersPoint-Loc		
18	NewRNewR-Loc	48	13GooseCrHartsville		
19	04ClearFkRobbins	49	CumbrGallatin-Loc		
20	BigSoFkLeatherwood-Loc	50	13BledsoeCrRogana		
21	BigSoFkStearns-Loc	51	13SpringCrLebanon		
22	06BuckCrHwy1003	52	OldHickory-Loc		
23	WolfCreek-Loc	53	JPercyPriest-Loc		
24	06BeaverCrMonticello	54	14EaFkStonesRLascassas		
25	08CrocusCrAmandaville	55	14WeFkStonesRMurfree		
26	CumbrBurkesville-Loc	56	14StewartCrSmyrna		
27	08MarrowboneCr	57	CumbrNashville-Loc		
28	Waterview	58	15MillCrAntioch		
29	DaleHollow-Loc	59	MillCrNashville-Loc		
30	07EaFkObeyRJamestown	60	15BrownsCrFairgrounds		
	WeFkObeyRALpin				

APPENDIX C

HEC-HMS CUMBERLAND RIVER BASIN MODEL SUB-BASIN AREAS

Subbasin	Area (mi²)	Subbasin	Area (mi²)
CumbRLoyall-Loc	173.85	CordellHull-Loc	319.36
MartinsFkHarlan-Loc	61.3	GreatFallsDm-Loc	731.73
01CrankCreek	24.89	10CollinsRMcMinnville	642.19
01MartinsFkSmith	23.67	10CalfkillerRSparta	172.24
MartinsFork-Loc	7.09	10CaneCrSpencer	133.73
02PoorFkCumberland	82.23	CenterHill-Loc	370.17
CumbRPineville-Loc	338.09	11FallingWaterRBurgssFalls	119.58
02YellowCrMiddlesboro	71.54	12SmithFkTemperanceHall	214.14
CumbRBarbourville-Loc	186.76	CaneyFkRStonewall-Loc	140.8
03ClearFkSaxton	320.17	CumbRCarthage-Loc	65.66
CumbRWilliamsburg-Loc	316.95	13RoundLickCrGrant	50.99
CumbRCumbFalls-Loc	369.71	CumbRHuntersPoint-Loc	302.68
06RockcastleRBillows	603.93	13GooseCrHartsville	64.39
05LaurelRKeavy	139.43	CumbRGallatin-Loc	168.94
05LynnCampCrCorbin	53.04	13BledsoeCrRogana	77.42
Laurel-Loc	89.5	13SpringCrLebanon	43.99
04NewRCordell	198.23	OldHickory-Loc	275.43
NewRNewR-Loc	184.16	JPercyPriest-Loc	384.49
04ClearFkRobbins	271.93	14EaFkStonesRLascassas	261.56
BigSoFkLeatherwood-Loc	151.99	14WeFkStonesRMurfreesboro	176.64
BigSoFkStearns-Loc	147.2	14StewartCrSmyrna	69.92
06BuckCrHwy1003	253.65	CumbRNashville-Loc	170.56
WolfCreek-Loc	1675.86	15MillCrAntioch	64.11
06BeaverCrMonticello	43.192	MillCrNashville-Loc	29.32
08CrocusCrAmandaville	86.67	15BrownsCrFairgrounds	11.95
CumbRBurkesville-Loc	144.24	17WhitesCrBordeaux	51.36
08MarrowboneCrWaterview	67.9	17RichlandCrCharlotteAve	24.7
DaleHollow-Loc	513.16	HarpethRUprFranklin-Loc	124.08
07EaFkObeyRJamestown	201.64	16HarpethRMcDaniel	66.57
07WeFkObeyRALpine	115.47	HarpethRLwrFranklin-Loc	19.69
07WolfRByrdstown	105.86	HarpethRBellevue-Loc	198.16
CumbRCelina-Loc	283.03	HarpethRKingstonSpr-Loc	272.97
CumbRPenitentiaryBr-Loc	130.71	Cheatham-Loc	462.42
09RoaringRGainesboro	275.14	17SycamoreCrAshlandCity	97.29
09JenningsCrWhitleyville	67.28		

APPENDIX D
HEC-HMS CUMBERLAND RIVER BASIN MODEL SUB-BASIN LOSS METHOD
PARAMETERS

Deficit Constant Loss Method

Subbasins	Initial Deficit (in)	Max Storage (in)	Constant Rate (in/hr)	Impervious (%)
CumbRLoyall-Loc	0.6	1.2	0.01	0
MartinsFkHarlan-Loc	0.6	1.1	0.02	0
01CranksCreek	0.6	1.51	0.01	0
01MartinsFkSmith	0.2	1.1	0.03	0
MartinsFork-Loc	0.42	1.1	0.01	7.22
02PoorFkCumberland	0.4	3	0.05	0
CumbRPineville-Loc	0.2	0.9	0.06	0
02YellowCrMiddlesboro	0.4	0.76	0	0
CumbRBarbourville-Loc	0.35	2	0	0
03ClearFkSaxton	0.37	1.25	0.04	0
CumbRWilliamsburg-Loc	0.45	0.9	0.02	0
CumbRCumbFalls-Loc	1.2	3	0	0
06RockcastleRBillows	1	0.84	0	0
05LaurelRKeavy	1	2.5	0	0
05LynnCampCrCorbin	0.77	1.02	0	0
Laurel-Loc	0.9	1	0	10.09
04NewRCordell	0.8	1.3	0	0
NewRNewR-Loc	0.8	1.3	0	0
04ClearFkRobbins	1.15	1.5	0.02	0
BigSoFkLeatherwood-Loc	0.8	1.3	0	0
BigSoFkStearns-Loc	0.8	1.3	0	0
06BuckCrHwy1003	1	1	0	0
WolfCreek-Loc	0.6	1.2	0	4.6
06BeaverCrMonticello	0	1.16	0	0
08CrocusCrAmandaville	0.3	1.14	0	0
CumbRBurkesville-Loc	0.3	1.35	0	0
08MarrowboneCrWatervie w	0.3	3	0	0
DaleHollow-Loc	1.4	1.4	0.01	8.43
07EaFkObeyRJamestown	1.2	1.83	0.02	0
07WeFkObeyRALpine	1.2	1.43	0.02	0
07WolfRByrdstown	2.2	3	0.01	0
CumbRCelina-Loc	0.5	3	0	0
CumbRPenitentiaryBr-Loc	0.7	1.8	0	0

Deficit Constant Loss Method

Subbasins	Initial Deficit (in)	Max Storage (in)	Constant Rate (in/hr)	Impervious (%)
09RoaringRGainesboro	1.45	2.62	0.06	0
09JenningsCrWhitleyville	1.6	3	0	0
CordellHull-Loc	0.5	1.8	0	5.85
GreatFallsDm-Loc	0.88	1.7	0.08	0
10CollinsRMcMinnville	1.5	2	0.08	0
10CalfkillerRSparta	0.88	1.3	0.09	0
10CaneCrSpencer	0.88	1.7	0.08	0
CenterHill-Loc	0.7	1.8	0.08	7.54
11FallingWaterRBurgssFalls	0.8	1.8	0.085	0
12SmithFkTemperanceHall	0.57	1.75	0.13	0
CaneyFkRStonewall-Loc	0	3	0	0
CumbRCarthage-Loc	0	3	0	0
13RoundLickCrGrant	0.1	1.8	0.04	0
13GooseCrHartsville	0	1.8	0.08	0
13BledsoeCrRogana	0	1.8	0	0
13SpringCrLebanon	0	1.55	0	0
JPercyPriest-Loc	1.1	1.8	0.025	5.77
14EaFkStonesRLascassas	1.15	1.8	0.04	0
14WeFkStonesRMurfreesboro	1.75	2	0	0
14StewartCrSmyrna	1.75	1.8	0	0
CumbRNashville-Loc	0.15	1.8	0.02	10
15MillCrAntioch	0	3	0	0
MillCrNashville-Loc	0	2.8	0	0
15BrownsCrFairgrounds	0.1	3.7	0	0
17WhitesCrBordeaux	0.5	1.82	0	0
17RichlandCrCharlotteAve	0.15	1.9	0	0
HarpethRUpFranklin-Loc	0.9	1.9	0.06	0
16HarpethRMcDaniel	0.5	1.9	0.03	0
HarpethRLwrFranklin-Loc	0.5	1.9	0.03	0
HarpethRBellevue-Loc	0.5	1.9	0.03	0
HarpethRKingstonSpr-Loc	0.5	1.9	0.03	0
17SycamoreCrAshlandCity	0.2	2.09	0	0

Initial Constant Loss Method

Subbains	Initial Loss (in)	Constant Rate (in/hr)	Impervious (%)
CumbRHuntersPoint-Loc	0	0	0
CumbRGallatin-Loc	0	0	0
OldHickory-Loc	0	0	0

Soil Moisture Accounting Method

Subbasin	Canpony (%)	Surface (%)	Soil (%)
Cheatham-Loc	0	0	5
	Groundwater 1 (%)	Groundwater 2 (%)	Canopy Storage (in)
	30	40	0.1
	Surface Storage (in)	Max Infiltration (in/hr)	Impervious (%)
	0.2	0.2	0
	Soil Storage (in)	Tension Storage (in)	Soil Percolation (in/hr)
	5	0.2	0.14
	Groundwater 1 Storage (in)	Groundwater 1 Percolation (in/hr)	GW 1 Coefficient (in/hr)
	4	0.12	60
	Groundwater 2 Storage (in)	Ground 2 Percolation (in/hr)	GW 2 Coefficient (hr)
	4	0.12	90

APPENDIX E

HEC-HMS CUMBERLAND RIVER BASIN MODEL SUB-BASIN TRANSFORM

METHOD PARAMETERS

ModClark Transform Method

Subbasin	Time of Concentration (hr)	Storage Coefficient (hr)
CumbRLoyall-Loc	12.7	16.5
MartinsFkHarlan-Loc	4.9	6.4
01CranksCreek	8.2	7.3
01MartinsFkSmith	11	6.5
MartinsFork-Loc	3.6	1.8
02PoorFkCumberland	8.1	9.7
CumbRPineville-Loc	6.5	26.8
02YellowCrMiddlesboro	7.3	10.4
CumbRBarbourville-Loc	26	17.6
03ClearFkSaxton	16	27
CumbRWilliamsburg-Loc	34	26.5
CumbRCumbFalls-Loc	18	12
06RockcastleRBillows	10	22.4
05LaurelRKeavy	5.1	26
05LynnCampCrCorbin	7.2	18.8
Laurel-Loc	9.4	9.4
04NewRCordell	9.7	5.8
NewRNewR-Loc	17.2	10.3
04ClearFkRobbins	14.7	11.7
BigSoFkLeatherwood-Loc	14	14
BigSoFkStearns-Loc	10.3	10.3
06BuckCrHwy1003	10.8	10.8
WolfCreek-Loc	22.4	11
06BeaverCrMonticello	7.1	10.5
08CrocusCrAmandaville	4	3
CumbRBurkesville-Loc	6.8	4
08MarrowboneCrWaterview	11	12.5
DaleHollow-Loc	8	7
07EaFkObeyRJamestown	10.7	7.7
07WeFkObeyRALpine	7.8	9.2
07WolfRByrdstown	6	7.3
CumbRCelina-Loc	5	6
CumbRPenitentiaryBr-Loc	3	5
09RoaringRGainesboro	4.6	16.8

ModClark Transform Method

Subbasin	Time of Concentration (hr)	Storage Coefficient (hr)
09JenningsCrWhitleyville	5	3
CordellHull-Loc	6.9	7.8
GreatFallsDm-Loc	17.7	23.1
10CollinsRMcMinnville	19.4	21.6
10CalfkillerRSparta	19	19
10CaneCrSpencer	6.4	6.4
CenterHill-Loc	10.6	10.6
11FallingWaterRBurgssFalls	14.5	14.6
12SmithFkTemperanceHall	9.4	8.1
CaneyFkRStonewall-Loc	5	8
CumbRCarthage-Loc	4.7	7.5
13RoundLickCrGrant	6	5.2
CumbRHuntersPoint-Loc	2	12
13GooseCrHartsville	8	7.4
CumbRGallatin-Loc	2	15
13BledsoeCrRogana	2.5	3.4
13SpringCrLebanon	6.5	3
OldHickory-Loc	2	12
JPercyPriest-Loc	18	12
14EaFkStonesRLascassas	12.5	9.9
14WeFkStonesRMurfreesboro	16.7	6.7
14StewartCrSmyrna	7.7	7.8
CumbRNashville-Loc	3.4	8
15MillCrAntioch	4.5	6.2
MillCrNashville-Loc	4.1	4.1
15BrownsCrFairgrounds	1	2
17WhitesCrBordeaux	3.2	3.3
17RichlandCrCharlotteAve	1	2.7
HarpethRUpFranklin-Loc	15	15
16HarpethRMcDaniel	14	15
HarpethRLwrFranklin-Loc	7	7.5
HarpethRBellevue-Loc	14	12.4
HarpethRKingstonSpr-Loc	12.2	12
Ceatham-Loc	27.2	32
17SycamoreCrAshlandCity	7.3	4

APPENDIX F

HEC-HMS CUMBERLAND RIVER BASIN MODEL SUB-BASIN BASEFLOW

METHOD PARAMETERS

Bounded Recession Baseflow

Subbasin	Initial Type	Initial Discharge (cfs)	Recession Constant	January (CFS)	February (cfs)	March (cfs)	April (cfs)	May (cfs)	June (cfs)	July (cfs)	August (cfs)	September (cfs)	October (cfs)	November (cfs)	December (cfs)		
CumbRLoyal-Loc	Discharge	400	0.98	299	282	187	440	400	68	24	22	62	69	143	247	140	240
MartinsFkHerdan-Loc	Discharge	140	0.98	106	99	66	150	140	24	24	22	66	24	50	87	60	80
01CranksCreek	Discharge	15	0.97	43	40	27	17	12	10	9	9	10	10	20	35	24	40
01MartinsFkSmith	Discharge	14	0.98	41	38	25	16	11	9	8	8	9	9	19	34	20	40
MartinsFk-Loc	Discharge	4	0.98	12	11	8	5	3	3	3	3	3	3	6	10	6	10
02PoorFkCumberland	Discharge	470	0.9	200	200	200	500	220	80	80	80	40	40	80	120	80	120
CumbRPineville-Loc	Discharge	800	0.94	582	548	364	850	800	132	120	135	278	278	481	280	480	480
02YellowCkMiddlesboro	Discharge	200	0.9	120	100	80	200	200	30	30	40	60	60	80	60	80	80
CumbRBarbourville-Loc	Discharge	1900	0.89	321	303	201	2000	2000	73	66	75	153	153	266	160	260	260
03ClearFkSaxton	Discharge	1000	0.91	650	600	500	1000	1000	150	150	150	100	200	400	400	200	400
CumbRWWilliamsburg-Loc	Discharge	174	0.98	546	514	341	221	153	124	113	126	260	260	451	260	460	460
CumbRCumbFalls-Loc	Discharge	1650	0.9	636	600	398	1650	1650	145	132	147	304	304	526	300	520	520
06RockcastleRBillows	Discharge	203	0.97	1300	1100	700	200	150	150	200	200	500	500	1000	500	1000	1000
05LaurelKKeavy	Discharge	500	0.91	120	120	100	500	500	30	30	50	50	150	150	150	160	160
05LynnCampCCorbin	Discharge	120	0.97	80	80	50	120	120	20	20	10	40	40	60	100	60	100
Laurel-Loc	Discharge	36	0.98	154	145	96	62	43	35	32	36	36	73	127	80	120	120
04NewRCordell	Discharge	198	0.98	341	321	213	138	96	78	71	79	79	163	282	160	280	280
NewRNewR-Loc	Discharge	184	0.98	317	299	198	128	89	72	66	73	73	151	262	160	260	260
04ClearFkRobbins	Discharge	260	0.97	468	441	292	189	131	106	97	108	108	223	387	160	340	340
04ClearFkLeatherwood-Loc	Discharge	152	0.98	262	246	163	106	73	59	54	61	61	125	216	120	220	220
BigSoFkSteamer-Loc	Discharge	147	0.98	253	239	158	103	71	58	52	59	59	121	209	120	200	200
06BackCkHwy1003	Discharge	76	0.98	437	411	273	177	122	99	90	101	101	208	361	280	280	280
WolfCreek-Loc	Discharge	838	0.98	2885	2718	1802	1167	808	656	597	669	669	1376	2385	1380	2380	2380

Bounded Recession Baseflow

Subbasin	Initial Type	Initial Discharge (cfs)	Recession Constant	January (CFS)	February (cfs)	March (cfs)	April (cfs)	May (cfs)	June (cfs)	July (cfs)	August (cfs)	September (cfs)	October (cfs)	November (cfs)	December (cfs)
06BeaverC	Monticello	22	0.97	60	50	30	20	20	20	20	10	10	40	80	80
08MarrowboneC	Waterview	34	0.98	117	110	73	47	33	27	24	27	27	56	97	100
08Hollow-Loc	Discharge	257	0.98	883	832	552	357	248	201	183	205	205	421	730	420
07EarlObeysR	Jamestown	323	0.97	450	450	200	150	100	100	80	80	80	200	300	200
07WeikObeyR	Alpine	64	0.97	280	300	150	100	50	50	20	20	20	80	100	80
07WalrR	Byrdstown	53	0.97	320	320	200	100	50	50	50	50	100	150	200	160
09RoaringK	Gainesboro	0	0.97	300	200	140	70	20	20	40	40	40	140	350	140
09JenningsCr	Whitleyville	34	0.9	116	109	72	47	32	26	24	27	27	55	96	60
10GreatFallsDm-Loc	Discharge	1024	0.98	1260	1187	787	510	353	286	261	292	292	601	1041	600
10CollinsMc	Minnville	878	0.97	1100	1100	800	450	450	450	350	350	350	600	800	600
10GalkillerR	Sparta	272	0.97	296	279	185	120	83	67	61	69	69	141	245	200
10CaneCr	Spencer	187	0.98	230	217	144	93	65	52	48	53	53	110	190	120
CenterHill-Loc	Discharge	555	0.98	637	600	398	258	179	145	132	148	148	304	527	300
11FallingWaterR	BurgessFalls	191	0.98	300	300	200	200	120	100	60	60	60	100	300	100
13BledsoeCr	Rogana	35	0.98	80	70	40	40	20	20	20	20	20	70	70	80
13SpringCr	Lebanon	23	0.97	50	50	40	10	10	10	10	10	10	30	100	30
JPercyPriest-Loc	Discharge	423	0.98	662	624	414	268	185	150	137	153	153	316	547	320
14EarlStonesR	Lascassas	190	0.97	250	200	150	100	50	50	50	50	100	100	150	100
14WeikStonesR	Murfreesboro	266	0.97	250	200	100	100	80	60	60	60	60	100	200	100
14StewartCr	Smyrna	35	0.98	120	113	75	49	34	27	25	28	28	57	100	40
14MillCr	Nashville-Loc	15	0.98	50	48	32	20	14	11	10	12	12	24	42	24
15BrownsCr	Fairgrounds	8	0.97	21	19	13	8	6	5	4	5	5	10	17	6
HamphR	LwrFranklin-Loc	12	0.98	34	32	21	14	9	8	7	8	8	16	28	16
17SycamoreC	AshlandCity	10	0.98	167	158	105	68	47	38	35	39	39	80	138	60

Linear Reservoir Baseflow

Subbasin	Initial Type	GW 1 Initial (cfs)	GW 1 Coefficient (hr)	GW 1 Reservoirs
Ceatham-Loc	Discharge	200	100	1
		GW 2 Initial (cfs)	GW 2 Coefficient (hr)	GW 2 Reservoirs
		200	150	1

APPENDIX G

HEC-HMS CUMBERLAND RIVER BASIN MODEL REACH NAMES

Reach #	Reach Name	Reach #	Reach Name
1	CranksCreekAbvMartinsFork	34	RoaringRAbvCordellHull
2	MartinsFkAbvMartinsFork	35	CumbRAbvJenningsCr
3	MartinsForkAbvHarlan	36	CumbrRAbvCordellHull
4	MartinsFkAbvLoyall	37	CumbRAbvCarthage
5	PoorFkAbvLoyall	38	CollinsRAbvGreatFallsDm
6	CumbRAbvPineville	39	CalfkillerRAbvCaneCr
7	YellowCrAbvPineville	40	CaneCrAbvGreatFallsDm
8	CumbRAbvBarbourville	41	CalfkillerRAbvGreatFalls
9	CumbRAbvWilliamsburg	42	GreatFallsDmAbvCenterHill
10	ClearFkAbvWilliamsburg	43	FallingWaterRAbvCenterHill
11	CumbRAbvCumbFalls	44	CaneyFkAbvStonewall
12	CumbRAbvRockCastle	45	SmithFkAbvStonewall
13	RockcastleRAbvWolfCreek	46	CaneyFkRAbvCarthage
14	LaurelRAbvLaurel	47	CumbRAbvHuntersPoint
15	CumbRAbvStearns	48	GooseCrAbvHuntersPoint
16	NewRAbvNewR	49	CumbRAbvGallatin
17	NewRAbvLeatherwood	50	BledsoeCrAbvGallatin
18	ClearFkAbvLeatherwood	51	SpringCrAbvGallatin
19	BigSoFkAbvStearns	52	CumbRAbvOldHickory
20	BigSoFkAbvWolfCreek	53	EaFkStonesRAbvJPercyPriest
21	BuckCrAbvWolfCreek	54	WeFkStonesRAbvJPercyPriest
22	CumbRAbvWolfCreek	55	StewartCrAbvJPercyPriest
23	BeaverCrAbvWolfCreek	56	CumbRAbvNashville
24	CumbRAbvCrocusConf	57	UprMillCrAbvNashville
25	CumbRAbvBurkesville	58	LwrMillCrAbvNashville
26	CumbRAbvMarrowCrConf	59	BrownsCrAbvNashville
27	CumbRAbvCelina	60	CumbRAbvCheatham
28	EaFkObeyRAbvDaleHollow	61	HarpethRAbvUprFranklin
29	WeFkObeyRAbvDaleHollow	62	HarpethRAbvLwrFranklin
30	WolfRAbvDaleHollow	63	HarpethRAbvBellevue
31	DaleHollowAbvCelina	64	HarpethRAbvKingstonSpr
32	CumbRAbvPenitentiaryBr	65	HarpethRAbvCheatham
33	CumbRAbvRoaringR	66	SycamoreCrAbvCheatham

APPENDIX H
HEC-HMS CUMBERLAND RIVER BASIN MODEL REACH ROUTING
PARAMETERS

Muskingum Routing Method

Reach	Muskingum K (hr)	Muskingum X	Number of Sub reaches
CranksCreekAbvMartinsFork	1	0.2	1
MartinsFkAbvMartinsFork	1	0.1	1
MartinsForkAbvHarlan	8	0.1	2
MartinsFkAbvLoyall	2	0.1	1
PoorFkAbvLoyall	4	0.1	2
CumbRAbvPineville	7	0	3
YellowCrAbvPineville	9	0	2
ClearFkAbvWilliamsburg	9	0	2
CumbRAbvRockCastle	2	0.1	1
LaurelRAbvLaurel	3	0	1
CumbRAbvStearns	1	0.1	1
NewRAbvNewR	4	0	2
NewRAbvLeatherwood	2	0.1	1
ClearFkAbvLeatherwood	2	0.1	1
BigSoFkAbvStearns	5	0	1
BigSoFkAbvWolfCreek	3	0.1	1
BuckCrAbvWolfCreek	5	0.1	1
CumbRAbvWolfCreek	4	0	1
BeaverCrAbvWolfCreek	5	0.1	3
CumbRAbvCrocusConf	5	0	2
CumbRAbvBurkesville	3	0	1
CumbRAbvMarrowCrConf	4	0	1
CumbRAbvCelina	12	0	4
EaFkObeyRAbvDaleHollow	3	0	1
WeFkObeyRAbvDaleHollow	4	0	1
WolfRAbvDaleHollow	3	0.1	1
DaleHollowAbvCelina	4	0	1
CumbRAbvPenitentiaryBr	2	0.2	1
CumbRAbvRoaringR	1	0.2	1
RoaringRAbvCordellHull	3	0.1	1
CumbRAbvJenningsCr	1	0.2	1

Muskingum Routing Method

Reach	Muskingum K (hr)	Muskingum X	Number of Subreaches
CumbrRAbvCordellHull	1	0.2	1
CumbRAbvCarthage	2	0	1
CollinsRAbvGreatFallsDm	4	0.1	1
CalfkillerRAbvCaneCr	3	0.1	1
CaneCrAbvGreatFallsDm	2	0.1	1
CalfkillerRAbvGreatFalls	3	0.1	1
GreatFallsDmAbvCenterHill	2	0	1
FallingWaterRAbvCenterHill	2	0	1
CaneyFkAbvStonewall	4	0	1
SmithFkAbvStonewall	4	0	1
CaneyFkRAbvCarthage	4	0	2
CumbRAbvHuntersPoint	9	0.1	2
GooseCrAbvHuntersPoint	5	0.1	1
CumbRAbvGallatin	5	0.1	1
BledsoeCrAbvGallatin	4	0.1	1
SpringCrAbvGallatin	6	0.05	2
CumbRAbvOldHickory	6	0.1	2
EaFkStonesRAbvJPercyPriest	7	0	2
WeFkStonesRAbvJPercyPriest	4	0	1
StewartCrAbvJPercyPriest	3	0.1	1
CumbRAbvNashville	5	0.1	1
UprMillCrAbvNashville	1	0	1
LwrMillCrAbvNashville	7	0	1
BrownsCrAbvNashville	8	0	2
CumbRAbvCheatham	5	0.18	4
HarpethRAbvUprFranklin	8	0.2	8
HarpethRAbvLwrFranklin	1.5	0.15	1
HarpethRAbvBellevue	4	0	1
HarpethRAbvKingstonSpr	15	0.1	3
HarpethRAbvCheatham	18	0.1	6
SycamoreCrAbvCheatham	3	0.1	1

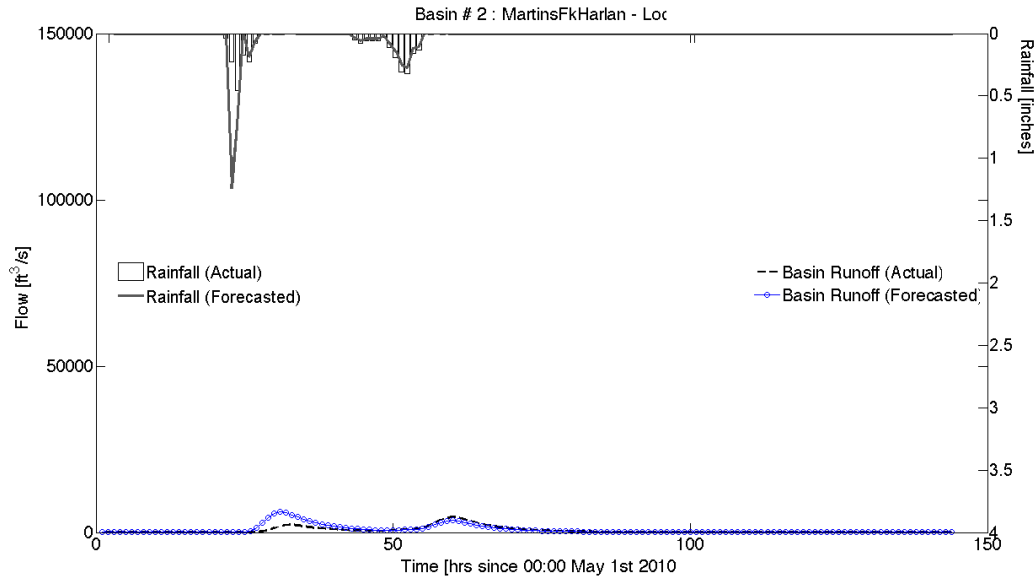
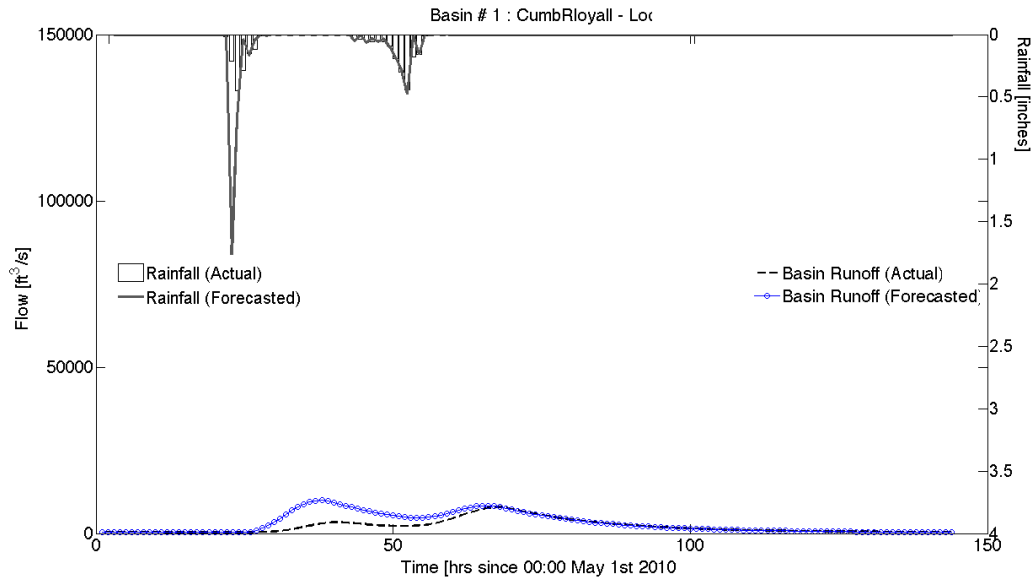
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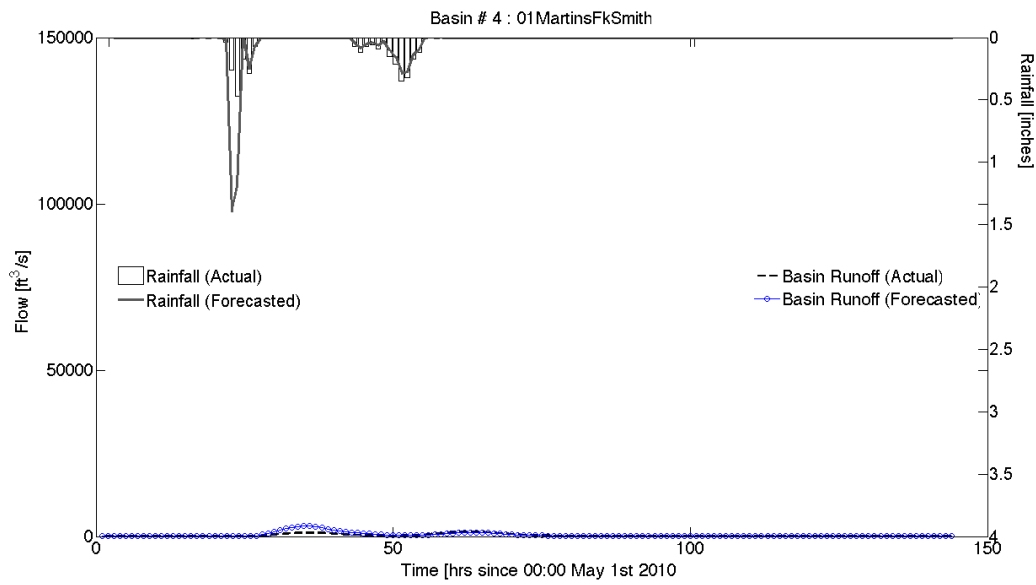
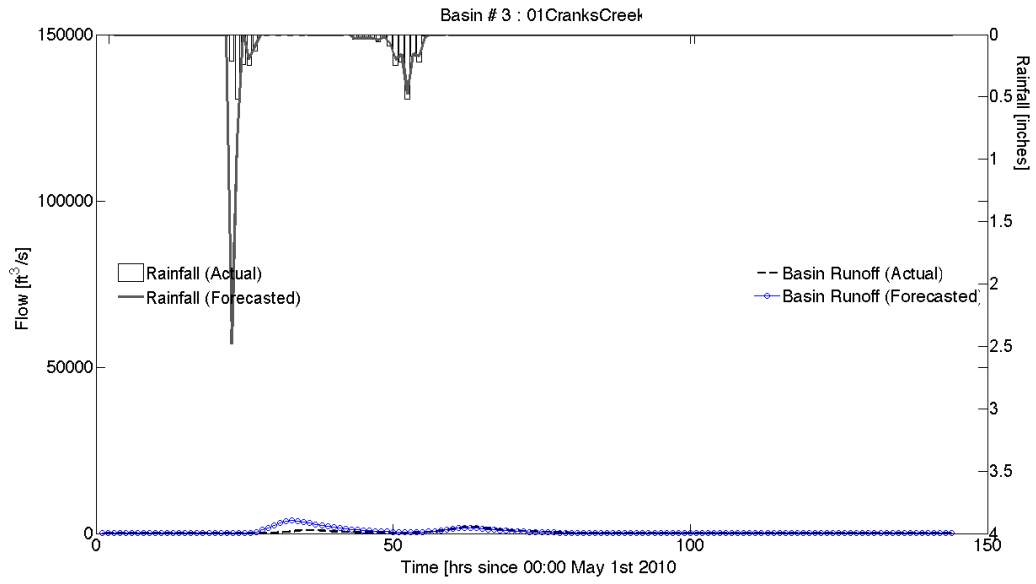
Reach	Length (ft)	Slope (ft/ft)	Manning n	Invert (ft)	Shape	L.B. Manning n	R.B. Manning n	Cross Section Table
CumbRAbvBarbourville	93472	0.0005	0.06	0	Eight Point	0.3	0.2	Barbourville
CumbRAbvWilliamsburg	116000	0.0002	0.06	0	Eight Point	0.17	0.17	Williamsburg
CumbRAbvCumbFalls	33000	0.0002	0.06	0	Eight Point	0.17	0.17	Williamsburg

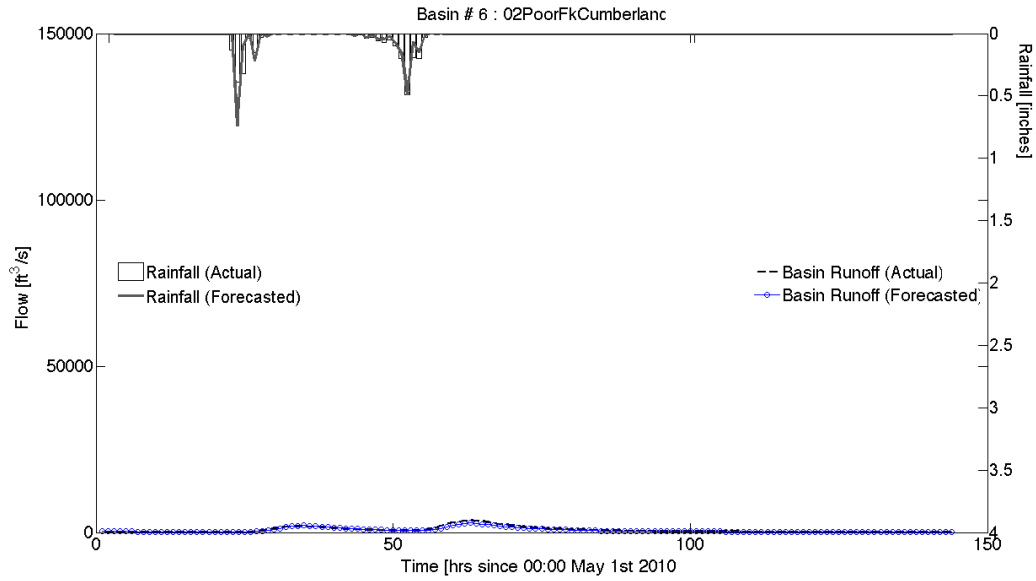
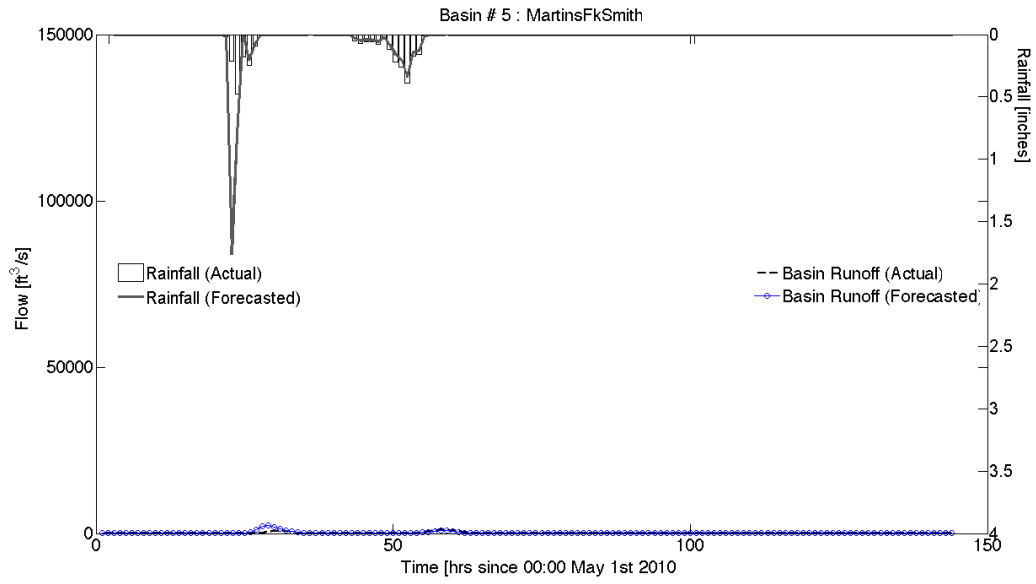
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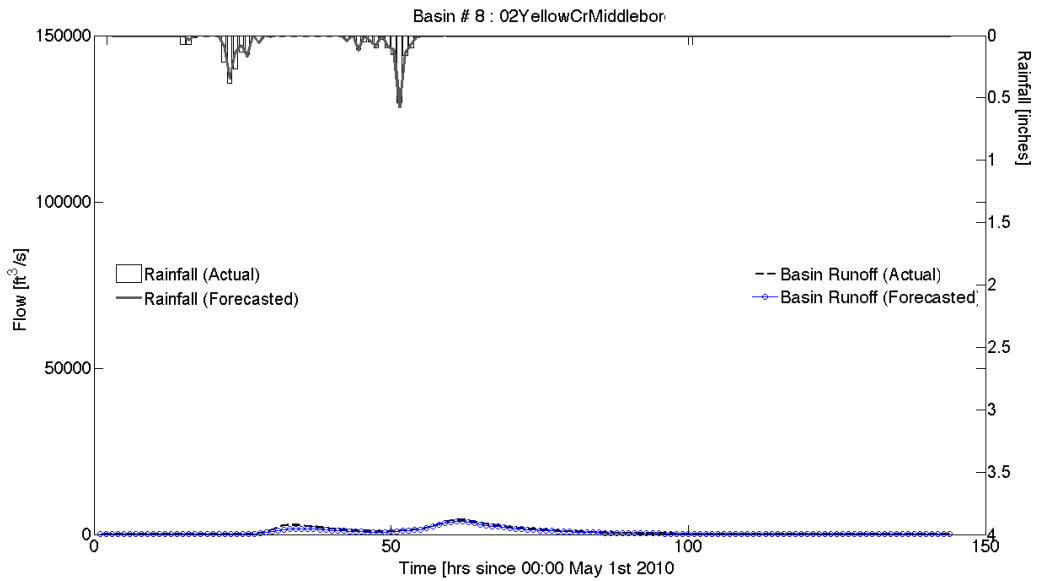
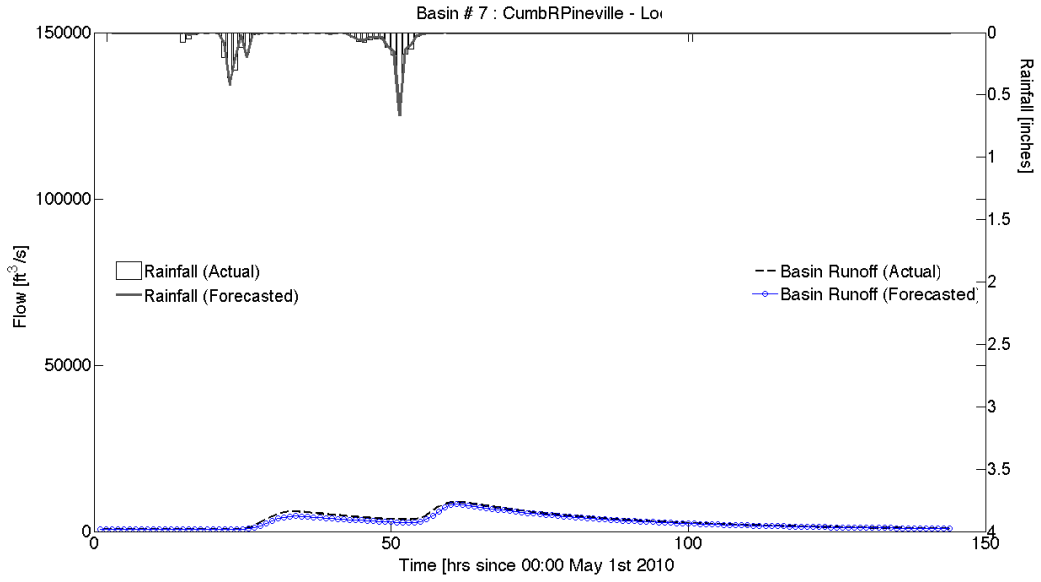
Reach	Storage-Distance Function	Subreaches	Initial
RockcastleRAbvWolfCreek	RockcastleBillows	1	Inflow = Outflow

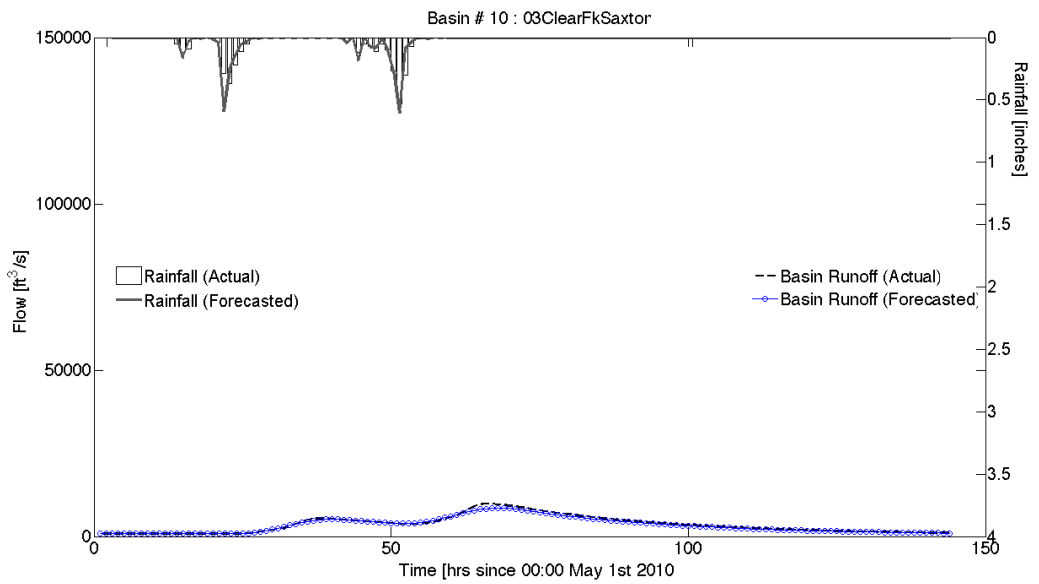
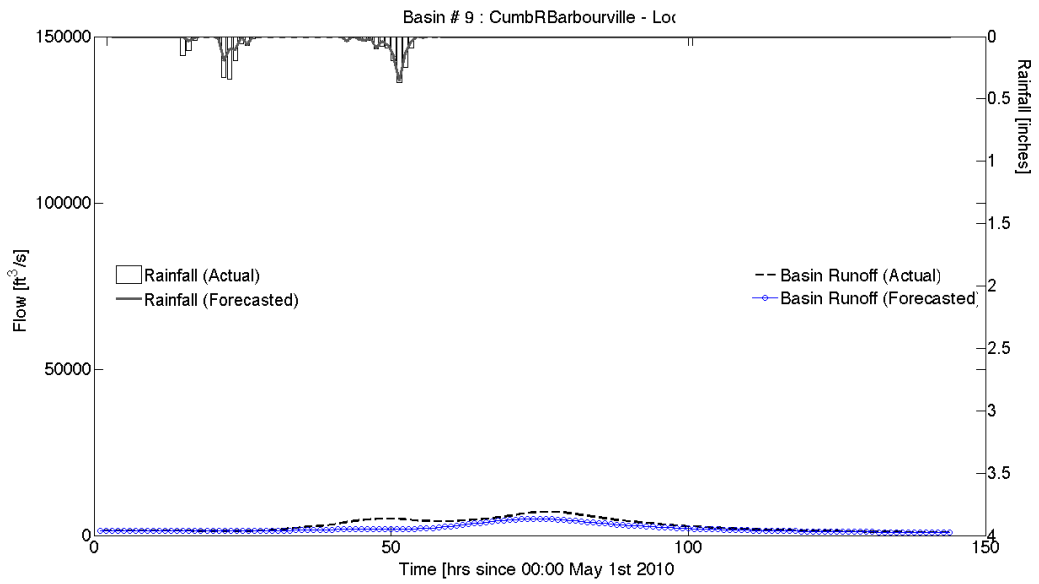
APPENDIX I
HEC-HMS CUMBERLAND RIVER BASIN MODEL RAINFALL-RUNOFF
RESULTS

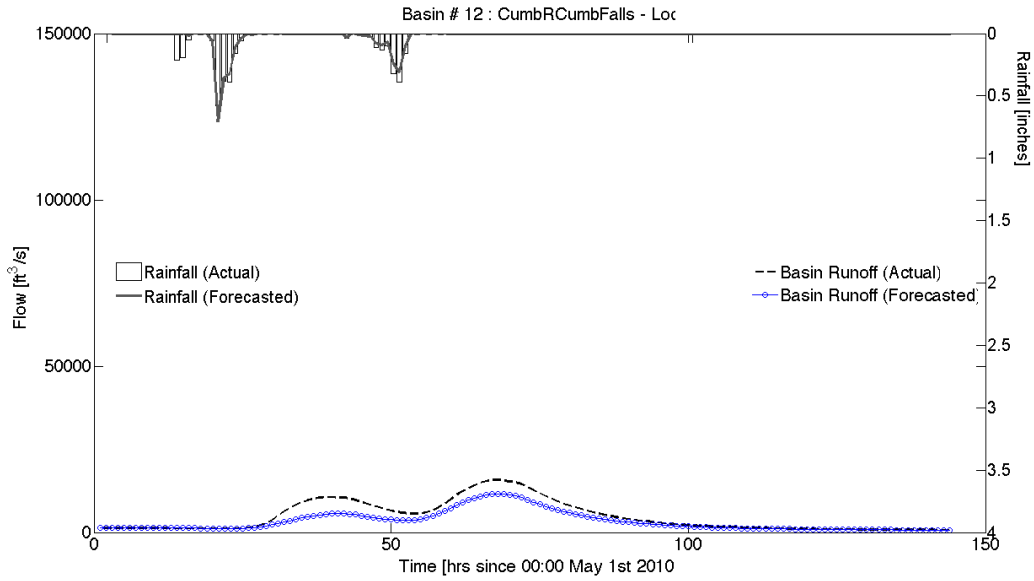
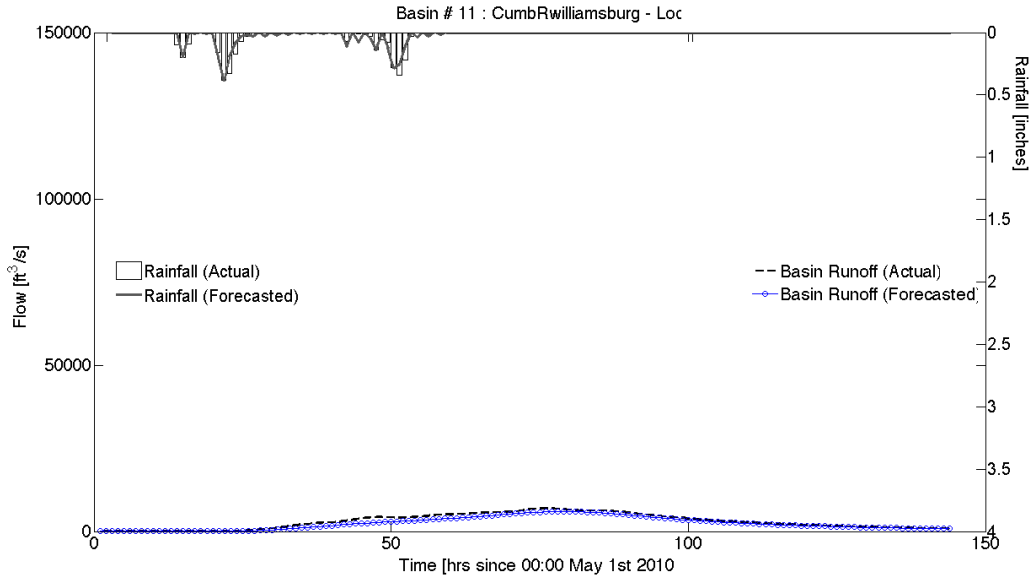


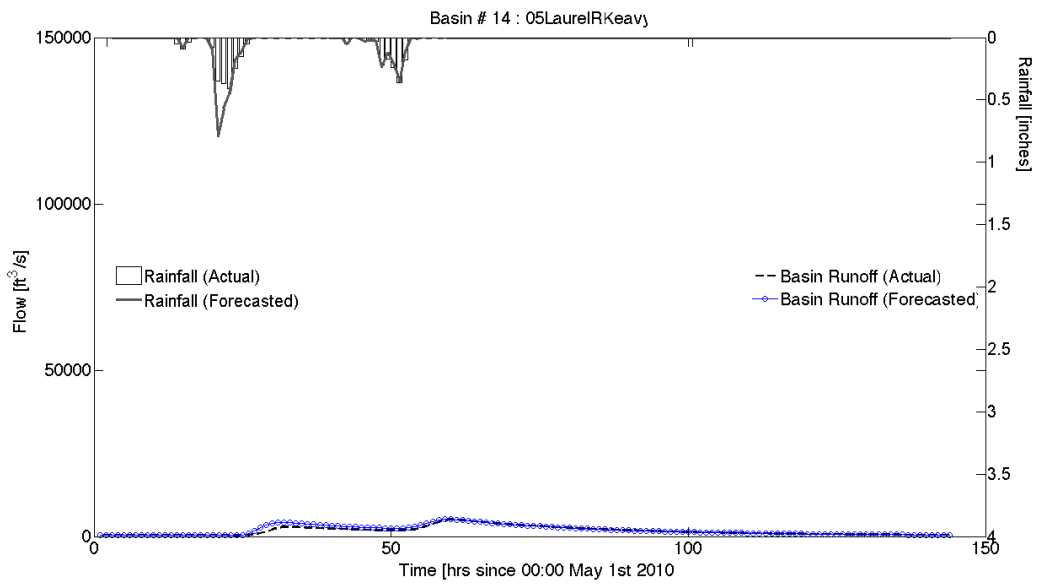
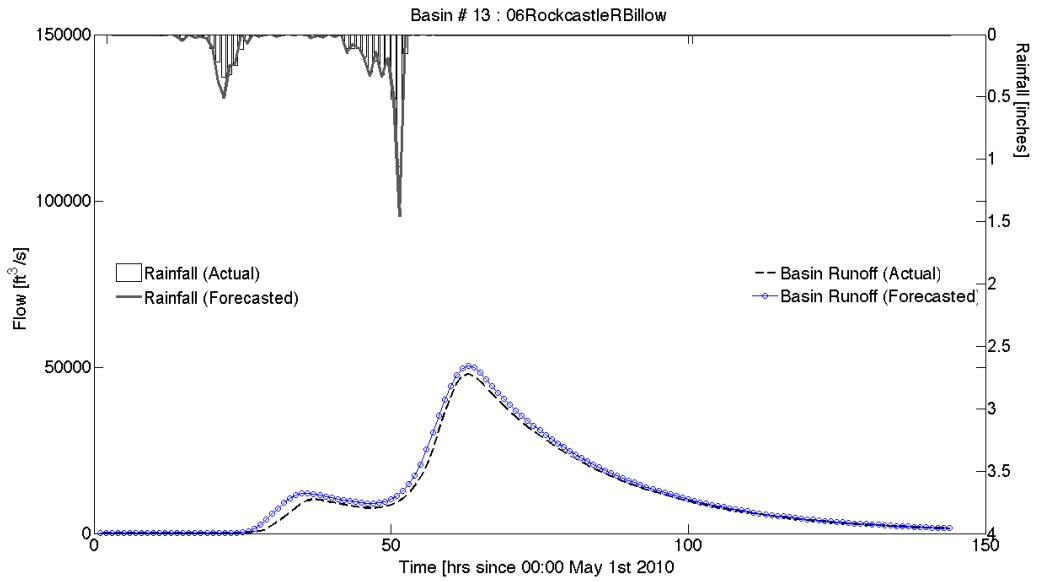


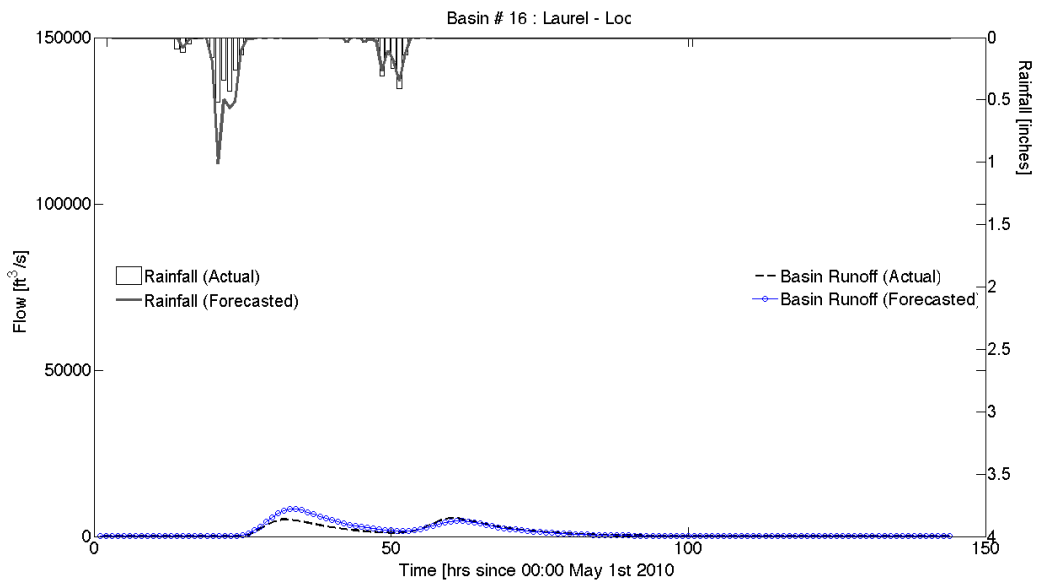
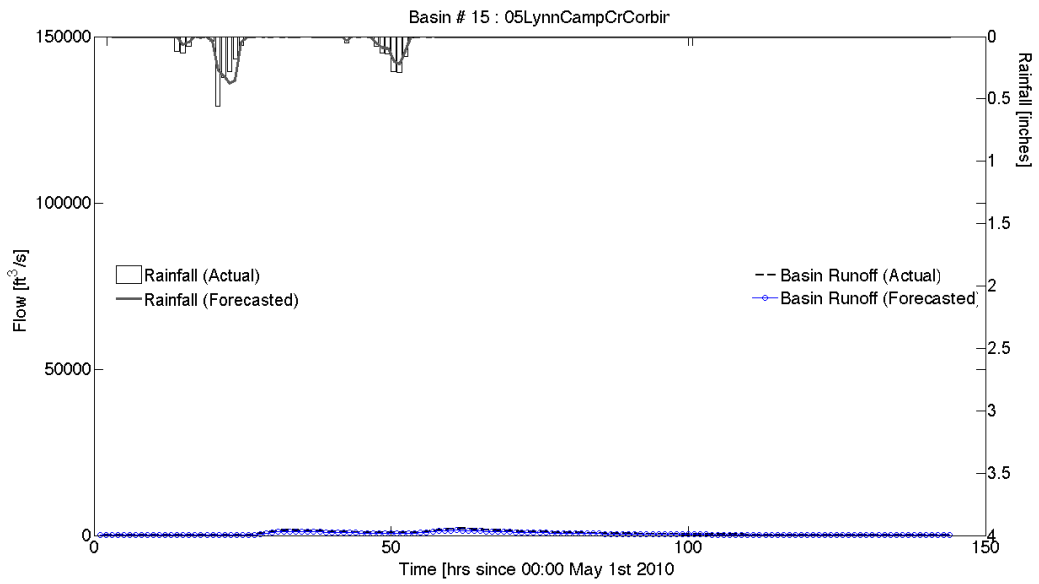


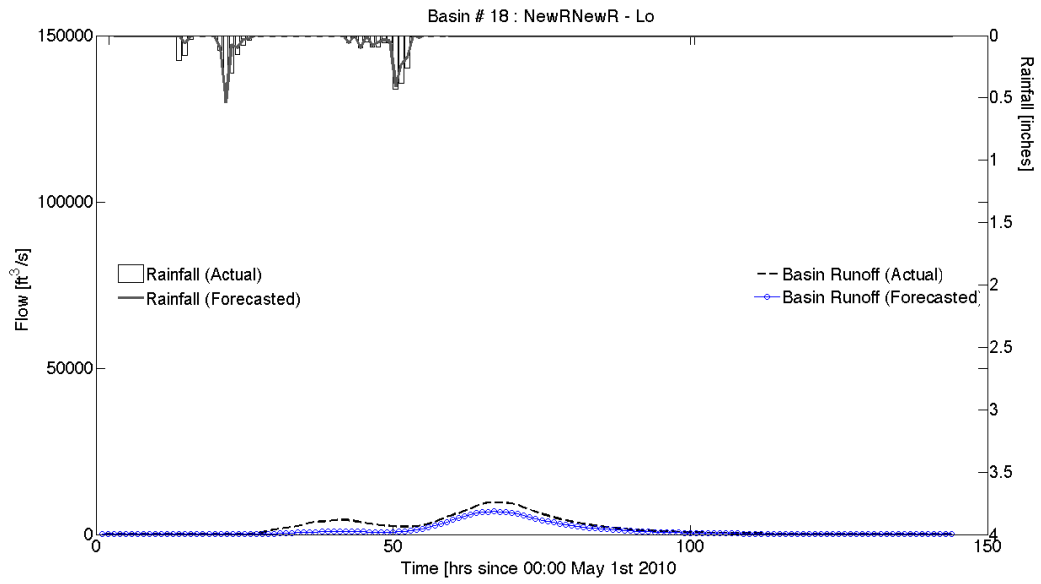
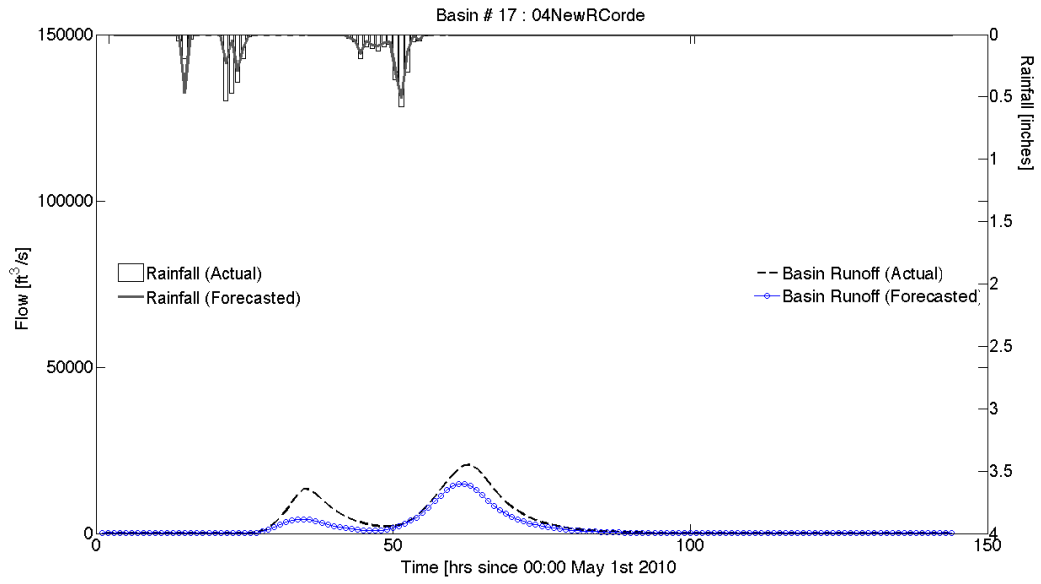


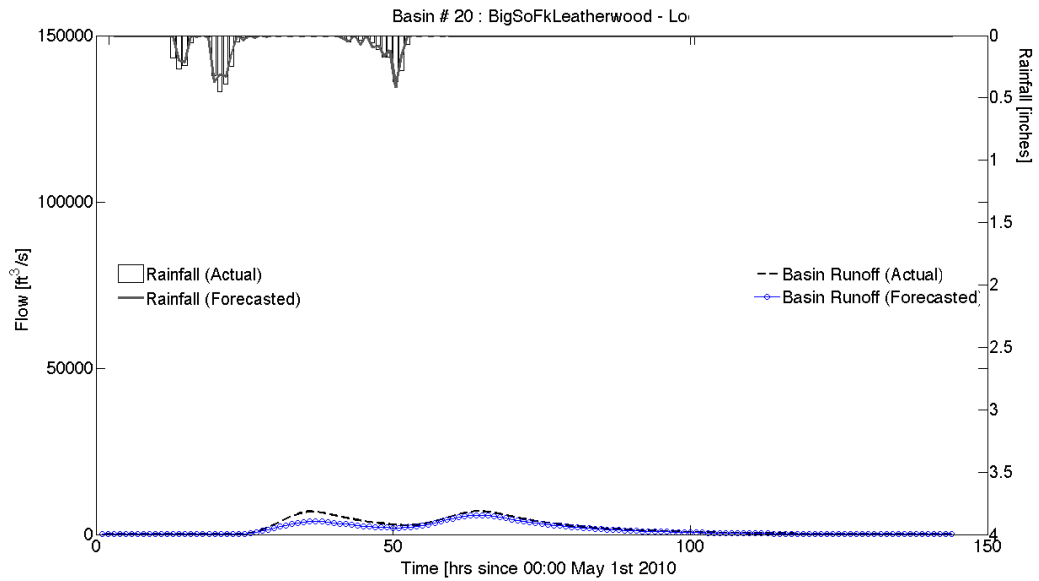
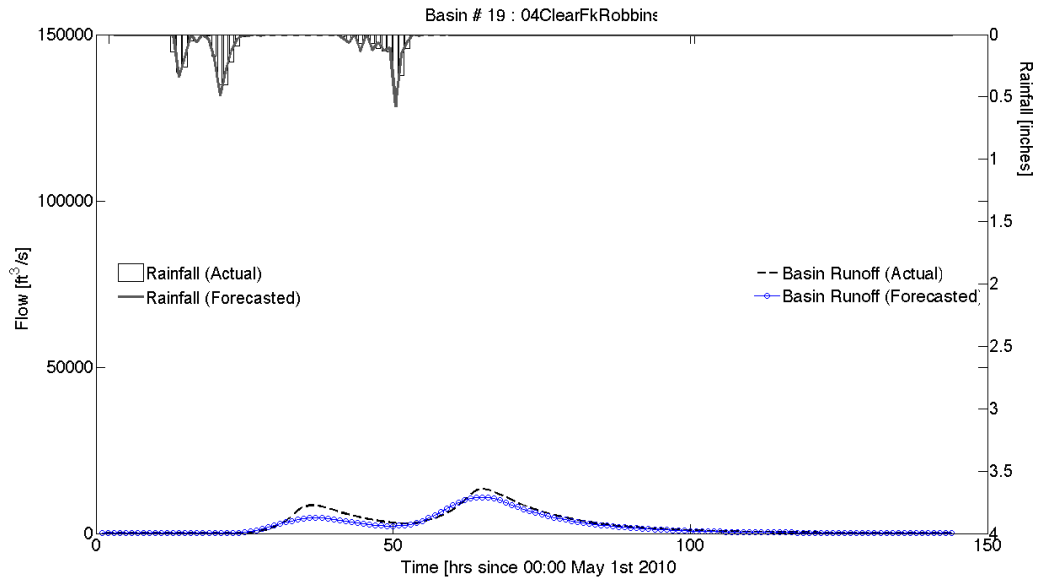


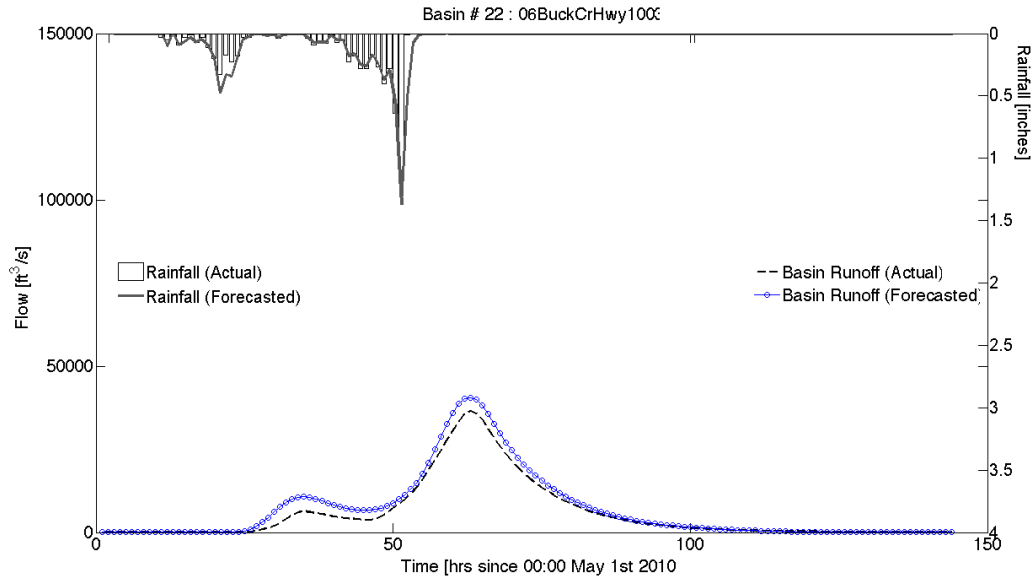
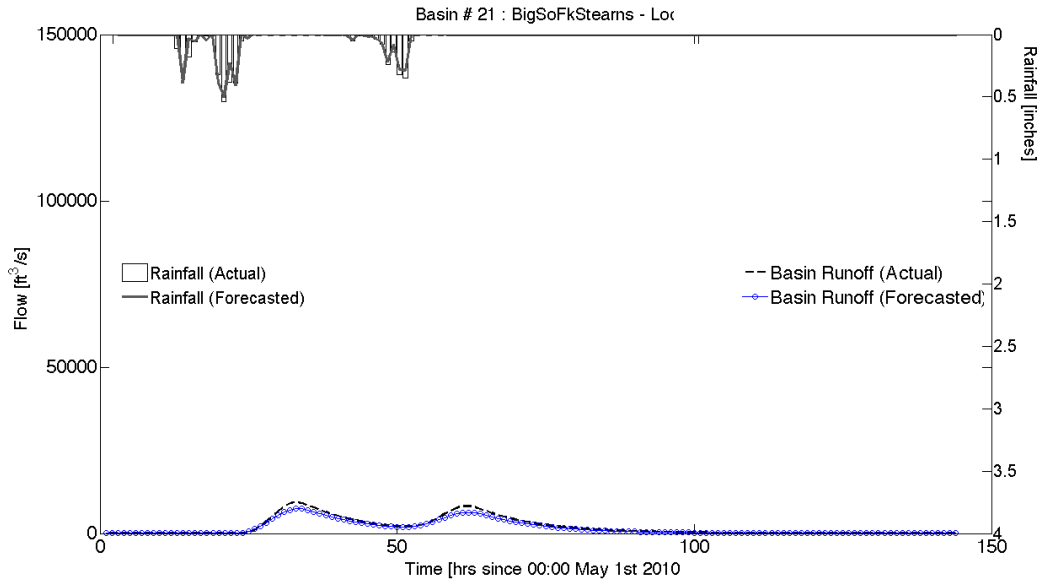


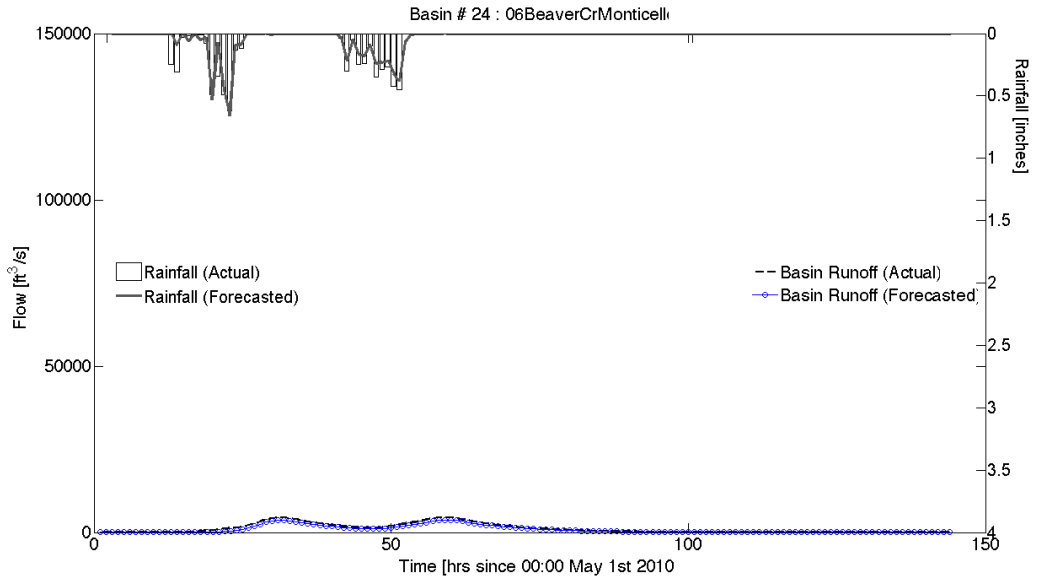
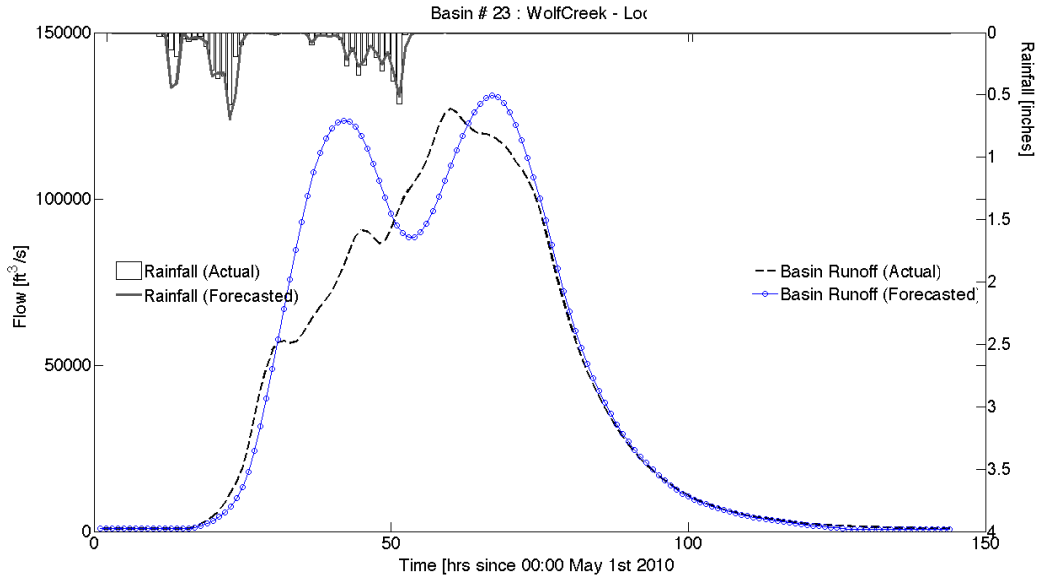


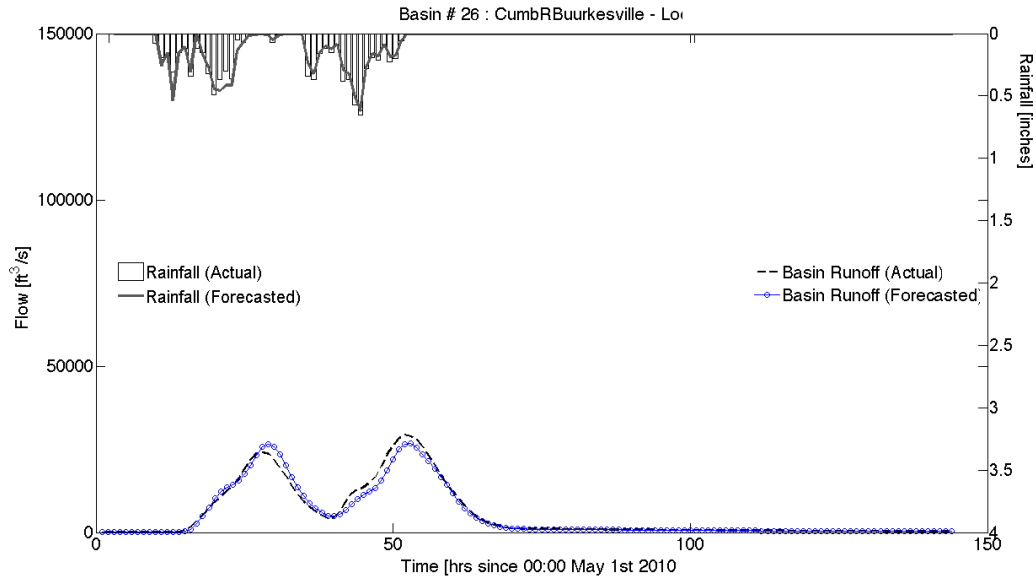
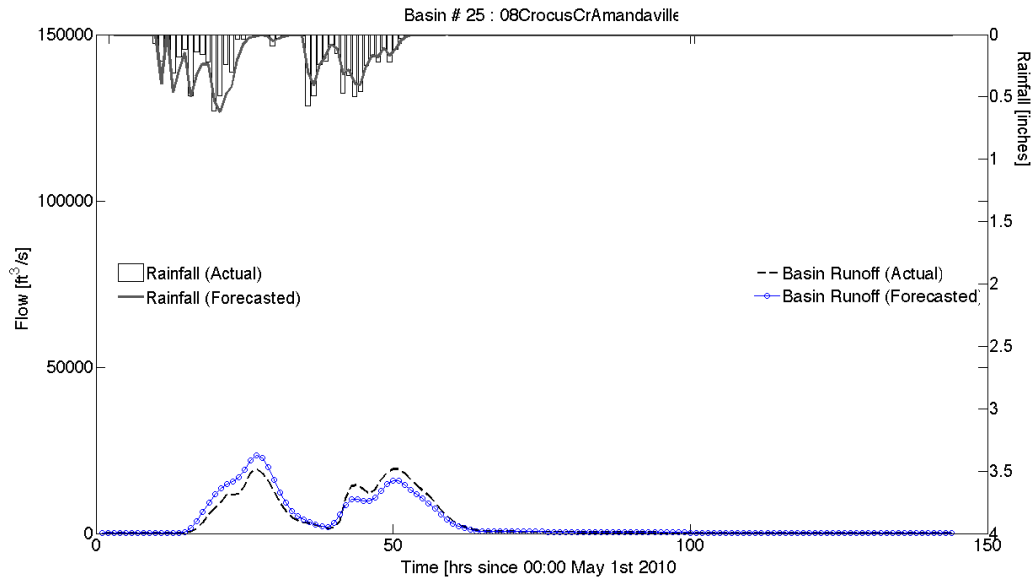


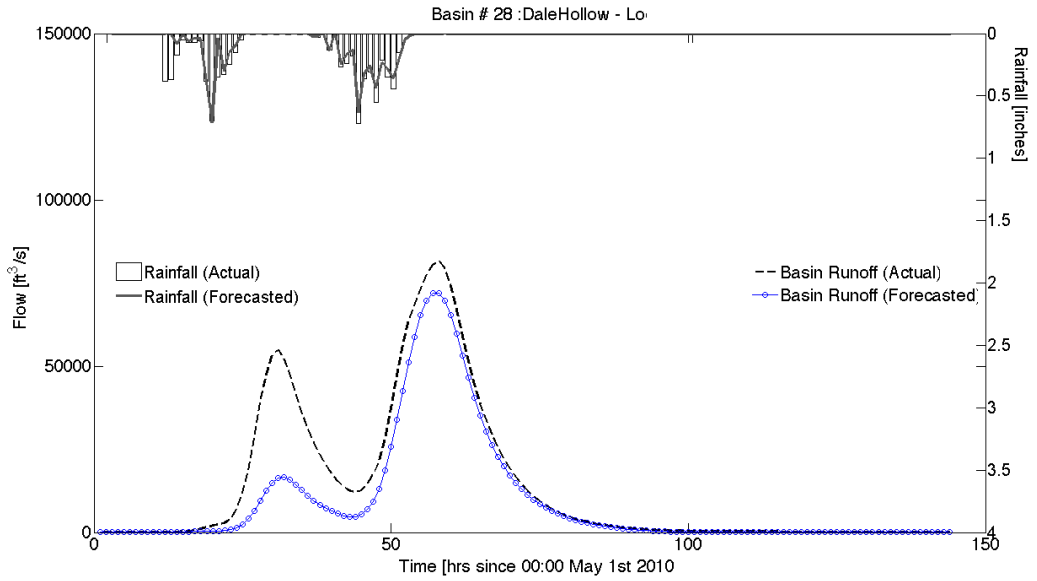
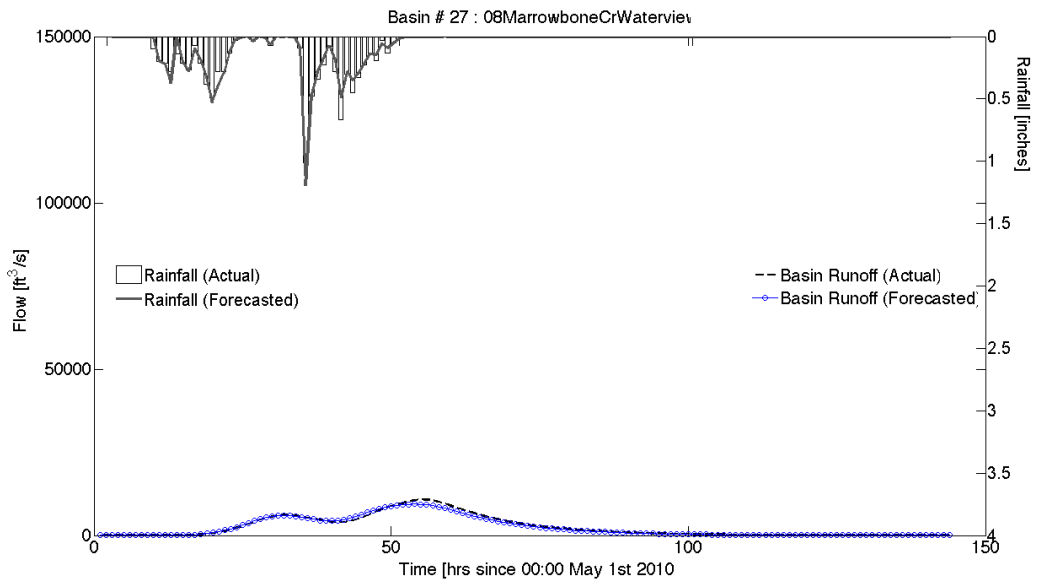


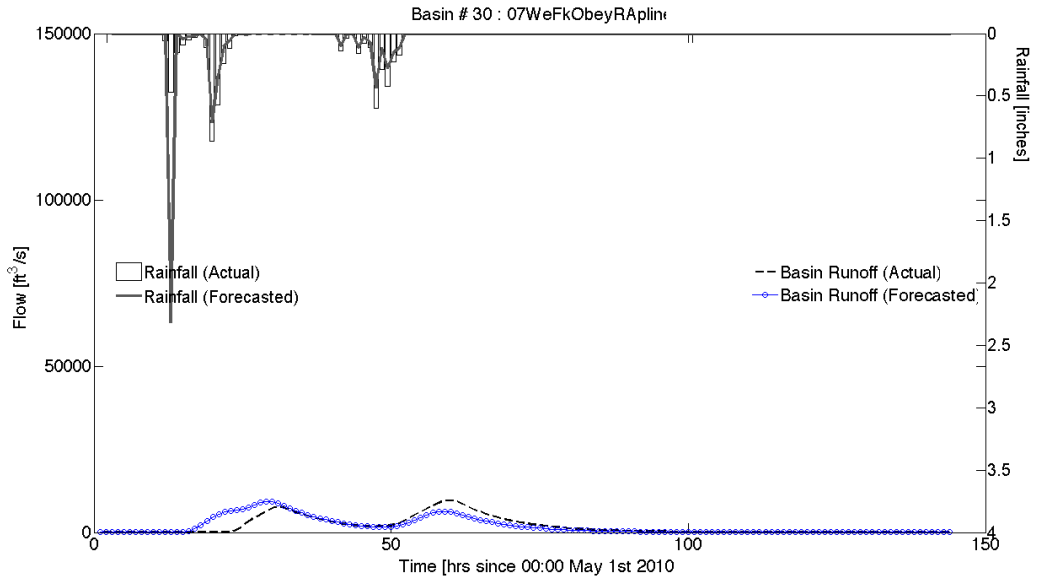
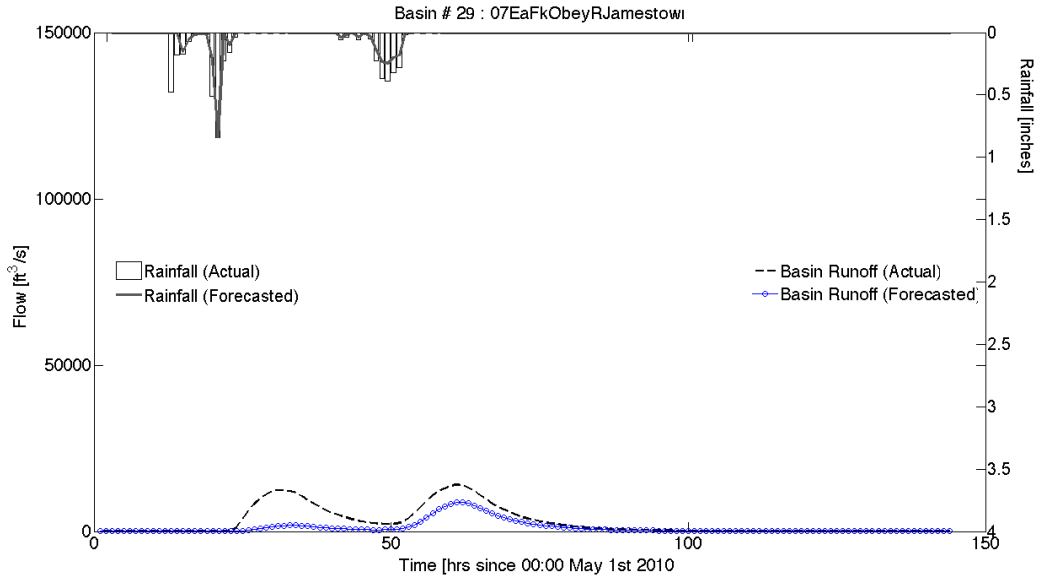


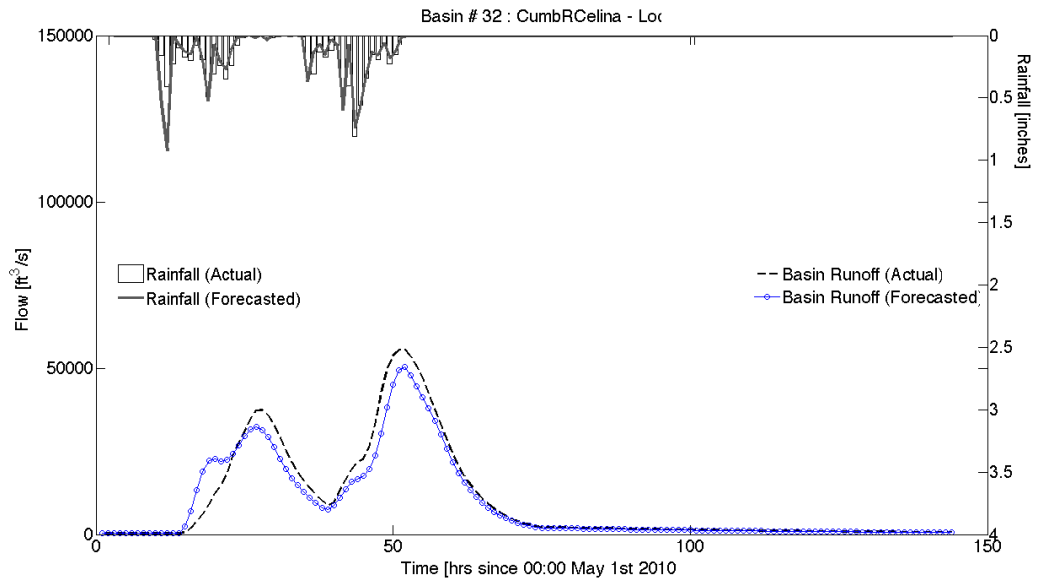
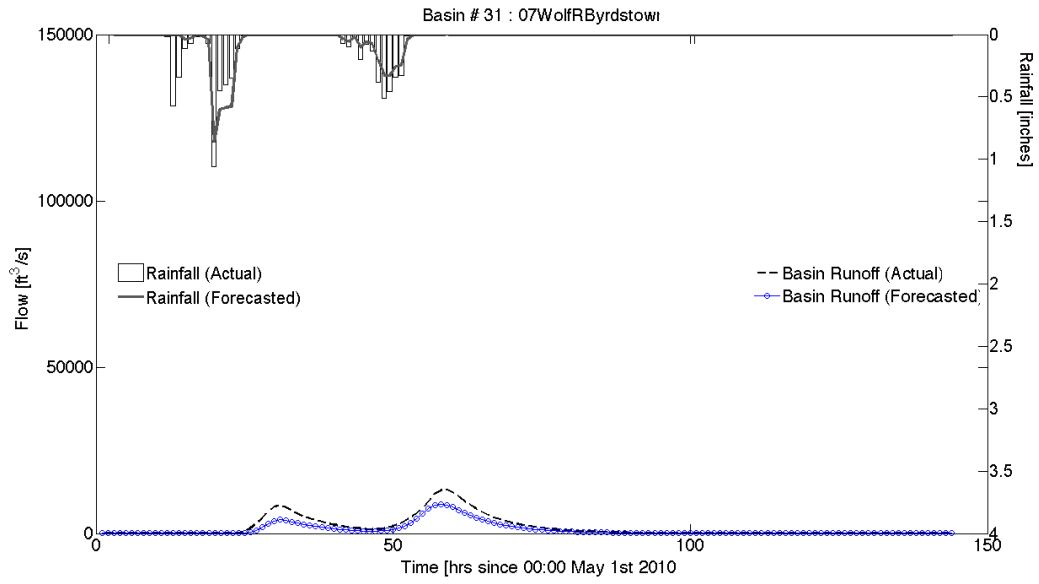


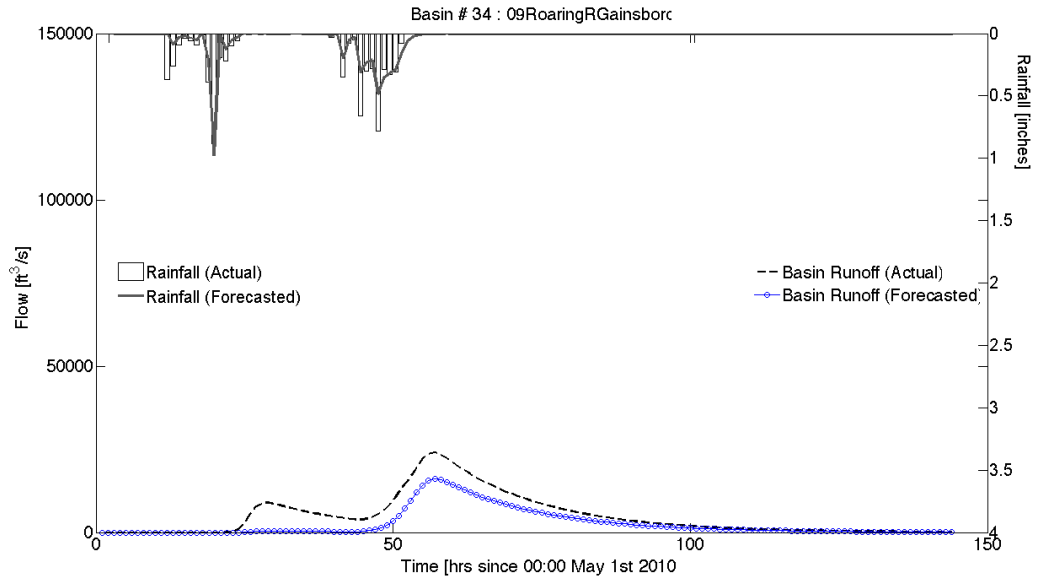
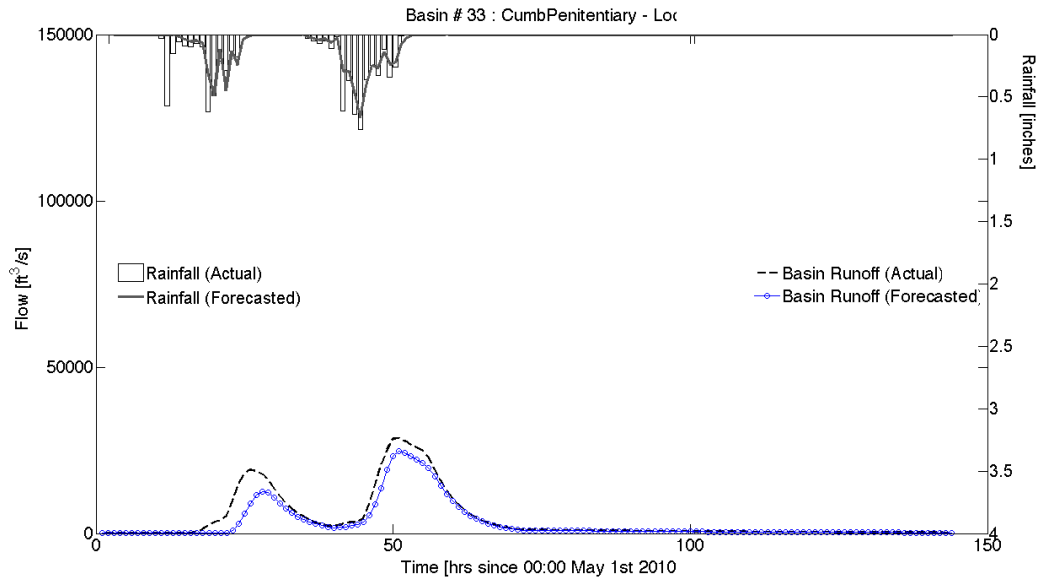


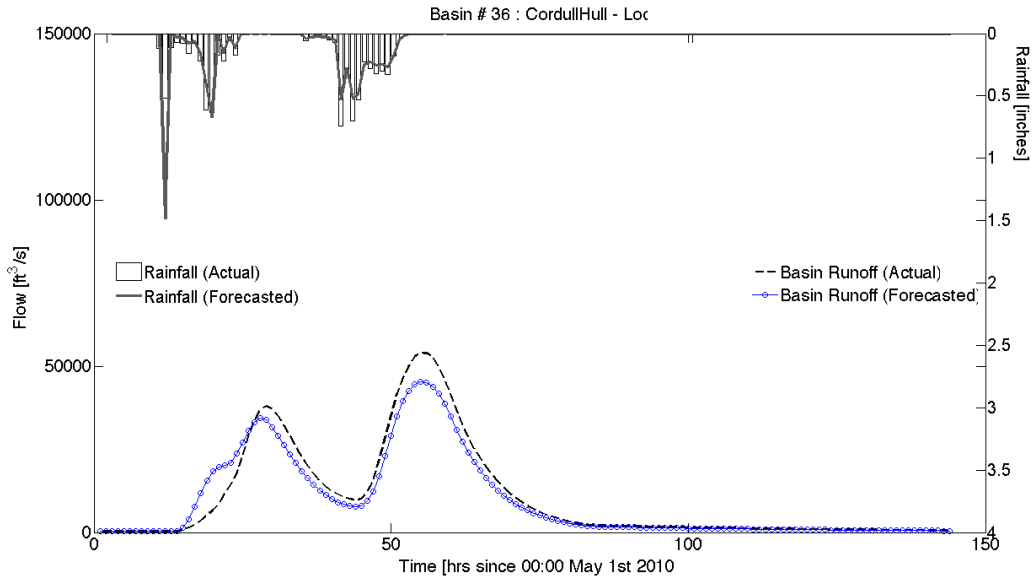
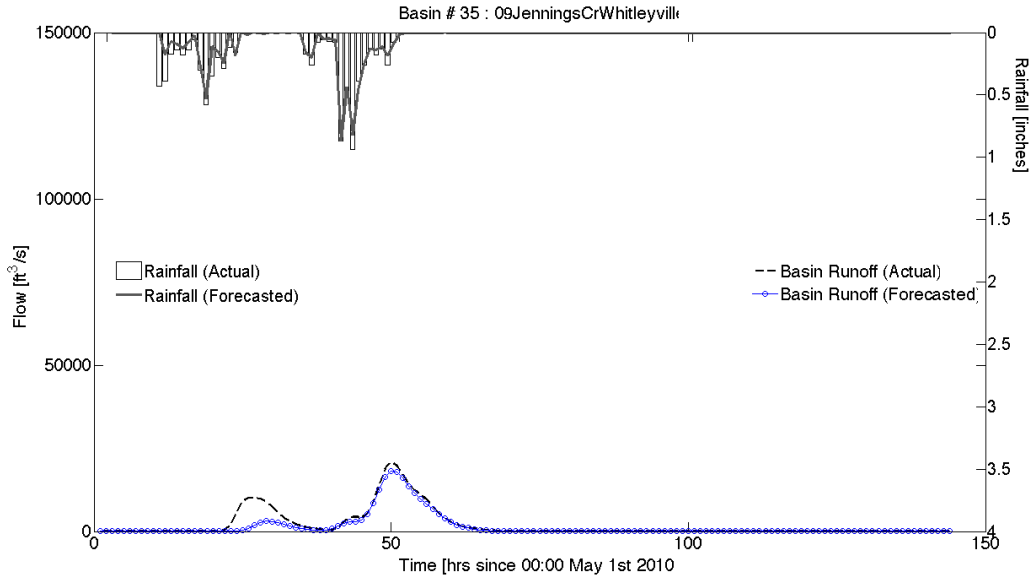


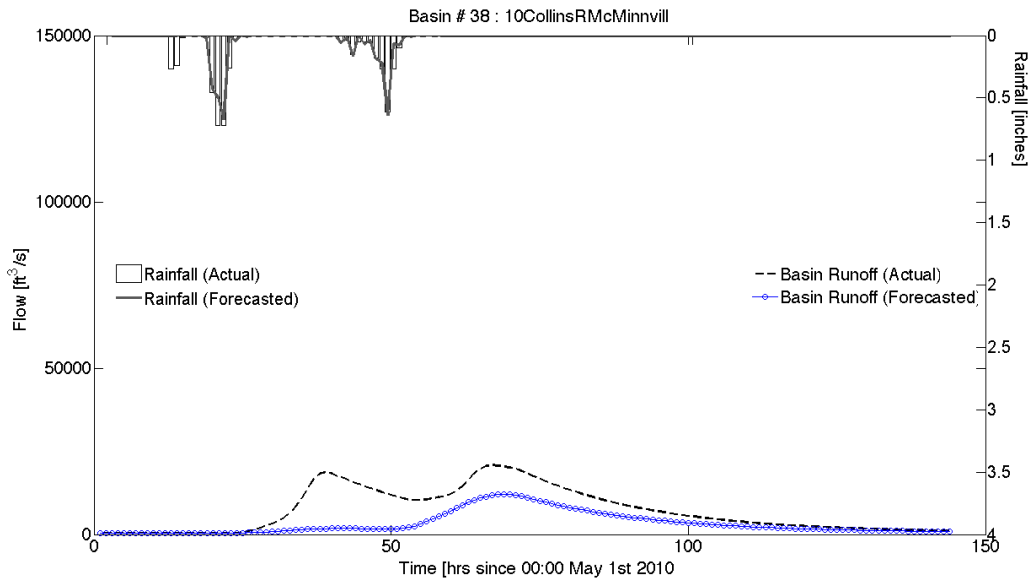
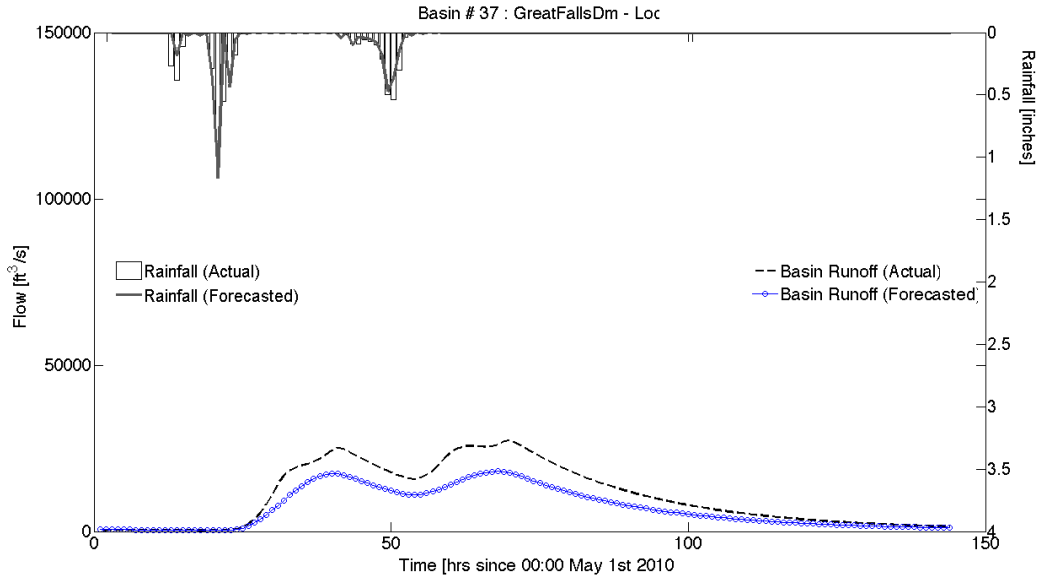


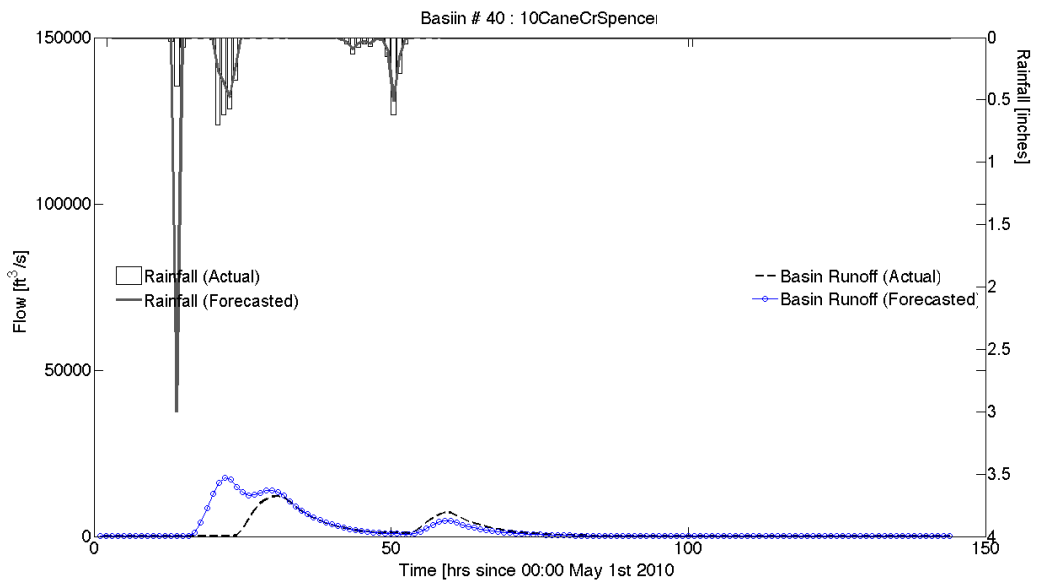
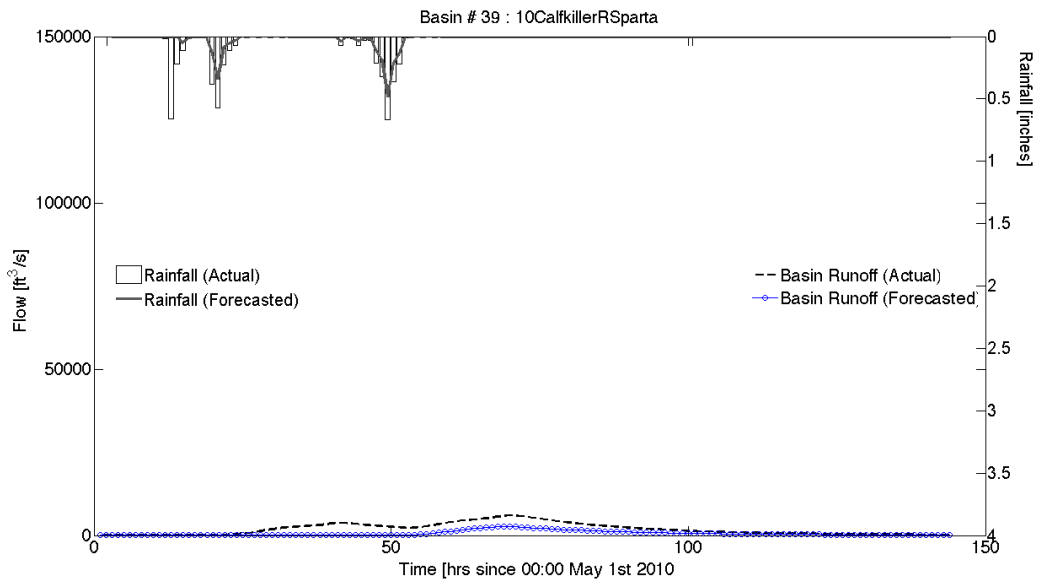


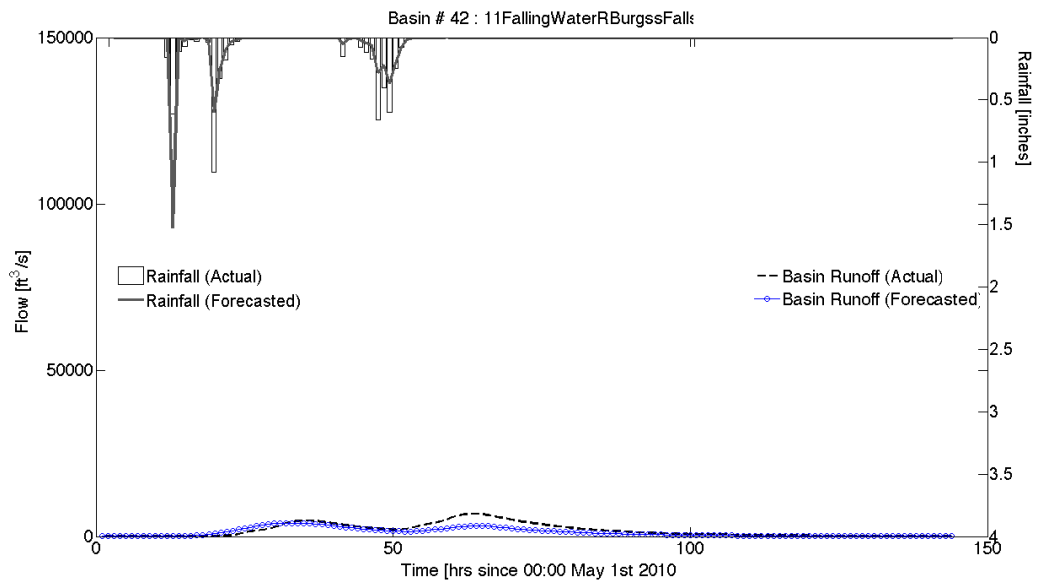
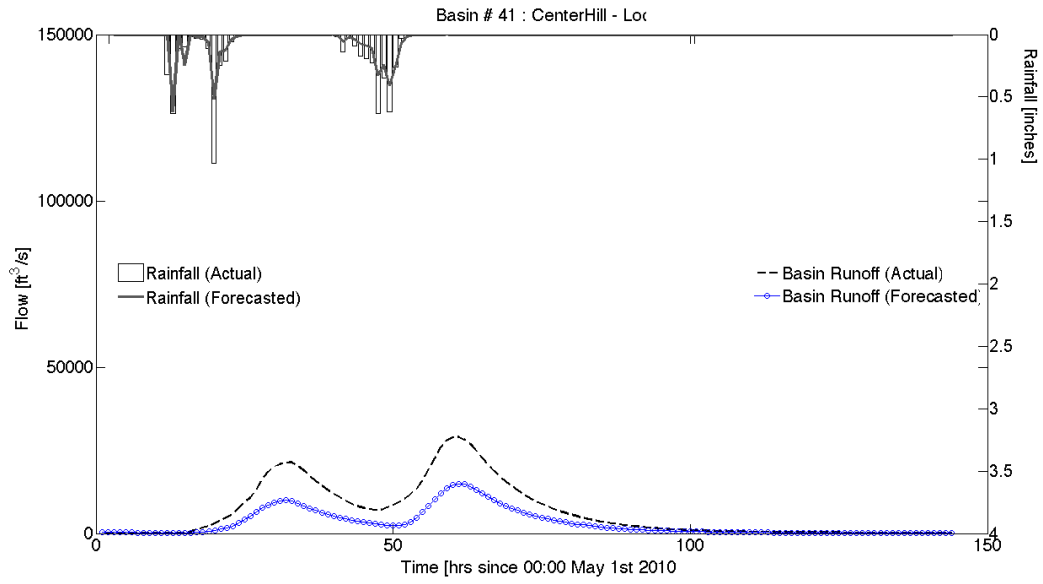


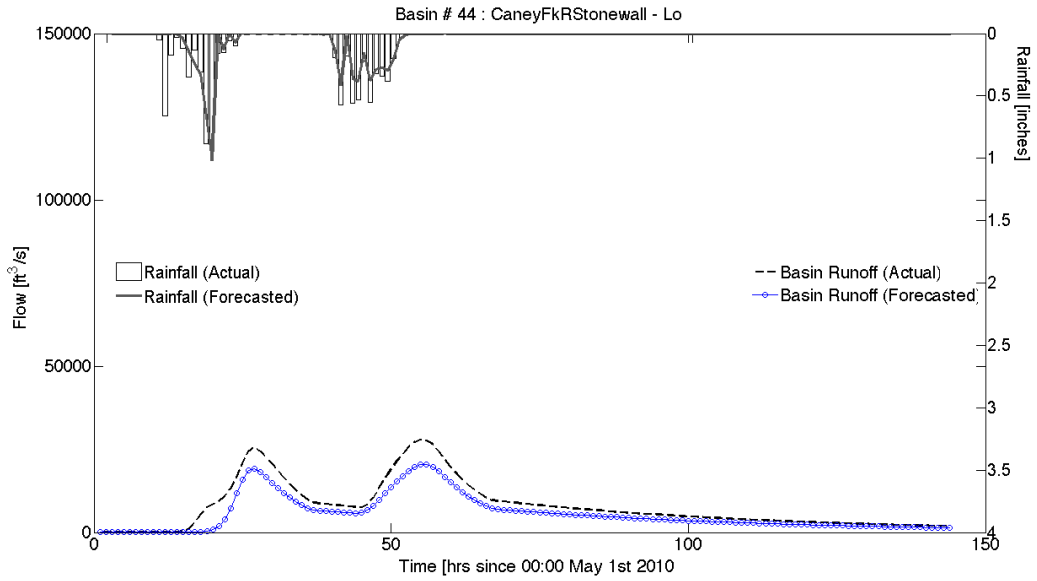
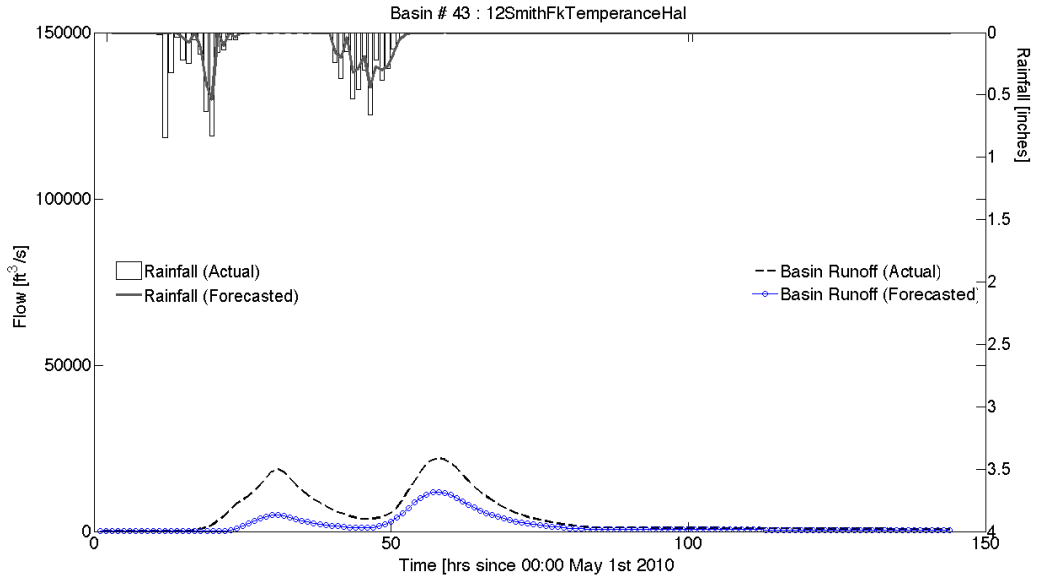


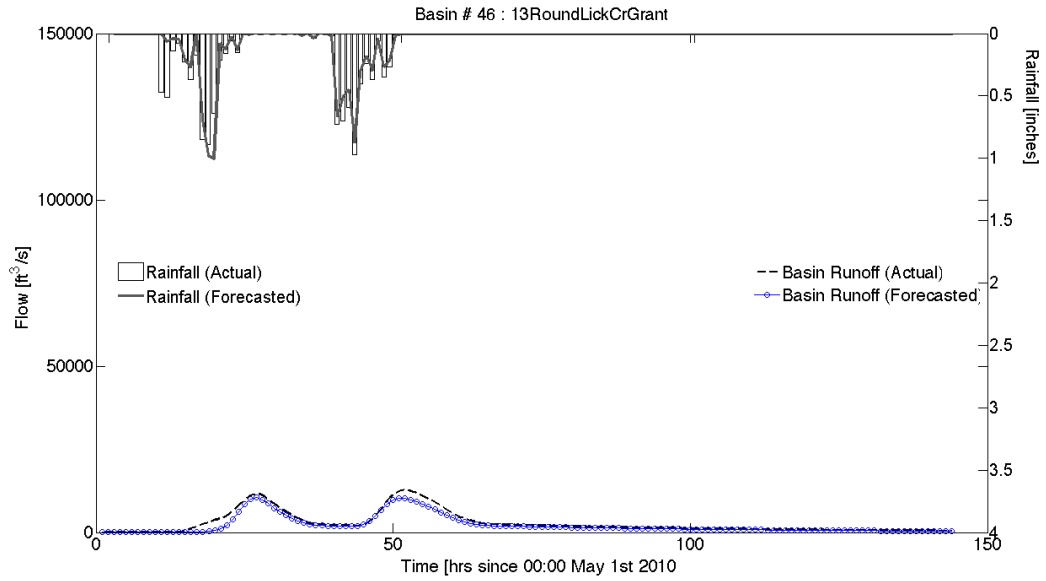
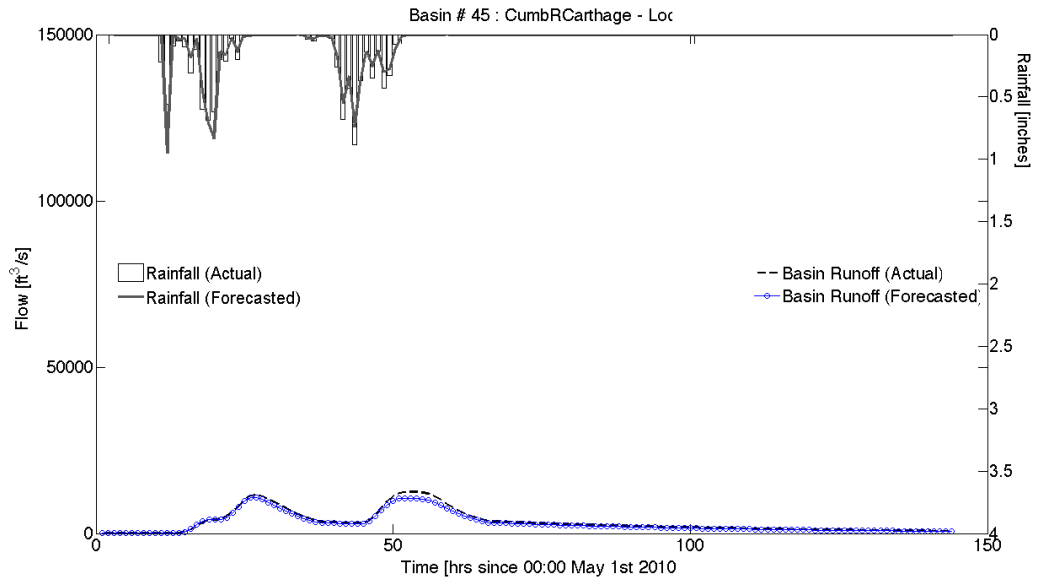


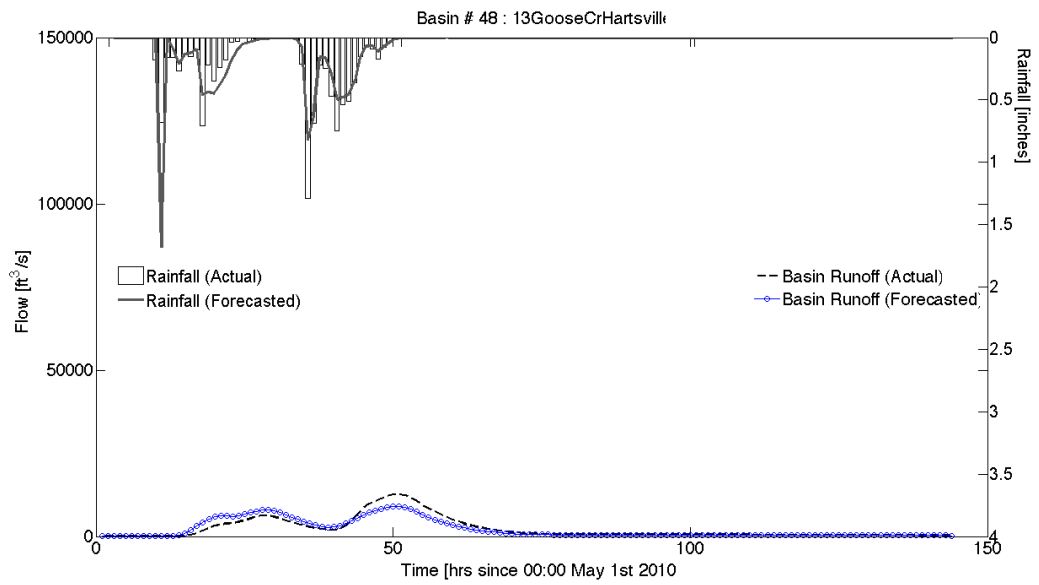
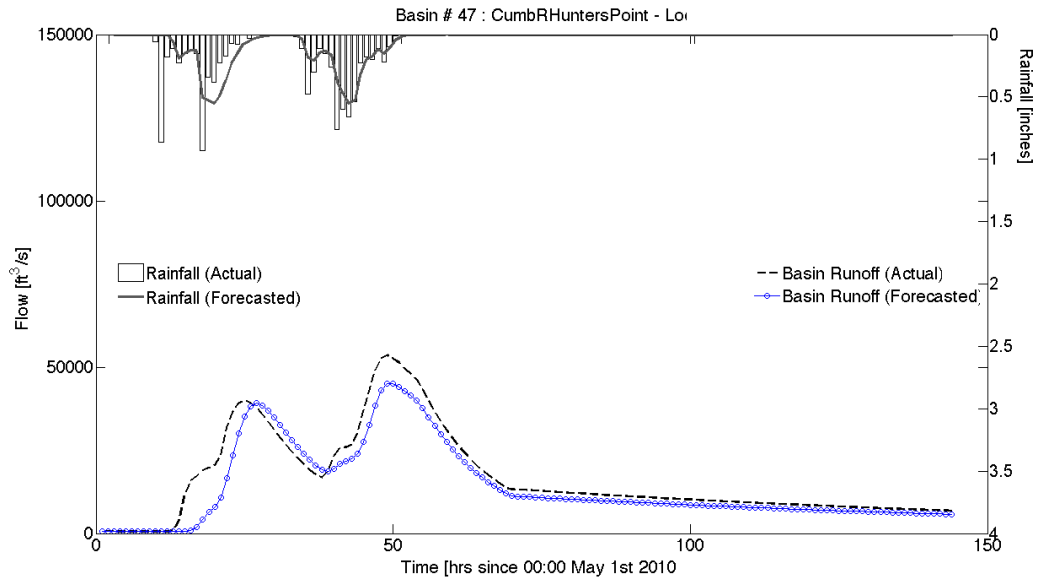


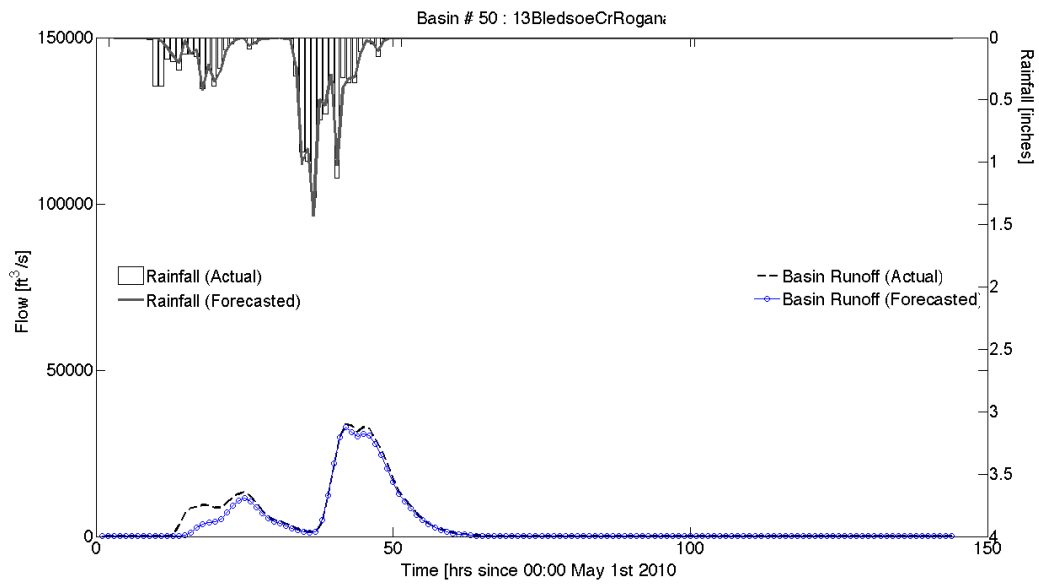
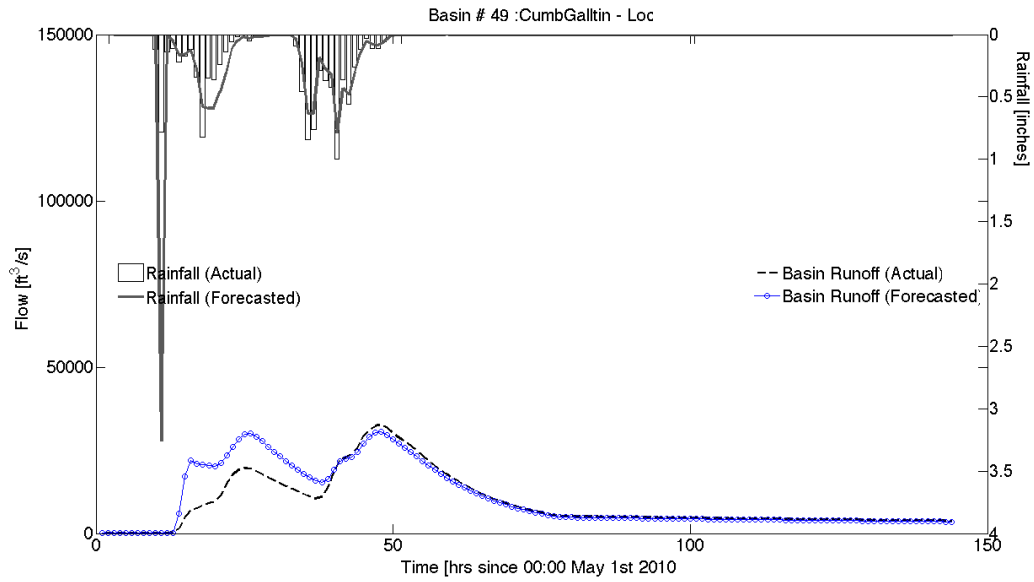


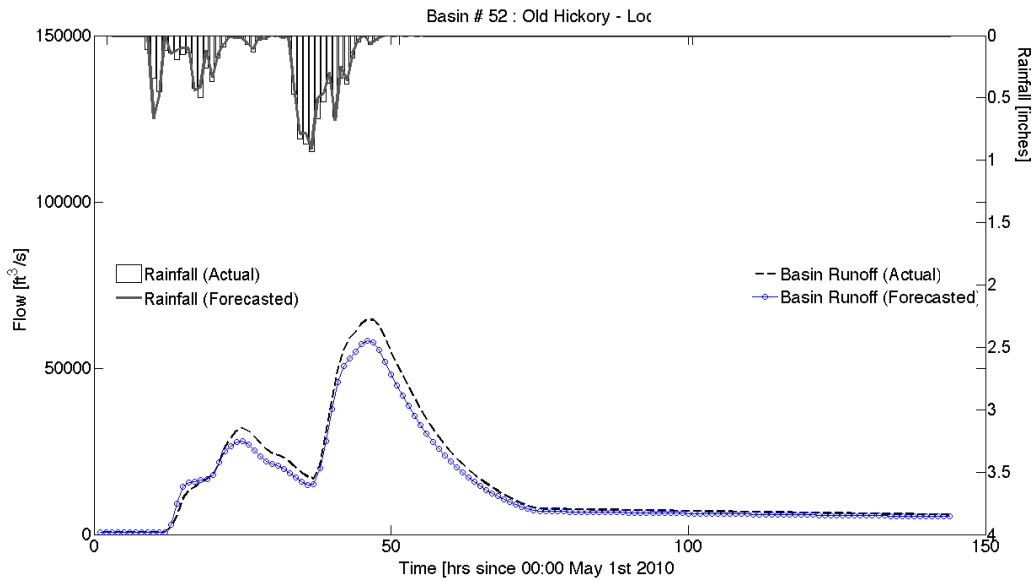
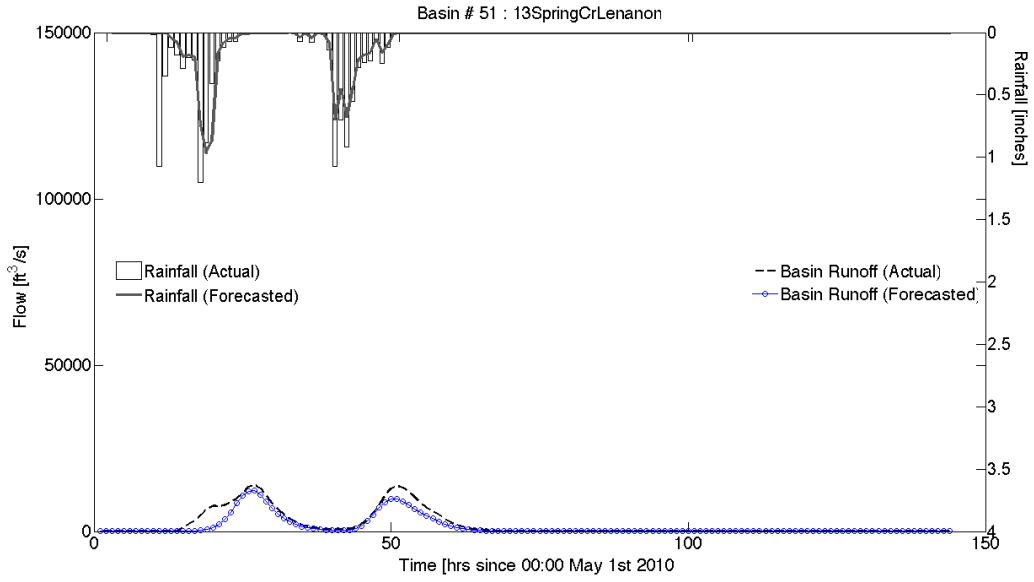


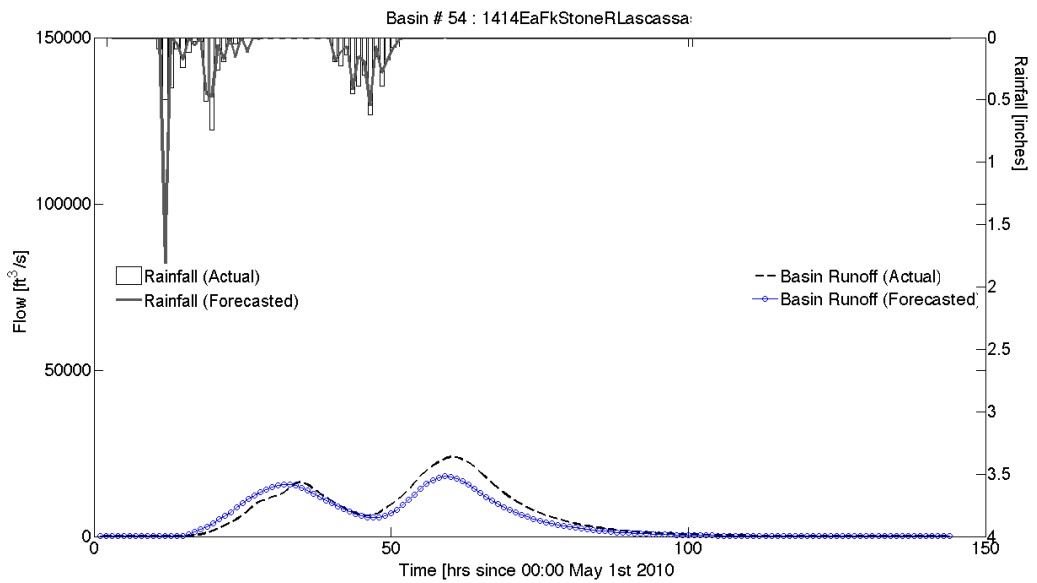
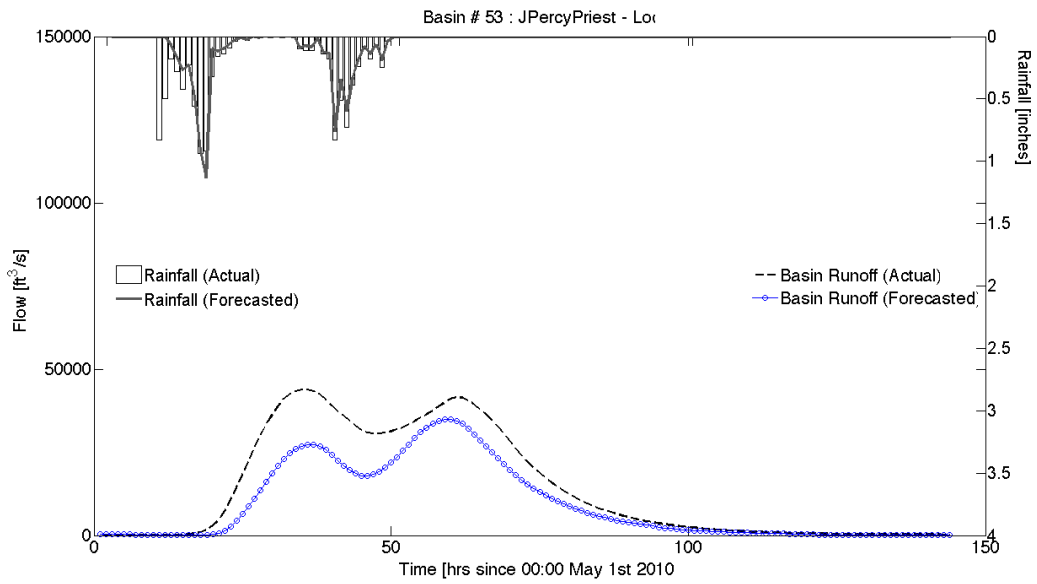


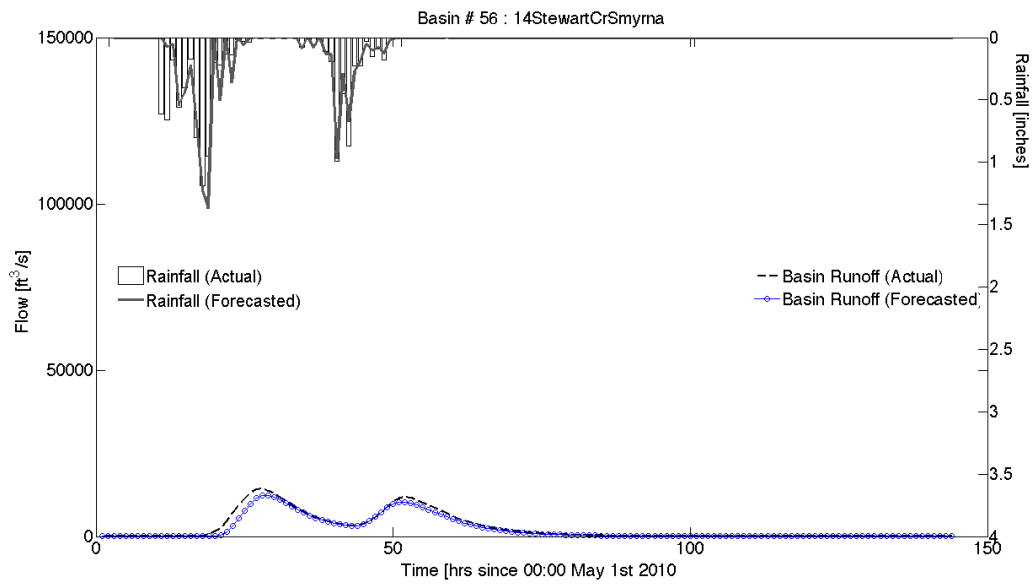
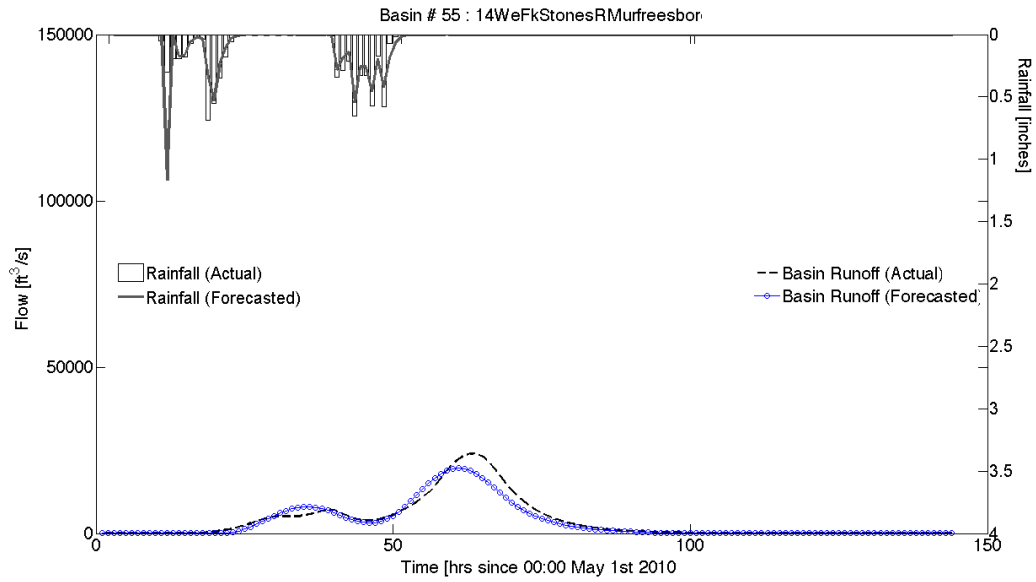


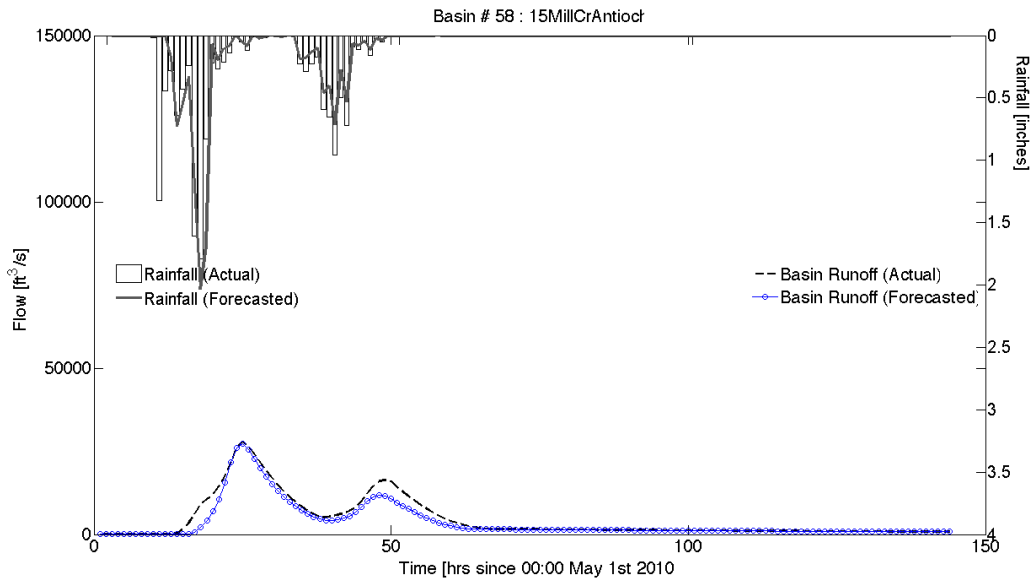
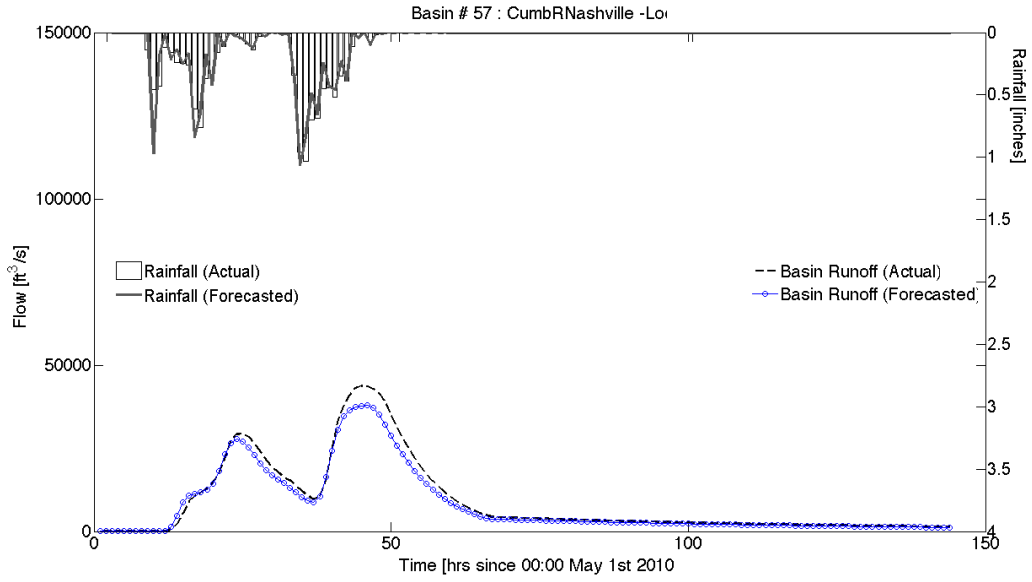


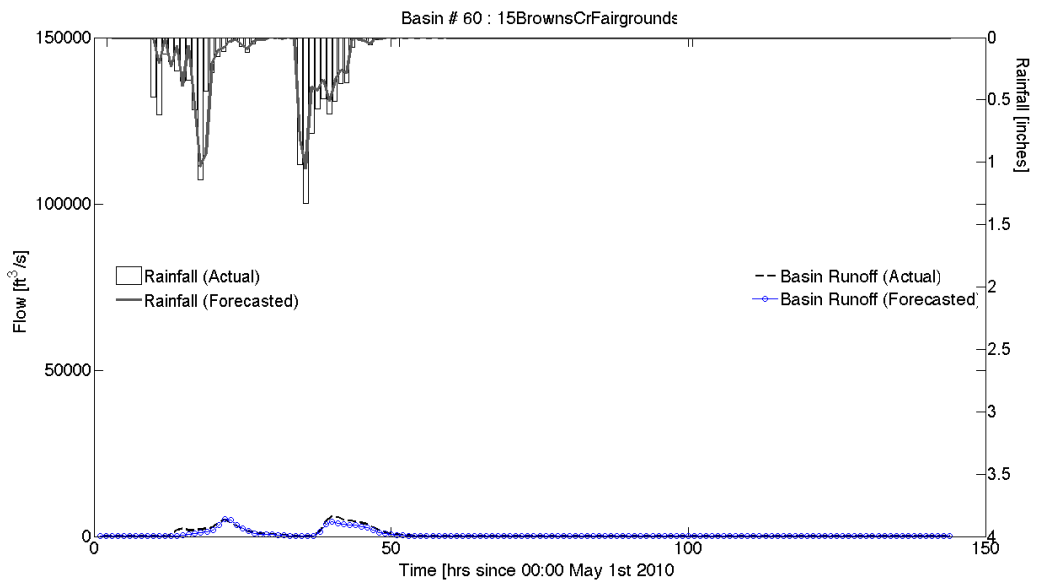
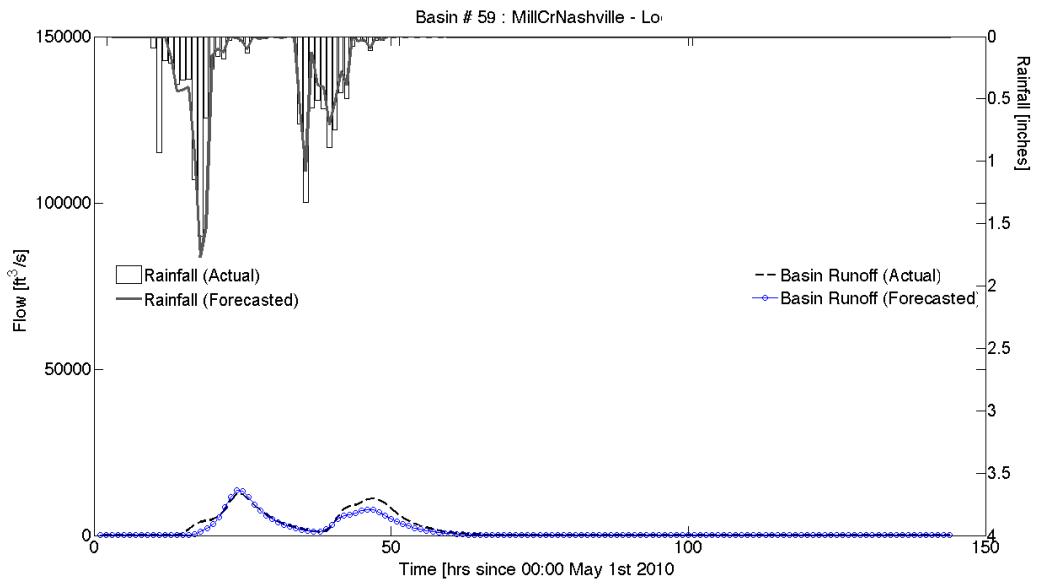


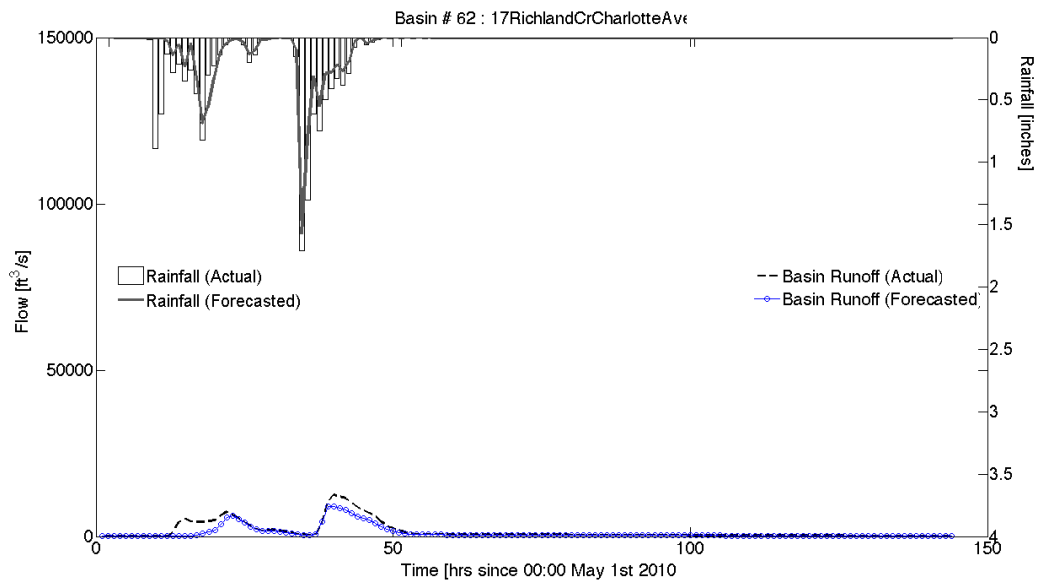
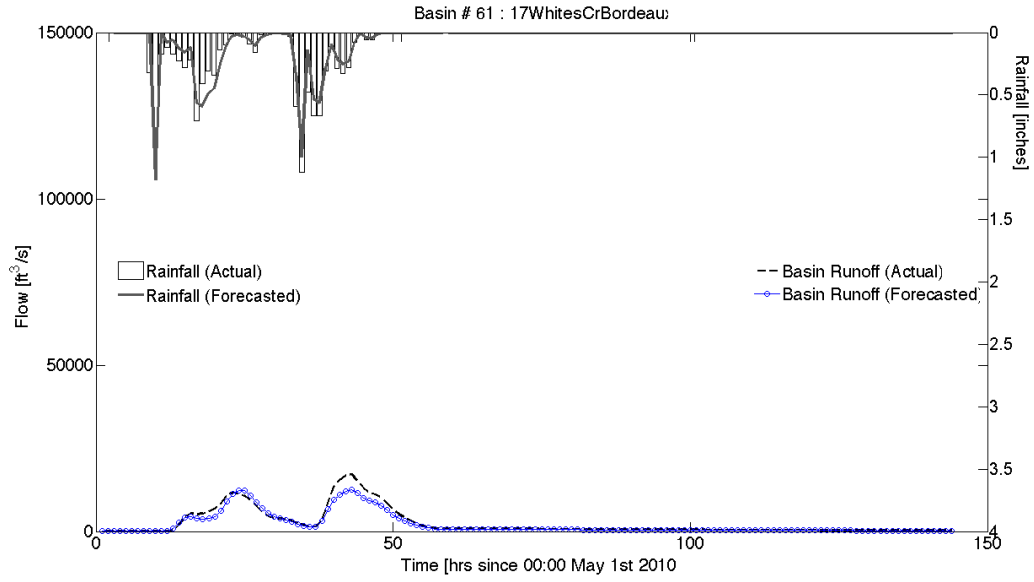


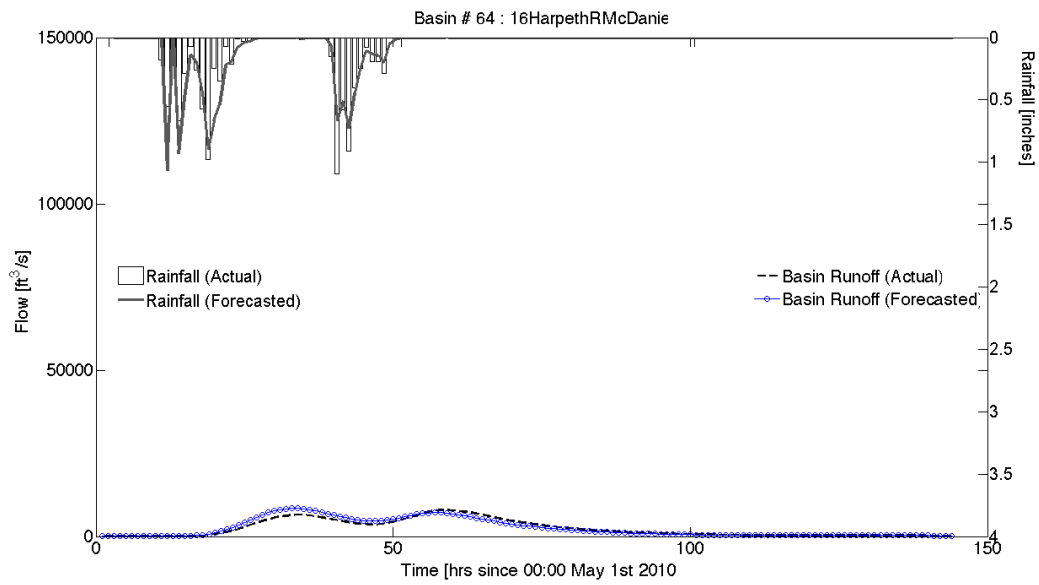
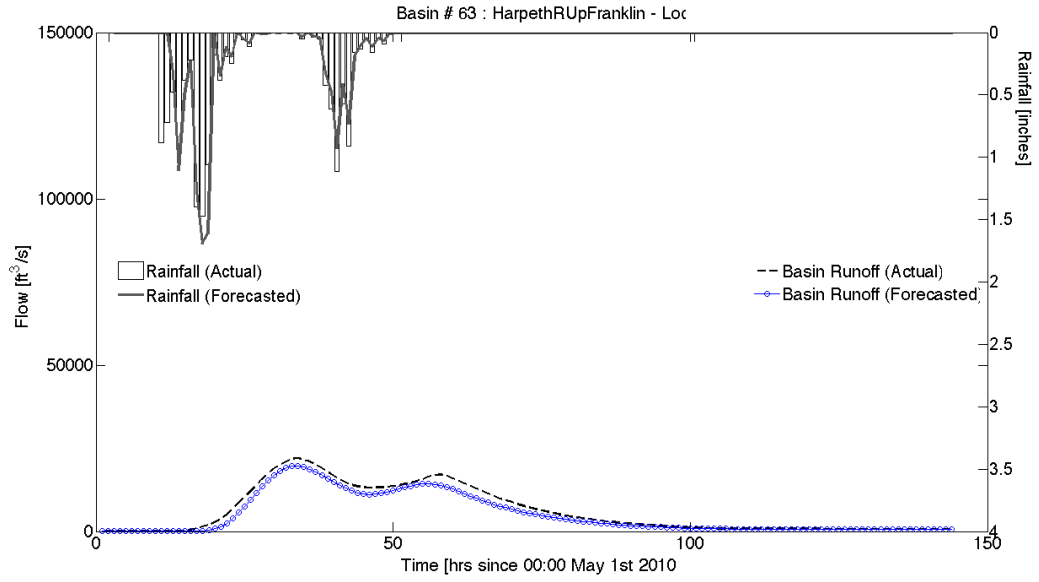


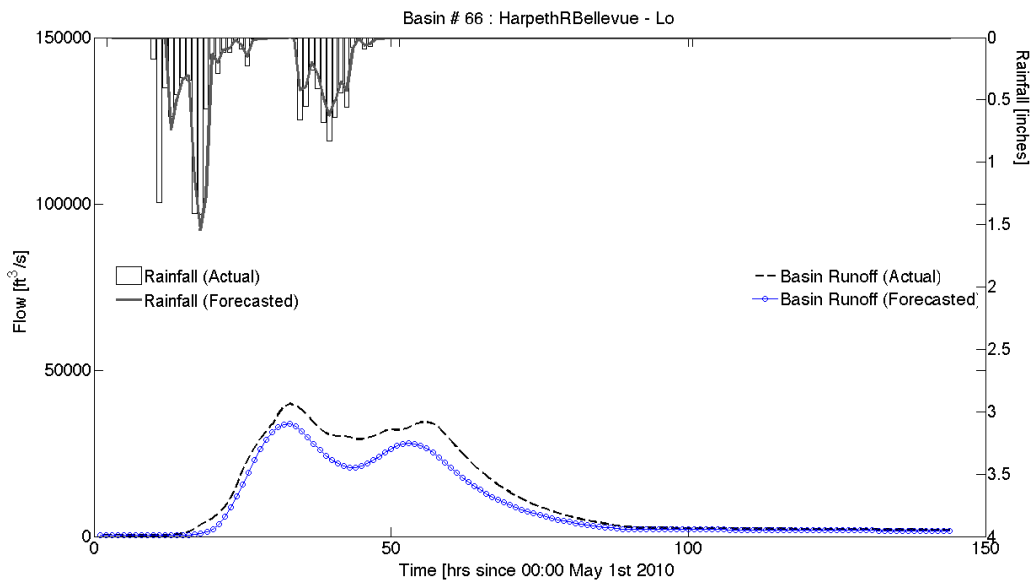
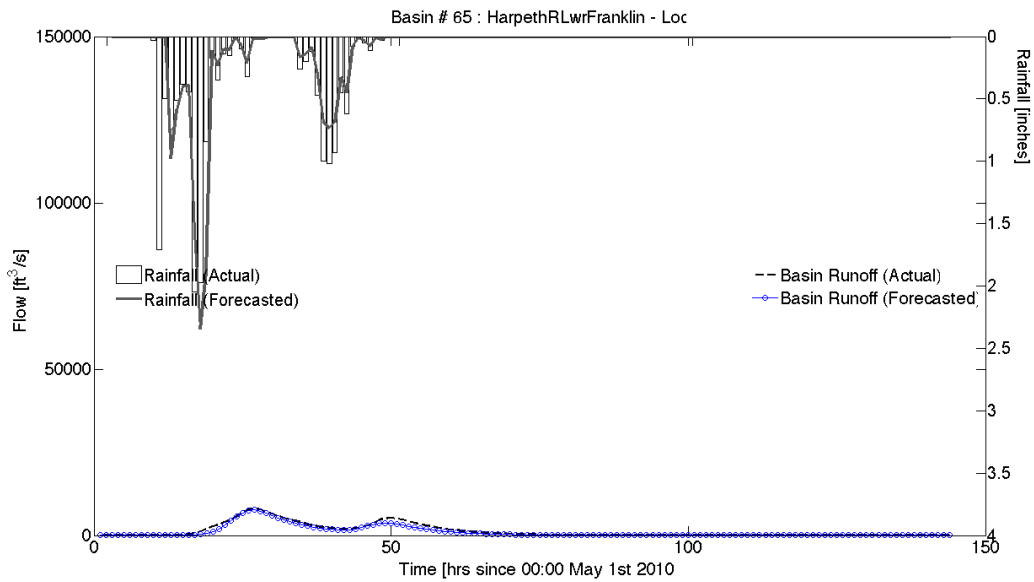


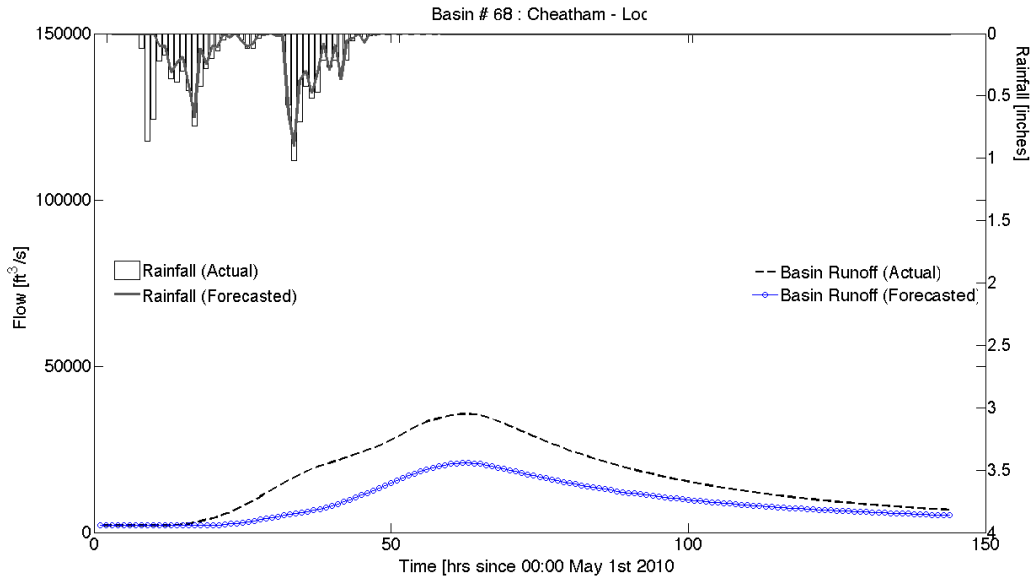
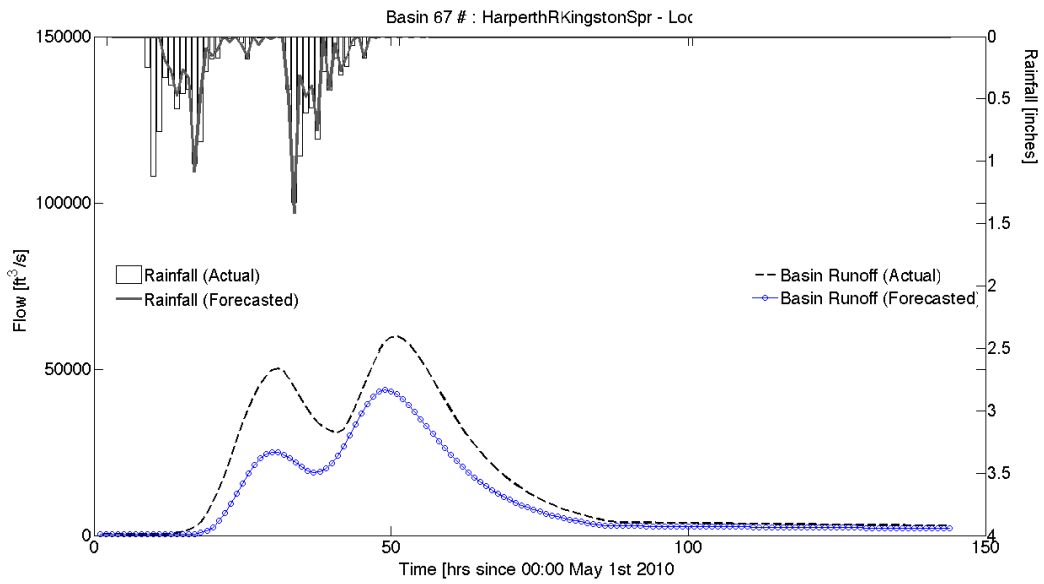


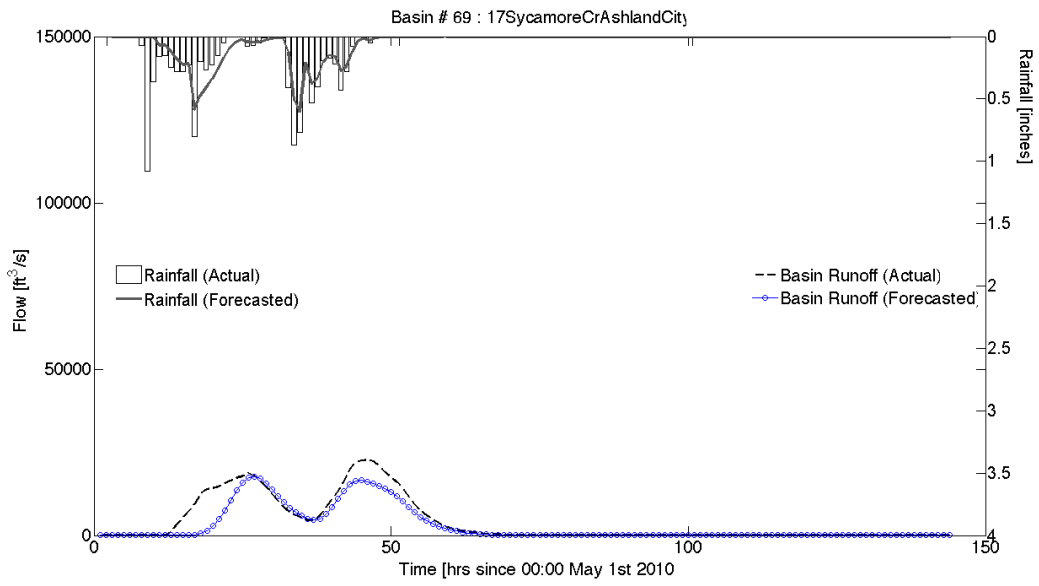






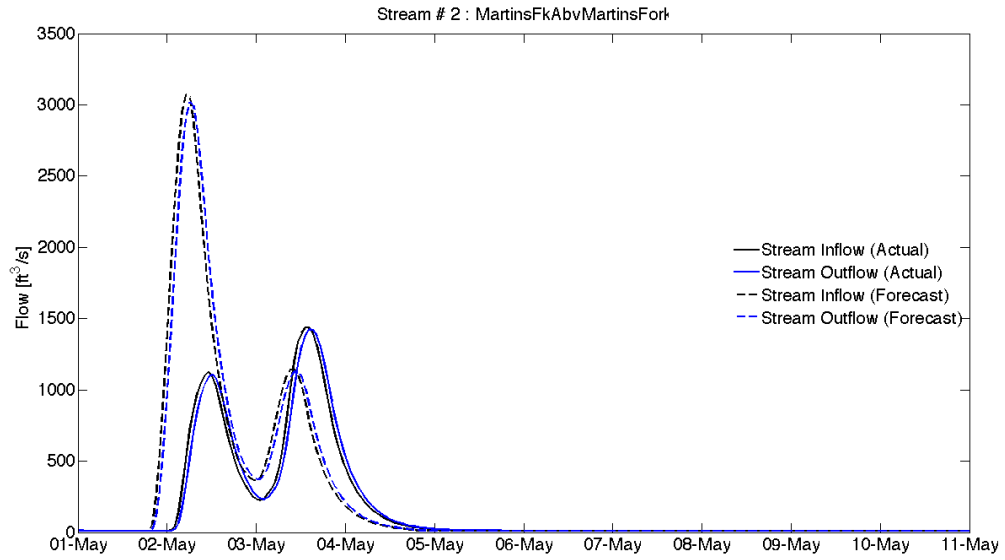
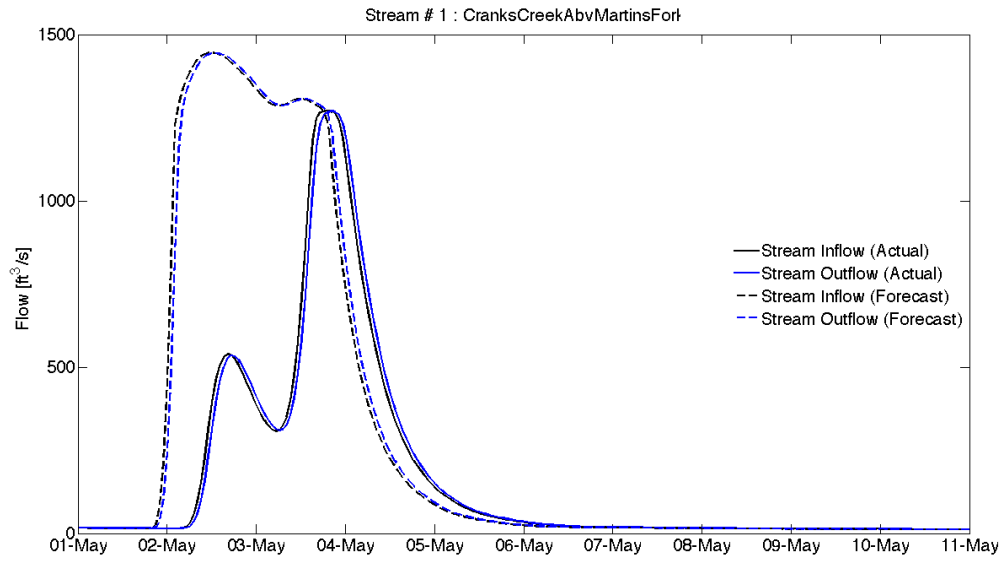


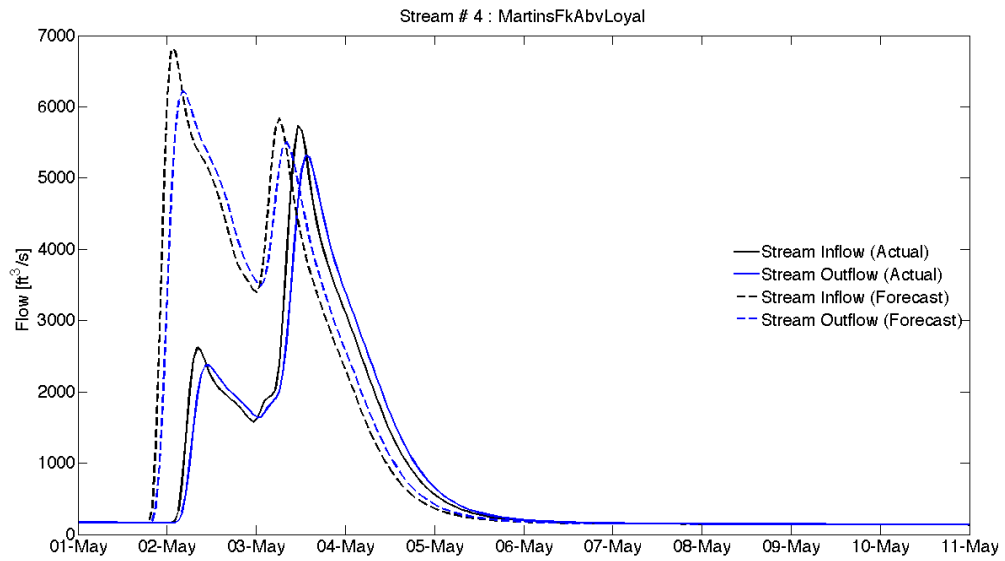
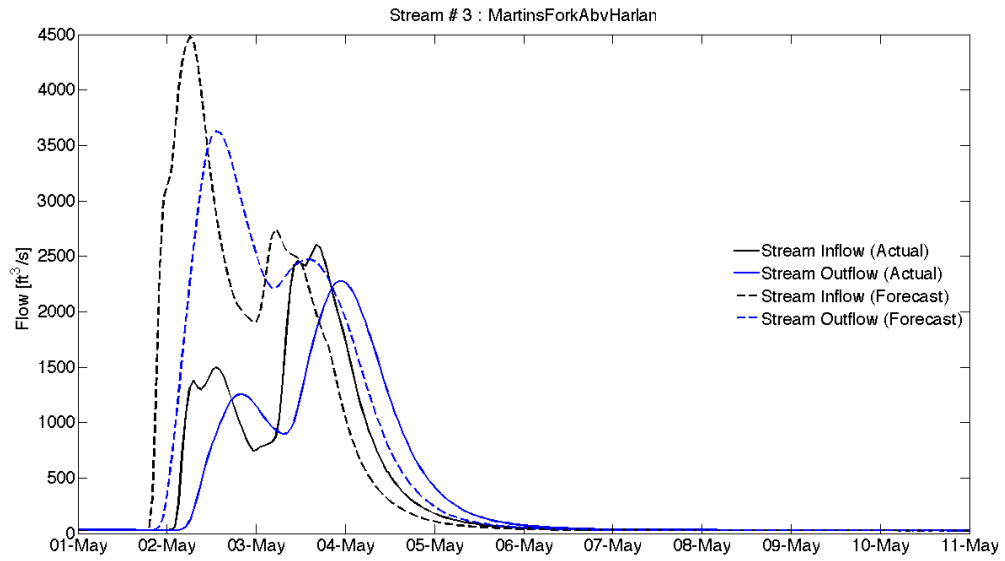


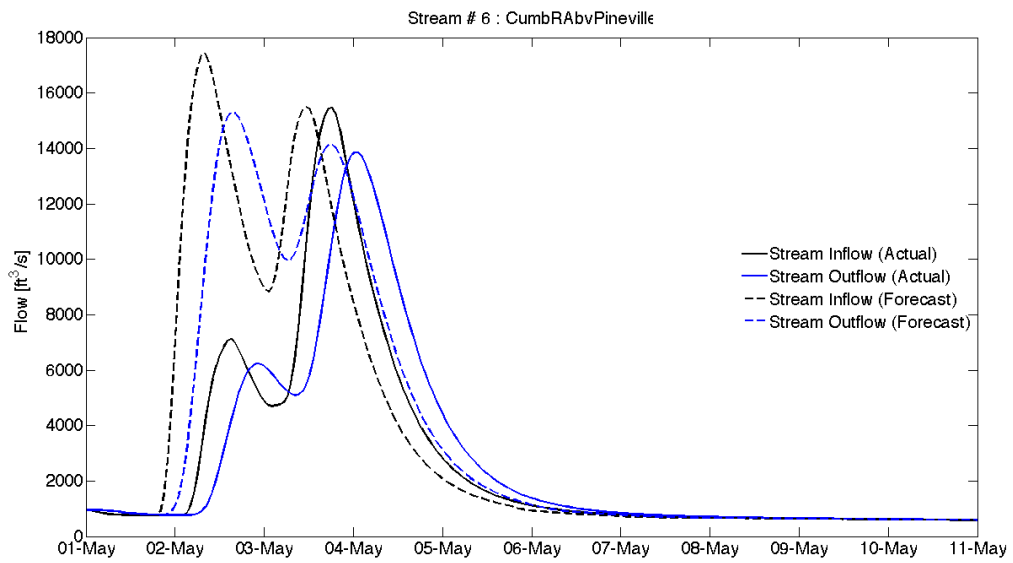
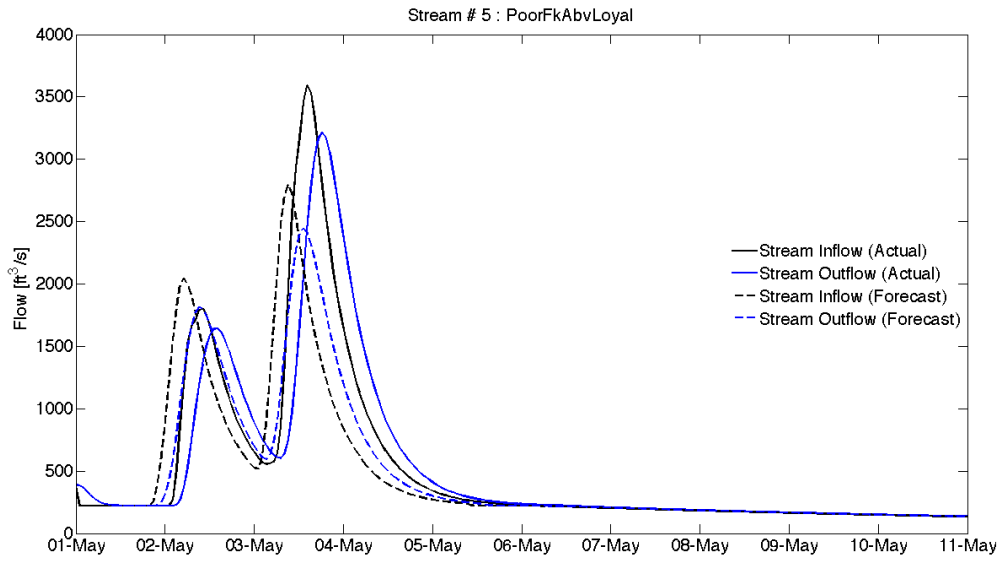


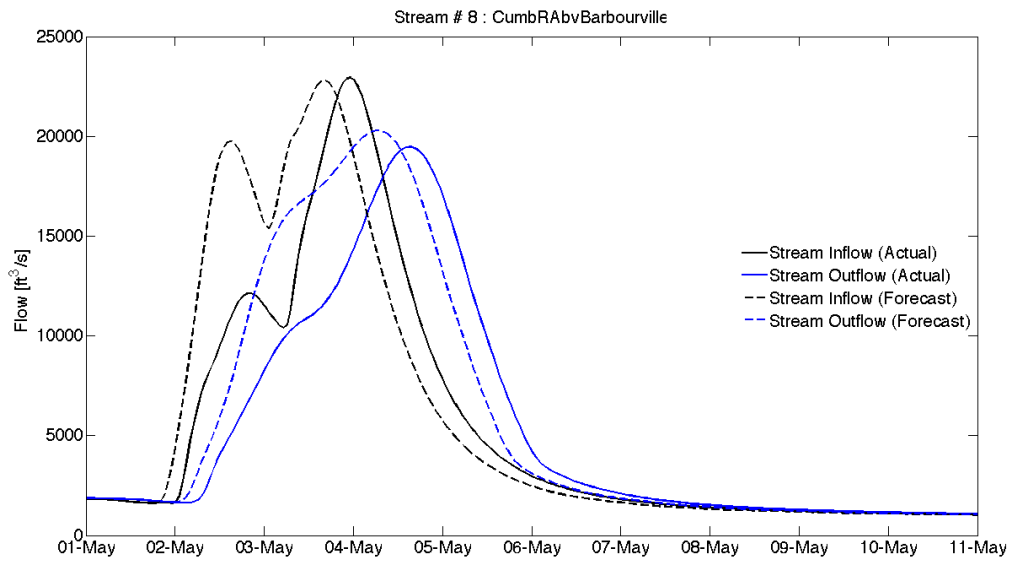
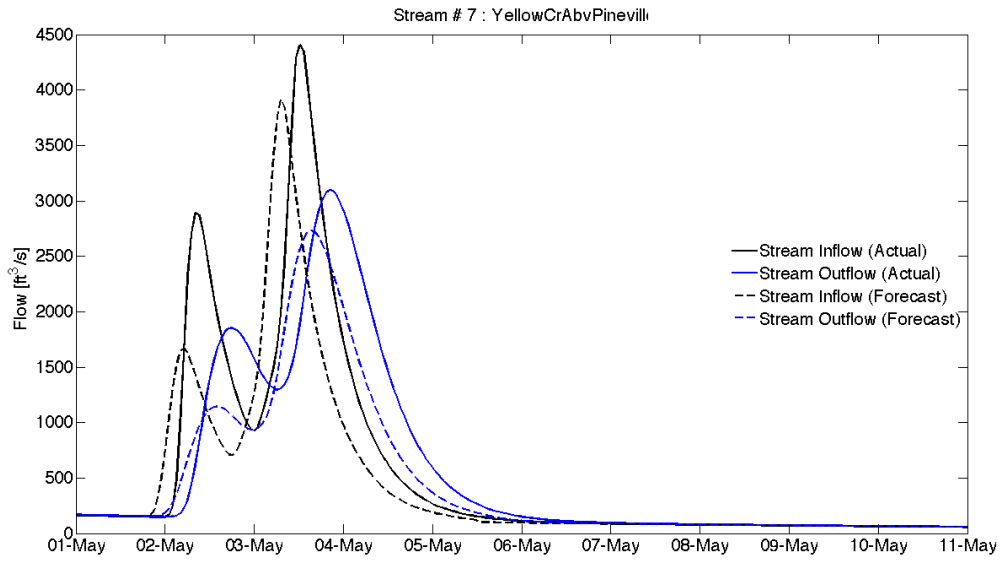
APPENDIX J

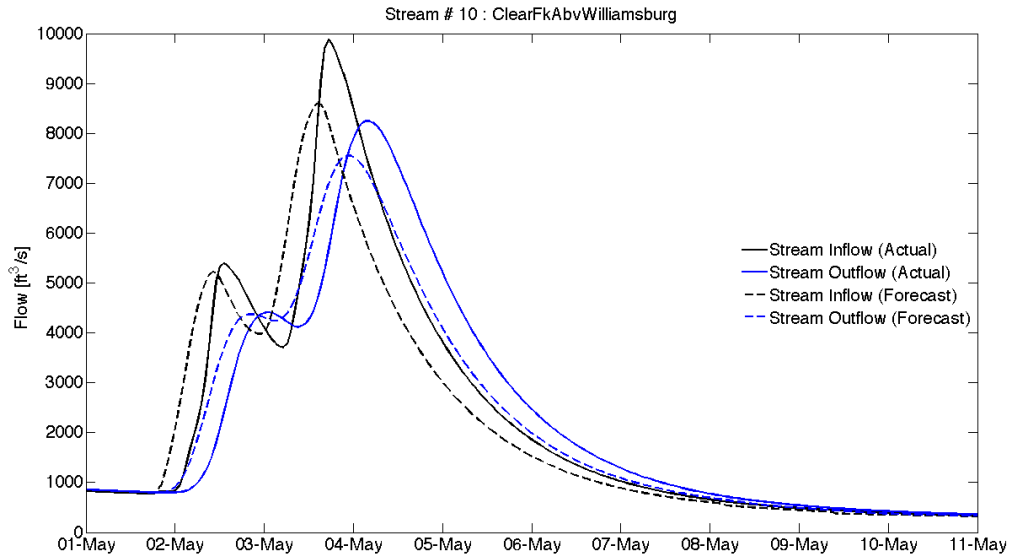
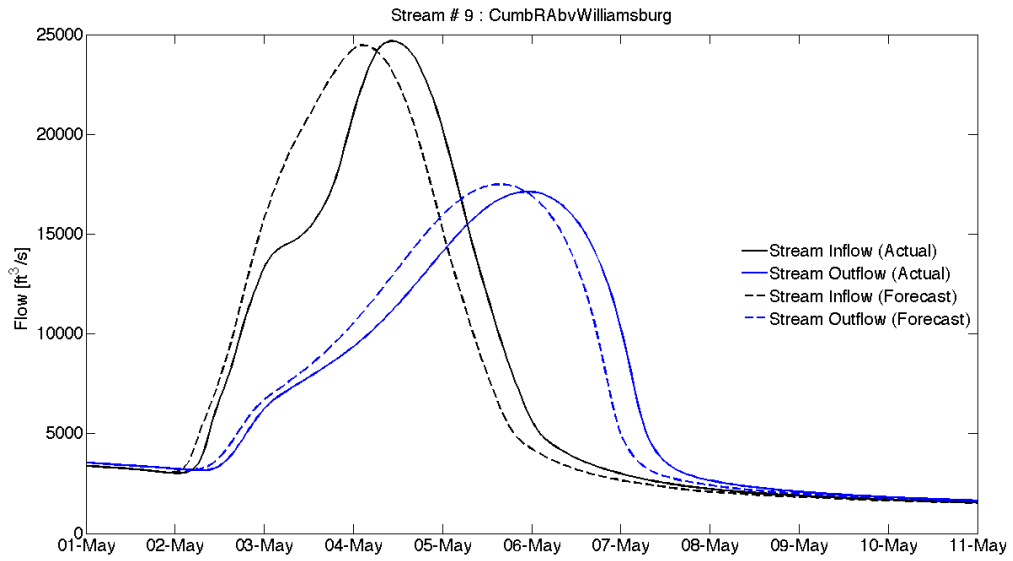
HEC-HMS CUMBERLAND RIVER BASIN MODEL STREAM FLOW RESULTS

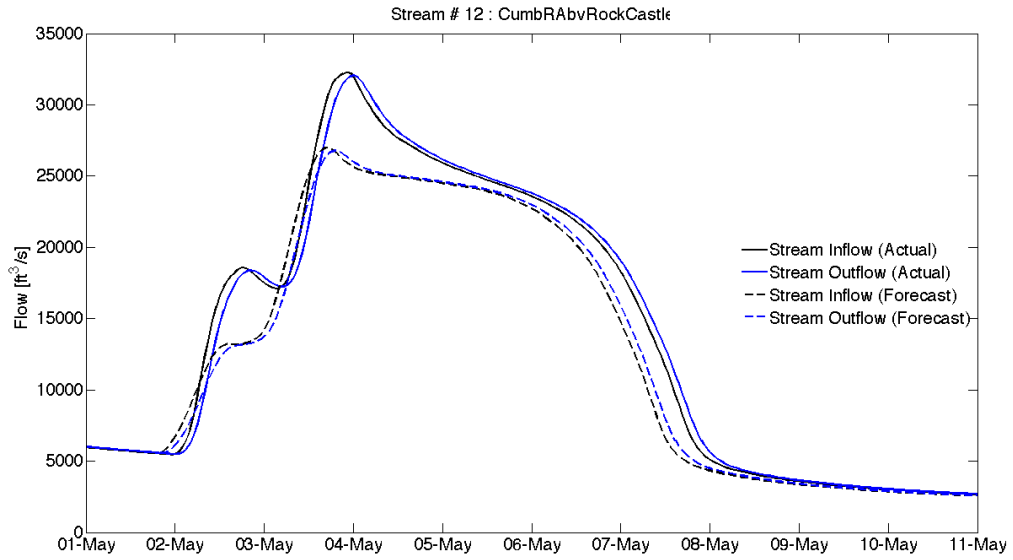
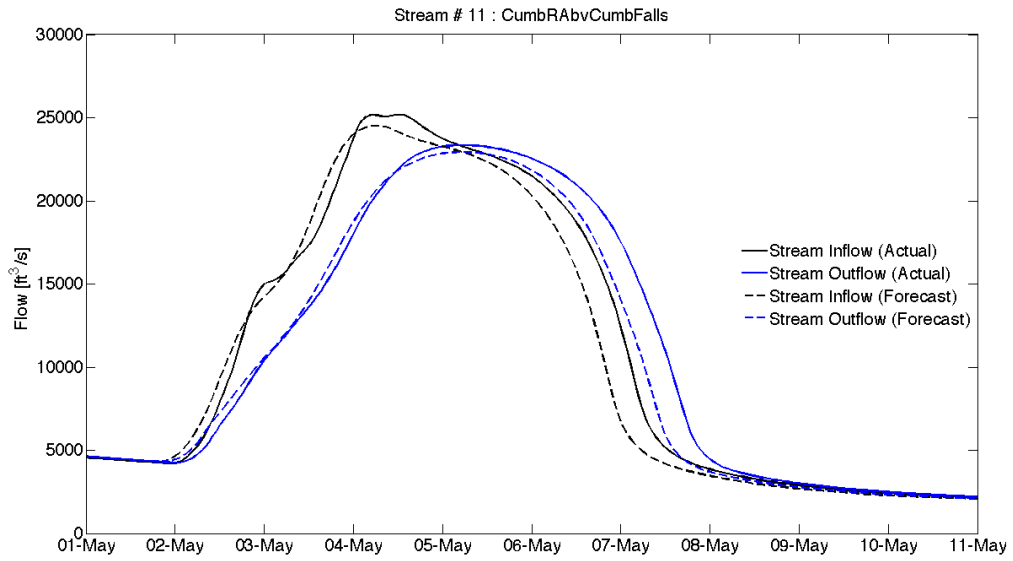


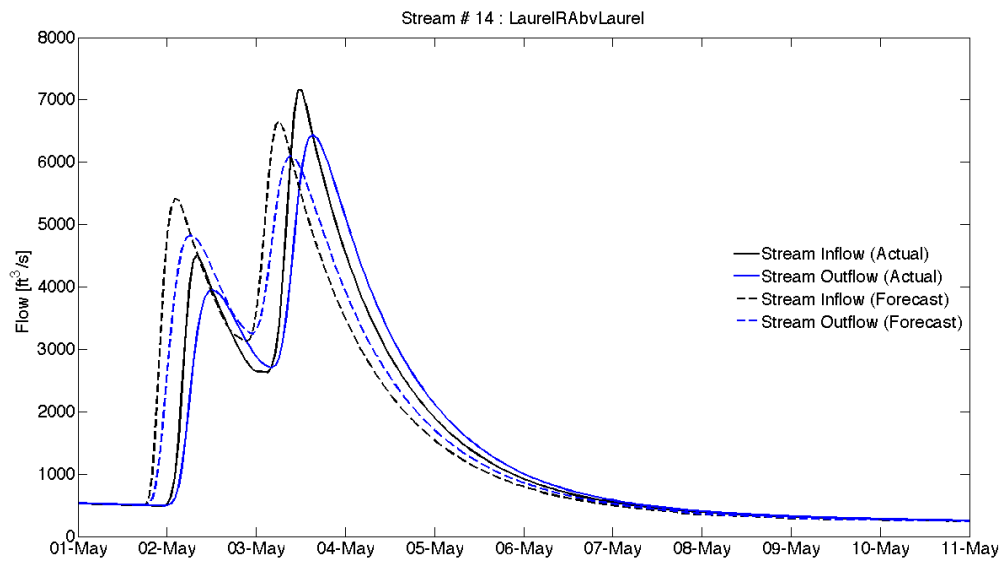
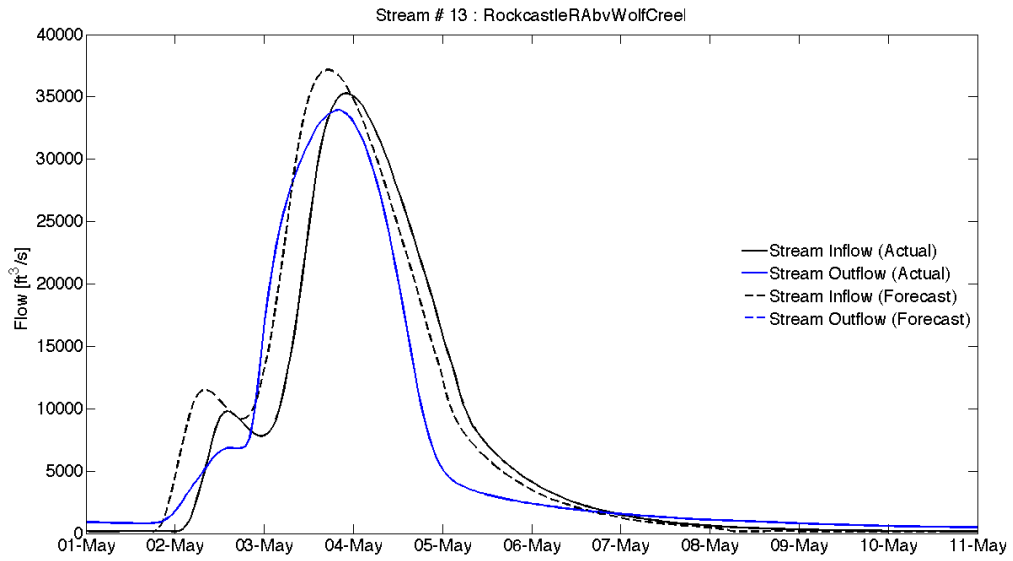


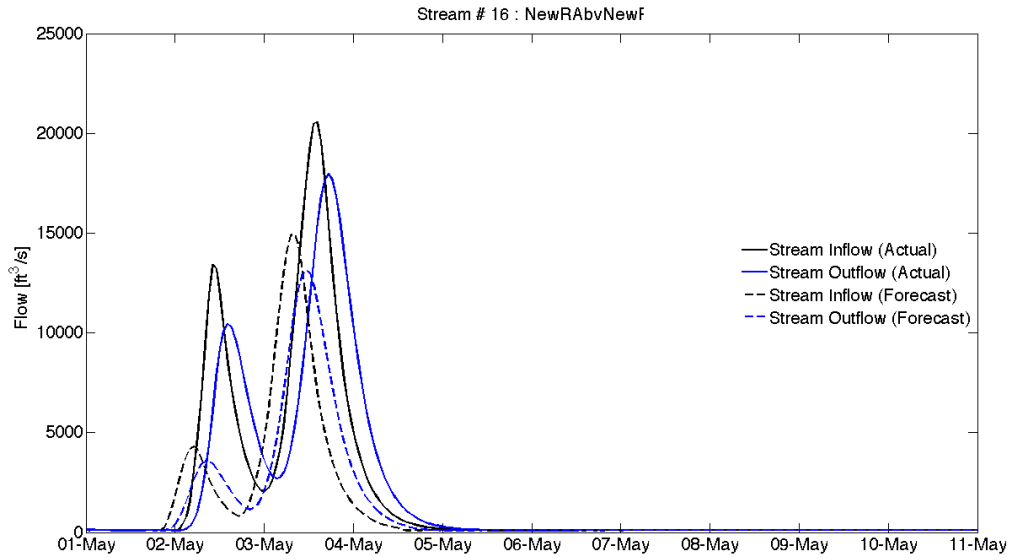
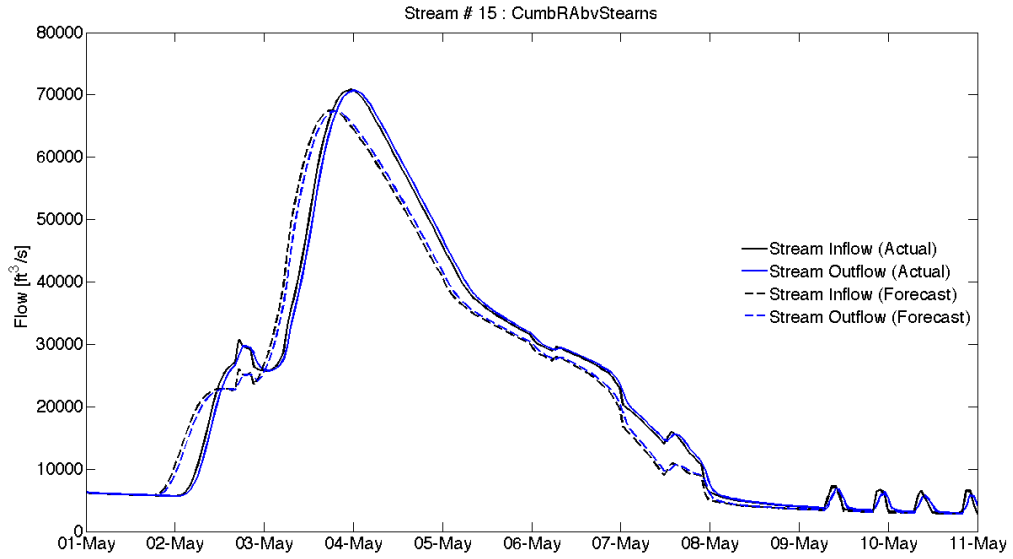


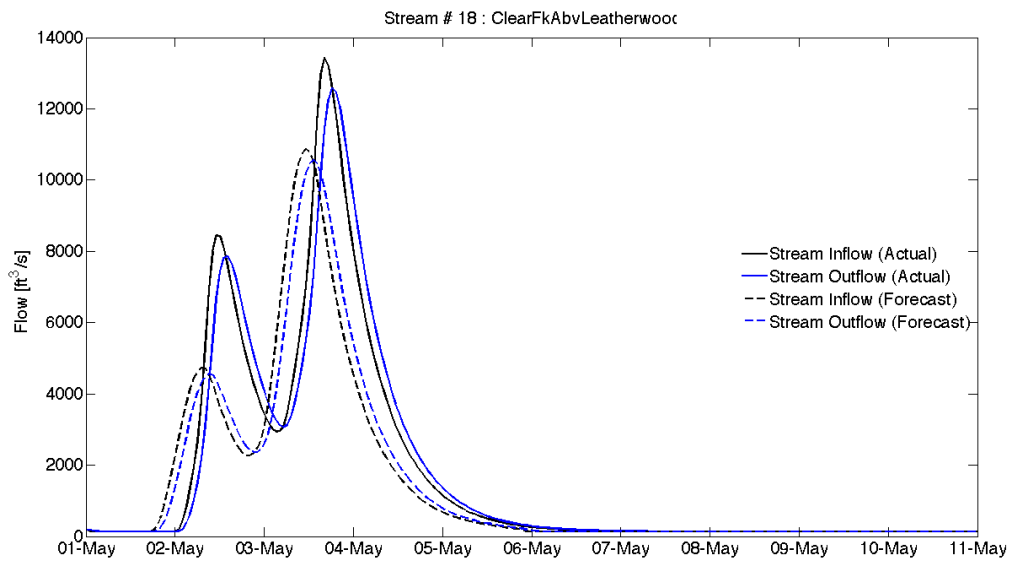
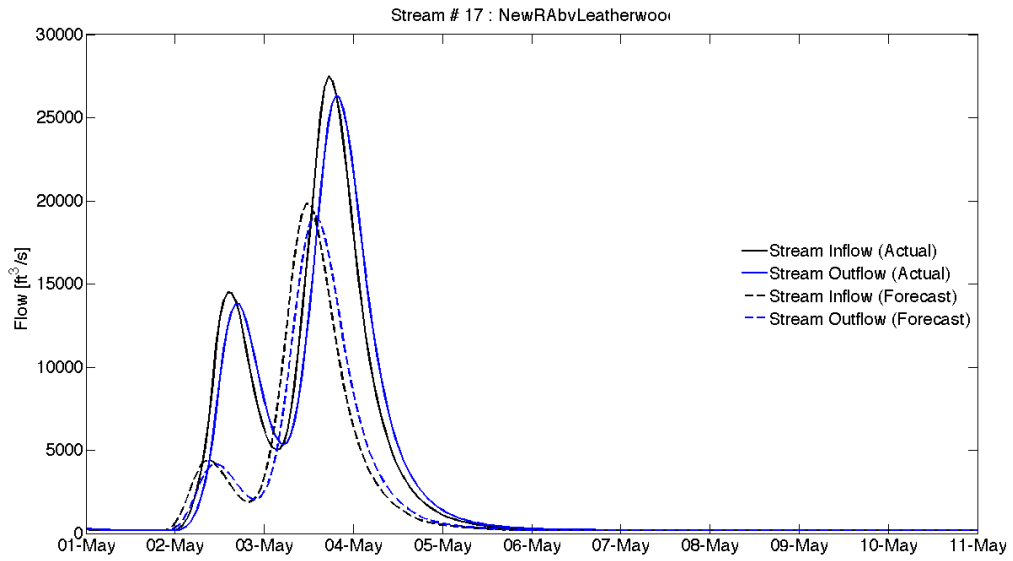


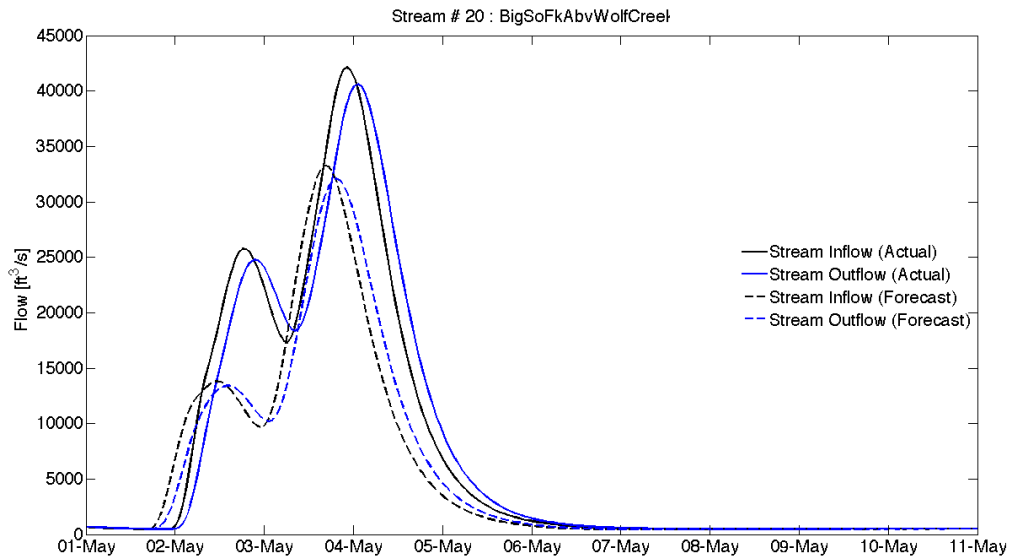
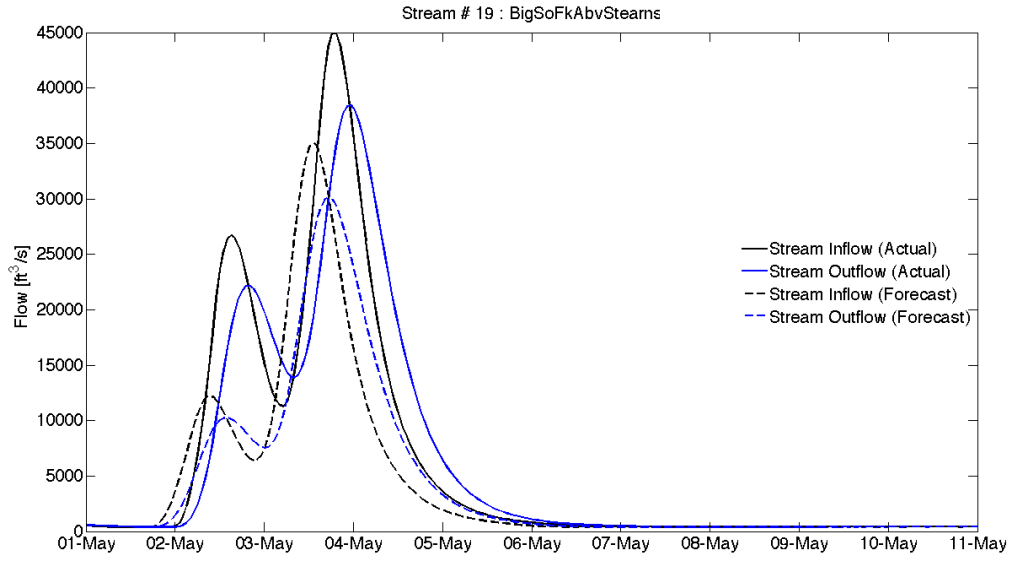


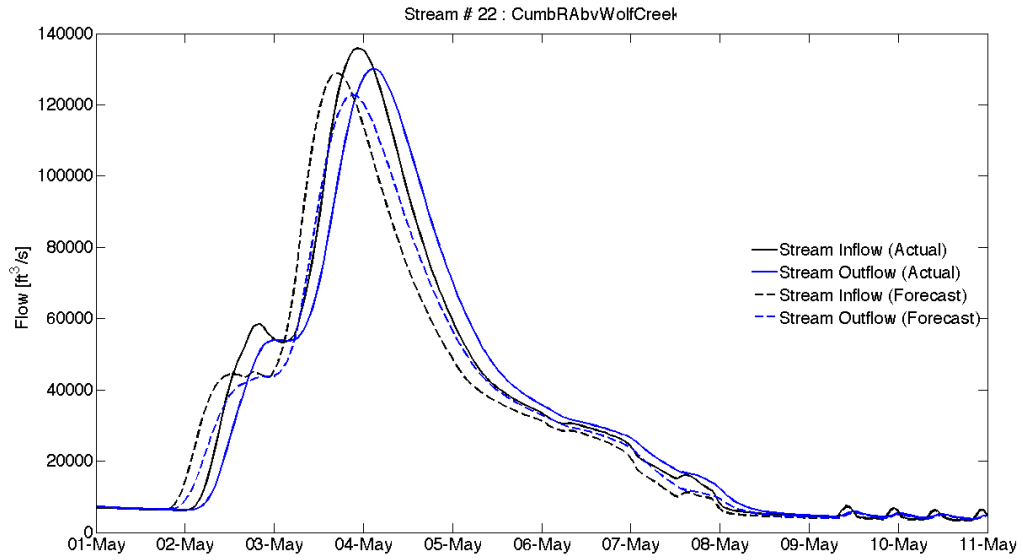
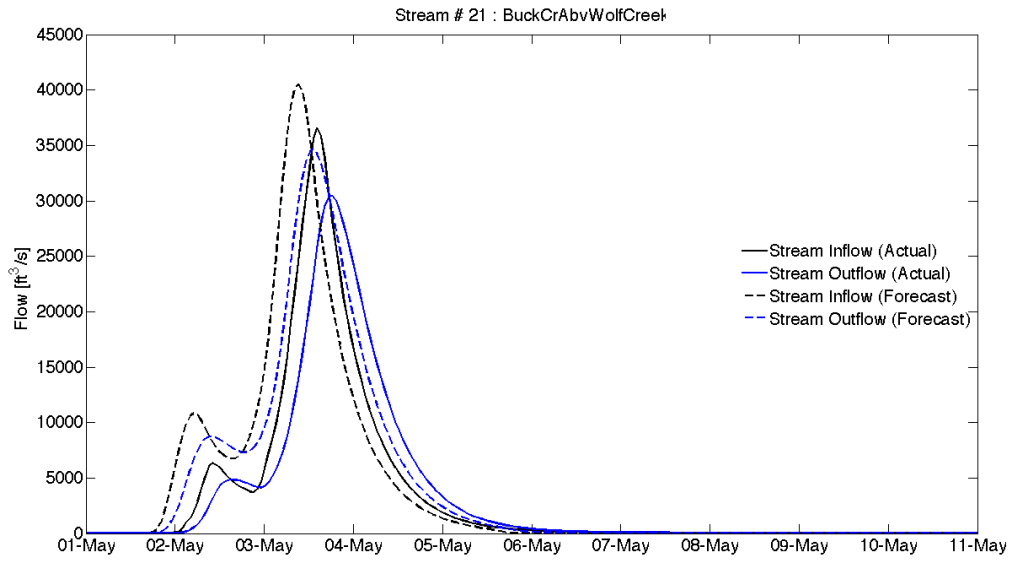


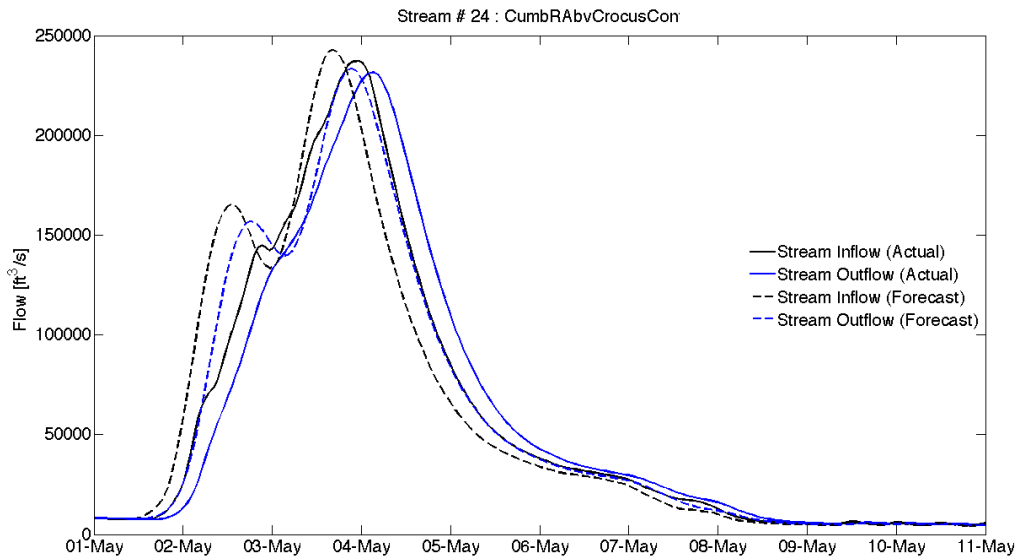
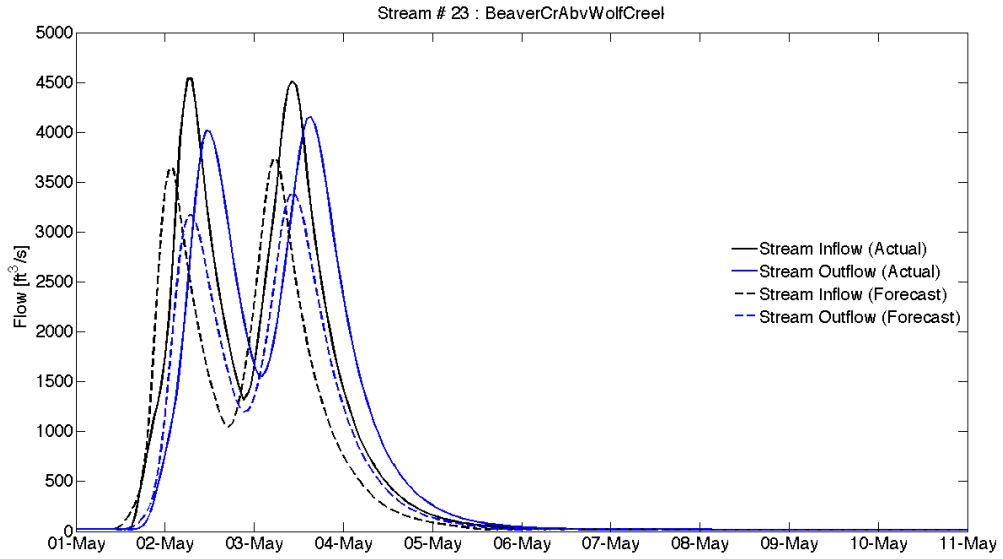


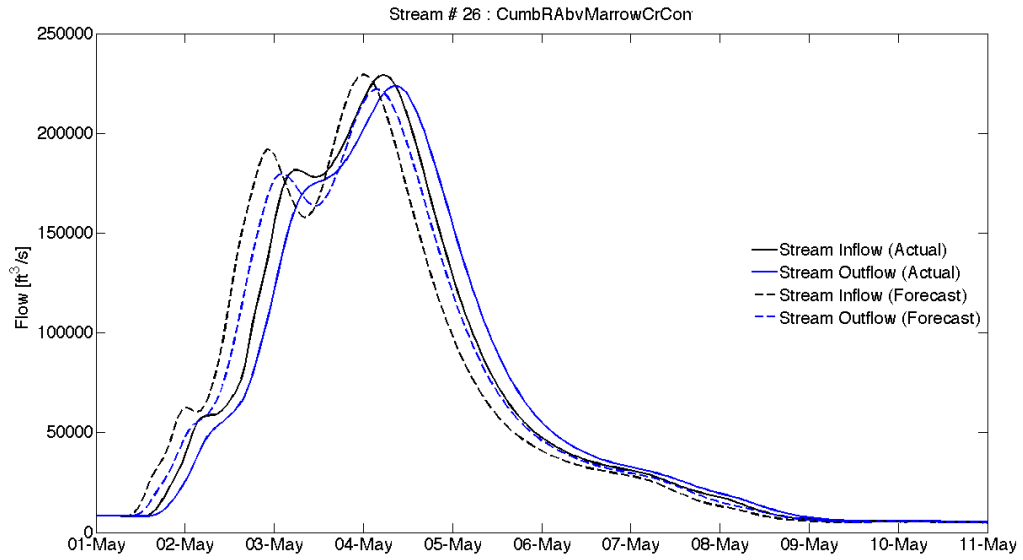
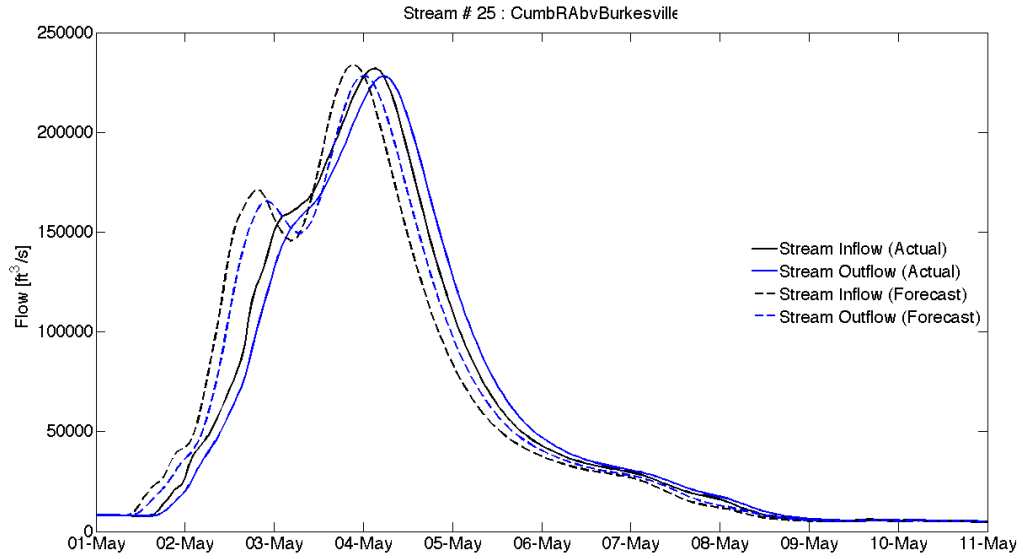


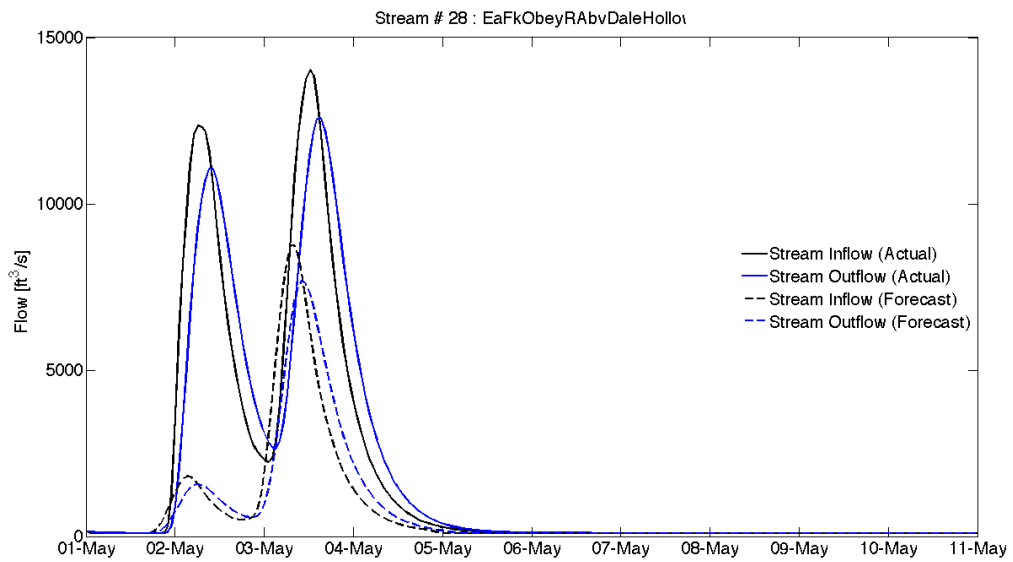
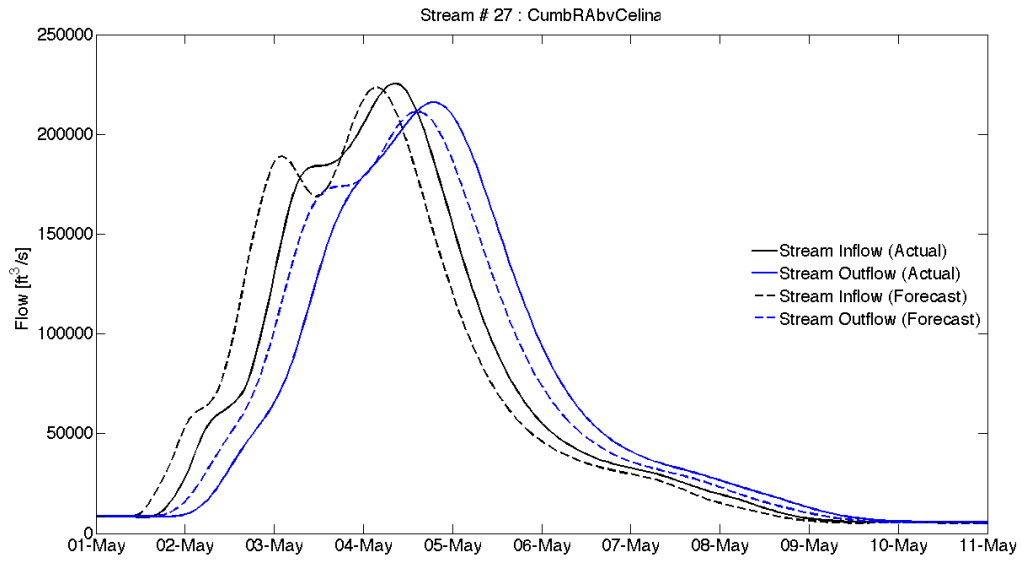


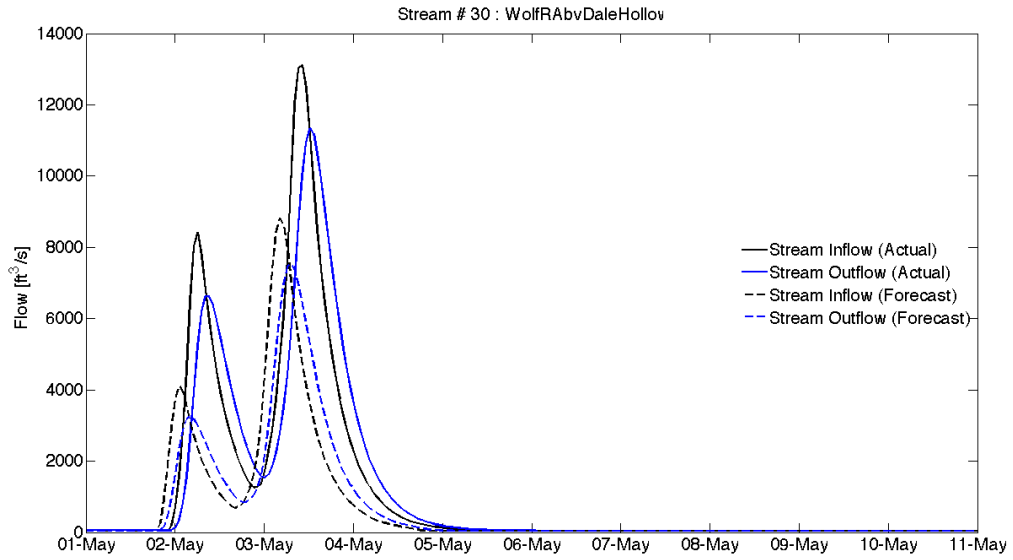
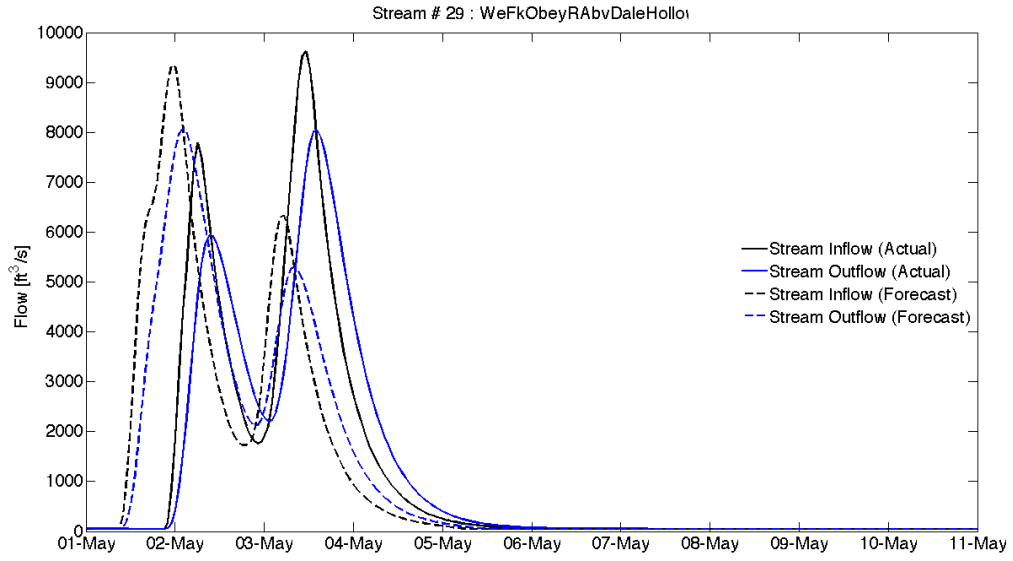


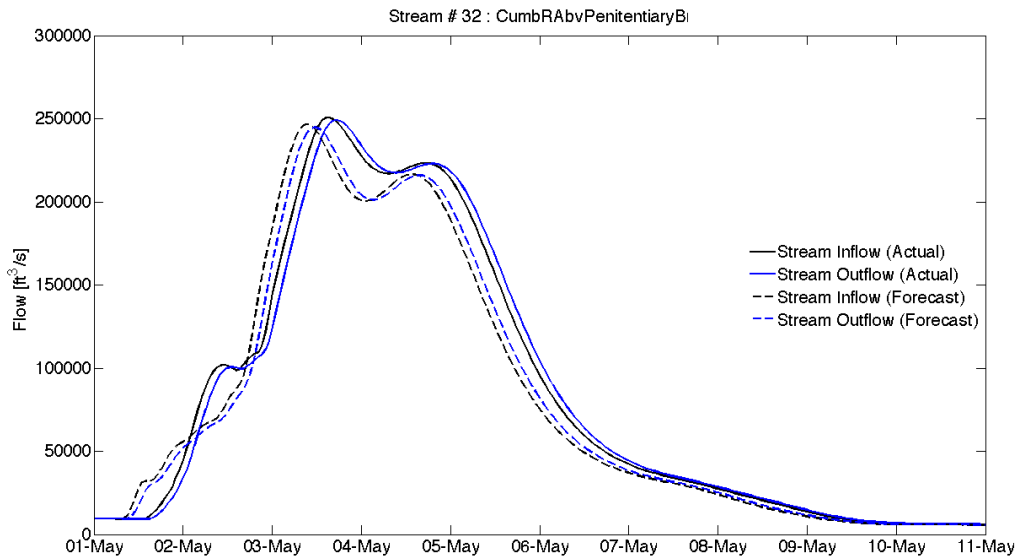
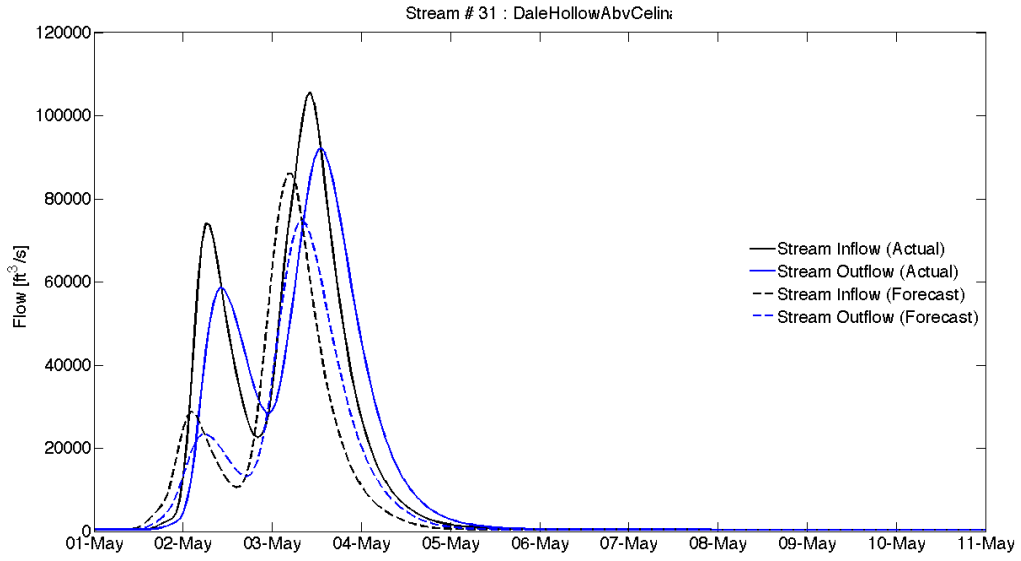


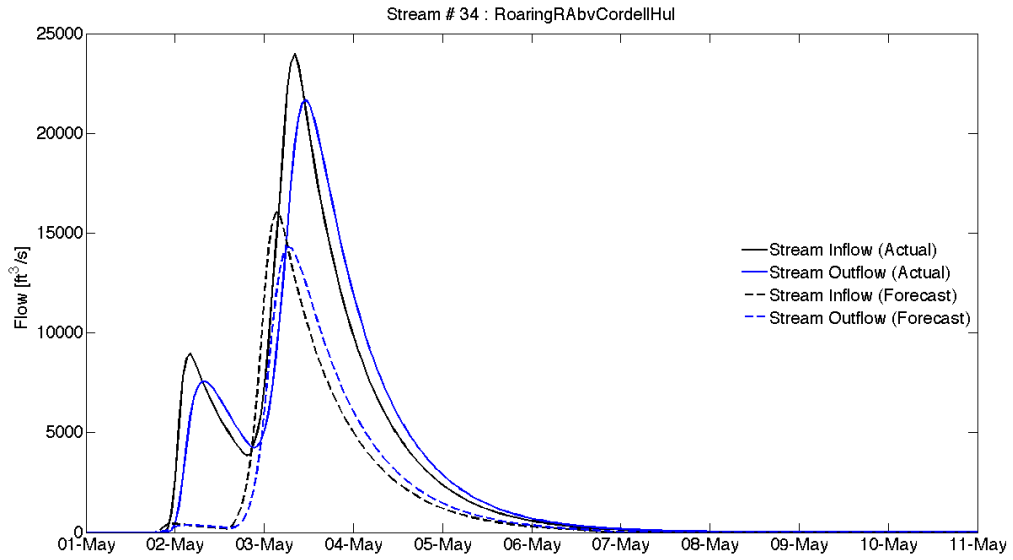
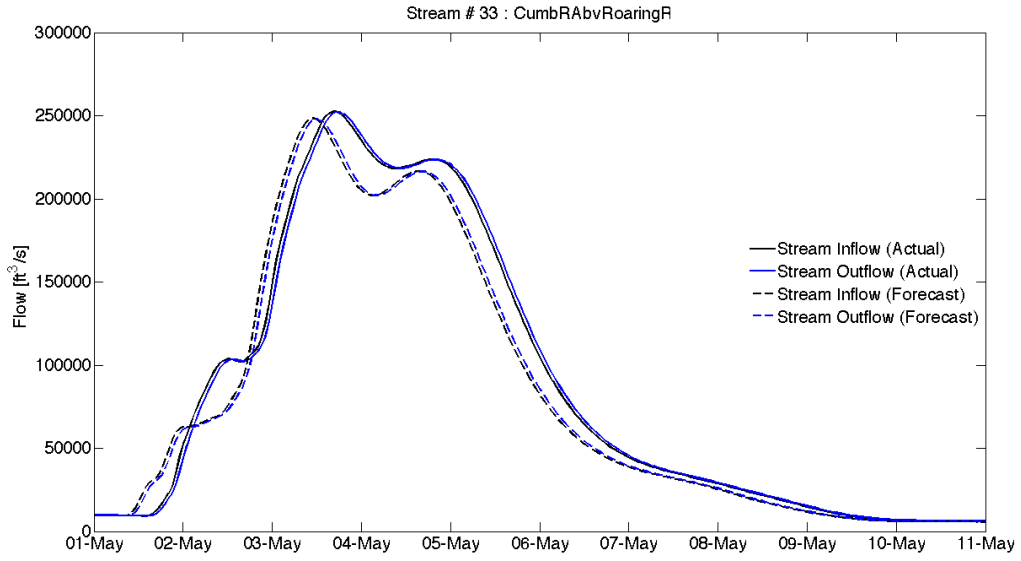


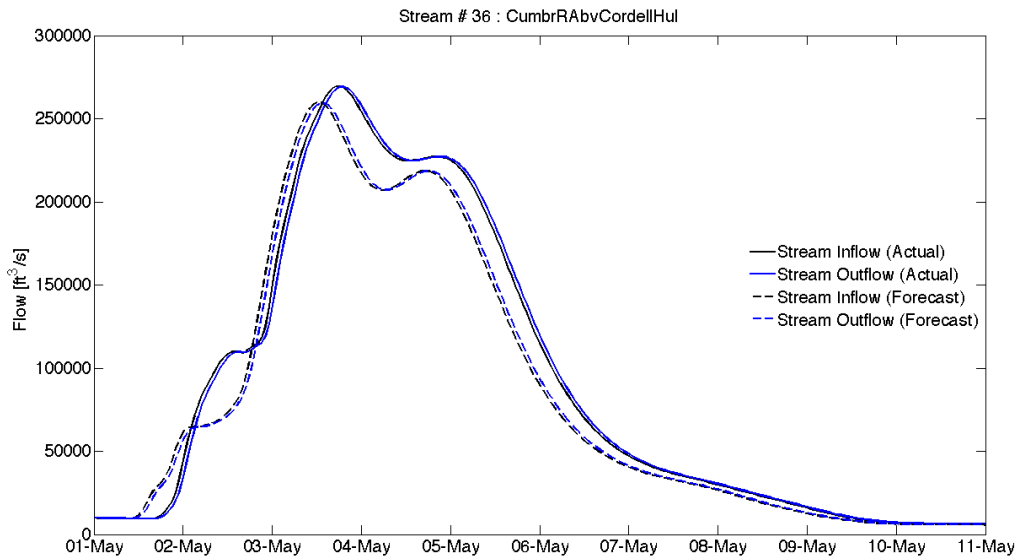
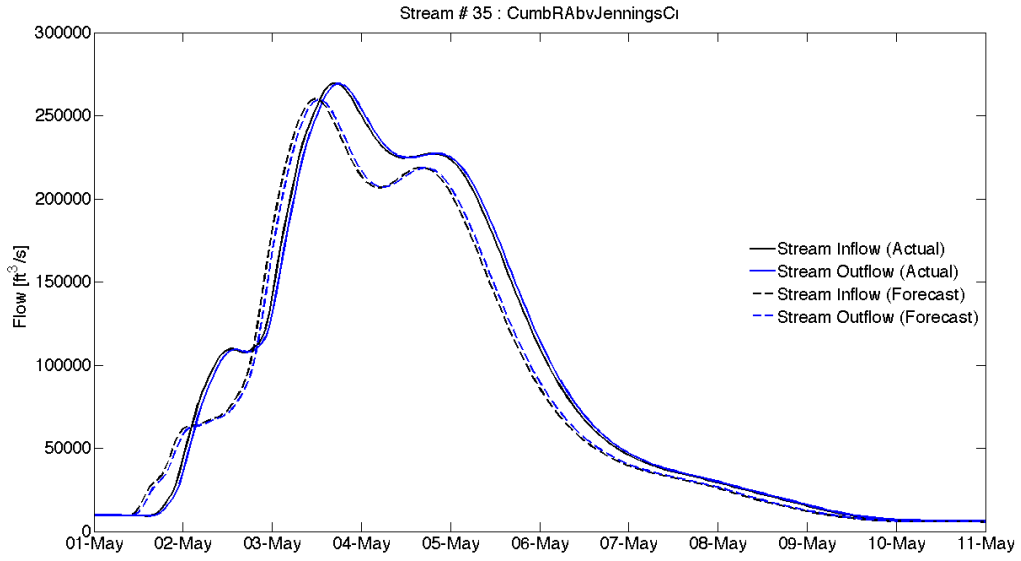


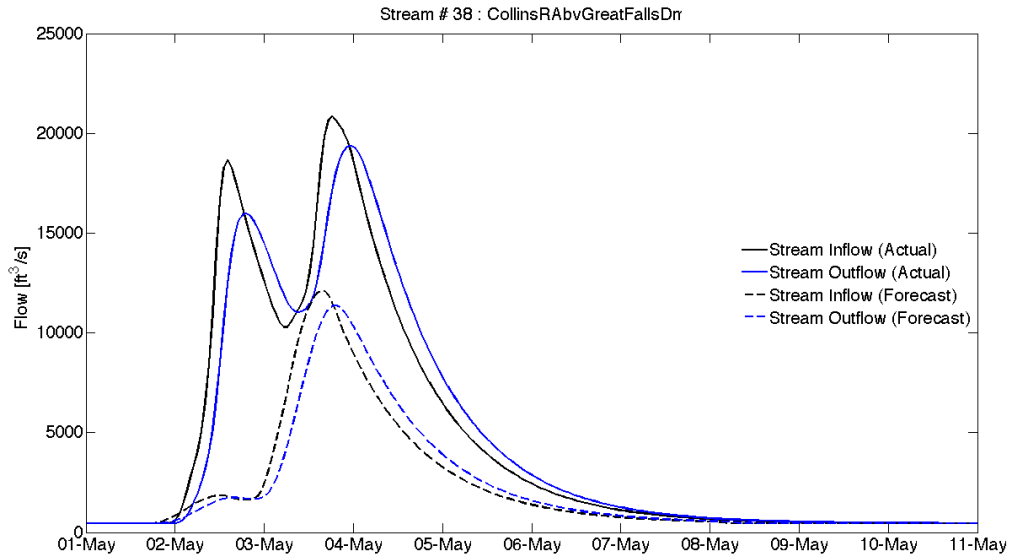
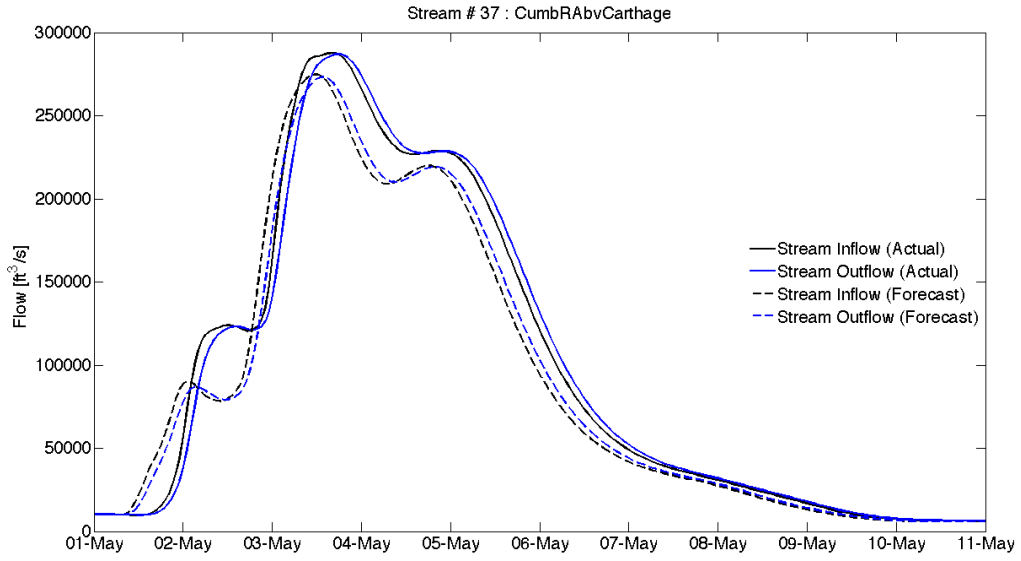


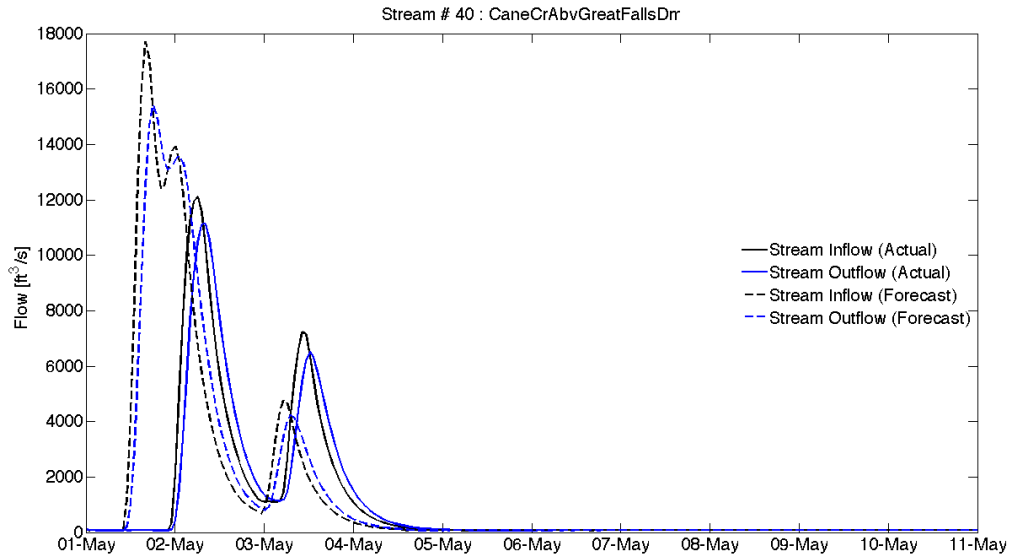
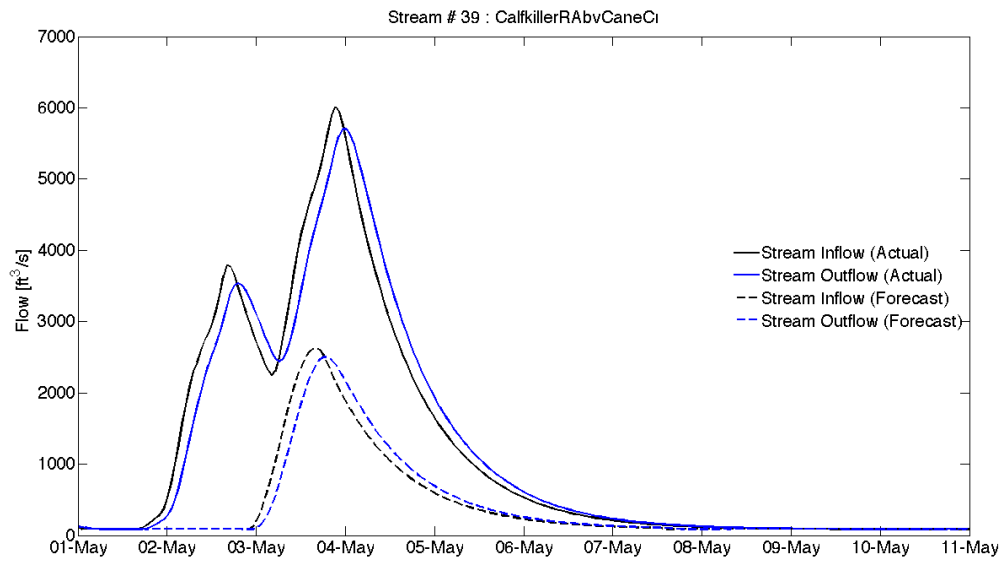


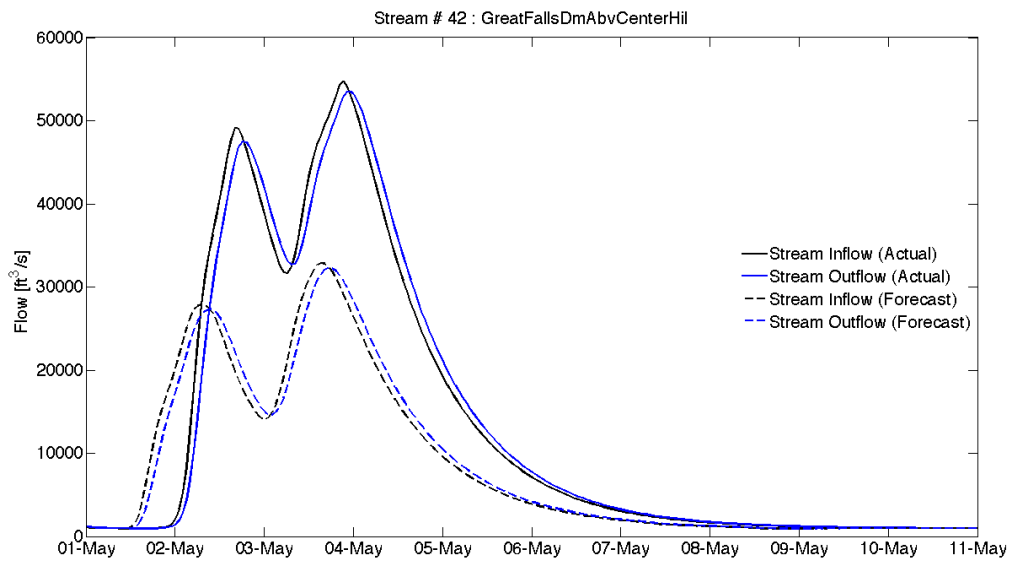
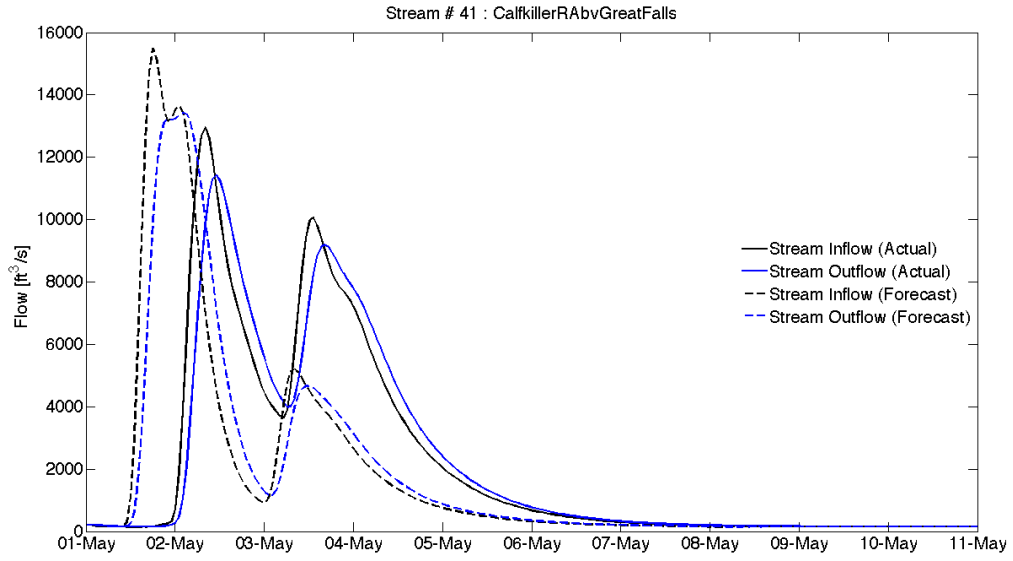


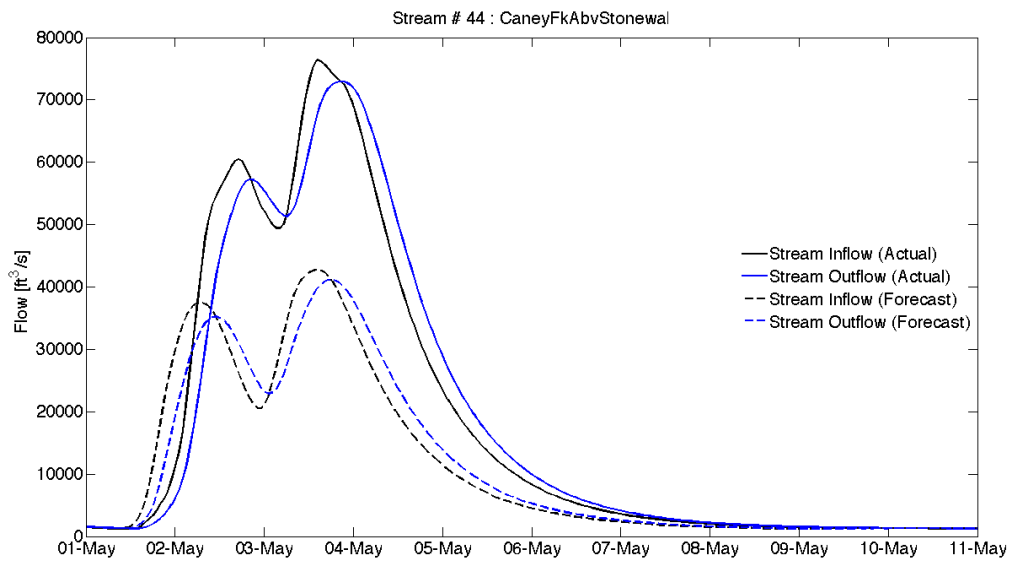
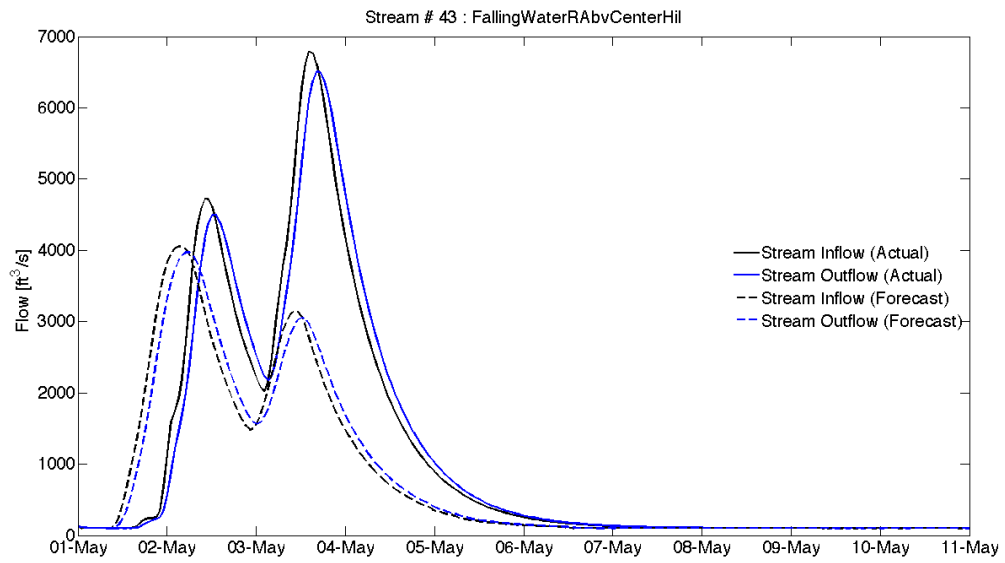


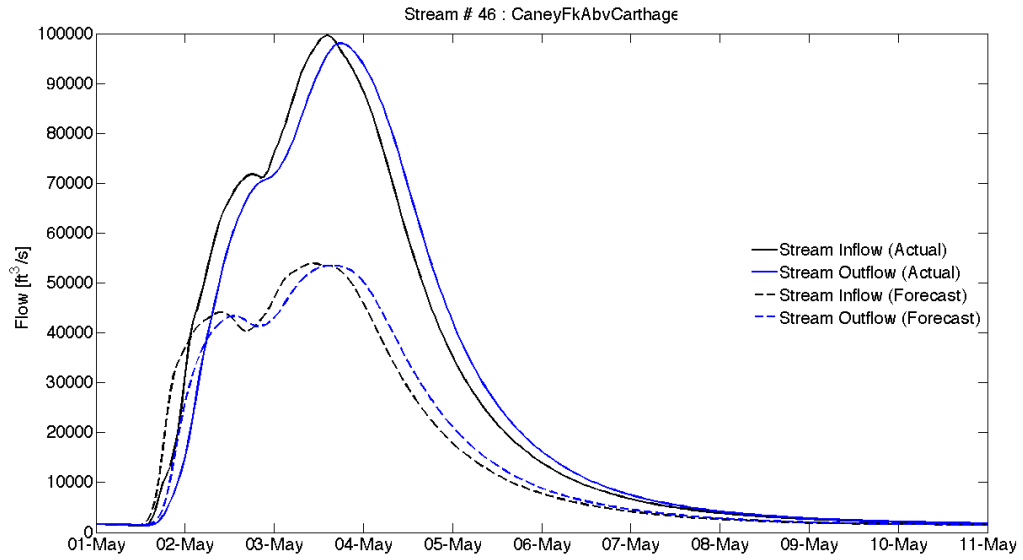
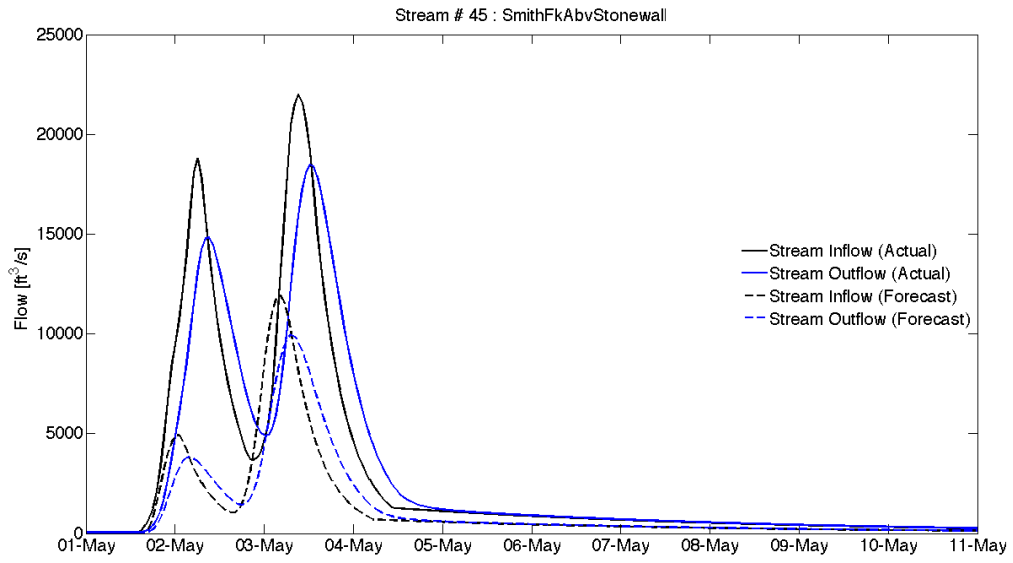


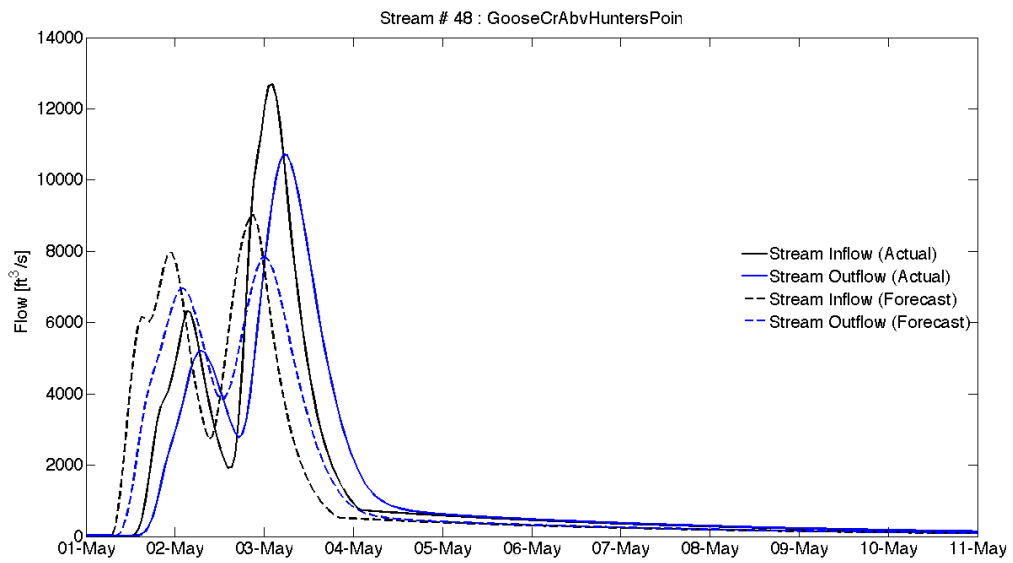
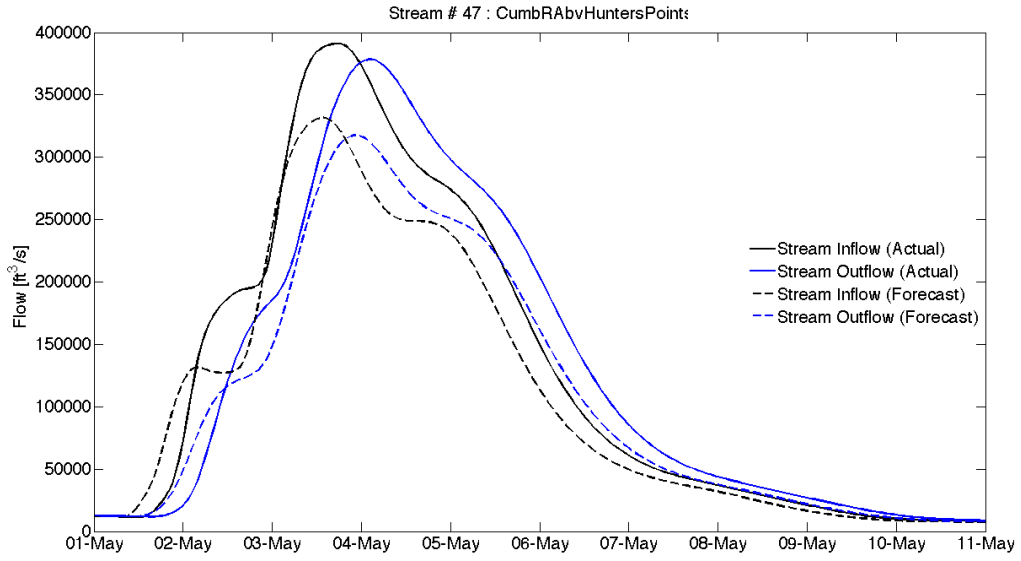


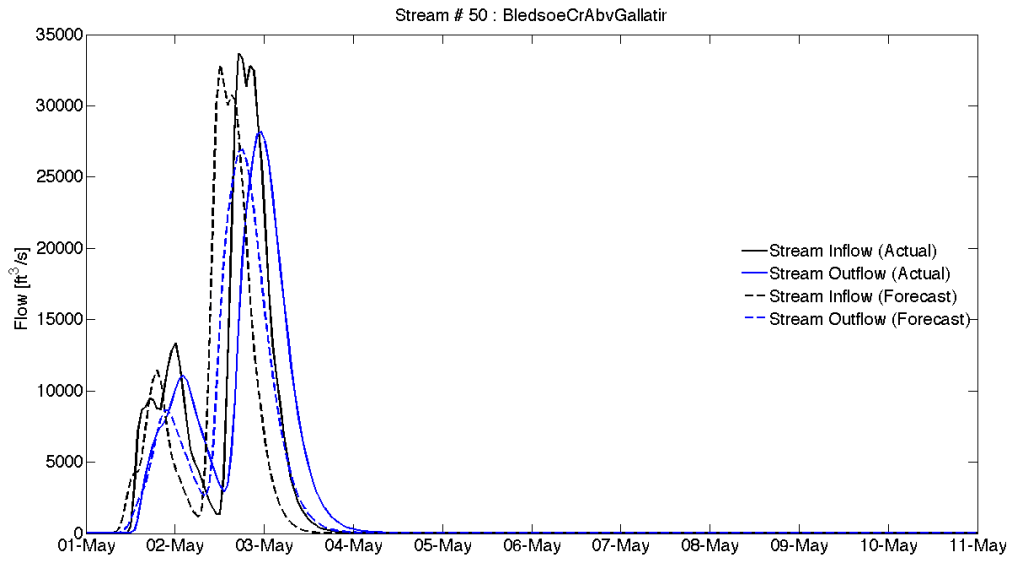
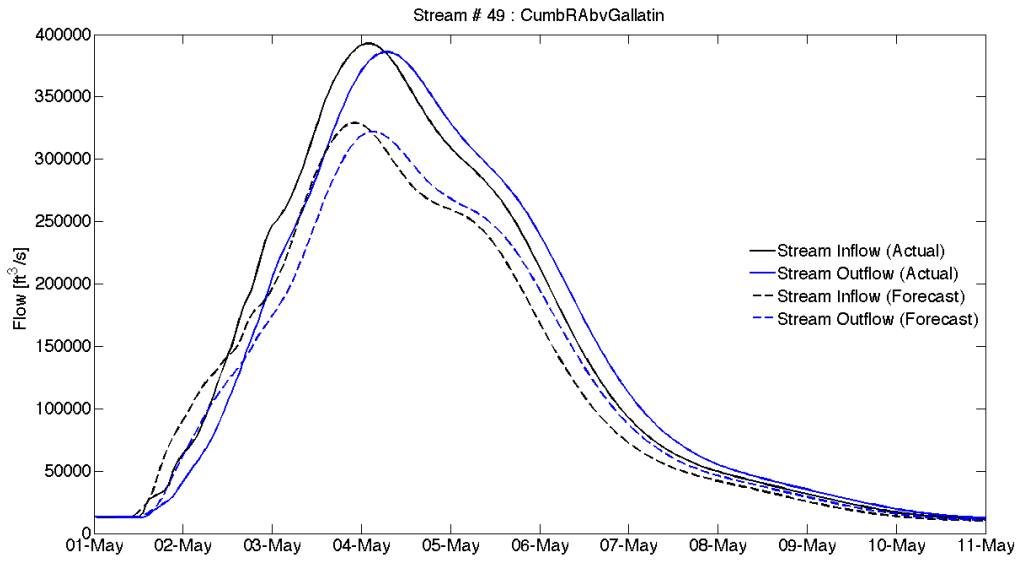


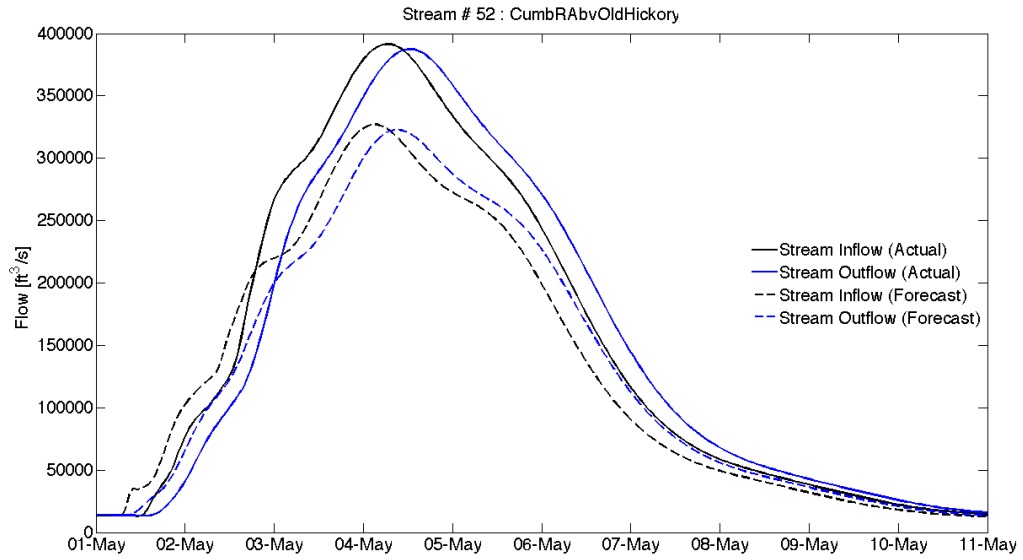
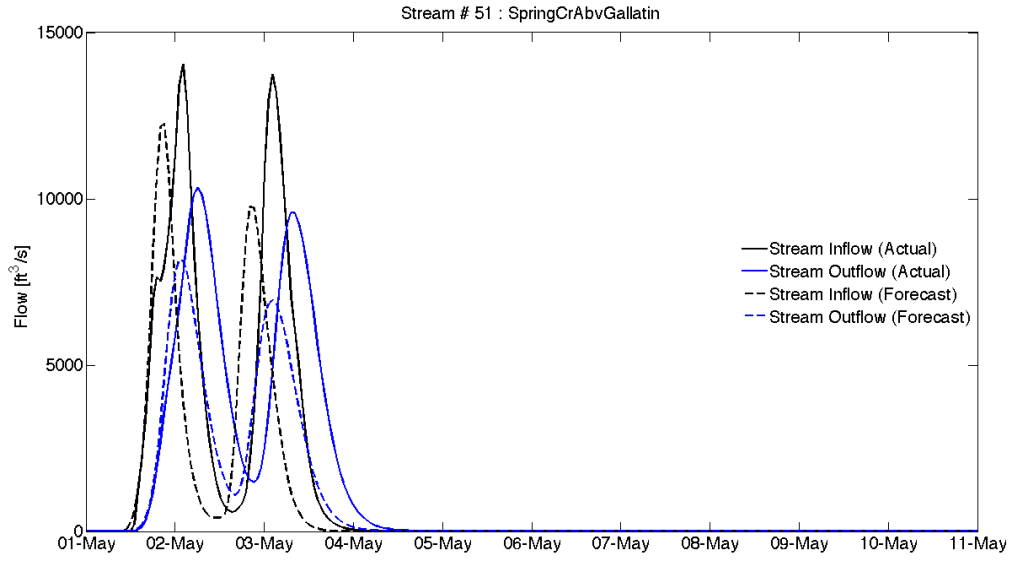


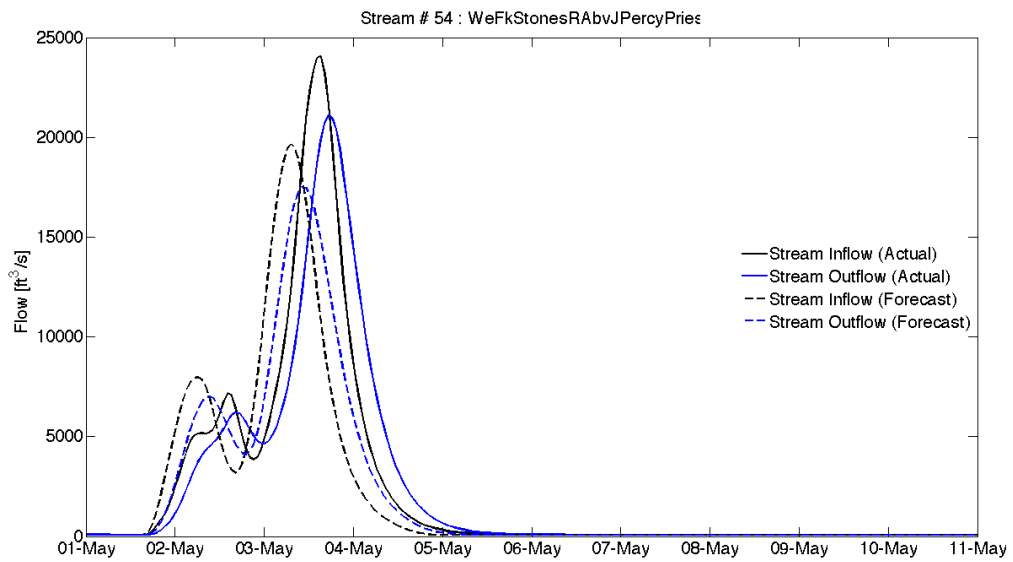
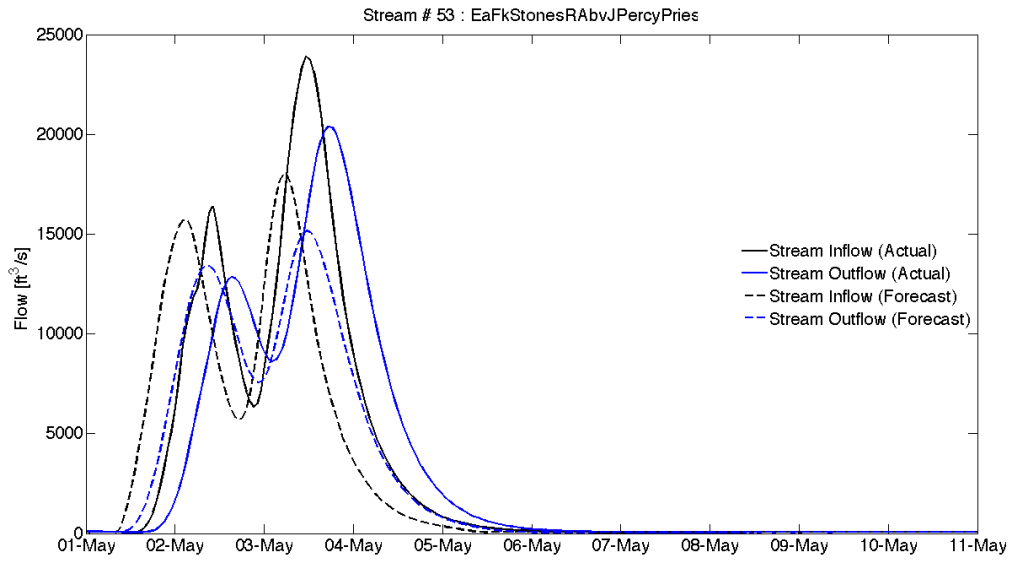


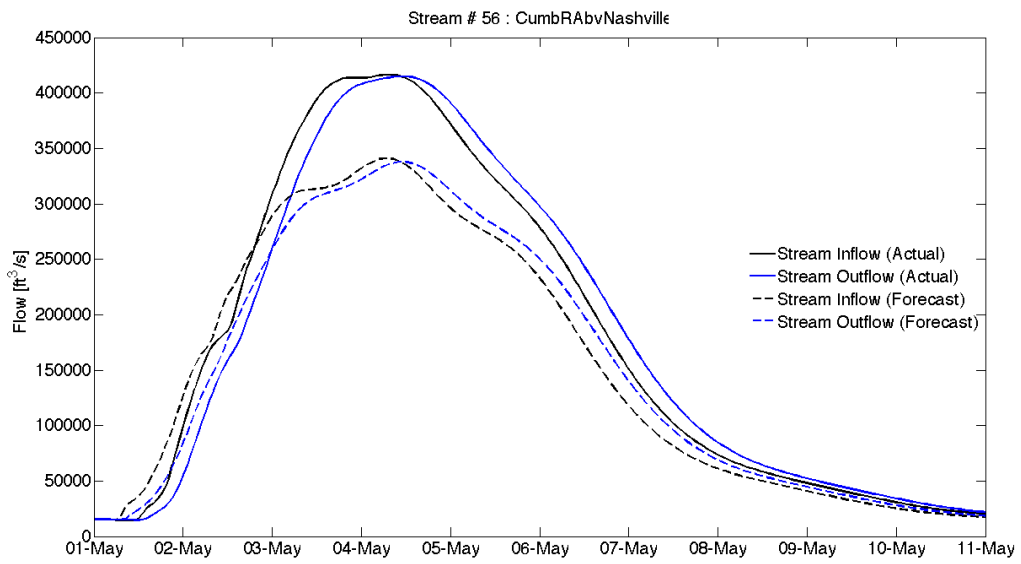
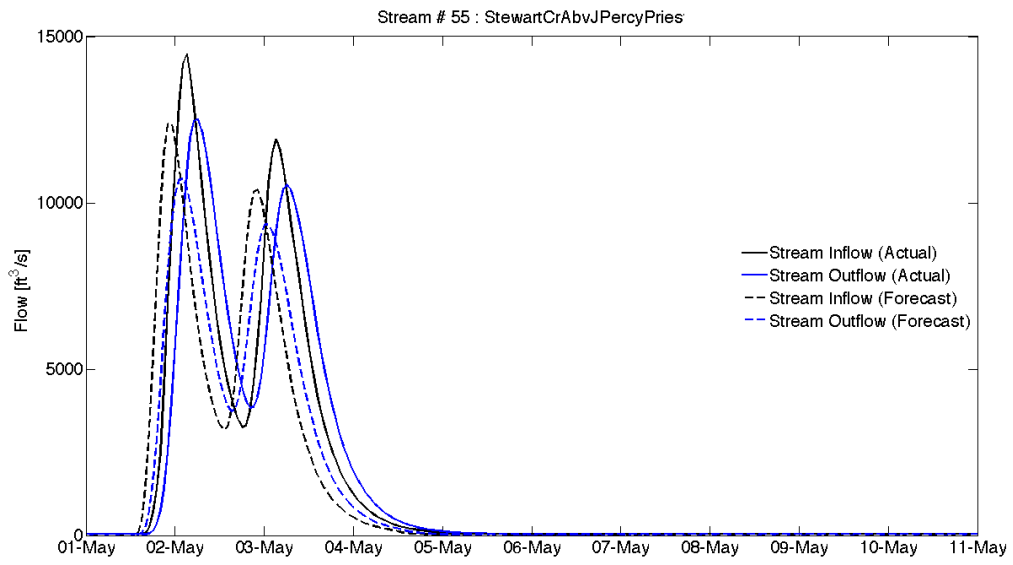


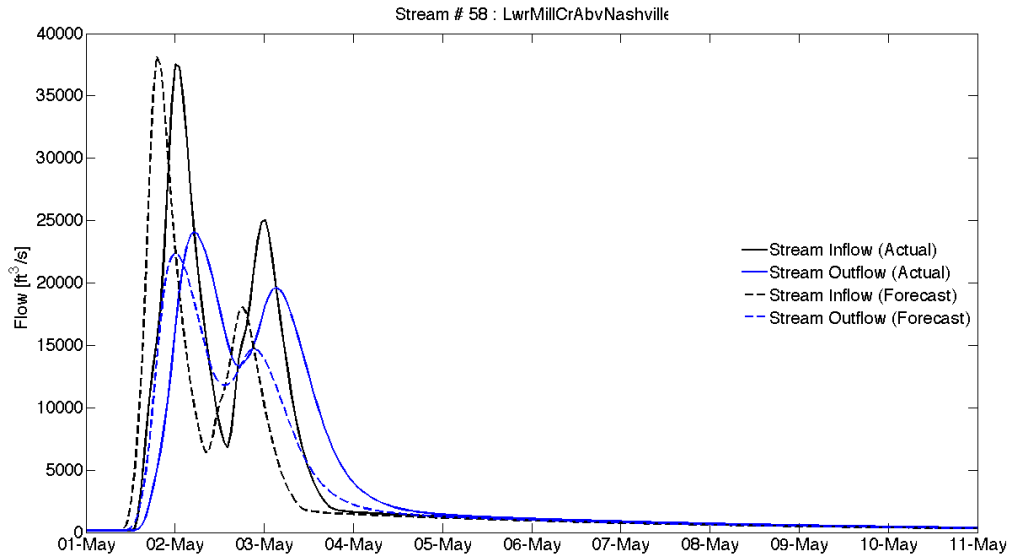
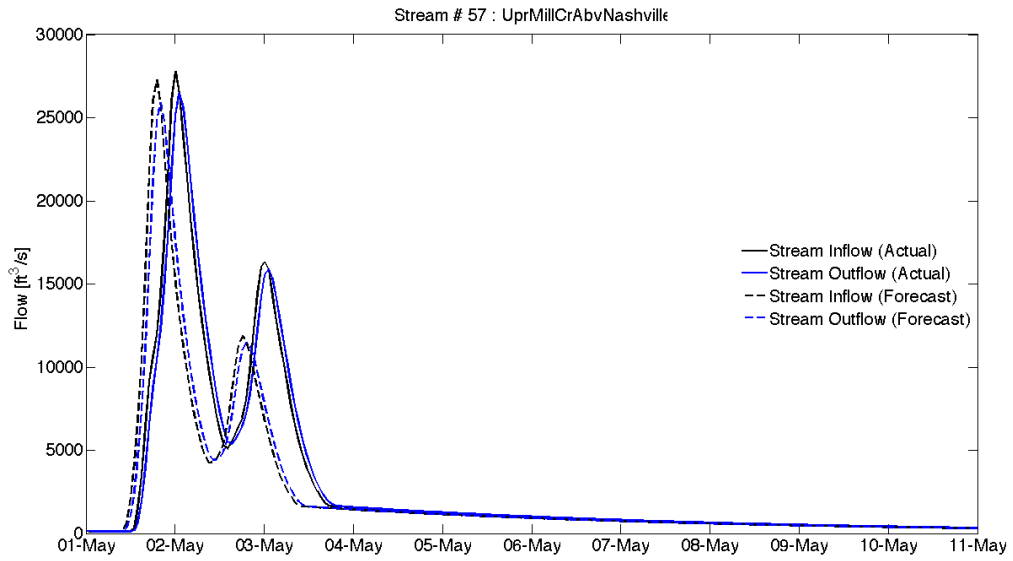


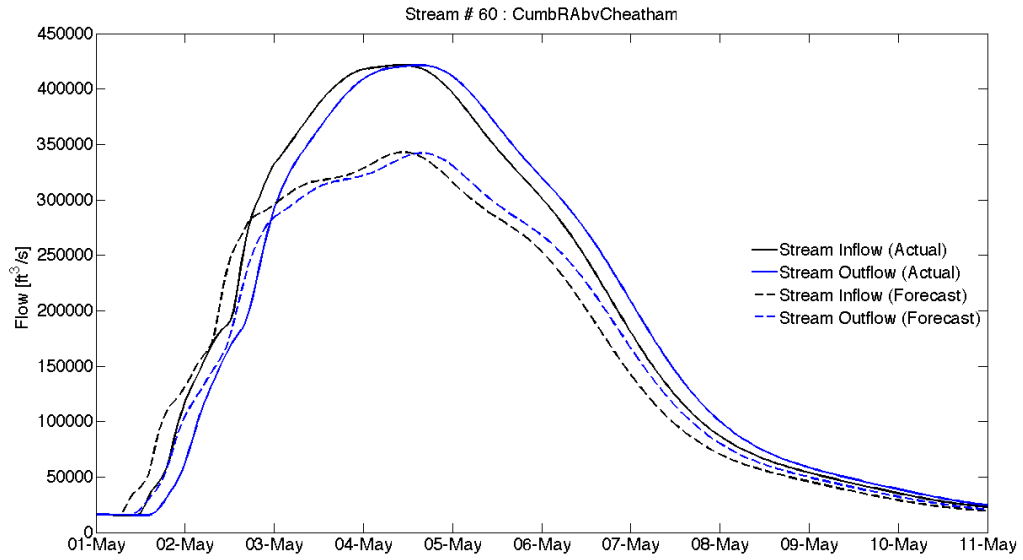
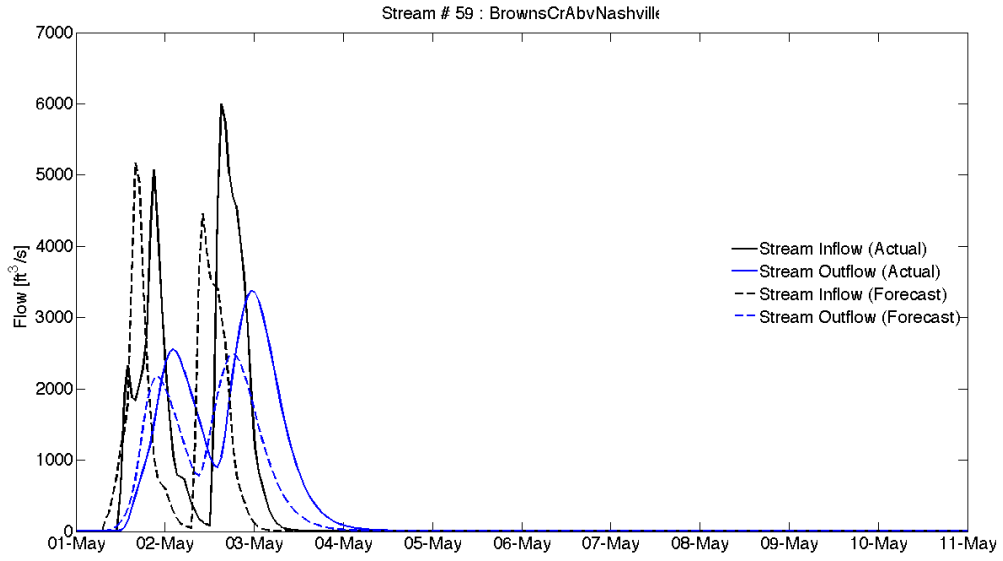


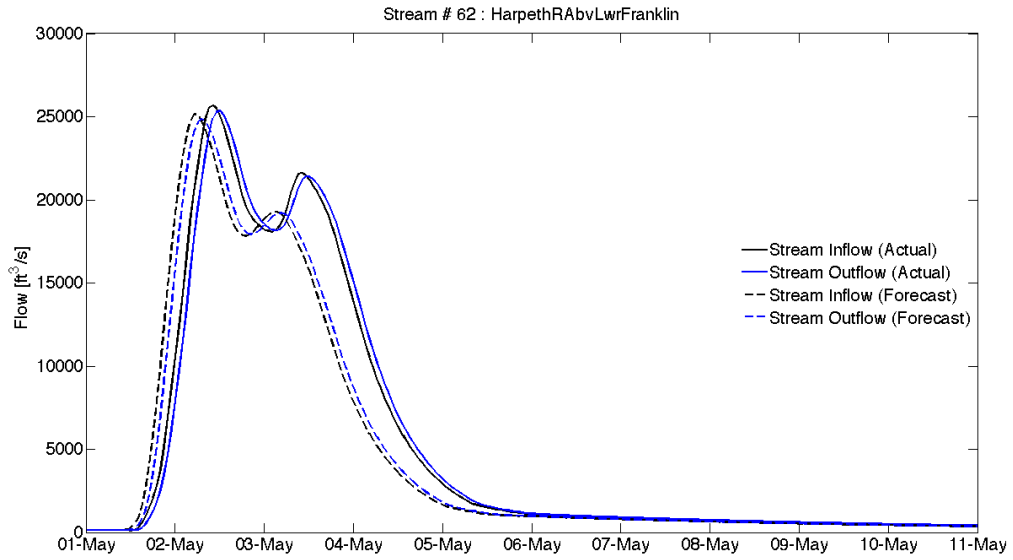
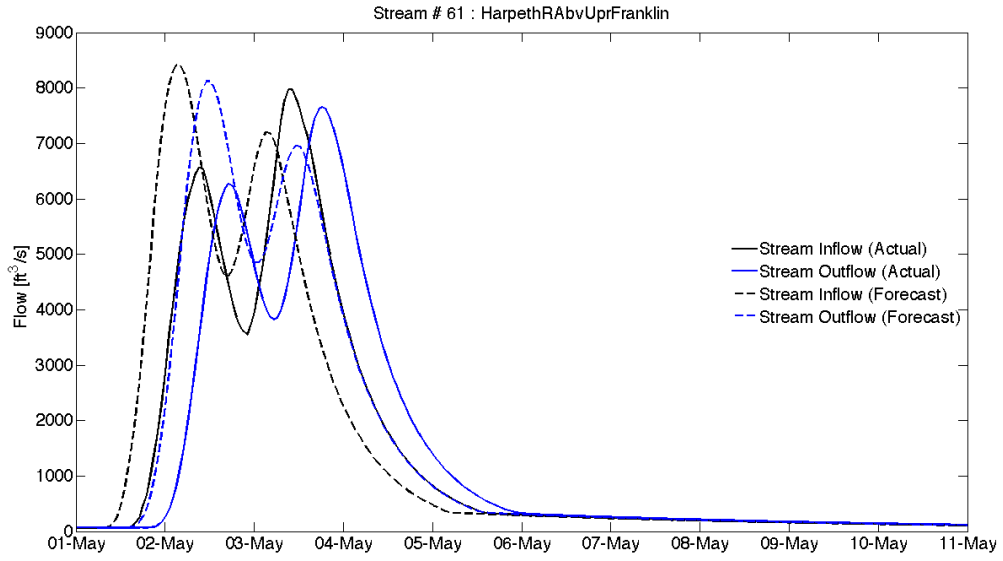


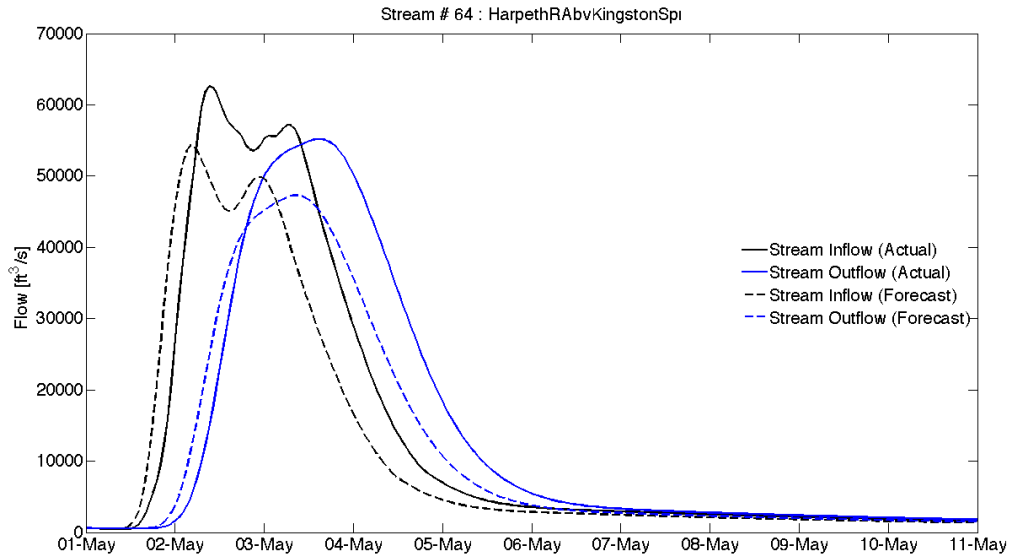
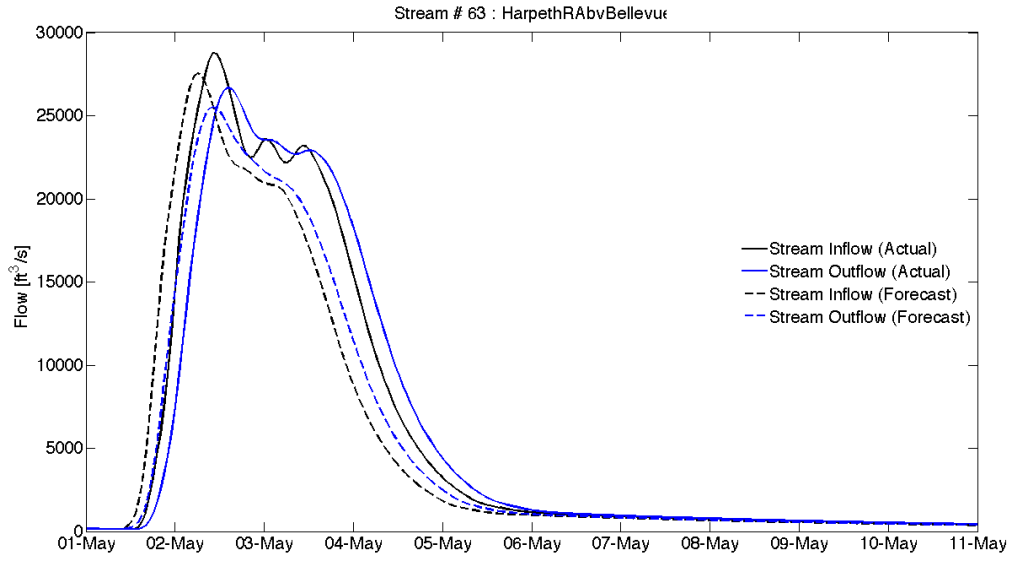


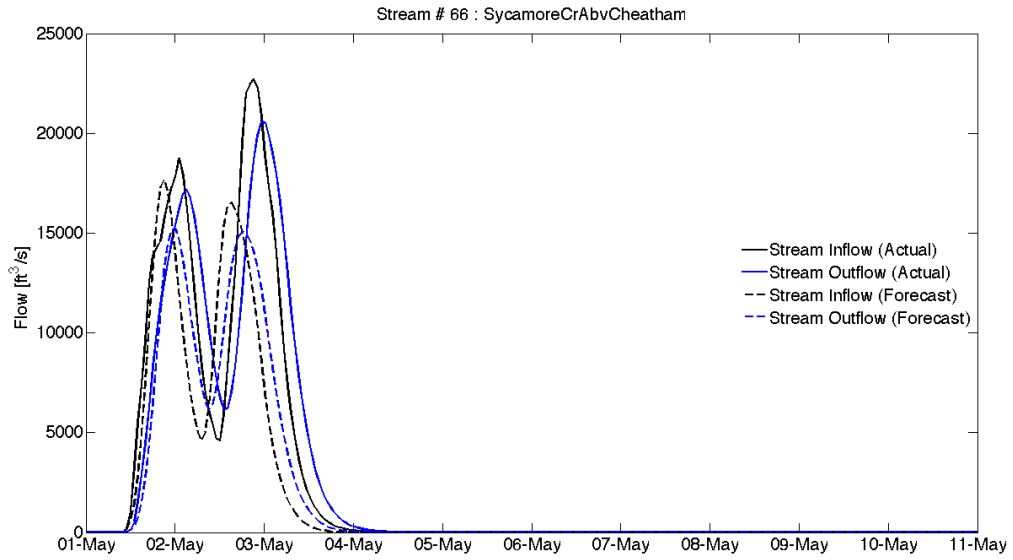
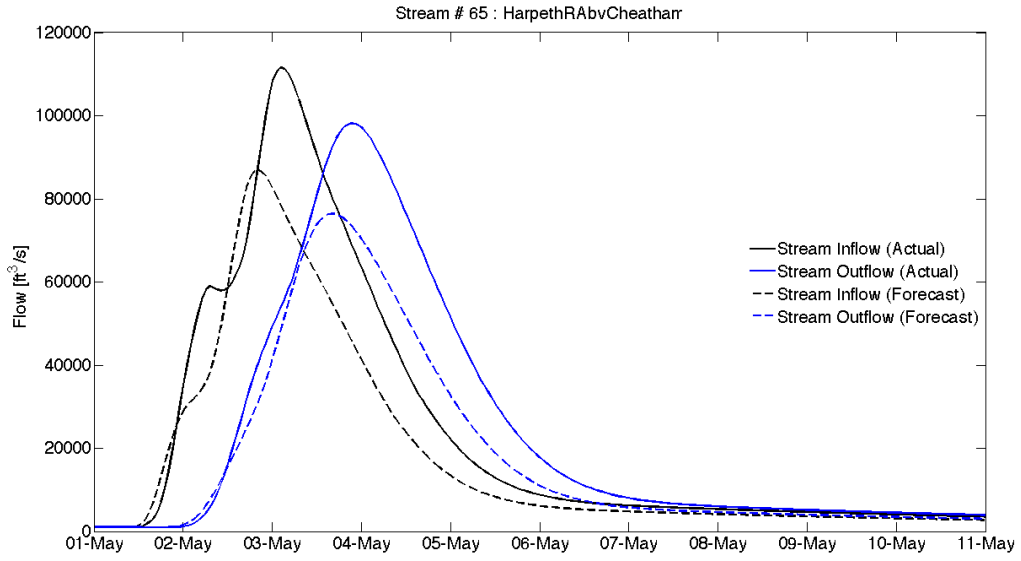












APPENDIX K
RESERVOIR STAGE-STORAGE RELATIONSHIP

Cordell Hull Reservoir

Volume (ac-ft)	Volume (ft³)	Elevation (msl)
0	0	424
8	348480	425
572	24916320	430
2160	94089600	435
4760	207345600	440
8380	365032800	445
13300	579348000	450
20000	871200000	455
28200	1228392000	460
37800	1646568000	465
49200	2143152000	470
63000	2744280000	475
80600	3510936000	480
103000	4486680000	485
131600	5732496000	490
168600	7344216000	495
214800	9356688000	500
271300	11817828000	505
339700	14797332000	510
420500	18316980000	515

Old Hickory Reservoir

Volume (ac-ft)	Volume (ft³)	Elevation (msl)
186000	8102160000	430
196000	8537760000	431
207000	9016920000	432
218000	9496080000	433
230000	10018800000	434
243000	10585080000	435
256000	11151360000	436
271000	11804760000	437
286000	12458160000	438
302000	13155120000	439
319000	13895640000	440
338000	14723280000	441
357000	15550920000	442
377000	16422120000	443
398000	17336880000	444
420000	18295200000	445
443000	19297080000	446
467000	20342520000	447
492000	21431520000	448
518000	22564080000	449
545000	23740200000	450
573000	24959880000	451
602000	26223120000	452
632000	27529920000	453
663000	28880280000	454
695000	30274200000	455

J. Percy Priest Reservoir

Volume (ac-ft)	Volume (ft³)	Elevation (msl)
268000	11674080000	480
279000	12153240000	481
290000	12632400000	482
302000	13155120000	483
313000	13634280000	484
326000	14200560000	485
338000	14723280000	486
351000	15289560000	487
364000	15855840000	488
378000	16465680000	489
392000	17075520000	490
406000	17685360000	491
421000	18338760000	492
436000	18992160000	493
452000	19689120000	494
468000	20386080000	495
485000	21126600000	496
502000	21867120000	497
519000	22607640000	498
538000	23435280000	499
557000	24262920000	500
577000	25134120000	501
597000	26005320000	502
618000	26920080000	503
640000	27878400000	504
663000	28880280000	505
687000	29925720000	506
711000	30971160000	507
736000	32060160000	508
762000	33192720000	509
789000	34368840000	510

APPENDIX L

OPERATION RULES OF RESERVOIR DAMS FOR FLOOD REGULATION

Rules of Operation for the Old Hickory Dam under Flooding Condition

The flood regulations for the Old Hickory Dam presented below are from the U.S. Army Corps of Engineers Cumberland River Basin Old Hickory Water Control Manual.

1) Flood Surcharge Storage

The Old Hickory Reservoir Dam does not have any flood control storage capabilities. However, the reservoir does have small amount of space dedicated to flood surcharge storage. The natural valley storage lost due the existence of the dam is replaced by surcharge storage. Flood surcharge storage space is used to restore downstream flood stages to those would have existed had the reservoir never been built. No overall improvements in downstream flood stage conditions are expected from the flood surcharge storage at the Old Hickory Dam.

2) Timing

The best time to utilize the flood surcharge storage is just prior to the peak of the flood, so that the dam peak outflow reduction can be maximized. If the surcharge storage is used in any other time, the river flow at non-peak time would be taken out, and may not reduce the peak stages downstream of the reservoir. If the surcharge storage is utilized too soon, there could be no storage available when the peak flood arrives.

3) Intended Use

The surcharge storage size was determined considering expected flows while the storage reservoirs were being drained. It was not, however, intended to compensate for the heavy local runoff simultaneous with peak releases from Wolf Creek Dam, Dale Hollow Dam, and Center Hill Dam. Even with the J. Percy Priest Dam being built in

1965, which controls the flow in the tributary Stone River, it remains prudent to utilize the surcharge storage only during the peak of a major flood event. In any case additional rain occur while using the surcharge storage for peak flow reduction of a moderate flood, full compensation for lost valley storage would not be possible. Consequently, the surcharge storage and any additional storage that can be gained by pre-flood release should be preserved until it is evident that the storm has passed. Moreover, priority should be given to evacuating surcharge storage over flood control storage.

4) Exception

There is, however, one exception to the policy of conserving all surcharge storage when it is desirable to allow the reservoir to rise above the top of the power pool prior to spill. If the rise is expected to be short term, and the reservoir level is anticipated to not exceed elevation of 445.15 feet, then the spillway releases are not required and the 0.15 feet of used surcharge storage will be evacuated via hydropower generation.

5) Pre-flood Drawdown

Capability of Pre-flood drawdown is limited by the quick response of the Cumberland River basin. There are often only hours between a precipitation event and the increase in the reservoir inflows. However, pre-flood drawdown to elevation 442 feet is allowed upon the direction of the Water Management Section. Since the pool typically maintains in the upper one foot of the three foot power pool, as preferred for recreation, it is doubtful that there will be enough time for the full pool to be emptied by pre-flood drawdown. Nonetheless, releases greater than natural flows should be made at the onset

of a flood to conserve storage for the peak. A maximum flood level rising rate of the one foot per hour in Nashville is used to guild pre-flood drawdown operations.

6) Induced Surcharge

The surcharge storage pool is between elevation of 445 feet and 450 feet. The tops of the spillway gates in the closed position are at 447 feet. In order to utilize the full surcharge storage pool, the gates must be opened as the headwater rises above elevation 447 feet. This is referred to as an induced surcharge operation. Adequate freeboard to prevent water from overtopping the gates must be preserved as the gates are being raised. For this reason, and also to insure appropriate stilling action in the spillway bucket, all gate openings should be operated uniformly. The following table shows the minimum possible spillway releases for various headwater levels, assuming no freeboard below the top of the gates

Headwater Elevation (feet)	Minimum Gate Opening (feet)	Minimum Spillway Discharge (cfs)
445	0	0
446	0	0
447	0	0
448	1	7500
449	2	14880
450	3	22440

7) Control Station

The primary control station of the Old Hickory Dam is Nashville, Tennessee, which is, located approximately 25 miles downstream of the dam. Flow passes through Nashville is directly affected by the releases from both the Old Hickory Dam and the J. Percy Priest Dam. J. Percy Priest Dam is designed for flood control therefore it has a greater flood capacity to affect the flow in Nashville than does the Old Hickory which has a small surcharge storage capacity. The total flow at Nashville is the combined by the Old Hickory and the J. Percy Priest discharge, in addition of the 275 square mile uncontrolled drainage area. Discharge from all three of these sources must be considered when developing an operation plan to achieve a desired flow or a rate of change in flow in Nashville.

8) Control Flow

Flood operations at the Old Hickory Dam are based on the peak stage and the rate of water level rise in Nashville. The Old Hickory Dam is operated in conjunction with the J. Percy Priest to limit the increase in combined releases to 5,000 cfs/hour, which in effect limits the rate of rise at Nashville to about one foot per hour. The Cumberland River basin flood control system above Nashville, which is by far the largest drainage in the basin, is operated to limit the flow at Nashville to a maximum of 90,000 cfs (stage of 35 feet) during flood season and 54,000 cfs (stage of 26 feet) during crop season. Crop season is generally defined as April 15 through December 15, however, actual conditions in the fields are more important than these dates. Flood season is defined as any time other than crop season, but is generally December 15 through April 15.

9) Rate of Release Change Limits

Hourly changes in combined spillway releases from the Old Hickory Dam and the J. Percy Priest are limited to a total of 5,000 cfs for increase and 10,000 cfs for decrease. It is desirable that the limit for decrease is 5,000 cfs per hour as well, and whenever practical, this limit is directly by the Water Management Section. The purpose of these restrictions is to reduce sudden surges downstream, reduce stream bank station erosion, and minimize impacts on navigation.

10) Use of Surcharge Storage

As a flood progresses, the Old Hickory Dam discharges are increased and Nashville flows are allowed to reach control levels before any surcharge storage is used. Once the control flow is reached, the J. Percy Priest Dam discharges are then reduced to maintain the control flow in Nashville. If the control flow in Nashville cannot be maintained while holding the water surface within the power pool, then the flood surcharge storage is utilized. If the headwater is rising faster than 0.15 feet per hour, the Old Hickory releases are increased and the Nashville control flow is exceeded, but the increase in maximum combined spillway releases from the Old Hickory Dam and the J. Percy Priest Dam is limited to 5,000 per hour. This operating constraint remains in effect until all surcharge storage is used, at which time the discharge is increased as necessary to maintain the water surface at the top of the flood surcharge pool, elevation 450. After the reservoir peaks, the maximum discharge reached is maintained until the headwater level recedes back to the top of the power pool or as instructed by the Water Management. The maximum combined decrease in the spillway discharge from the Old

Hickory Dam and the J. Percy Priest Dam is limited to 10,000 cfs per hour, however, decreases of no more than 5,000 cfs per hour are desirable, if possible.

11) Maximum Headwater

Under no circumstances should the headwater be allowed to rise above the top of the structure pool at elevation of 450 feet. During a flood event, this requirement takes precedent over all other operating criteria. When the headwater rises to elevation at 450 feet, the 5,000 cfs per hour limitation will no longer apply and releases may be increased as necessary to avoid any further rises in the headwater.

12) After Flood Crest

After the reservoir level has peaked, the gate setting normally will remain unchanged until the pool level returns to the power pool. Discharges are then to be reduced until the reservoir levels stabilize. Under some circumstances, following this procedure may cause a rapid drop in stages in the Nashville harbor which can adversely impact navigation. If forecast show that following normal procedures would result in undesirable conditions in the harbor, project discharges may start to be reduced before all surcharge storage being evacuated. This results in discharge reductions being spread out over a longer period of time and reduces the rate of fall of river levels in the area near Nashville. The procedures of discharge reduction schedule should follow the Water Management Section.

13) Spillway Gate Operation

During the flood event, the power plant is generally run at full available capacity, 24 hours a day. Adjustments to flow are then made by operating the spillway gates.

Instructions for these operations come from the Water Management Section directly to the powerhouse and may change periodically depending on hydrologic conditions. These can be in one of the two forms. First, the Water Management Section can issue a release schedule. Second, a headwater elevation schedule can be issued. If preferred, Water Management could issue a schedule, which combine the two.

14) Reservoir Release Schedule

If the Water Management Section issues a required release rate schedule, project personnel determine gate openings required to meet such rate, while maintaining the actual flow within ± 2000 cfs of target flows. Releases rate will be rates of flow past the dam and will include hydropower releases.

15) Headwater Release Schedule

If the Water Management Section issues a required headwater elevations schedule, project personnel determine gate openings required to achieve these elevations while maintaining the headwater within ± 0.2 feet of the target elevations. In addition, they must maintain the $-5,000$ cfs and $+10,000$ cfs per hour net change restriction in spillway discharge from the Old Hickory Dam and the J. Percy Priest Dam combined. If the above becomes unfeasible, the power plant operator may increase the headwater variation to ± 0.5 feet of the target elevations. If it becomes necessary to reduce turbine releases to keep the reservoir within 0.5 feet of the designed elevation, plant personnel should advise the Water Management Section.

Rules of Operation for the J. Percy Priest Dam under Flooding Condition

The flood regulations for the J. Percy Priest Dam presented below are from the U.S. Army Corps of Engineers Cumberland River Basin J. Percy Priest Water Control Manual.

- 1) There are two specific modes of operation regarding to flood regulation:
 - 1.1. Normal flood operation where outflows are reduced to provide flood protection for the primary control station in Nashville, Tennessee.
 - 1.2. Emergency flood operation where downstream flood reduction is an objective, but protection of the dam is the main concern.
- 2) Control Station

The primary control location of the J. Percy Priest Dam during flood events is Cumberland River at Nashville, Tennessee. This is 14.7 miles downstream of the confluence of the Stone River with the Cumberland River, or a total of 21.5 miles downstream of the J. Percy Priest Dam. Nashville is the primary damage center within the Cumberland River basin.

- 3) Control Flow

The control flow for Nashville has been established at 90,000 cfs (at the corresponding stage of 35 feet) during flood season and 54,000 cfs (at the corresponding stage of 26 feet) during crop season. Crop season is generally defined as April 15 through December 15, however, actual conditions in the fields are more important than these dates. Flood season is defined as any time other than crop season, but is generally December 15 through April 15. The official flood stage is about 40 feet. The channel

capacity of the Stone River below the J. Percy Priest is approximately 17,000 cfs, disregarding the flooding of low areas at the mouths of the smaller tributary streams.

4) Rate of Release Change Limits

Increase in combined total spillway releases from the Old Hickory Dam and the J. Percy Priest Dam are limited to 5,000 cfs per hour. Hourly decreases are limited to 10,000 cfs per hour. The 5,000 cfs per hour increase limit, which corresponds to approximately one foot per hour rise in the Nashville harbor, can be waived during severe flooding events. Moreover, to reduce surges and prevent excessive bank station in the Stone River due to flooding, increases in spillway releases are limited to 2,000 cfs per hour; and the decreases are limited to 4,000 cfs per hour. These limits also represent J. Percy Priest's maximum contribution to the flow increase limit (5,000 cfs per hour) and decrease limit (10,000 cfs per hour) at the Nashville harbor. Nevertheless, when the flood conditions are extreme, spillway increases greater than 2,000 cfs per hour from the J. Percy Priest are allowed if they are needed to get the outflow to the level designed on the Emergency Operation Schedule, which is discussed in the following section.

5) Gate Operation

When spillway operations are in effect, all gates should be operated uniformly. This type of operation improves the hydraulic efficiency of the spillway and minimizes downstream scouring of the channel. However, to facilitate the computation of spillway discharge, the gates should be operated a whole foot increments even this could result in some unbalanced spillway flow.

6) Normal Flood Operation

The Nashville flow is allowed to reach the maximum desired amount without flood control procedures being implemented. If the Nashville flow is forecasted to exceed the maximum desired level, releases from the J. Percy Priest Dam are curtailed, and flood control storage utilized in a manner that will reduce the flood crest at Nashville as much as the system allows. After the flood crest has passed, utilized flood control storage is evacuated as fast as possible to prepare for any potential floods. When evacuating flood control storage consideration is given to preventing a second flood crest at Nashville, allowing Nashville flows to recede to the maximum desired amount, and limiting the J. Percy Priest Dam discharges to the Stone River channel capacity.

7) Emergency Flood Operation

If forecasts indicate that limiting the project discharge to 17,000 cfs or the Nashville flow to the maximum desired amount would result in water surface in the reservoir rises above the top flood pool level, then emergency operation should be initiated. The Emergency Operations Schedule (EOS) presented in the Water Control Manual shows the guideline of this operation. The purpose of this plan is to prevent the overtopping of the dam while minimizing the discharges as much as the system allows. This is accomplished by utilizing induced surcharge storage by simultaneously opening all spillway gates so that any inflow in excess of the discharge will be stored above the normal top of the flood control pool. When operating according to the EOS projects discharges are increased until the reservoir level peaks. Spillway gates should then remain in their existing opening until the pools falls to the elevation of 504.5 feet. At that time, all gates should

be operated uniformly such that outflow approximates inflow releases recedes to 17,000 cfs. When these conditions are met, normal flood control procedures are to be continued.

Rules of Operation for the Cordell Hull Dam under Flooding Condition

The flood regulations for the Cordell Hull Dam presented below are from the U.S. Army Corps of Engineers Cumberland River Basin Cordell Hull Dam Water Control Manual.

1) Flood Surcharge Storage

The Cordell Hull Dam does not have any flood control storage capabilities. It does have a small amount of space dedicated to flood surcharge storage. These two terms are often confused with each other, but in reality they are quite different. The most significant difference between the two is its intended purpose. Reservoir dams with flood control storage are intended to hold back vast amounts of water during flood events. These projects can substantially reduce downstream flood stages by providing a space to hold floodwaters until a flood crest has passed and the excess water can be released at a rate such that the potential damage is minimized flood surcharge storage. However, a reservoir dam with just surcharge storage does not do that. Surcharge storage replaces natural valley storage lost due to existent of a reservoir. A flood crest moving downstream tends to be accelerated by the existence of the reservoir in the river system. The loss of valley storage can send flood waters into a reach of river quicker than would be the case under natural conditions and subsequently cause stages at downstream points to be higher than would be the case had the dam not been built. To prevent Cordell Hull reservoir from causing such increase in downstream flood depths, the flood surcharge storage space is used to store this excess water and thus return downstream flood stages to those that would have existed had Cordell Hull reservoir never been built. Thus no

overall improvement in downstream flood stage conditions is expected from the flood surcharge storage at Cordell Hull.

2) Size

During the flood season, Cordell Hull has a volume of about 86,000 acre-feet assigned as flood surcharge storage. This equates to about 0.11 inches of runoff from the project drainage area. As a comparison the four major flood control projects in the Cumberland Basin, they include Wolf Creek Dam, Dale Hollow Dam, Center Hill Dam, and J. Percy Priest Dam, are capable of storing from five to seven inches of runoff from their respective drainage basins in their flood pools. This demonstrates the vast difference in capacity to hold back floodwaters between dams with flood control and flood surcharge storage.

3) Wave Travel Time

The typical travel time of a wave through the Cordell Hull subbasins is about 12 hours from Dale Hollow Dam and approximately 30 hours from Wolf Creek Dam. This translates to be an average wave velocity through Cordell Hull Reservoir about six miles per hour and an average velocity in the Cumberland River upstream of the impoundment of about four miles per hour.

4) Timing

Flood surcharge storage is best used just before the peak of the flood to maximize decrease of the peak outflow from the dam. At any other time, use of the surcharge storage will result in taking flow out of the river at non-peak times and may not reduce

the peak stage downstream. If the flood surcharge storage is used too quickly, there could be no storage space remaining when peak arrives.

5) Timing

Flood surcharge storage is best used just before the peak of the flood to maximize reduction of the peak outflow from the dam. Any other time, use of surcharge storage will result in taking out flow out of the river at non-peak periods and may not reduce the peak stages downstream. If the flood surcharge storage is utilized too soon, there could be no space available when the flood peaks arrived.

6) Intended Use

The size of the surcharge storage was determined considering expected flows while the storage reservoirs were being emptied. It was not intended to also compensate for heavy local runoff simultaneous with peak releases from the Wolf Creek Dam, the Dale Hollow Dam, and the Center Hill Dam. Therefore, it is prudent to utilize the surcharge storage only during major flood. If additional rain were to occur while using surcharge storage to reduce the peak of a moderate flood, full compensation for the lost valley storage would not be possible. Thus, the surcharge storage and any additional storage that can be gained by pre-flood drawdown should be preserved until it is clearly evident that the storm has passed. Additionally, priority should be given to evacuating surcharge storage over flood control storage.

7) Exceptional Case

There is one exception to the policy of conserving all surcharge storage where it is advisable to allow the reservoir to rise above the top of the power pool level prior to the

spill. If the rise is anticipated to be short term, and the reservoir level is projected to not exceed 501.15 feet in winter or 504.65 in summer, then spillway releases are not required and the 0.15 feet of used surcharge storage will be evacuated via hydropower generation.

8) Induced Surcharge

The surcharge storage pool is between elevations 501 feet and 508 feet in the winter, and between elevations 504.5 feet and 508 feet in the summer. The tops of the spillway gates in the closed position are at the elevation of 505.72 feet. In order to use the full surcharge storage pool, the gates must be open as the headwater rises above the elevation of 505.72 feet. This is referred to as an induced surcharge operation. Adequate freeboard to prevent water from overtopping the gates must be maintained as the gates are being raised. In addition to this reason, to insure proper stilling action in the spillway bucket, all gates should be operated uniformly. The following tabulation shown below is the minimum possible spillway releases for various headwater levels, assuming no freeboard below the top of gates.

Headwater Elevation (feet)	Minimum Gate Opening (feet)	Minimum Spillway Discharge (cfs)
504	0	0
505	0	0
506	0.3	1710
507	1.3	7540
508	2.3	13505

9) Control Station

The primary control point for releases from the Cordell Hull Dam is Carthage, Tennessee, which is located 5.3 miles downstream of the dam. Flow past Carthage is directly affected by the releases from both the Cordell Hull Dam and Center Hill Dam. Center Hill Dam is a flood control structure therefore it has a greater capacity to affect the flow in Carthage than does Cordell Hull reservoir, which has only a small surcharge storage capacity. The total flow at Carthage Dam is the combination of Cordell Hull Dam and the Center Hill Dam discharges, plus runoff from the 420 square mile of uncontrolled drainage area. Discharge from all three of these sources must be considered when developing an operating plan to achieve a desired flow or a rate of change in flow in Carthage.

10) Control Flow

The control flow for Carthage has been established at 45,000 cfs with a corresponding stage of 20 feet during crop season, and 72,000 cfs with a corresponding stage of 29 feet during the flood season. Crop season is generally understood to be from April 15 through December 15, however these dates may be adjusted depending on actual conditions of the fields. Flood season is designated as anytime other than crop season, generally from December 15 through April 15. The official flood stage in Carthage is 40 feet. It is recognized that control flows (desired maximum flow) for Carthage result in river stages significantly below damage levels. These control flows have been set to leave room in the channel to accommodate additional runoff from subsequent rainfall events during periods when flood control storage is being evacuated from upstream projects. This criterion was set primarily to minimize damage in the areas near Nashville.

11) Rate of Release Change Limits

Hourly changes in controlled spillway releases from the Cordell Hull Dam and Center Hill are limited to a total of 5,000 cfs for increases and 10,000 cfs for decreases. It is desirable to limit decreases to 5,000 cfs per hour as well. The Water Management Section directs this 5,000 cfs /hour limit. The purposes of these restrictions are to reduce sudden surges downstream, reduce stream bank erosion, and minimize impacts on navigation.

12) Use of Surcharge Storage

As flood progress, Cordell Hull Dam discharges are increased and Carthage flows are allowed to reach control levels before any surcharge storage is used. Once the control flow is reached, Center Hill Dam discharges are then reduced to maintain the flow in Carthage. If the control flow at Carthage cannot not be maintained while holding the water surface within the power pool, then flood surcharge storage is utilized. If the headwater is rising faster than 0.15 feet per hour, Cordell Hull releases are increased and the Carthage control flow is exceeded, but the increases in maximum combined spillway releases from the Cordell Hull Dam and the Center Hill Dam is limited to 5,000 cfs per hour. This operating constraint remains in effect until all surcharge storage is used, at which time the discharge is increased as necessary to maintain the water surface at the top of the flood surcharge storage pool level at the elevation of 508 feet. After the reservoir peaks, the maximum reached discharge is maintained until the headwater level recedes back to the top of the power pool. The maximum combined decreases in spillway discharges from the Cordell Hull Dam and the Center Hill Dam is limited to

10,000 cfs per hour, however, if applicable, decreases of no more than 5,000 cfs per hour are desired.

13) Instructions to Operators during Flood

During a flood event, the power plant is generally run at full available capacity, 24 hours per day. Adjustments to flow are then made by manipulating the spillway gates. Instructions for these operations come from the Water Management Section directly to the powerhouse and may change periodically depending on the hydrologic conditions. These can be in one of the two different forms. The Water Management Section can issue a release schedule or issue a headwater elevation schedule. If preferred, Water Management could issue a schedule, which combined the two.

14) Reservoir Release Schedule

If the Water Management Section issues a required release rate schedule, reservoir operation personnel determine gate openings required to meet such rates, while maintaining the actual flow within $\pm 2,000$ cfs of the target flows. Release rates will be rates of flow past the dam and will include hydropower releases.

15) Headwater Elevation Schedule

If the Water Management Section issues a required headwater elevation schedule, project personnel determine gate openings required to achieve these elevations while maintaining the headwater within ± 0.2 feet of the target elevations. In addition, they must maintain the $+ 5,000$ cfs and $- 10,000$ cfs per hour net change restriction in spillway discharge from the Cordell Hull Dam and the Center Hill Dam combined. If the above becomes infeasible, the power plant operator may increase the headwater variations to \pm

0.5 feet of the target elevations. If it becomes necessary to reduce turbine releases to keep the reservoir within 0.5 feet of the designated elevation, plant personnel should seek guidance from the Water Management Section.

16) Maximum Headwater

Under no circumstances should the headwater be allowed to rise above the top of the surcharge pool elevation of 508 feet. During a flood, this requirement takes precedent over all other operating criteria. When the headwater rises to the elevation of 508 feet, the 5,000 cfs per hour limitation will no longer apply and releases may be increased as necessary to prevent any further rises in the headwater.

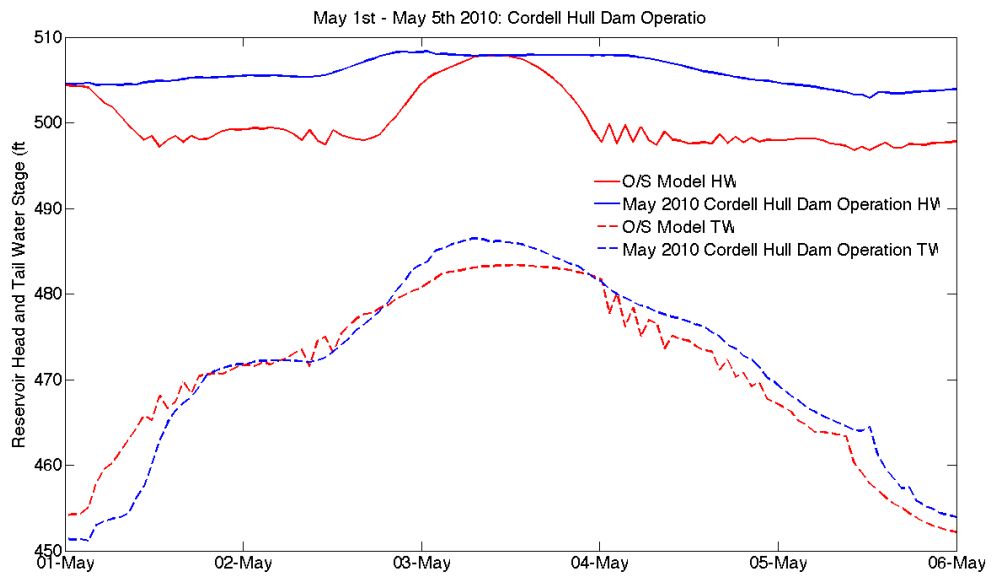
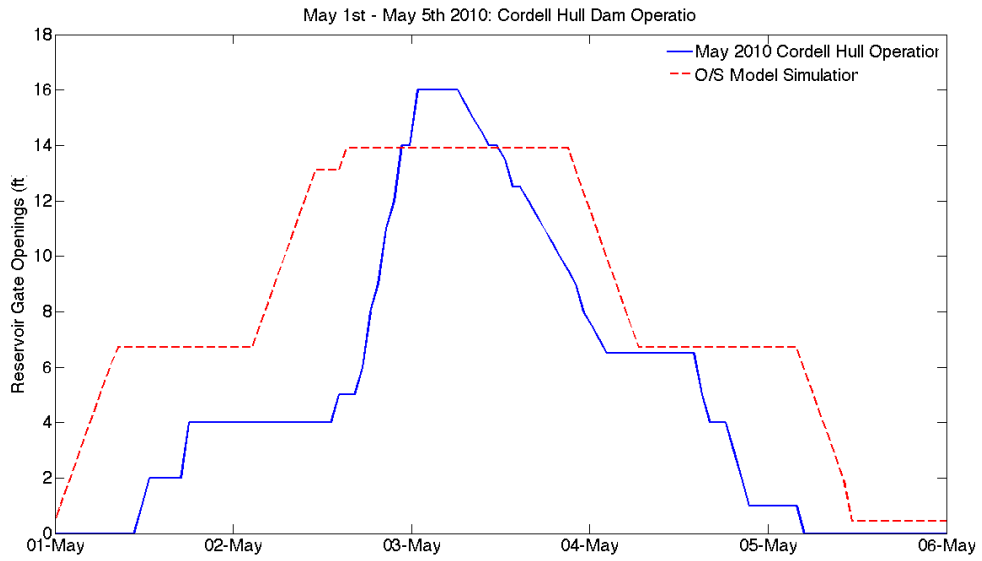
17) After Flood Crests

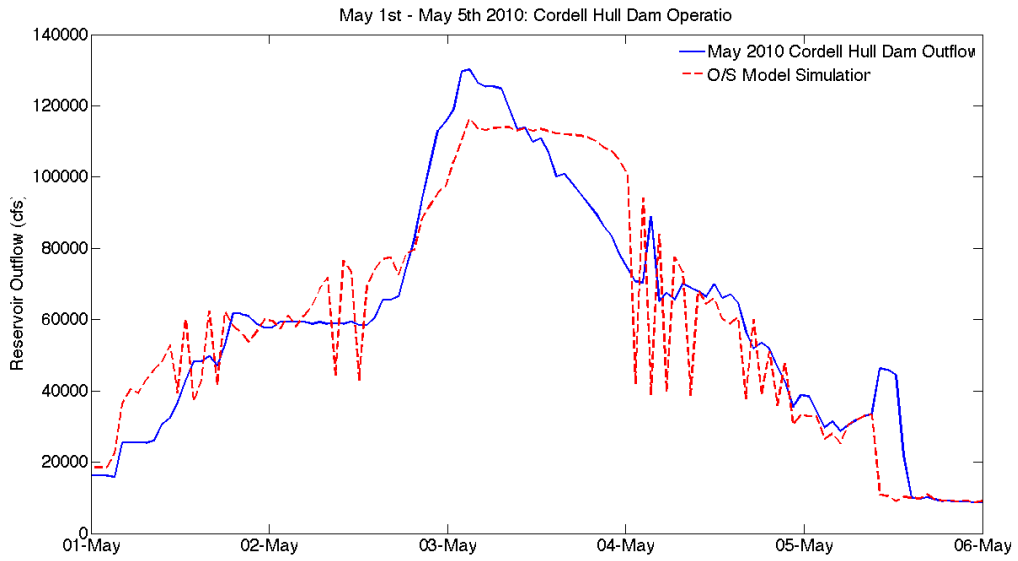
After the lake has peaked, the gate setting normally will remain unchanged until the pool returns to the top of the power pool. Discharges are then to be reduced until lake levels stabilize. Under some circumstances, following this procedure may cause a rapid drop in levels downstream, which can adversely impact navigation. If forecasts show that following normal procedures would result in undesirable navigation conditions, project discharges may start to be reduced prior to all surcharge storage being evacuated. This results in discharge reductions being spread out a longer period of time and reduces the rate of fall of river levels downstream.

APPENDIX M

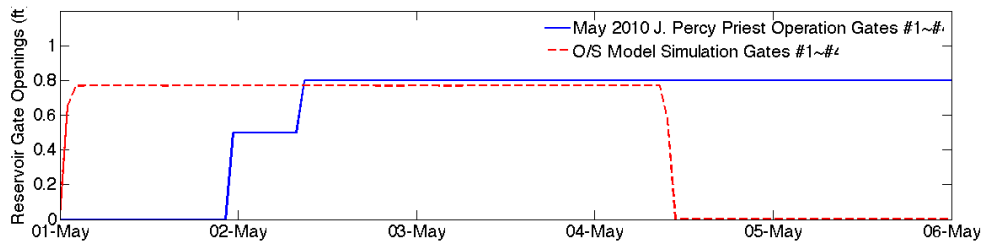
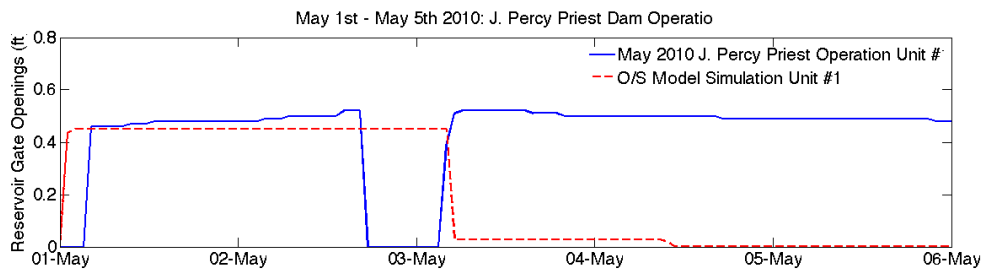
RESERVOIRS SIMULATION RESULTS (CORDELL HULL & J. PERCY PRIEST)

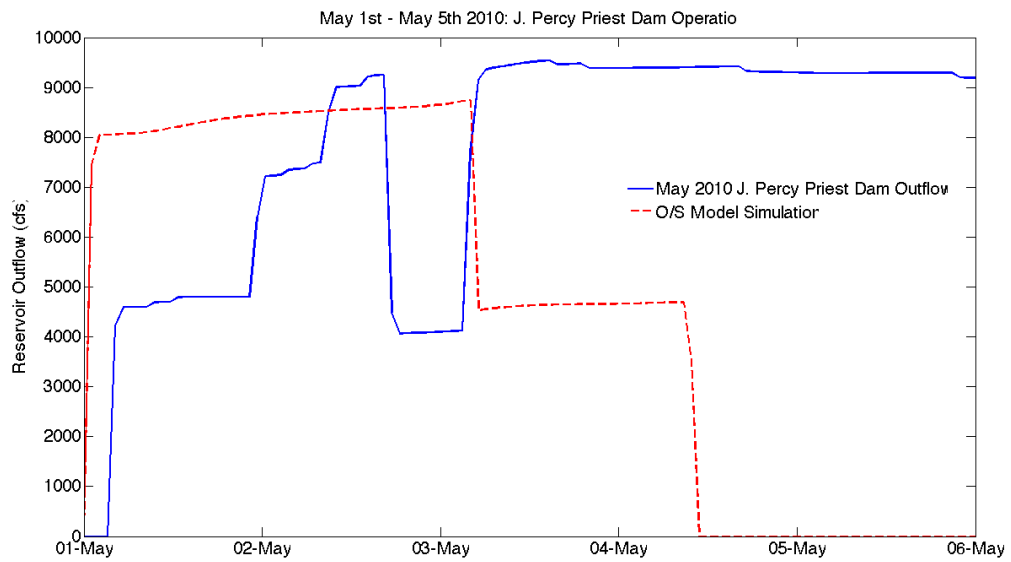
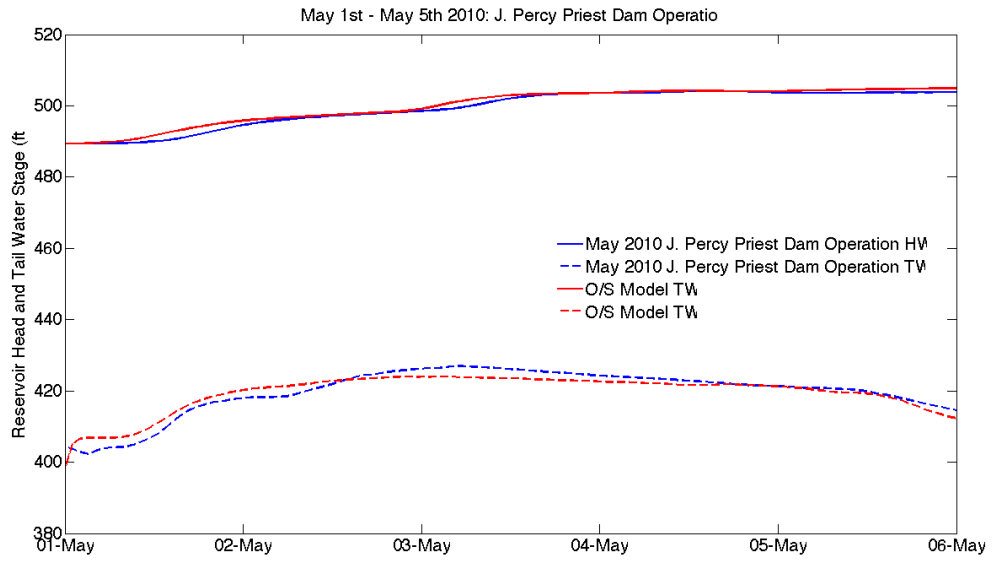
Cordell Hull Reservoir





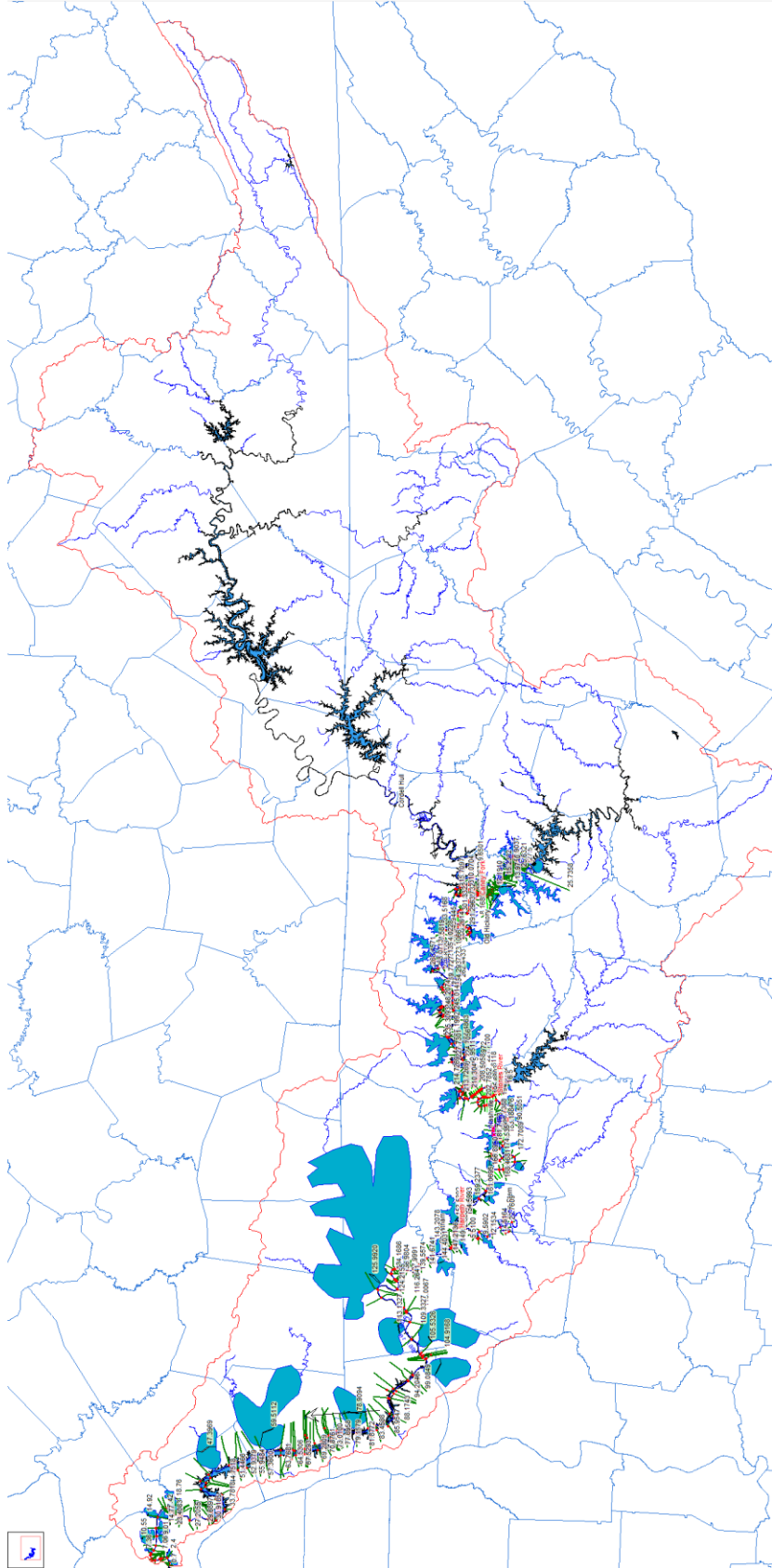
J. Percy Priest Reservoir





APPENDIX N

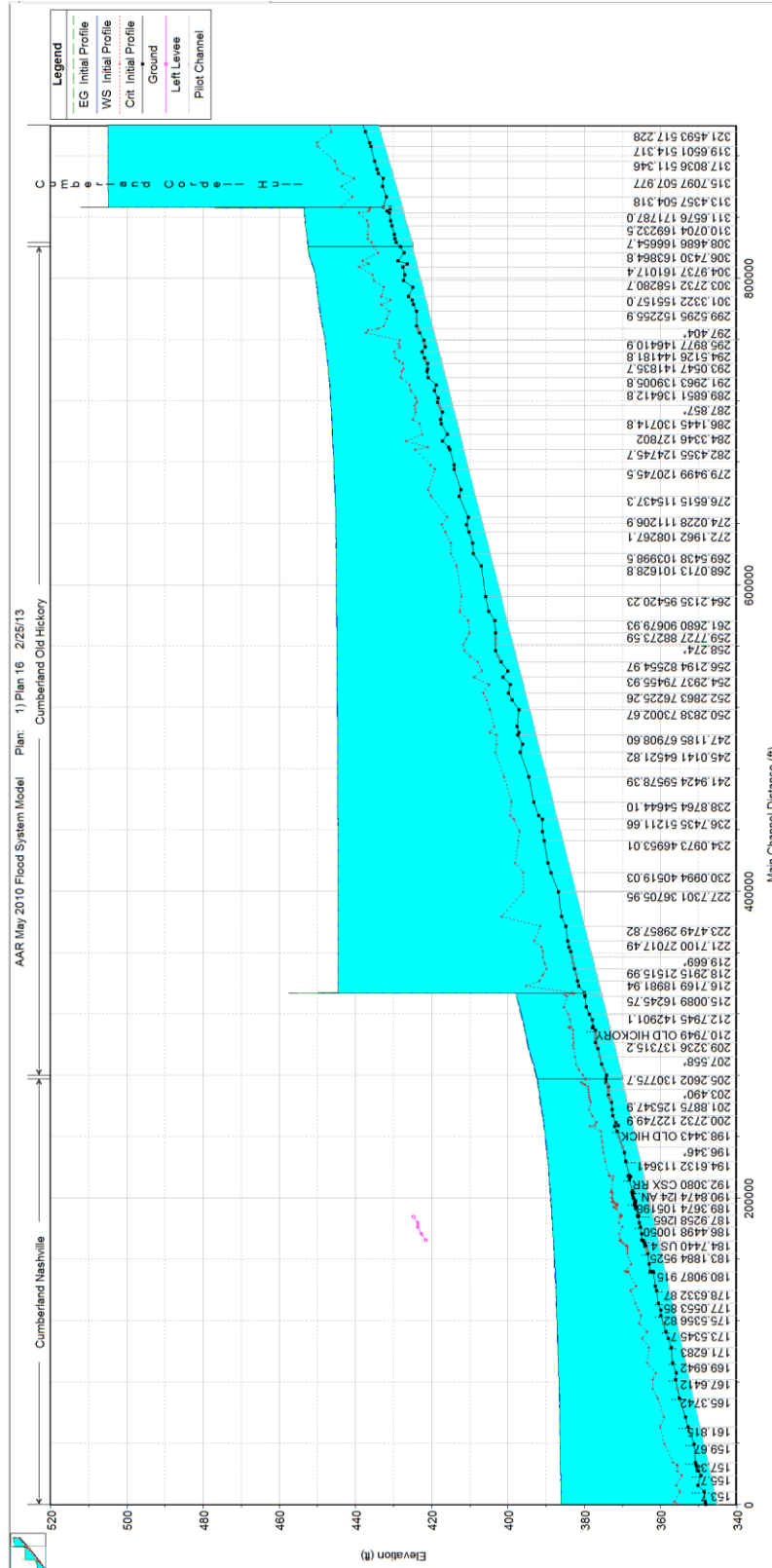
HEC-RAS CUMBERLAND RIVER BASIN MODEL DOMAIN



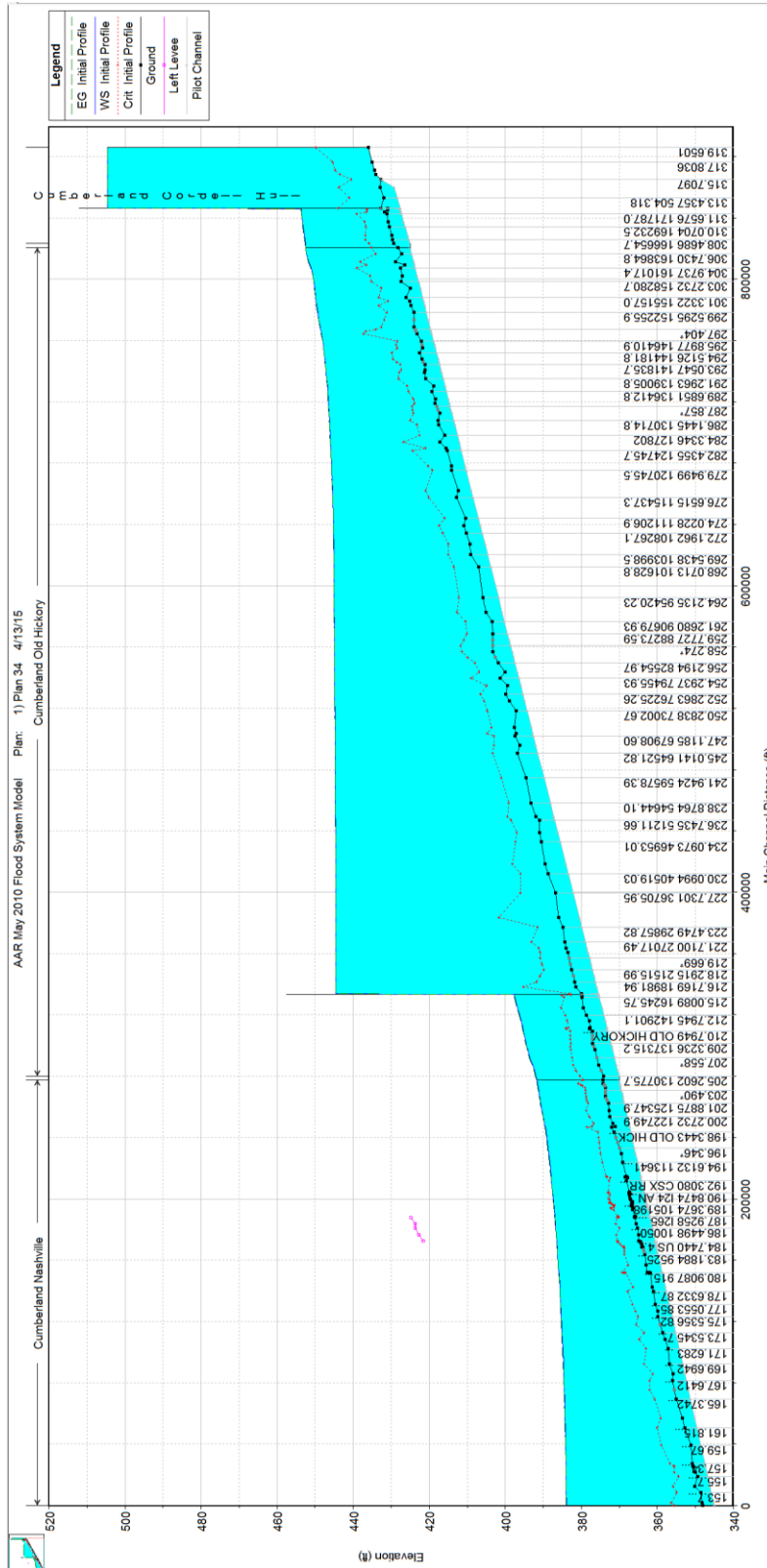
APPENDIX O

HEC-RAS SIMULATION RESULTS (RIVER PROFILE)

May 2010 Flood Event (Maximum Water Surface Elevation)



Optimization/Simulation Model Result (Maximum Water Surface Elevation)



APPENDIX P
COMPUTATIONAL TIME OF MODEL RUNS

Time Index	Simulation Period		Iterations	Local/Global Optimal Solutions	Computation Time [s]
1	1-May-10	7:00	32	6	1669
2	1-May-10	8:00			
3	1-May-10	9:00	20	4	1426
4	1-May-10	10:00			
5	1-May-10	11:00	30	6	1693
6	1-May-10	12:00			
7	1-May-10	13:00	26	5	1416
8	1-May-10	14:00			
9	1-May-10	15:00	29	6	1543
10	1-May-10	16:00			
11	1-May-10	17:00	17	3	1793
12	1-May-10	18:00			
13	1-May-10	19:00	25	5	1429
14	1-May-10	20:00			
15	1-May-10	21:00	15	3	1543
16	1-May-10	22:00			
17	1-May-10	23:00	28	6	1436
18	2-May-10	0:00			
19	2-May-10	1:00	31	6	1638
20	2-May-10	2:00			
21	2-May-10	3:00	15	3	1779
22	2-May-10	4:00			
23	2-May-10	5:00	12	2	1675
24	2-May-10	6:00			
25	2-May-10	7:00	33	7	1544
26	2-May-10	8:00			
27	2-May-10	9:00	17	3	1524
28	2-May-10	10:00			
29	2-May-10	11:00	21	4	1766
30	2-May-10	12:00			
31	2-May-10	13:00	24	5	1620
32	2-May-10	14:00			
33	2-May-10	15:00	23	5	1654
34	2-May-10	16:00			
35	2-May-10	17:00	10	2	1525
36	2-May-10	18:00			
37	2-May-10	19:00	24	5	1770
38	2-May-10	20:00			

39	2-May-10	21:00	14	3	1500
40	2-May-10	22:00			
41	2-May-10	23:00	27	5	1494
42	3-May-10	0:00			
43	3-May-10	1:00	6	1	1528
44	3-May-10	2:00			
45	3-May-10	3:00	17	3	1712
46	3-May-10	4:00			
47	3-May-10	5:00	24	5	1615
48	3-May-10	6:00			
49	3-May-10	7:00	29	6	1446
50	3-May-10	8:00			
51	3-May-10	9:00	19	4	1737
52	3-May-10	10:00			
53	3-May-10	11:00	17	3	1584
54	3-May-10	12:00			
55	3-May-10	13:00	2	1	1680
56	3-May-10	14:00			
57	3-May-10	15:00	20	4	1534
58	3-May-10	16:00			
59	3-May-10	17:00	25	5	1366
60	3-May-10	18:00			
61	3-May-10	19:00	3	1	1638
62	3-May-10	20:00			
63	3-May-10	21:00	15	3	1618
64	3-May-10	22:00			
65	3-May-10	23:00	22	4	1781
66	4-May-10	0:00			
67	4-May-10	1:00	4	1	1653
68	4-May-10	2:00			
69	4-May-10	3:00	33	7	1609
70	4-May-10	4:00			
71	4-May-10	5:00	19	4	1445
72	4-May-10	6:00			
73	4-May-10	7:00	16	3	1512
74	4-May-10	8:00			
75	4-May-10	9:00	32	6	1785
76	4-May-10	10:00			
77	4-May-10	11:00	28	6	1446
78	4-May-10	12:00			
79	4-May-10	13:00	29	6	1671

80	4-May-10	14:00			
81	4-May-10	15:00	5	1	1473
82	4-May-10	16:00			
83	4-May-10	17:00	15	3	1747
84	4-May-10	18:00			
85	4-May-10	19:00	27	5	1354
86	4-May-10	20:00			
87	4-May-10	21:00	30	6	1602
88	4-May-10	22:00			
89	4-May-10	23:00	24	5	1742
90	5-May-10	0:00			
91	5-May-10	1:00	16	3	1624
92	5-May-10	2:00			
93	5-May-10	3:00	31	6	1384
94	5-May-10	4:00			
95	5-May-10	5:00	26	5	1479
96	5-May-10	6:00			
97	5-May-10	7:00	22	4	1496
98	5-May-10	8:00			
99	5-May-10	9:00	8	2	1701
100	5-May-10	10:00			
101	5-May-10	11:00	4	1	1534
102	5-May-10	12:00			
103	5-May-10	13:00	12	2	1472
104	5-May-10	14:00			
105	5-May-10	15:00	27	5	1587
106	5-May-10	16:00			
107	5-May-10	17:00	5	1	1587
108	5-May-10	18:00			
109	5-May-10	19:00	8	2	1553
110	5-May-10	20:00			
111	5-May-10	21:00	3	2	1495
112	5-May-10	22:00			
113	5-May-10	23:00	7	1	1645
114	6-May-10	0:00			

APPENDIX Q
MATLAB CODE

Data Exchange

```
%% Import data from HEC-DSS.
% Script for importing data from the following text file:
%
% Daniel Che (Arizona State University - April 2013)
%
% C:\Users\Daniel Che\Desktop\Cumberland River
% Basin\AAR_Model\Baseline_Models\System_Model\May2010_System.u18
%
% To extend the code to different selected data or a different text
file,
% generate a function instead of a script.

%% Initialize variables.
filename = 'C:\Users\Daniel Che\Desktop\Danny PhD Research
Shit\Cumberland River
Basin\AAR_Model\Baseline_Models\System_Model\May2010_System.u18';
delimiter = ' ';

%% Read columns of data as strings:
% For more information, see the TEXTSCAN documentation.
formatSpec = '%s%s%s%s%s%s%s%s%s%s%[\n\r]';

%% Open the text file.
fileID = fopen(filename,'r');

%% Read columns of data according to format string.
% This call is based on the structure of the file used to generate this
% code. If an error occurs for a different file, try regenerating the
code
% from the Import Tool.
dataArray = textscan(fileID, formatSpec, 'Delimiter', delimiter,
'MultipleDelimsAsOne', true, 'ReturnOnError', false);

%% Close the text file.
fclose(fileID);

%% Convert the contents of columns containing numeric strings to
numbers.
% Replace non-numeric strings with NaN.
raw = [dataArray{:,1:end-1}];
numericData = NaN(size(dataArray{1},1), size(dataArray,2));

for col=[1,2,3,4,5,6,7,8,9,10]
    % Converts strings in the input cell array to numbers. Replaced
non-numeric
    % strings with NaN.
    rawData = dataArray{col};
    for row=1:size(rawData, 1);
        % Create a regular expression to detect and remove non-numeric
prefixes and
        % suffixes.
```

```

        regexstr = '(?<prefix>.*?)(?<numbers>([-
]*(\d+[\,]*)+[\.]{0,1}\d*[eEdD]{0,1}[-+]*\d*[i]{0,1})|([-
]*(\d+[\,]*)*[\.]{1,1}\d+[eEdD]{0,1}[-+]*\d*[i]{0,1})) (?<suffix>.*)';
    try
        result = regexp(rawData{row}, regexstr, 'names');
        numbers = result.numbers;

        % Detected commas in non-thousand locations.
        invalidThousandsSeparator = false;
        if any(numbers==' ');
            thousandsRegExp = '^(\d+?(\,|\d{3})*\.{0,1}\d*$)';
            if isempty(regexp(thousandsRegExp, ' ', 'once'));
                numbers = NaN;
                invalidThousandsSeparator = true;
            end
        end
        % Convert numeric strings to numbers.
        if ~invalidThousandsSeparator;
            numbers = textscan(strrep(numbers, ' ', ''), '%f');
            numericData(row, col) = numbers{1};
            raw{row, col} = numbers{1};
        end
    catch me
    end
end
end

```

```

%% Replace non-numeric cells with NaN
R = cellfun(@(x) ~isnumeric(x) && ~islogical(x), raw); % Find non-
numeric cells
raw(R) = {NaN}; % Replace non-numeric cells

```

```

%% Create output variable
May2010System = dataset;
May2010System.Flow = cell2mat(raw(:, 1));
May2010System.TitleMAYS = cell2mat(raw(:, 2));
May2010System.Old = cell2mat(raw(:, 3));
May2010System.Hickory = cell2mat(raw(:, 4));
May2010System.Gate = cell2mat(raw(:, 5));
May2010System.Ops = cell2mat(raw(:, 6));
May2010System.A4 = cell2mat(raw(:, 7));
May2010System.VarName8 = cell2mat(raw(:, 8));
May2010System.VarName9 = cell2mat(raw(:, 9));
May2010System.VarName10 = cell2mat(raw(:, 10));
%% Clear temporary variables
clearvars filename delimiter formatSpec fileID dataArray ans raw
numericData col rawData row regexstr result numbers
invalidThousandsSeparator thousandsRegExp me R;

```


Forecasting Model

```
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
% Rainfall Forecasting Comparison
% Written by Daniel Che (Feb 2015)
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

clear
clear mem
%%
tic
%%      User Defined Inputs
% Forecasting Period in Hours
T=input('What is the forecasting period in hours? ');
% Choosing a Forecasting Model
disp('1 --> Auto Regressive')
disp('2 --> Auto Regressive Exogenous')
disp('3 --> Auto Regressive Moving Average')
disp('4 --> State-Space Model')
Method=input('Which Method would you like? from 1 to 4: ');
%% Input Data from Excel
data=xlsread('RainInput');
P=data(:,2);
%%
% Rainfall Forecasting Process
for i=7:T:length(P)
    if Method == 1
        sys1=ar(P(1:i),1)
    elseif Method == 2
        sys1=arx(P(1:i),1);
    elseif Method == 3
        sys1=armax(P(1:i),[1 1]);
    else
        sys1=ssest(P(1:i),1);
    end
    f(i:i+T-1)=forecast(sys1,P(1:i),T);
end
%%
f_1=f';
% Plot Forecasted Result
fontsize = 12;
bar(P,'w')
hold on
x=1:1:length(f);
plot(x(1:end),f(1:end),'b-o','LineWidth',2)
xlim([0 72])
ylim([0 1.2])
xlabel('Time [hrs]','fontsize',fontsize)
ylabel('Rainfall [inches]','fontsize',fontsize)
title('Hypothetical Rainfall','fontsize',fontsize)
set(gca,'YTick',[0 0.2 0.4 0.6 0.8 1.0 1.2]);
set(gca,'XTick',[0 6 12 18 24 30 36 42 48 54 60 66 72]);
if Method == 1
    legend('Hypothetical','Forecasted (AR)')
elseif Method == 2
```

```

        legend('Hypothetical','Forecasted (ARX)')
elseif Method == 3
    legend('Hypothetical','Forecasted (ARMAX)')
else
    legend('Hypothetical','Forecasted (SSEST)')
end
legend('boxoff')
set(legend,...
    'Location','NorthEast',...
    'Color',[1 1 1]);
toc
%%
RMSE=sqrt(sum((f(7:end))-P(7:end)).^2)/length(P(7:end))
Cumulative_Forecast_Error=(sum(P(7:end))-sum(f(7:end)))/sum(P(7:end))
%%

%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
% Rainfall Forecasting For the Cumberland River Basin
% Written by Daniel Che (July 2014)
% HEC-HMS 69 Sub-Basins
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

clear all
clear mem
clc
tic
% Forecasting time series using a OLS prediction model
data=xlsread('Random','Actual');
% Create Initial Storage Matrix for Forecasted Rainfall
Forecasted=zeros(size(data));
% Forecasting Period
T = 2;      % delta t
% Rainfall Forecasting Process
for j=1:length(data(1,:))
    for i=2:T:length(data(:,1))
        sys1=ar(data(1:i,j),1)
        Forecasted(i:i+T-1,j)=forecast(sys1,data(1:i,j),T)
    end
end
end
toc



---



function [YF, varargout] = forecast(model, data, K, varargin)
%FORECAST Forecasts linear system response into future.

arginchk(3,6)

[model, data, K, Unext, Options, DoubleData] = ...
    localValidateInputs(model, data, K, varargin{:});

no = nargout;
if isequal(Options,[])

```

```

    Init = 'e';
else
    Init = Options.InitialCondition;
end
[YF, varargout{1:no-1}] = forecast_(model, data, K, Unext, Init,
Options);
if ~DoubleData
    Tstart = pvget(data, 'Tstart'); N = size(data,1);
    Ts = pvget(data, 'Ts'); %nu = size(model,2);
    if ~isequal(numel(Unext),numel(YF))
        Unext = repmat(Unext,[1,numel(N)]);
    end
    nu = size(model,2);
    for kexp = 1:numel(N)
        Tstart{kexp} = Tstart{kexp} + N(kexp)*Ts{kexp};
        if nu>0 && isempty(Unext{kexp})
            Unext{kexp} = zeros(size(YF{kexp},1),nu);
        end
    end

    Warn =
ctrlMsgUtils.SuspendWarnings('Ident:iddata:MoreOutputsThanSamples');
%#ok<NASGU>
    YF = iddata(YF, Unext, Ts, 'InterSample',data.InterSample,...
        'TimeUnit',data.TimeUnit,'InputName',data.InputName,...
        'OutputName',data.OutputName,'InputUnit',data.InputUnit,...
        'OutputUnit',data.OutputUnit,'Tstart',Tstart,...
        'ExperimentName',pvget(data,'ExperimentName'));
elseif isscalar(YF)
    YF = YF{1};
end

if no>1 && isscalar(varargout{1})
    varargout{1} = varargout{1}{1};
    if no>2, varargout{2} = varargout{2}{1}; end
end

%-----
----
function [model, data, K, Unext, Options, DoubleData] = ...
    localValidateInputs(model, data, K, varargin)
% Validate input arguments.

Options = [];
if nargin>3 && isa(varargin{end},'ltioptions.Generic')
    Options = varargin{end};
    if ~isa(Options,'idoptions.x0est')
        try
            Options = cast(forecastOptions,Options);
        catch %#ok<CTCH>
            ctrlMsgUtils.error('Ident:general:optionFormat','forecast',...
                'forecastOptions','forecastOptions')
        end
    end
end
varargin = varargin(1:end-1);

```

```

end

[ny, nu] = size(model);
Ts = abs(model.Ts);
DoubleData = isnumeric(data);
if DoubleData
    if size(data,2)~=ny+nu
        ctrlMsgUtils.error('Ident:general:modelDataDimMismatch')
    else
        Tsdats = getDefaultTDDDataTs(model);
        data = iddata(data(:,1:ny),data(:,ny+1:end),Tsdats,...
            'InterSample',getDefaultISB(model),'TimeUnit',model.TimeUnit);
    end
end
data = idpack.utValidateData('forecast', data, 'time', false);
Tsdats = pvget(data,'Ts');

[~,Nydat,Nudat,Nexp] = size(data);
if Nydat==0
    ctrlMsgUtils.error('Ident:general:noOutputChannel')
elseif ~isequal([Nydat, Nudat],[ny, nu])
    ctrlMsgUtils.error('Ident:general:modelDataDimMismatch')
end

% Check name and unit compatibility and reconcile time units
if ~DoubleData
    [model, data] = idpack.utAlignNamesUnits(model,data);
    Ts = abs(model.Ts); % Ts might have changed
end
% RE: scaleTime does not change sample time of a model with Ts = -1.
if model.Ts<0, model.Ts = Tsdats{1}; Ts = Tsdats{1}; end

if Ts~=0 && abs(Tsdats{1}-Ts)>10*eps
    % Allow mismatch only when model is CT
    ctrlMsgUtils.error('Ident:general:dataModelTsMismatch2','forecast')
end

if ~idpack.isPosIntScalar(K)
    ctrlMsgUtils.error('Ident:analysis:forecastChk1')
end

% Estimate initial conditions by default. The initial conditions are
then
% computed to minimize the 1-step prediction error to the known
(observed)
% data.
Unext = {};
ni = length(varargin)+3;
if ni>3
    Unext = varargin{1};
    if isa(Unext,'iddata')
        Unext = pvget(Unext,'InputData');
    elseif isnumeric(Unext)
        Unext = {Unext};
    end
end

```

```

elseif ~iscell(Unext)
    ctrlMsgUtils.error('Ident:analysis:forecastChk2')
end

if ~any(numel(Unext)==[1 Nexp])
    ctrlMsgUtils.error('Ident:analysis:forecastChk4',Nexp)
end

for kexp = 1:numel(Unext)
    sz2 = size(Unext{kexp});
    if ~isnumeric(Unext{kexp}) || ~any(sz2(1)==[K,0])
        ctrlMsgUtils.error('Ident:analysis:forecastChk2')
    elseif ~any(sz2(2)==[Nudat,0])
        ctrlMsgUtils.error('Ident:analysis:forecastChk3',Nudat)
    else
        Unext{kexp} = double(full(Unext{kexp}));
    end
end

if isscalar(Unext) && Nexp>1
    Unext = repmat(Unext,[1 Nexp]);
end

if ni>4 && ~isequal(varargin{2},[])
    ctrlMsgUtils.error('Ident:general:optionFormat','forecast',...
        'forecastOptions','forecastOptions')
end
elseif Nexp>1
    Unext = repmat(Unext,[1 Nexp]);
end

if ~isempty(Options)
    Options = checkConsistency(Options, model, Nexp, true, 'forecast');
end

```

```

function [th,ref] = ar(data,n,varargin)

```

```

ni = nargin;
narginchk(2,Inf)

PVStart = 0; varg = {};
pt = true; % estimate covariance flag
Ts = []; I = [];
if ni>2
    I = find(cellfun(@(x) isa(x,'idoptions.ar'),varargin));
end

if ~isempty(I)
    options = varargin{I(end)};
    varargin(I) = [];
    ni = length(varargin)+2;

```

```

else
    options = arOptions;
end

if ni<3 || isempty(varargin{1})
    % no-op
elseif ischar(varargin{1})
    % Could be "approach" or start of PV pair
    v1 = varargin{1};
    if ~isempty(v1) && v1(end)=='0'
        pt = false; % obsolete syntax, where ending '0' denoted no
covariance
        v1 = v1(1:end-1);
    end
    Value = ltipack.matchKey(v1,{'fb','ls','yw','burg','gl'});
    if isempty(Value)
        % Assume PV start
        PVStart = 1;
    else
        options.Approach = Value;
    end
end

if ni>3
    v2 = varargin{2};
    Window = [];
    if PVStart==0
        if isempty(v2), v2 = 'now'; end
        if ~ischar(v2)
            ctrlMsgUtils.error('Ident:general:InvalidSyntax','ar','ar')
        end
        if v2(end)=='0'
            pt = false; % obsolete syntax, where ending '0' denoted no
covariance
            v2 = v2(1:end-1);
        end
        Window = ltipack.matchKey(v2,{'now','prw','pow','ppw'});
    end

    if isempty(Window)
        if PVStart==0
            % Assume PV start
            PVStart = 2;
        end
    else
        options.Window = Window;
    end

    % Trap obsolete syntax: Model = AR(Y,N,Approach,Win,Maxsize,T)
    if PVStart==0 && ni>4 && ni<=6 && isnumeric(varargin{3}) && ...
        (ni<6 || (isnumeric(varargin{4}) && isscalar(varargin{4})))
        options.MaxSize = varargin{3};
        if ni==6
            Ts = varargin{4};
        end
    end
end

```

```

else
    if PVStart>0
        varg = varargin(PVStart:end);
    else
        varg = varargin(3:end);
    end
    % Find Ts
    TsInd = idpack.findOptionInList('Ts',varg,2);
    if ~isempty(TsInd)
        Ts = varg{TsInd(end)+1};
        varg([TsInd, TsInd+1]) = [];
    end
end
elseif PVStart~=0
    ctrlMsgUtils.error('Ident:general:InvalidSyntax','ar','ar')
end

% If PV pairs are supplied, look for 'IntegrateNoise' since its value
can
% affect estimation results.
NI = false;
if ~isempty(varg)
    NIInd = idpack.findOptionInList('IntegrateNoise',varg,3);
    if ~isempty(NIInd)
        if length(varg)>NIInd(end)
            NI = varg{NIInd(end)+1};
            if isscalar(NI)
                if isnumeric(NI) && isequal(NI, logical(NI))
                    NI = logical(NI);
                elseif ~islogical(NI)
                    ctrlMsgUtils.error('Ident:estimation:arNI')
                end
            else
                ctrlMsgUtils.error('Ident:estimation:arNI')
            end
        else
            ctrlMsgUtils.error('Ident:general:InvalidSyntax','ar','ar')
        end
    end
end
end

% Checks on data and order
if isa(data,'frd') || (isa(data,'iddata') &&
strcmp(pvget(data,'Domain'),'Frequency'))
    ctrlMsgUtils.error('Ident:estimation:estUsingFrequencyData','ar')
else
    if isa(data,'double') && isvector(data)
        data = data(:);
    end
    data = idpack.utValidateData('ar', data, 'time', true);
    if isempty(data.Name), data.Name = inputname(1); end
    if ~isempty(Ts), data.Ts = Ts; end
    [~, ny, nu] = size(data);
    if ny>1
        ctrlMsgUtils.error('Ident:estimation:arMultiOutput','ar')
    end
end

```

```

elseif nu>0
    ctrlMsgUtils.error('Ident:estimation:IODataNotAllowed','ar')
end

yor = pvget(data,'OutputData');
Ne = numel(yor); Ncaps = cellfun('length',yor);
end

if ~idpack.isPosIntScalar(n)
    ctrlMsgUtils.error('Ident:estimation:arInvalidOrder')
end

if ~isempty(varg)
    if rem(length(varg),2)~=0

ctrlMsgUtils.error('Ident:estimation:CompleteOptionsValuePairs','ar')
    else
        % look for maxsize
        maxsizeInd = idpack.findOptionInList('MaxSize',varg,1);
        if ~isempty(maxsizeInd)
            options.MaxSize = varg{maxsizeInd+1};
        end
    end
end

options.EstCovar = pt; pt1 = pt;

% Perform estimation.
ref = [];
maxsize = options.MaxSize;
if ischar(maxsize), maxsize = 250e3; end
approach = options.Approach;
win = options.Window;
yOff = options.DataOffset;

if NI
    for kexp = 1:Ne
        yor{kexp} = diff(yor{kexp});
    end
    Ncaps = Ncaps-1;
elseif ~isempty(yOff) || ~isequal(yOff,0)
    yOff = idpack.checkOffsetSize(yOff,'DataOffset',[1 Ne]);
    for kexp = 1:Ne
        yor{kexp} = yor{kexp} - yOff(kexp);
    end
end

y = yor; % Keep the original y for later computation of e

if strcmp(approach,'yw'), win = 'ppw'; end
if strcmp(win,'prw') || strcmp(win,'ppw')
    for kexp = 1:Ne
        y{kexp} = [zeros(n,1);y{kexp}];
    end
end

```



```

    Ncaps = Ncaps+n;
end

if strcmp(win,'pow') || strcmp(win,'ppw')
    for kexp = 1:Ne
        y{kexp} = [y{kexp};zeros(n,1)];
    end
    Ncaps = Ncaps+n;
end

% First the lattice based algorithms
if any(strcmp(approach,{'burg','gl'}))
    ef = y; eb = y;
    rho = zeros(1,n+1);
    r = zeros(1,n);
    A = r;
    [ss,1] = sumcell(y,1,Ncaps);
    rho(1) = ss/1;
    for p = 1:n
        nef = sumcell(ef,p+1,Ncaps);
        neb = sumcell(eb,p,Ncaps-1);
        if strcmp(approach,'gl')
            den = sqrt(nef*neb);
        else
            den = (nef+neb)/2;
        end
        ss = 0;
        for kexp = 1:Ne
            ss = ss+(-eb{kexp}(p:Ncaps(kexp)-
1)*ef{kexp}(p+1:Ncaps(kexp)));
        end

        r(p) = ss/den;
        A(p) = r(p);
        A(1:p-1) = A(1:p-1)+r(p)*conj(A(p-1:-1:1));
        rho(p+1) = rho(p)*(1-r(p)*r(p));
        efold = ef;
        for kexp = 1:Ne
            Ncap = Ncaps(kexp);
            ef{kexp}(2:Ncap) = ef{kexp}(2:Ncap)+r(p)*eb{kexp}(1:Ncap-1);
            eb{kexp}(2:Ncap) = eb{kexp}(1:Ncap-
1)+conj(r(p))*efold{kexp}(2:Ncap);
        end
    end
    Apoly = [1 A]; %th = pvset(th,'a',[1 A]);
    ref = [0 r ; rho];
else
    pt1 = true; % override pt for the other approaches
end

covR = [];

% Now compute the regression matrix
if pt1
    nmax = n;

```

```

M = floor(maxsize/n);
R1 = zeros(0,n+1);
fb = strcmp(approach,'fb');
if strcmp(approach,'fb')
    R2 = zeros(0,n+1);
    yb = cell(1,Ne);
    for kexp = 1:Ne
        yb{kexp} = conj(y{kexp}(Ncaps(kexp):-1:1));
    end
end
for kexp = 1:Ne
    Ncap = Ncaps(kexp);
    yy = y{kexp};
    for k = nmax:M:Ncap-1
        jj = (k+1:min(Ncap,k+M));
        phi = zeros(length(jj),n);
        if fb
            phib = zeros(length(jj),n);
        end
        for k1 = 1:n
            phi(:,k1) = -yy(jj-k1);
        end
        if fb
            for k2 = 1:n
                phib(:,k2) = -yb{kexp}(jj-k2);
            end
        end
        if fb
            R2 = triu(qr([R2; [[phi;phib],[yy(jj);yb{kexp}(jj)]]]));
            [nRr,nRc] = size(R2);
            R2 = R2(1:min(nRr,nRc),:);
        end
        R1 = triu(qr([R1; [phi,yy(jj)]]));
        [nRr,nRc] = size(R1);
        R1 = R1(1:min(nRr,nRc),:);
    end
end
end

covR = R1(1:n,1:n);
P = pinv(covR);
if ~any(strcmp(approach,{'burg','gl'}))
    if ~fb
        A = (P * R1(1:n,n+1)).';
    else
        A = (pinv(R2(1:n,1:n)) * R2(1:n,n+1)).';
    end
    Apoly = [1 A]; % th = pvset(th,'a',[1 A]);
end
%P = P*P';
end

e = [];
for kexp = 1:Ne
    tt = filter([1 A],1,yor{kexp});
    tt(1:n) = zeros(n,1);
end

```

```

    e = [e; tt];
end

lam = e'*e/(length(e)-n);
if pt
    cov = idpack.FactoredCovariance(covR/sqrt(lam), [], true(n,1));
else
    cov = [];
end

S = pmodel.polynomial({Apoly}, [], [], [], [], zeros(1,0));
S.IntegrateNoise = NI;
Tsdats = pvget(data, 'Ts'); Tsdats = Tsdats{1};
PolyData = idpack.polydata(S, Tsdats);
PolyData.Covariance = cov;
PolyData.EstimationOptions = options;
PolyData.NoiseVariance = lam;
PolyData.EstimationStatus = 1;

Info = idresults.GenericParametric;
[Info.Fit.FitPercent, Info.Fit.MSE] =
getFitPercent(PolyData, unpack(data), 1, 'e');
Info.Fit.FPE = lam*(1+2*n/sum(Ncaps));
Info.Fit.LossFcn = lam;
Info.Status = 'Estimated using AR';
Info.Method = sprintf('AR (''%s/%s'')', approach, win);
Info.RandState = rng;
Info = setParameterInfo(Info, PolyData);

ed = Info.DataUsed;
ed.Name = data.Name;
ed.Length = Ncaps;
ed.Ts = cell2mat(pvget(data, 'Ts'));
ed.InterSample = cell(0, Ne);
ed.OutputOffset = yOff;
Info.DataUsed = ed;
PolyData.Report = Info;

th = idpoly.make(PolyData, [1 0]);
if ~isempty(varg)
    th = set(th, varg{:});
    th = setcov(th, PolyData.Covariance);
end

th = copyEstimationDataMetaData(th, data);
%th = timemark(th);

%-----
function [s, ln] = sumcell(y, p, N)

ln = 0;
s = 0;
for kexp = 1:length(y)
    y1 = y{kexp};

```

```

    s = s+y1(p:N(kexp))'*y1(p:N(kexp));
    ln = ln + length(y1);
end

```

```

%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
% Rainfall Forecasting For the Cumberland River Basin
% MATLAB Built-In Model
% Forecasting Model - Autoregressive Moving Average (ARMA)
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

```

```

function sys = armax(varargin)
%ARMAX Estimate ARMAX polynomial model using time domain data.
%
% M = ARMAX(Z, [na nb nc nk])
% estimates an ARMAX model, M, represented by:
%     A(q) y(t) = B(q) u(t-nk) + C(q) e(t)
% where:
%     na = order of A polynomial      (Ny-by-Ny matrix)
%     nb = order of B polynomial + 1  (Ny-by-Nu matrix)
%     nc = order of C polynomial      (Ny-by-1 matrix)
%     nk = input delay (in number of samples, Ny-by-Nu entries)
%     (Nu = number of inputs; Ny = number of outputs)
%
% The estimated model, M, is delivered as an @idpoly object. M
contains
% the estimated values for A, B, and C polynomials along with their
% covariances and structure information.
%
% Z is the time-domain estimation data given as an IDDATA object.
Type
% "help iddata" for more information. You cannot use frequency-
domain
% data for estimation of ARMAX models. na, nb, nc and nk are the
% polynomial orders associated with the ARMAX model.
%
% M = ARMAX(Z, [na nb nc nk], 'Name1', Value1, 'Name2', Value2,...)
% specifies additional model structure properties as name-value
% pairs. You can specify as one or more of the following:
% 'InputDelay': Specify input delay as a double vector of length
equal
%                 to number of inputs. Entries must be nonnegative
%                 integers denoting the delay as multiples of sample
%                 time.
% 'ioDelay': Input-to-output delay (double matrix). Specify as an
% Ny-by-Nu matrix of nonnegative integers denoting the
% delays as multiples of sample time. Useful as a
% replacement for "nk" order - max(nk-1,0) lags can be
% factored out as "ioDelay" value.
% 'IntegrateNoise': Add integrator to noise channel. Logical vector
of
%                 length Ny. Default: false(Ny,1). Setting
IntegrateNoise
%                 to true (for a particular output) results in models
of
%                 structure:

```



```

end

% Validate input arguments and create a template system if required.
try
    [sys, EstimData, Orders] = validatePEMInputs('armax',varargin{:});
catch E
    throw(E)
end

Options = getDefaultOptions(sys);
Disp = ~strcmpi(Options.Display,'off');
if Disp
    W = Options.ProgressWindow;
    Str =
ctrlMsgUtils.message('Ident:estimation:msgDispPolyest1','ARMAX');
    idDisplayEstimationInfo('Intro',{Str, ' '},W);
end

%% Perform estimation.
try
    sys = pem_(sys, EstimData, Orders);
catch E
    if Disp
        S{1} = sprintf('<font color="red">%s</font>',E.message);
        S{2} =
ctrlMsgUtils.message('Ident:estimation:msgAbortEstimation');
        idDisplayEstimationInfo('Error',S,W);
    end
    throw(E)
end
if Disp, W.STOP = true; end

%% Reconcile metadata between model and data.
sys = copyEstimationDataMetaData(sys, EstimData);
Report = sys.Report; Report.Method = 'ARMAX'; sys = setReport(sys,
Report);

```

PLOTTING

```

%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
% Optimization/Simulation Model Results (Nashville Gage)
% Written by Daniel Che (August 2014)
% HEC-RAS Result
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

clear all
clear mem
clc

fontsize = 12;

Data=xlsread('Nashville_Gage');
Q100=ones(1,length(Data(:,1)))*48;

```

```

startDate = datenum(2010,05,01,0,0,0);
endDate = datenum(2010,05,06,0,0,0);
tData = linspace(startDate,endDate,114);

plot(tData, Data(:,1), 'b', 'Linewidth',2)
hold on
plot(tData,Q100, 'r--', 'Linewidth',2)
hold on
plot(tData,Data(:,2), 'k-', 'Linewidth',1.5)
hold off
datetick('x','dd-mmm')
title('May 1st - May 6th 2010: Nashville','fontsize',fontsize)
ylabel('Flood Stage (ft)','fontsize',fontsize)
legend('May 2010 Flood Event','100-Year Flood','O/S Model')
set(legend,'YColor',[1 1 1],'XColor',[1 1 1],...
      'Location','NorthEast',...
      'Color',[1 1 1]);

plot(tData,Data(:,10), 'r-', 'Linewidth',1.5)
hold on
plot(tData, Data(:,9), 'b', 'Linewidth',2)
hold off
ylim([0 200000])
datetick('x','dd-mmm')
title('May 1st - May 6th 2010: Nashville','fontsize',fontsize)
ylabel('Flow (cfs)','fontsize',fontsize)
legend('May 2010 Flood Event','O/S Model')
set(legend,'YColor',[1 1 1],'XColor',[1 1 1],...
      'Location','NorthEast',...
      'Color',[1 1 1]);
set(gca,'YTickLabel',num2str(get(gca,'yTick')))
set(gca,'YTick',[0 100000 150000 200000 160000 180000 200000 200000]);

figure(2)
plot(tData, Data(:,13)*(3600/45360), 'b', 'Linewidth',2)
hold on
plot(tData,Data(:,12)*(3600/45360), 'r', 'Linewidth',1.5)
hold off
datetick('x','dd-mmm')
title('May 1st - May 6th 2010: Nashville','fontsize',fontsize)
ylabel('Cumulative Flow Volume (acre-ft)','fontsize',fontsize)
legend('May 2010 Flood Event','O/S Model')
set(legend,'YColor',[1 1 1],'XColor',[1 1 1],...
      'Location','NorthEast',...
      'Color',[1 1 1]);
set(gca,'YTickLabel',num2str(get(gca,'yTick')))
set(gca,'YTick',[0 100000 150000 200000 160000 180000 200000 200000]);

```

```

%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
% Optimization/Simulation Model Results (Old Hickory)
% Written by Daniel Che (August 2014)
% HEC-RAS Result
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

clear all
clear mem
clc

fontsize = 12;

Data=xlsread('Old Hickory.xlsx');
Data2=xlsread('Old Hickory.xlsx','Sheet2');
Data3=xlsread('Old Hickory.xlsx','Sheet3');
Data4=xlsread('Old Hickory.xlsx','Sheet4');

startDate = datenum(2010,05,01,0,0,0);
endDate = datenum(2010,05,06,0,0,0);
tData = linspace(startDate,endDate,114);
tData3 = linspace(startDate,endDate,864);

figure(1)
plot(tData, Data(:,2),'b','Linewidth',1.5)
hold on
plot(tData,Data(:,1),'r-','Linewidth',1)
hold off
datetick('x','dd-mmm')
title('May 1st - May 6th 2010: Old Hickory Dam
Operation','fontsize',fontsize)
ylabel('Reservoir Outflow (cfs)','fontsize',fontsize)
legend('May 2010 Old Hickory Dam Operation','O/S Model')
set(legend,'YColor',[1 1 1],'XColor',[1 1 1],...
'Location','NorthEast',...
'Color',[1 1 1]);
set(gca,'YTickLabel',num2str(get(gca,'yTick')))
set(gca,'YTick',[0 50000 100000 150000 200000 250000]);

figure(2)
plot(tData, Data(:,5)*(3600/45360),'b','Linewidth',2)
hold on
plot(tData,Data(:,4)*(3600/45360),'r-','Linewidth',2)
hold off
datetick('x','dd-mmm')
title('May 1st - May 6th 2010: Old Hickory Dam
Operation','fontsize',fontsize)
ylabel('Reservoir Cumulative Outflow (acre-ft)','fontsize',fontsize)
legend('May 2010 Old Hickory Dam Operation','O/S Model')
set(legend,'YColor',[1 1 1],'XColor',[1 1 1],...
'Location','NorthEast',...
'Color',[1 1 1]);
set(gca,'YTickLabel',num2str(get(gca,'yTick')))
set(gca,'YTick',[0 400000 800000 1200000]);

```



```

ylim([0 1200000])

figure(3)
plot(tData3, Data3(:,1), 'b', 'Linewidth',1.5)
hold on
plot(tData,Data2(:,1), 'r-', 'Linewidth',1)
hold off
datetick('x','dd-mmm')
title('May 1st - May 6th 2010: Old Hickory Dam
Operation','fontsize',fontsize)
ylabel('Reservoir Gate Openings (ft)','fontsize',fontsize)
legend('May 2010 Old Hickory Dam Operation','O/S Model')
set(legend,'YColor',[1 1 1],'XColor',[1 1 1],...
'Location','NorthEast',...
'Color',[1 1 1]);
ylim([0 40])
%set(gca,'YTickLabel',num2str(get(gca,'YTick'))')
%set(gca,'YTick',[0 50000 100000 150000 200000 250000]);

figure(4)
plot(tData, Data4(:,2), 'b', 'Linewidth',1.5)
hold on
plot(tData,Data4(:,1), 'r-', 'Linewidth',1)
hold off
datetick('x','dd-mmm')
title('May 1st - May 6th 2010: Old Hickory Dam
Operation','fontsize',fontsize)
ylabel('Reservoir Tail Water Stage (ft)','fontsize',fontsize)
legend('May 2010 Old Hickory Dam Operation','O/S Model')
set(legend,'YColor',[1 1 1],'XColor',[1 1 1],...
'Location','NorthEast',...
'Color',[1 1 1]);
%ylim([0 40])

%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
% Cumberland River Basin
% Rainfall and Runoff Plots
% Written by Daniel Che (May 2015)
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%%%
clc
clear all
clear mem

% Load Data from Excel File
data1=xlsread('CumberlandRainFall1.xlsx','Actual Rainfall');
data2=xlsread('CumberlandRainFall1.xlsx','Forecasted Rainfall');
data3=xlsread('CumberlandRainFall1.xlsx','Actual Runoff');
data4=xlsread('CumberlandRainFall1.xlsx','Forecasted Runoff');

x=1:1:length(data1(:,1));

```

```

fontsize = 20;

for i=1:x(end)

figure(i)
plot(x(1:end),data3(:,i),'k--','LineWidth',2)
hold on
plot(x(1:end),data4(:,i),'b-o','LineWidth',1)
hold all
%xlim([0 72])
ylim([0 150000])
xlabel('Time [hrs since 00:00 May 1st 2010 ]','fontsize',fontsize)
ylabel('Flow [ft^3/s]','fontsize',fontsize)
title('Rainfall-Runoff (Basin 2)','fontsize',fontsize)
%set(gca,'XTick',[0 6 12 18 24 30 36 42 48 54 60 66 72]);
set(gca,'YTickLabel',num2str(get(gca,'YTick').'))
%set(gca,'YTick',[0 30000 60000 90000 120000 150000]);
legend('Basin Runoff (Actual)','Basin Runoff (Forecasted)')
legend('boxoff')
set(legend,'Location','East','Color',[1 1 1]);

ax1 = gca;
set(ax1,'XColor','k','YColor','k');
sec_ax1 =
axes('Position',get(ax1,'Position'),'XAxisLocation','top','YAxisLocation',
'right','Color','none','XColor','k','YColor','k');
hold on;
bar(data1(:,i),'w','Parent',sec_ax1)
line(x,data2(:,i),'Color',[.35 .35
.35],'LineWidth',3,'Parent',sec_ax1);
legend('Rainfall (Actual)','Rainfall (Forecasted)')
legend('boxoff')
set(legend,'Location','west','Color',[1 1 1]);
hold all;
grid off;
ylabel('Rainfall
[inches]','FontSize',20,'FontName','Arial','VerticalAlignment','cap','R
otation',270);
ylim([0 4]);
%xlim([0 72]);
set(gca,'YDir','reverse','FontSize',20,'FontName','Arial','Xticklabel',
' ');
%legend1 = legend(sec_ax1,'show');
%set(legend1,'FontSize',fsize,'FontName','Arial','Color',[1 1
1],'YColor',[1 1 1],'XColor',[1 1 1],'Position',[0.75 0.7433 0.06625
0.06632]);
set(gca,'YTick',[0 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0]);

end

```