Real-Time Operation of River-Reservoir Systems During Flood Conditions Using Optimization-Simulation Model with One- and Two-Dimensional Modeling

by

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ABSTRACT

Flooding is a critical issue around the world, and the absence of comprehension of watershed hydrologic reaction results in lack of lead-time for flood forecasting and expensive harm to property and life. It happens when water flows due to extreme rainfall storm, dam breach or snowmelt exceeds the capacity of river system reservoirs and channels. The objective of this research was to develop a methodology for determining a time series operation for releases through control gates of river-reservoir systems during flooding events in a real-time using one- and/or two-dimensional modeling of flows through river-reservoir systems.

The optimization-simulation methodology interfaces several simulation-software coupled together with an optimization model solved by genetic algorithm coded in MATLAB. These software include the U.S. Army Corps of Engineers HEC-RAS linked the genetic algorithm in MATLAB to come up with an optimization-simulation model for time series gate openings to control downstream elevations. The model involves using the one- and two-dimensional ability in HEC-RAS to perform hydrodynamic routing with high-resolution raster Digital Elevation Models. Also, the model uses both real-time gridded- and gaged-rainfall data in addition to a model for forecasting future rainfall-data.

This new model has been developed to manage reservoir release schedules before, during, and after an extraordinary rainfall event that could cause extreme flooding. Further to observe and control downstream water surface elevations to avoid exceedance

of threshold of flood levels in target cells in the downstream area of study, and to minimize the damage and direct effects in both the up and downstream.

The application of the complete optimization-simulation model was applied to a portion of the Cumberland River System in Nashville, Tennessee for the flooding event of May 2010. The objective of this application is to demonstrate the applicability of the model for minimizing flood damages for an actual flood event in real-time on an actual river basin. The purpose of the application in a real-time framework would be to minimize the flood damages at Nashville, Tennessee by keeping the flood stages under the 100-year flood stage. This application also compared the three unsteady flow simulation scenarios: one-dimensional, two-dimensional and combined one- and two-dimensional unsteady flow.

DEDICATION

I DEDICATE THIS WORK TO THE MOST PASSIONATE AND LOVING TWO PERSONS TO SEE ME FINISH IT, MY PARENTS.

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I would like to express my deepest appreciation to the committee chair Professor Larry W. Mays, who has the substance of a genius. He continually and convincingly conveyed a spirit of adventure regarding research and scholarship and an excitement in regard to teaching. He has provided me extensive personal and professional guidance and taught me a great deal about both scientific research and life in general. He taught us more than we could ever give him credit for here and shown, by his example, what a good engineer (and person) should be. Without his smart guidance and persistent help, this dissertation would not have been possible.

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CHAPTER 1 INTRODUCTION

1.1 Background

Flooding, according to the Cambridge Dictionary, is defined as the condition of a region or piece of land being filled or covered with a significant amount of water, especially from the rain. This can happen in many different ways. Most commonly, flooding occurs when a river stream overflows its banks due to excessive rain. Throughout history, several ways to mitigate the impact flooding have been invented, but the most effective one was the construction of a dam and then a reservoir at a specific location on the river basin or watershed. However, this river-reservoir system needs to be carefully managed; otherwise, the problem could be worse than it was before the reservoir system was built, and could result in multiple losses.

When flooding happens, the downstream areaa of a river or reservoir systems are inundated by the overflow and rise in water level. It most commonly happens when water flows due to extreme rainfall, a dam breach or snow melting exceeding the capacity of river system channels, lakes, or the way into which it runs. Usually, the combined flow of several tributaries causes flooding along river banks or floodplains. Flooding is a critical issue around the world and the absence of comprehension of watershed hydrologic reaction and demonstrating constraints result in a lack of lead-time for flood forecasting and expensive property damage as well as life. Global warming and climate change problems have promoted extreme weather events in the recent past and underscored the need for accurate predictions of flood levels before it is too late to happen.

Despite the extensive studies that many previous researchers have done to analyze floods and flow management, flooding problems still occur, causing tremendous devastation of life and property in both the short and long terms. As of late, there have been new approaches and systems created in GIS techniques that are considered to have more productive storage, processing of information, and joint examination of various datasets. The extensive use of data has shown that it can be reliable in producing rainfall intensities and patterns. That enables and enhances the application of real-time flood forecasting and merges the use of real-time and forecast rainfall and streamflow data into hydrologic and hydraulic simulation models to predict flow rates and stages of the river-reservoir system for a period of time, ranging from hours to days in advance.

1.2 Real-Time Flood Forecasting

One of the most effective measures for flood management is real-time flood forecasting. As a focused activity in the hydro-meteorological sector, flood forecasting is a relatively recent development that might indicate a growing seriousness of flood impacts. Formerly, in many different European countries, response focused on flood forecasting and warning through the global meteorological forecasting of severe weather. The estimations of flood forecasts regarding quantity and time have grown with the acknowledgment of the consequence of flood warnings as a contribution to flood management. That implies that the conventional methods of basic extrapolation of forecasts from gauged sites are no longer sufficient (Moore, Cole, Bell, & Jones, 2006). However, the occurrence of numerous severe events resulted in the founding of many national flood forecasting and warning centers. These warning centers contributed to enhancing the development of monitoring networks particularly for flood forecasting and warning purposes. Hydrological systems consist of instruments that have electronic facilities for data storage and transmission (rain gauges and water surface elevations), and meteorological effort has focused on collection and delivery of satellite and radar data. Due to the nature of Hydrological and meteorological phenomena, flood forecasting and warning is exposed to uncertainty, as it is somehow based on the principles of probability (World Meteorological Organization, 2011).

The process of any hydrologically related forecast is an estimation of the future state of a hydrological event. Like the flow rate, the water volume and level of an area that would be affected or inundated by water and average velocity of flow in a particular region or location of a stream. The lead time for such a forecast can be defined as the interval of time starting from making the forecast to the future point in time for which the forecast is applied; Figure 1.1 depicts the procedures of developing flood forecasting model. Determining a lead-time requires many constraints to be considered; the main one of them is the size of the catchment within a particular region or even country. However, for instance, a short-term forecast of lead time between 2 to 48 hours is considered in the United States of America, while between 2 to 10 days is classified as medium-term forecast, and a long-term forecast would be exceeded by ten days (World Meteorological Organization, 2011).

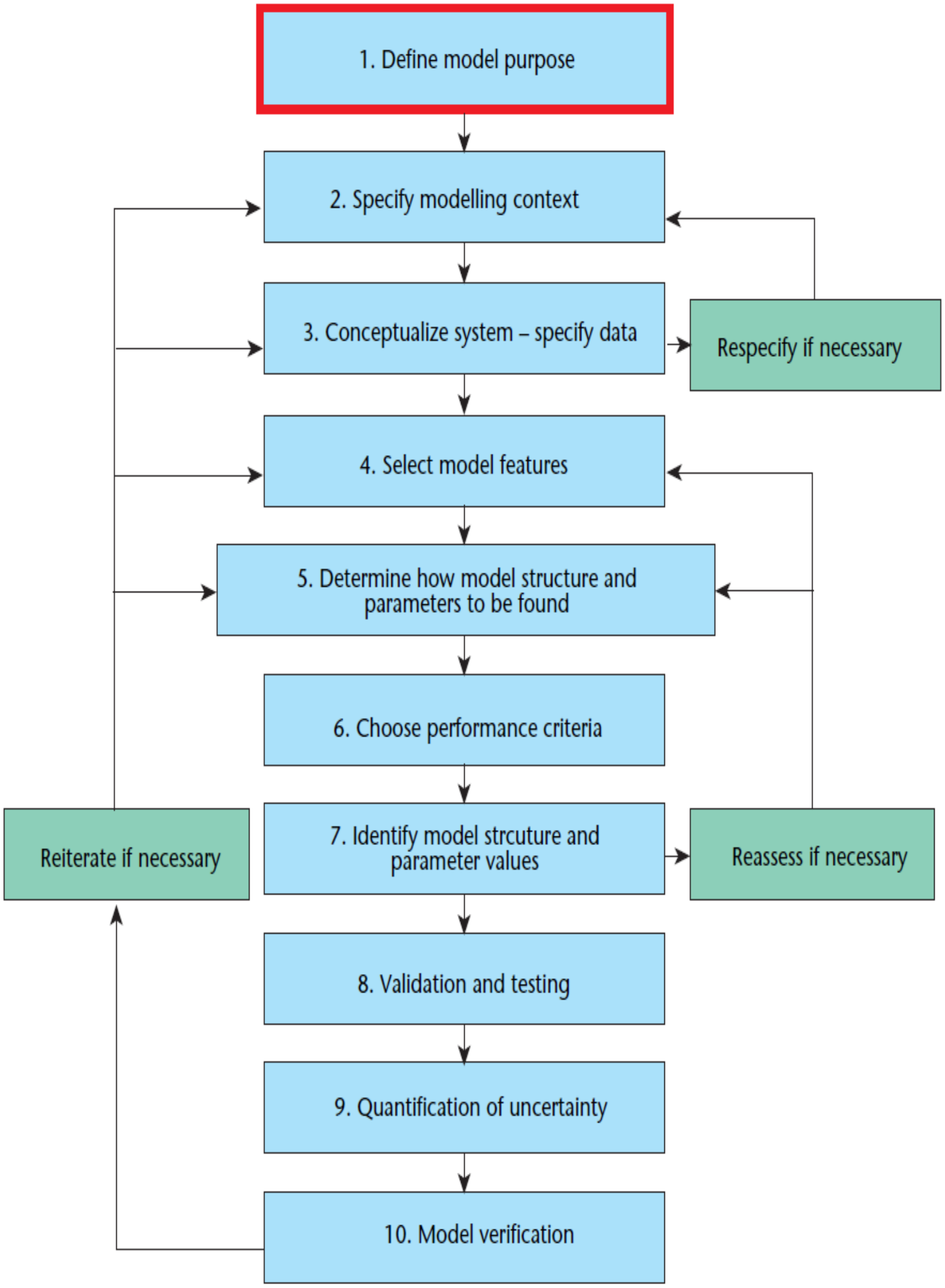


Figure 1.1 Process for Developing a Flood Forecasting Model, (World Meteorological Organization, 2011)

To formulate a real-time flood forecasting problem for a river-reservoir system, Figure 1.2, meteorological and flow data are observed and then transmitted to the determining station through a different method of information correspondences. The meteorological and flow data obtained in real-time are then used in the flood forecasting model to estimate the movement of the flood and the corresponding water levels. The lead-time, as mentioned above, ranges from a few hours to days depending on the catchment size and aim of the forecast. The modeling framework should not have excessive input requirements, but at the same time, the forecasted flood should be as accurate as possible.

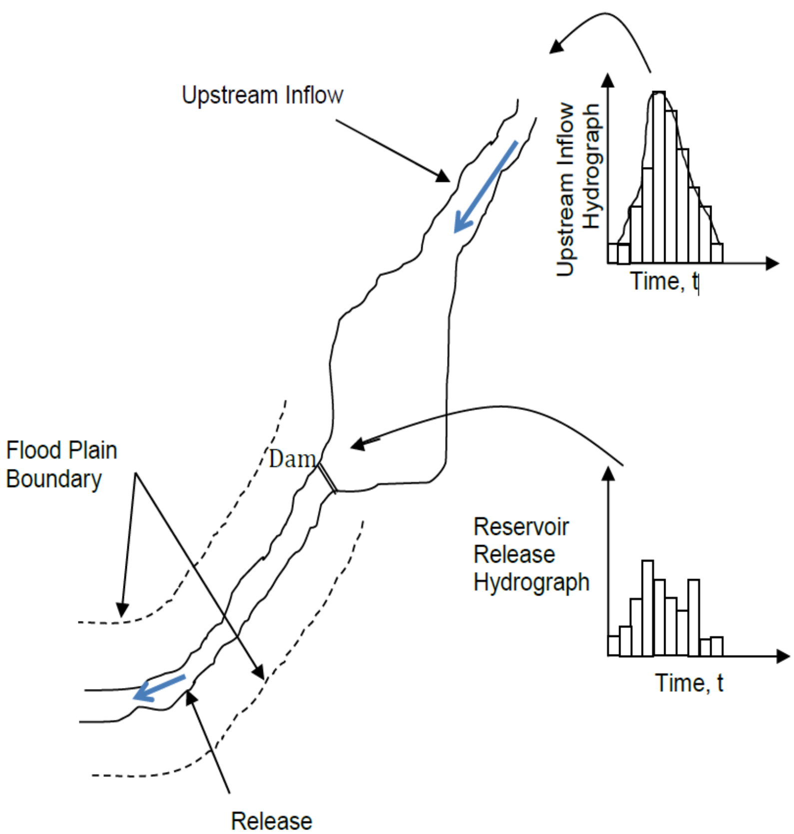


Figure 1.2 Schematic of River-Reservoir System, (Ahmed, 2006)

In many cases, modeling which considers one-dimensional flows is difficult to model floodplain flows accurately, as there are numerous directions of water flow on a flat plain. Therefore, the hydraulics of the flood plain needs to be precisely predicted. Usually, modeling flow in a network of channels can be performed using one-dimensional modeling. It can somehow reach a solution but does not take the change of direction of flow at a point of interest into consideration, while the modification in a direction is considered to be a part of the solution in two-dimensional models (Beffa & Connell, 2001).

Diffusion of flow in a flood plain includes many issues to be considered, especially in complex topography. During flooding conditions, allocating water stream flow at a particular time can exceed the flood level and then propagate horizontally onto the flood plain in different directions, so it is going to be difficult to model in one direction. Starting with a dry flood plain is another important feature of two-dimensional solutions. Based on the topography of the flooding area, water spreads out in the flood plain in different directions at the beginning of modeling. Two-dimensional flow models were first developed and applied to estuary flows in 1967 by Leendertse, see (Beffa & Connell, 2001). They succeeded in implementing the finite difference method to solve the problem of subcritical flow regime. Later, in the eighties and nineties, that type of modeling became widely applied to the simulation of tidal flows and lowland flows. However, later on, the use of these schemes receded because of their inability to predict the critical and supercritical flow regimes accurately. In other words, they could not model flow in the steep slope channels.

The one-dimensional modeling could only determine one resulting water surface elevation at a cross-section. Therefore, the fluctuations across the section will not occur in the model as they would in the case of a real event. However, the one-dimensional analysis can predict good results for river reaches. This research will use both one, and two-dimensional unsteady flow routing for river reaches. The river segments are modeled using one-dimensional flow equations, while floodplains will be modeled using the two-dimensional analysis.

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 the momentum equations, (V. Te Chow, Maidment, & Mays, 1988):

$$\frac{\partial Q}{\partial x} + \frac{\partial s_{co}(A + A_0)}{\partial t} - q = 0 \tag{1.1}$$

Momentum Equation

$$\frac{\partial (s_m Q)}{\partial x} + \frac{\partial (\beta Q^2/A)}{\partial x} + gA\left[\frac{\partial h}{\partial x} + S_f + S_e + s_i\right] - L + W_f B = 0 \tag{1.2}$$

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 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.

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

Two-Dimensional Conservation of Mass:

$$\frac{\partial h}{\partial t} + \frac{\partial hu}{\partial x} + \frac{\partial hv}{\partial y} = 0 \tag{1.3}$$

X-Direction Momentum

$$\frac{\partial u}{\partial t}+u\frac{\partial u}{\partial x}+v\frac{\partial u}{\partial y}+g\frac{\partial(h+z)}{\partial x}+\frac{gn^2u\sqrt{u^2+v^2}}{h^{4/3}} - \frac{v}{h}\left(2\frac{\partial^2 hu}{\partial x^2}+\frac{\partial^2 hu}{\partial x^2}+\frac{\partial^2 hv}{\partial x\partial y}\right)=0 \tag{1.4}$$

Y-Direction Momentum

$$\frac{\partial v}{\partial t}+u\frac{\partial v}{\partial x}+v\frac{\partial v}{\partial y}+g\frac{\partial(h+z)}{\partial y}+\frac{gn^2v\sqrt{u^2+v^2}}{h^{4/3}} - \frac{v}{h}\left(\frac{\partial^2 hv}{\partial x^2}+2\frac{\partial^2 hv}{\partial y^2}+\frac{\partial^2 hu}{\partial x\partial y}\right)=0 \tag{1.5}$$

Where

u and v are the components of the horizontal velocity in the X and Y direction.

h is the flow depth.

z is the bed elevation.

x and y are horizontal distances in the x and y directions respectively.

t is the time.

g is the acceleration due to gravity.

n is Manning's coefficient of roughness.

There are various types of one and two-dimensional unsteady flow models, most of which are commercial models and used in practice are presented in details in Chapter 5. The two-dimensional unsteady flow equations solver (HEC-RAS) uses the Implicit

Finite Volume Algorithm. The algorithm of implicit solution allows for longer computational time steps than explicit methods. The Finite Volume Method gives an increment of improved stability and robustness over traditional finite difference and finite element techniques. The wetting and drying of 2D cells are very robust; two-dimensional flow areas can start completely dry and handle a sudden rush of water into the area. Additionally, the algorithm can handle subcritical, supercritical, and mixed flow regimes (flow passing through critical depth, such as a hydraulic jump), (G. Brunner, 2016).

Going back to lead time, which is considered as the most criterion in flood forecasting and as (L. Mays & Tung, 1992) define it as the interval of time between the issuing the of a forecast and expected arrival of the forecasted event. In flood forecasting, both the location and time are necessary to include. For instance, 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 1.3 (L. Mays & Tung, 1992). There are three urban areas: A, B, and C; with a major rainfall in the upper region of the watershed. 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 1.4, urban area A needs for rainfall forecasts, where urban area C requires one.

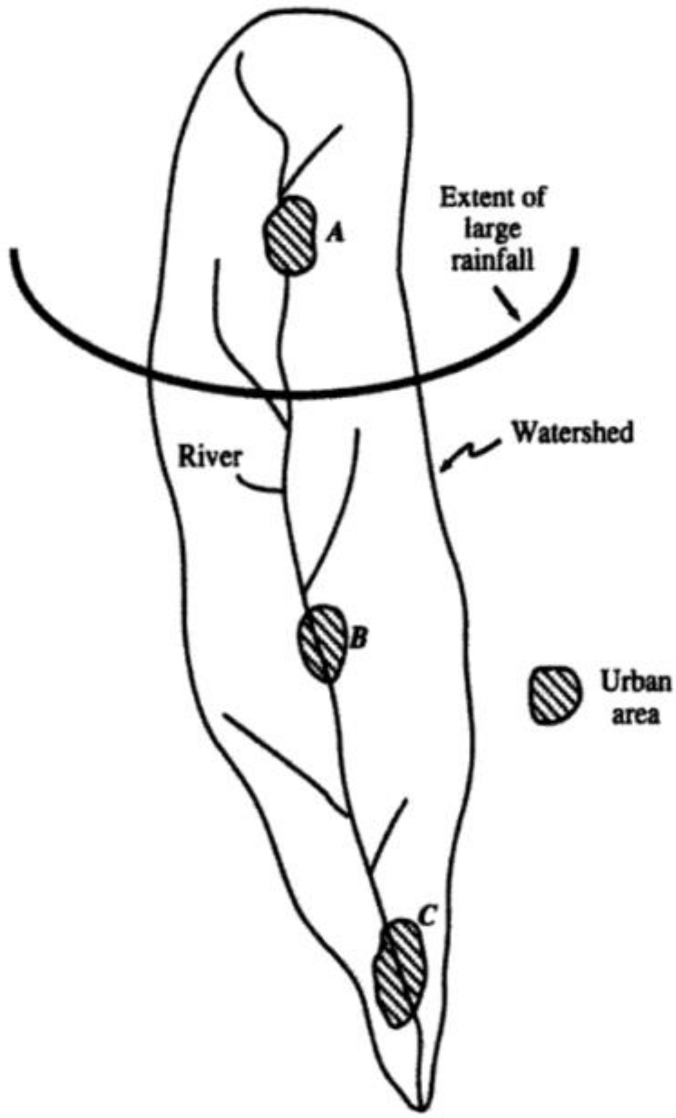


Figure 1.3 Effect of Lead Time (L. Mays & Tung, 1992)

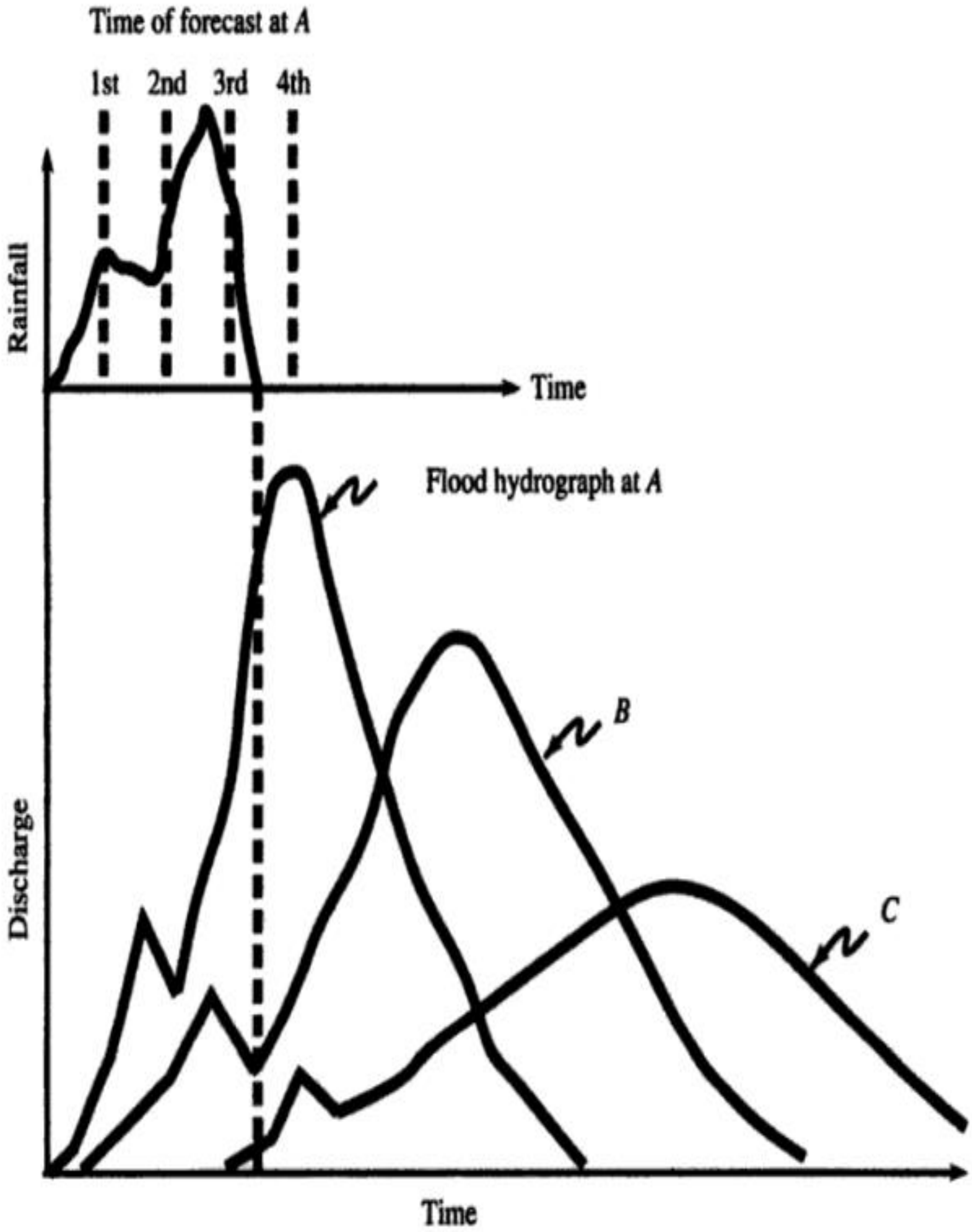


Figure 1.4 Flood hydrograph at Downstream Location in a Watershed (L. Mays & Tung, 1992)

1.3 Research Objective

The objective of the research is to define a strategy for determining a time series operation for releases through control gates of a river-reservoir system during a flooding event in a real-time fashion. That will be done through interfacing several simulation software coupled together with an optimization technique (genetic algorithm) by a MATLAB code. These software are HEC-RAS 5.0.3, and HEC-HMS linked the genetic algorithm in the MATLAB environment to come up with a simulation-optimization model for time series gate openings to control downstream elevation. The model involves using the two-dimensional (2D) ability in HEC-RAS 5.0.3 to perform hydrodynamic routing with high-resolution raster Digital Elevation Models. This new model will be developed to manage reservoir releases schedules before, during, and after an extraordinary rainfall storm event causing flooding, to observe and control downstream water surface elevations to avoid exceedance of threshold of flood levels in target cells in the area of study. The Hydrologic Engineering Center of the US Army Corps of Engineer has added this feature of 2D simulation and analysis recently to the HEC-RAS.

Accurate rainfall data in real-time should be available in order to forecast the upcoming inflow to the reservoir and then to decide the outflow from the flood control gates of the reservoir. The gates opening of the reservoir’s spillway are considered in the problem formulation as the decision variable to determine the optimal control in real-time operations.

Real-time simulation performed in discrete time with constant step (fixed-step simulation) as time moves forward in an equal duration of time to estimate the rainfall over the watershed and then use it as input data for the next phase of the study which is the rainfall-runoff simulation model. Runoff modeled by using the U. S. Army Corps of Engineers Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS). HEC-HMS integrated with GIS technique model as well as the topographic and the parameter of the way of computing the runoff to produce the outflow hydrograph from the watershed. Basic steps of the proposed model are depicted in Figure 1.5.

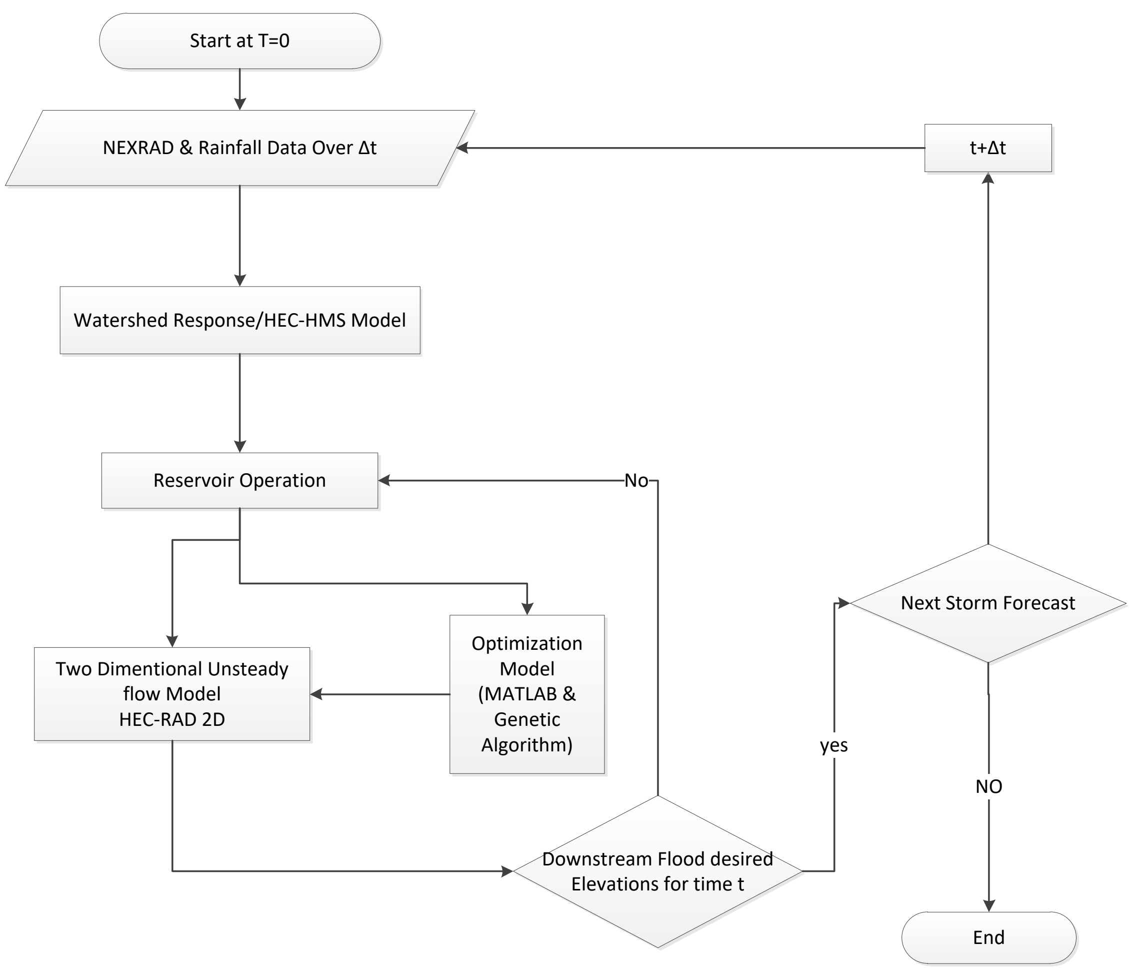


Figure 1.5 General Flow Chart of Simulation-Optimization Model

To start simulating the runoff, accurate real-time data of rainfall (NEXRAD data) should be available to obtain the output hydrographs of the watershed at each time step (Δt). The obtained watershed hydrographs are then used as input data, into the two-dimensional unsteady flow model to be routed downstream to the location of the reservoirs. By getting the watershed hydrographs and routing them downstream to the site of a reservoir throughout the unsteady model (HEC-RAS), adequate information and data of how much water, that will input to the reservoir into a period will be computed. Especially, in the case of the intended reservoir has a specified flood control volume as illustrated in Figure 1.6. Before high quantities of water flow reached the reservoir in real time, a decision should be made through the optimization model, to decide how much water should be released, in advance from the gate openings. The gate openings are the decision variables of the optimization model. The purpose is to ensure that there is enough volume in the reservoir to accommodate the upcoming water quantities without causing flooding downstream.

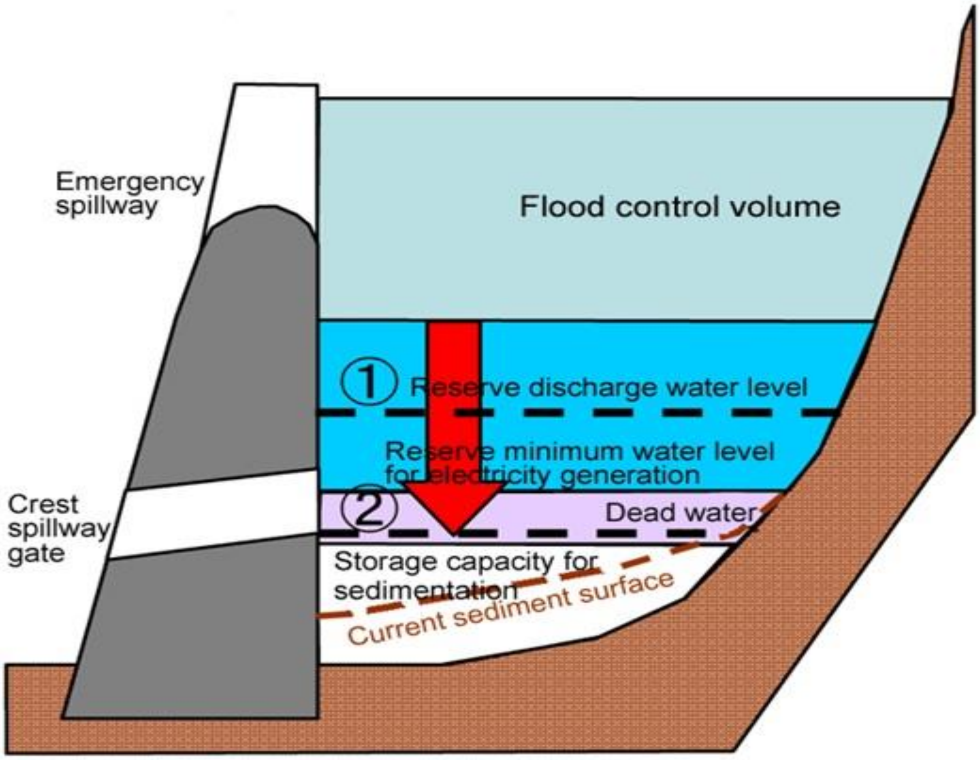


Figure 1.6 Allocation of Reservoir Capacity in Flood Season, (Asia Pacific Adaptation Network (APAN), 2013)

A simple hypothetical model as illustrated in Figure 1.7 and will be applied to the flooding event of the 2010 Cumberland River in Nashville, Tennessee. Four major components are brought together to form the optimization-simulation model for operating a river-reservoir system in a real-time fashion. Figure 1.8 illustrates the connecting among those components. The first element of the model is the simulation of rainfall-runoff from the watershed system by using the HEC-HMS (Hydrologic Modeling System), (U.S. Army Corps of Engineers, 2007) & (U.S. Army Corps of Engineers, 2010a). The second one is the unsteady flow routing from the watershed to the reservoir by using HEC-RAS (River Analysis System) which can be performed in both one and two-dimensional unsteady flow computations studies, (G. Brunner, 2016). The third and fourth

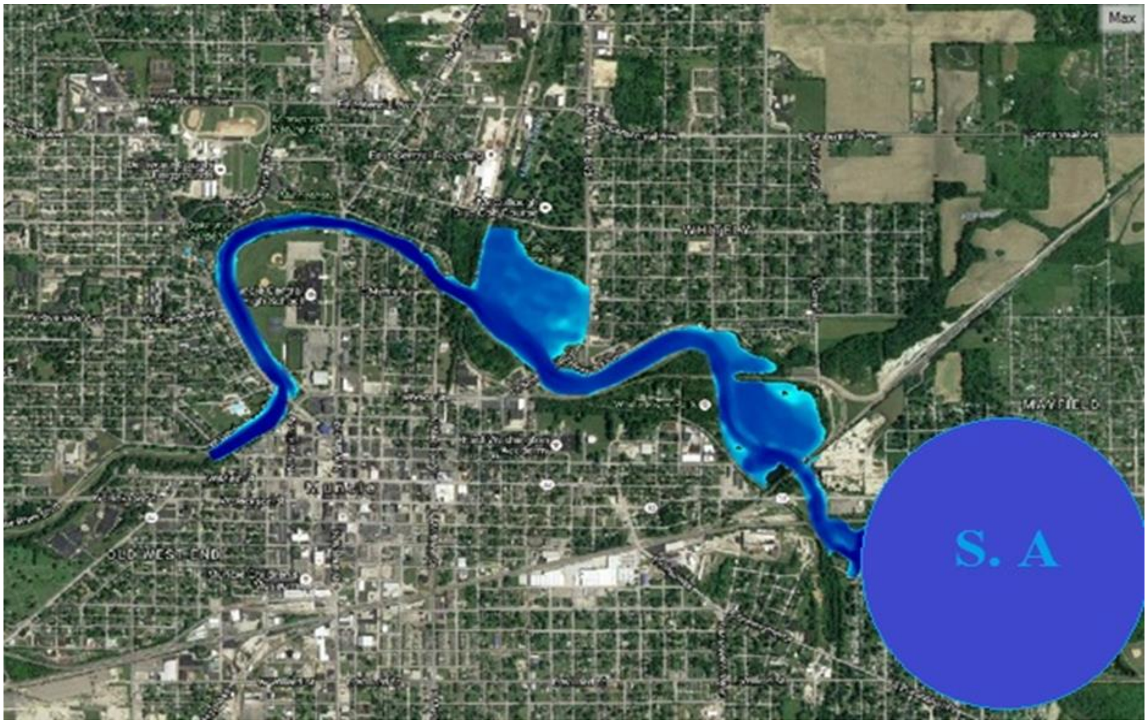


Figure 1.7 Simplified Hypothetical Model

Components which are the most important part of the overall model are the operation of the reservoir gate and the optimization model for determining the optimal gates opening as a function of time; which is the decision variable of the objective function. The rainfall forecasting model produces rainfall in the future based on the actual real-time rainfall up to real-time of the operation are then taken from the National Weather Services gridded rainfall values, and a rainfall gage network until the time of resuming reservoir operation. The model will also be responsible for measuring the real-time flood elevation in the selected cell in a river and the two-dimensional area. After that, an approach of projecting short-term rainfall using the forecasting model should be developed in the next minutes or maybe a few hours after resuming the reservoir operation.

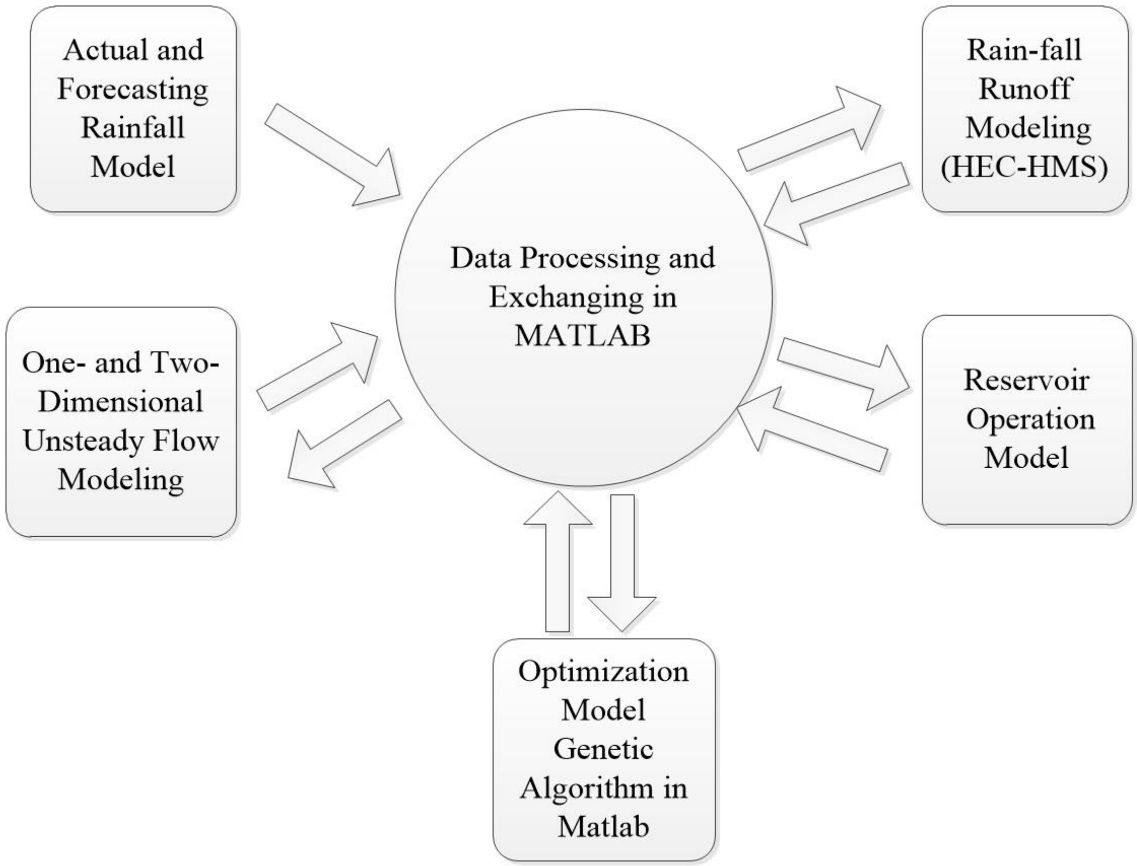


Figure 1.8 Connecting Among Model Components

The optimization approach adopted in this research is to use the genetic algorithm optimizer programmed through MATLAB software to interface the other components of the model to decide the optimal gate opening of the reservoir system in the real-time operation. Figure 1.9 illustrates the interfacing between the model components. Rainfall-runoff simulation is done by using HEC-HMS model, depending on The National Weather Service gridded rainfall data (NEXRAD) to produce the hydrographs which will be utilized as input to the optimization model to determine the optimal gate openings at series of time along with the event of the storm in a river-reservoir system. After the optimal operation of the reservoir's gate opening obtained, the decision variables (gate openings) enter into the two-dimensional unsteady model HEC-RAS, for flood simulations in the river downstream of the reservoir system at target locations.

The genetic algorithm optimizer though MATLAB is used as an optimization method to determine the feasible solution. Different from the conventional optimization methods, like gold section and simplex methods. The genetic algorithm does not necessarily require a well-defined function. Historically, the genetic algorithm firstly developed by Holland in the middle of 1970s as model simulates the Natural Selection of Charles Darwin's Theory of Evolution. In general, the genetic algorithm consists of three operators: selections, crossover, and mutations. The reason behind using the genetic algorithm in this research is of the advantages over traditional optimization methods and its ability to solve very complex optimizations problems and parallelism, (Deb, 2011).

The main objective of the research proposes a simulation/optimization methodology for controlling the flood flows and flood elevations at various locations of a river-reservoir system using one- and or two-dimensional simulation in the river-reservoir system. One example might be to keep the flow rates and flood elevations below the 100-year level. If the objective is not met, the genetic algorithm optimization through MATLAB will reiterate its process to determine the reservoir's gate release until the maximum downstream water surface elevation is reached. The moment that the objective is attained, the model proceeds to the next iteration. At that very moment, the short-term projected rainfall is used to run the precipitation-runoff model to produce the hydrograph that used to operate the reservoirs for the following forecasting period. At this point, the real rainfall data have been processed and ready to be used to calculate the actual runoff from the watershed, the water surface elevation, reservoir gate releases, and the unsteady flows. The model will be reiterated and proceed until the goal is obtained with fulfilling all constraints for the whole simulation time frame. The explanation behind the model begins the simulation days before the storm event is started. it can figure out which activities are fundamental for the reservoir to take to get ready for the floodwaters for the coming days. A detailed description of the real-time reservoir operation model will be presented later in chapter seven of this dissertation.

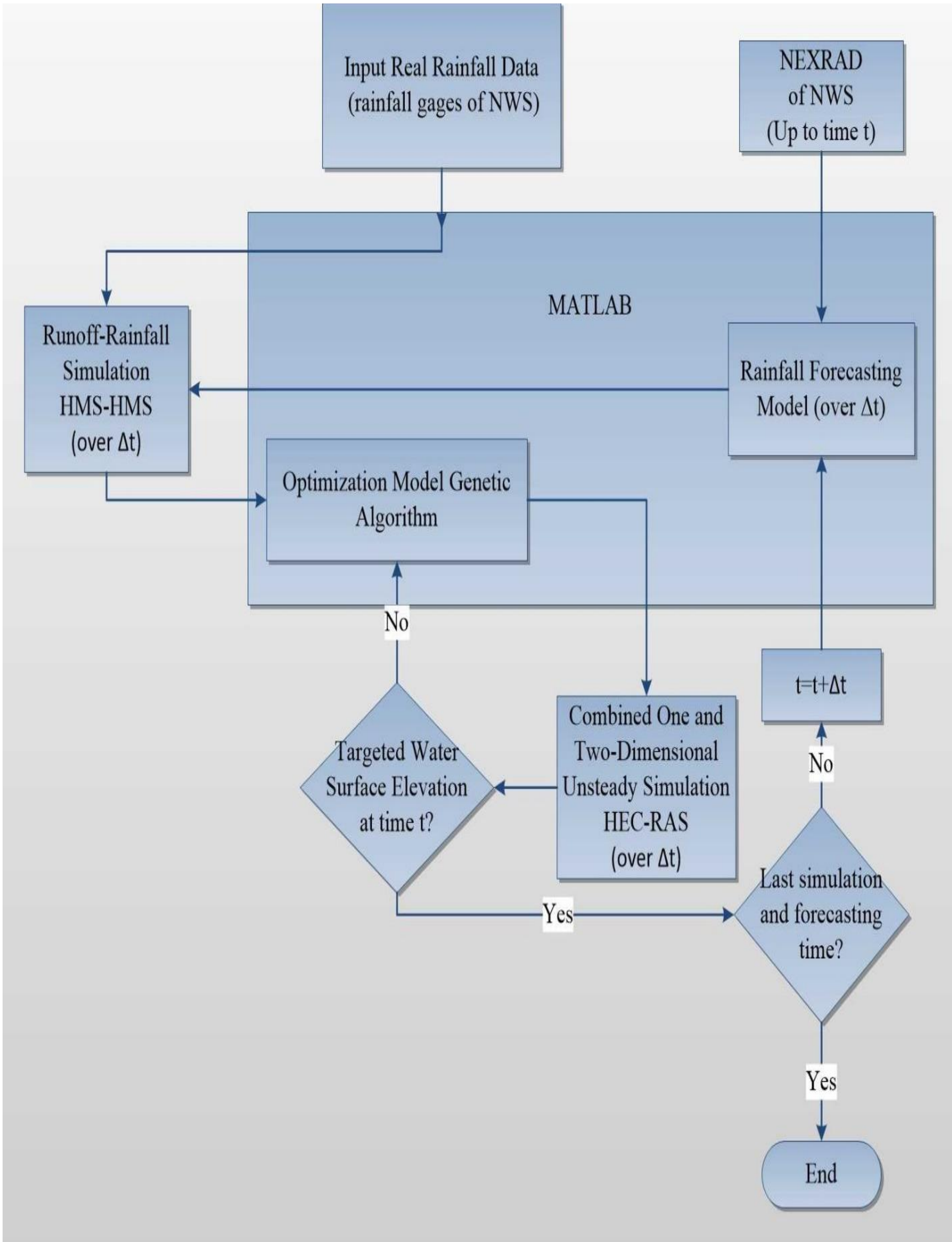


Figure 1.9 Interfacing of Model Components

1.4 Research Phases

The phases of the research are categorized into two types; phases of model development and phases application

1.4.1 Model Development Phases

1. Literature review on conducted works on two-dimensional flow modeling, real-time flood forecasting, GIS technique patterns in flood management, rainfall-runoff models and one and two-dimensional unsteady flow models.
2. Develop a georeferenced terrain model for the study area using the BASINS model for the purpose of the two-dimensional model.
3. Build a model based upon using the HEC-HMS and HEC-RAS, genetic algorithm Solver through MATLAB, so that a data exchange system can be programmed to interface data among these components of the modeling system.
4. Interface 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 through a MATLAB's code needed to perform the interfacing.
5. Search for the space of all feasible solutions in which the genetic algorithm can be used in selecting gate operations of the various reservoirs.
6. Develop a model to forecast short-term future rainfall for hours in advance of a known rainfall considering the lead time.
7. Examine the accuracy of the sub models and overall model.

8. Create a simplified hypothetical model, after the model components had been tested.

The study will focus on the using of two-dimensional feature the HEC-RAS and the understudy the flow downstream of a reservoir, as well as the importance of using real-time and forecasting data for an extreme flood event in the real-time flood control operation of a river-reservoir system.

1.4.2 Phases of Model Application

A demonstration of the model will be performed using the data from the May 2010 flood event on the Cumberland River system. The U.S. Army Corps of Engineers set up the HEC-HMS model input and the one-dimensional HEC-RAS models for the 2010 flood event. So,

1. The optimization/simulation model developed herein will be 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 1.10 and Figure 1.11). These are the three dams that have the most impact on the Cumberland River upstream of Nashville, Tennessee.
2. A detailed study of the Old Hickory dam operations during the 2010 flood event while considering the actual operation done by the U.S. Army Corps of Engineers and the operation rules established years prior.

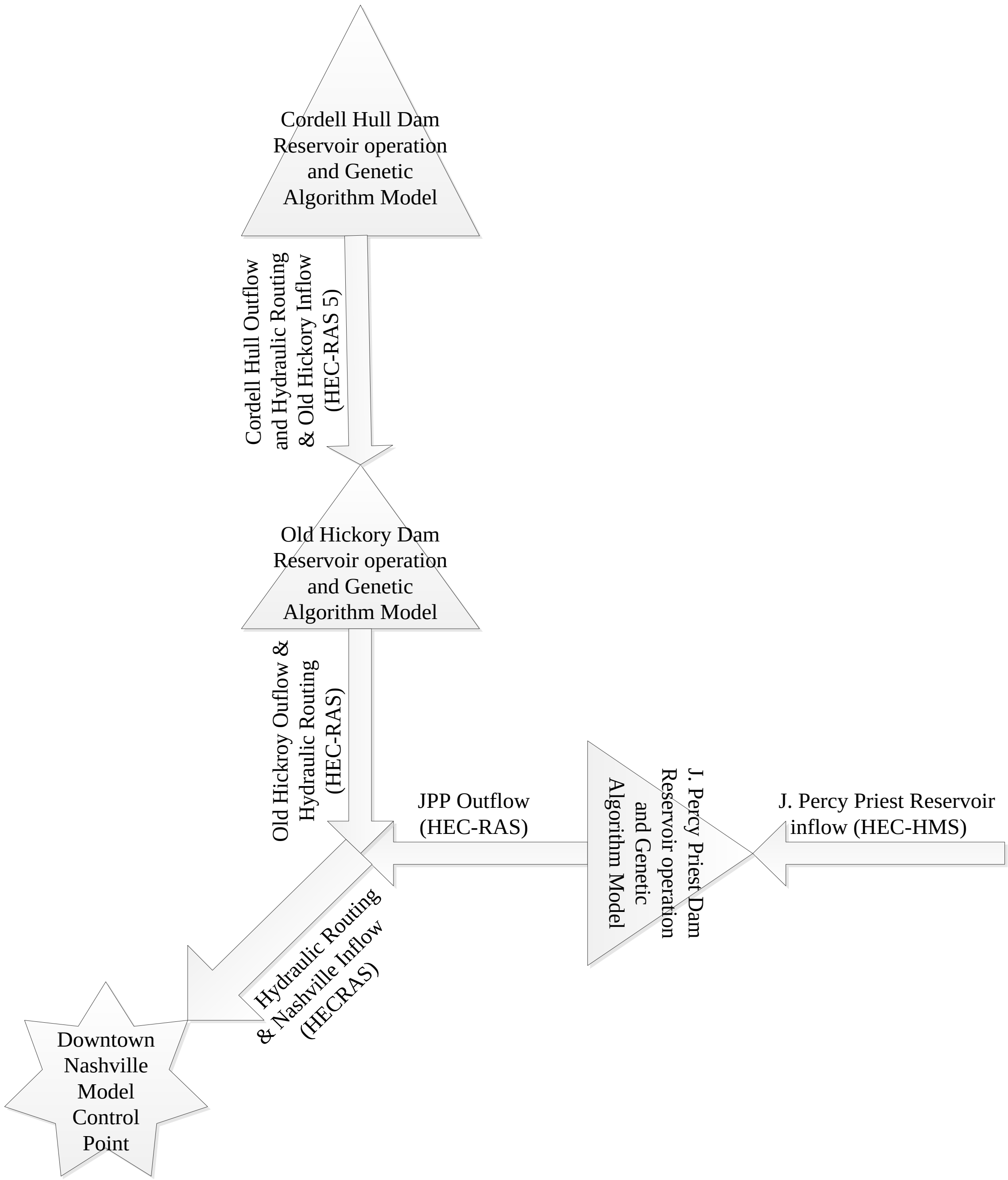


Figure 1.10 Reservoirs on the Cumberland River Near Nashville

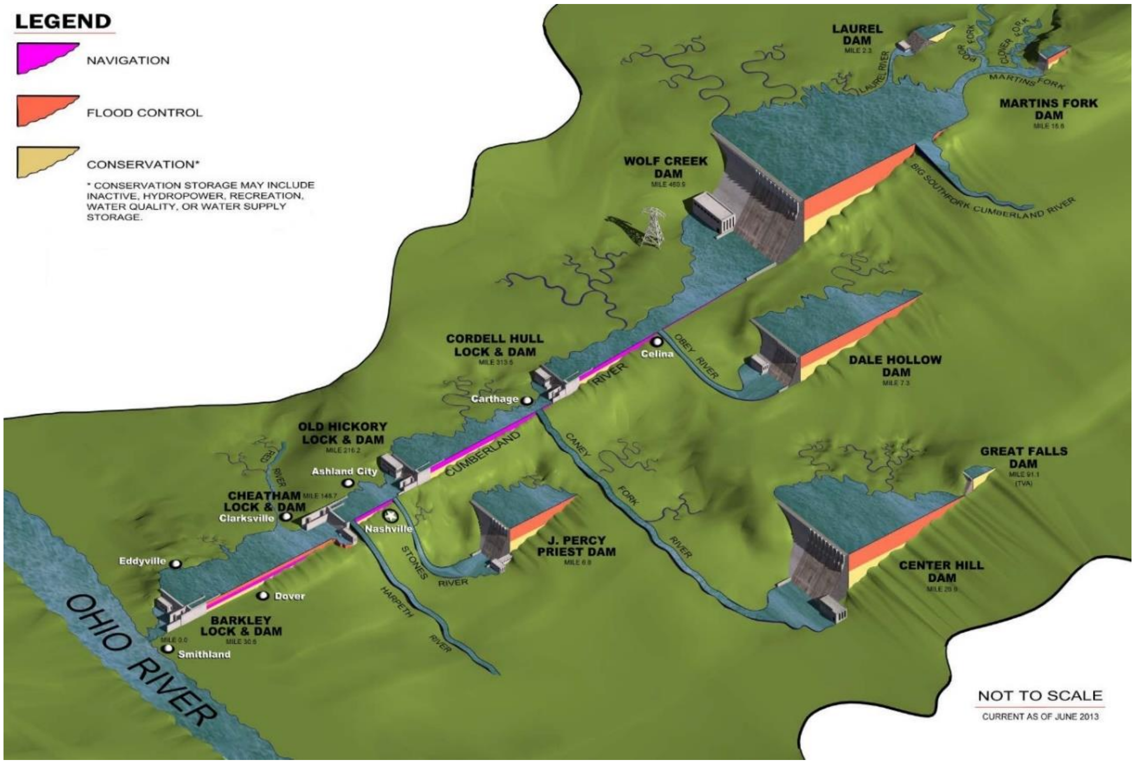


Figure 1.11 Cumberland River System (USACE, 2013)

CHAPTER 2 STATE-OF-THE-ART OF REAL-TIME FLOOD MANAGEMENT AND PREVIOUS MODELS

2.1 Flood Management

Floods are natural disasters that affect millions of people all over the world. More than ninety million of people are affected by floods every year worldwide, that accounts for 39% of all natural catastrophes after 2000, causing life losses and injuries, home displacement, destruction of properties and infrastructures, and many other problems (Emerton et al., 2016). Over the years, humans somehow succeeded to diminish the effect of floods, by constructing different kinds of dams somewhere on the riverway and later on they developed these hydraulic structures by adding more facilities for operation purposes to control and manage the flood water. However, full protection against floods is almost impossible. Flood management strategies generally encompass policy, design, planning, and management. In the literature, there are two deferent kinds of flood management strategies: planned management and real-time management. This research focuses on real-time management and reservoir operation under flood condition.

2.2 Real-Time Flood Management and Forecasting

Real-time flood management needs real-time inflow data to determine how much water should be released from the control facilities. Sometimes inflow data would not be available at the event time so, forecasting the required data for short-term depending on the availability may be required. This research will also focus on the flooding caused by heavy rainfall events requiring very fast response. Flood forecasting studies endeavor to produce as accurate as possible futures estimate of how much water will be discharged from a reservoir-river system based on the present state and past behavior of the river-reservoir scheme. (Bálint, 2002) defines flood forecasting as "an operational, result-oriented activity and as such pays less attention to the modeled system than to the output of the forecasting procedure." The forecasting outputs are peak stage or flood crest, flood flows, and stage or discharge hydrographs, flood volume, and floodplain inundation maps. Flood forecasting and warning processes are also, in most cases, to provide timely reliable information to the related department of civil and general public protection. The entire forecasting and warning process should be performed with enough lead time to allow the decision makers to take possible measures to prevent or to minimize the prospective flooding by taking appropriate actions.

2.3 Global-Scale Flood Forecasting Systems

In recent years, forecasts at the global scale have become possible because of the integration abilities of meteorological and hydrological modeling, satellite observations and land-surface hydrologic modeling, data collection improvements, and increases in resources and computer capabilities. Global hydrological forecasting and modeling are complicated because of the various processes of rainfall runoff and river systems. However, flood forecasting in terms of large-scale, with the advanced technologies and increased integration of hydrological and meteorological stations, has become easier, with an allowable percentage of uncertainty from the meteorological data input to the forecasted downstream flow rates in rivers.

The forecasting proficiency of the numerical models of global weather prediction has remarkably increased recently. The proficient medium scale precipitation numerical forecasting models have been shifted to large-scale forecasting models for the use of early warning. Table 2.1, depicts the resolutions and forecast ranges of some of the main quantitative precipitation forecasting products used in operative large-scale flood forecasting regimes.

Table 2.1 large-scale Flood Forecasting, Quantitative Precipitation Forecasts, (Emerton et al., 2016)

Product Type	Spatial Extent	Spatial Resolution	Temporal Resolution	Forecast Range	Uncertainty
Radar nowcasting	~10,000–50,000km^2	1–4 km	5–60 min	1–6 h	Low
Ensemble radar nowcasting	~10,000–50,000 km^2	1–4 km	5–60 min	1–6 h	↓
Radar-NWP blending	Regional	~2 km	15–60 min	~6 h	
Limited-area NWP	Regional–Continental	2–25 km	1–6 h	1–3 days	
Ensemble limited-area NWP	Regional–Continental	2–25 km	3–6 h	~5–30 days	
Global NWP	Global	~15–100 km	~3–6 h	~5–30 days	
Seasonal forecasts	Global	~15–100 km	~6–24 h	Months	High

The process of forecasting is challenging because of the atmosphere nature, as a small change in the initial condition in the system will produce an unpredictable result. The process of generating precipitation is difficult to model because the implicit physical processes of how precipitation is generated are complex to model, and that can result in deficiencies and forecast inaccuracies, particularly at longer lead times.

2.4 State of the Art of Real-Time Flood Forecasting

Floods are considered more than half of the world natural disaster, threatening millions of people’s lives and properties each and every year. Thus, real-time flood forecasting is essential to integrate flood risk management. Several types of the current state of the art of real-time flood forecasting models around the world have been reviewed here.

2.4.1 Real-Time Flood Warning Using High-Resolution Radar in Denmark

The Municipality of Hvidovre is a southwest suburb of Copenhagen, Denmark. During summer 2008, a real-time online alert system has been set up to provide information on the risk of basement flooding. The alert system based on local area weather radar (LAWR)8 rainfall forecast and a hydrological book-keeping model for twenty-two urban catchments. The high-resolution radar pictures are retrieved every 5 minutes to produce and update a projection for the next hour. This forecast is used by the decision support system (DSS) together with historical data to calculate the accumulated rainfall for each sub-catchment and to issue a warning if any of the pre-defined critical levels are exceeded. The Hvidovre citizens can be either warned automatically by the DSS, SMS and e-mail or access to the current status information through a web page developed with Dashboard Manager.

2.5 National Weather Services (NWS)

The task of the National Weather Service (NWS) can be summarized as providing water, weather, and climate forecasts and warnings for all the United States' territories, adjacent waters, and ocean areas, for life and property proection and the enhancement of the national economy. NWS data and products form a national information database and infrastructure which can be used by other governmental agencies, the private sector, the public, and the global community" (NWS, 2011b). Public, marine, and aviation forecasts are provided routinely by the NWS, as well as unscheduled short- and long-fused advisories and life-saving warnings when conditions warrant. NWS also provides seasonal and longer-term climate forecasts and warnings, and its observations are a critical part of the long-term climate record. (National Weather Service's Modernization Program, 2012).

2.5.1 Weather Prediction Center (WPC)

One of the nine centers of National Centers for Environmental Prediction (NCEP) is Weather Prediction Center (WPC), which belongs to 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 weather forecasting 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 an impact over the next seven days. The WPCQPF prepares and issues forecasts of quantitative of precipitation accumulation, heavy rain, heavy snow, and highlights areas with the possibility 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 model's preparation of short-range precipitation forecasts. To produce QPFs, the WPC meteorologists analyze the current condition of the atmosphere. Then they use a 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 2.1, illustrates an example of a Day 1 QPF on March 1st, 2017.

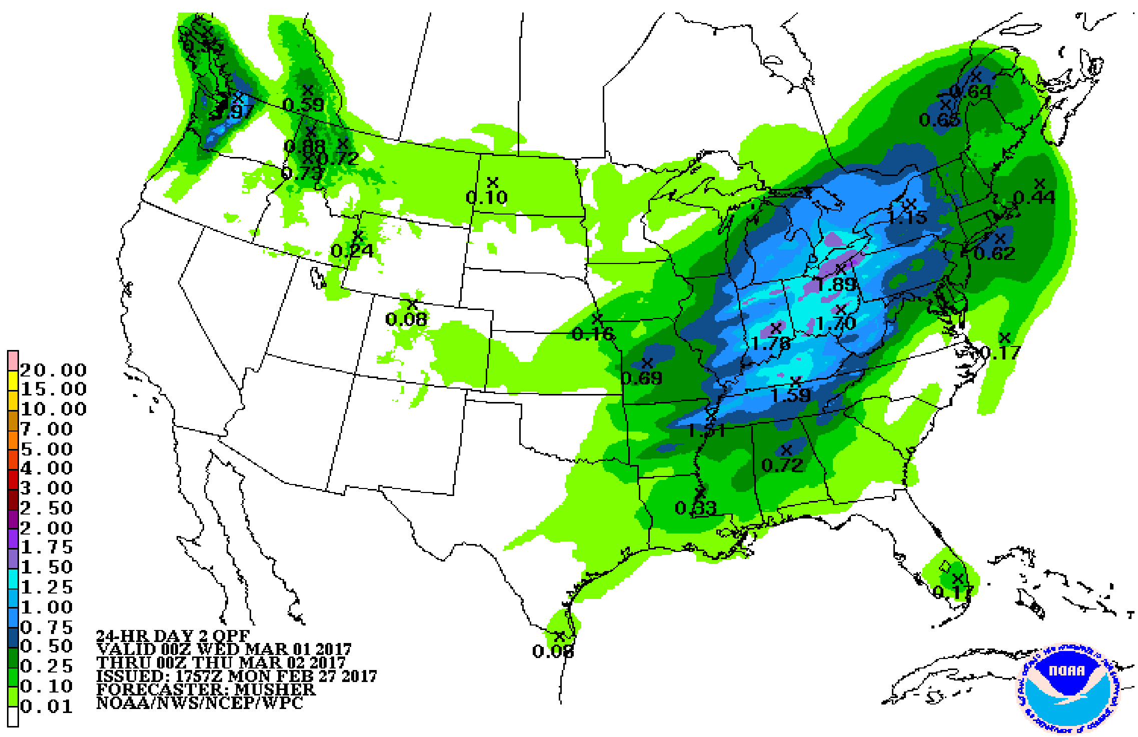


Figure 2.1 Example of a Quantitative Precipitation Forecast (Weather Prediction Center, 2017)

2.5.2 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 amounts of data from a variety of sources such as supercomputers, automated gauges, geostationary satellites, Doppler radars, weather observation stations, and the computer and communications system, called the Advanced Weather Interactive Processing System (AWIPS). National Weather Services provides hydrologic forecasts for almost 4,000 locations across the CONUS, (National Weather Service, 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, like the flood forecast level to which a river will rise and when it is most likely to reach its peak or crest, is shown through hydrographs. Other information includes but not limited to (National Weather Service, 2002)

a) The probability of a river exceeding minor, moderate, or major flooding.
b) The likelihood of a river exceeding a certain level, volume, and flow of water at certain points on the river during 90 day periods.
c) 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.

2.5.3 River Forecasting System (NWSRFS)

The National Weather Service River Forecasting System (NWSRFS) comprises programs and techniques for developing river forecasts (National Weather Service, 2005). The NSWRFS 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 (National Weather Service, 2005):

a) the Calibration System,

b) the Operational Forecast System (OFS), and

c) the Ensemble Streamflow Prediction System (ESP).

Every system interrelates with other different models and could be used to determine rivers forecast. Figure 2.2, shows the major components of the NWSRFS.

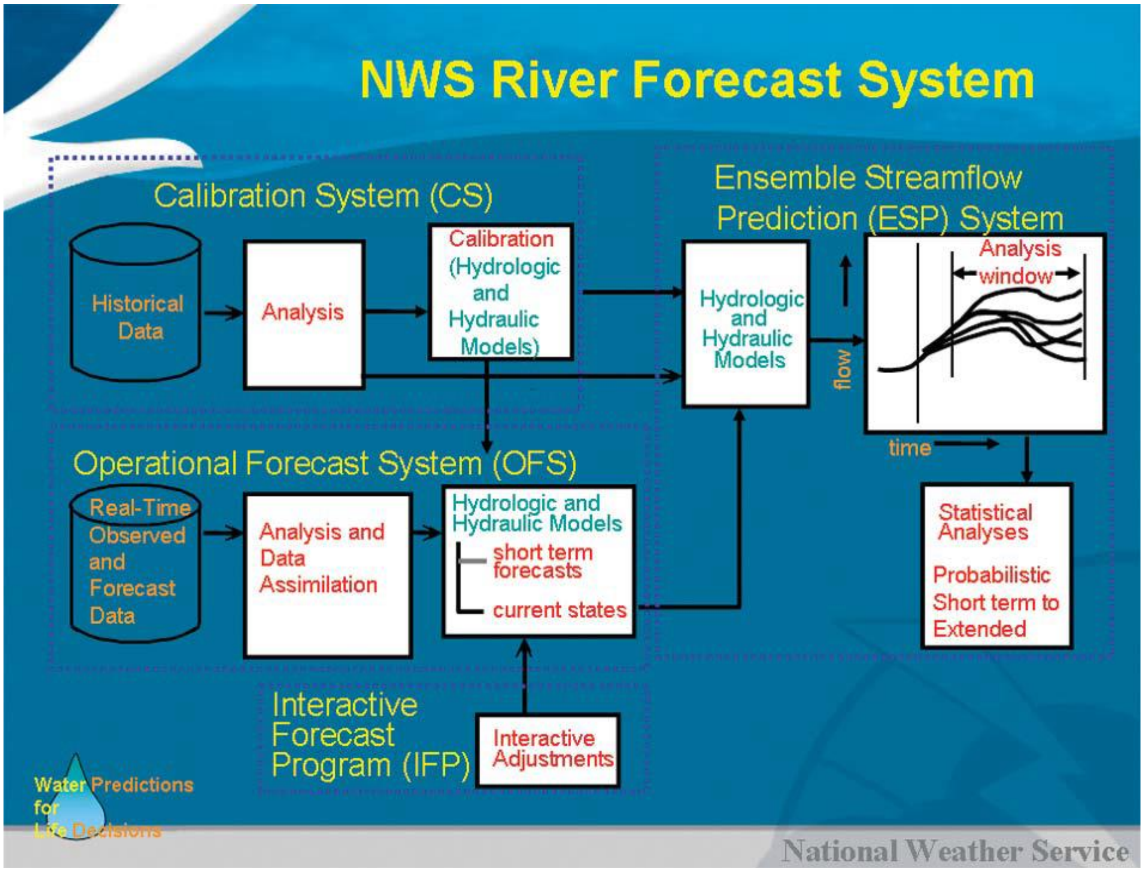


Figure 2.2 Operational flow diagram of the NWS River Forecast System. (McEnery, Ingram, Duan, Adams, & Anderson, 2005)

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 2-2 to specify the physics of water movement for any sub-basin (NWS, 2005):

Table 2.2 Hydrologic Operations in the NWS River Forecast System, (National Weather Service, 2005)

Types of Operation	Methods
Snowmelt Models	HYDRO 17 Snow Model
Rainfall-Runoff Models	Sacramento Soil Moisture Accounting
	NWS RFC Antecedent Precipitation Index Model
	Xinanjiang Soil Moisture
Temporal Runoff	Unit Hydrograph
Channel Losses and Gains	Simplified Loss/Gain Method, Consumptive Use
Routing Model	Lag and K; Muskingum; Layered Coefficient; Tatum
	Dynamic wave routing models (DWOPER/FLDWAV)
Baseflow Simulation	The base flow simulation model
	Single, independently controlled reservoir under
Reservoir Regulation	Various modes of operation
	Multiple reservoirs operated jointly
Adjustment Procedures	Simplified flow adjustment and blend
	Single-valued rating curve with log or hydraulic

Stage-Discharge Conversion	Extensions and loop ratings
Time Series Computation	Computation of mean discharge; Weight time series

The National Weather Service River Forecast Centers (RFCs) use 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 following (National Weather Service, 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).

2.5.4 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 substituted through the Community Hydrologic Prediction System. CHPS has been developed by the NWS in collaboration with Deltares (Delft Hydraulics as formerly known) in the Netherlands. The Delft-Flood Early Warning System (FEWS) serves as the infrastructure for CHPS with NWS hydrologic models and the U.S. Army Corps of Engineers (USACE) hydraulic models providing the forecasting core. Figure 2.3 illustrates the core idea of the relationship between CHPS and FEWS (NWS, 2010):

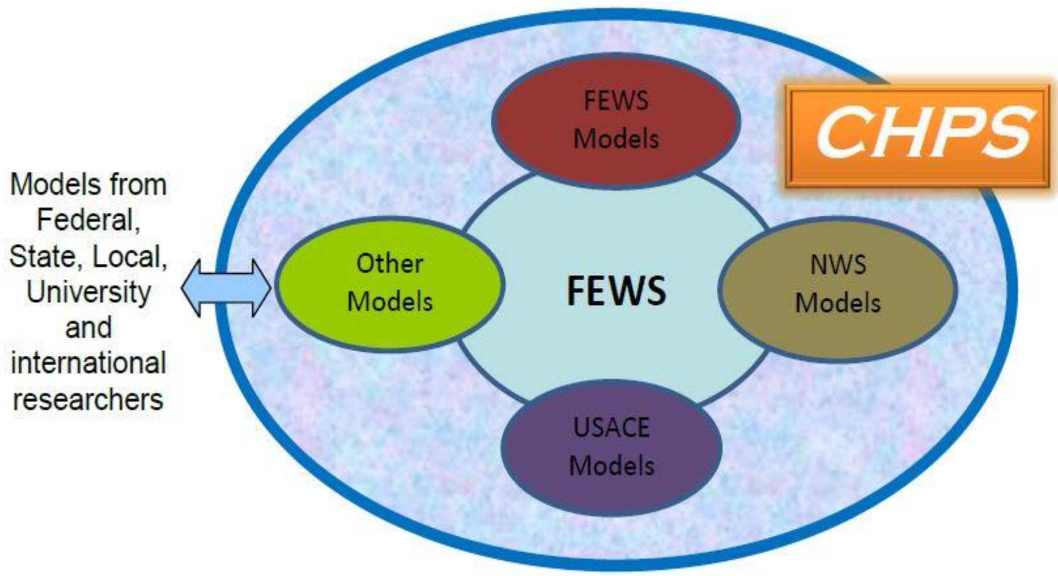


Figure 2.3 Relationship Between CHPS and FEWS, (National Weather Service, 2010)

CHPS is a system and a concept. The community concept of CHIP indicates a desire on the part of National Oceanic and Atmospheric Administration (NOAA) to get to the 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 (National Weather Service, 2010).

2.6 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 2.4, illustrates the basic structure of the real-time flood management model.

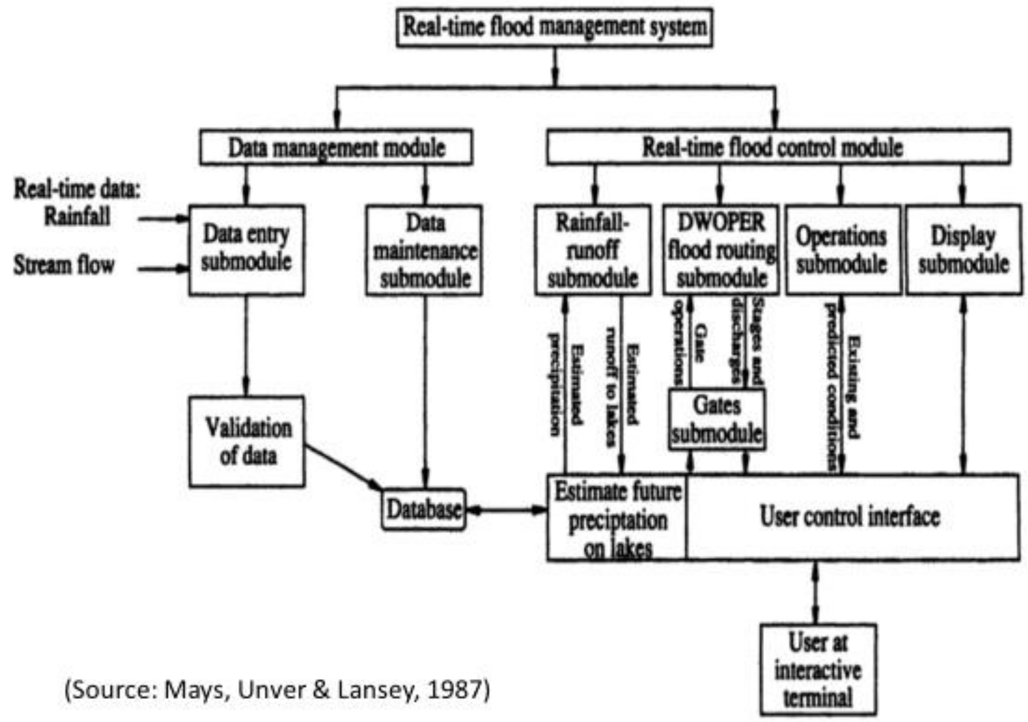


Figure 2.4 Structure of the LCRA Highland Lake System Real-Time Flood Management Model

The real-time flood control module contains the following submodules shown in Figure 2.4 Rainfall-runoff submodule rainfall-runoff model developed by the University of Texas at Austin for ungauged drainage area.

1. Unsteady flow routing submodule NWS, Dynamic Wave Operational Model (DWOPER).
2. 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.

3. Display submodule graphical 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 the 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 (L. W. 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 model's 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 (L. W. Mays, 1991).

2.7 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 principal 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.

2.7.1 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

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:

- ❖ A hydro-meteorological survey by running computer systems and collecting and collating data from the survey network;

- ❖ Hydro-meteorological forecast which produces and disseminates 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 a local and global scale;
- ✓ Numerical modeling for flood forecasting on the main river system.

Flood forecasting is conducted using the MIKE-FLOODWATCH system.

2.8 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 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.

2.8.1 The Anglian Flow Forecasting Modeling System (AFFMS)

The Environment Agency Anglian Region is in charge of flood forecasting and flood warning in the region (Todini, et al., 2005). The Environment Agency Anglian Region covers an area of 10,502 square miles, 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):

- ❖ A highly accessible internet-based user interface that can be 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 public.
- ❖ Comprehensive forecast databases with forecast analysis archive.
- ❖ An external data interface to allow visualization and application of a variety of data types from different sources.
- ❖ A generic modeling interface which allows applications of different forecast modeling tools including the MIKE 11 system.

- ❖ User-defined scenarios to evaluate alternative operation policies and uncertainty analysis.

2.9 Real-Time Flood Forecasting Previous Studies

The need for reliable flood forecasting in real time has recently increased, especially in, because of the high costs caused by flood events damages. Since the early eighties of the past century, the subject of real-time flood forecasting has gained increasing attention of many researchers.

2.9.1 Integrated Simulation and Optimization Models

The idea of connecting simulation and optimization models together first started by (L. W. Mays, Unver, & Lansey, 1987). They developed at that time new methodology for the real-time optimal flood operation of river-reservoir systems. it 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 a 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 & Mays, 1990) for the operation of multi-reservoir systems under flooding conditions was formulated as follows:

1. Objective:

$$Min\ Z = f(h, Q) \tag{2.1}$$

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,

$$g(h, Q, r) = 0 \tag{2.2}$$

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

$$\underline{Q} \leq Q \leq \overline{Q} \tag{2.3}$$

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

$$\underline{h} \leq h \leq \overline{h} \tag{2.4}$$

    d) Physical and operational bounds on gate operations,

$$0 \leq \underline{r} \leq r \leq \overline{r} \leq 1 \tag{2.5}$$

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

$$W(r) \leq 0 \tag{2.6}$$

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 damage at each location. The mathematical expression for this objective function is:

$$\text{Min } Z = \sum_i \sum_t c_i h_i^t \tag{2.7}$$

which $i \in I_c$ and $t \in T$, where Z is the objective function value; I is the location index; I_c is the set that contains flood control locations; t is the time index; T is the time domain; and 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.

Ahmed and Mays (2013), developed a newer model and applied it to the same lake system. This newer model methodology was for evaluating real-time optimal reservoir releases under flooding condition to minimize flood damages for a river-reservoir system. They formulated the problem as a discrete-time optimal control problem in which reservoir releases were considered as the control variables and water surface elevations and discharges as state variables. They also used the penalty function method to impose the reservoir's water surface elevations and discharges to the downstream reaches into the objective function, Equation 2.8. Their model approach consisted of two primary interfaced components, Figure 2.5. These components are the

U.S. Geological Survey Full Equation routing model (Franz & Melching, 1997), to simulate the dynamic of unsteady flow of the river-reservoir system in one-dimensional and simulated annealing method as an optimization technique to optimize reservoir releases by flood control gates operating subjected to the mentioned above system constraints. The constraints of the were the same as the (Unver & Mays, 1990) model. The methodology solved an augmented control case. The model was applied to the Lake Travis on the Lower Colorado river-reservoir system in Texas,

Figure 2.6. The application of the model revealed the advantage of the model in improving a given operation policy, no matter what kind of objective function was, whether it was linear or nonlinear. 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.

$$\text{Min}\, Z = \sum_{i} \sum_{j} c_i h_i^j + c_i' Q_i^j \qquad (2.8)$$

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's the cost coefficient as a function of discharge.

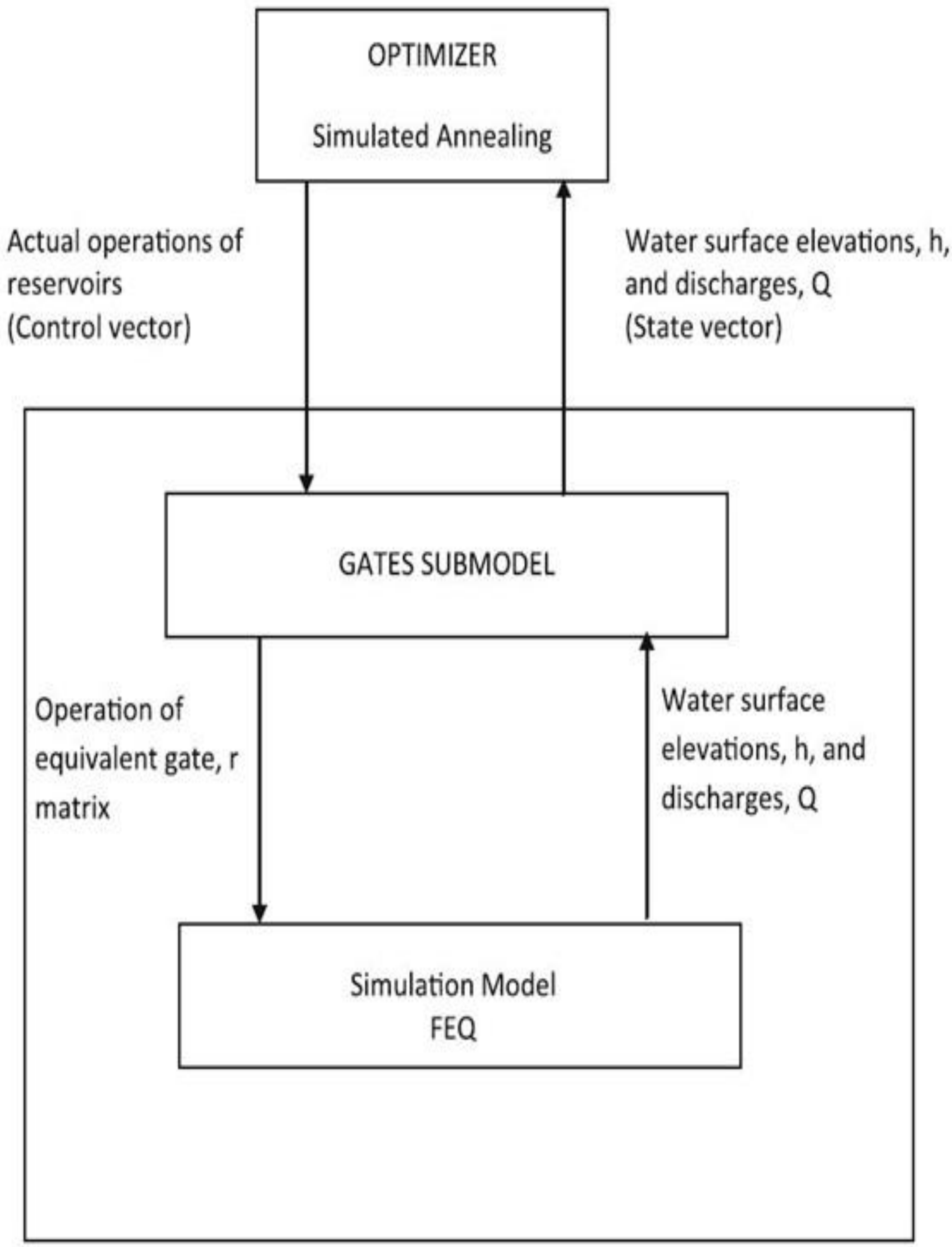


Figure 2.5, Optimal control approach to operations problem, (Ahmed & Mays, 2013)

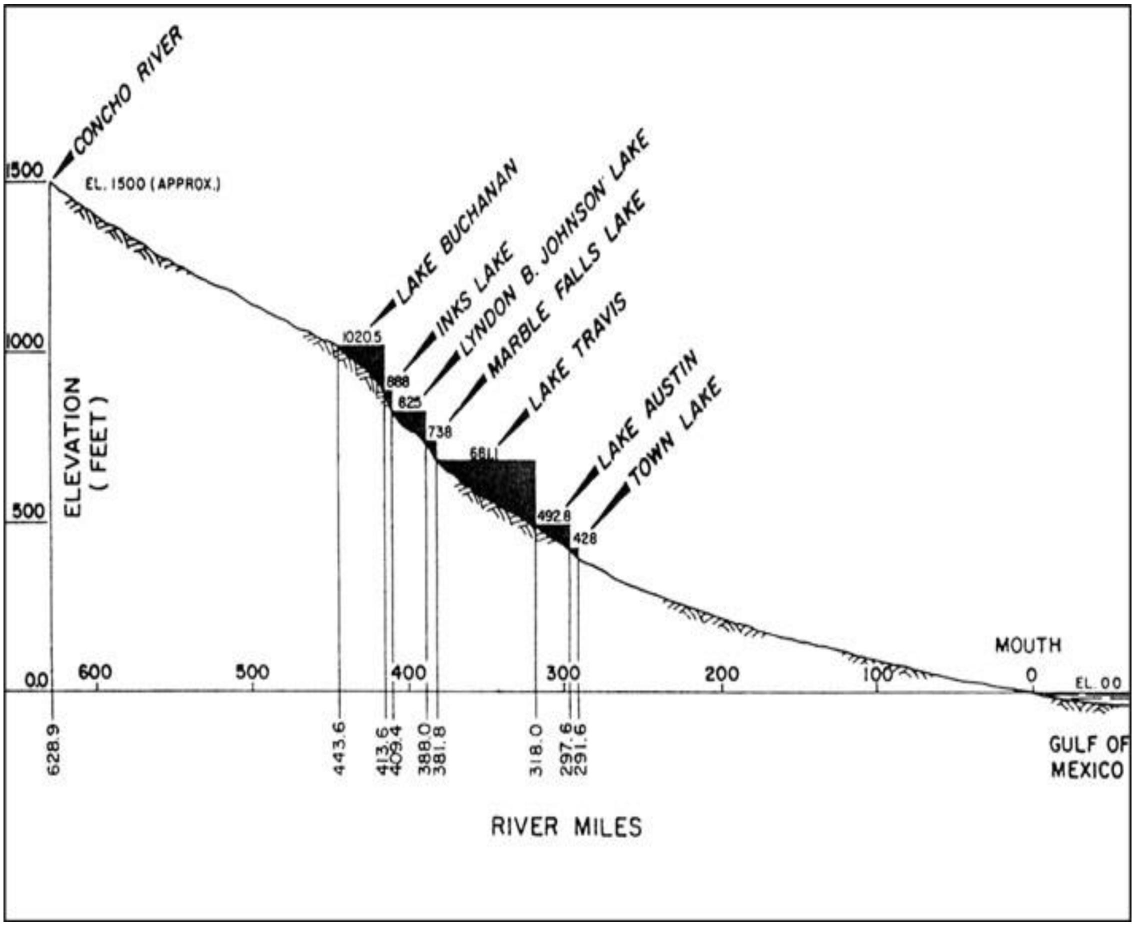


Figure 2.6, Highland Lakes System, Texas

A newer approach, later on, developed and tested by (Che & Mays, 2015), for determining of river-reservoir systems release schedules that is before during and after an extreme flood event in real-time. They formulated the problem as a real-time optimal control problem in which the releases from a reservoir represented the decision variables of their optimization model. There were five components in their model: the first component was the U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center - Hydrologic Modeling System (HEC-HMS), That models rainfall-runoff processes of watershed systems; the second was the U.S. Army Corps of Engineers

Hydrologic Engineering Center - River Analysis System (HEC-RAS) for one-dimensional unsteady flow dynamics and routing; the third components was a reservoir release operation model followed by the fourth part which was a short-term rainfall forecasting model to forecast rainfall over the next a few hours during a rainfall event; and the last components was a genetic algorithm (GA) (in Microsoft Excel) optimizer interfaced with the other components that determine the real-time operation of a river-reservoir systems.

Figure 2.7, depicts the model structure is an interconnection of model components.

The mechanism of the model lets it start with the known real-time rainfall as the actual rainfall input up to the time of decision making. Thereafter, the forecasting process of the next time period will start to generate forecasted rainfall, also flows in the system using real-time rainfall of NEXRAD or rainfall gages network will be determined. The model considers both observed and forecasted rainfall data to use it as the input of HE-HMS model to simulate the rainfall runoff process of the watershed, the output though of HEC-HMS which is the hydrographs will be used as inputs for the optimization model which is here the genetic algorithm into Microsoft Excel to determine the releases of the river-reservoir system, Figure 2.8, shows the basic steps of the optimization and simulation model mechanism.

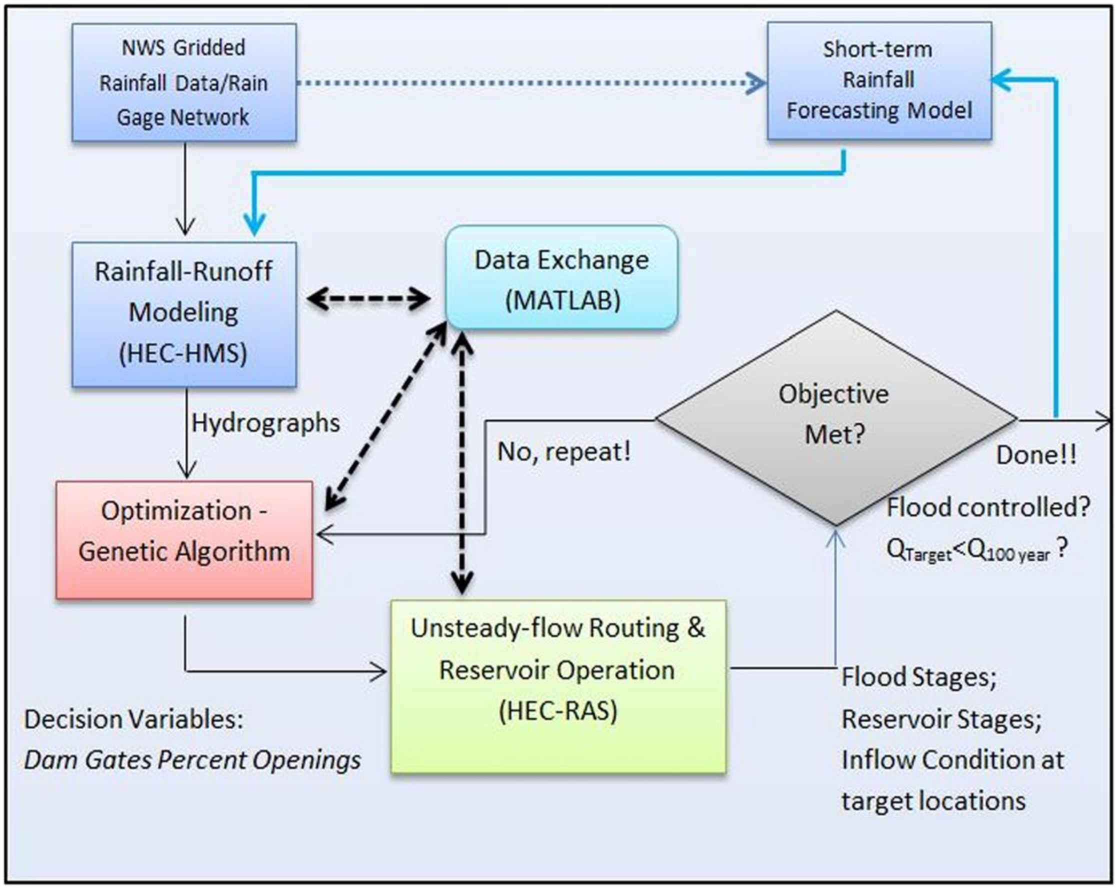


Figure 2.7 Model structure and interconnection of model components, (Che & Mays, 2015, 2017)

The model uses MATLAB version 2014b to perform the interfacing between HECHMS, the reservoir operation model, and HEC-RAS. They used free open software Pullover's Macro Creator version 4.1.0 to automate the overall communicating processes among all model components. The model employed the genetic algorithm (GA) of Microsoft Excel as optimization method which is not as efficient as MATLAB's (GA) that we are using in this research, more details about MATLAB' (GA) is explained in chapter seven.

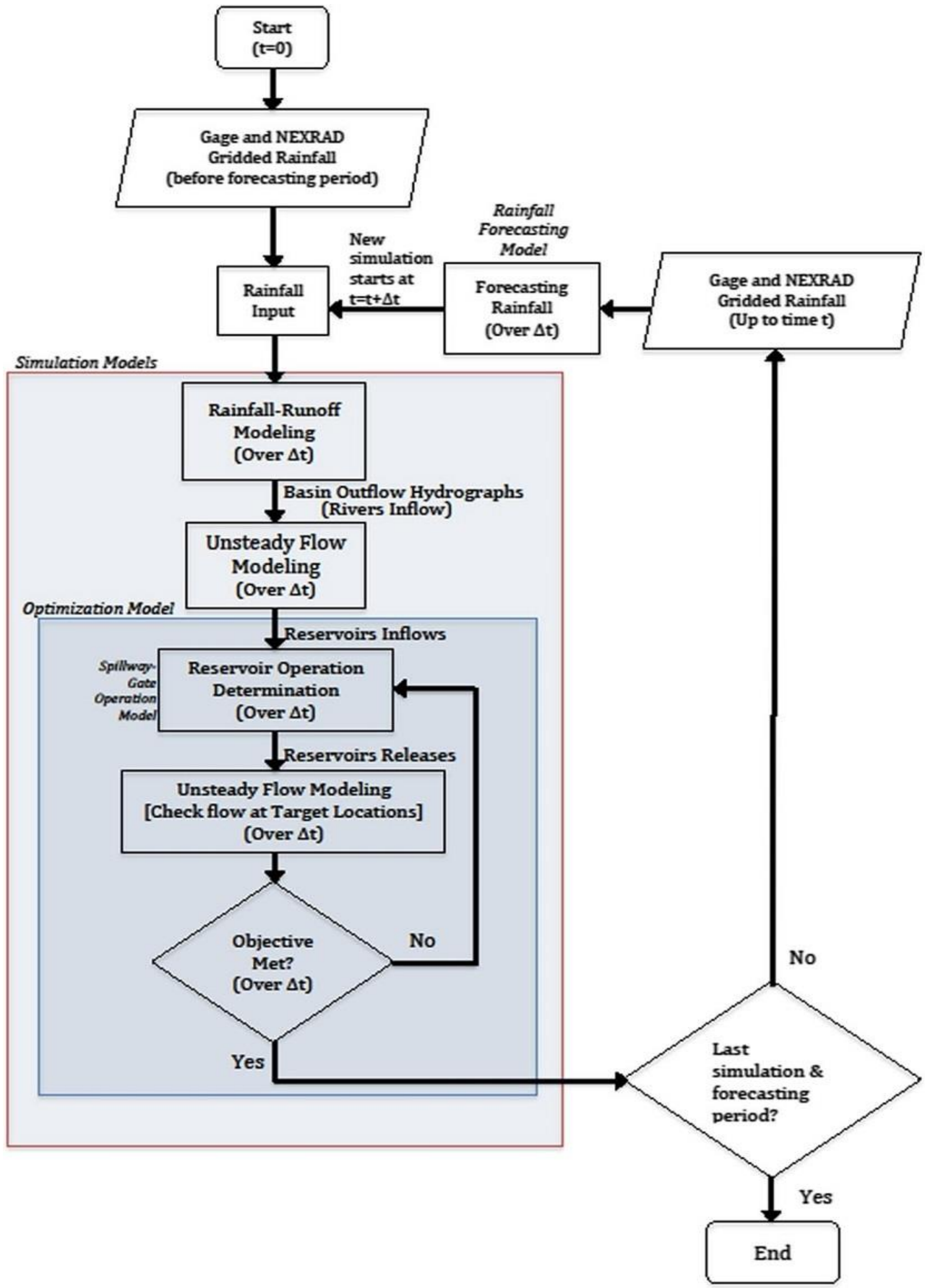


Figure 2.8, Basic steps of the optimization and simulation model, (Che & Mays, 2015)

The authors tested the model in a real-time flood control operation of a river-reservoir system and for quality assurance as well by suggesting a small example application. They proposed two watersheds with two reservoirs system 1 and 2. Reservoir 1 and 2 located downstream watershed 1 and 2 respectively, so that the runoff of every watershed discharged directly in its respective reservoir. Figure 2.9 illustrates the schematic of the example application. Then, the releases from each reservoir are routed downstream through the reaches 1, 2 which in reach up to the city A where the damage has to be minimized.

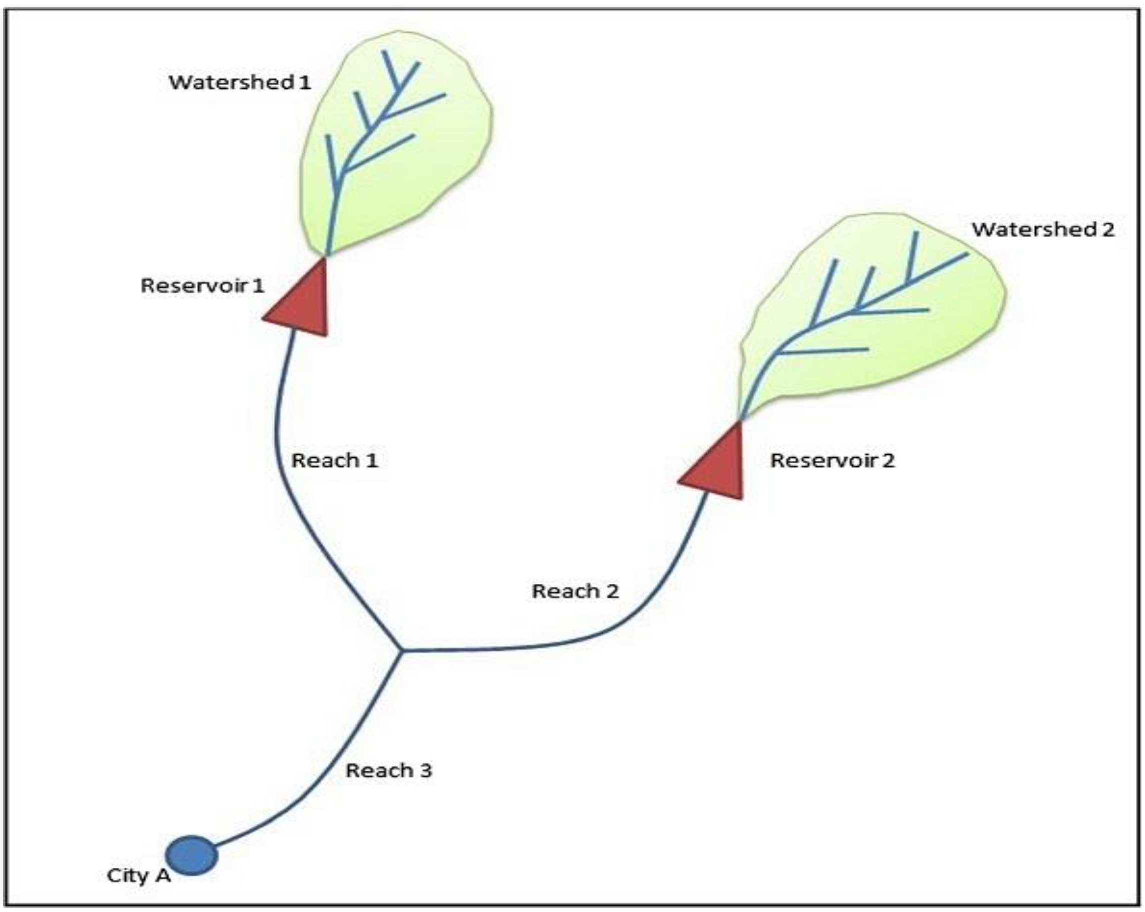


Figure 2.9, Example system

Thereafter, (Che & Mays, 2017) applied the same model to a real-world problem which was the catastrophic May 2010 flood on the Cumberland River at Nashville, Tennessee, Figure 2.10, which is described in more details in the next chapter, using the same components and the methodology above.

Figure 2.10, Cumberland River Basin

The application of the model was to a portion of the Cumberland River upstream of Nashville, Tennessee for the May 2010 extreme flood event which is used is this researched as well. The optimization/simulation model has been applied to a portion of the large river-reservoir system, the Cumberland River basin as in Figure 2.11. The objective (Equation 2.9) of the optimization and simulation model was to minimize the peak flood stage at Nashville, subsequently to keep the flood elevation at downtown Nashville below the 100-year stage of 48 ft during the whole simulation period. The reservoir operations rules were set in the model according to the water control USACE manual.

$$\text{Min}\, Z = Min(Max\ h_i^t) \tag{2.9}$$

The process of simulating and optimizing of the river-reservoir system starts with the HEC-HMS model the (hydrologic modeling), that simulates the Cumberland River Basin rainfall-runoff process. The rainfall-runoff model (HEC-HMS) includes all sub-basins and reaches upstream to the Cordell Hull Reservoir, where the reservoir inlet node is the outlet node of the HEC-HMS model. Then the HEC-HMS model generates the Cordell Hull reservoir inflow hydrograph. These inflow hydrographs is transmitted to the Cordell Hull reservoir operation and optimization models for determining its optimal operation. Once the gate release decisions are computed, the information becomes the input of the unsteady flow HEC-RAS model for downstream hydraulic routing up to Old Hickory Reservoir inlet.

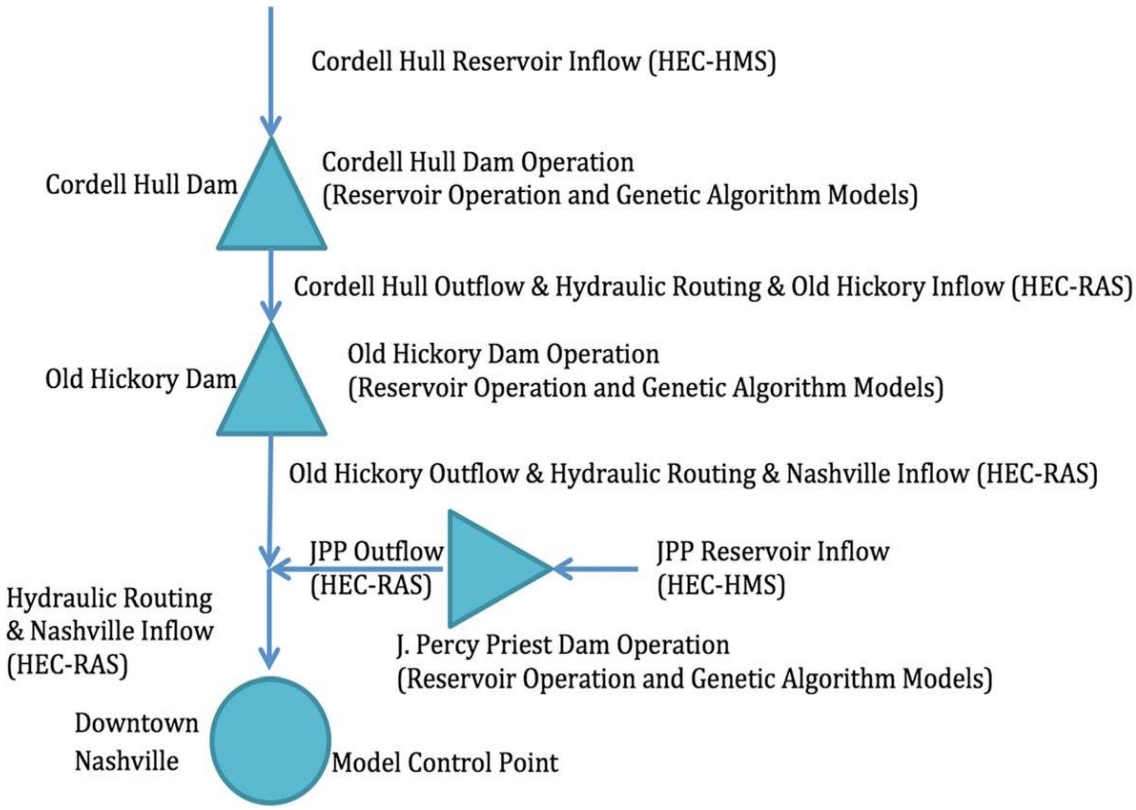


Figure 2.11, Schematic of the portion of the Cumberland River Basin used in the optimization/simulation model, (Che & Mays, 2017)

The inflow hydrographs now enter the Old Hickory reservoir operation and optimization model like Cordell Hull Dam and also for J. Percy Priest Dam to determine the optimal operation. The condition of the system control point at Nashville, Tennessee determines the optimization model decision variables. Thus, the process will be repeated until the objective is met. The objective here is to keep the levels of flood peak at Nashville under the 100-year level through the event of May 2010 which is 48 ft with associated flow rate 172000 cfs as seen in Figure 2.12.

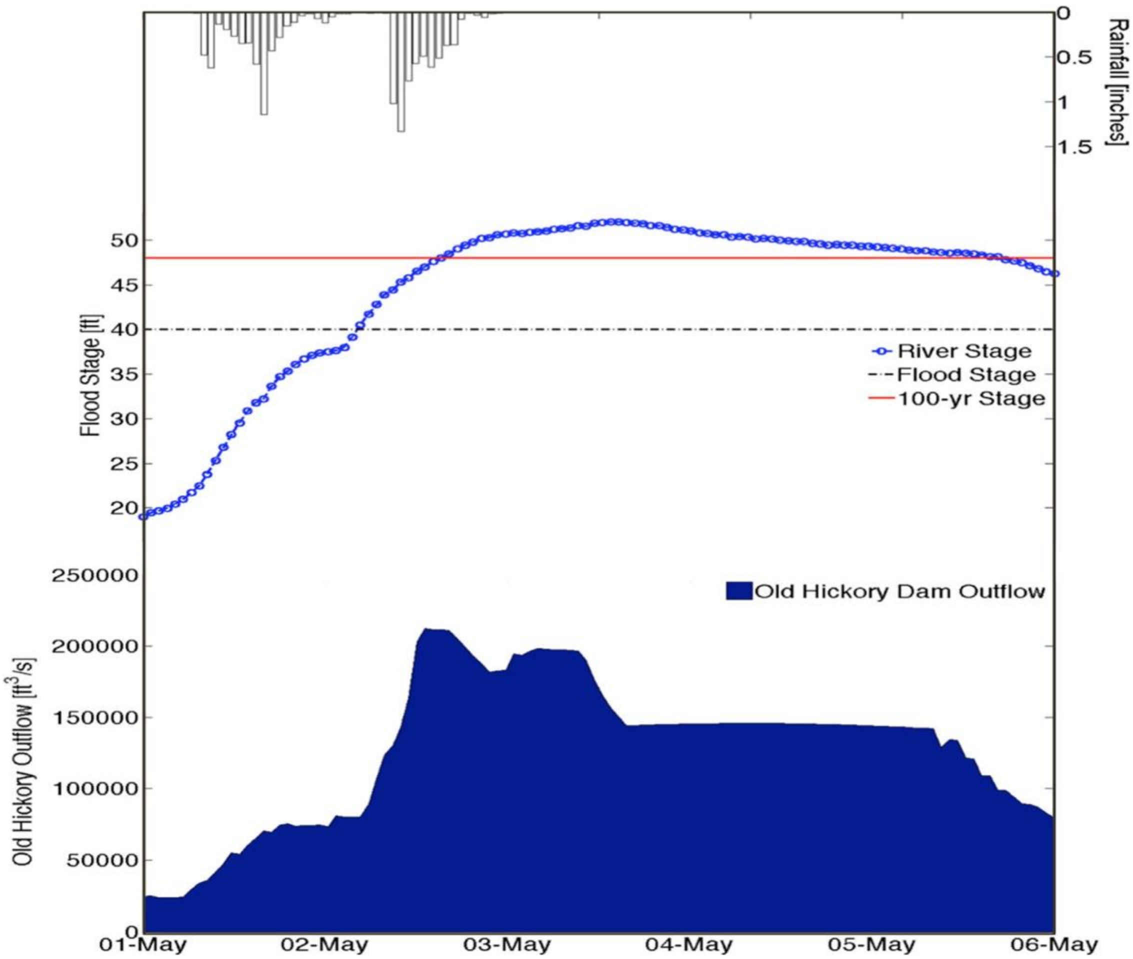


Figure 2.12, Discharge and flood stage as a function of time in Nashville, Tennessee during the April 29 – May 7, 2010 event, (U.S. Army Corps of Engineers, 2010a)

The optimization/simulation operation model showed an efficient methodology, algorithm, numerical, and optimal reservoir systems operation model for determining reservoir release schedules prior to, during, and after an extreme flood event in a real-time fashion.

CHAPTER 3 RAINFALL-RUNOFF MODEL

3.1 Introduction

Reliable estimation of a river flow generated from the watershed is required as part of a data set that help to make a decision for planning and to manage river-reservoir systems. The characteristic of the time series of a river flow that can affect modeling, simulation, and planning of river-reservoir system can include the sequencing of flows hourly and longer time steps, the spatial and temporal changes in flows, season allocation and flows characteristics. The better prediction of streamflow would be expected to come from water level observations got at a gauge station, converted to flow estimates using a well defined and stable stage-discharge curve. However, such observations are only available for a limited number of gauging locations and for the relatively short time range. Estimates for ungauged locations and a much longer period are required for contemporary water management, and ways to make estimates for possible future conditions are required as well (Vaze, Jordan, Beecham, Frost, & Summerell, 2012).

A variety of methods are available to determine the run-off from catchments, using either observed or forecasted data wherever possible, or estimating by experimental and statistical techniques, and more commonly using rainfall-runoff models. The methodology of modeling used to estimate streamflow depends on the purpose of the modeling, time, and skills and instruments are available within the organization. With increasing levels of inter-agency collaboration in water planning and management, development of a best practice approach to rainfall-runoff modeling is desirable to provide a consistent process and improve interpretation and acceptability of the modeling results.

3.2 KINEROS (The Kinematic Runoff and Erosion Model)

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 a small urban and agricultural watersheds (Woolhiser, Smith, & Goodrich, 1990). A cascade of planes and channels represents the watershed; the partial differential equations are describing overland flow, channel flow, and erosion, and finite difference techniques solve sediment transport. Spatial variability of rainfall and infiltration, runoff, and erosion parameters can be determined. KINEROS can also be used to compute the effects of a number of artificial features such as urban developments, small detention reservoirs, or lined channels on flood hydrographs and sediment yield. (Memarian et al., 2013) used three storm events in different intensities and durations were required to calibrate KINEROS2. They showed that the calibration results were excellent and very good fittings for runoff and sediment simulations based on the aggregated measure. Validation results demonstrated that the INEROS2 is reliable for runoff modeling, while KINEROS2 application for sediment simulation was only valid for the period 1984–1997. Later on, (Guber et al., 2014) has tested KINEROS2 fitting to the experimental cumulative runoff data. They considered infiltration in unsaturated soil is accounted for by a net capillary drive parameter in the Parlange equation,G. Results showed that the most accurate prediction was obtained when the G parameter was matched to the cumulative ranfall-runoff. The KINEROS2-recommended parameter slightly overestimated the calibrated value of parameter G and yielded less accurate predictions of runoff. The pad transfer functions estimated parameters systematically deviated from calibrated G values that caused high uncertainty in the KINEROS2 predictions. The flow chart below Figure 3.1 illustrates the solution solving algorithm of the KINROS2 model:

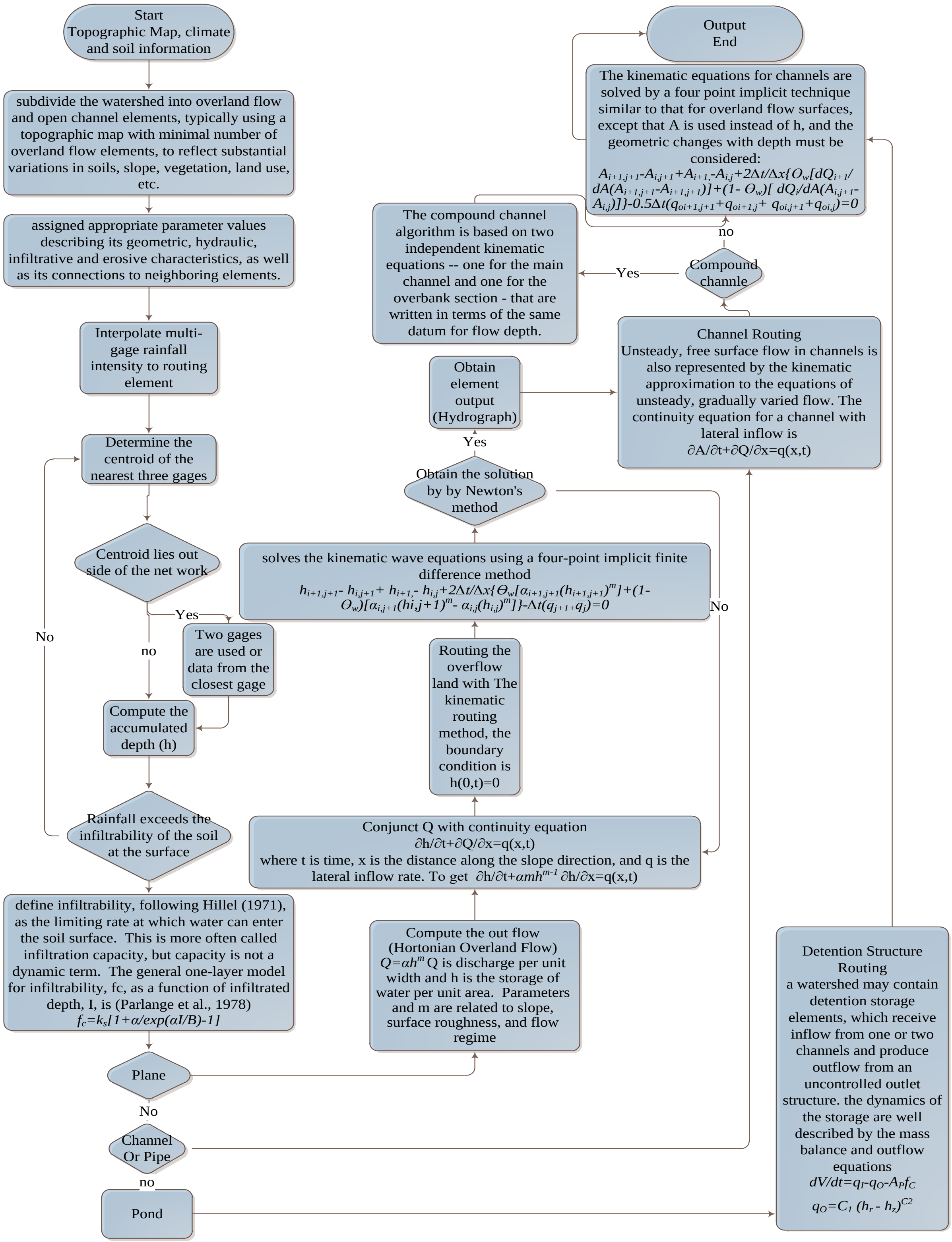


Figure 3.1 Detailed Flowchart of the Procedure Used in KINEROS

3.3 MIKE SHE Model

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. 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 groundwater flow. MIKE SHE can be characterized as a deterministic, physically-based, distributed model. The distributed model MIKE SHE has been used to model wetlands for a lowland wet grassland, the Elmley Marshes, in southeast England and demonstrated by (Thompson, Sørenson, Gavin, & Refsgaard, 2004). The results suggested that improvements could be made to the MIKE SHE bypass flow routine to enable it to represent macropore flow associated with soil cracking and swelling more accurately. (Butts et al., 2005) Presented new developments of grid-based hydrological modeling have been spurred by increasing access to meteorological modeling, radar, and satellite remote sensing. The approach adopted in the flood relief project has been to develop a flexible, hydrological modeling framework based on the MIKE SHE that permits changes in the model structure, including both conceptual and physics-based process descriptions, to be made within the same modeling tool. The model framework was used to derive a distributed sub-catchment based conceptual model for modeling the rainfall-runoff process and a comprehensive hydraulic model for the highly flood-prone and complex Upper and Middle Odra River in Poland. This model has been successfully calibrated against measurements both in the main river system and tributary catchments, including the extreme flooding in July 1997. Figure 3.2 shows how a catchment is represented in an integrated fashion by the major processes and their interaction (MIKE by DHI, 2008). 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 (Danish Hydraulic Institute, 2003).

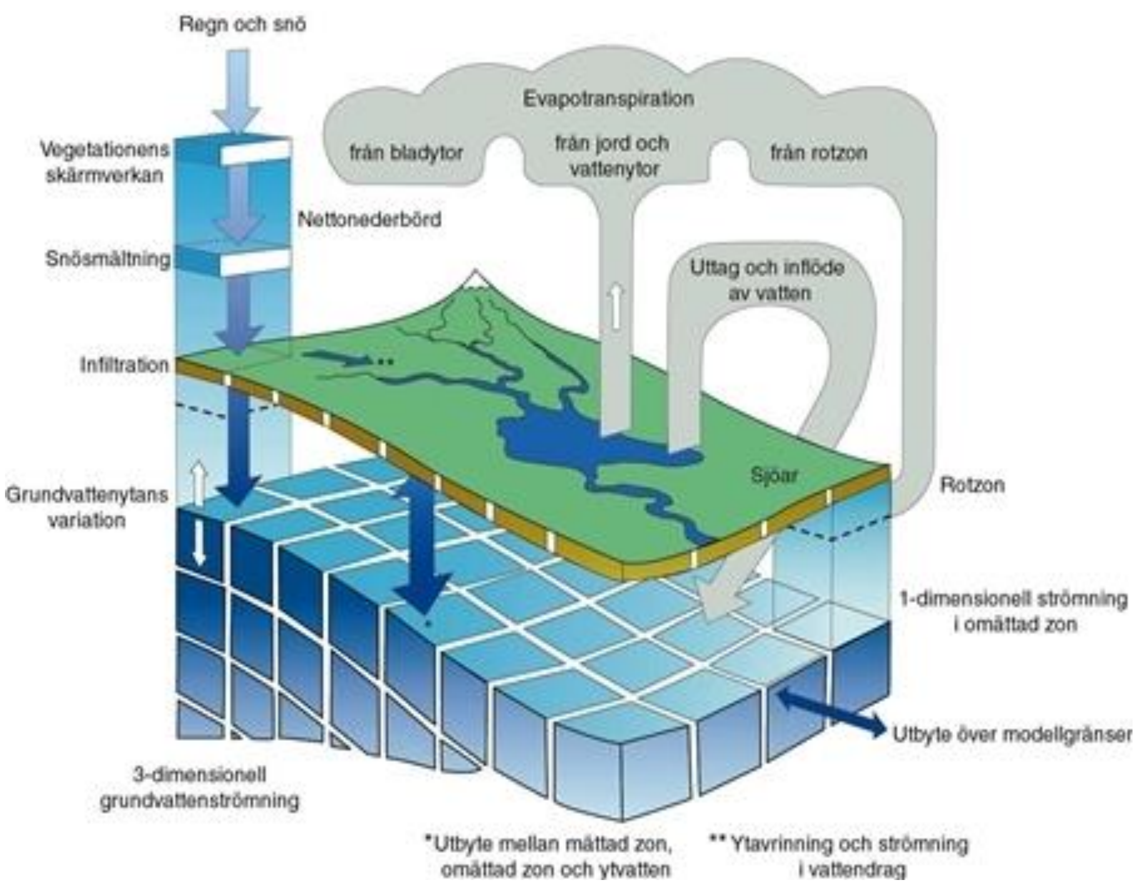


Figure 3.2 Process-Based Structure of the MIKE SHE Hydrological Modeling System

3.4 BASINS (Better Assessment Science Integrating point & Non-point Sources)

The Better Assessment Science Integrating Point and Nonpoint Sources (BASINS) is a multipurpose environmental analysis system intended to help regional, state, and local agencies perform watershed- and water quality-based studies. It was developed by the U.S. Environmental Protection Agency to assist in watershed management and TMDL development by integrating environmental data, analysis tools, and watershed and water quality codes. A geographic information system (GIS) provides the integrating framework for BASINS. GIS organizes spatial information so it can be displayed as maps, tables, or graphics. GIS allows the user to analyze landscape information and display relationships among data. Figure 3.3 depicts BASINS's Geographic Information System Interface. By using GIS, BASINS has the flexibility to display and integrate a wide range of information (e.g., land use, point source discharges, and water supply withdrawals) at a scale chosen by the user (U.S. Environmental Protection Agency's, 2015)

BASINS models watershed and water quality studies easier by putting together key data and analytical components in one tool. BASINS allow users to access national environmental information efficiently, incorporate local site-specific data, apply assessment and planning tools and run a variety of proven, robust nonpoint loading and water quality models. The BASINS model is a useful tool for those interested in watershed management, development of total maximum daily loads (TMDLs), coastal zone management, nonpoint source programs, water quality modeling, and National

Pollutant Discharge Elimination System (NDPES) permitting, (U.S. Environmental Protection Agency's, 2015).

BASINS, through its extensible component-based architecture, is a dynamic system whose capabilities have increased as technology has allowed, and needs have demanded. Another implication of the extensible architecture is that each BASINS tool can be developed independently of each other BASINS tool, greatly increasing the potential for independent groups to develop compatible BASINS extensions simultaneously, (Kinerson, Kittle, & Duda, 2009), Figure 3.4 shows BASIN4 system overview.

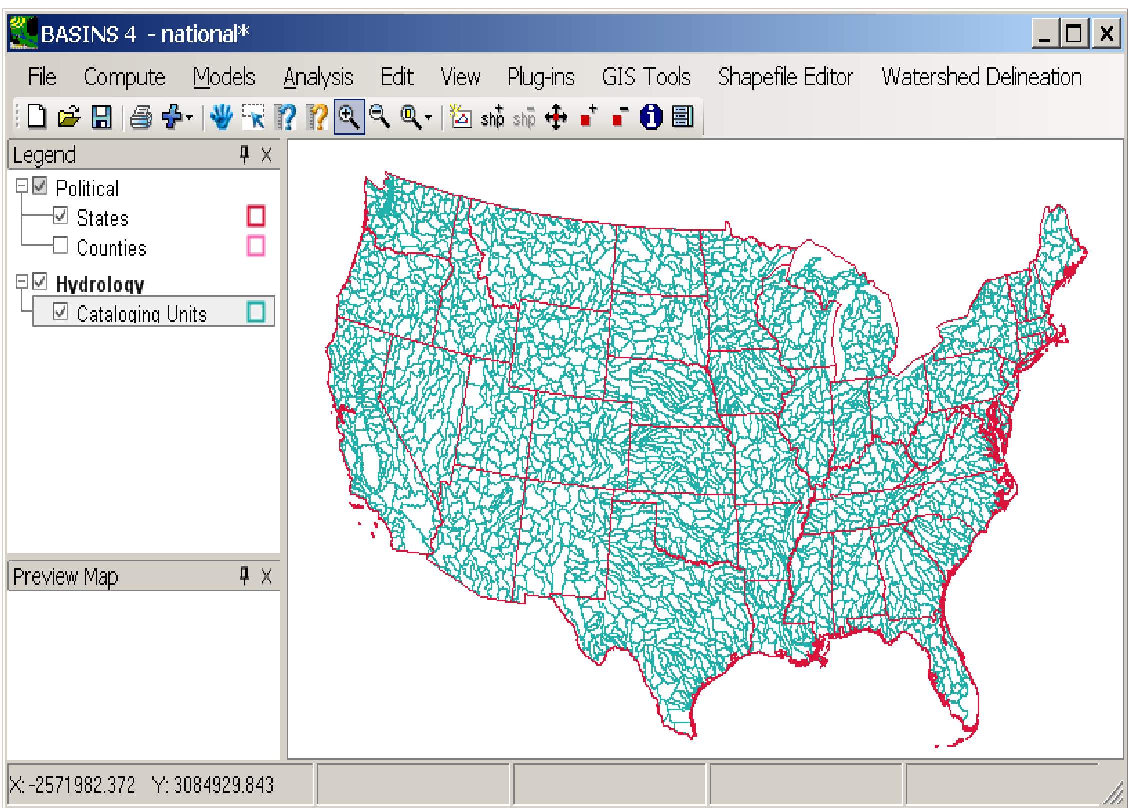


Figure 3.3 BASINS Geographic Information System Interface

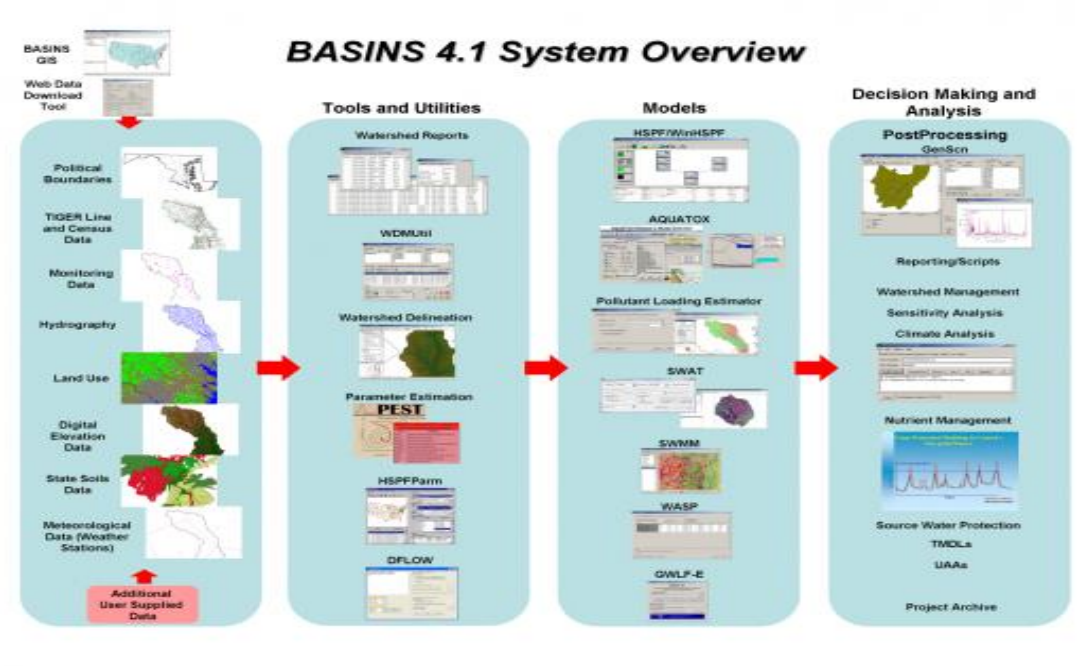


Figure 3.4 BASIN System Overview

3.5 Hydrologic Modeling System (HEC-HMS)

The Hydrologic modeling system (HEC-HMS) program is a product of the US Army Corps of engineer' research and development program, (Feldman, 2000). HEC-HMS is intended to simulate the precipitation-runoff processes of a dendritic watershed. It is also designed to be applicable in a wide range of geographic areas for solving a broad range of problems and examples. This includes large river basin water supply and flood hydrology to small urban or natural watershed runoff. Hydrographs produced by the program can be used directly or in conjunction with other software for studies of water availability, urban drainage, flow forecasting, future urbanization impact, reservoir spillway design, flood damage reduction, floodplain regulation, wetlands hydrology, and systems operation, (Fleming & Brauer, 2016).

For precipitation-runoff-routing simulation, the program provides the following components, (Feldman, 2000):

- ✓ Precipitation-specification options which can describe an observed (historical) precipitation event, a frequency-based hypothetical precipitation event, or an event that represents the upper limit of precipitation possible at a given location.
- ✓ Loss models which can estimate the volume of runoff, given the precipitation and properties of the watershed.
- ✓ Direct runoff models that can account for overland flow, storage, and energy losses as water runs off a watershed and into the stream channels.
- ✓ Hydrologic routing models that account for storage and energy flux as water moves through stream channels.
- ✓ Models of naturally occurring confluences and bifurcations.
- ✓ Models of water-control measures, including diversions and storage facilities.

HEC-HMS has been used in this research to model the rainfall-runoff as the first component of the optimization simulation model. Figure 3.5 reparents watershed scale rainfall-runoff process represented by HEC-HMS.

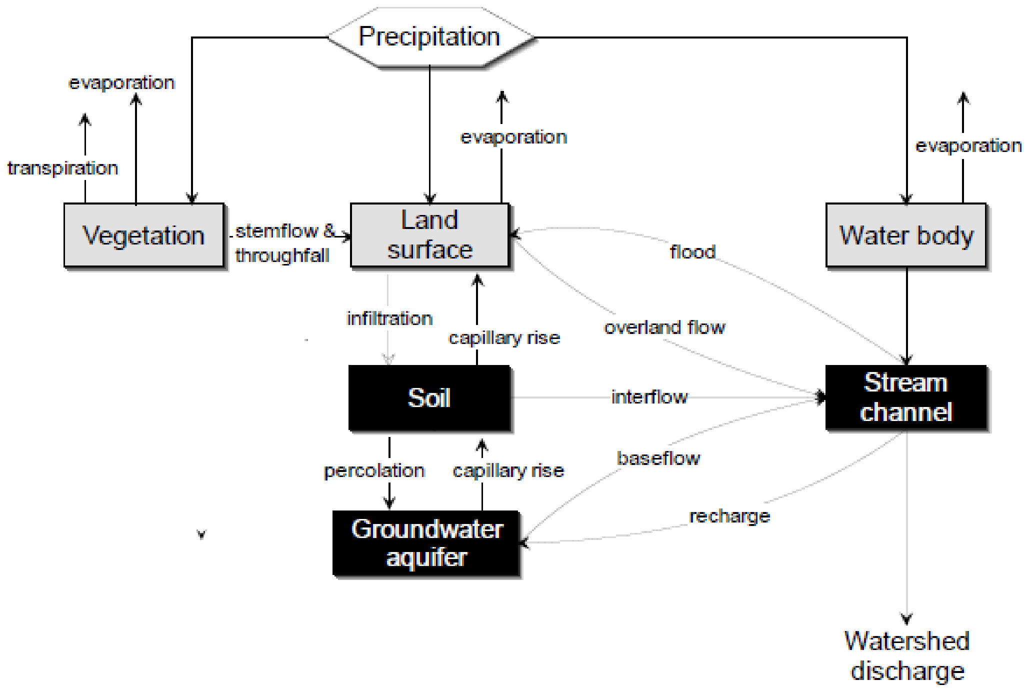


Figure 3.5 Watershed Scale Rainfall-Runoff process Represented by HEC-HMS (Feldman, 2000).

When modeling a storm event using HEC-HMS, precipitation falls on the land surface, and water may pond. For continuous, non-event-based simulation, evapotranspiration may be included in the model. Depending on soil type, land surface type, antecedent moisture and other properties of the watershed, a portion of the water may infiltrate. The infiltrated water 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 accounts for vertical percolation of water from the 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 HECHMS serves as the first component to compute the watershed runoff after a storm event, with a 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.

3.5.1 Water Description in HEC-HMS

Physically, a watershed can be represented in the HEC-HMS using the BASIN model. The is a number of hydrologic elements can be added and connected to each other to model the natural flow of water for a real watershed. Table 3.1 describes each hydrologic element of HEC-HMS.

The HEC-HMS program deals with each element through parameter data so that that program models the hydrologic processes assigned to each element. For example, the sub-basin element has many mathematical models available in the program for determining the losses of precipitation, transforming excess precipitation to the subbasin outlet also adding baseflow. Table 3.2 shows the available method for reach and sub-basin elements. The parameter data can be entered the component editor or through the global editor which can be used for viewing the parameter data.

Table 3.1 The Hydrologic Elements Kinds, (U.S. Army Corps of Engineers, 2016d)

Hydrologic Element	Description
Subbasin	The subbasin is used to represent the physical watershed. Given precipitation, outflow from the subbasin element is calculated by subtracting precipitation losses, calculating surface runoff, and adding baseflow.
Reach	The reach is used to convey streamflow in the basin model. Inflow to the reach can come from one or many upstream elements. Outflow from the reach is calculated by accounting for translation and attenuation. Channel losses can optionally be included in the routing.
Junction	The junction is used to combine streamflow from elements located upstream of the junction. Inflow to the junction can come from one or many upstream elements. Outflow is calculated by summing all inflows.
Source	The source element is used to introduce flow into the basin model. The source element has no inflow. Outflow from the source element is defined by the user.
Sink	The sink is used to represent the outlet of the physical watershed. Inflow to the sink can come from one or many upstream elements. There is no outflow from the sink.
Reservoir	The reservoir is used to model the detention and attenuation of a hydrograph caused by a reservoir or detention pond. Inflow to the reservoir element can come from one or many upstream elements. Outflow from the reservoir can be calculated using one of three routing methods.
Diversion	The diversion is used for modeling streamflow leaving the main channel. Inflow to the diversion can come from one or many upstream elements. Outflow from the diversion element consists of diverted flow and non-diverted flow. Diverted flow is calculated using input from the user. Both diverted and non-diverted flows can be connected to hydrologic elements downstream of the diversion element.

Table 3.2 Sub Basin and Reach Elements Method in HEC-HMS

Hydrologic Element	Calculation Type	Method
Subbasin	Canopy	Dynamic
		Simple (also gridded)
	Surface	Simple (also gridded)
	Loss Rate	Deficit and constant (also gridded)
		Exponential
		Green and Ampt (also gridded)
		Initial and constant
		SCS curve number (also gridded)
		Smith Parlange
		Soil moisture accounting (also gridded)
	Transform	Clark unit hydrograph
		Kinematic wave
		ModClark
		SCS unit hydrograph
		Snyder unit hydrograph
		User-specified s-graph
		User-specified unit hydrograph
	Baseflow	Bounded recession
		Constant monthly
		Linear reservoir
		Nonlinear Boussinesq
		Recession
Reach	Routing	Kinematic wave
		Lag
		Modified Puls
		Muskingum
		Muskingum-Cunge
		Straddle stagger
	Gain/Loss	Constant
		Percolation

3.5.2 HEC-HMS Meteorology Modeling

The meteorological model determines the precipitation input data that sub-basin element requires. The meteorological model uses both gridded and point precipitation and the ability to simulate frozen and liquids precipitation along with evapotranspiration. Table 3.3 below briefly describes the available methods for determining grid cell precipitation or basin average precipitation.

Table 3.3 HEC-HMS Precipitation Methods

Precipitation Methods	Description
Frequency storm	Used to develop a precipitation event where depths for various durations within the storm have a consistent exceedance probability.
Gage weights	User specified weights applied to precipitation gages.
Gridded precipitation	Allows the use of gridded precipitation products, such as NEXRAD radar.
Inverse distance	Calculates subbasin average precipitation by applying an inverse distance squared weighting with gages.
HMR52	Probable maximum precipitation (PMP) using the HMR52 procedure.
SCS storm	Applies a user specified SCS time distribution to a 24-hour total storm depth.
Specified hyetograph	Applies a user defined hyetograph to a specified subbasin element.
Standard project storm	Uses a time distribution to an index precipitation depth.

CHAPTER 4 ONE AND TWO-DIMENSIONAL UNSTEADY FLOW MODELS

4.1 Introduction

Most open channel flow is unsteady (such as natural streams, drainage channels, and even storm sewers) is unsteady as the flow conditions when the hydraulic properties are varying with respect to time. These variations are significant, especially during and after a storm event. In practice, for flood studies, sometimes the steady flow equations use to determine the maximum flow depths in a stream channel, assuming the flow is steady at peak discharge. Nevertheless, this approach is conservative, since it does not account for the attenuation of flood waves due to the storage effect of the channel, on the one hand. On the other hand, the timing of the peak flows in the steady flow approach cannot be determined, while it is very important to time the flood elevations at a specific location as well as to the water surface elevation. Prediction of how a flood wave propagates in a channel precisely is possible only through the use of the unsteady open channel flow equations. Usually, referred to unsteady flow calculations in open channels as flood routing or channel routing calculations, (Akan, 2006).

Unsteady flow equations are very complicated, and for the most part, are not docile to analytical solutions. So, the numerical methods are being used for solving unsteady flow equations. Because of the enormous progress in computer technology and numerical methods that have been achieved in recent years, the use of numerical simulation methods in water resources engineering gets extreme importance. In general, it can be applied to all engineering disciplines. Numerical computations in many cases offer a cost-efficient and, therefore, a very attractive possibility for the investigation optimization of processes and research operations.

An important aspect when using numerical simulation techniques is the “proper” mathematical modeling of the processes to be investigated. If there is no following proper model, even a perfect numerical method will not yield reasonable results. Another crucial issue related to modeling is that frequently it is possible to significantly reduce the computational effort by certain simplifications in the model, (Schäfer, 2006).

The unsteady one and two-dimensional Saint-Venant equations are solved by an implicit finite difference scheme and finite volume method respectively to handle wide open channel range and river flows. The complete derivation of the Saint- Venant equations can be found in text Applied Hydrology, (V. Te Chow et al., 1988). There are many models (software) have been developed by different agencies, which are very accurate and useful to simulate river flow in both one and two-dimensional directions computations. All most, all of them solve the conservation of mass and momentum equations.

4.2 One-Dimensional Unsteady Flow

Steady and unsteady flow classification is useful in describing the flows of interest. The uniform flows simplest steady flow, in which there are no changes in hydraulics properties with respect to distance. For a uniform steady flow, every flow variable is constant on distance and time. If any modification in the flow variable occurred, then it is classified and called as nonuniform and can be further subdivided into a rapidly varied and gradually varied flow. The flow variables may change with distance, in a gradually varied steady flow, but all variables are still constant in time. The series of backwater profiles explained in details in open channel hydraulics (V. T. Chow, 1959). On the other hand, rapidly varied flow, remarkable variations happen in the vertical and transversal flow. A hydraulic jump downstream a hydraulic structures is a good example for that. This flow can still be analyzed as one-dimensional flow, but the zone that including the rapidly varied of the flow must be recognized and isolated in the analysis. The last case of unsteady uniform flow cannot exist, so the hydraulic analysis focus only on nonuniform unsteady flow, (V. T. Chow, 1959).

Due to the high development in computer processing, a huge amount of work has been done during the past few decades in developing computer models especially for use in water resources planning and management. Powerful software packages are now playing an increasingly essential role in all aspects of water resources management and providing a broad range of analysis capabilities as reported in the literature.

4.2.1 Dynamic Wave Operational Model (DWOPER)

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 model is a dynamic wave flood routing model can be used to routes an inflow hydrograph to a point downstream. It can also be used on a one or multiple river systems of 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 applied to get a solution to the St. Venant equations using a Newton- Raphson iterative technique.

DWOPER has some features (Fread, 1978) that make it applicable to a variety of natural river systems for real-time forecasting. It is designed to accommodate various boundaries conditions and irregular cross-sections at unequal distances along a single multiple-reach or many 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 the desired accuracy since the implicit finite difference techniques are not restricted to the tiny time steps of explicit technique due to numerically stability considerations. However, this can enable 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

$$\frac{\partial Q}{\partial x} + \frac{\partial (A + A_0)}{\partial t} - q = 0 \quad (4.1)$$

Momentum Equation

$$\frac{\partial Q}{\partial x} + \frac{\partial (Q^2/A)}{\partial x} + gA\left[\frac{\partial h}{\partial x} + S_f + S_e\right] - qv_x + W_f B = 0 \quad (4.2)$$

Where:

Q the discharge;

A the cross-sectional area;

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

h the water surface elevation;

q the lateral inflow or outflow;

x the distance along the channel;

t the time;

g the acceleration of gravity;

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

W_f the wind term;

B the top width of the channel;

S_f the energy grade line slope.

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

4.2.2 Dam-Break Flood Forecasting Model (DAMBRK)

Forecasting downstream flash floods due to dam failure is an application of flood routing that has gained considerable attention in recent decades. The most widely used dam-break model in the 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:

a. Temporal and geometric description of the dam breach.

b. Computation of the breach outflow hydrograph.

c. 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 Qs:

$$Q = Q_b + Q_s \tag{4.3}$$

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

$$Q_s = 3.1 B_W t_b C_v K_s \frac{(h - h_b)}{T} + 2.45 z C_v K_s (h - h_b)^{2.5} \tag{4.4}$$

Where:

t_b the time after dam breaching.

B_W the width of the breach bottom.

C_v the correction factor for the approaching velocity.

K_S the submergence correction for the tailwater effects on weir flow.

h the reservoir water surface elevation.

h_b the breach bottom elevation.

T the failure time interval.

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

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

$$Q_s = C_s L_s (h - h_s)^{1.5} + \sqrt{2g} C_g A_g (h - h_s)^{0.5} + C_d L_d (h - h_d)^{0.5} + Q_t \tag{4.5}$$

Where

C_s the uncontrolled spillway discharge coefficient.

Ls the of the uncontrolled spillway length.

h_s the uncontrolled spillway crest elevation.

C_g the gated spillway discharge coefficient.

A_g the area of the gate opening.

h_g the centerline elevation of the gated spillway.

C_d the discharge coefficient of the dam crest flow.

L_d the crest length.

h_d the dam crest elevation.

Q_t 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:

$$\frac{\partial(K_c Q)}{\partial x_c} + \frac{\partial(K_l Q)}{\partial x_l} + \frac{\partial(K_r Q)}{\partial x_r} + \frac{\partial(A_c + A_l + A_r + A_0)}{\partial t} - q = 0 \qquad (4.6)$$

And

$$\frac{\partial Q}{\partial t} + \frac{\partial(K_c^2 Q^2/A_c)}{\partial x_c} + \frac{\partial(K_l^2 Q^2/A_l)}{\partial x_l} + \frac{\partial(K_c^2 Q^2/A_c)}{\partial x_c} + gA_c\left[\frac{\partial h}{\partial x_c} + S_{fc} + S_e\right] + gA_l\left[\frac{\partial h}{\partial x_l} + S_{fl}\right] + gA_r\left[\frac{\partial h}{\partial x_r} + S_{fr}\right] = 0 \qquad (4.7)$$

The subscripts, *l*, *r*, and *c* denoted in equations 5.5 and 5.6 are the left flood plain, the right floodplain, 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 floodplain flow, and the right floodplain 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.

4.2.3 Flood Wave Routing Model (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 models. 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 human-made or natural flow controls. The expanded Saint-Venant equations, which govern the FLDWAV model (Fread, 1998) are:

Conservation of Mass Equation:

$$\frac{\partial Q}{\partial x} + \frac{\partial s_{co}(A + A_0)}{\partial t} - q = 0 \tag{4.8}$$

Momentum Equation:

$$\frac{\partial(s_m Q)}{\partial x} + \frac{\partial(\beta Q^2/A)}{\partial x} + gA\left[\frac{\partial h}{\partial x} + S_f + S_e + s_i\right] - L + W_f B = 0 \qquad (4.9)$$

Where:

Q the discharge;

A the cross-sectional area of flow;

A_0 the inactive off-channel cross-sectional area;

h the water surface elevation;

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

q the lateral inflow or outflow per lineal distance;

x the longitudinal distance along the river;

t the time;

β the momentum correction coefficient;

g the acceleration of gravity;

L the momentum effect of lateral flow;

W_f the surface wind resistance;

B the top width of the channel;

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

S_e the contraction/expansion slope;

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

The FLDWAV model includes the several capabilities which are not found in DAMBRK and as the following:

1) The flood may occur in a system of interconnected rivers like the 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.
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, floodplain 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).

4.2.4 USGS Model (FEQ)

The Full Equations (FEQ) model by the U.S. Geological Survey (USGS) for the simulation of one-dimensional unsteady flow through control structures and in open channels 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 simulated by FEQ model can be subdivided into stream reaches, some of the stream system for which complete data 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, dams, bridges, culverts, spillways, waterfalls, weirs, pumps, and side weirs. 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. An implicit finite-difference technique solves the FEQ at fixed points. The equations represented in the FEQ model are the integral form of the conservation of mass (the continuity equation) and conservation of momentum (motion equation) (Franz and Melching, 1997a and 1997b):

Conservation of Mass:

$$\int_{x_1}^{x_2} [(A)_{t2} - (A)_{t1}]\, dx = \int_{t_1}^{t_2} \left[(uA)_{x_1} - (uA)_{x_2}\right] dt \tag{4.10}$$

Conservation of Momentum:

$$\begin{aligned}\int_{x_1}^{x_2} [(uA)_{t2} - (uA)_{t1}]\, dx &= \int_{t_1}^{t_2} \left[(u^2A)_{x_1} - (u^2A)_{x_2}\right] dt \\ &+ g\int_{t_1}^{t_2} \left[(I_1)_{x_1} - (I_2)_{x_2}\right] dt \\ &+ g\int_{t_1}^{t_2}\int_{x_1}^{x_2} I_2 dxdt + g\int_{t_1}^{t_2}\int_{x_1}^{x_2} A\left(S_0 - S_f\right) dxdt\end{aligned} \tag{4.11}$$

where

u the velocity;

A the cross-sectional area of flow;

x the distance along the channel;

t the time;

g the acceleration due to gravity;

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

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

S_0 the bottom slope of the canal, positive with decline downstream;

S_f 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.

4.2.5 MIKE11

MIKE11 is software developed by DHI (Danish Hydraulic Institute) to simulate 1D flow problems. The program enables the user to simulate unsteady flow in river networks as well as looped networks using an implicit finite difference scheme. Sub-critical as well as supercritical flow conditions can be calculated. The big advantage of the model is that most of the input parameters can be variable in time (for example K-values). Furthermore, MIKE11 can simulate mass transport processes as well as sedimentation and erosion processes, (Monninkhoff, 2014).

The model applied with the fully dynamic descriptions solves the vertically integrated equations of the conservation of mass and conservation of momentum (Saint

Venant equations), which are based on the following assumptions, (Danish Hydraulic Institute, 2009):

- Incompressible flow and homogeneous.
- Very mild channel bed slope.
- Wavelengths are significant compared to water depth.
- Open channel flow regime is sub-critical.
- MIKE 11 uses the implicit 6-point Abbott scheme to solve the governing equations, (Danish Hydraulic Institute, 2009)

Conservation of Mass Equation:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = q \tag{4.12}$$

Momentum Equation:

$$\frac{\partial Q}{\partial x} + \frac{\partial(\beta Q^2/A)}{\partial x} + gA\frac{\partial h}{\partial x} + \frac{gQ|Q|}{C^2AR} = 0 \tag{4.13}$$

Where:

Q is the discharge;

A is the flow area;

x the distance along the channel;

t the time;

g the acceleration due to gravity;

q the lateral flow;

h the flow depth;

R the hydraulic radius;

C the Chezy resistance coefficient;

B 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, (Danish Hydraulic Institute, 2007).

4.3 Two-Dimensional Models

There are many models that can perform two-dimensional flow analysis and simulation. Almost, all of them are commercial models except for HEC-RAS, it is available for free for all users. Here are some of them and the way that they function:

4.3.1 TUFLOW

TUFLOW is a computer program to simulate depth-averaged, one and two-dimensional free-surface flows such as occurs from floods and tides, with the two-dimensional solutions occurring over a regular grid of square elements. TUFLOW was originally developed for modeling two-dimensional flows and stands for two-dimensional unsteady flow.

TUFLOW couples two grid based solvers; a CPU based second order semi-implicit solution often referred to as TUFLOW Classic, and a heavily parallelized first order explicit solver built for speed referred to as TUFLOW GPU.

TUFLOW also contains the full functionality of the ESTRY one-dimensional network or quasi-two-dimensional modeling system based on the full one-dimensional free-surface Saint Venant flow equations. In addition to the ESTRY one-dimensional engine, TUFLOW has also been dynamically linked (fully integrated) with the following external one-dimensional solvers: Flood Modeller (formerly known as ISIS); XP-SWMM one-dimensional solution engines; and 12D Civil Engineering Solutions 1D drainage module,(Forum et al., 2016).

TUFLOW's implicit two-dimensional solver is based on (Stelling, 1984) and is documented in (Syme, 1991). It solves the full two-dimensional, depth-averaged, momentum and continuity equations for free-surface flow using a 2nd order semi-implicit matrix solver. The solution scheme includes the viscosity or sub-grid-scale turbulence term that other mainstream software deleting. The first development was done as a joint development and research project between WBM Oceanics Australia (now BMT WBM) and The University of Queensland in 1990. The project was successfully developed a 2D/1D dynamically linked modeling system(Syme, 1991). Later improvements from 1998 up to nowadays converges on hydraulic structures, flood modeling, linking one and two and two and two-dimensional models, and using GIS for data management (Syme, 2001). TUFLOW has also been submitted to extensive extermination and validation by WBM Pty Ltd and others (Barton, 2001; Huxley & Syme, 2004; Neelz, S. & Pender, 2013). TUFLOW is particularly orientated towards establishing flow and inundation patterns in floodplains, coastal waters, estuaries, rivers and urban regions where the flow behavior is essentially two-dimensional in nature and cannot or would be awkward to represent using a one-dimensional model.

A good feature of TUFLOW is its capaility to dynamically link to one-dimensional networks using the hydrodynamic solutions of ESTRY, Flood Modeler, XP-SWMM, and 12D. The users prepare a model as a combination of one-dimensional network domains linked to two-dimensional domains. As such, the two-dimensional and one-dimensional domains are linked to form one overall model.

4.3.2 Two-Dimensional Schematization of TUFLOW

TUFLOW topography in a two-dimensional domain is defined by elevations at the cell centers, corners, and mid-side. Every cell has the following elevations assigned to it, as shown in Figure 4.1:

- "C" (ZC) – middle of cell
- "U" (ZU) – middle right of cell
- "V" (ZV) – middle top of cell
- "H" (ZH) – top right-hand corner of cell

One of the very important aspects of TUFLOW modeling is to understand the roles of the elevation (Zpt) points.

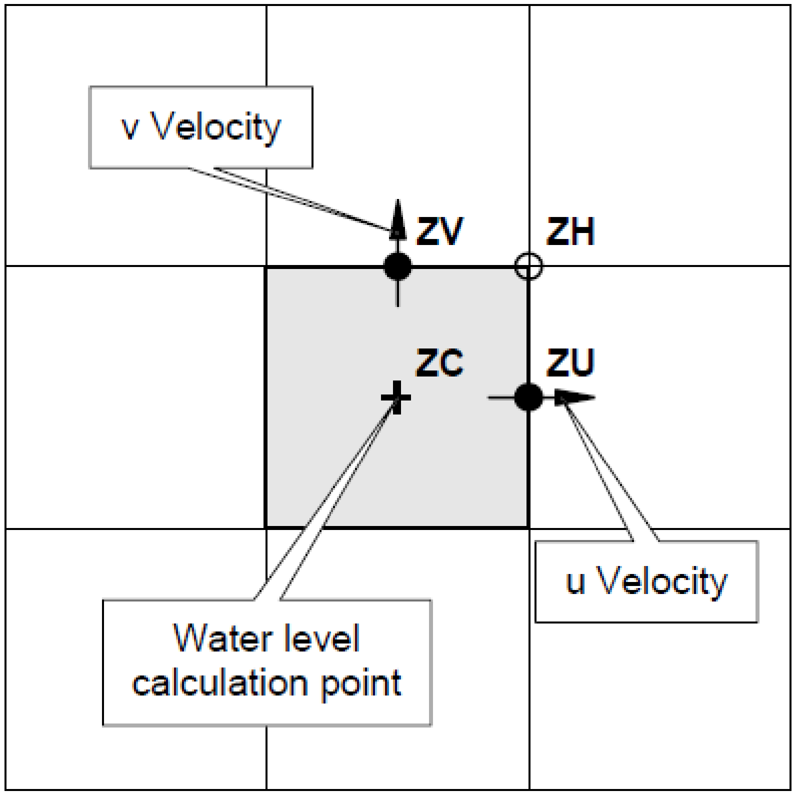


Figure 4.1 Location of Computation Points and Zpts

ZC point:

- Defines the volume of water (cell volume is based on a flat square cell that wets and dries at the height of ZC in addition to the Cell Wet/Dry Depth);
- Controls when a cell becomes wet and dry (note that cell sides can also wet and dry); and
- Determines the slope of the bed when testing for the upstream controlled flow regime

The ZU and ZV points:

- Control how water is conveyed from one cell to another;

- Represent where the momentum equation terms are centered and where upstream controlled flow regimes are applied;
- Deactivate if the cell has dried (based on the ZC point) and cannot flow; and
- Dry and wet independently of the cell wetting or drying. This allows for the modeling of "thin" obstructions such as fences and thin embankments about the cell size (e.g., a concrete levee).

4.3.3 TUFLOW Solving Scheme

TUFLOW solves the depth-averaged 2D shallow water equations. The shallow water equations are the equations of fluid motion used for modeling long waves like floods, ocean tides, and storm surges. They are derived using the hypotheses of vertically uniform horizontal velocity and negligible vertical acceleration (i.e., a hydrostatic pressure distribution). These assumptions are valid where the wavelength is greater than the depth of water. In the case of the ocean tide, the SWE are applicable everywhere.

The two-dimensional SWE in the horizontal plane is described by the partial differential equations of mass continuity and momentum conservation in the X and Y directions for an in-plan Cartesian coordinate frame of reference as in the following. The equations are:

Two-Dimensional Conservation of Mass:

$$\frac{\partial \zeta}{\partial t} + \frac{\partial (Hu)}{\partial x} + \frac{\partial (Hv)}{\partial y} = 0 \tag{4.14}$$

X-Direction Momentum:

$$\frac{\partial u}{\partial t}+u\frac{\partial u}{\partial x}+v\frac{\partial u}{\partial y}-c_f v+g\frac{\partial \zeta}{\partial x}+gu\left(\frac{n^2}{H^{\frac{4}{3}}}+\frac{f_1}{2g\Delta x}\right)\sqrt{u^2+v^2}$$

$$-\mu\left(\frac{\partial^2 u}{\partial x^2}+\frac{\partial^2 u}{\partial y^2}\right)+\frac{1}{\rho}\frac{\partial p}{\partial x}=F_x \quad (4.15)$$

Y-Direction Momentum:

$$\frac{\partial u}{\partial t}+u\frac{\partial u}{\partial x}+v\frac{\partial u}{\partial y}-c_f u+g\frac{\partial \zeta}{\partial y}+gv\left(\frac{n^2}{H^{\frac{4}{3}}}+\frac{f_1}{2g\Delta y}\right)\sqrt{u^2+v^2}$$

$$-\mu\left(\frac{\partial^2 v}{\partial x^2}+\frac{\partial^2 v}{\partial y^2}\right)+\frac{1}{\rho}\frac{\partial p}{\partial y}=F_y \quad (4.16)$$

Where:

ζ	Water suface elevation;
u and v	Depeth avrage velocity components in x and y directions;
H	Depth of water
T	Time
x and y	Distance in X and Y directions
Δx and Δy	Cell dimensiona in x and y directions
C_f	Coriolis force Cefficient
n	Manning's n
f_1	Form energy loss coefficient
μ	Horizontal diffusion of momentum coeffient
p	Atmospheric pressure

ρ Density of Water

F_x *and* F_y Sum of comoponents of external forces (wind) in X and Y directions.

4.3.4 Flood Modeler CH2M

Flood Modeler program is a commercial flexible and comprehensive package of tools for deriving flood maps, flood forecasting, designing flood management schemes, developing catchment strategies and many other flood and non-flood applications including modeling low flows, sediment, and water quality.

Flood Modeller can also be used to solve systems under both unsteady and steady flow routing. Steady flow solutions are discussed further in the Steady Flows topic. For unsteady solutions, Flood Modeller uses the governing hydraulic equations for each unit. These equations are inevitably a combination of empirical and theoretical equations many of which are non-linear. The non-linear equations are first linearized, and the solution of the linear version of the problem is then found via matrix inversion. An iterative procedure is used to account for the non-linearities.

Flood Modeller two-dimensional solver and many other models of its type represent shallow water hydraulics. The shallow water equations can describe the following:

Two-Dimensional Conservation of Mass:

$$\frac{\partial h}{\partial t} + \frac{\partial hu}{\partial x} + \frac{\partial hv}{\partial y} = 0 \qquad (4.17)$$

X-Direction Momentum:

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} + g\frac{\partial(h+z)}{\partial x} + \frac{gn^2u\sqrt{u^2+v^2}}{h^{4/3}}$$
$$- \frac{v}{h}\left(2\frac{\partial^2 hu}{\partial x^2} + \frac{\partial^2 hu}{\partial x^2} + \frac{\partial^2 hv}{\partial x \partial y}\right) = 0 \tag{4.18}$$

Y-Direction Momentum:

$$\frac{\partial v}{\partial t} + u\frac{\partial v}{\partial x} + v\frac{\partial v}{\partial y} + g\frac{\partial(h+z)}{\partial y} + \frac{gn^2v\sqrt{u^2+v^2}}{h^{4/3}}$$
$$- \frac{v}{h}\left(\frac{\partial^2 hv}{\partial x^2} + 2\frac{\partial^2 hv}{\partial y^2} + \frac{\partial^2 hu}{\partial x \partial y}\right) \ldots . = 0 \tag{4.19}$$

Where:

u and v two components of the horizontal velocity;

h flow depth;

z bed elevation;

x and y horizontal distances in the x and y directions;

t time;

g acceleration due to gravity;

n Manning's coefficient of roughness;

The kinematic eddy viscosity used to parameterize horizontal turbulent momentum transport, with a value between 0 and 1. The default value in the interface for this parameter is zero, meaning that viscosity is excluded from the Shallow Water equations. This can improve model run times, but a better solution would be gained using a value of 0.15 or higher (can be much higher for open sea).

The movement of water in terms of a depth-averaged two-dimensional velocity and the water depth are described by that equations, in response to the forces of gravity and friction. These equations typically represent situations where the flow is approximately horizontal, is uniform with depth, and where vertical accelerations are small. The velocity components represent the water velocity averaged:

- Over the depth of the water column, ignoring variations in flow direction and magnitude with the depth
- Over time, ignoring short-term turbulent velocity variations

One important property of shallow water flows is the different behaviors of subcritical and supercritical flows. Supercritical flows tend to develop jumps (sudden changes in velocity and water level), which are difficult to represent in the model without causing any instability.

Flood modeler uses the Preissmann four-point implicit finite difference scheme for solving the channel equations and sparse matrix solver to invert the matrix.

4.3.5 MIKE 21

MIKE 21 Flow Model FM is a new modeling system based on a flexible mesh approach. The modeling system has been developed for applications within oceanographic, coastal and estuarine environments. MIKE 21 Flow Model FM is composed of following modules: Hydrodynamic Module, Transport Module, ECO Lab/Oil Spill Module, Particle Tracking Module, Mud Transport Module, and Sand Transport Module. The Hydrodynamic Module is the basic computational component of the entire MIKE 21 Flow Model FM modeling system providing the hydrodynamic basis for the other modules.

The Hydrodynamic Module is based on the numerical solution of the two-dimensional shallow water equations - the depth-integrated incompressible Reynolds-averaged Navier-Stokes equations. However, the model consists of continuity, momentum, temperature, salinity and density equations. In the horizontal domain, both Cartesian and spherical coordinates can be used. Below the governing equations are presented using Cartesian coordinates, (Danish Hydraulic Institute, 2013).

Mass Conservation:

$$\frac{\partial u}{\partial x}+\frac{\partial v}{\partial x}+\frac{\partial w}{\partial z}=S \tag{4.20}$$

X-Direction Momentum:

$$\begin{aligned}&\frac{\partial u}{\partial t}+\frac{\partial u^2}{\partial x}+\frac{\partial vu}{\partial y}+\frac{\partial wu}{\partial z}\\&=fv-g\frac{\partial \eta}{\partial x}-\frac{1}{\rho_0}\frac{\partial p_a}{\partial x}-\frac{g}{\rho_0}\int_z^\eta\frac{\partial \rho}{\partial x}dz+F_u\\&+\frac{\partial}{\partial z}\left(v_t\frac{\partial u}{\partial z}\right)+u_sS\end{aligned} \tag{4.21}$$

Y-Direction Momentum:

$$\begin{aligned}&\frac{\partial v}{\partial t}+\frac{\partial v^2}{\partial x}+\frac{\partial vu}{\partial y}+\frac{\partial wu}{\partial z}\\&=fu-g\frac{\partial \eta}{\partial y}-\frac{1}{\rho_0}\frac{\partial p_a}{\partial y}-\frac{g}{\rho_0}\int_z^\eta\frac{\partial \rho}{\partial y}dz+F_v\\&+\frac{\partial}{\partial z}\left(v_t\frac{\partial v}{\partial z}\right)+u_sS\end{aligned} \tag{4.22}$$

Where:

t time

x, y, z Cartesian coordinates

u, v, w flow velocity components

The spatial discretization of the primitive equations is performed using a cell-centered finite volume method. The spatial domain is discretized by subdivision of the continuum into non-overlapping element/cells. In the horizontal plane, an unstructured grid is used comprising of triangles or quadrilateral element. An approximate Riemann solver is used for computation of the convective fluxes, which makes it possible to handle discontinuous solutions. For the time integration an explicit scheme is used, (Danish Hydraulic Institute, 2011).

4.4 Hydrologic Engineering Center's River Analysis System (HEC-RAS)

HEC-RAS is an integrated system of software, programmed for interactive use in a multi-tasking environment. The HEC-RAS system contains the following river analysis components for (1) steady flow water surface profile computations; (2) one-dimensional and two-dimensional unsteady flow simulation; (3) Quasi unsteady or fully unsteady flow movable boundary sediment transport computations; and (4) water quality analysis. A key element is that all four components use a common geometric data representation and common geometric and hydraulic computation routines. Besides the four river analysis components, the system contains several hydraulic design features that can be invoked once the water surface profiles are computed. HEC-RAS is designed to perform one-dimensional and two-dimensional hydraulic calculations for a full network of natural and constructed channels, overbank/floodplain areas, levee-protected areas, (U.S. Army Corps of Engineers, 2016a).

HEC has introduced the ability to perform two-dimensional (2D) hydrodynamic flow routing within the unsteady flow analysis portion of HEC-RAS. Users can now perform one-dimensional (1D) unsteady flow modeling, two-dimensional (2D) unsteady flow modeling (Full Saint Venant equations or Diffusion Wave equations), as well as combined one-dimensional and two-dimensional (1D/2D) Unsteady-Flow routing. The two-dimensional flow areas in HEC-RAS can be used in some ways. The following are examples of how the 2D Flow Areas can be utilized to support modeling with HEC-RAS, (G. W. Brunner, 2014):

- Detailed 2D channel modeling
- Detailed 2D channel and floodplain modeling
- Combined 1D channels with 2D floodplain areas
- Combined 1D channels with 2D Flow Areas behind levees
- Directly connect 1D reaches into and out of 2D Flow Areas.
- Directly connect a 2D Flow Area to 1D Storage Area with a hydraulic structure
- Multiple 2D Flow Areas in the same geometry
- Directly connect multiple 2D Flow Areas with hydraulic structures
- Simplified to very detailed Dam Breach analyses
- Simplified to very detailed Levee Breaching analyses

- ❖ Mixed flow regime. The 2D capability (as well as the 1D) can handle the supercritical and subcritical flow, as well as the flow transitions from subcritical to supercritical and supercritical to subcritical (hydraulic jumps).

Two-dimensional flow modeling is accomplished by adding two-dimensional flow area elements into the model in the same manner as adding a storage area. A two-dimensional flow area can be added by drawing a two-dimensional flow area polygon; developing the 2D computational mesh; then linking the two-dimensional flow areas to one-dimensional model elements and directly connecting boundary conditions to the 2D areas. In this research, HER-RAS represents the second major components of the optimization simulation model to simulate flow routing in downstream two-dimensional area.

4.4.1 Routing Inflow Flood Throughout the Reservoir

HEC-RAS can be used to route inflow flood hydrograph through a reservoir by using one of the following methods:

- ❖ unsteady flow routing in one-dimensional using Saint Venant equations
- ❖ unsteady flow routing two-dimensional using Diffusion wave equations or Saint Venant equations.
- ❖ Routing level reservoir pool.

The unsteady flow routing one or two-dimensional will be more accurate for both the with and without breach process. That method can capture the water surface slope through the reservoir pool as the inflowing hydrograph arrives, the change in water surface slope that occurs during a dam breach too. However, reservoirs with long narrow pools will exhibit greater water surface slope upstream of the dam than reservoirs that are short and wide. Thus, the most accurate modeling technique to capture pool elevations and outflows of long narrow reservoirs is full dynamic wave (unsteady flow) routing. For wide and short reservoirs, level pool routing may be appropriate.

4.4.2 Two-Dimensional Unsteady Flow Hydrodynamic

The movements of fluid in three-dimensional can be described by the Navier-Stokes equations. In the context of the flood and channel modeling are imposed. A simplified set of equations is the Shallow Water (SW) equations. The assumption if incompressible flow, hydrostatic pressure, and uniform density are assumed, and the equations are Reynolds averaged so that turbulent motion is approximated using eddy viscosity. Likewise, the vertical length scale is assumed much smaller than the horizontal length scales. Consequently, the vertical velocity is relatively small and pressure is hydrostatic, leading to the differential form of the SW equations.

Furthermore, to improve computation time, a sub-grid bathymetry approach can be used. The reason behind this method is to use a relatively coarse computational grid and finer scale information about the underlying topography (Casulli, 2008). The conservation of mass equation is discretized using a finite volume technique. The fine grid details are factored out as parameters representing multiple integrals over volumes and face areas. As a result, the transport of fluid mass accounts for the fine-scale topography inside of each discrete cell. Since this idea relates only to the mass equation, it can be used independently of the version of the equation of momentum.

The bottom surface elevation is given by z(x,y); the water depth is h(x,y,t); and the water surface elevation as in Figure 4.2 below.

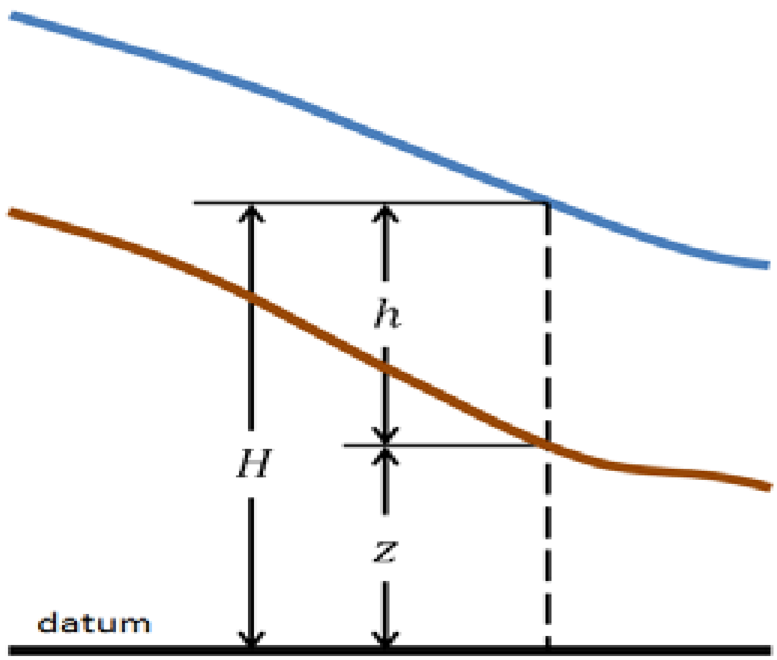


Figure 4.2 Datum of Surface Elevation

So,

$$H(x,y,t) = z(x,y) + h(x,y,t) \tag{4.23}$$

Mass Conservation:

$$\frac{\partial H}{\partial t} + \frac{\partial (hu)}{\partial x} + \frac{\partial (hv)}{\partial y} + q = 0 \tag{4.24}$$

Where:

t time

u and v the velocity components in the x- and y- direction respectively

q a source/sink flux term.

In vector form, the continuity equation takes the form:

$$\frac{\partial H}{\partial t} + \nabla \cdot hV + q = 0 \tag{4.25}$$

Where:

V=(u,v) is the velocity vector, and the differential operator del (∇) is the vector of the partial derivative operators given by $\nabla = (\partial/\partial x, \partial/\partial y)$.

Integrating over a horizontal region with boundary normal vector *n* and using Gauss' Divergence theorem, the integral form of the equation is obtained:

$$\frac{\partial}{\partial t}\iiint_{\Omega} d\Omega + \iint_{S} V \cdot n dS + Q = 0 \tag{4.26}$$

The volumetric region Ω represents the three-dimensional space occupied by the fluid. The side boundaries are given by S. It is assumed that Q represents any flow that crosses the bottom surface (infiltration) or the top water surface of Ω (evaporation or rain). The source/sink flow term Q is also convenient to represent other conditions that transfer mass into, within or out of the system, such as pumps. Following the standard sign conventions, sinks are positive, and sources are negative, (U.S. Army Corps of Engineers, 2016a).

HEC-RAS uses the subgrid bathymetry approach to solve such a problem. The computational grid cells have some extra data such as cross-sectional area, volume, and even the hydraulic radius volume and cross-sectional area that can be pre-computed from the fine bathymetry.

4.4.2.1 Momentum Conservation

When the horizontal length scales are much larger than the vertical length scale, volume conservation implies that the vertical velocity is small. The Navier-Stokes vertical momentum equation can be used to justify that pressure is nearly hydrostatic. In the absence of baroclinic pressure gradients (variable density), strong wind forcing and non-hydrostatic pressure, a vertically-averaged version of the momentum equation is adequate. Vertical velocity and vertical derivative terms can be safely neglected (in both mass and momentum equations). The shallow water equations are obtained,(U.S. Army Corps of Engineers, 2016a).

X-Direction Momentum:

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} = -g\frac{\partial H}{\partial x} + v_t\left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2}\right) - c_f u + fv \qquad (4.27)$$

Y-Direction Momentum:

$$\frac{\partial v}{\partial t} + u\frac{\partial v}{\partial x} + v\frac{\partial v}{\partial y} = -g\frac{\partial H}{\partial x} + v_t\left(\frac{\partial^2 v}{\partial y^2} + \frac{\partial^2 v}{\partial y^2}\right) - c_f u + fv \qquad (4.28)$$

Where:

u and v	the velocities in the Cartesian directions,
g	the gravitational acceleration,
v_t	horizontal eddy viscosity coefficient,
c_f	the bottom friction coefficient,
R	the hydraulic radius and,
f	the Coriolis parameter.

The left-hand side of the equation contains the acceleration terms. The right-hand side represents the internal or external forces acting on the fluid. The left and right-hand side term are typically organized in such a way in accordance with Newton's second law, from which the momentum equations are ultimately derived. The momentum equations can also be rendered as a single differential vector form. The advantage of this presentation of the equation is that it becomes more compact and easily readable. The vector form of the momentum equation is:

$$\frac{\partial V}{\partial t} + V \cdot \nabla V = -g\nabla \mathrm{H} + v_t \nabla^2 V - c_f V + fk \times V \tag{4.29}$$

Where the differential operator del (∇) is the vector of the partial derivative operators given by $\nabla=(\partial/\partial \mathrm{x}, \partial/\partial \mathrm{y})$ and k is the unit vector in the vertical direction.

4.5 Controlling HEC-RAS Through MATLAB

The interfacing between the various simulation programs used in the optimization-simulation model is one of the objectives of this research. the interfacing and data exchange between HEC-RAS and MATLAB in a challenging task. The users of HEC-RAS often have unique applications for using this software that may include the coupling with other software to perform systems analysis such as flood risk analysis, optimization of flooding structures under uncertainty and multi-objective reservoir operation under uncertainty, multi-objective reservoir operation under uncertainty, among others (Leon & Goodell, 2016). One state-of-the-art environment for integrating proprietary software and/or open source codes is MATLAB. MATLAB is a high-performance language for technical computing. It integrates computation, visualization, and programming in an easy-to-use environment, (Mathworks, 2016).

CHAPTER 5 COMPONENTS OF MODELING APPROACH

The various components of the optimization-simulation model developed herein include the rainfall-runoff model, the unsteady flow routing models, the rainfall forecasting model, the reservoir operation model and the optimization model. The interfacing of these components was first introduced in Figure 1.9. In chapter 3 the rainfall-runoff model has been discussed in detail and in chapter 4 the unsteady flow models have been discussed. This chapter discusses the rainfall forecasting model, the reservoir operation model and the optimization model.

5.1 Rainfall Forecasting Model

Forecasting natural phenomena is not an easy task and is a challenging subject, particularly the forecasting of information about water availability such as rainfall forecasting, which is very useful for real-time modeling of flooding events. This research proposes a methodology to design a system that can provide a sequence of rainfall data that represents the future scenario of predicted rainfall.

Rainfall forecasting is another necessary component in the optimization - simulation model. Forecasted precipitation is needed for flood forecasting since reservoir management personnel would have to make reservoir release~~s~~ decisions based upon the forecasted information prior to the actual rainfall event and floodwater arrival. In this research, a statistical regression analysis approach is used for the rainfall forecasting model.

5.1.1 Forecasting Approach

The rainfall forecasting approach used herein was adopted from (Che, 2015) which based on~~,~~ (Montgomery et al., 2012) for forecasting rainfall in time period t+Δt, knowing the actual rainfall in the current time period t is expressed as:

$$\hat{P}_{t+\Delta t} = \hat{\Phi} P_t + (1 - \hat{\Phi})\hat{\beta}_0 + \hat{\beta}_1\left[(t + \Delta t) - \hat{\Phi} t\right] \tag{5.1}$$

Where:

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

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

t is the current time period.

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

Φ is an autocorrelation parameter defined as:

$$\Phi = \sum_{t=2}^{t} \frac{e_t e_{t-1}}{\sum_{t=1}^{t} e_t^2} \tag{5.2}$$

e_t is the vector of residuals from the prediction.

$\hat{\beta}_0, \hat{\beta}_1$ are model parameter.

The general procedure of rainfall forecasting model is illustrated in Figure 5.1 where P_t in this model is the known rainfall at time t. First, the model obtains the actual rainfall up to current time~~,~~ *t*. Then the rain data is ~~entered into the step that~~ used in the prediction model equation 5.1 to generate rainfall over time period t+*Δt.* Once the prediction model of time *t+Δt* 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 the optimization/simulation model. 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 rainfall prediction will be generated for each simulation period. The process repeats until the very last simulation period when forecasting is no longer needed.

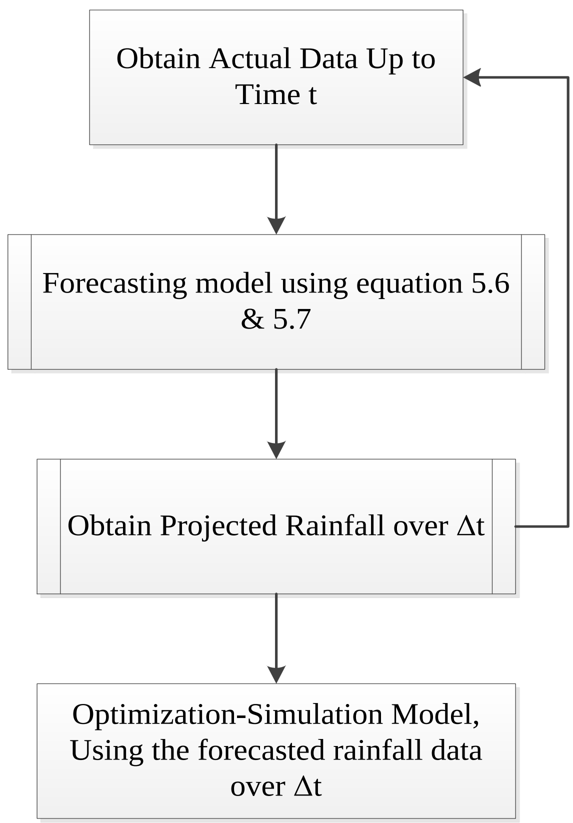


Figure 5.1 General Procedure of Rainfall Forecasting Model.

5.1.2 Suggested Rainfall Forecasting Methods

The optimization simulation model needs an accurate time series rainfall data so that can continue produce a reliable output for making the final decision. Four time series forecasting models are suggested by Che & Mays (2015) for their optimization/simulation model 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 Figure 5.2 is used for the comparison of the four times series forecasting models. The time span is set to be 72 hours, and the forecasting starts at t equal to 7 hours.

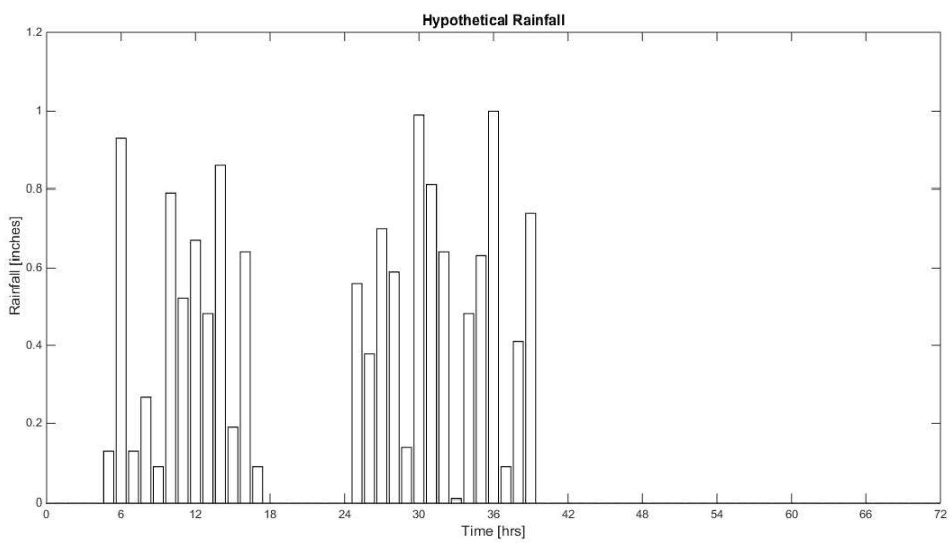


Figure 5.2 Hypothetical Rainfall

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

$$P_{t+\Delta t} + a_1 P_t + a_2 P_{t-\Delta t} + a_3 P_{t-2\Delta t} \ldots + a_N P_{t-(N-1)\Delta t} = e_{t+\Delta t} \qquad (5.3)$$

Where:

$P_{t+\Delta t}$ is the forecasted rainfall values over the time ($t + \Delta t$).

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

t is the current time period.

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

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

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

The advantage~~s~~ of the autoregressive models are flexibility in handling a wide range of different time series patterns. Figure 5.3 depicts the forecasting result based on the hypothetical rainfall hyetograph.

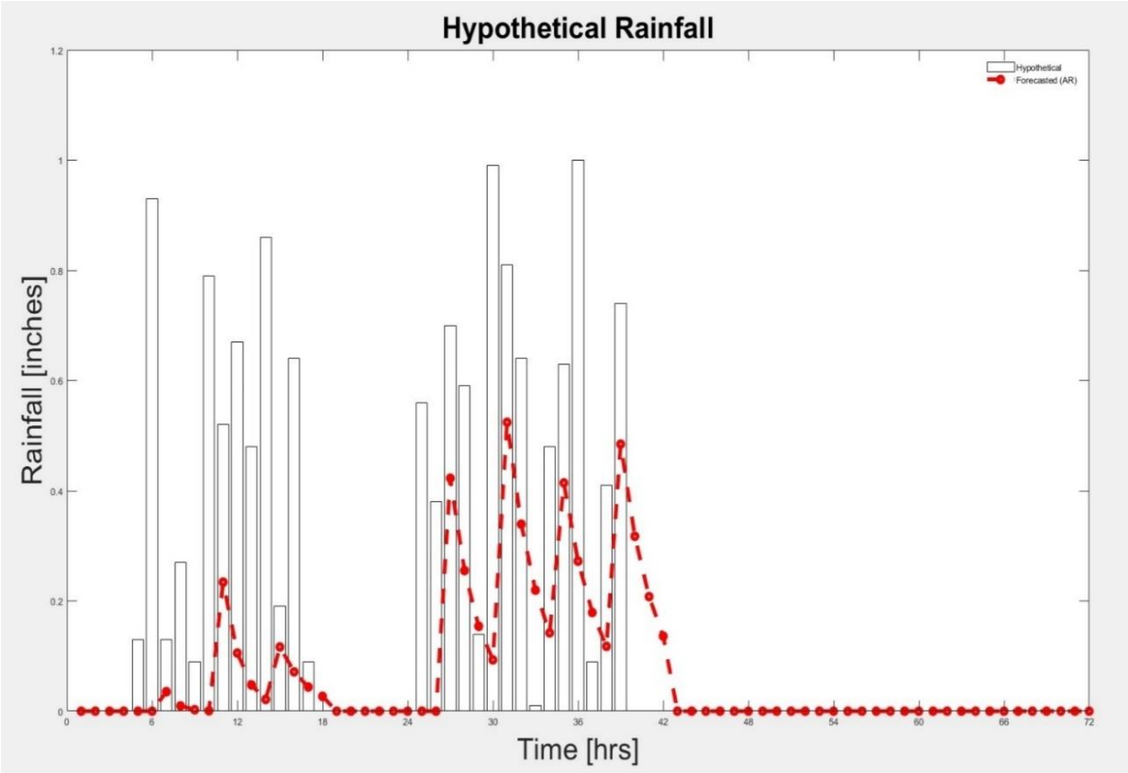


Figure 5.3 Forecasting Result of the AR Model, Δt = 4 hours

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

$$\begin{aligned} P_{t+\Delta t} + a_1 P_t + a_2 P_{t-\Delta t} + a_3 P_{t-2\Delta t} \dots + a_N P_{t-(N-1)\Delta t} \\ = b_1 u_t + b_2 u_{t-\Delta t} + b_3 u_{t-2\Delta t} \dots + b_N u_{t-(N-1)\Delta t} + e_{t+\Delta t} \end{aligned} \tag{5.4}$$

Where:

$P_{t+\Delta t}$ is the forecasted rainfall values over the time $(t + \Delta t)$.

u_t is the exogenous values at the end of 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 generated rainfall is purely hypothetical, the exogenous variable used here is the cumulative rainfall up to time t. Figure 5.4 depicts the forecasting result based on the hypothetical rainfall hyetograph.

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 & Li, 2006). The basic formulation of an ARMAX model is as follows:

$$\begin{aligned} P_{t+\Delta t} + a_1 P_t + a_2 P_{t-\Delta t} + a_3 P_{t-2\Delta t} \ldots + a_N P_{t-(N-1)\Delta t} \\ = b_1 u_t + b_2 u_{t-\Delta t} + b_3 u_{t-2\Delta t} \ldots + b_N u_{t-(N-1)\Delta t} + c_1 e_t \\ + c_2 e_{t-\Delta t} + c_3 e_{t-2\Delta t} \ldots + c_N e_{t-(N-1)\Delta t} + e_{t+\Delta t} \end{aligned} \tag{5.5}$$

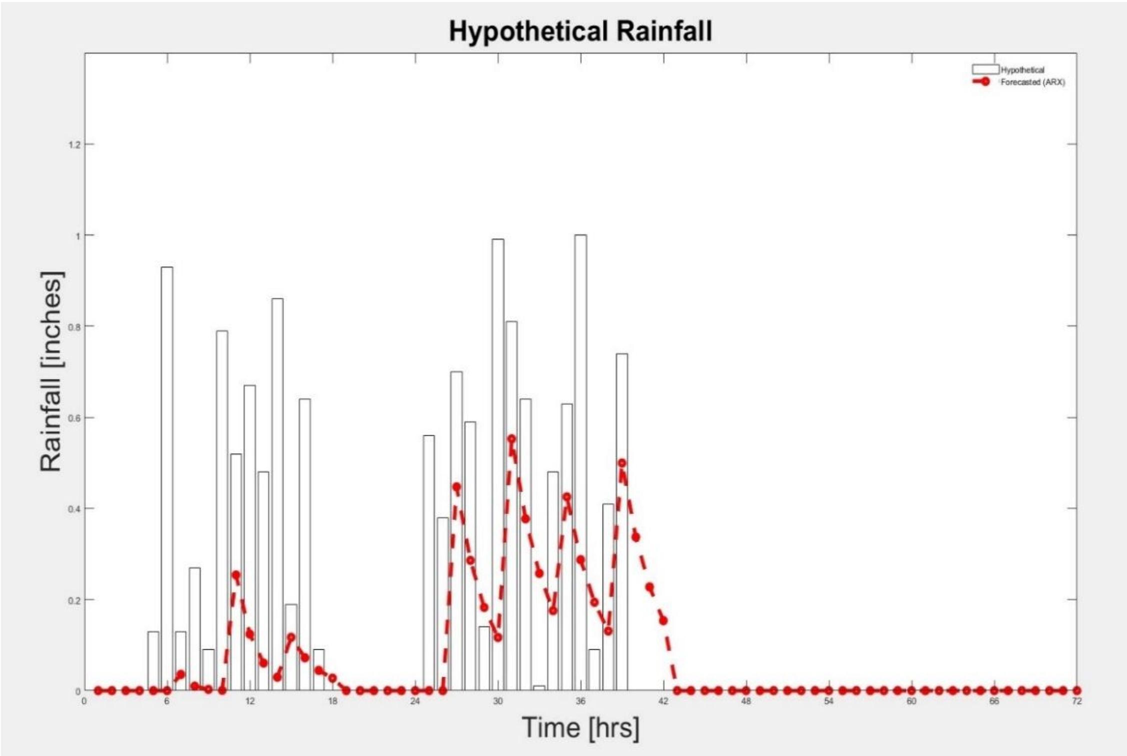


Figure 5.4 Forecasting Result of the ARX Model, Δt = 4 hours

Where:

$P_{t+\Delta t}$ is the forecasted rainfall values over the time $(t + \Delta t)$.

u_t is the exogenous values at the end of 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.

Figure 5.5 depicts the forecasting result based on the hypothetical rainfall hyetograph.

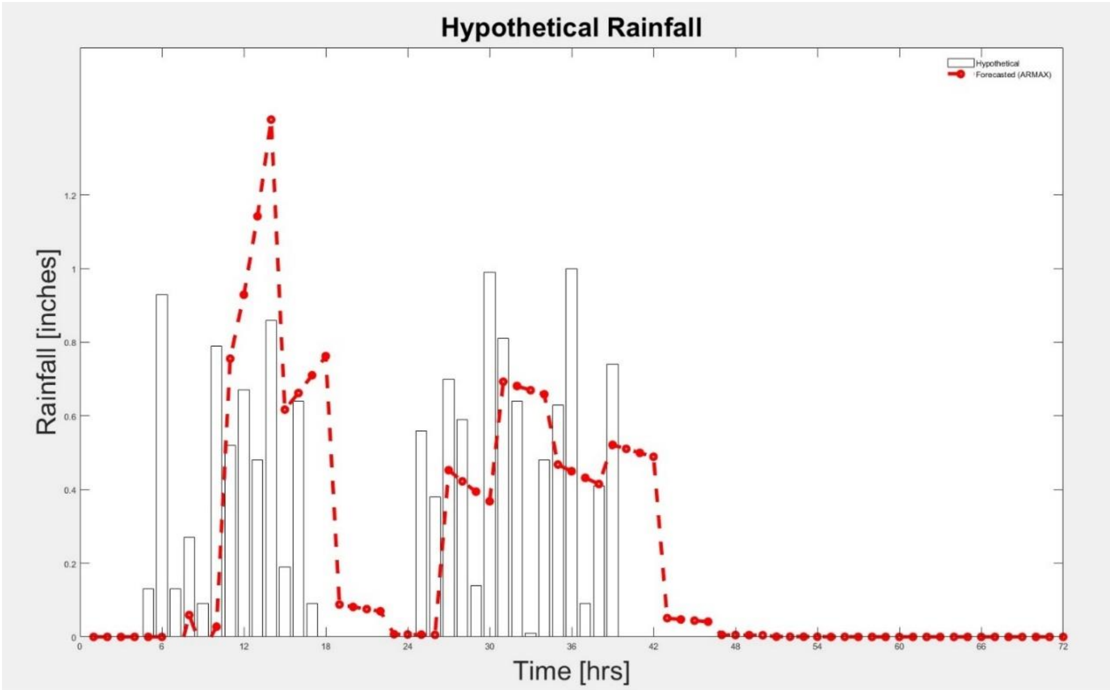


Figure 5.5 Forecasting Results of the ARMAX Model, Δt = 4 hours

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:

$$\frac{dx}{dt} = Ax_t + Bu_t + Ke_t \tag{5.6}$$

$$P_t = Cx_t + Du_t + e_t \tag{5.7}$$

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 are the model parameters.

A, B, C, K are the model parameters.

Figure 5.6 depicts the forecasting result based on the hypothetical rainfall hyetograph.

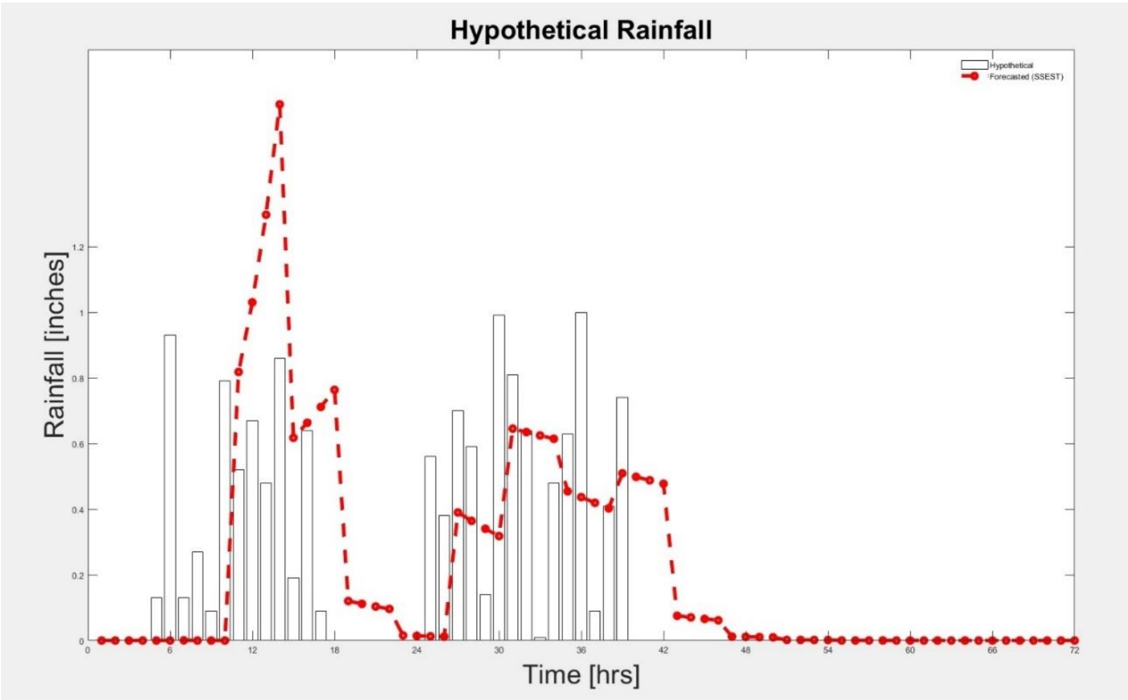


Figure 5.6 Forecasting Result of the SSEST Model, Δt = 4 hours

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:

$$CEF = \frac{\sum_{\tau} P_t - \sum_{\tau} \hat{P}_t}{\sum_{\tau} P_t}, \forall t \in \tau \tag{5.8}$$

Where:

if CFE $\begin{cases} > 0, Under\ forecast \\ < 0, over\ forecast \end{cases}$

P_t is the hypothetical rainfall up to time t.

$\hat{P}_t$ is the forecasted rainfall upto time t.

τ is the 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:

$$RMSE = \sqrt{\frac{\sum_{\tau}(P_t - \hat{P}_t)^2}{n}}, \forall t \in \tau \tag{5.9}$$

where

P_t is the hypothetical rainfall up to time t.

$\hat{P}_t$ is the forecasted rainfall up to time t.

τ is the 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 the forecasted rainfall, thus resulting unwanted uncertainties in the rest of the optimization model.

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

Table 5.1 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 similar; therefore the four models each produce a similar quality of forecasting. There are significant differences in the CFEs for the four models. The autoregressive (AR) model and the autoregressive exogenous (ARX) model both have large CFE values, which indicates that both methods are ~~way~~ under forecasting as compared to the actual (hypothetical) rainfall. 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, but only over forecast 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 forecast produced by the AR and the ARX models, the ARMAX and the SSEST models ~~is~~ were then compared to each other. The ARMAX takes significantly less time than the SSEST for this hypothetical rainfall example; thus the ARMAX is the most desired method of all four methods. The optimization - simulation model, would use the ARMAX approach for its rainfall forecasting component due to its quality of forecast and lesser computational time required. The rainfall forecasting component of the optimization-simulation model incorporates an updating procedure for projected rainfall in real time similar to the procedure such as presented by (Madsen & Skotner, 2005), as well as the discussion in Section 6.4.

5.2 Reservoir Operation Model

The principle of any reservoir operation, whether in real-time or for planning purposes, is governed by is the conservation of mass (the continuity equation). Reservoir operation in real time is a process of continuously determining the releases through the reservoir gates to keep the water surface elevations at the upstream and downstream within the desired levels. Strategic planning defines the operation rules. While the reservoir operation in some parts of it based upon forecasting information, so the process not without an error. Uncertainty and inaccuracy are unavoidable in real-time reservoir operation and even in the planning processes, that come from the forecasting and then determining the net inflow into the reservoir and that is the source of uncertainty and inaccuracy.

The reservoir operation model governed by the conservation of mass (the continuity equation). Conservation of mass for a control volume Figure 5.7 states that the net rate of flow into the volume be equal to the rate of change of storage inside the volume. The rate of inflow into the control volume is:

$$Q - \frac{\partial Q}{\partial x}\frac{\Delta x}{2}$$

The outflow rate is:

$$Q + \frac{\partial Q}{\partial x}\frac{\Delta x}{2}$$

The rate of storage changes is:

$$\frac{\partial A_T}{\partial t}\Delta x$$

Assuming that Δx is small, the change in mass in the control volume is equal to:

$$\frac{\partial A_T}{\partial t}\Delta x = \rho\left[\left(Q - \frac{\partial Q}{\partial x}\frac{\Delta x}{2}\right) - \left(Q + \frac{\partial Q}{\partial x}\frac{\Delta x}{2}\right)\right] + Q_1 \qquad (5.10)$$

Where Q_1 is the lateral flow entering the control volume and ρ is the fluid density, simplifying and dividing through by $\rho \Delta x$ yields the final form of the continuity equation:

$$\frac{\partial A_T}{\partial t} + \frac{\partial Q}{\partial x} - q_1 = 0 \tag{5.11}$$

In which A_T is the area of the control surface and q_1 is the lateral inflow per unit length.

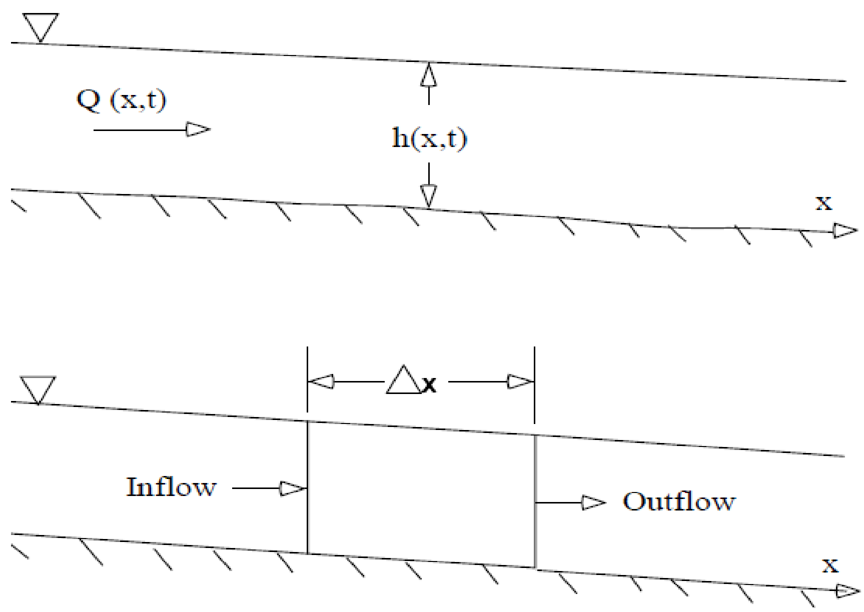


Figure 5.7 Elementary Control Volume for Derivation of Continuity and Momentum Equations

The reservoir operation model is coupled with an optimization model to determine the flow rate that should be released from the reservoir over the Δt time. The importance of the optimization model is to compute the optimal decision for the releases schedule from the reservoir to keep the downstream water surface elevation under the flood level. The optimization technique used is the genetic algorithm into MATLAB interfaced with other model components that provides the best solution for the aim of the problem.

5.2.1 Principle of Reservoir Operation Model

The principle that governs any reservoir operation is the conservation of mass (continuity) expressed mathematically for a control volume (as shown in Figure 5.8).

$$\frac{d(m_{cv})}{d_t} + \sum_{cs} m_{out} - \sum_{cs} m_{in} = 0 \tag{5.12}$$

Where:

$d(m_{cv})$	the accumulation of mass in the control volume.
$\sum_{cs} m_{out}$	the total mass inflow through the control surface.
$\sum_{cs} m_{in}$	the total mass outflow through the control surface.

Equation (5.12) states that the accumulation rate of mass in the control volume plus the net rate of the outflow of mass through the control surface is equal to zero.

Instead of using the flow of mass rate, equation (5.12) can be written in terms of inflow volume by dividing all terms by the density of the water ρ, as the in following:

$$\frac{1}{\rho}\frac{d(m_{cv})}{d_t} + \frac{1}{\rho}\sum_{cs} m_{out} - \frac{1}{\rho}\sum_{cs} m_{in} = 0 \tag{5.13}$$

In other words, equation 5.13 is simplified to

$$\frac{d(V_{cv})}{d_t} + \sum_{cs} Q_{out} - \sum_{cs} Q_{in} = 0 \tag{5.14}$$

Where:

$\frac{d(V_{cv})}{d_t}$	the volume change in the control volume.

$\sum_{cs} Q_{out}$ the total volumetric in flow through the control surface.

$\sum_{cs} Q_{in}$ the total volumetric outflow through the control surface.

Equation (5.14) is the basis of gated spillway model. The schematic reservoir with components of flows and the storage of the reservoir is shown in Figure 5.8 below.

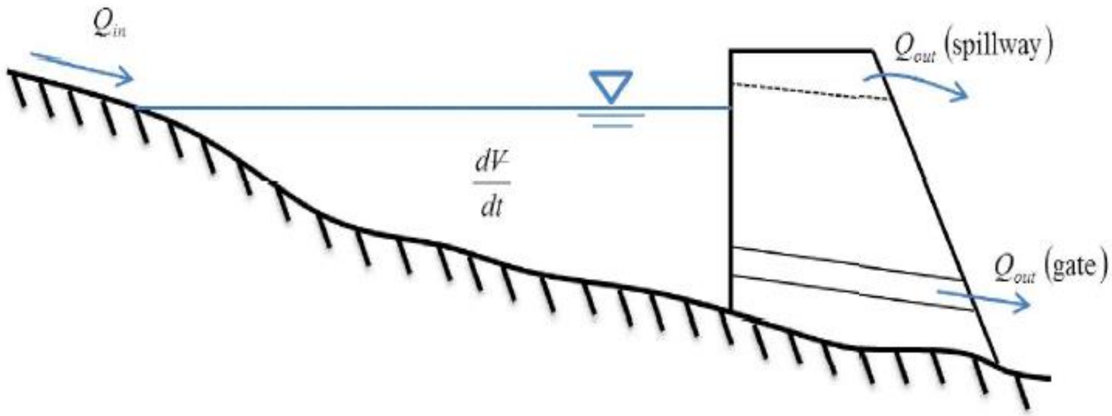


Figure 5.8 Reservoir Inflow, Outflow, and Storage

In the case of a flooding situation, the operation of the gated spillway is concerned with the optimal operation of an existing reservoir system, and the decisions releases for various purposes have to be made in a considerably shorter time period based on the optimization model to obtain the optimal releases for the reservoir gates.

An optimization procedure based on a genetic algorithm (GA) optimizer interfaces the other components 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, with formulation explained below.

5.2.2 Reservoir Operational Constraints

There are many types of constraints in the world of optimization including bound constraints. The reservoir operation constraints can be assorted as greater than and less than sort of constraints, which define the operational target, upper and lower variables bounds, limitation of the outlet and spillway gates, and the reservoir storage capacity. All the constraints mentioned above are included in the problem formulation in the optimization model (genetic algorithm).

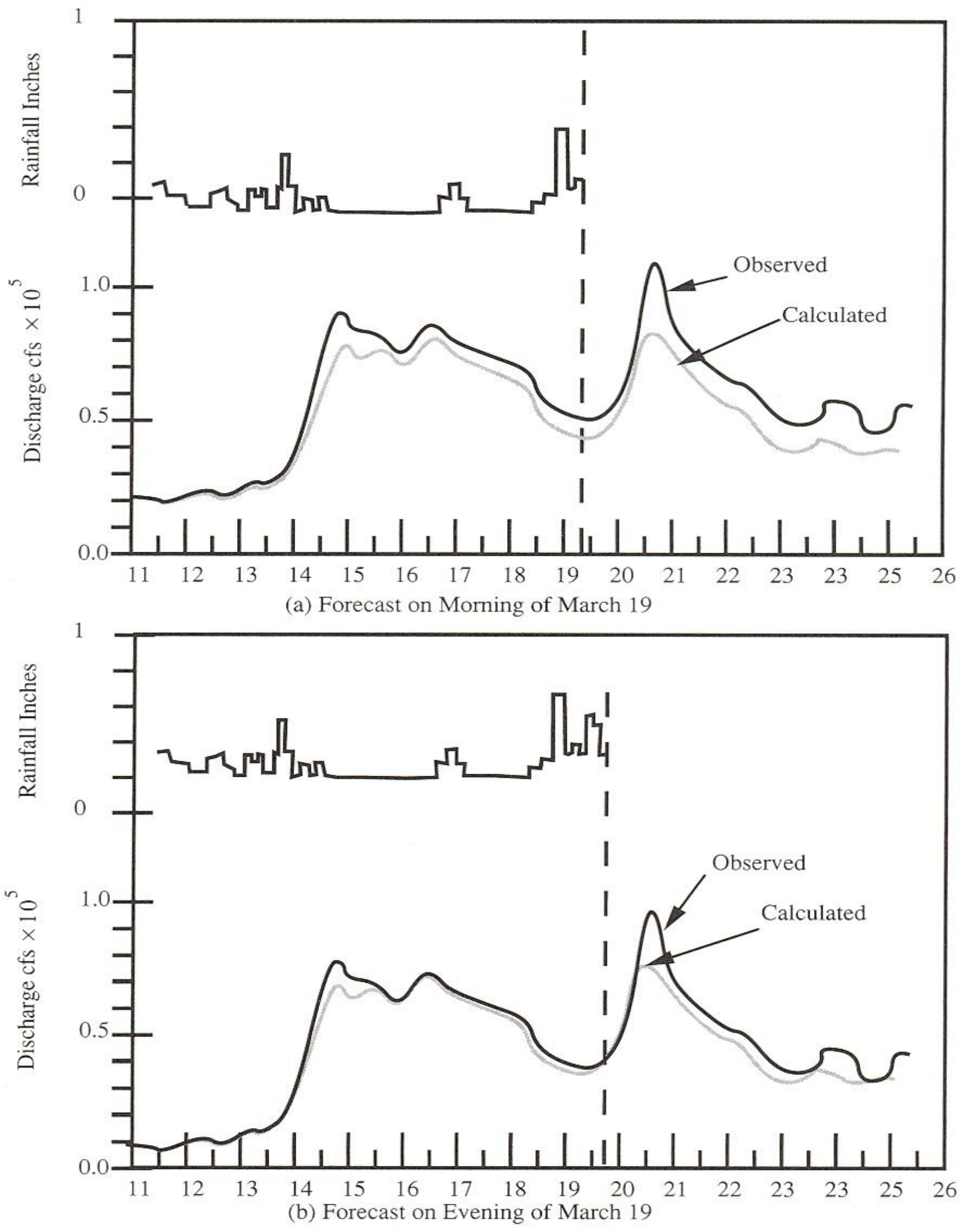
The process of gate operation (openings) is a main operational constraint designed for operation between a range of minimum and maximum allowed gate opening. The gate operation limits depend on the physical limitation of gate operation at which the minimum and maximum allowed rate of change in gate opening is predetermined.

The optimization simulation model used to determine real-time operation decisions (gate operations) incorporates real-time precipitation and streamflow data and forecasted rainfall throughout the system. The model consists of the following components:

- Forecasting model, to predict the precipitation for next the time period of simulation.
- Rainfall-runoff model, to demonstrate the hydrologic response of a watershed and then determine the discharge come off from the watershed.
- Unsteady one or two-dimensional flow model for the river-reservoir system, (in our case here two-dimensional model), to route the water flow in the river stream and further in the flood plain area in case one-, two- and combined one-and two-dimensional.

- ❖ Optimization model for determining the optimal reservoir spillways' gates 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. As one observes from the graph, precipitation events, as shown in Figure 5.9, 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 vertically through the graph.



(a) Forecast on Morning of March 19

(b) Forecast on Evening of March 19

Figure 5.9 a, the precipitation forecasts were made on the morning of March 19th resulting in the ability to make forecasts of flood hydrographs. A similar phenomenon is seen in Figure 5.9 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. The real-time operation of multi-reservoir systems involves many considerations, such as hydrologic, hydraulic, operational, technical, and institutional considerations. That will enable engineers in the field to make critical decisions about releases from the reservoirs 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 a significant problem in hydrologic engineering, especially when flood-control reservoir operations are involved.

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

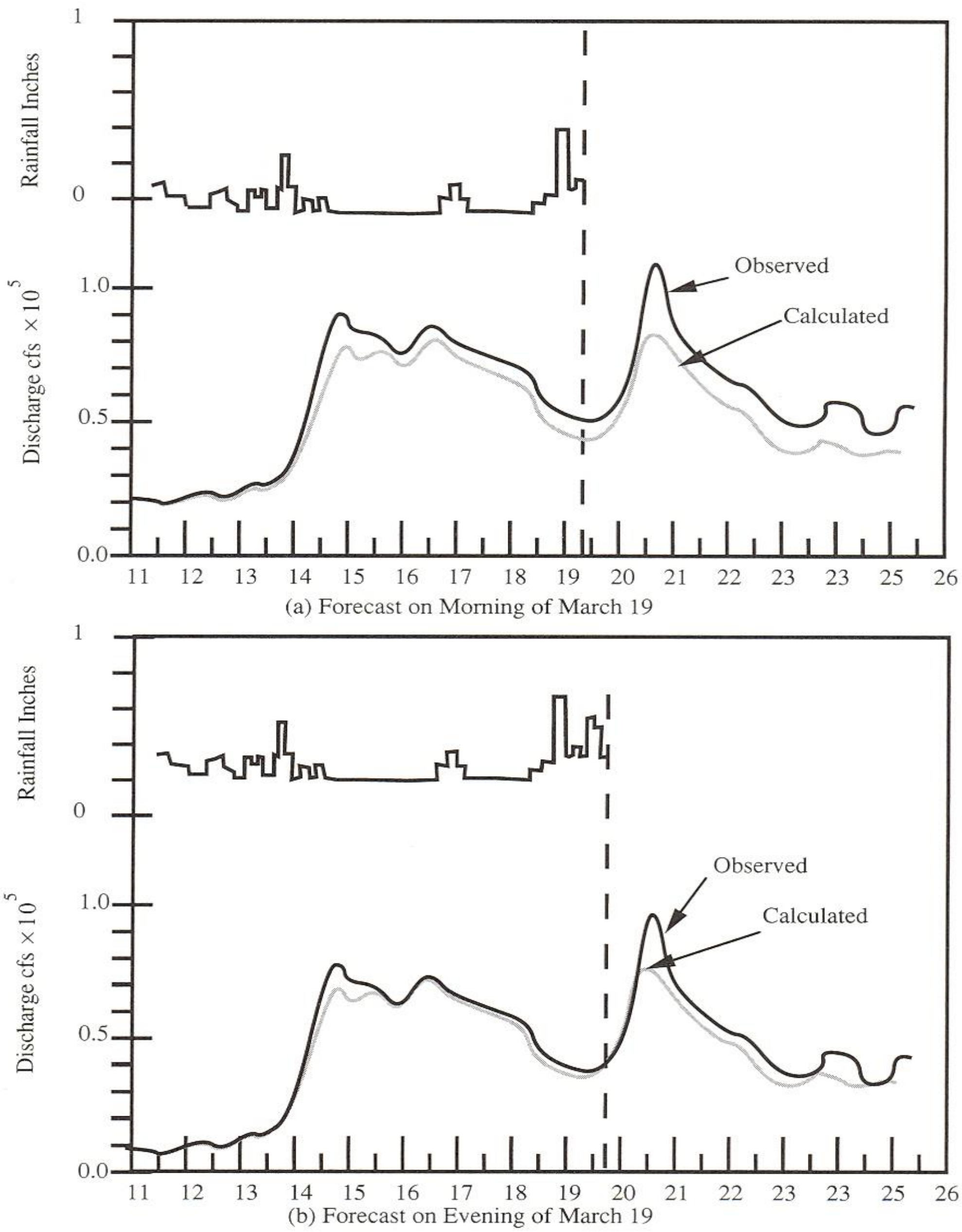


Figure 5.9 Observed and Forecasted Hydrographs at Kanawha Falls, Resulting from a Forecast of the March 1967 Flood Event, (L. Mays & Tung, 1992)

5.3 Optimization Model

The optimization technique adopted in this research is the genetic algorithm in MATLAB. A genetic algorithm is an optimization method that is a heuristic solution-

search, basically based on the principle of Darwinian evolution through genetic selection. Genetic algorithms use a highly abstract version of the evolutionary processes to evolve solutions. Each genetic algorithm (GA) operates on a population of
artificial chromosomes which are strings in a finite alphabet, usually binary. Each chromosome represents a solution to a problem and has fitness, a real number which is a measure of how good a solution is to the problem. Genetic algorithms differ from other optimizing methods such as gradient-based method and the simplex method, GA does not necessarily require a well-defined fitness function (objective function). There is no global definition describes GA; however, in general, genetic algorithm constructed of a number of operators.

5.3.1 Genetic Algorithm Structure

A genetic algorithm consists of several distinct components. This is a particular strength because it means that standard components can be re-used, with trivial adaptation in much different GAs, thus easing implementation. The main components are the chromosome encoding, the fitness function, selection, recombination and the evolution scheme (McCall, 2005).

5.3.1.1 Encoding of Chromosomes

The most critical issue in using a genetic algorithm is to find a suitable encoding to a chromosome of the examples in the problem domain. A decent decision of representation will help the search to be easier by limiting the search range; a poor choice will result in a large search range. Thus, chromosomes encoding is one of the problems, when someone starts solving genetic algorithm problem. Encoding very depends on the problem.

5.3.1.2 Fitness function

The fitness function is a computation that evaluates the quality of the chromosome as a solution to a particular problem. By analogy with biology, the chromosome is referred to as the genotype, whereas the solution it represents is known as the phenotype. The translation process can be quite complicated. In timetabling and manufacturing scheduling GAs, for example, a chromosome is translated into a timetable or set of scheduled activities involving large numbers of interacting resources. The fitness computation will then go on to measure the success of this schedule in terms of various criteria and objectives such as completion time, resource utilization, cost minimization and so on. This complexity is reminiscent of biological evolution, where the chromosomes in a DNA molecule are a set of instructions for constructing the phenotypical organism.

5.3.1.3 Selection

A GA uses fitness as a discriminator of the quality of solutions represented by the chromosomes in a GA population. The selection component of a GA is designed to use fitness to guide the evolution of chromosomes by selective pressure. Chromosomes selected for recombination by fitness. Those with higher fitness should have a greater chance of selection than those with lower fitness, thus creating a selective pressure towards more highly fit solutions. Selection is usually with replacement, meaning that highly fit chromosomes have a chance of being selected more than once or even recombined with themselves. The traditional selection method used is the Roulette Wheel (or fitness proportional) selection. This allocates each chromosome a probability of being selected proportional to its relative fitness, which is its fitness as a proportion of the sum of finesses of all chromosomes in the population (Goldberg,1989). There are many different selection schemes.

5.3.1.4 Recombination

Recombination is the process by which chromosomes selected from a source population are recombined to form members of a successor population. The idea is to simulate the mixing of genetic material that can occur when organisms reproduce. Since selection for recombination is biased in favor of higher fitness, the balance of probabilities (hopefully) is that more highly fit chromosomes will evolve as a result. There are two main components of recombination, the genetic operator crossover, and mutation. Genetic operators are nondeterministic in their behavior. Each occurs with a certain probability, and the exact outcome of the crossover or mutation is also non-deterministic.

5.3.1.5 Evolution

After recombination, resultant chromosomes are passed into the successor population. The processes of selection and recombination are then iterated until a complete successor population is produced. At that point, the successor population becomes a new source population (the next generation). The GA is iterated through a number of generations until appropriate topping criteria are reached. These can include a fixed number of generations having elapsed, observed convergence to a best-fitness solution, or the generation of a solution that fully satisfies a set of constraints. There are several evolutionary schemes that can be used, depending on the extent to which chromosomes from the source population are allowed to pass unchanged into the successor population. These range from *complete replacement*, where all members of the successor population are generated through selection and recombination to *steady state*, where the successor population is created by generating one new chromosome at each generation and using it to replace a less-fit member of the source population. The choice of the evolutionary scheme is an important aspect of GA design and will depend on the nature of the solution space being searched. A widely-used scheme is *replacement-with-elitism*. This is almost complete replacement except that the best one or two individuals from the source population are preserved in the successor population. This scheme prevents solutions of the highest relative fitness from being lost from the next generation through the nondeterministic selection process.

5.3.1.6 Genetic Algorithm Design

There are many choices that have to be made in designing a GA for a given application. The choice of encoding will depend on the nature of the problem. Nonbit-string representations are now common and include sequences of integer or floating-point values as the size of the allele set expands, for example, where the strings consist of floating-point numbers, the set of possible chromosomes becomes considerably greater. Many modern (or non-classical) GAs use a range of representational approaches to ensure that the set of possible chromosomes is a close match for the set of possible solutions to the problem. Having selected an encoding, there are many other choices to make. These include the form of the fitness function; population size; crossover and mutation operators and their respective rates; the evolutionary scheme to be applied; and appropriate stopping/re-start conditions. The usual design approach is a combination of experience, problem-specific modeling, and experimentation with different evolution schemes and other parameters. Several examples of non-classical GAs can be found in [9] and [21]. Part G of [1] contains several examples of real-world applications. A typical design for a classical GA using complete replacement with standard genetic operators might be as follows:

(1) Randomly generate an initial source population of P chromosomes.

(2) Calculate the fitness, *F(c)*, of each chromosome c in the source population.

(3) Create an empty successor population and then repeat the following steps until P chromosomes have been created.

5.3.1.7 Resources

There are a number of resources freely available on the Internet for those interested in applying Gas in their own area. A good place to start is the Genetic Algorithms Archive, maintained by the US Navy Centre for Applied Research in Artificial Intelligence (http://www.aic.nrl.navy.mil/galist/). The site is a long-established resource for the genetic algorithm and evolutionary computation communities and contains lists of research groups, downloadable software, and links to related sites of interest. The Archive also maintains an archive of postings to the EC Digest mailing list (formerly GA-List).

A wide range of downloadable software is available to assist the rapid development of GAs. Toolkits are available in many programming languages and vary widely in the level of programming skill required to utilize them. Mathematicians are likely to find GAOT, the Genetic Algorithm Toolbox for MATLAB, the easiest way to begin experimenting with GAs. The Toolbox implements a GA as a set of MATLAB functions, which can be redefined and reconfigured to suit different applications. GAs using binary and floating-point encodings can be defined, and a range of standard recombination operators are also supplied. The Toolbox website also provides some example of GAs to solve some standard problems. GAOT can be downloaded from http://www.ie.ncsu.edu/mirage/GAToolBox/gaot/.

5.3.2 Genetic Algorithm in MATLAB

Genetic algorithms are an inbuilt function of the Global Optimization Toolbox in MATLAB. It is a method available in MATLAB to solve both constrained (which have to be converted to an unconstrained problem) as well as unconstrained optimization problems. A GA only solves unconstrained problems. The genetic algorithm function repeatedly modifies a population of individual solutions, selecting individuals at random from the current population to be parents and using them to produce the children for the next generation, at each successive generation, Figure 5.10.

The genetic algorithm solution methodology Figure 5.11 starts with creation of a random initial population, then creating a sequence of new populations and computing the values of a fitness function for each new population. The algorithm then chooses members called parents based on their fitness and chose individuals in the current population with the lower fitness as elite. The current population is replaced with the children of the elite by combining the vector entries of a pair of parents – crossover. The replaced population then forms the next generation. This runs through an infinite loop type sequence until a predefined ‘stopping criteria’ is met.

The algorithm chooses the initial population with a size of the ‘population size,’ which is an input for the genetic algorithm. If an approximate location of the minimal point for a function is known, an ‘Initial range’ is required to be set so that the point lies near the middle of the range. Thereafter, at each step, the genetic algorithm selects individuals that have better fitness values as parents. The algorithm usually selects individuals that have better fitness values as parents. The ‘Selection Function’ field in the ‘Selection’ options could be used to provide selection criteria for the selection of the elite individuals for every generation. The selection function chooses the parents for the next generation based on their scaled values from the fitness scaling function. Thus, this function plays a vital role of comparing the various individuals of a generation based on their individual fitness function values. The selection option lays out a line in which each parent corresponds to a section of the line f length proportional to its scaled value. The algorithm moves along the line in steps of equal size.

Another category of options required to be set are the ‘Reproduction Options. Reproduction options control how the genetic algorithm creates the next generation. The breeding options include the elite count (number of elite individuals determined from each generation), crossover function (fraction of individuals in the next generation other than the elite children). The genetic algorithm uses the individuals in the current generation to create the children that make up the next generation.

The various steps involved in developing a Genetic Algorithm in MATLAB include:

1. Identification of the objective function and the constraints of the problem

   The first phase in the formulation of a genetic algorithm is to define the problem in the form of a standard minimization optimization problem; this includes identifying all the decision and state variables as well as the parameters in the model and then formulating the objective function and the constraints as required.

2. Conversion to an unconstrained form

   As explained earlier, the problem formulated in step 1 is required to be converted to an unconstrained optimization problem format. The objective function of the unconstrained problem (including the penalty functions for the various constraints).

3. Creation of 'Fitness function' using a function (.m) file in MATLAB.

   A fitness function file created in MATLAB for computation of the value of the objective function defined in step 2. This function is used to compute the individual fitness values for each generation.

4. Identification and setting the various parameters for the Genetic Algorithm.

   This process requires a deep analysis of the different terms associated with the fitness function of the genetic algorithm. The parameters necessary for the GA as explained earlier are computed using an exhaustive regime of sensitivity analysis for the variously considered parameters.

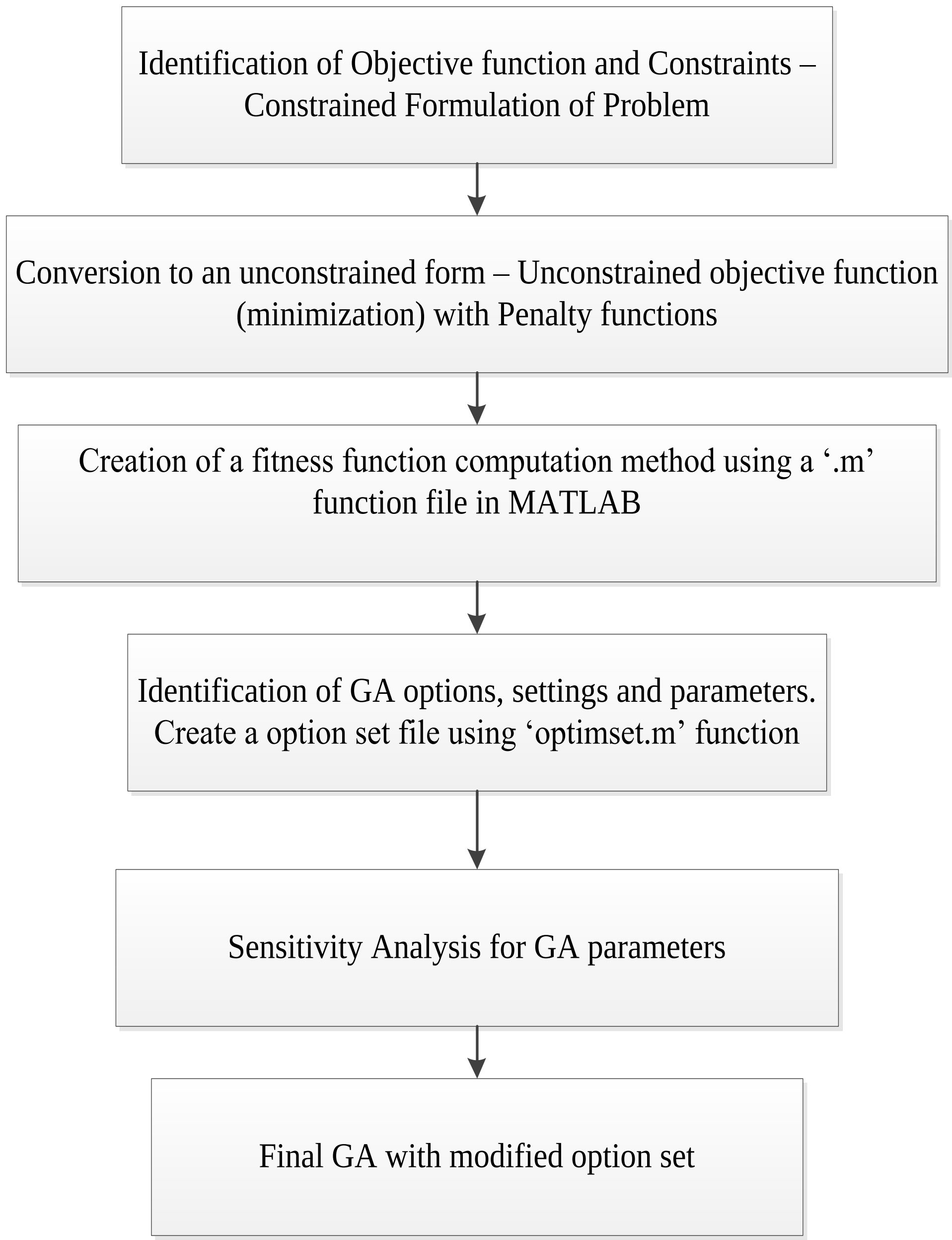


Figure 5.10 Genetic Algorithm

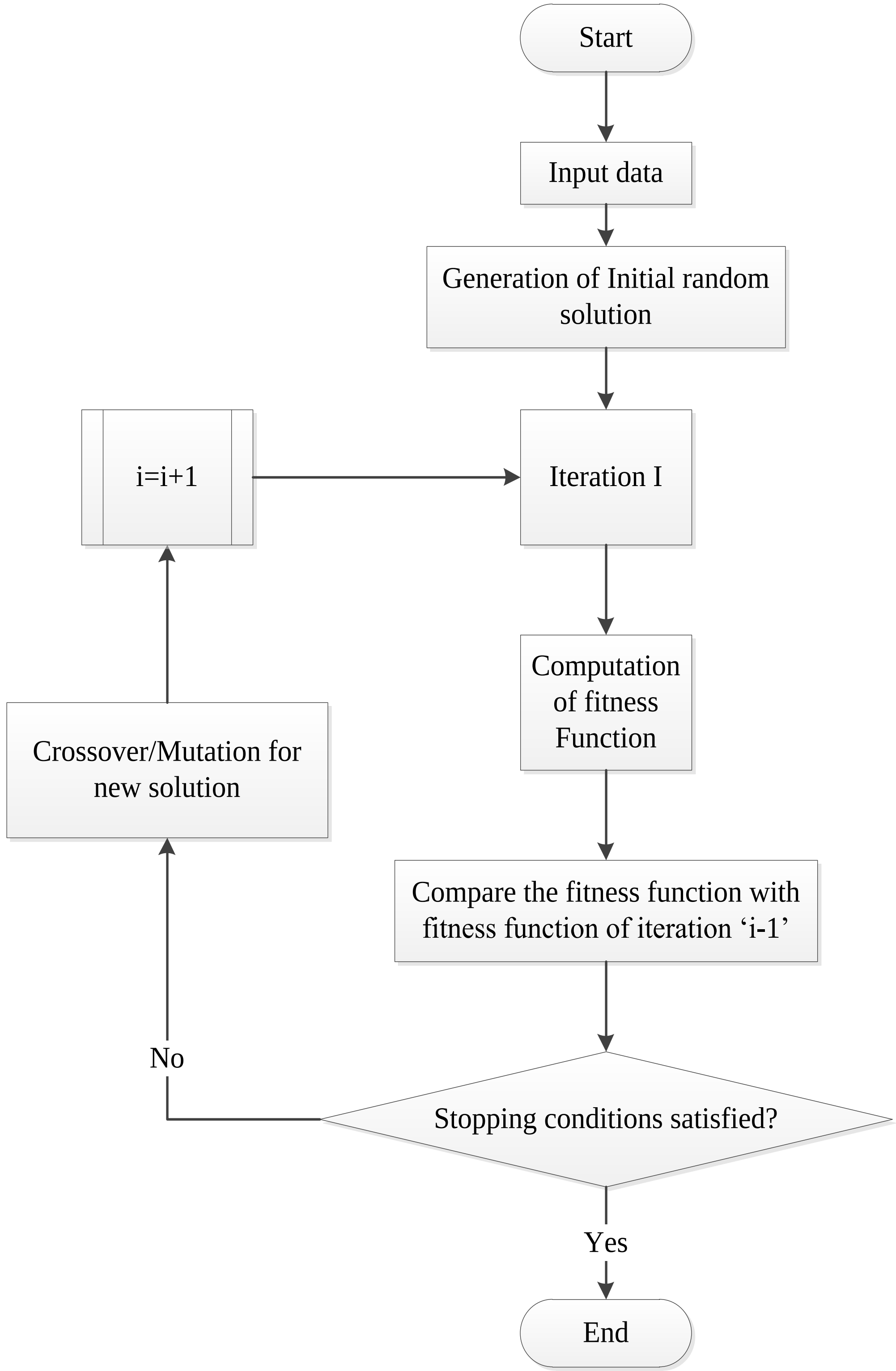


Figure 5.11 General Procedure of Genetic Algorithm

CHAPTER 6 FORMULATION OF THE PROBLEM

6.1 Statement Theoretical

The expected damages caused by a natural flood event that exceeds the hydrologic design return period of the reservoir to a certain degree cannot be avoided. Using optimization techniques in managing and operating the flooding facilities can tremendously minimize the effect and damage cost. The general theoretical formulation for optimizing and simulation of the flood releases condition from a river-reservoir system that minimizes the expected flood damage are subjected to the following:

1. Hydrologic constraints for rainfall-runoff are simulated using HEC-HMS.
2. Diffusion-Wave or Saint-Venant Equations for simulating unsteady flow modeling in a two-dimensional form which predicts the flow and its various components in a river -reservoir system simulated using HEC-RAS 5.0.6.
3. The allowable releases from the reservoir, i.e., the maximum and minimum and the flow at particular locations or computational cells.
4. The maximum allowable water surface elevations at specific computational cells in the flood plain and minimum water surface elevation in the river system.
5. The operation limits of reservoir gates during flooding conditions.
6. The storage capacity of the reservoir.

The simulation part of the problem is described through the first two constraints and the fifth constraint of the model. The HEC-RAS 5.0 two-dimensional computational module has an option of either using the 2D diffusion-wave equations, or the full 2D

Saint-Venant equations (sometimes referred to as the full 2D shallow water equations) to run a simulation model. The 2D diffusion-wave equation is set as the default. Generally, the 2D diffusion-wave equations are considered for most flooding applications. The diffusion wave equation set runs faster, more stable than 2D Full Saint Venant equation. Definitely, that does not mean it works the same for all applications. However, there are many applications and cases where the two-dimensional Full St. Venant equations should be used for more accuracy. The choice between the two options is not that hard task; it is simply a matter of selecting the equation set. Both can be used in two separate runs and compare the results, (Bunner, 2016).

There are three important dependent variables form the optimization-simulation model, which are the water surface elevations and the discharges in all computational cells in the downstream floodplain, the discharges values at all computational cells and the reservoir releases for all reservoirs. The mathematical expression of the optimization-simulation problem for operating a river-reservoir system is described as in the following section.

6.2 The Objective Function

The objective function is to minimize the flow rate at control points in addition to minimizing the water surface elevation at specified cells, i, in all time t during the flooding condition. Then the mathematical expression of that objective function can be represented in term of flow for to that minimizing the total damage caused by the raising water elevation or depth at a certain location as below:

$$Min\ Z = \sum_{x=1,y=1}^{x,y} \sum_{t=1}^{T} [C_{x,y}\ Q_{x,y,t}] \tag{6.1}$$

Where: -

- $Q_{x.y,t}$ is the flow rate time-series at the the control point, (x, y), of the river-reservoir system that affects the cell, i, in the downstream two-dimensional area.
- $C_{x,y}$ is the penalty coefficient at control cells, x, y.
- x, y & t are spatial and temporal indices, respectively.

Alternative objective function has been adopted to solve the problem that maximizes the downstream water surface elevation (H) at specified control cells while satisfying all the constraints stated above can also minimize the total damage, and the objective function could also be written as in the following form:

$$Min\ Z = Max\left[h_{x,y,t}\right] \tag{6.2}$$

The objective minimizes the total damage in the entire river system at all times, t, including the upstream side, as it reliefs the headwater upstream reservoir gate

6.3 Problem Constraints

To achieve the objective function stated above a number of constraints and limitations must be specified. In general, the constraints that governed the objective function and as it has been described earlier in this chapter can be classified as four main types:

a) Hydrologic constraints: represented by the rainfall-runoff relationships defined by the sub-basin areas, rainfall losses due to canopy interceptions, depression storage, soil infiltration, excess rainfall transform methods, watershed runoff routing method, internal boundary conditions and initial conditions that depict the rainfall-runoff process in different components of a watershed system and function as generally stated as:

$$h\left(P_{x,t}, L_{i,t}, Q_{i,t}\right) = 0 \tag{6.3}$$

where ($P_{x,y,t}$) is the matrix of precipitation data in the system; ($L_{x,y,t}$) is the rainfall losses of the watershed system and ($Q_{x,y,t}$) are the watershed and reaches discharges.

All the hydrologic can be expressed constraints are in matrix form because the problem has dimensions of space, x, y, and time, t solved using HEC-HMS.

b) Hydraulic constraints define the unsteady 1-D and 2-D flow in the river-reservoir system. They are defined by the Saint-Venant equations for one- (equations 1.3-1.5) and two-dimensional unsteady flow, and related relationships of upstream boundary condition, downstream boundary condition, external two-dimensional flow area boundary conditions, internal two-dimensional area boundary conditions, and initial conditions that depict the flow in different components of a river-reservoir system,

$$g\left(h_{x,y,t}, Q_{x,t}\right) = 0 \tag{6.4}$$

where $(h_{x,y,t})$ is the matrix of water surface elevations in the system; $(Q_{x,y,t})$ is the discharge matrix of the system. All the hydraulic constraints are in matrix form is because of the problem has dimensions of space, *i*, and time, *t* solved using HEC-RAS.

c) Bound constraints include upper and lower discharge limits that define the maximum and minimum allowable reservoir releases and flow rates at target locations:

$$\underline{Q}_{x,y} \leq Q_{x,y,t} \leq \overline{Q}_{x,y} \tag{6.5}$$

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

Another significant hydraulic constraint is the water surface elevation bounds defined by the allowable the upper limit and lower limit at specified locations in the downstream two-dimensional area, including reservoir levels:

$$\underline{h}_{x,y,} \leq h_{x,y,t} \leq \overline{h}_{x,y} \tag{6.6}$$

d) Operation constraints which include the rules of reservoir operation and releases, reservoir storages and the beginning and the end of the simulation period, reservoir storage capacities, etc., are also important to be included in the optimization-simulation model:

$$\mathrm{W}\left(Q_{x,y,t}, h_{x,y,t}\right) \leq 0 \tag{6.7}$$

6.3.1 Reduced Optimization Model

The previous optimization model is an optimal control problem solved by interfacing an optimization solver which is here the genetic algorithm in MATLAB and the unsteady flow simulation model (HEC-RAS). Usually, the genetic algorithm used to solve an unconstrained problem which is a reduced optimization problem combining equation 6.5 -6.7, and HEC-RAS solves equation 6.4 implicitly to the optimization model at iteration it is called. A reduced optimization model with constraints equations 6.5 – 6.7 in the form of penalty function can be solved by the genetic algorithm which solves an unconstrained problem. The unconstrained reduced optimization model is in the following:

$$
\begin{aligned}
Minimize\ Z = \ & W_{1,t} \sum_{x=1,\,y=1}^{X,Y} \sum_{t=1}^{T} C_{x,y} Q_{x,y,t} \\
& + \ W_{2,t} \sum_{i=0}^{I} \sum_{t=0}^{T} ((max(0, Q_{i,t} - Q_{max,i}))^{\ n} \\
& + (\max(0, Q_{min,i,t} - Q_{i,t}))^{n}) \qquad (6.8) \\
& + W_{3,t} \sum_{i=0}^{I} \sum_{t=0}^{T} ((max(0, h_{i,t} - h_{max,i}))^{n} \\
& + (\max(0, h_{min,i,t} - h_{i,t}))^{n})
\end{aligned}
$$

Where:

$W_{1,t} - W_{3,t}$ are penalty weights

6.4 Optimization/Simulation Solution Methodology

A real-time optimation and simulation to manage a river-reservoir system by determining the releases before, during and even after the extreme flood event through an optimization simulation model that could help the decision maker to control reservoir releases and consequently the water surface elevations and keep it within the desired level as much as possible. The decision variable of the problem formulation is the water discharge (Q), based on the reservoir gate or gates opening that represents the control variable (state variable).

The overall model starts inputting actual rainfall data, at time that it is needed to make decision on how much water should be released to so that to prepare enough volume in the reservoir to accommodate the upcoming flood water wave, meanwhile the model generates a short-term rainfall forecast and then the floods that can be happened by using the real-time rainfall of the precipitation gages and expects the real-time flood water elevation in the river-reservoir system to avoid accumulating a headwater on the gate of the reservoir. A methodology of projecting future rainfall within the next few minutes to hours has been developed by Danny Che, 2015, as a part of the methodology. Forecasted rainfall data will be used to simulates the watershed rainfall-runoff through HEC-HMS model, and then to produce hydrographs as time series of the reservoir inflows that is going to be used as inputs of the optimization model to compute the releases of the reservoirs gates in a river-reservoir system. The optimization model will come up with sets of a feasible solution of how much water should be released to satisfy some the problem constraints. Once these set of feasible solutions for the decision variable which may or might not contain the optimal sets are obtained, the obtained data then is to use as the input of one- and two-dimensional unsteady model to be routed downstream and simulated though HEC-RAS 5 model to check if the flooding will occur or not. If the answer was yes, the model would repeat the process until the target water surface elevations are achieved without or with minimum damage effect for the system or for the area downstream. So, the main objective of this study is to control the reservoir resales and keep the water surface elevations acceptable level. For instance, 100-year flood elevation might be one of the targeted levels in the processed system. For, the next iteration, the model uses the hourly projected real rainfall data for running the HEC-HMS model to compute the actual runoff quantities from the watershed and then the reservoir water level and consequently the releases from the reservoir and hence routing them downstream to make a decision for the next iteration of the operation period. This process continues and repeats until the objective is met at all time with satisfying all of the constraints for the entire period of simulation. The reason behind that model enabled to forecast and run the simulation in advance of the storm event is the can help to make pre-decision and take the necessary action to minimize the flood condition as much as possible. The stages of the simulation and optimization model algorithm are illustrated in Figure 6.1.

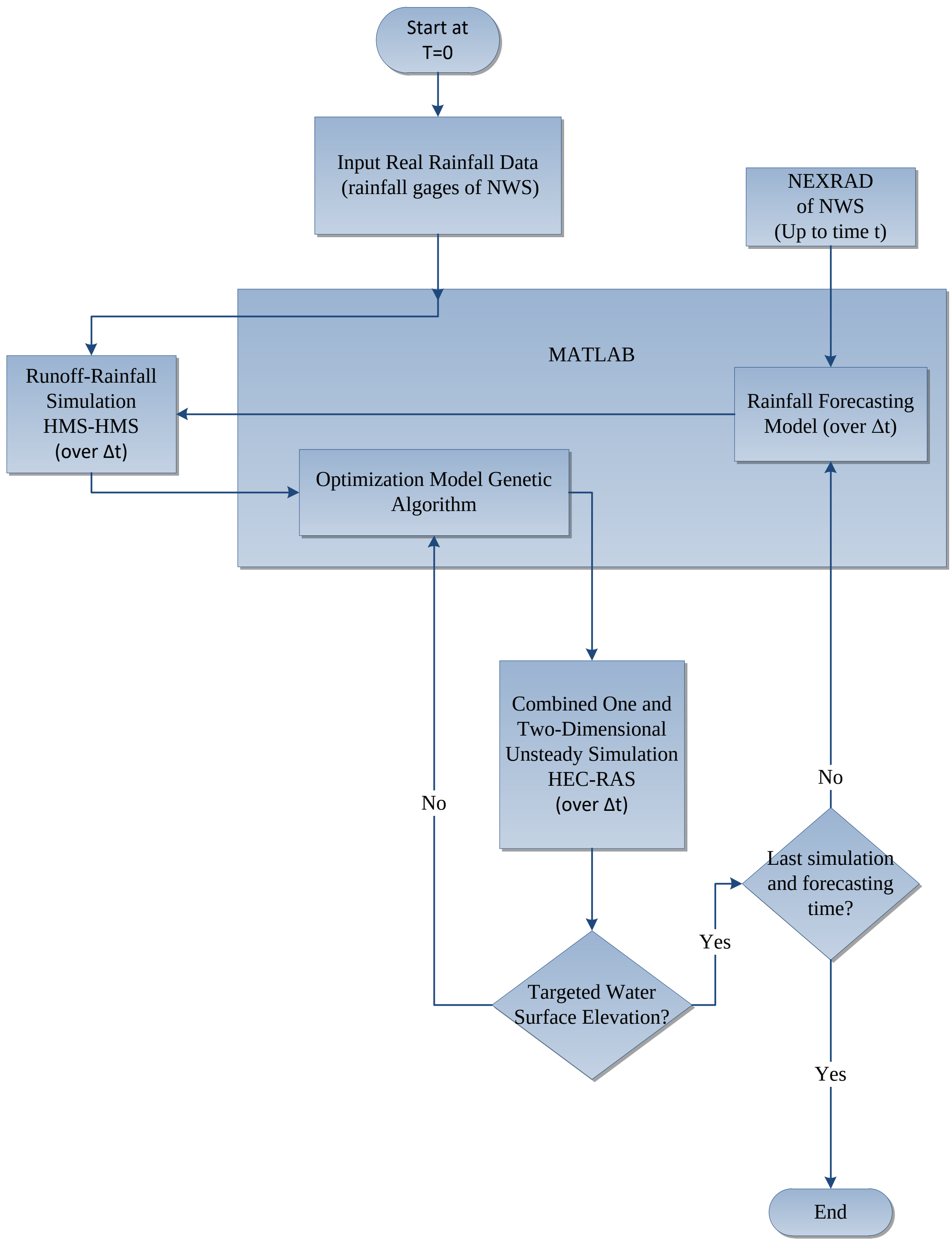


Figure 6.1 Overall Optimization-Simulation Stages

The optimization simulation model initiates to obtain real-time rainfall data from precipitation gages stations to get for the HEC-HMS to start simulating the rainfall-runoff model and to produce the required hydrographs to input them into one dimensional unsteady model for routing them from the watershed exist point to the reservoir location. Now, the optimization model has the required data and ready to run and search for the optimal solution to make the proper decision of real-time releases from the reservoir. The optimization model starts to generate the possible operation to the reservoir until and determine the gate opening. These possible solutions represent the releases from the reservoir gates, so the next step is to route these discharges downstream into one and/or two-dimensional unsteady model through HEC-RAS to simulate further downstream to the two dimensional areas of study where other some constraints should be stratified and meet the objective of the targeted water level at the control cell in the study area. This stage in the most important operation in the model where the model began to test the objective and repeats the process to readjust the releases from the reservoir. For example, if the releases of the first iteration after routed in the unsteady flow models produced water level in the downstream control cell higher than the target level, the model will repeat the process and reduce the releases and vice versa until acceptable water levels are obtained. The process continuous iterating and the optimization and simulation (HEC-RAS) model. Once the objective is satisfied and obtained the target water levels up this time t, the overall process repeats for the next period $t+\Delta t$ of simulation and optimization until the last time of Δt. The process of forecasting rainfall is a complementary process to the model to ensure the continuity of model operation, right after they enter the actual rainfall data. The model of rainfall forecasting starts to forecast the upcoming rainfall storm over the next Δt, while the model uses the known rainfall up to the immediate simulation time. t. the forecasting rainfall model generates and provides the required precipitation data for the next iteration that prompts the simulation process to start and simulate the hydrologic model. The overall process of the simulation-optimization model process will continue until the last period of simulation, and then the model will be stopped. Figure 6.2 below illustrate the various component of River-Reservoir System operation in a real-time pattern.

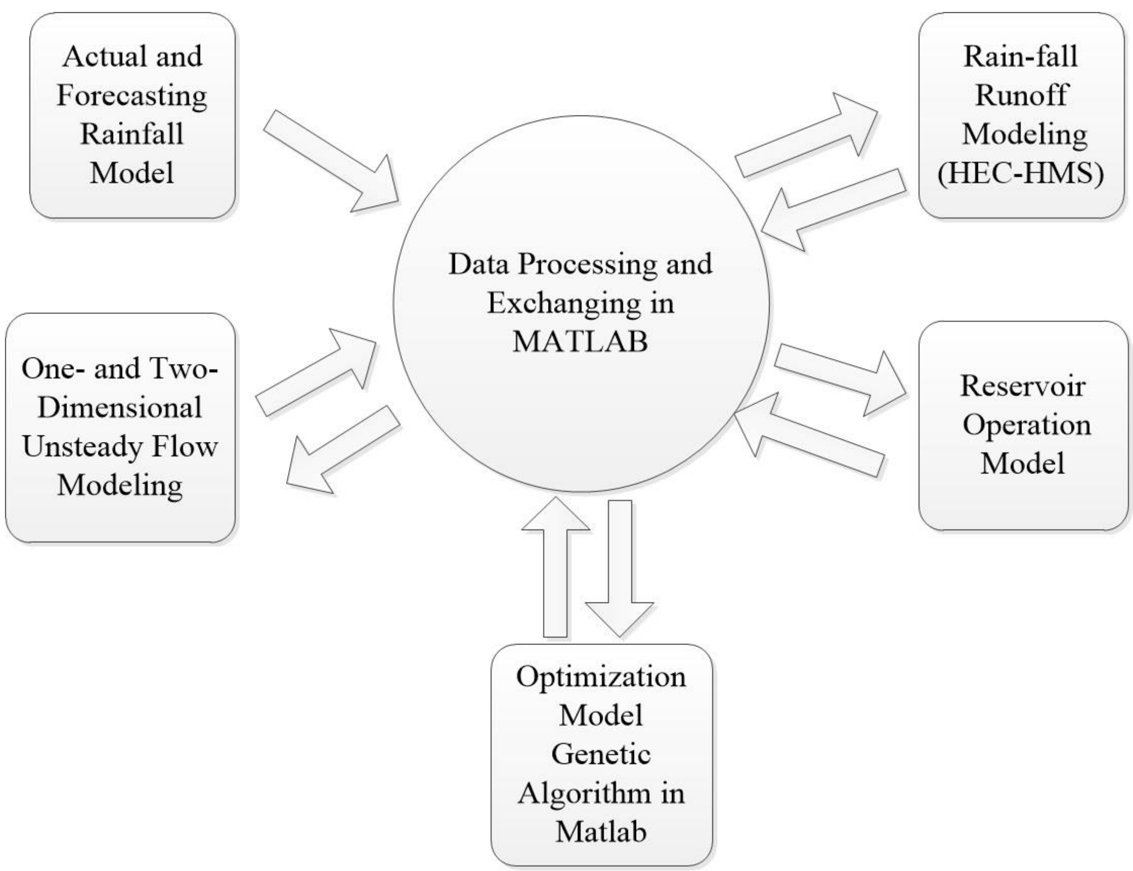


Figure 6.2 Model Components Interfacing

6.4.1 Model Development

The overall model is developed by using the MATLAB environment for interfacing and exchanging data among the aforementioned submodels. Both actual (measured) rainfall data to time t are known, the forecasted rainfall is used to obtain future rainfall estimation and the time interval t to t+Δt. The model starts with inputting the first actual available rainfall data and the forecasted rainfall data of time Δt into the MATLAB model. MATLAB sends the rainfall data to the HEC-HMS model through an algorithm programmed into MATLAB. Once the HEC-HMS model receives the rainfall data, the rainfall-runoff is simulated. The results of the rainfall-runoff model (HEC-HMS) will be resent to the MATLAB through the same algorithm which represents the watershed outflow hydrographs in a matrix form so that the MATLAB can deal with it and sent it to the next sub model. These results now need to be routed to the reservoir location, so MATLAB is going to communicate HEC-RAS model and sent it the produced hydrographs to route it to where the reservoir is located and resent the data which is reservoir inflow hydrographs back to the MATLAB to use in the next sub-model which is the reservoir operation model. The model of reservoir operation started once the data of floodwater being arrived at the reservoir location to initiate set feasible solutions of how much water is going to be released for the reservoir gate. Genetic algorithm solver into MATLAB is coded to carry out this most important task of generating the feasible and optimal solutions and according to the operating constraints discussed before. These feasible solutions will be resent to the on- and two-dimensional unsteady flow model of the HEC-RAS model to route it further downstream where the control cells and the targeted water elevations should be. The feasible solutions generated tested one after another by communicating and resending the data back and forth between the reservoir operation model and the unsteady model of HEC-RAS until the optimal solution reached with satisfying all hydraulic and operation constraints, and the flood elevation at the control cell in the area downstream. Figure 6.3 shows the operation optimization model process into MATLAB. The process will exit the optimization subroutine of the reservoir operation optimization model once the objective is met and move to the next Δt of rainfall forecasting and the repeat the same all over the process until the end of the storm.

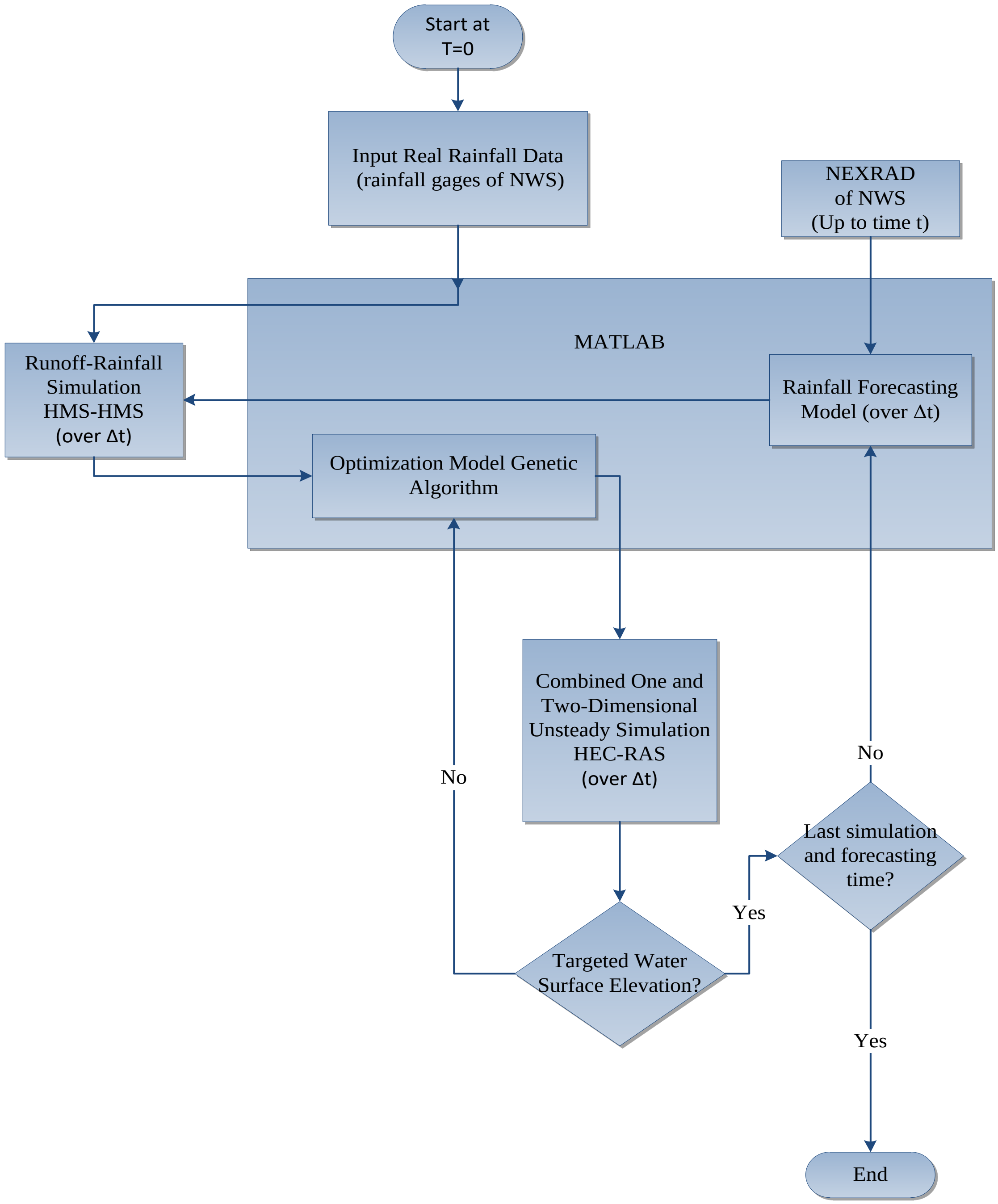


Figure 6.3 The Operation Optimization Model Process in MATLAB

CHAPTER 7 MODEL APPLICATION

An example application is introduced in this chapter for demonstrating the optimization-simulation model for real-time operation of a flood event. The application is designed specifically for testing the optimization-simulation model to determine the optimal operation of the floodgates in a reservoir with possible downstream flooding in the river and floodplain of an urban area. The river reach is real, but the reservoir is a hypothetical reservoir with one floodgate. The application assumes an inflow hydrograph to the hypothetical reservoir so that the HEC-HMS portion of the overall model is not employed in this application. The unsteady flow modeling is performed using the combined one and two-dimensional approach.

The other very important purpose of this chapter is to describe in detail the development of the two-dimensional mesh for the two-dimensional modeling. Movement of water through the two-dimensional area is governed by the computational mesh.

7.1 Model Description

The example application, referred to as the Muncie Project (for Muncie, Indiana), is based upon information from (U.S. Army Corps of Engineers, 2016b). Muncie is located on the West and East Forks of the White River, which flows through Central and Southern Indiana as shown in Figure 7.1, creating the largest watershed contained entirely within the state. Muncie is located in East Central Indiana, about 50 miles (80 km) northeast of Indianapolis.

This example model application includes a hypothetical reservoir (modeled as a storage area) upstream of Muncie. Figure 7.2 illustrates the one-dimensional area along the main channel shown in blue and the two-dimensional flow area is outlined in red downstream of Muncie, Indiana and the main channel area in blue is the one-dimensional flow area. Figure 7.3 shows the location of the hypothetical reservoir (storage area) and the connection to the hypothetical dam (inline structure). The modeling purpose is to determine the reservoir operation (gate operations of the inline structure) in real-time for the hypothetical reservoir using an optimization-simulation model.

Figure 7.1 Location of Muncie, Indiana and White River

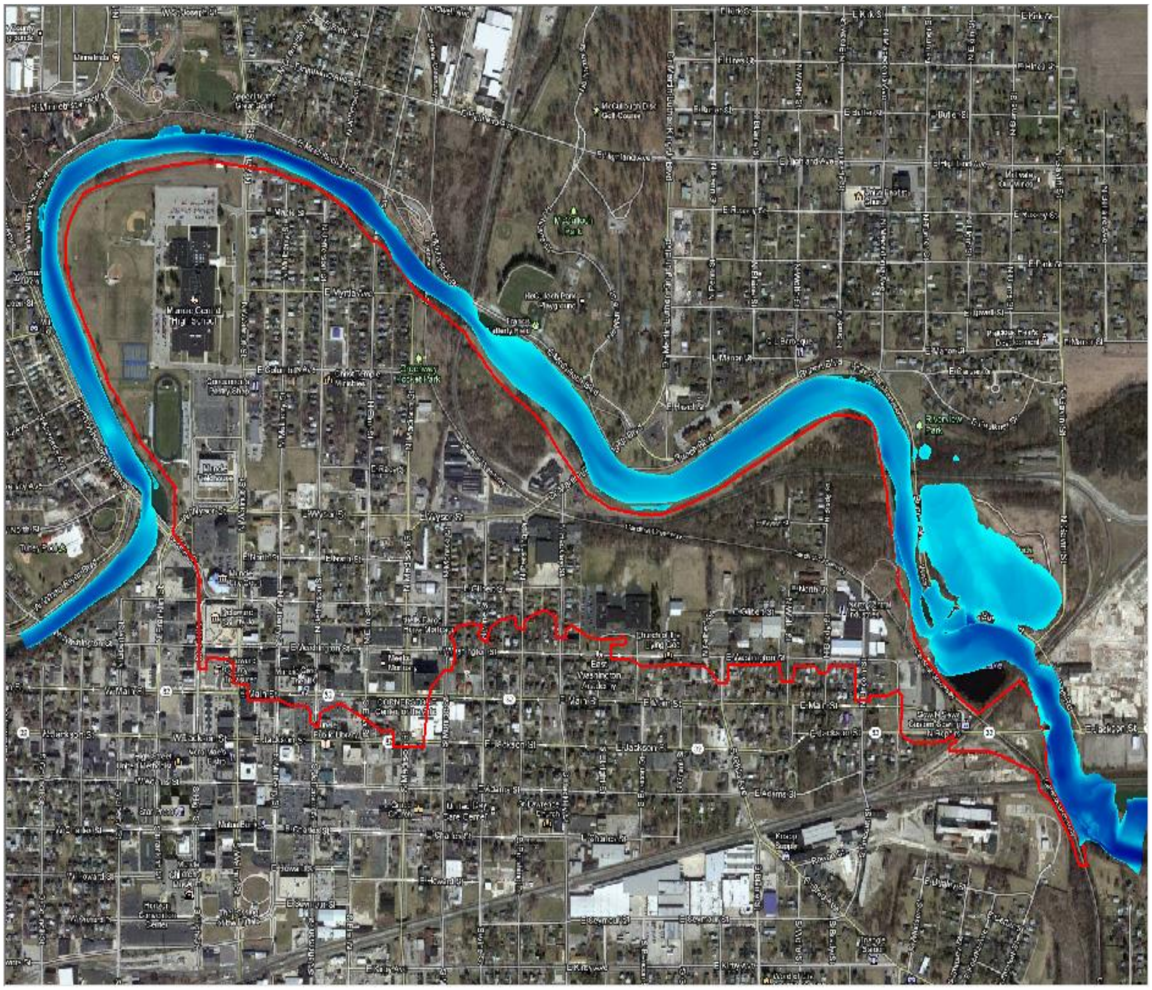


Figure 7.2 One-Dimensional Flow Area in Blue and Two-Dimensional Flow Area Outlined in Red Downstream of Muncie, Indiana

As a part of the Mississippi River system, the White River Basin drains 11,350 square miles of central and southern Indiana. The average streamflow is about 12,300 cubic feet per second close to the White River's confluence with the Wabash River in Southwestern Indiana. Changes in streamflow generally occurred seasonally and moderated. The peak flows are typically recorded in April and May, whereas in Summer and Fall are the lost flows. The annual precipitation rates in average from between 40 inches in the northern part of the basin and 48 inches in the south-central part. The precipitation is evenly distributed through the year, the winter and early spring rainfall.

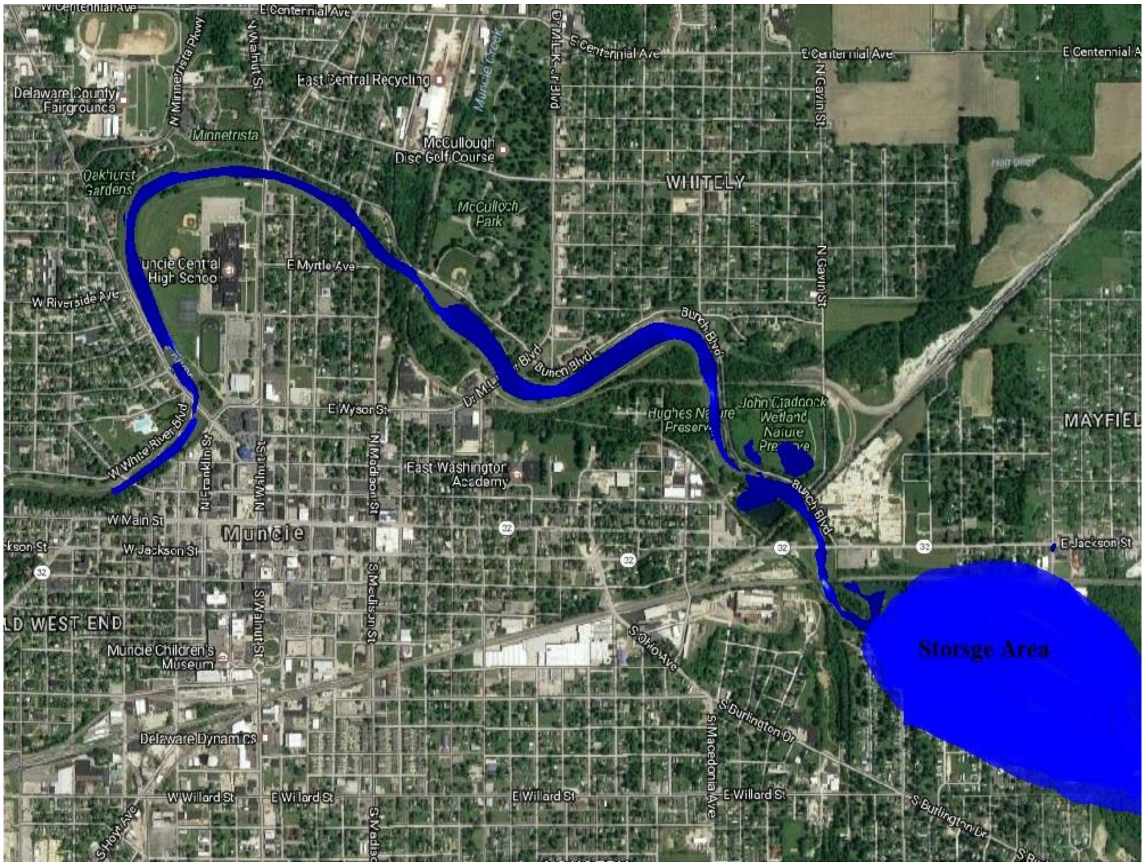


Figure 7.3 Hypothetical Storage Area and Connection to the Hypothetical Dam (Inline Structure)

As a part of the Mississippi River system, the White River Basin drains 11,350 square miles of central and southern Indiana. The average streamflow is about 12,300 cubic feet per second close to the White River's confluence with the Wabash River in Southwestern Indiana. Changes in streamflow generally occurred seasonally and moderated. The peak flows are typically recorded in April and May, whereas in Summer and Fall are the lost flows. The annual precipitation rates in average from between 40 inches in the northern part of the basin and 48 inches in the south-central part. The precipitation is evenly distributed throughout the year. The winter and early spring rainfall are generally characterized by a long-term duration, steady and of moderate intensity, while during late spring and summer rainfall seems to be of high intensity of short duration, see https://in.water.usgs.gov/nawqa/wr00002.htm.

7.2 Hypothetical Model.

The elevation-storage area of the hypothetical reservoir is presented in Table 7.1 and **Error! Reference source not found.**. The system is modeled using the combined one and two-dimensional modeling approach. The river reach is modeled using one-dimensional unsteady flow, while the floodplain area and/or the probable flood inundation areas are modeled using the two-dimensional unsteady flow approach, Figure 7.5 shows the one-dimensional area and the cross-sections of the river reach. Figure 7.6 shows the two-dimensional finite difference grid system of the floodplain and possible flood inundation area. The two-dimensional modeling is based upon the diffusion-wave modeling described in Chapter 4.

Table 7.1 Storage Area Elevation Volume Relationship

Elevation	Volume (Acre-Feet)
840	0
885.4	1657
894.8	5591
904.2	12228
913.6	22746
923	38559
932.4	60089
941.8	86922
951.2	119473
960.6	156859
970	201293
979.4	246078
988.8	295791
998.2	347839
1007.6	401953
1017	457592
1026.4	514117
1035.8	571196
1045.2	628516
1054.6	685971
1064	743449

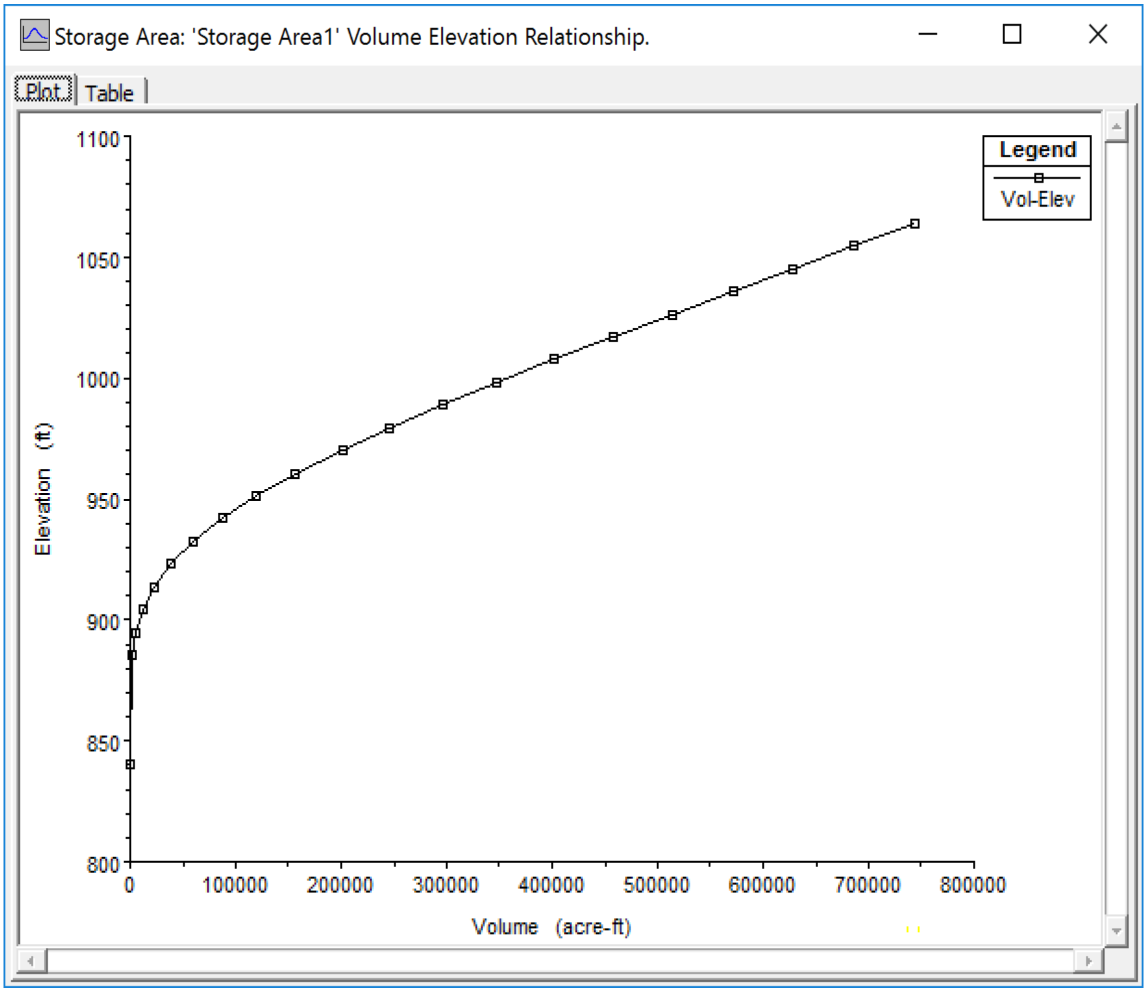


Figure 7.4 Hypothetical Model Storage Area Elevation-Volume Curve

The hypothetical reservoir is connected to river reach at the first upstream cross-section see Figure 7.7, through a gated spillway (inline structure) with a large radial gate to allow passing a wide range of significant discharges for the purpose of simulation process of a flooding event. One radial gate type is assumed to regulate the flow from the dam to the downstream channel. The total gates width is assumed to be 30 feet with maximum opening 21 feet, and the discharge coefficient of the gate is 0.98, as shown in Figure 7.8 and Figure 7.9 respectively.

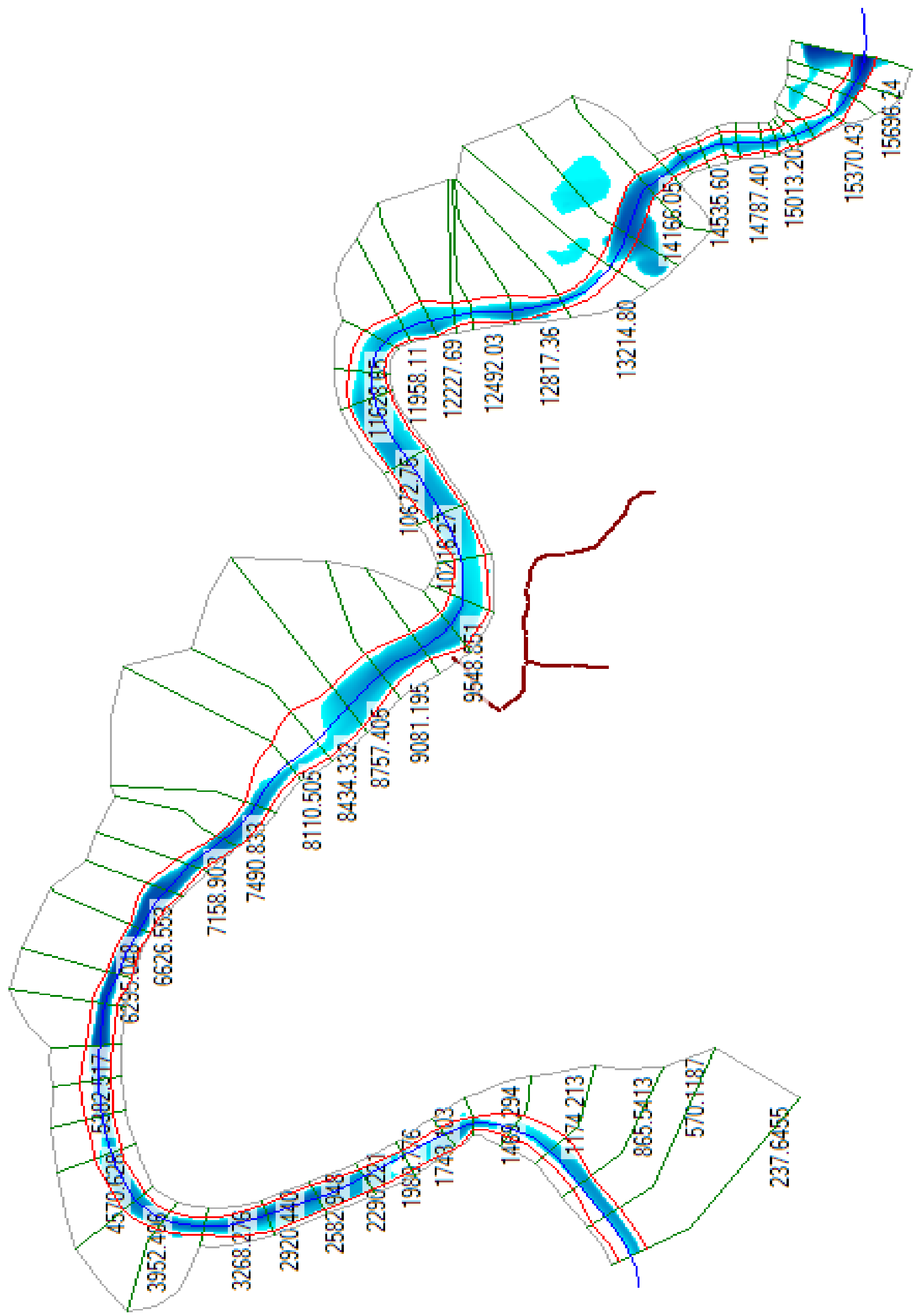


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Figure 7.5 One-Dimensional River Reach.

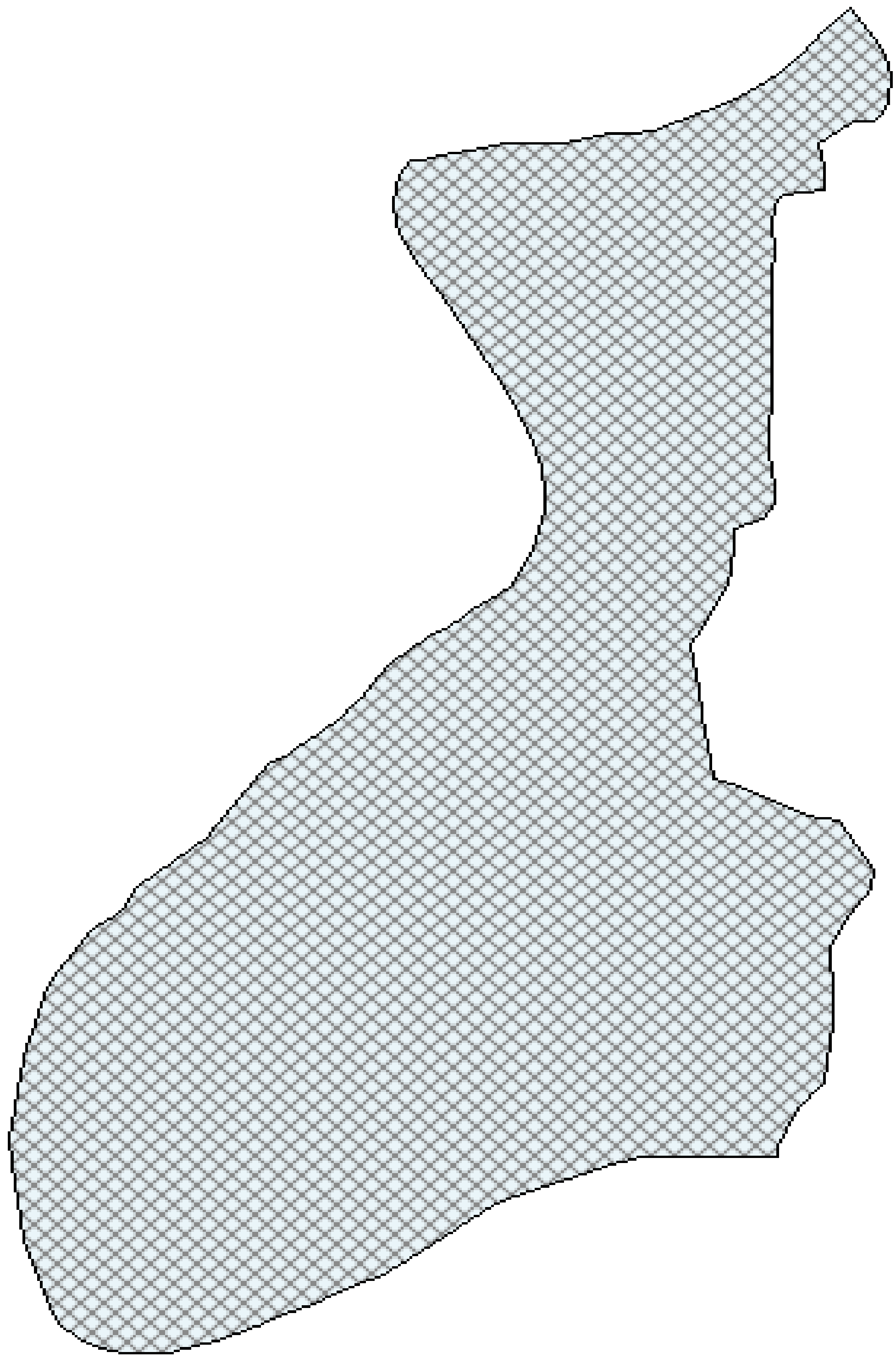

Figure 7.6 Finite Difference Grid System of the Model Area of Floodplain and Possible Inundation Area

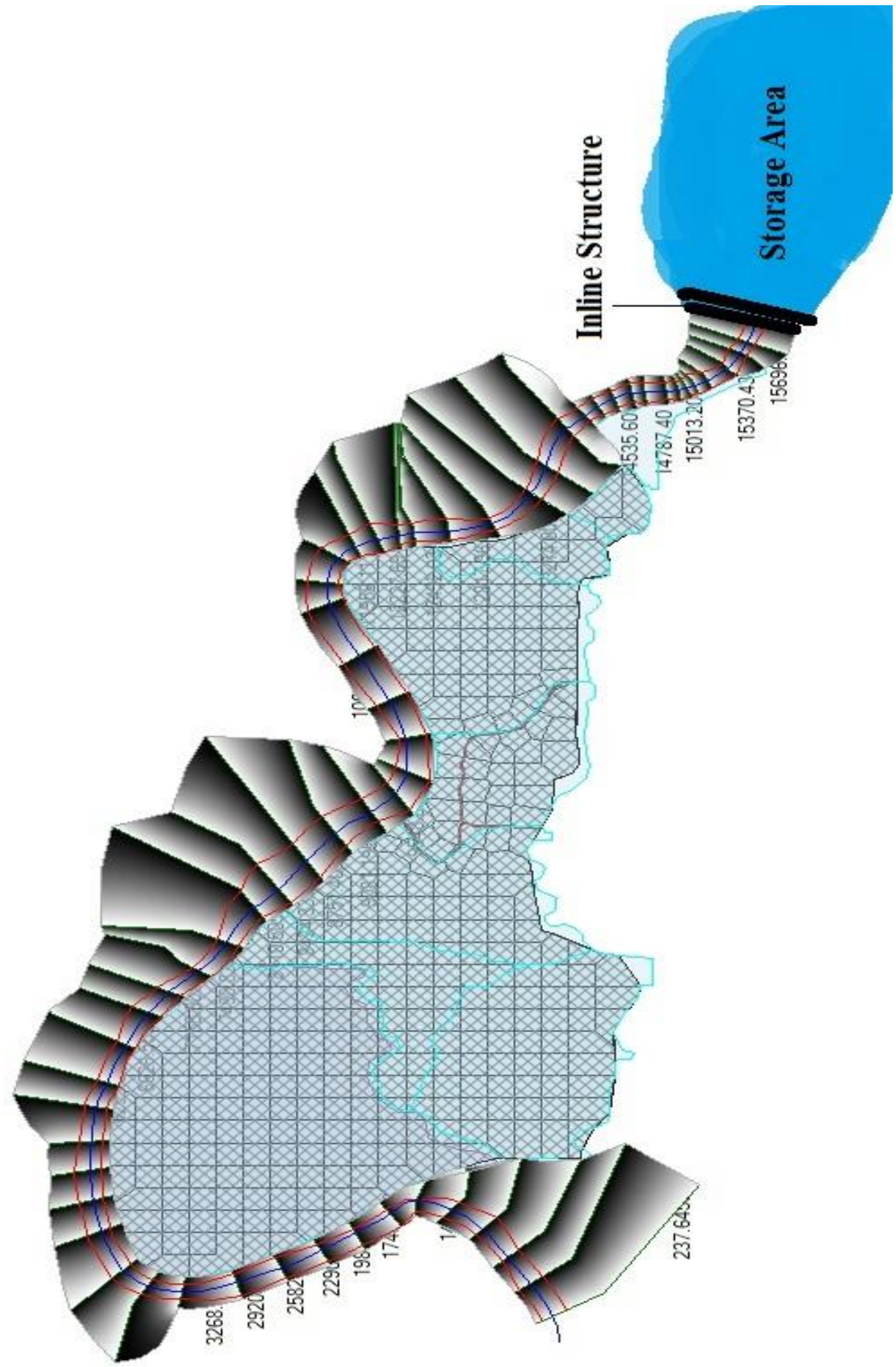

Figure 7.7 Storage Area Connected Through an Inline Structure

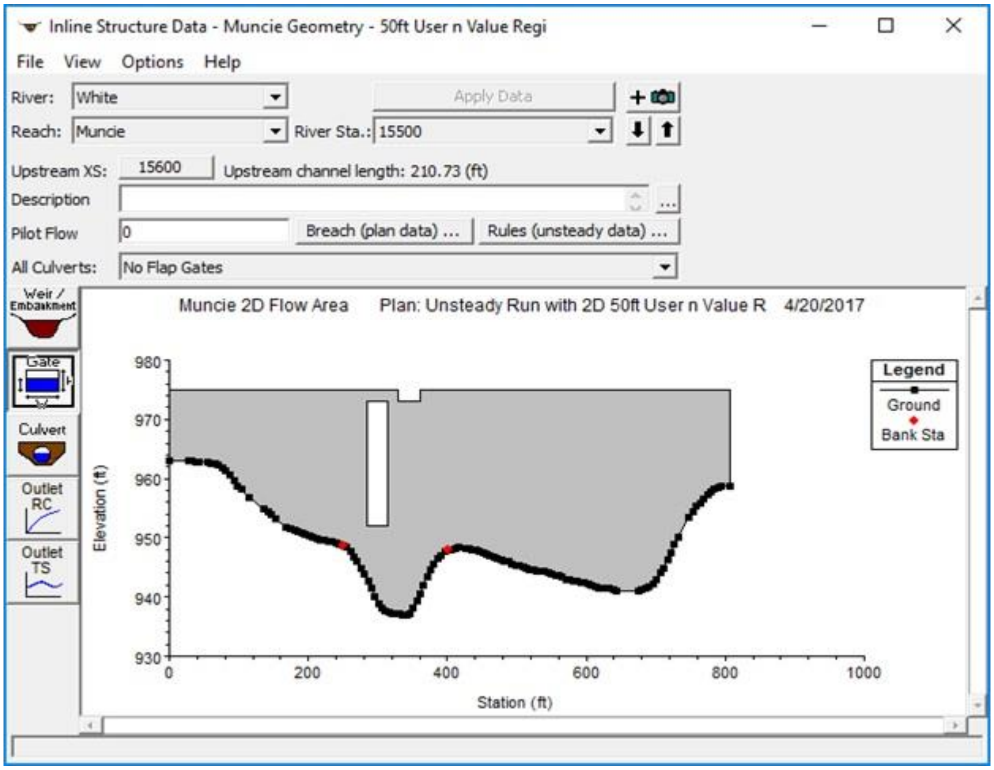

Figure 7.8 The inline regulating structure with gate specifications

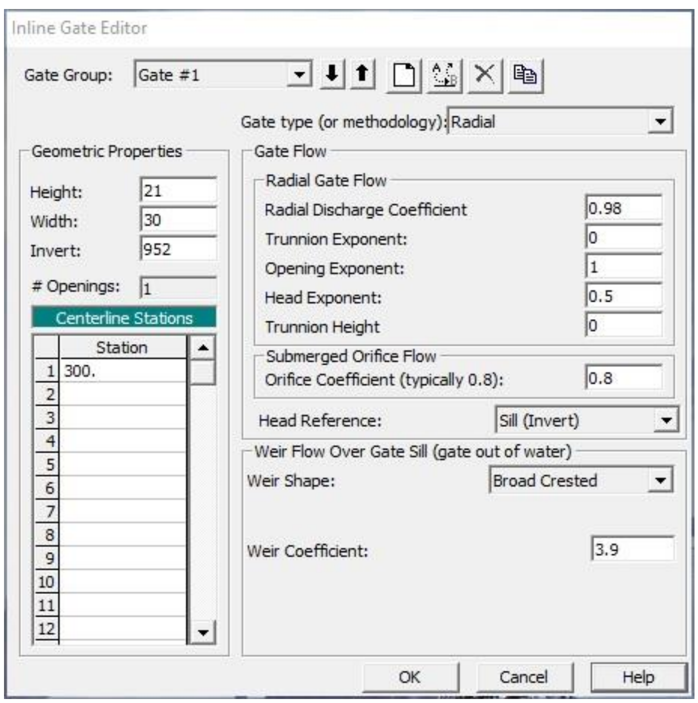

Figure 7.9 Gate Specifications

7.3 Two-Dimensional Area Terrain Model Development

The two-dimensional hydraulics model essentially requires a detailed and accurate terrain model in order to have accurate results and closer to reality. So, the terrain fitness (resolution) is the most important factor that can affect the hydraulic model results. There are many different sources, formats, and levels of details to obtain terrain data. Gridded data is what HEC-RAS uses to describe the terrain. The U.S. Geological Survey is a good source to download a digital elevation in ASTER GDEM form which is a product of METI and NASA. Another source to download a digital elevation model is the US Environmental Protection Agency through the model of Better Assessment Science Integrating Point & Nonpoint Sources (BASINS) which can download a digital elevation model in TIF format and then can be directly uploaded to the HEC-RAS mapper to create a terrain model. A user can collect data from multiple sources, create an accurate terrain model, and they can convert and export it into a gridded data format which can be processed in HEC-RAS. Creating a terrain model in HEC-RAS mapper before the user can perform his model computations that contain 2D flow areas is highly recommended, or even before the user can display two-dimensional or combined mapping results~~,~~ (G. Brunner, 2016). The user can also visualize the terrain layer as a background image in the HEC-RAS geometry editor. It is also recommended to create background images that assist~~s~~ the user in defining the 2D flow area boundaries. This allows the modeler to recognize the tops of levees, floodwalls, and any high ground that could act as a barrier to flow. Figure 7.10 shows the RAS Mapper with a terrain data layer of the hypothetical model.

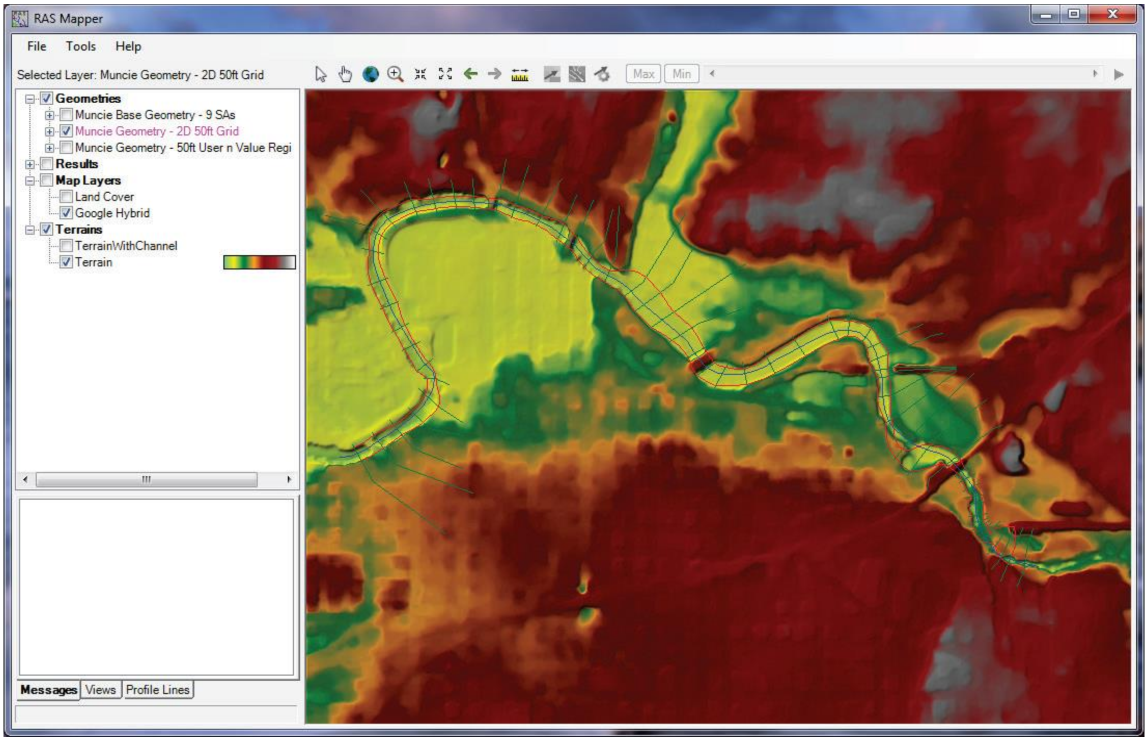

Figure 7.10 The RAS Mapper with the Terrain Data Layer of the Hypothetical Model

7.4 Creating Computational Mesh Dimensions

The movement of water through the two-dimensional area is governed by the computational mesh so that for each cell of the grid is going to be a single water surface elevation at each time step. Each cell in the grid may have several faces depends on where it is located within the grid. Normally, lock cells have a regular rectangular shape which also called structured cells, while the terminal cells take unregular shapes and named as unstructured cells as well, as shown in Figure 7.11. However, the movement of water flow between any adjacent cells depends on the cells faces. HEC-RAS processes the underlying terrain and the computational mesh before doing any hydraulics process to develop individual relationships for elevation–volume for each cell, and then detailed hydraulic aspects relations for every face of a cell in the grid such as roughness (Manning n), elevation vs. wetted perimeter and area.

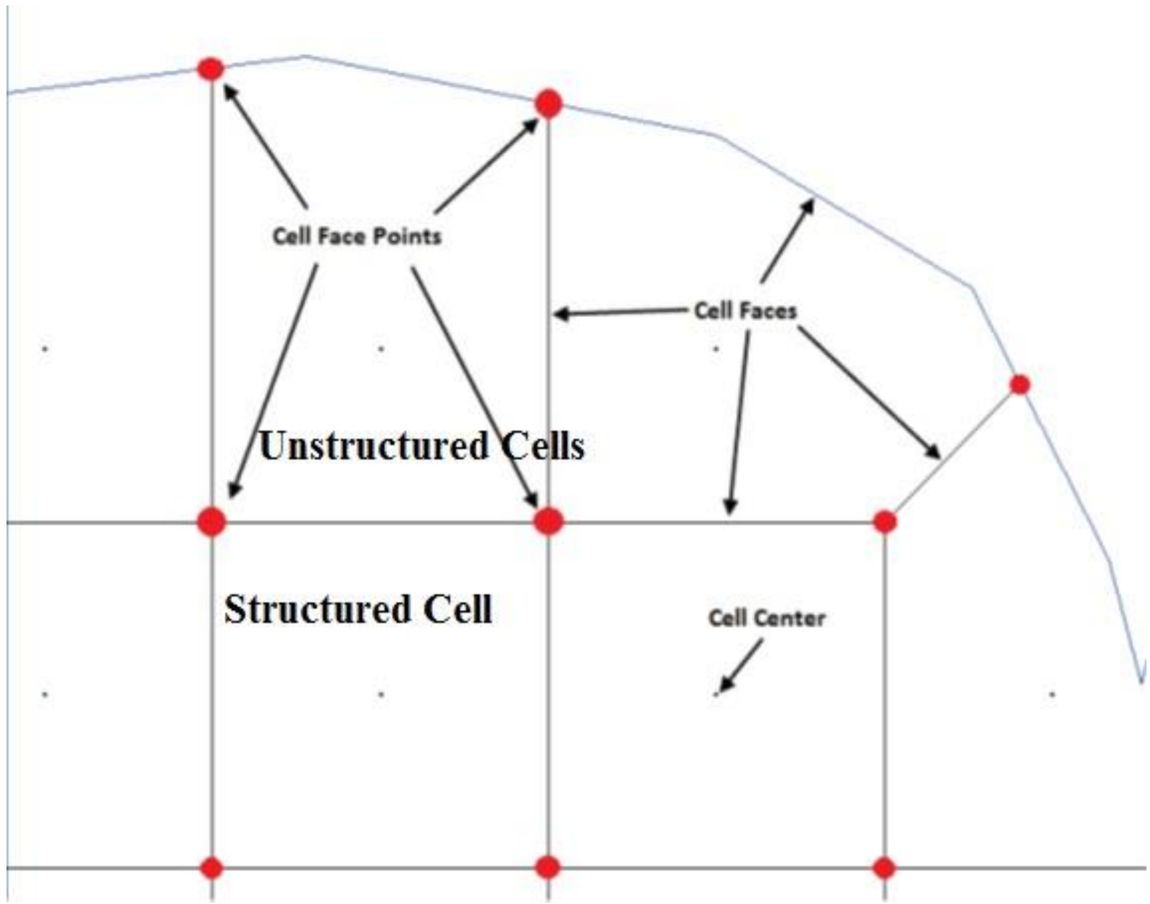

Figure 7.11 HEC-RAS 2D Modeling Computational Mesh Terminology, (Brunner, 2016)

7.5 Mesh Size Selection

The underlying terrain defines the hydraulic parameter in table form so that they will be used in water conveyance and storage but without considering the computational cell size. How big or small the cell size is still pawned with few important considerations and limits to decide where large coarse cells can be used versus the little small cells are needed. The two-dimensional modeling ability of HEC-RAS 5.0.3 uses a finite-volume solution scheme algorithm. The development of this algorithm allows for the users using structured and unstructured computational mesh. Furthermore, the computational mesh can be a mixture of three to eight-sided computational cells as HEC-RAS has a maximum of eight sides in a computational cell. However, the user will most likely select a simple grid resolution to use (e.g., 200 x 200 ft cells) as what adopted herein the hypothetical model, and the automated tools within HEC-RAS will build the computational mesh. However, the user can refine the grid after the initial mesh is built through break lines and the mesh editing tools, (Brunner, 2016).

Generally, the slope of the water surface is one of the important factors to define the cell size for a given area. The obstruction in another factor affects choosing the cell size in terrain as well. In other words, larger grid cell sizes are recommended where the slope of the water surface is nearly flat and changing gradually. For steeper slopes area, where the water surface elevation changes more rapidly, smaller grid cell sizes are required to monitor that rapid change in the water surface elevation. Because of the movement of flow depends on the cell faces of the computational cell, significant changes to geometry and quick changes in flow dynamics required to be defined by the smaller cells.

7.6 Hypothetical Model Application Input and Output

As mentioned above the system is modeled using the combined one and two-dimensional modeling approach. The river reach is modeled using one-dimensional flow, and the floodplain is modeled using the two-dimensional modeling approach based upon the diffusion wave model. A hypothetical reservoir is connected to the river reach at the very first upstream cross-section, through a gated spillway inline structure with large enough radial gate to allow passing a range of significant discharges for simulation process of a flooding event. A radial gate type is assumed to regulate the flow from the dam to the downstream channel. The total gate width is 30 feet long with a maximum opening of 21 feet, and the discharge coefficient of the gate is 0.98. The river channel has an initial steady flow of 1000 cfs.

The hypothetical inflow hydrograph to the reservoir is shown in Figure 7.12, which is used for this example application.

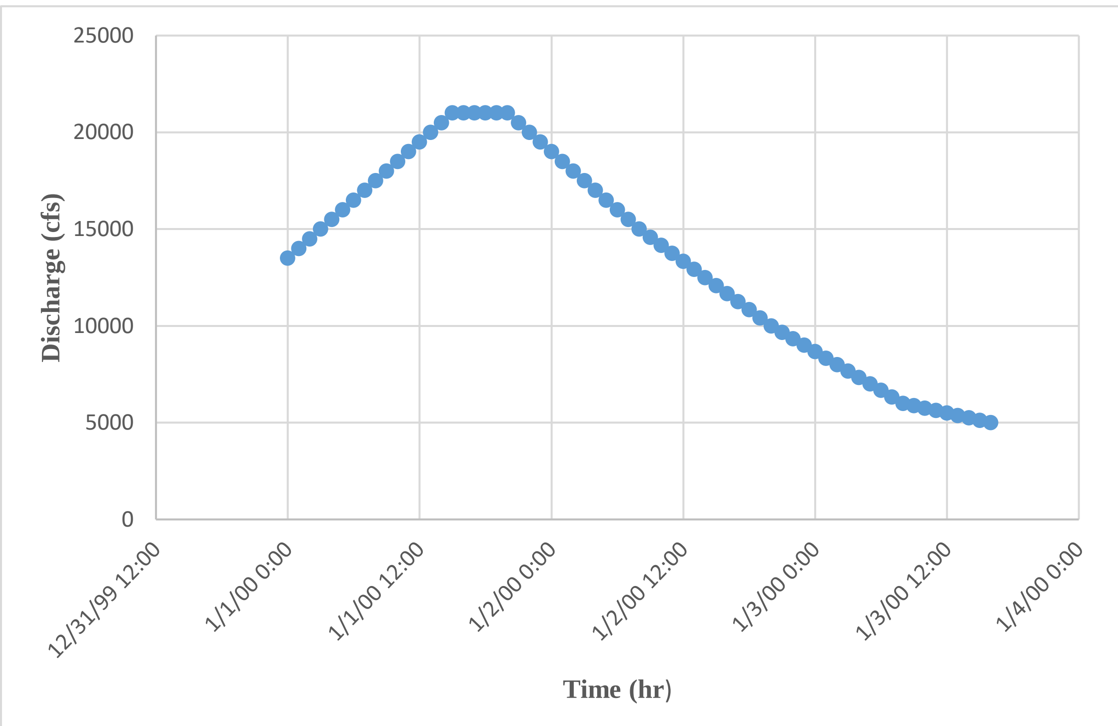

Figure 7.12 Reservoir Hypothetical Inflow Hydrograph

A computation interval Δt of one hour is used, starting at time t=0. To determine the optimal sequence of reservoir gate openings for the targeted downstream water surface elevations, an adequate theoretical optimization scheme should be presented and the potential constraints that must be satisfied during a flooding event. These time series of gates openings should achieve downstream water surface elevations within the allowable range of elevations at the points of interest, which is here in the model represented by Muncie City. So, the problem objective is to minimize the flow rates at the city of Muncie, while satisfying all hydraulic, hydrological and operational constraints during the entire period of flooding event simulation. The input parameters that have been used the simple model are listed in Table 7.2, and the inflow hydrograph shown in **Error! Reference source not found.** is applied as hypothetical inflow hydrograph to reservoir pool for 64 hours with Δt of one hour.

Table 7.2, Hypothetical Model Reservoir Input Parameters

Parameter	Value
Max reservoir level	974 ft
Min reservoir level	952 ft
Max reservoir discharge	14500 cfs
Max water surface elevation at Muncie	950 ft

On the other hand, The optimization-simulation model is responsible for keeping the storage level above the inactive storage (minimum water surface elevation), and below the maximum flood storage, also the stage at the control point (Muncie city) must always be under the flood stage, 950 ft. This phase includes the reservoir operation model with global optimization model solved by the genetic algorithm solver in the MATLAB program. So that, each optimization iteration results, which is here the gate opening as decision variable and hence the released discharges, can be simulated and routed downstream to the control point by HEC-RAS 5.0.3. That is to check and satisfy the hydraulic constraints downstream.

The process of simulation starts at t=0, where the initial level of the storage area was set to be as 971 ft above the sea level.

After that, the forecasting starts at t=5 h. The hypothetical hydrographs of watershed outflow which is supposed to be computed by the HEC-HMS used as reservoir inflow hydrograph. The data exchanges among the submodels though MATLAB code. At this time the reservoir operation model link with GA in MATLAB starts generating the possible solution for determining the reservoir releases over next Δt. The reservoir operation model and the genetic algorithm solver succeeded to keep the reservoir storage within the desirable range, which the storage for the reservoir was kept above the inactive storage and below the maximum flood storage, thus preventing any potential dam failure. The time series of the gate openings of the inline structure is depicted in Figure 7.13, with these results.

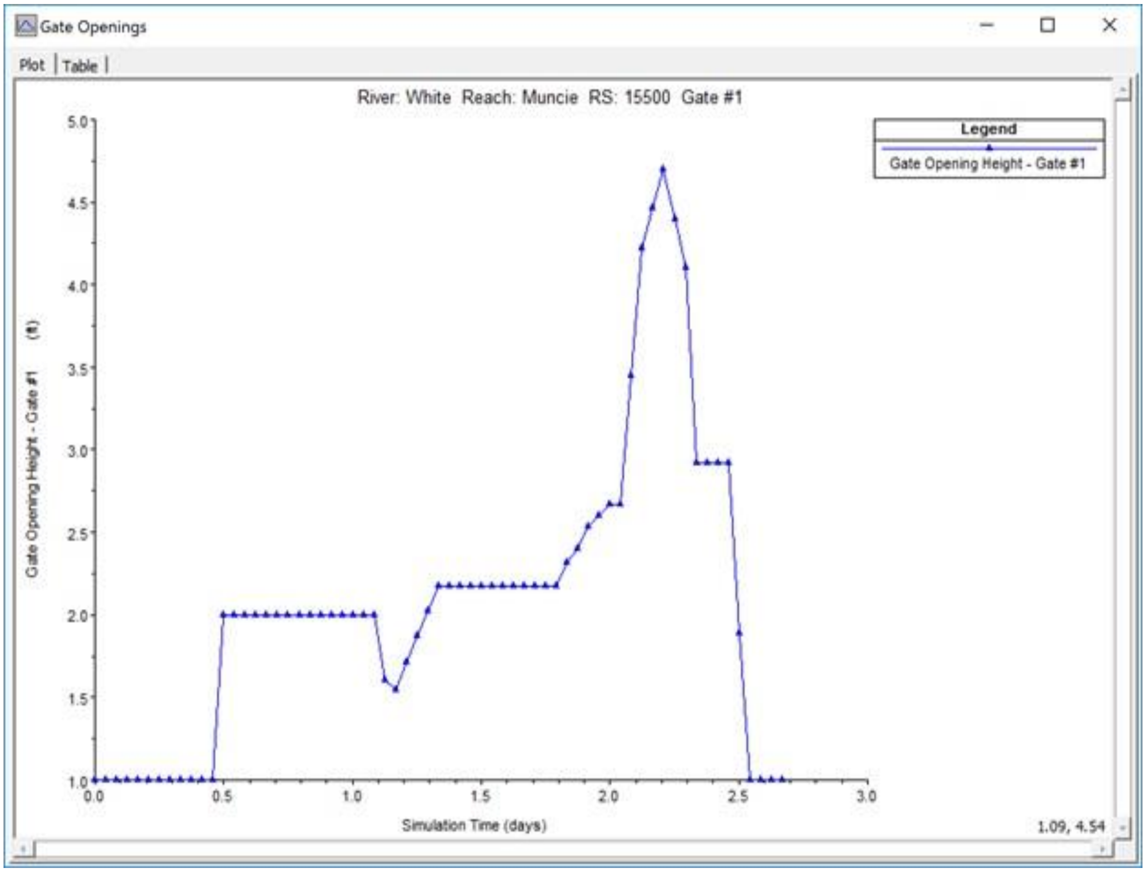

Figure 7.13 Time Series of The Gate Openings

The reservoir operation model and the genetic algorithm solver succeded to keep the reservoir storage within the desirable range, which means the storage for the reservoir was kept above the inactive storage and below the maximum flood storage, thus preventing any potential dam failure, Figure 7.14. Moreover, the stages at the control point Muncie city were under the flood stage at all time, and the area of inundation before and after the add the reservoir depicted in Figure 7.15 and Figure 7.16 respectively.

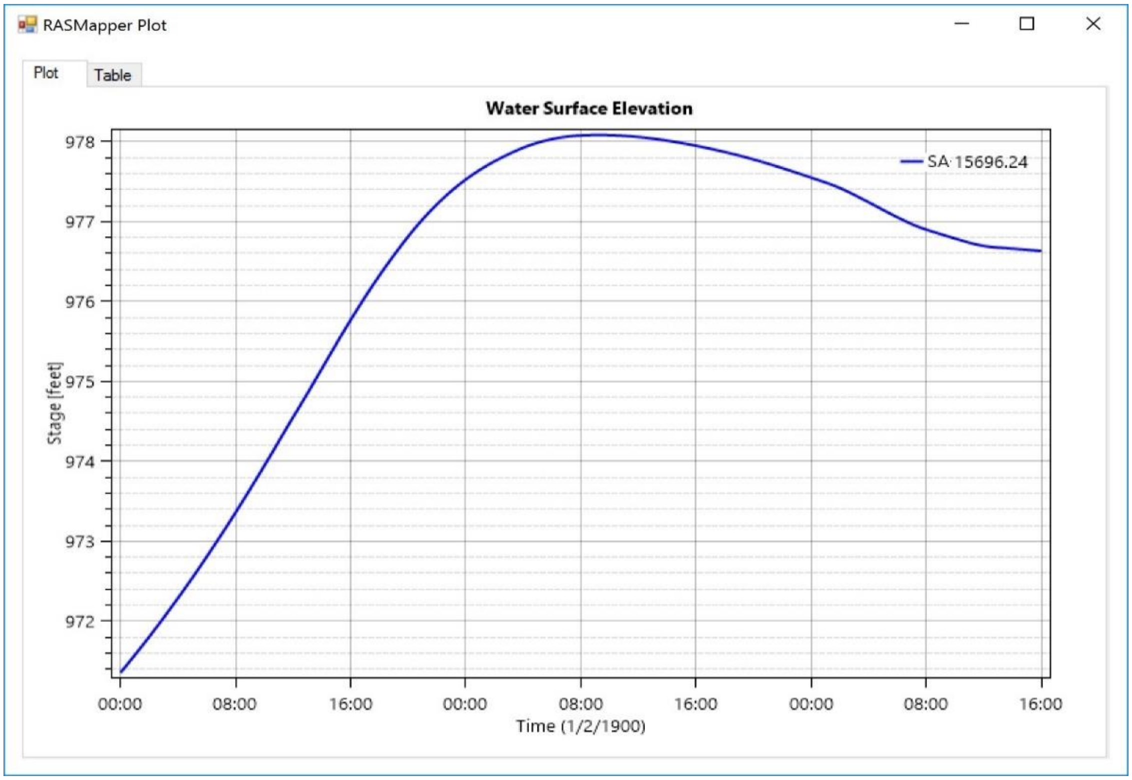

Figure 7.14 Storage Area Stages Time Series

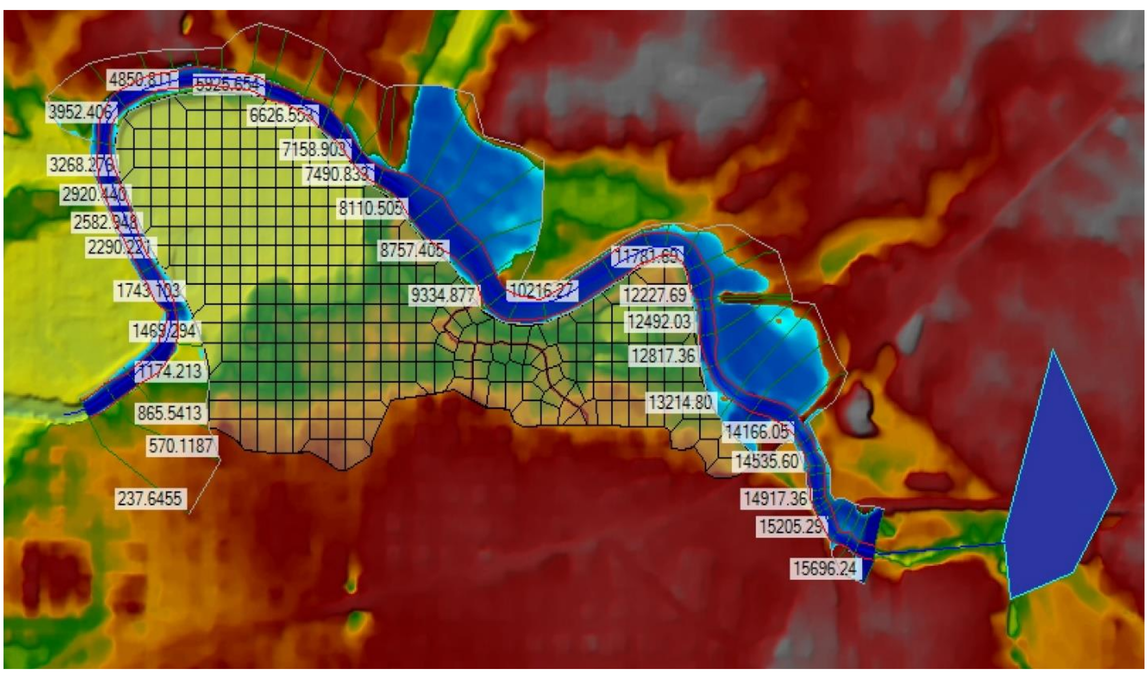

Figure 7.15 Maximum Simulated Stage at Muncie City Using the optimization-Simulation Model

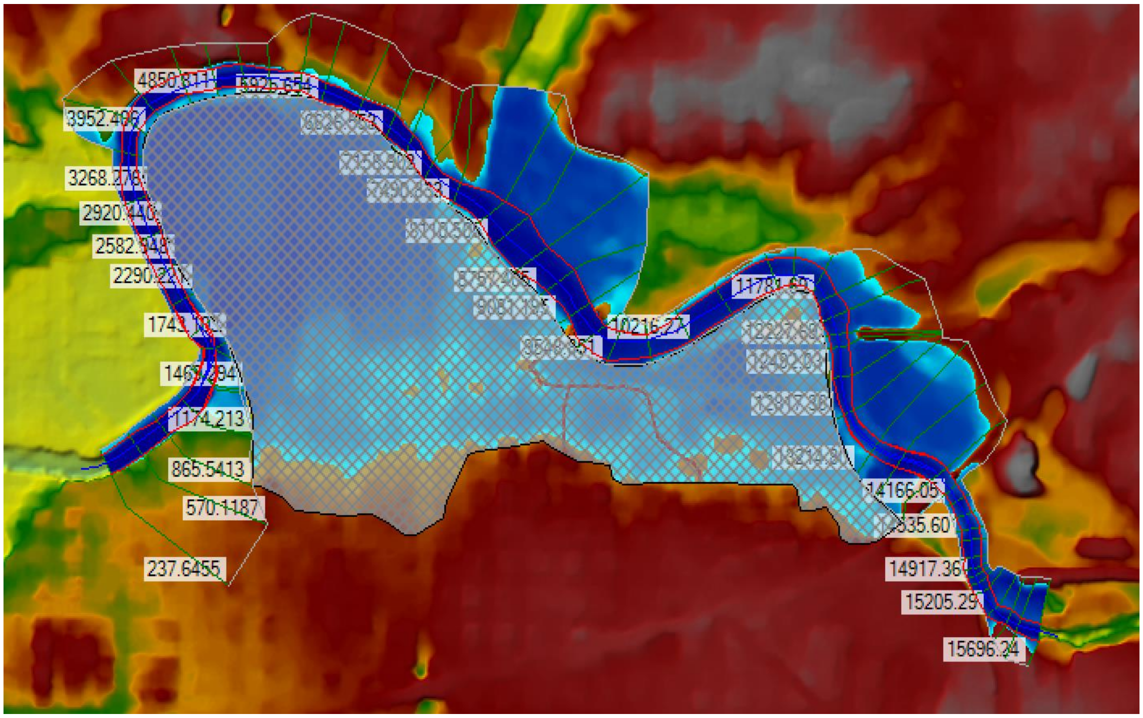

Figure 7.16 Maximum Simulated Stage at Muncie City without Using the Optimization-Simulation Model

CHAPTER 8 THE STUDY AREA AND THE FLOOD EVENT

8.1 May 2010 Flooding Event at Nashville

The application of the complete optimization-simulation model was applied to a portion of the Cumberland River System in the vicinity of Nashville Tennessee, Figure 2.10 and Figure 8.1, for the flooding event that occurred during May 2010. This event caused severe flooding in Nashville, surrounding areas. Many lives and home were lost and ruined, but Nashville rose above the tragedy. The rainfall occurred on the weekend of May 1-2, 2010. According to the National Weather Service, 13.57 inches of rain was recorded in just a 36 hour period. The rainfall total doubled the previous 48-hour rainfall record in Nashville. Twenty-six lives passed away; as a result, the flooding in Tennessee and Kentucky, including 11 in the Nashville area. Final, body of a victim, Danny Tomlinson of Pegram found Sept. 26, 2010, about five months after the flood. Nearly 11,000 properties were damaged or destroyed in the flooding, and 10,000 people were displaced from their homes. The flood caused over $2 billion in private property damage and $120 million in public infrastructure damage in Nashville. One year after the event, The Tennessean, citing the Nashville, TN Area Chamber of Commerce, reported that 300 to 400 businesses stayed closed and 1,528 jobs were "very unlikely" to return, (Grigsby, 2015).

A significant weather system brought torrential rains and severe thunderstorms on Saturday, May 1st through Sunday morning, May 2nd. A stalled frontier accompanied with very humid air streaming northward from the Gulf set the stage for repeated rounds of rain. Numerous locations along the I-40 corridor across western and middle Tennessee reported more than ten to fifteen inches, with some places receiving up to twenty inches per Doppler radar estimates, (NWS, 2010).

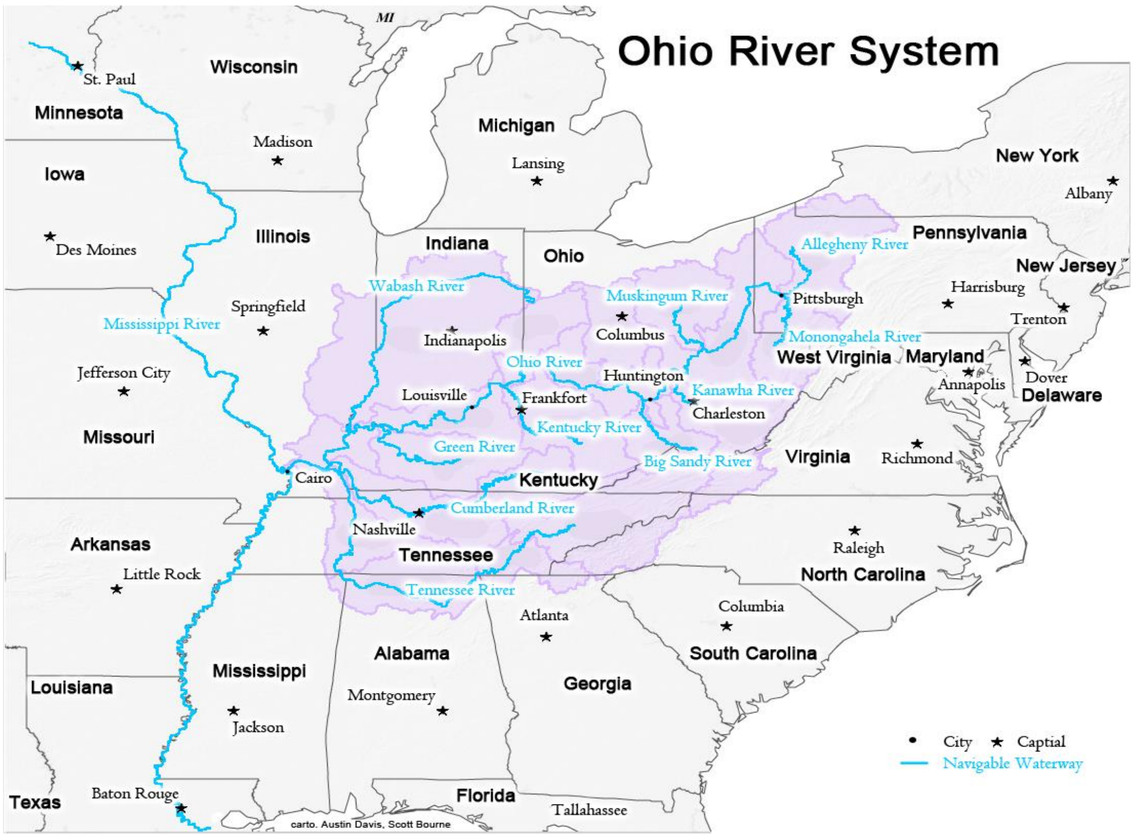

Figure 8.1 Ohio River System (U.S. Environmental Protection Agency's, 2017)

8.2 Cumberland River Basin

The Cumberland River, Figure 2.10, is a major river in the Ohio River Basin. The 688-mile-long (1,107 km) river drains almost 18,000 square miles (47,000 km^2) of the south of Kentucky and north-central Tennessee. The river flows to the west from the Appalachian Mountains to its confluence with the Ohio River near Paducah, Kentucky, and the mouth of the Tennessee River. Major tributaries include the Obey, Caney Fork, Stones, and Red rivers, (USGS, 2011).

The Ohio River starts at the confluence of the Monongahela and Allegheny Rivers near Pittsburgh, Pennsylvania. The Ohio River flows along the borders of states: Indiana, Illinois, Kentucky, West Virginia, and Ohio to its confluence with the Mississippi River at Cairo, Illinois. Figure 8.1 illustrates the Ohio River Basin. 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 mi in long and has a total drainage area of about 204,000 mi2 converging parts of 15states. The Cumberland River enters the Ohio River 58 miles upstream of its junction with the Mississippi River as seen in Figure 8.1, (U.S. Environmental Protection Agency's, 2017).

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 Figure 2.10, lies entirely within the states of Kentucky and Tennessee and has a total area of 17,914 mi^2 miles, of which 10,695 square miles (60%) are in the state of Tennessee. The topography of the Cumberland River Basin changed from rugged mountains in the eastern upstream portion to rolling low plateaus in western, or downstream, sector. Elevations range from 4150 ft above mean sea level (MSL) in the Cumberland Mountains to 302 ft in the pool at the start of the river (USACE, 2010c and 2012).

Five projects on the Cumberland River mainstream are maintained and operated by the U.S. Army Corps of Engineers Nashville District, plus five other projects on the tributaries. The mainstream projects are the Cordell Hull, Barkley, Cheatham, Old Hickory, and Wolf Creek. Wolf Creek and Barkley are only congressionally authorized in terms of flood risk management, while Congress authorizes Barkley, Cheatham, Old Hickory, and Cordell Hill for hydropower generation 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 8.2 and Figure 2.10, illustrates the current U.S. Corps of Engineers projects in the Cumberland River Basin, and Table 8.1 summarizes of current purposes of these congressionally authorized projects.

The system for control of the Cumberland River and its tributaries is comprised of ten dams, five on the main stem, and the other five are on the tributaries. All of them produce hydropower, except the Martin's Fork Dam. Four of the projects have navigation locks, and six do not. All the projects enhance the water supply of the Cumberland River Basin. However, the U.S. Congress for water supply purposes specifically authorizes none. All projects contribute to improving water quality, but only Martin's Dam, specifically permitted for water quality improvement. The entire Corps' projects in the Cumberland provide recreation, fish, and wildlife enhancement. Only six dams have been authorized for flood control purposes. The storage reservoirs of Wolf Creek, Dale Hollow, Center Hill, and J. Percy projects provide 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 impounded by the 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).

Table 8.1 Currently Congressionally Authorized Projects Purposes (U.S. Army Corps of Engineers, 2010a)

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

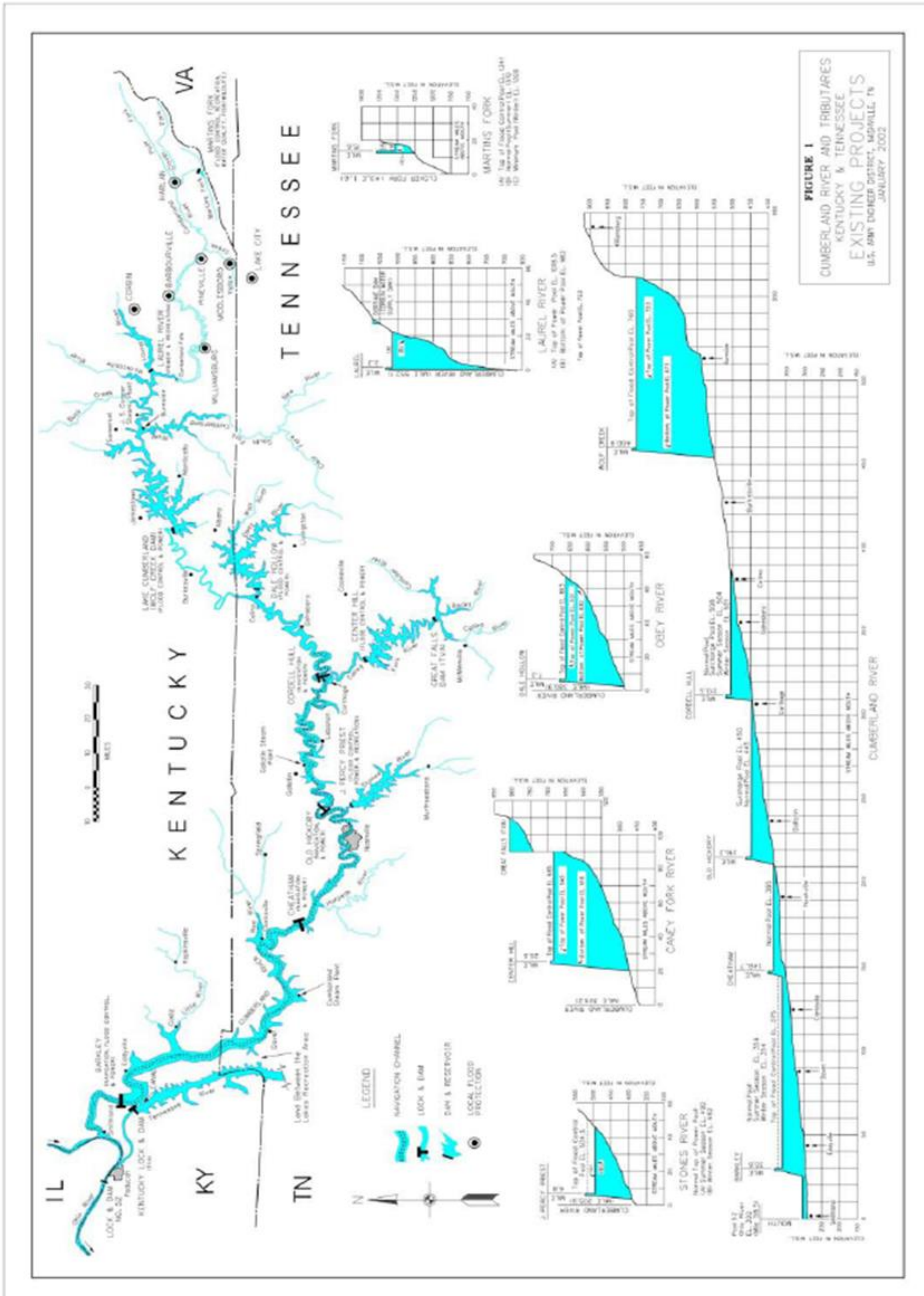

Figure 8.2 U.S. Army Corps of Engineers' Projects in the Cumberland River Basin (U.S. Army Corps of Engineers, 2010a)

Cumberland lake is designed to retain over 6.5 inches of rainfall runoff within its flood control pool elevation. During the early flood season (springtime), 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 is located 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 flooding at Celina, Tennessee. Center Hill Lake contains 15% of the Cumberland River Basin's flood storage capacity.

The main control point for flood control 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 have sufficient storage 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 inflows, which significantly exceed damage levels, mainly on the lower parts of the river.

Reservoirs continue to store incoming upstream floodwaters during 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 less than in the upper portions of the river. For instance, early parts of a flood may exceed flood levels before upstream discharge reductions become more efficient 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 efficient for flood control purposes. A new philosophy and new approaches for flood control are therefore necessary to response to potential future extreme rainfall and consequential flooding events. The occurrence of these extreme events will become more frequent and more intense primarily due to climate change.

8.3 Rainfall and Flood Event in May 2010

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

8.3.1 Antecedent Moisture Conditions

In most cases, an 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 8.3 to Figure 8.7 are the high-resolution precipitation images 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 8.2 for the 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 8.2 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 8.2 shows the previous rain event restored the area to normal condition, and the antecedent conditions were irrelevant due to the massive amount of rainfall which followed on May 1st and 2nd.

Table 8.2 Cumberland River Basin Project Drainage Basin Rainfall/Runoff Values, (U.S. Army Corps of Engineers, 2012)

Drainage	Rainfall (in.)			Runoff (in.)		
Basin	Observed	Normal	Difference	Observed	Normal	Difference
Barkley L&D						
March	3.46	4.96	-1.50	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	0.70	1.26	-0.56
Cheatham L&D						
March	3.87	5.30	-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	3.50	2.10	1.40
June	3.08	4.19	-1.11	0.42	1.02	-0.60
J. Percy Priest Dam						
March	3.20	5.57	-2.37	2.10	3.65	-1.55
April	2.08	4.18	-2.10	0.55	2.20	-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
Old Hickory L&D						
March	3.11	5.35	-2.24	1.67	3.40	-1.73
April	3.42	4.10	-0.68	1.26	2.55	-1.29
May	12.86	5.17	7.69	5.14	2.01	3.13

June	3.35	4.30	-0.95	0.41	0.98	-0.57
<u>Center Hill Dam</u>						
March	3.60	5.94	-2.34	2.21	3.77	-1.56
April	2.12	4.41	-2.29	1.20	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	5.20	-2.15	1.52	3.44	-1.92
April	3.22	4.01	-0.79	1.33	2.62	-1.29
May	11.40	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	-0.20	0.38	0.90	-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	5.10	2.01	4.25	2.00	2.25
June	4.29	4.47	-0.18	0.26	1.07	-0.81

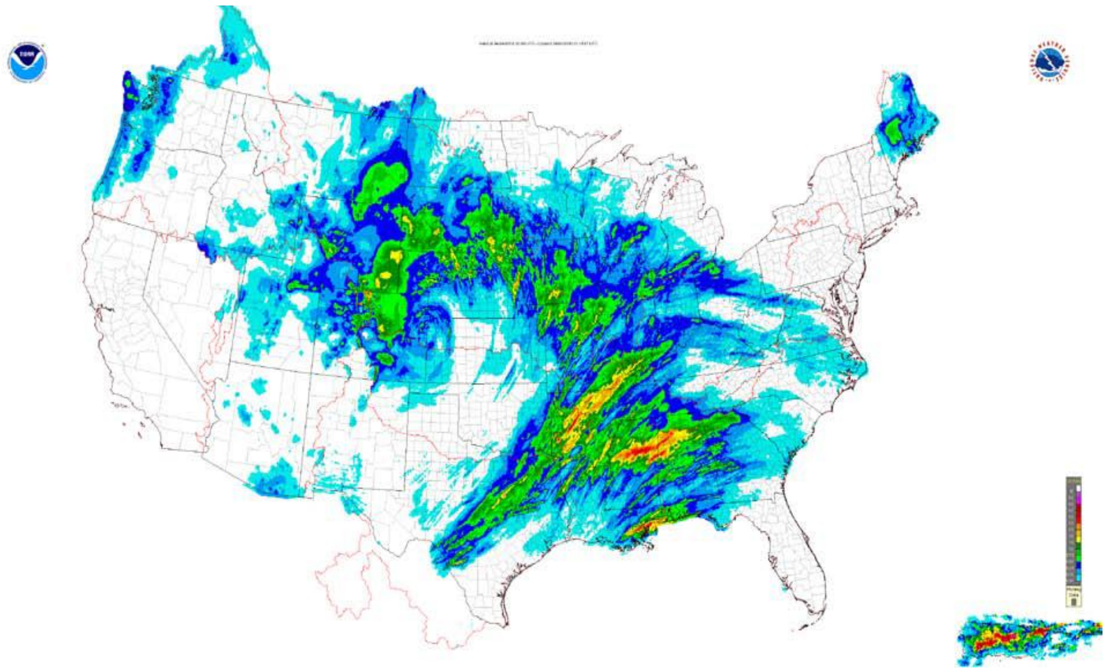

Figure 8.3 Composite High-Resolution Precipitation Image at April 24, 2010, 12:00 UTC (U.S. Army Corps of Engineers, 2012)

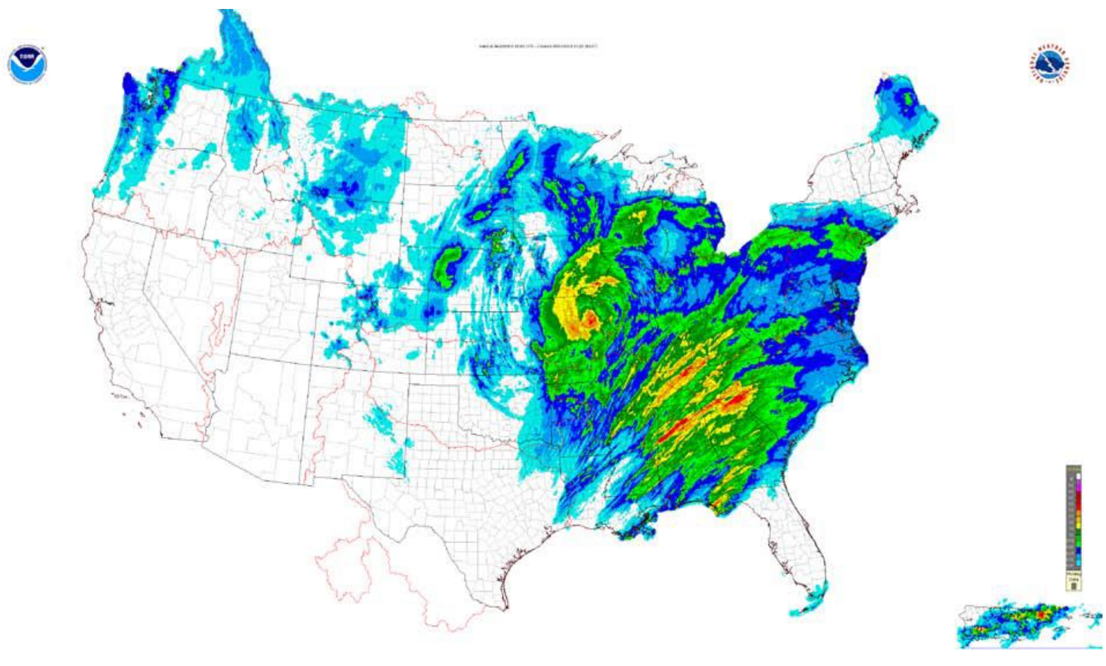

Figure 8.4 Composite High-Resolution Precipitation Image on April 25, 2010, 12:00 UTC (U.S. Army Corps of Engineers, 2012)

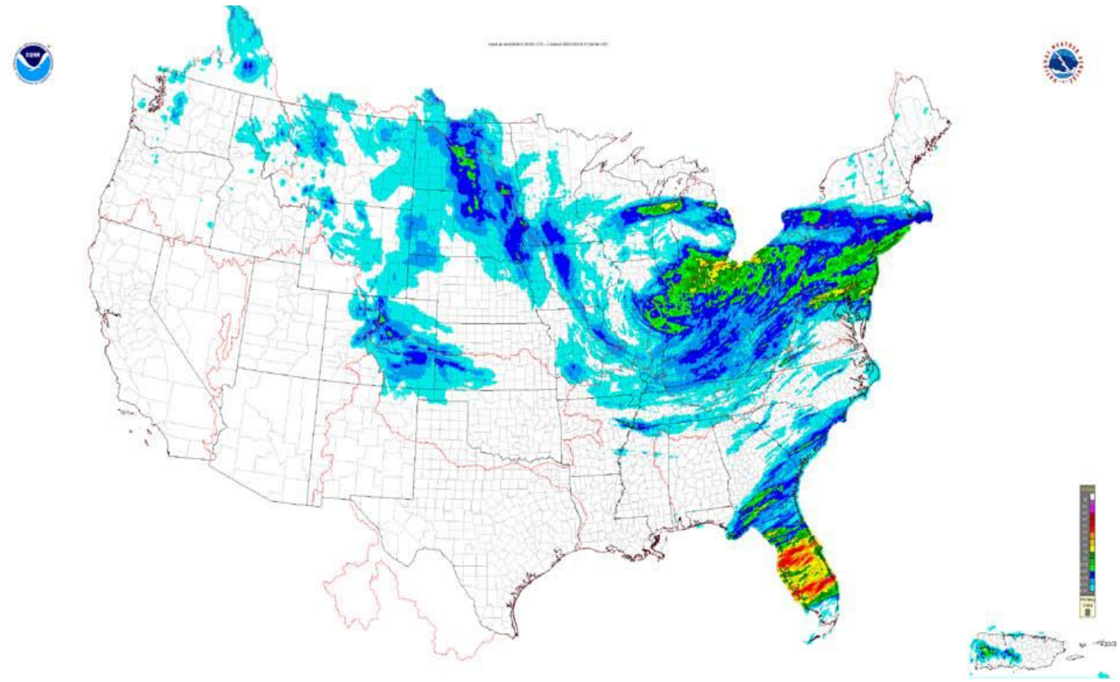

Figure 8.5 Composite High-Resolution Precipitation Image on April 26, 2010, 12:00 UTC (U.S. Army Corps of Engineers, 2012)

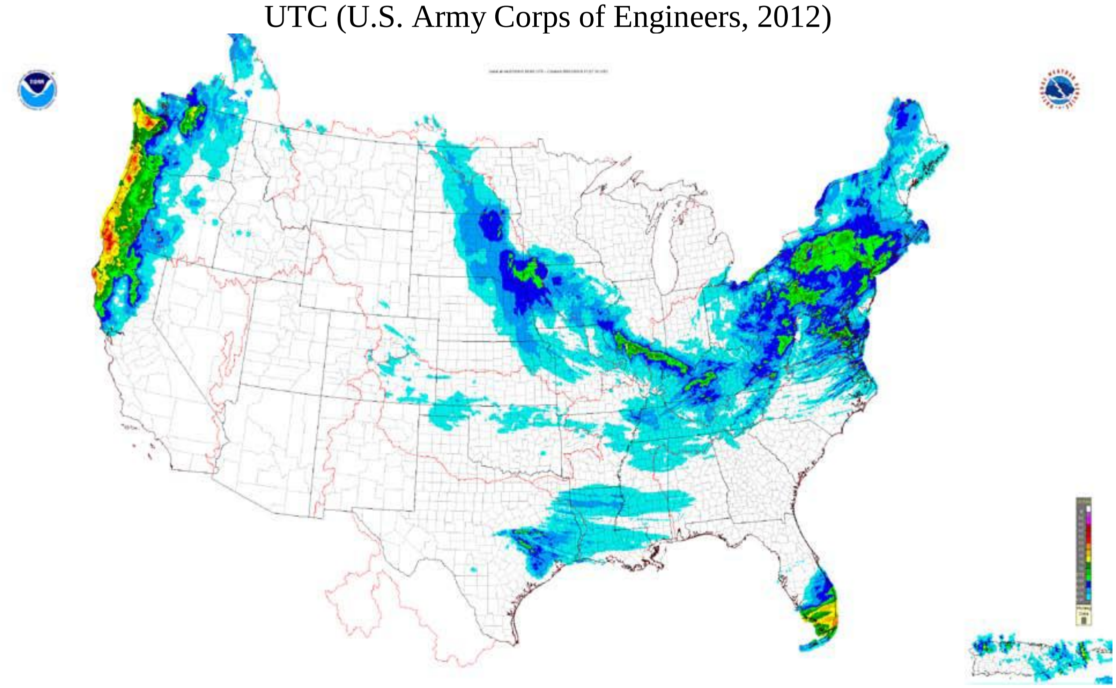

Figure 8.6 Composite High-Resolution Precipitation Image on April 27, 2010, 12:00 UTC (U.S. Army Corps of Engineers, 2012)

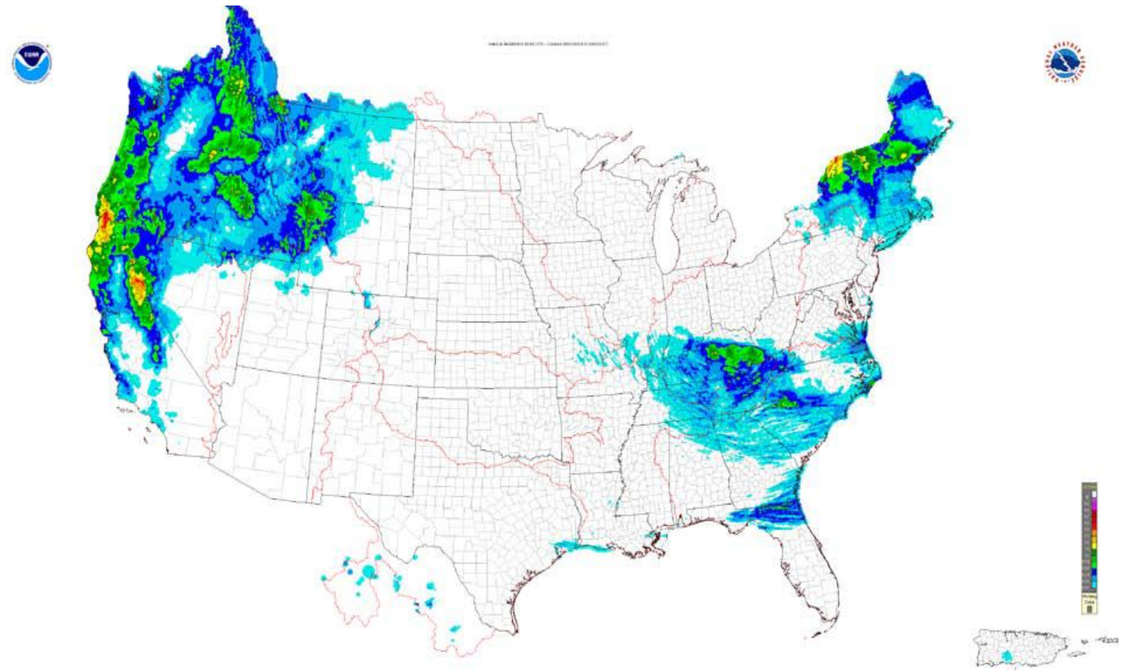

Figure 8.7 Composite High-Resolution Precipitation Image on April 28, 2010, 12:00 UTC (U.S. Army Corps of Engineers, 2012)

8.3.2 Meteorological Conditions

Weather disturbances in the mid-level 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 historical precipitation and flooding across Tennessee and Kentucky. Primary factors that contributed 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 unseasonable maintained 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 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 the second round of heavy rain and intense thunderstorm activities on May 2nd, 2010. Figure 8.8 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).

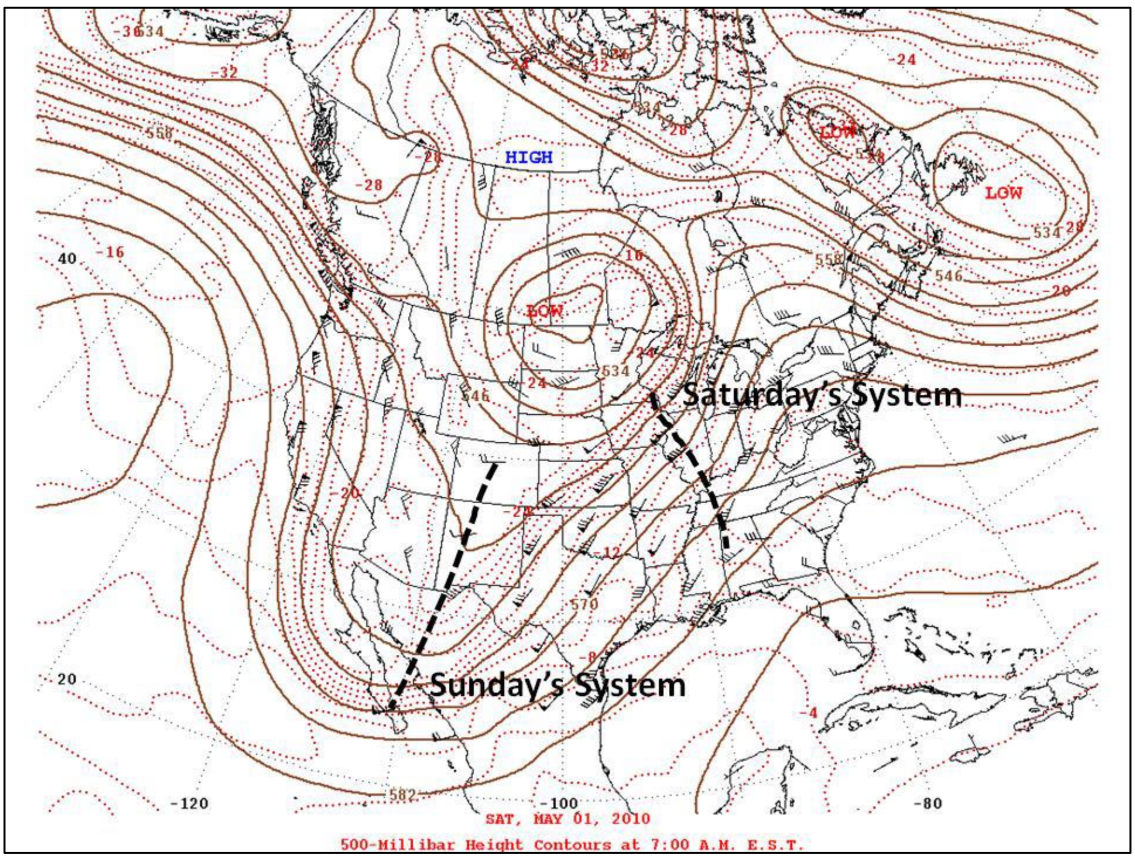

Figure 8.8 Upper Air Chart Showing Flow and Disturbances at Approx. 18000 Ft. AGL, May 1st, 7:00 a.m. (Service, 2011)

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 was illustrated in Figure 8.9. 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).

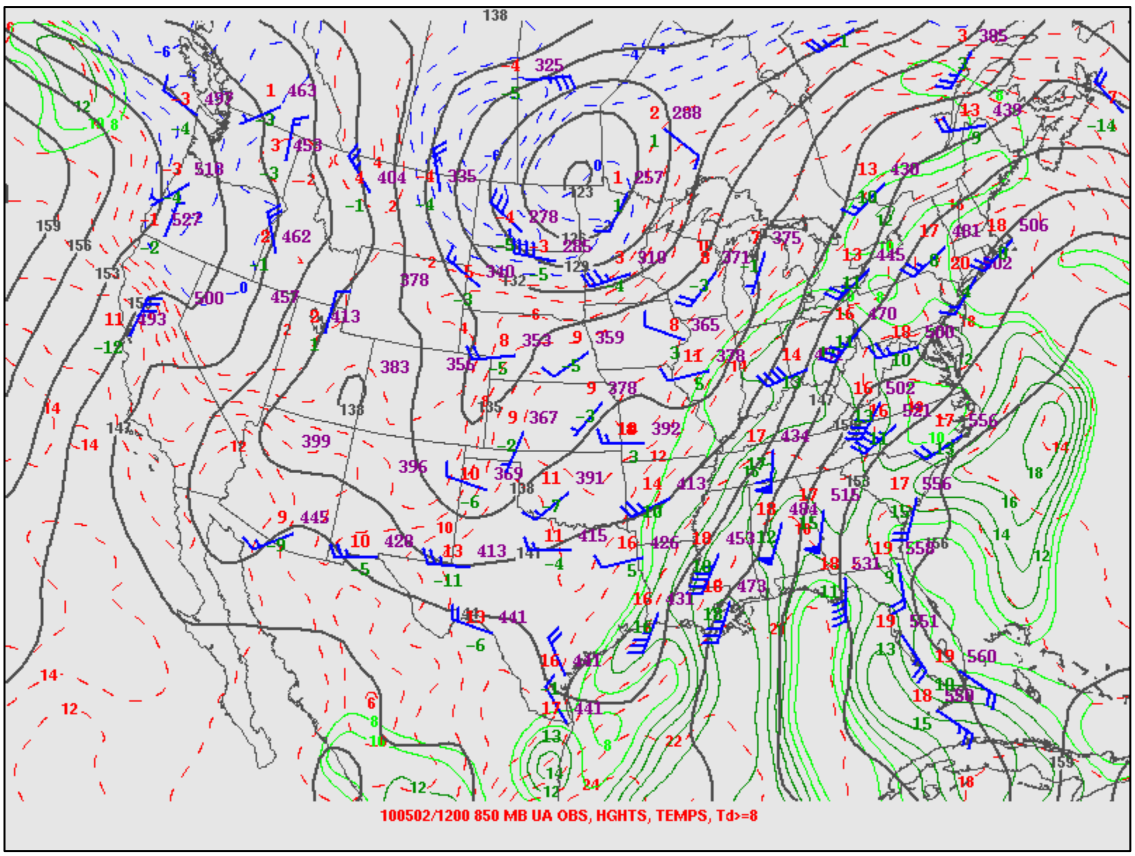

Figure 8.9 Lower Levels Atmosphere Showing Moisture Transport (Green Lines) at Approx. 5000 Ft. AGL, May 1st, 7:00 a.m. (Service, 2011)

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 8.10 and Figure 8.11 illustrate the total spatial precipitation data in the Cumberland River Basin on May 1st and May 2nd, 2010; and Figure 8.12 shows the total rainfall received over the two days, (U.S. Army Corps of Engineers, 2012).

Hourly and accumulative rainfall data at the Nashville International Airport are shown in Figure 8.13. 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-hour 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 19.41inches. Figure 8.13 also depicts the resultant river level rise (the brown curve) on the Cumberland River at Nashville, Tennessee.

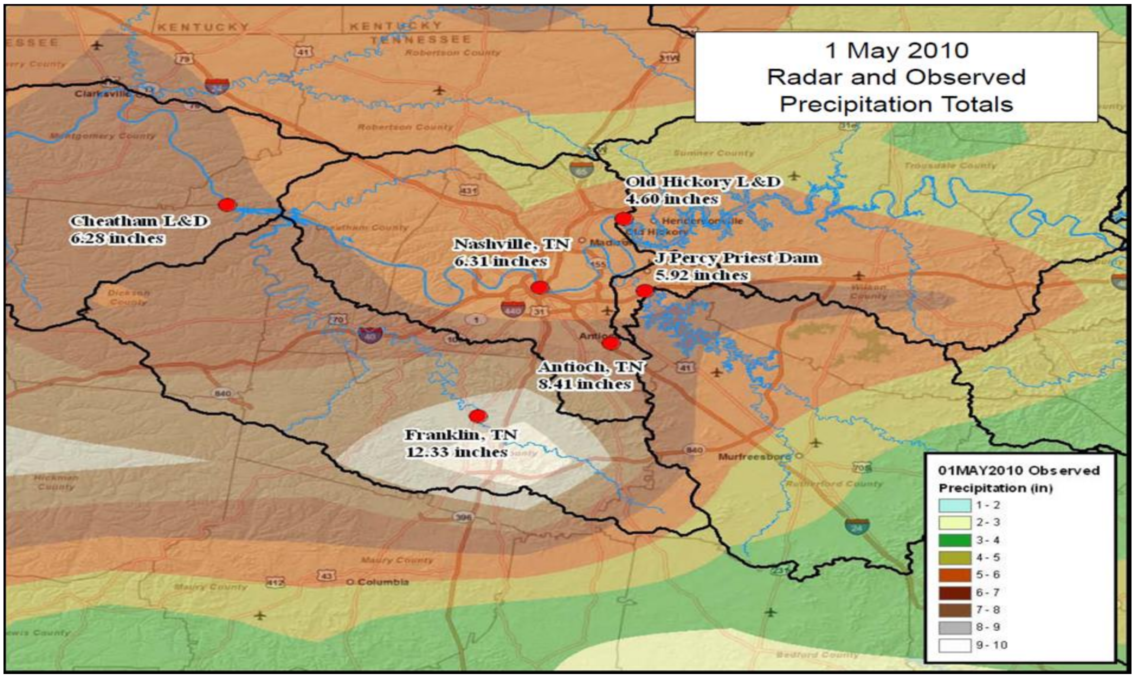

Figure 8.10 Total Precipitation Data in the Cumberland River Basin on May 1st, 2010, (U.S. Army Corps of Engineers, 2012)

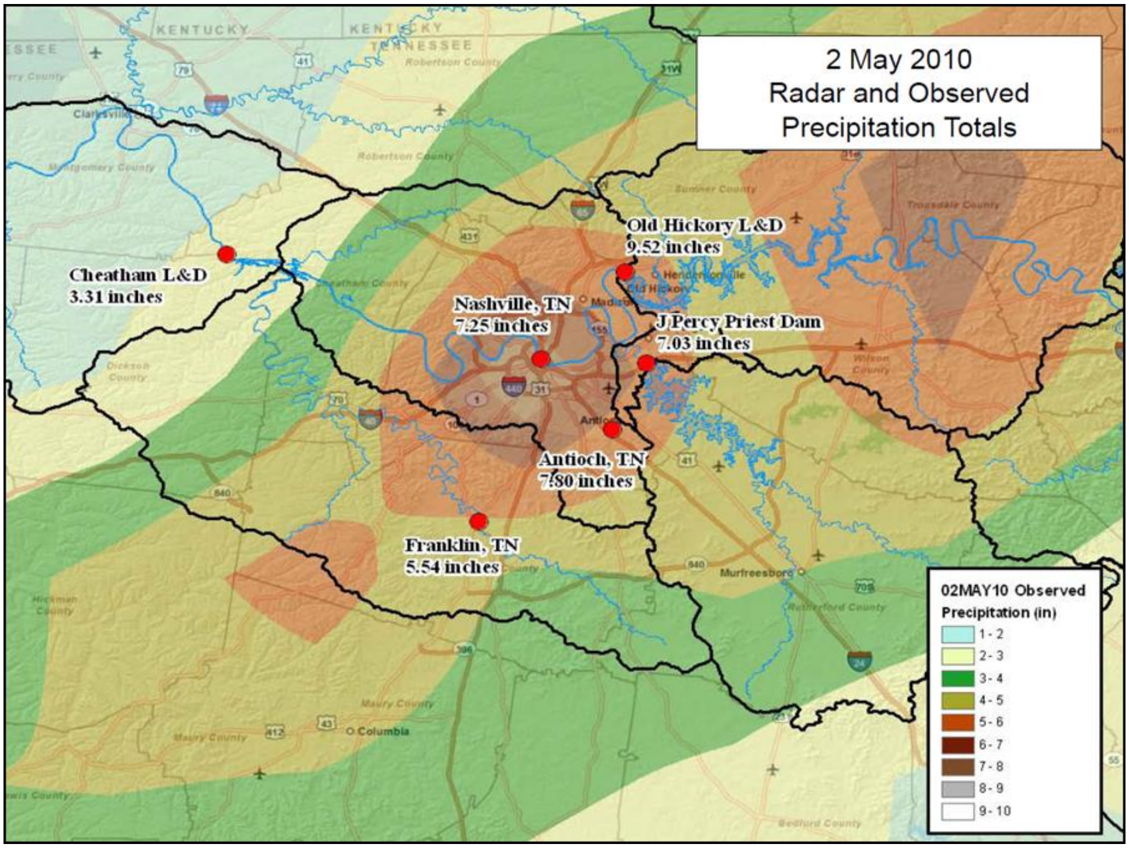

Figure 8.11 Total Precipitation Data in the Cumberland River Basin on May 2nd, 2010, (U.S. Army Corps of Engineers, 2012)

As seen in Figure 8.13, the flood crest of 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.

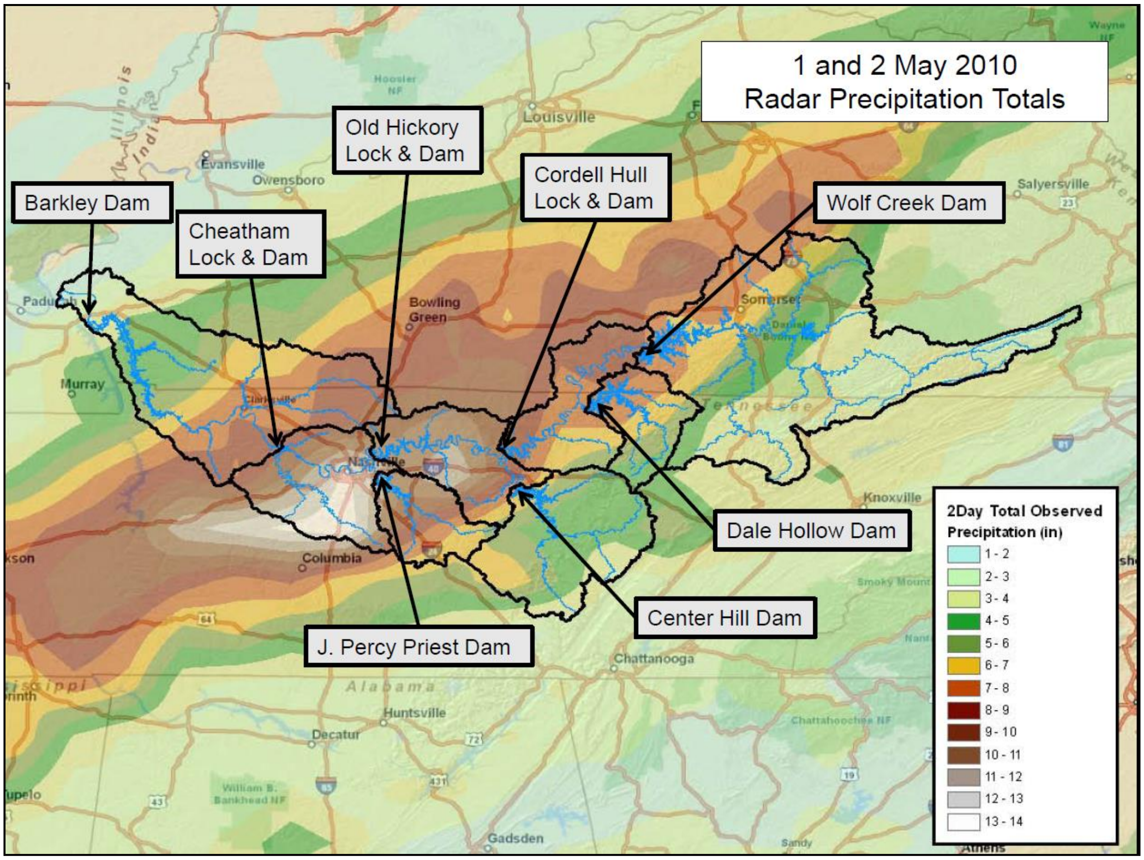

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Figure 8.12 Total Precipitation Data in the Cumberland River Basin over May 1st and 2nd, 2010, (U.S. Army Corps of Engineers, 2012)

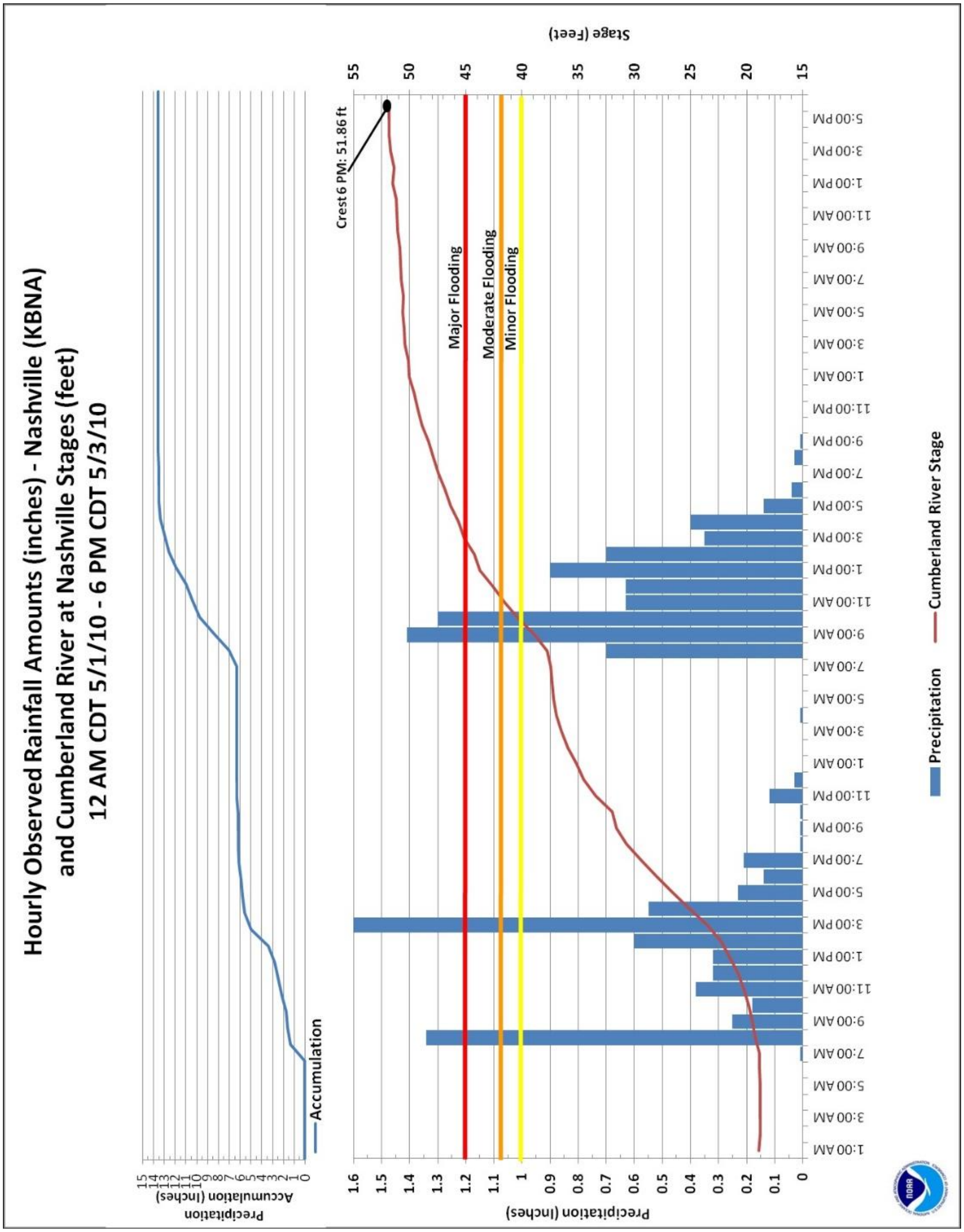

Figure 8.13 Hourly and Accumulative Rainfall at Nashville International Airport from 12:00 a.m., May 1st to 12:00 a.m., May 3rd, (Service, 2011)

Table 8.3 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 Clarksville	62.58 ft	57.1 ft	Mar, 14 1975	46 ft	270 year
Cumberland River at Nashville	51.86 ft	47.6 ft	Mar, 15 1975	40 ft	300 year

Table 8.2, summarizes some of the rainfall totals across the region over the two-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.

Figure 8.14 and Figure 8.15 illustrate the base condition of the Cumberland River levels and the peak stage inundation and the severity of the May 2010 flood event.

Table 8.4 Rainfall Total from May 1st to May 3rd, 2010, (U.S. Army Corps of Engineers, 2010b)

Gage Location Cumberland River Basin	Total Rainfall (in)
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 8.14 Nashville Area during Base Condition, (U.S. Army Corps of Engineers, 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. 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 quick exceeded on Sunday when the river stage at Nashville, TN, rose rapidly through moderate and major flood levels as seen in Figure 8.13 (USACE 2010c and 2012). The next section, the actions and reservoir operations of the USACE during the flood event is described in detail.

Figure 8.15 Nashville Area during Peak Stage Condition, (U.S. Army Corps of Engineers, 2012)

8.4 Actions 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; including uncontrolled downstream tributary drainage areas, 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 being operated 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 8.16 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 8.1) 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).

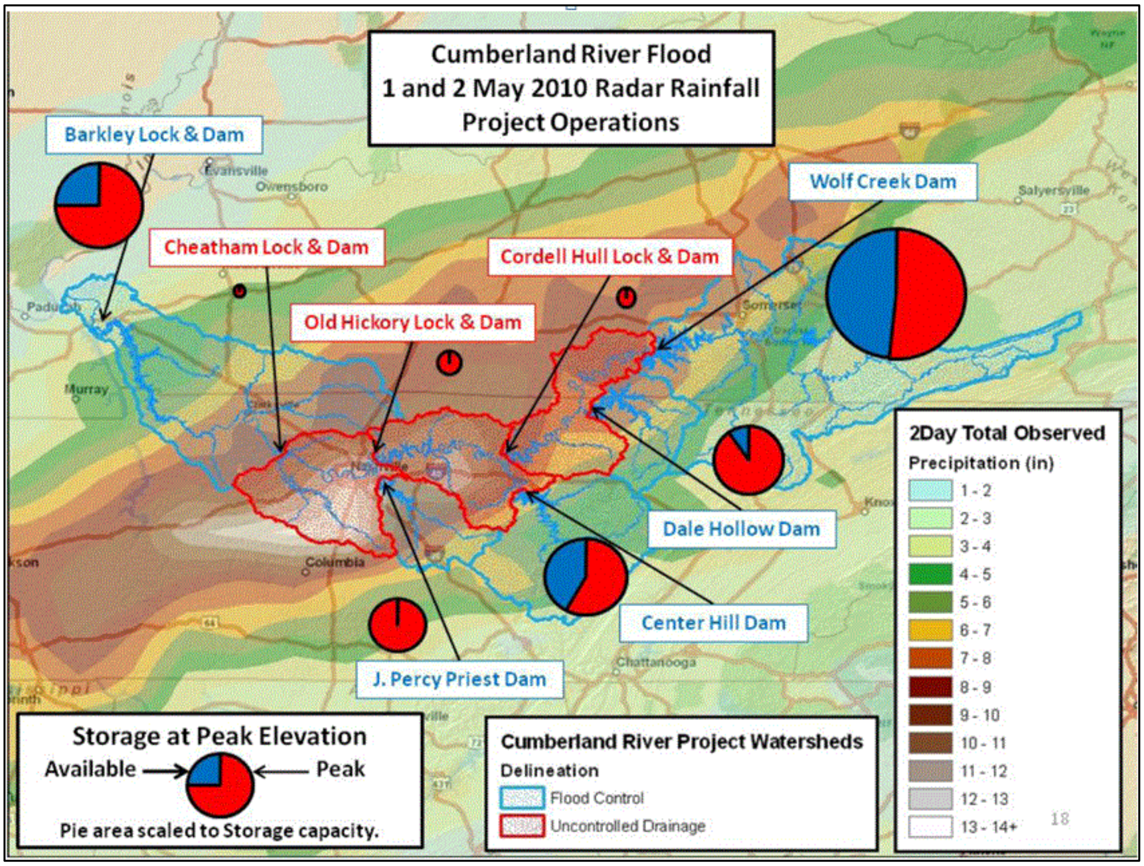

Figure 8.16 Cumberland River Basin Projects, Controlled and Uncontrolled Drainage Areas: May 1st and 2nd, 2010 (*Source*: UASCE, 2010c)

During the May 2010 flooding event, J. Percy Priest Dam, located just upstream of Nashville, TN, the spillway gates were nearly overtopped (USACE 2010c and 2012). The flood storage capacity was exceeded requiring the 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 ft^3/s. During the flood event, the project was visually inspected twice a day. Old Hickory Lock and Dam experience a tremendous water load coming with 6.6 inches of complete dam failure. A maximum historical discharge of 212,260 ft^3/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 in uncontrolled flow and increased downstream damage impact. Figure 8.17 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 ft^3/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, with a maximum historical discharge of 240,000 ft^3/s along with a maximum historical headwater elevation of 404.15 feet (USACE 2010c and 2012).

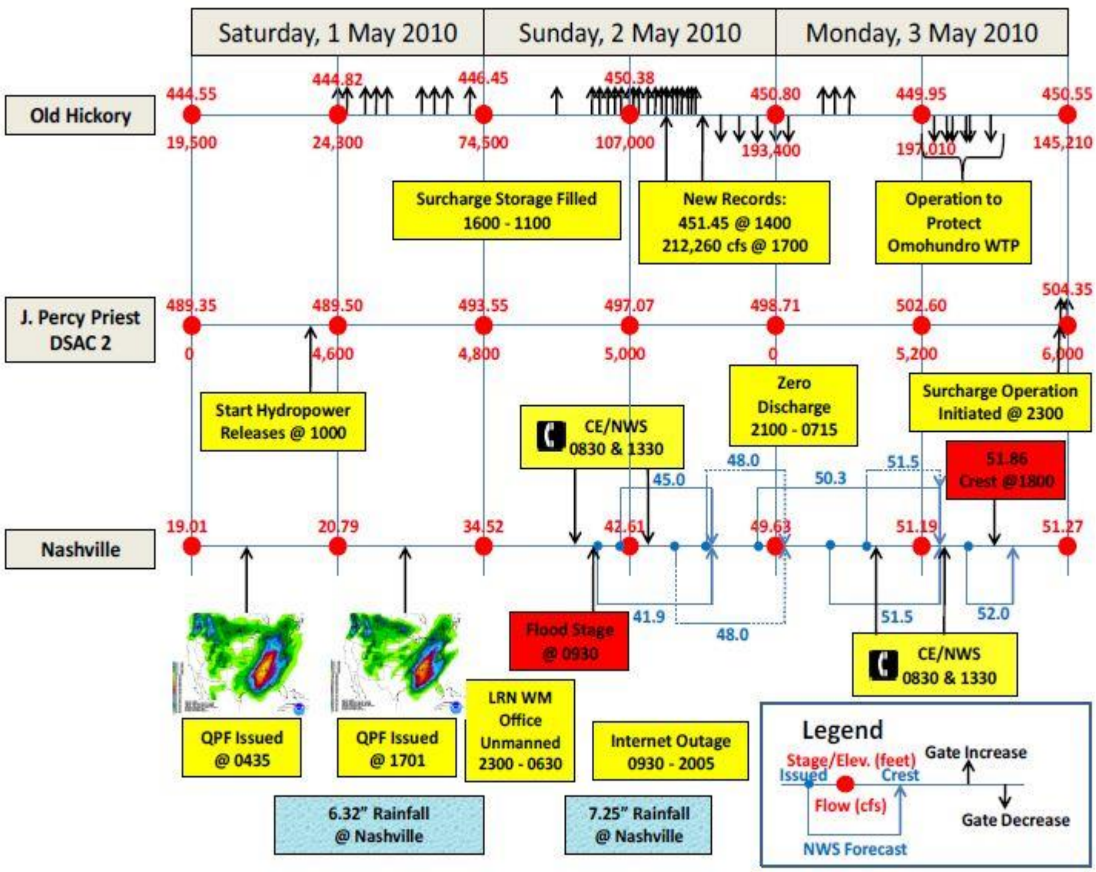

Figure 8.17 Old Hickory, J. Percy Priest, and Nashville Gage, (U.S. Army Corps of Engineers, 2010b)

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 mainstream of the Cumberland River, the operation policies do not extend to the full range such as when extreme events. The ability to sustain the 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 a reservoir within 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 gravity 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, (U.S. Army Corps of Engineers, 2010b, 2010a, 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 8.18 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 processes 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.

8.5 The Flooding Damages

The May 2010 flood event established the new flood record for much of middle Tennessee. Figure 8.19 shows the aftermath on Cumberland River near downtown Nashville. The immediate concern was issued regarding the quality of municipal water supplies. It was reported that 42 water supply systems were adversely affected. Ten of these systems were completely offline with several being out of service for two weeks or more. The 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.

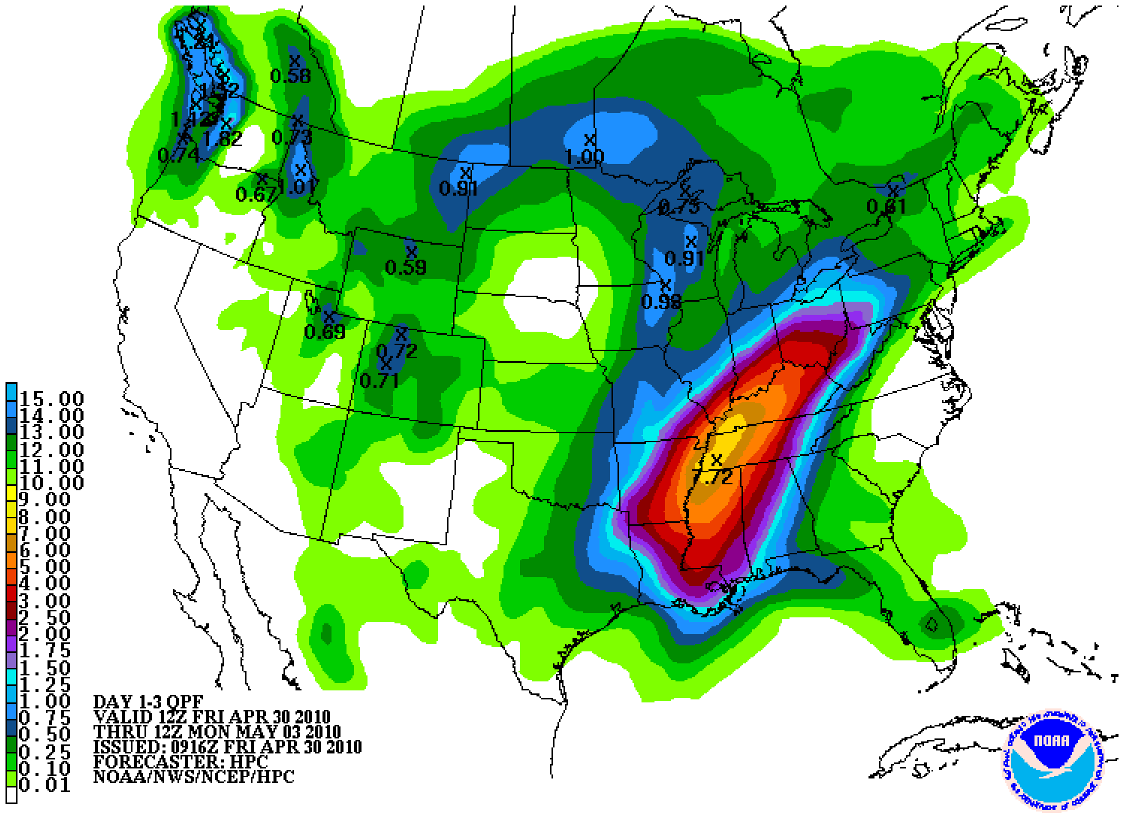

Figure 8.18 NWS QPF published on April 30, 2010, (U.S. Army Corps of Engineers, 2010a)

Numeral 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 a few weeks. Although water and wastewater contamination were 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. The 52-county region was affected by the flooding. The flooding within the Cumberland River Basin impacted thousands of homes and businesses. An estimated $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 the west and middle Tennessee and, western and central Kentucky, 18 of which occurred within the USACE Nashville District boundaries (USACE, 2010c and 2012).

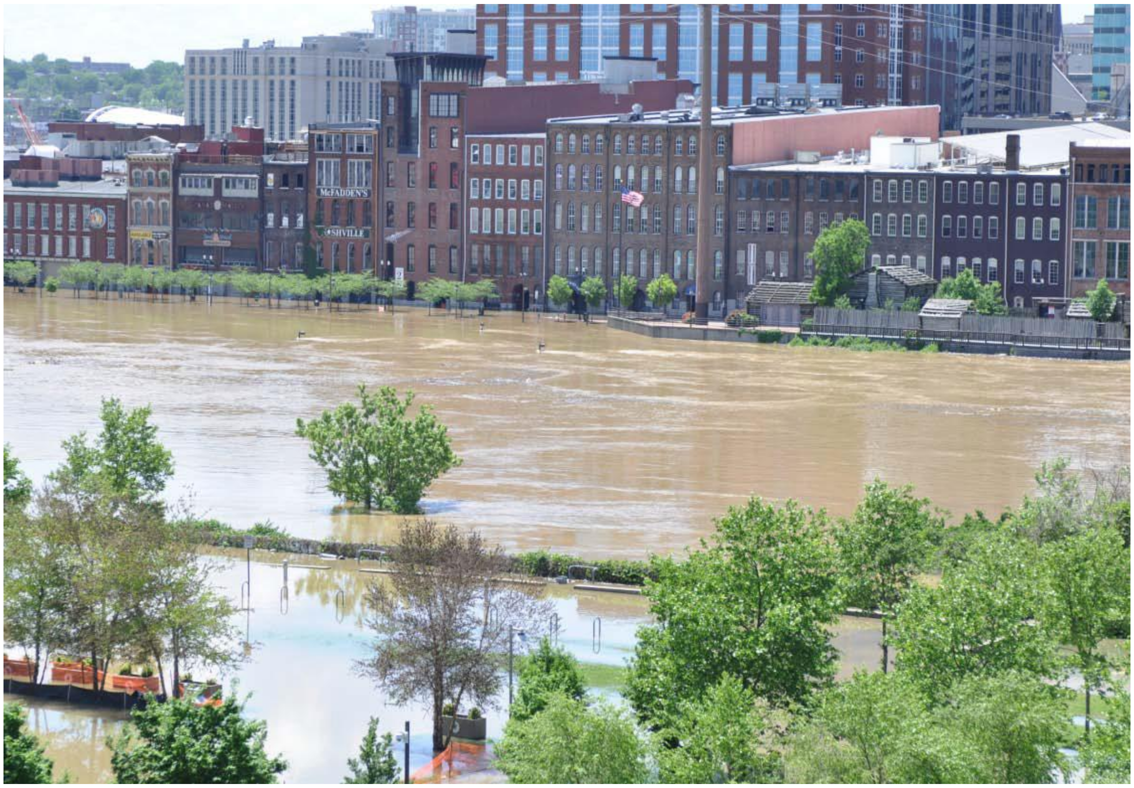

Figure 8.19 Flooding along First Avenue on the Cumberland River near Downtown Nashville, (U.S. Army Corps of Engineers, 2012)

8.6 Real-Time Operation Situation in the Cumberland River System

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 8.20. 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 mainstream 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 the elevation of 447 feet as shown in

Table 8.5. 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 are reached, J. Percy Priest discharge 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 ft^3/s per hour. The maximum combined decrease in spillway discharges from Old Hickory and J. Percy Priest is limited to 10,000 ft^3/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 the 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 the 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 rising 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 8.5.

Table 8.5 Spillway Releases for various Headwater Levels, (U. S. Army Corp of Engineer, 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 personnel was sent to the reservoirs and flood sites to observe 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 8.17, 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 optimal real-time operation of the river-reservoir system, as described briefly in Chapter 1 and in detailed in chapter 7, been adopted, the flood damage during the May 2010 might have been minimized. By employing the optimal real-time operation of the 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.

8.7 Old Hickory Dam Impact

The USACE operation at the Old Hickory Dam can be further analyzed. The Old Hickory Dam is immediately upstream of Nashville, see Appendix A.

Figure 8.20 shows the gate opening over the five-day span from the start of May 1st to the end of May 5th. Figure 8.21 shows the reservoir discharge over the same five-day span. Obviously, the USACE did not start operating the gates at the dam until well after the storm had started (see Figure 1.11 and Figure 8.13). It was not until later in the day on May 1st that the USACE started to release water from the reservoir gates. The late response at the Old Hickory Dam to the storm was one of the main reasons why Nashville was flooded. The USACE needed to release quickly; thus the gates were opened rapidly on May 2nd, causing huge discharges from the dam as seen in Figure

8.21. Figure 8.22 shows the flow comparison of the Old Hickory Dam discharges and the flow in downtown Nashville. There was a strong correlation between the two flow time series, and it was evident that the huge rapid increase in discharges from the Old Hickory Dam was the major 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 time span between May 2^{nd} and May 4^{th} as shown in Figure 8.23.

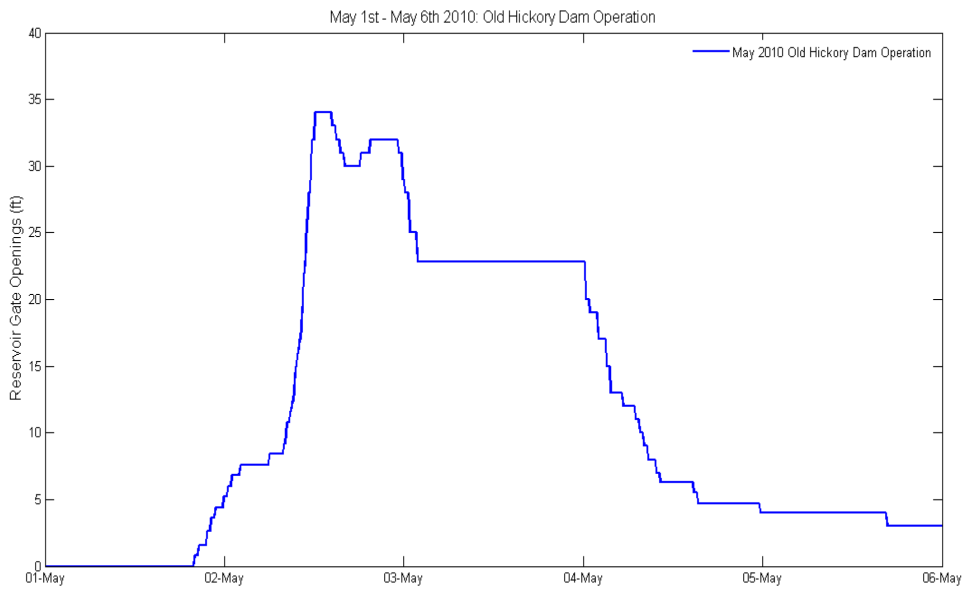

Figure 8.20 The Gate Openings at the Old Hickory Dam during the May 2010 Storm Event (U.S. Army Corps of Engineers, 2010b)

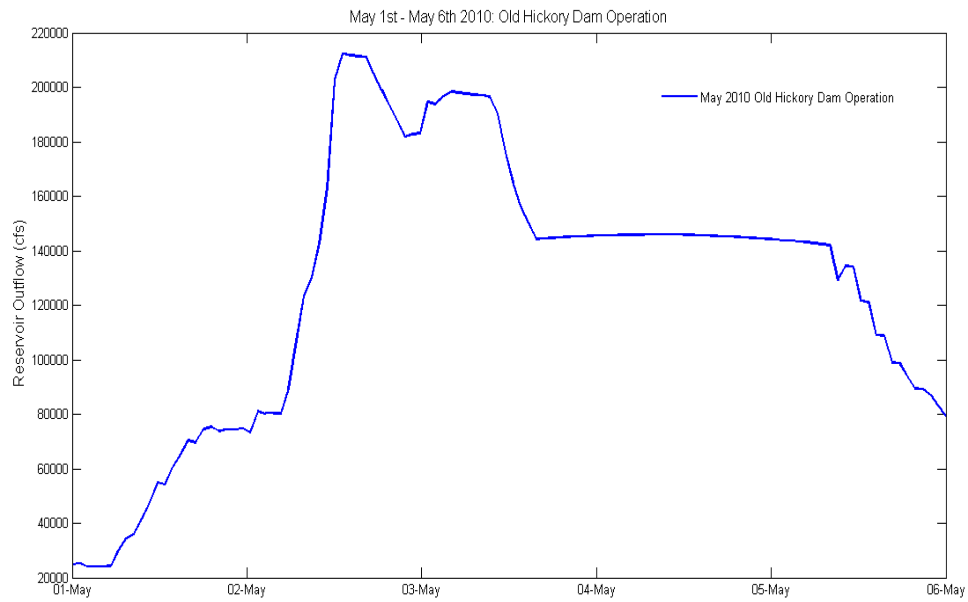

Figure 8.21 Reservoir Outflow at the Old Hickory Dam during the May 2010 Storm Event (U.S. Army Corps of Engineers, 2010b)

In fact, not only the Old Hickory Dam operation during the May 2010 storm event was flawed, but the existence of the dam was also problematic under the flooding conditions. According to the study and simulations conducted by Dr. Larry W. Mays of Arizona State University with the dam in place and the USACE operation caused a 2.2 ft. increase in maximum water surface elevation for the May 2010 storm event , as illustrated in Figure 8.23. Figure 8.25 shows the flooding inundation in downtown Nashville and at Pennington Bend/Opryland.

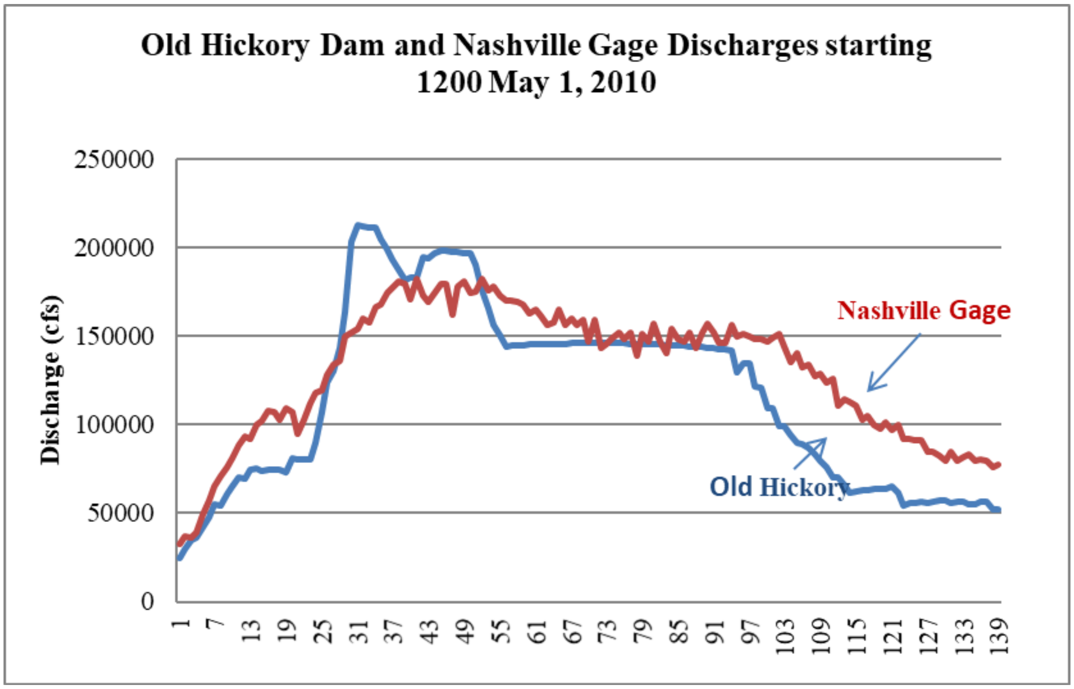

Figure 8.22 Reservoir Outflow at the Old Hickory Dam and Flow at Nashville during the May 2010 Storm Event, (Che & Mays, 2015)

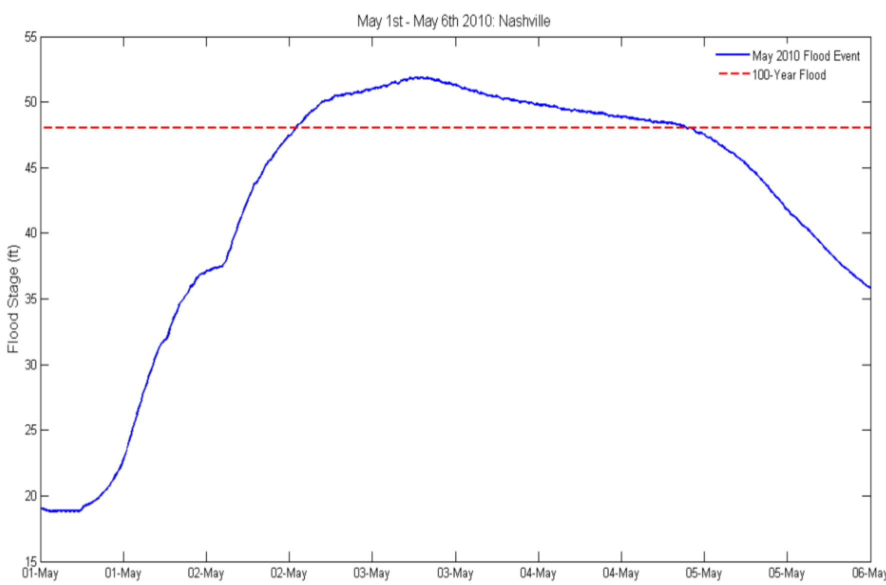

Figure 8.23 Flood Stage Condition at Nashville during the May 2010 Storm Event

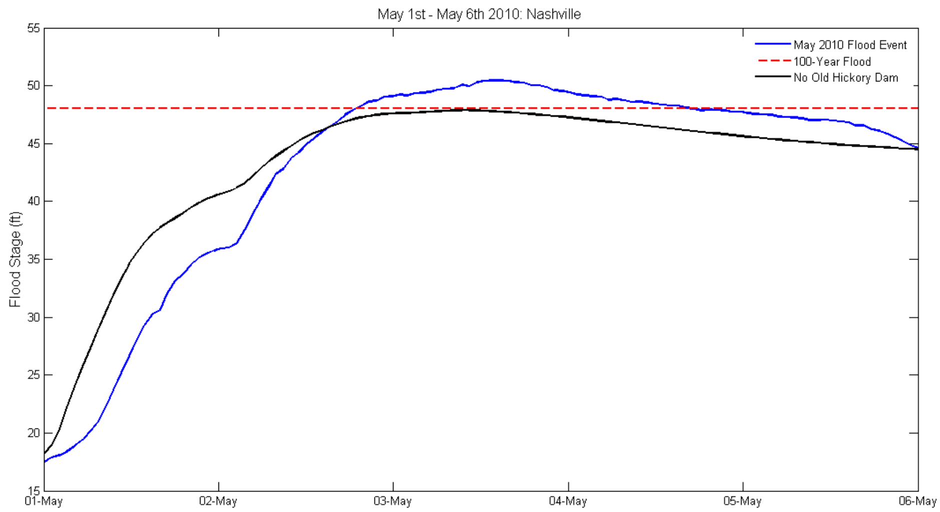

Figure 8.24 Flow Comparison (with and without Old Hickory Dam) at Nashville during the May 2010 Storm Event

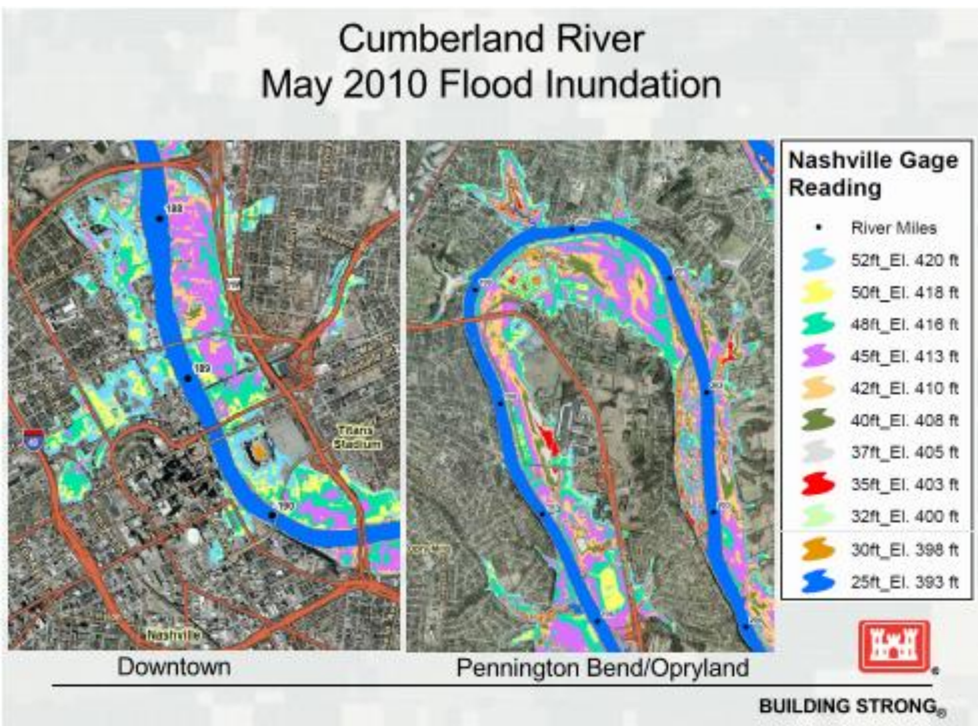

Figure 8.25 Flooding in Downtown Nashville and Pennington/Opryland by the

U.S. Army Corps of Engineers

CHAPTER 9 MODEL APPLICATION TO THE CUMBERLAND RIVER BASIN

9.1 Model Applications

The optimization – simulation model, described in chapters 5 and 6 and tested in chapter 7, has been applied to the Cumberland River system described in chapter 8 using the May 2010 flood event, also explained in the previous chapter. The main objective of this model application is to demonstrate the applicability of the model for minimizing flood damages for an actual flood event in a real-time fashion on an actual river basin. The purpose of the application in a real-time framework was to minimize the flood damages at Nashville, Tennessee by keeping the flood stages (water surface elevations) under the 100-year flood stage of 48 feet at the Nashville Woodland station during the storm event.

The model application compared the three unsteady flow simulation scenarios: one-dimensional unsteady flow, two-dimensional unsteady flow (diffusion-wave model), and combined one- and two-dimensional unsteady flow (diffusion-wave model) utilizing the current version of HEC-RAS 5.7. This allowed a comparison of the three unsteady flow simulation methodologies. The reservoir regulation and operation rules prepared by Sverdrup Corporation in 1998 for the U.S. Army Corps of Engineers, Nashville District were used in the model to set the operation rules constraints. The domain or simulated portion of the model application on the Cumberland River system is shown in Figure 9.1, and Figure 9.2 shows the entire Cumberland River system. The HEC-RAS two-dimensional module has the option of either running the two-dimensional diffusion-wave equations. Basically, the diffusion wave equations run faster and more stable. Thus, the diffusion wave equations set have been used in this model.

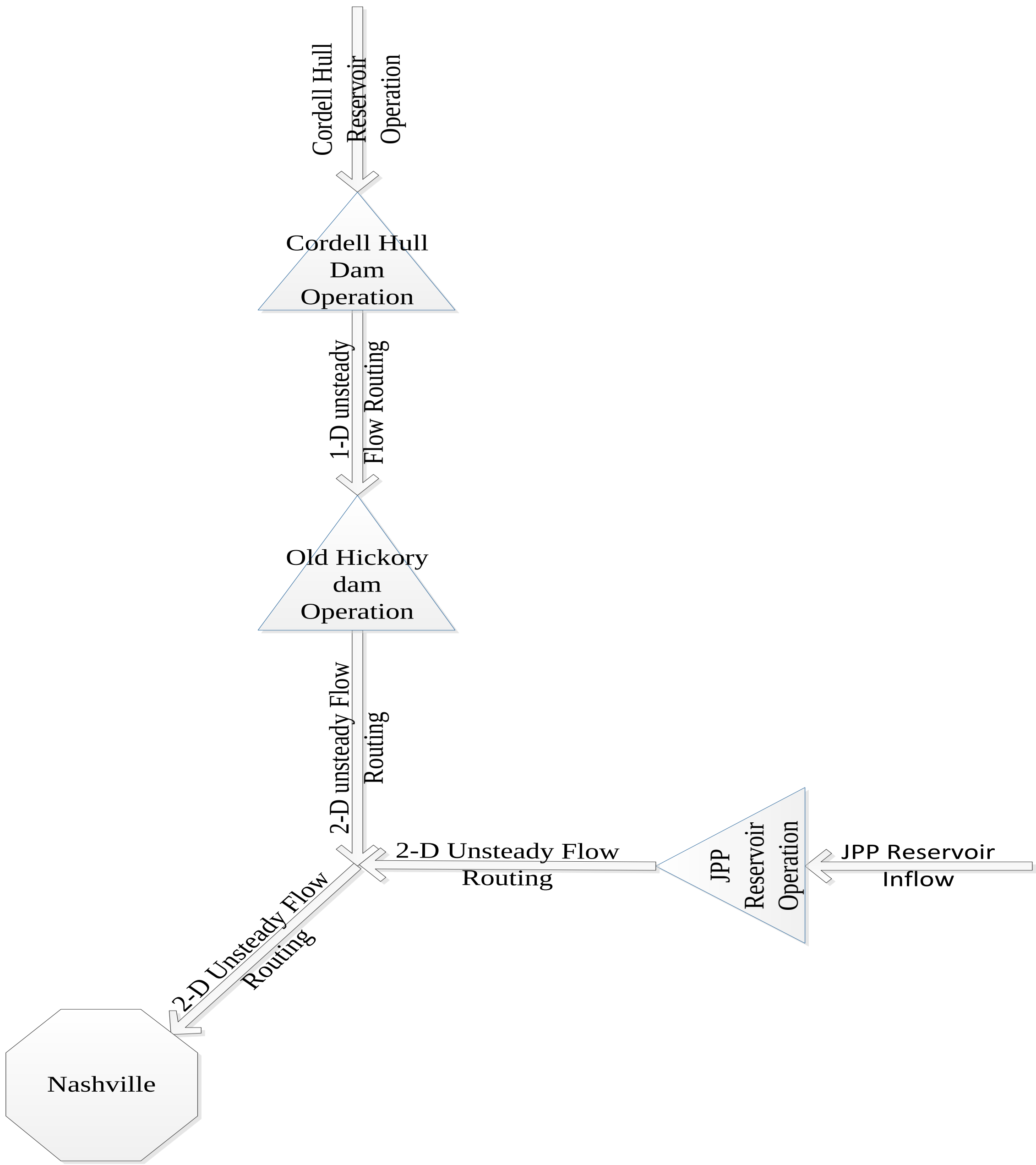

Figure 9.1 Cumberland River Simulated Portion

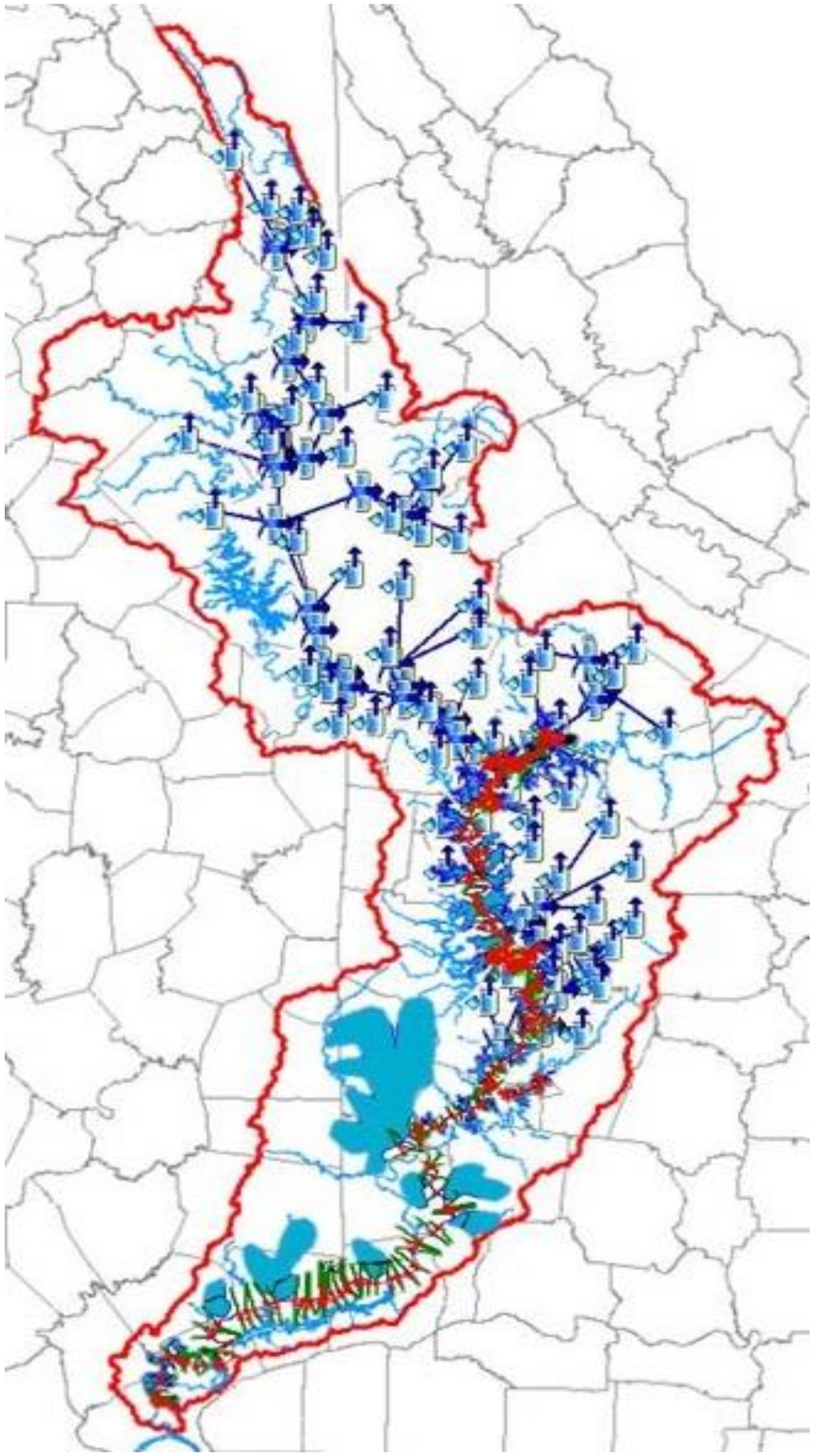

Figure 9.2 Cumberland River Basin HEC-HMS and HEC-HMS Model Domain

The approximate watershed area modeled is 14,160 mi^2 of the Cumberland River system. The modeled watershed area included headwater of the basin starts in Lechter County, Kentucky to its end at Cheatham Dam around 30 miles downstream from Nashville, Tennessee. The hydrologic model in HEC-HMS is already set up and consists of 66 reaches and 69 basins of areas ranging from 7 to 17 mi^2. The hydrologic simulation methods adopted in the HEC-HMS model for the Cumberland River basin are listed in Table 9.1.

Table 9.1 HEC-HMS Hydrologic Processes and Methods Used

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

The real-time rainfall data source of the May 2010 event is the high resolution gridded generated by Next Generation Radar (NEXRAD), which is used for the hydrologic modeling. Figure 9.3 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, (Che, 2015).

The forecasted rainfall is determined using gridded area weighted rainfall forecasting. The rainfall was extracted from each cell of grid areas up to time t, from time series (hyetograph). The weights of each grid cells, w, within a subbasin are determined Figure 9.4.

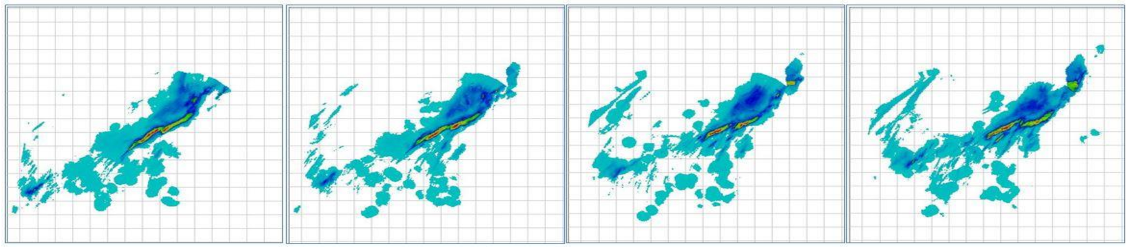

Figure 9.3 Sample Time Revolution of the May 2010 Storm Event from NEXRAD (May 1st 10 a.m. to 1 p.m.)

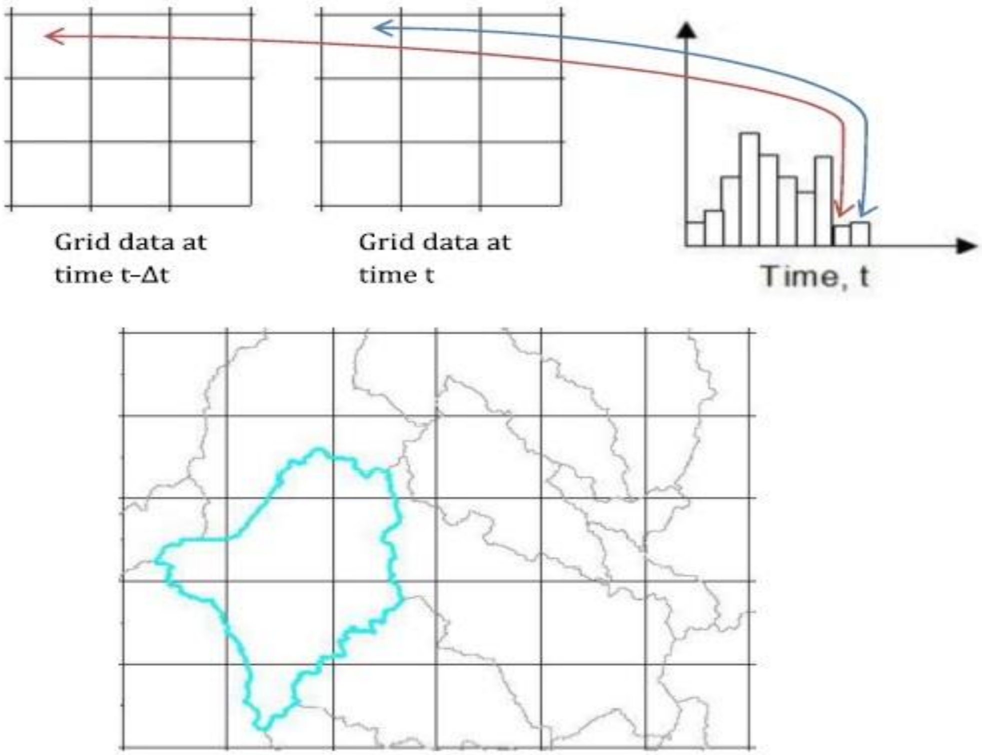

Figure 9.4 Hyetograph Generation for a Cell using Grid Data Extraction, (Che, 2015)

The time series up time t of rainfall for i subbasin is computed using the following equation:

$$P_{i,t} = \sum_{j} w_{i,j} P_{J,t} \tag{9.1}$$

Where:

$P_{i,t}$ is time series of rainfall up to current time t for the i-th sub basin

$w_{i,j}$ weight of the j-th grid overlaying the i-th sub basin

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

The HEC-HMS was already calibrated and validated for the May 2010 storm event by (Che, 2015). He compared the simulated, and the observed Dale Hollow reservoir inflow hydrograph during the May 2010 storm event. He utilized the root mean square error (RMSE) at the Dale Hollow reservoir for the HEC-HMS model is 6174 ft^3/s, which is acceptable considering the magnitude of the storm event. Equation 9.1 determines the root square:

$$RMSE = \sqrt{\frac{\sum_{i=1}^{N}(Q_{observed} - Q_{simulated})^2}{N}} \tag{9.2}$$

Where:

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

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

N is the number of hydrograph ordinate for the model validation

Modeling efficiency has been used as well to validate the HEC-HMS Cumberland River basin, which tests the quantitative measure of performance and goodness of fit. (Nash & Sutcliffe, 1970) described the modeling efficiency based on the deviation variance, Equation 9.2:

$$E = 1 - \frac{\sigma_e^2}{\sigma_o^2} \tag{9.3}$$

Where:

E	is model efficiency.
σ_e^2	is the variance of the deviation between observation and simulation
σ_o^2	is the variance of the observations.

The HEC-HMS model of the Cumberland River Basin is well validated with a model efficiency of 0.853 as shown in Figure 9.5, (Che, 2015).

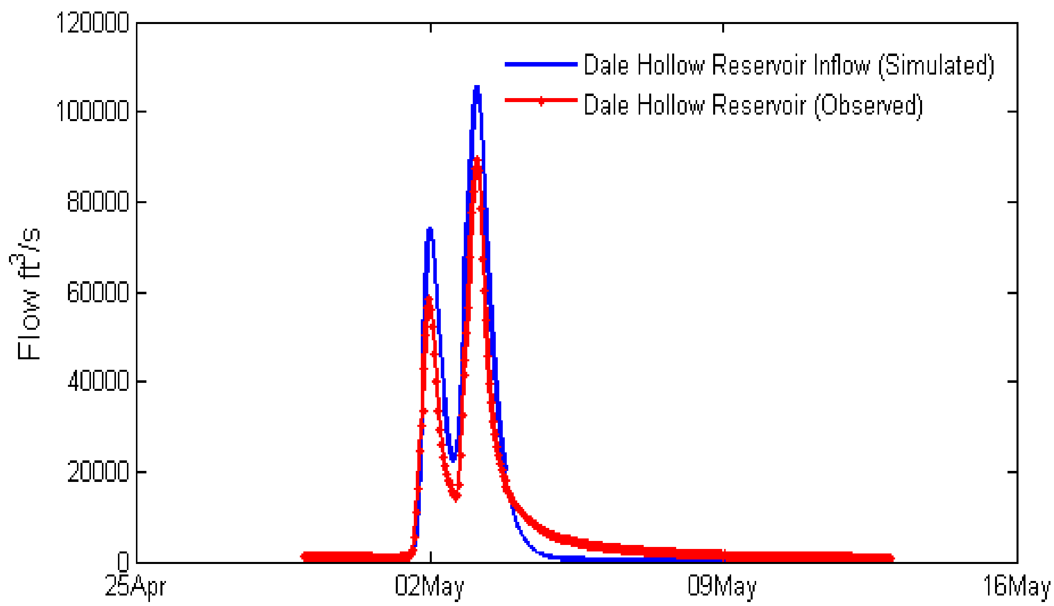

Figure 9.5 HEC-HMS Model Validation for the May 2010 Storm Event at Dale Hollow Dam, (Che, 2015)

9.2 One-Dimensional Unsteady Flow Model Application

The one-dimensional unsteady flow (HER-RAS) model of the Cumberland River system consists of computational components. These components are 675 cross sections, 8 inline structures, 117 lateral structures, and 1 bridge. Che (2015) has also calibrated and validated the 1-D unsteady flow model for the May 2010 storm event. He computed the root mean square error (RMSE) at Nashville for HEC-RAS model as 14,550 ft^3/s, which is in the range of acceptance considering the magnitude of the storm event and the nature of unsteady flow modeling. The model efficiency for the HEC-RAS model is 89%. The difference between the observed and simulated flow for the May 2010 storm is illustrated in Figure 9.6. The root means square error (RMSE) at the Nashville stage for this simulation is 1.777 ft, as in Figure 9.7

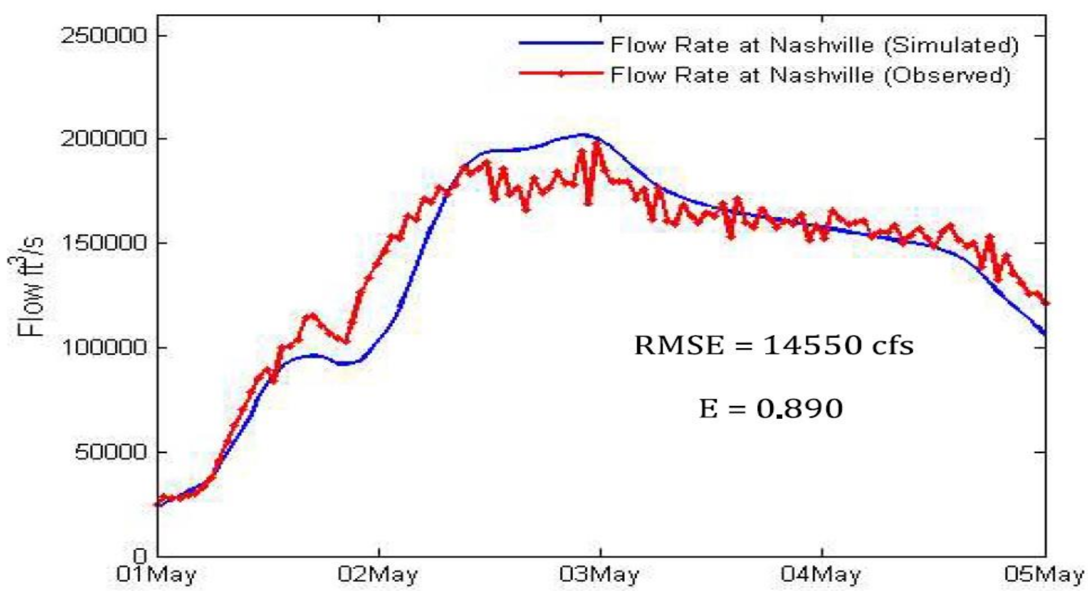

Figure 9.6 Simulated VS Observed Flow at Nashville, (Che, 2015)

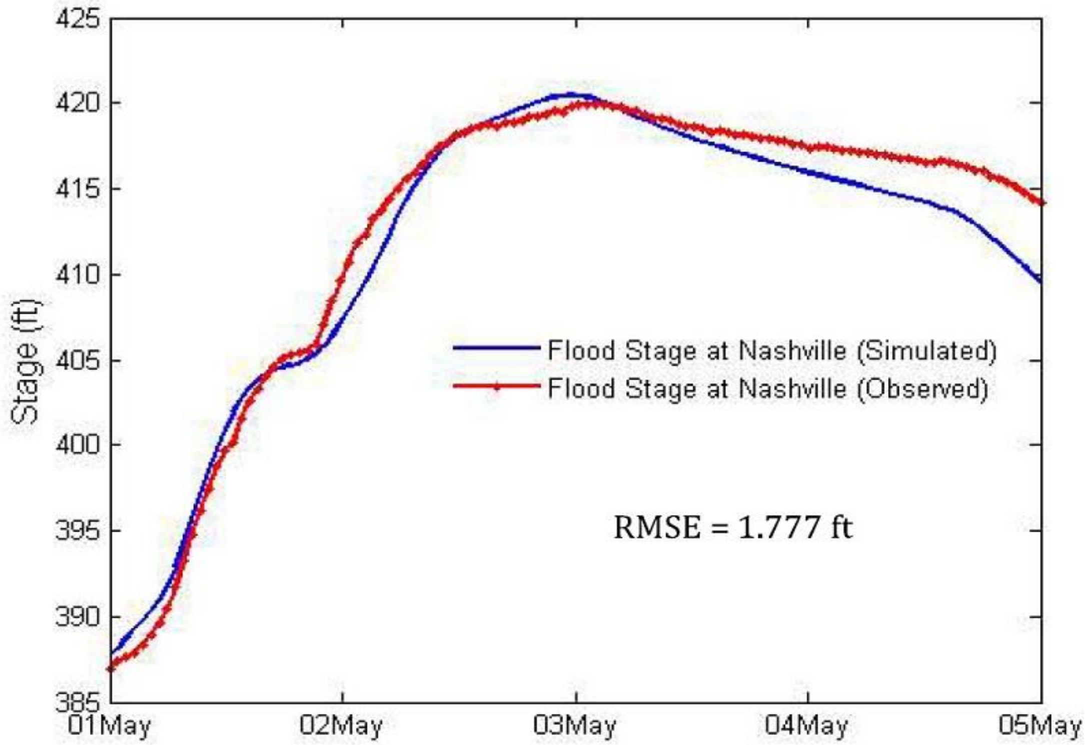

Figure 9.7 Observed and Simulated Stages at Nashville, (Che, 2015)

9.3 Unsteady Flow Simulation Approach

As stated above the optimization - simulation model was applied to a portion of the Cumberland River basin including Old Hickory and Cordell Hull reservoirs and J. Percy Priest lake as shown in Figure 9.1. The portion of the Cumberland River modeled in HEC-RAS using the two-dimensional approach is from Old Hickory Dam downstream through Nashville to Cheatham Dam.

The flows downstream of Old Hickory and J. Percy Dams were modeled using the two-dimensional unsteady flow approach and combined one, and two-dimensional unsteady flow and the area upstream of Old Hickory has been modeled using only one-dimensional unsteady flow routing.

Three scenarios have been adopted to simulate the unsteady flow for the May 2010 flood event. The first scenario uses combined one- and two-dimensional unsteady flow modeling, in which the Cumberland River, Nashville reach has been model in one dimensional, while its flood plain was modeled using two-dimensional unsteady flow simulation. The two-dimensional area was divided into four subregions, two the for the north side of the river reach and two for the south side and as NE, NW, SE, and SW. These sub-areas were gridded into 832, 1068, 235 and 1208 cells respectively, Figure 9.8. The maximum cell size of 2.19 M square feet minimum cell area of 714 k square feet, and the average cell area of 1 M square feet. The total area of these cells that cover two-dimensional modeling is around 106 square miles. The input spacing into the 2-D flow area editor for generating these cells is 1000 x 1000 feet, Figure 9.11.

The other components include two reaches, cross sections, storage areas, laterals, inline structures, and one junction. The Nashville reach connects Old Hickory Dam at the upstream to Cheatem at the downstream using 76 cross sections over the total length of 51 miles. The other reach is Stoned River reach that links the J. Percy Priest Lake to the Nashville reach via 22 cross sections. The cross sections were extracted from the terrain model and modified with the actual cross sections surveyed by U.S.AC.E. The terrain does not accurately represent the actual bathymetry of a river reach because the LIDAR technology does not have the ability to penetrate the water surface elevation.

Each of the two-dimensional areas is connected to the river reach (one-dimensional area) through a lateral structure, Figure 9.8. The Nashville’s reach connected form the upstream to the Old Hickory reservoir using inline structure representing the

Old Hickory Dam as shown in Figure 9.8, and the and the same for J Percy Priest Lake. Each run iteration of a combined one- and two- dimensional model takes 5 to 6 minutes to run one unsteady flow simulation for this portion of the Cumberland river system shown in Figure 9.1.

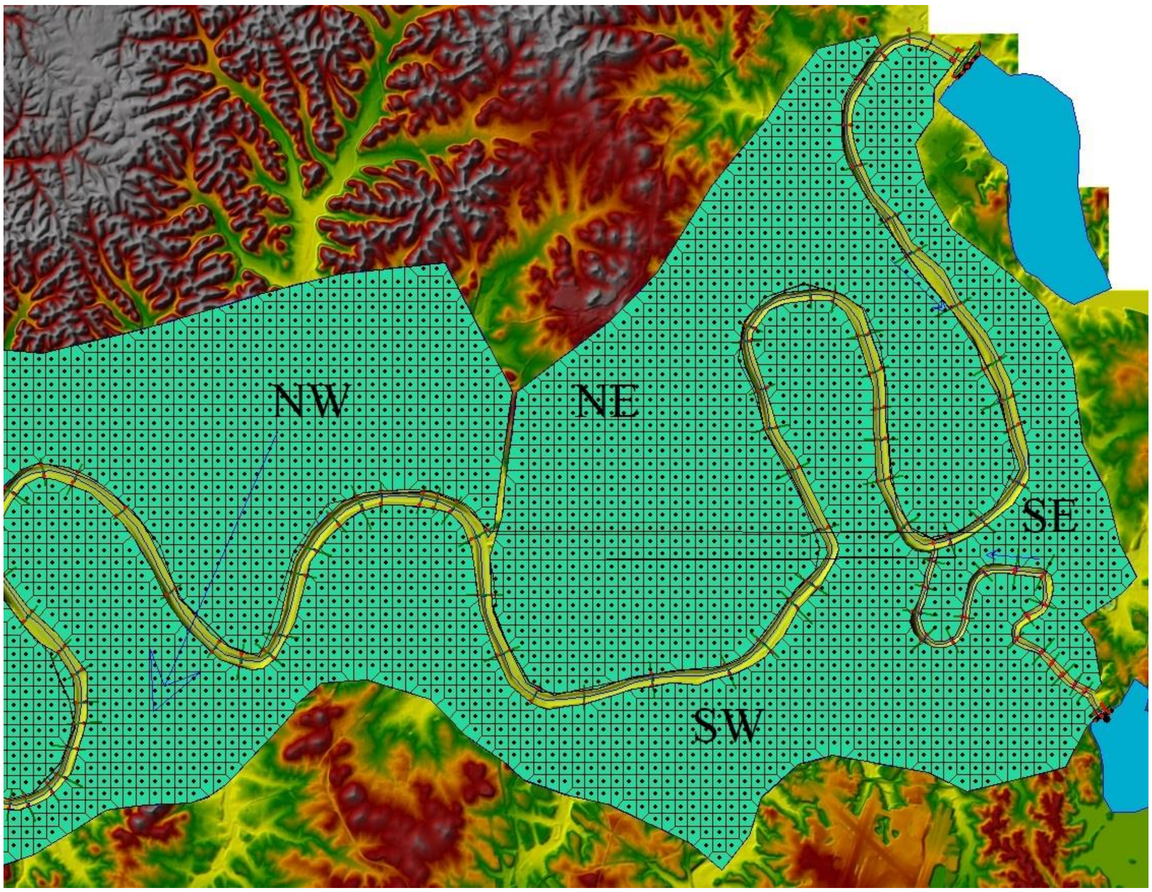

Figure 9.8 Combined One- and Two- Dimensional Areas

The second scenario is using only the two-dimensional unsteady flow modeling. One of the problems in using only the two-dimensional approach is that terrain data does not often include the actual terrain underneath the water surface in the channel region (river bathymetry) due to the fact that LIDAR processing is not capable of penetrating the water surface elevation (U.S. Army Corps of Engineers, 2016c). As a result, many HEC-RAS users do not prefer only the two-dimensional approach. Thus, the terrain model of the only two – dimensional of the Nashville reach has been modified through RAS Mapper by creating a terrain model of the channel region only from the cross sections surveyed and measured in the field by U.S Army Corp Engineers and the cross-section interpolation surface. As mentioned above the general surface terrain model does not accurately depict the terrain below the water surface so, it is merged with the created channel region terrain to improve the whole terrain model for hydraulic modeling purposes. Figure 9.9 shows Nashville's terrain before modification while Figure 9.10, shows the modification on Nashville's terrain model which reflexes the existing bathymetry of Nashville' reach of Cumberland river.

Though, the second scenario was set up with only two-dimensional area enhanced with 2-D break lines along the river reach to enforce the mesh generation tools to align the computational cell faces along the break lies. The two-dimensional flow element connected directly to the storage areas: Old Hickory Reservoir and J. Percy Priest Lake using the storage area and 2-D area connections that allow inputting the data of hydraulic structures such as gates to weirs as it set in normal inline structures to control the flow between the two elements of the area. The 2-D area was divided into 3211 cells, with a maximum cell area of 2.2 M square foot, minimum cell area of 439 K square foot, and the average cell area of 974165 square foot. The total area of these cells that cover the two-dimensional modeling is around 112.2 square miles, see Figure 9.11. The input spacing into the 2-D flow area editor for generating these cells is 1000 x 1000 feet, which is considered relatively course, the reason behind that is any finer cell size will take longer time to run the simulation and that may cause exceeding the lead time in which the decision for reservoir releases has to be made. However, the model ran well with the suggested spacing.

Due to the limited availability of the LIDAR that was used to develop the terrain model for Nashville, the area upstream of Old Hickory Dam was modeled using 1-D unsteady flow simulation. The terrain resolution used in the model was 2.5 X 2.5 feet which is considered high enough to produce more accurate and detailed hydraulic table properties for two-dimensional computational cells and cell faces.

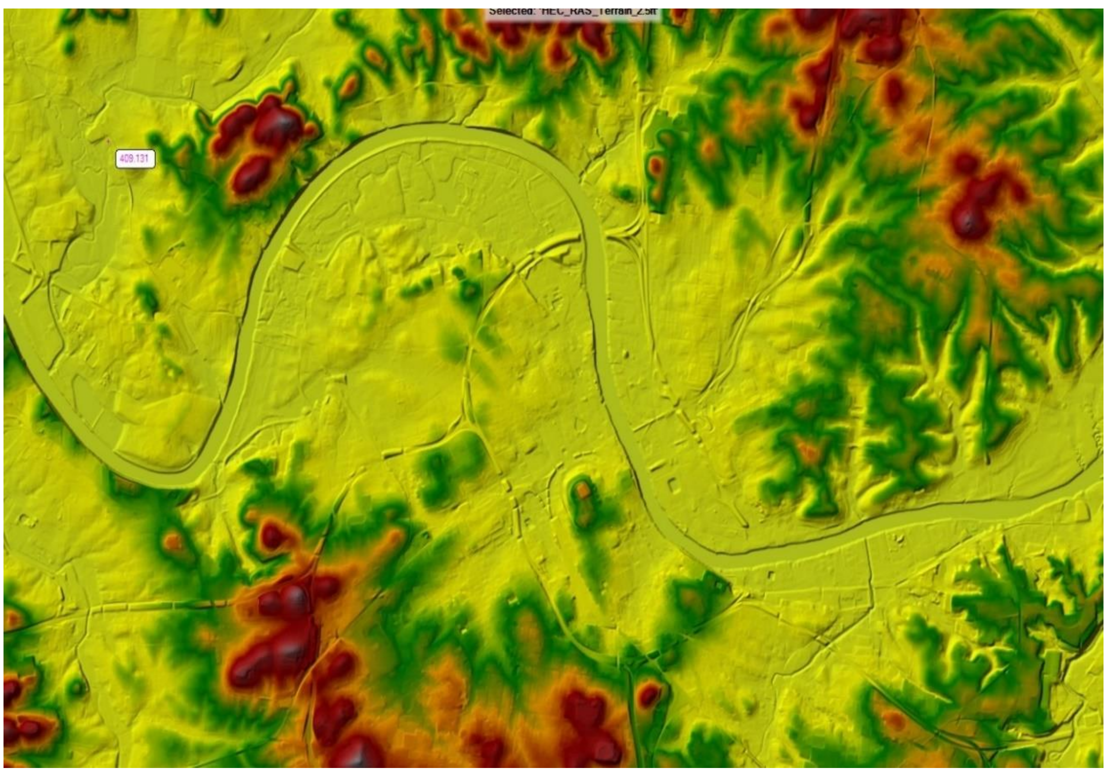

Figure 9.9 Nashville Orphan Terrain

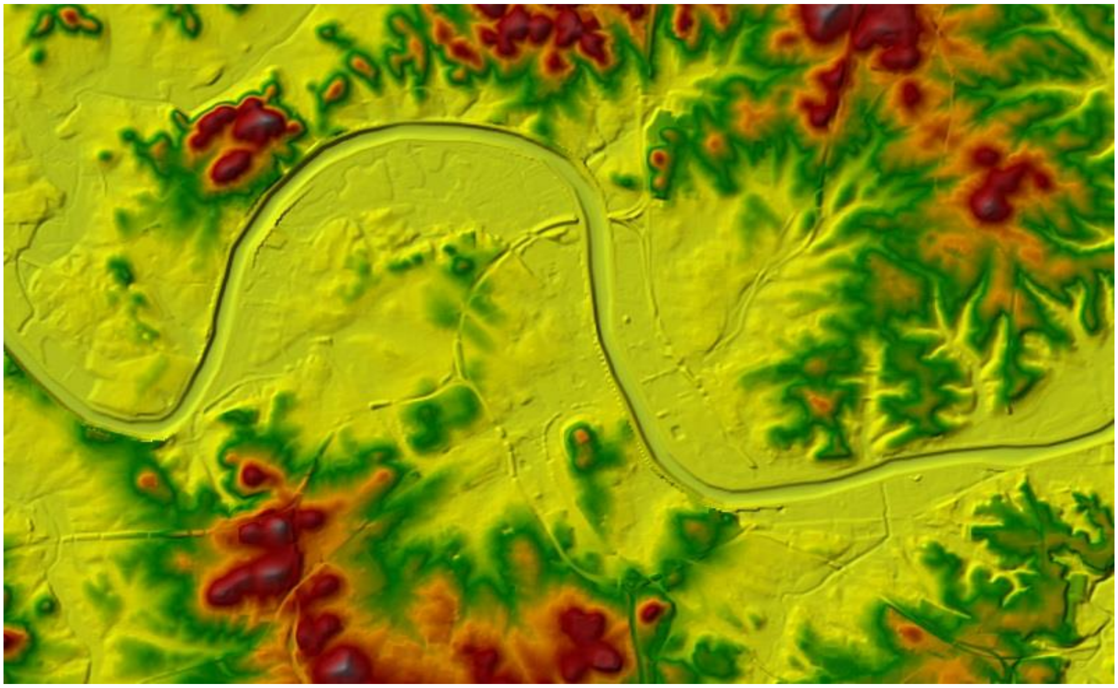

Figure 9.10 Nashville's Modified Terrain

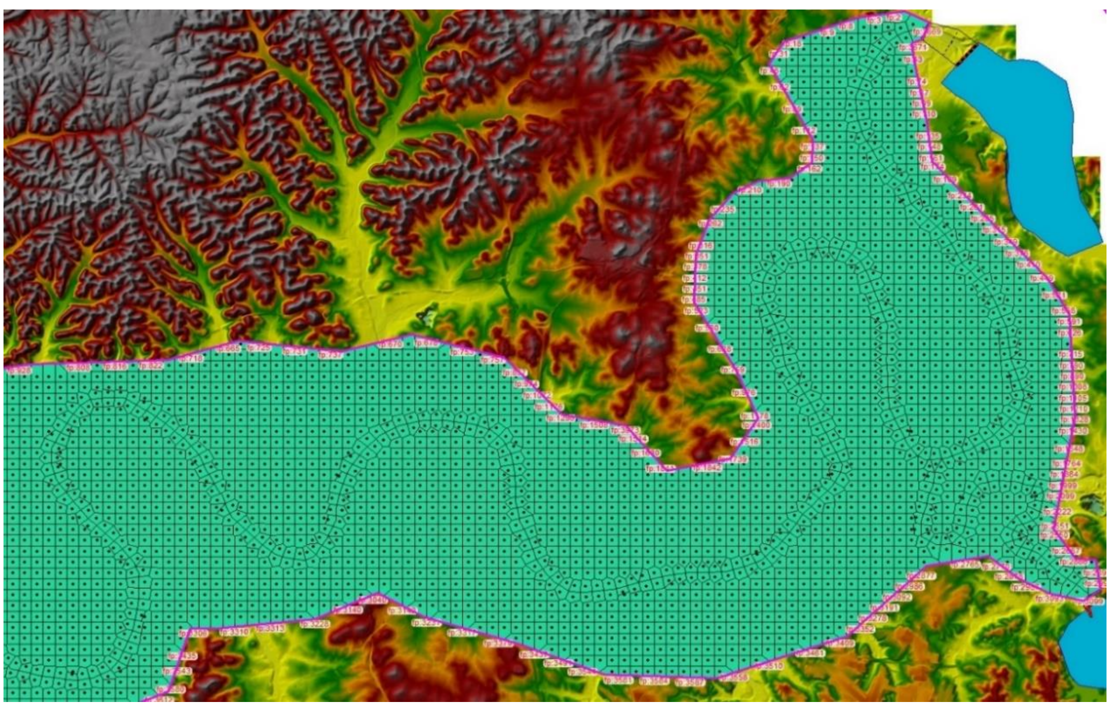

Figure 9.11 Nashville Two-Dimensional Area

9.4 Solution Approach

The basic objective of the optimization-simulation model for river-reservoir system operation in real time is to keep the discharges and water surface elevations below specified or target values during an extreme storm event. For this application, the 100-year frequency values Nashville, Tennessee are 48 feet for the stage, 417.52 for elevation and 172,000 cfs for Woodland station. The process of simulation starts with the hydrologic modeling for the system using the hydrologic modeling system HEC-HMS, which simulates the process of Cumberland River basin rainfall runoff. The HEC-HMS model for entire Cumberland River Basin was previously developed by the U.S. Army Corp of Engineers.

The solution processes start with the available actual rainfall data up to time t for the area upstream from Cordell Hull reservoir and J. Percy Priest Lake to the model through a MATLAB code. The MATLAB code sends the actual rainfall data to the HEC-HMS to simulate the rainfall-runoff process. The discharge hydrograph of the HEC-HMS model becomes the inflow for the Cordell Hull Reservoir. The inflow hydrograph enters the optimization and operation model to determine the optimal releases from the Cordell Hull Dam gates. The model now calls HEC-RAS to route the releases up to Old Hickory reservoir, where it considered as the inflow to the Old Hickory reservoir. Once the inflow hydrograph for Old Hickory reservoir is determined, the operation and optimization model determines the releases for the next 4 hours from the Old Hickory Dam. The optimization model employs the genetic algorithm in MATLAB to generate the initial solution considering all the operating rules constraints described previously and, calls the unsteady flow model (HEC-RAS) to test the generated solutions which are the time series of the Old Hickory gates openings. The unsteady flow model routes through the gate openings at both reservoirs downstream to the Woodland station at Nashville. The process continues iteratively until the objective function is satisfied. Then the model steps to the next time $t + \Delta t$. The model continues to run until the last Δt of the storm.

The optimization model (GA) uses the last generation or the optimal solution at time t as the initial solution for time $t+\Delta t$ to reduce the search time of the next step. This saves around 17 minutes of computation time for each iteration for this application. This computational time savings may be very valuable in real-time river-reservoir operation under flood conditions.

9.5 Model Results and Discussion

The most important factor that could limit this model is the simulation time. Shorter simulation times allows the optimization model (GA) to increase the number of objective function evaluations, which means the number of times that the simulator (HEC-RAS) is called. Producing faster simulation model taking into consideration the accuracy of the mode was a priority of this research. The most time-consuming part of the overall model application was the unsteady flow simulations. Every factor that may affect the time of simulation time, including the mesh size, computation interval, mapping output, and even hydrograph interval was considered.

To obtain a faster two-dimensional unsteady simulations, the diffusion wave model was utilized using the current version of HEC-RAS instead of the full two-dimensional simulation. The portion of the Cumberland River modeled in HEC-RAS using the one- and two-dimensional approaches is from Old Hickory Dam downstream through Nashville to Cheatham Dam.

All the simulation scenarios showed close simulation results for the flood situation at Nashville during the May 2010 flooding event. The optimization and the combined one- and two- dimensional simulation as well as the one-dimensional model successfully kept the discharge at or below 171,809 cfs, after 64 iterations. This is a little higher than the one-dimensional result of the simulation-optimization model of (Che, 2015) with 171,076 cfs.

Each simulation run of the combined one- and two- dimensional simulation and took from 6 to 8 minutes depending on the decided discharges for 4 hours' time span, for which the optimization model could perform around 23 iterations except the first iteration which took longer. The optimized and simulated water surface elevations for the three scenarios at Nashville are depicted in Figure 9.13. The inundation map of the observed water surface of May 2010 for Nashville, that simulated using the combined one- and two-dimensional simulation approach is depicted in Figure 9.14. The simulated and optimized water surface elevations for May 2010 at Nashville is shown in Figure 9.15.

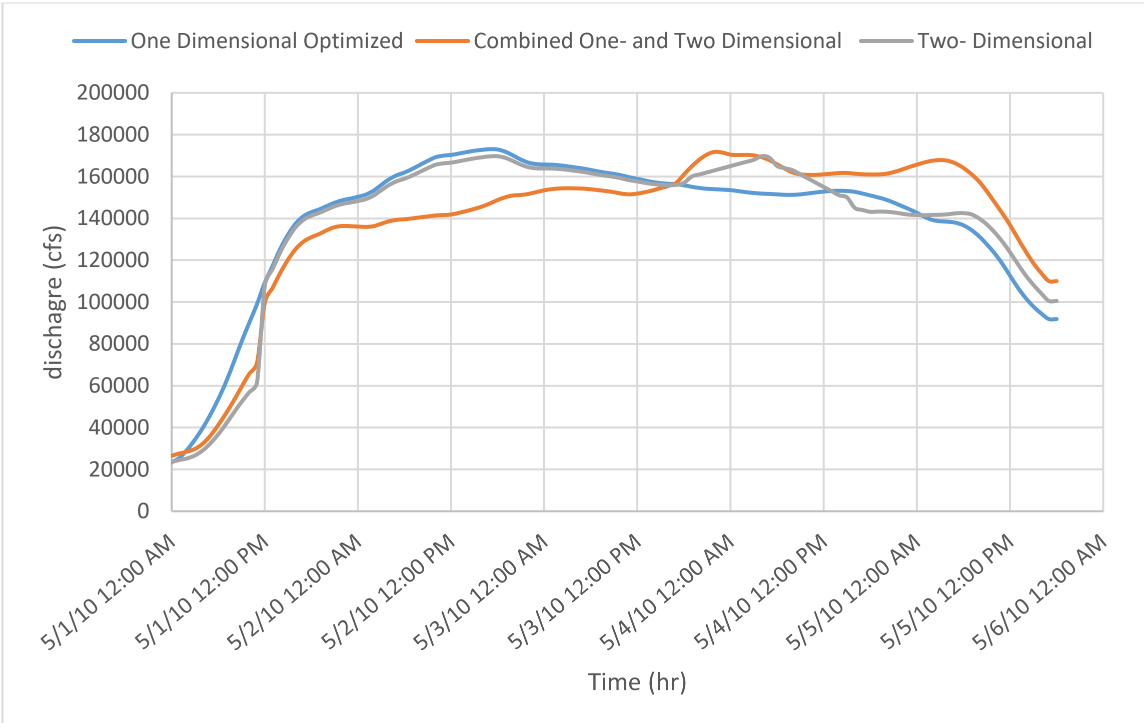

Figure 9.12 Optimized Discharges at Nashville for the May 2010 Flood Event

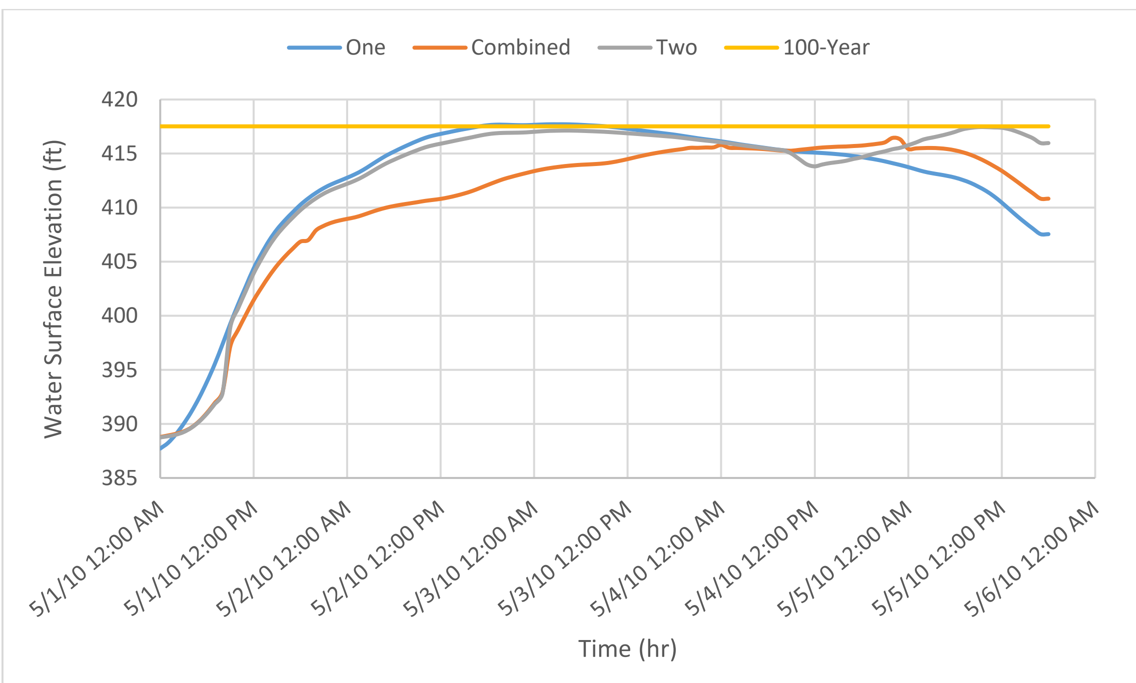

Figure 9.13 Optimized Water Surface Elevations for May 2010 Flood Event at Nashville

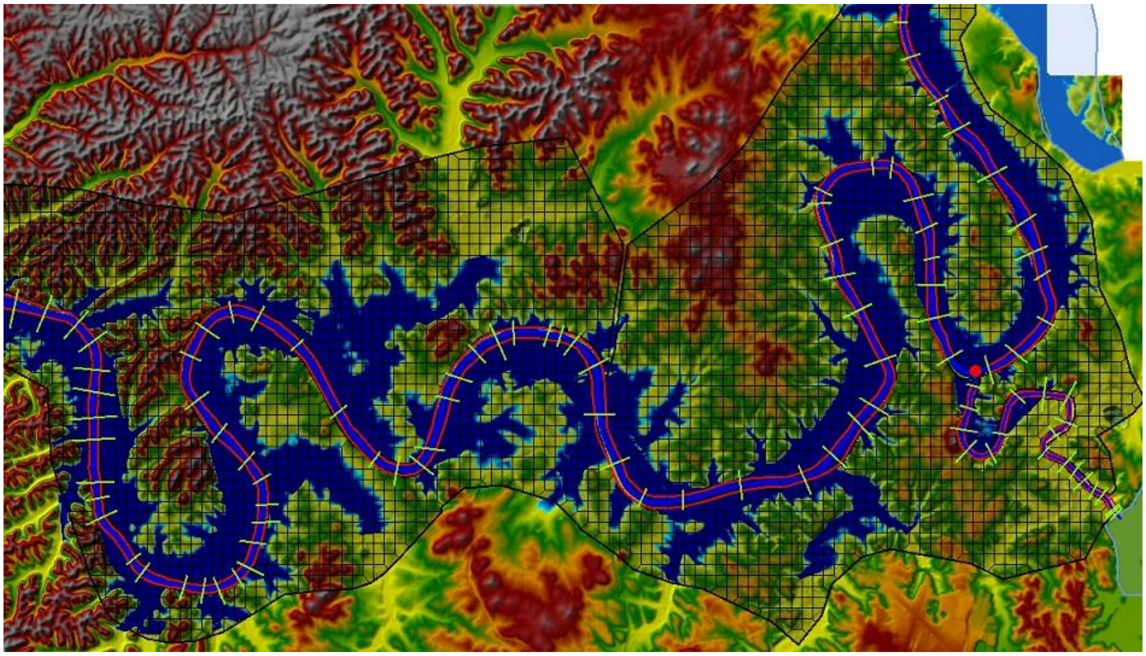

Figure 9.14 Simulated Inundation Map Using Combined One- and Two-Dimensional Approach in HEC-RAS for the May 2010 Flood Event

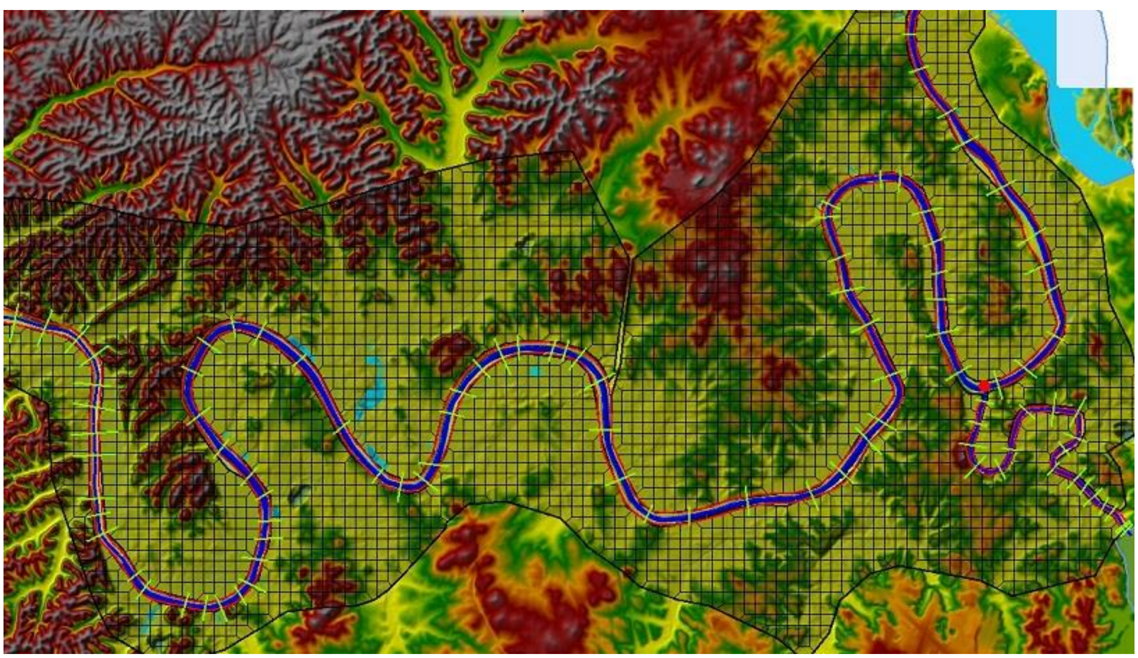

Figure 9.15 Optimized Water Surface Elevations (Inundation) Using Combined One- and Two-Dimensional Approach in HEC-RAS for the May 2010 Flood Event

Contrary to expectations, the two-dimensional simulation model linked with the optimization model resulted in peak discharges that did not exceed 169,694 cfs during the entire period of simulation of May 2010 storm event. This two- dimensional unsteady flow model ran faster than the combined one- and two-dimensional, so the optimization model had more time to improve the solution. The reason why the previous model is slower than this one is because of the connection way between the two-dimensional and one-dimensional areas, which is modeled as very long lateral structures.

The observed water surface elevations in the form of inundation map for the May 2010 event at Nashville using the two-dimensional unsteady flow modeling approach (HEC-RAS) is depicted in Figure 9.16. Figure 9.17 shows the flood inundation area resulting from the application of the optimization-simulation model.

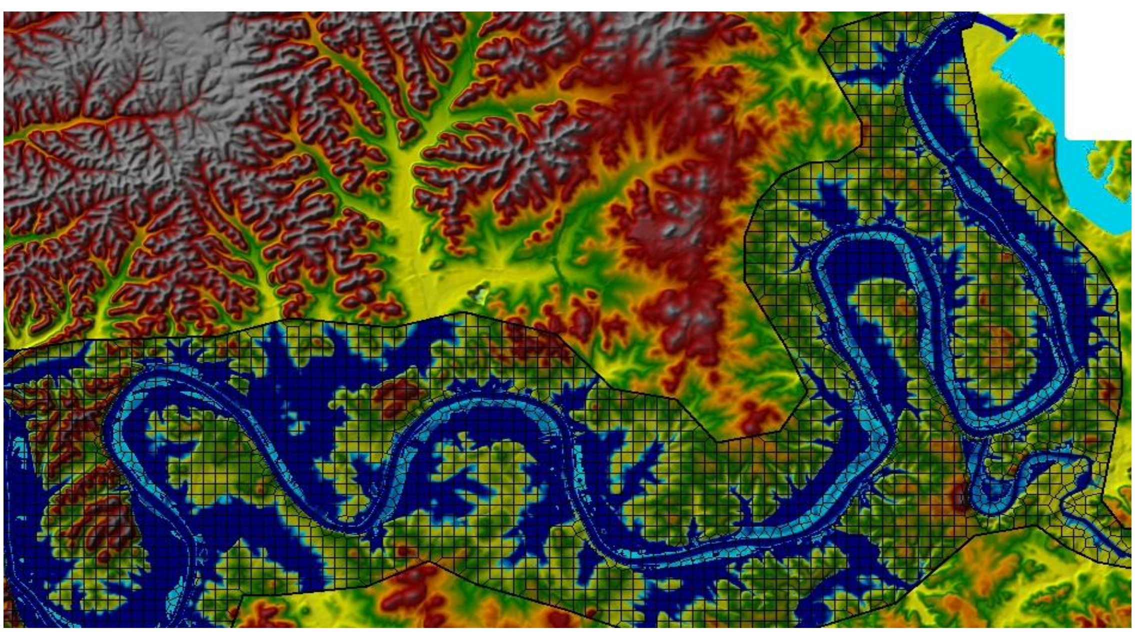

Figure 9.16 Simulated Flooding (Inundation Areas) Using the HEC-RAS Two-Dimensional Approach for the May 2010 Flood Event

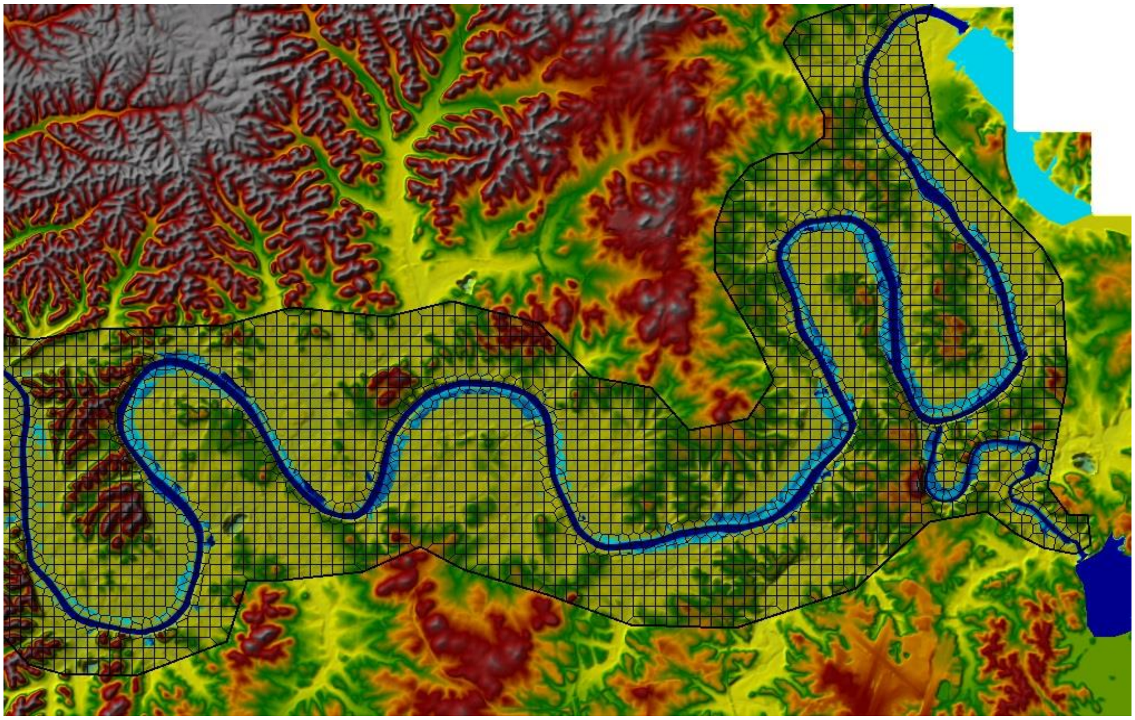

Figure 9.17 Optimized Flooding (Inundation Areas) Using the Two-Dimensional Approach in the Optimization-Simulation Model for the May 2010 Flood Event.

9.6 Operations of Old Hickory Dam

The time series of the dam's gate openings are the decision variables of the optimization–simulation model including the constraints of reservoirs constraints such as gates openings discharge relationships, operation rules of the gates under flooding condition, the gate height hourly rate of change and reservoirs stage storage relationship. The severe storm event that hit Nashville, Tennessee in May 2010 was a very high-frequency storm. Thus, the objective of the optimization-simulation model is to reduce the river flows down to 100- year frequency flows or less by determining optimal gate releases. The model determines the operation for each forecasting period Δt, which is 4 hours for the Cumberland River. The gate operations and the discharges from the Old

Hickory Dam determined using the one- and two and combined one- and two-dimensional optimization-simulation during the May 2010 event versus the actual operation and discharges at Nashville, TN are shown in Figure 9.18 and

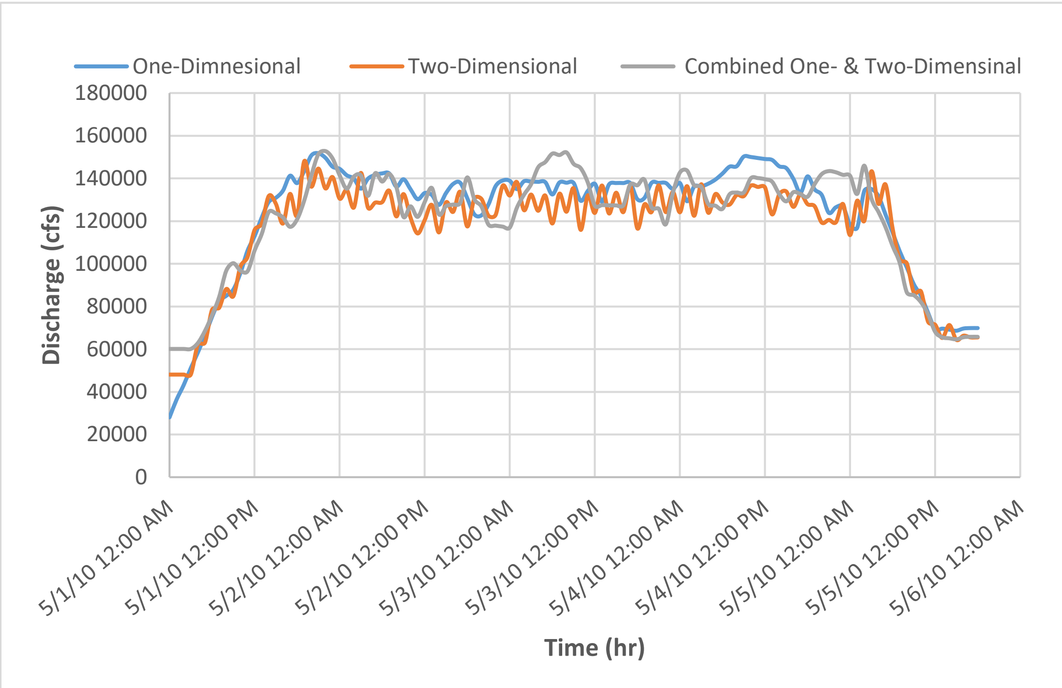

Figure 9.19 respectively.

The actual operation of the Old Hickory Dam during the event started the releases at night on May 1, 2010, despite the forecast warnings from severe rainfall several days in advance. Using the optimization-simulation model with the available forecasting information could have helped the U.S. Army Corps of Engineers make a decision at Old Hickory Dam before the actual storm entered the Old Hickory Reservoir in a timely manner.

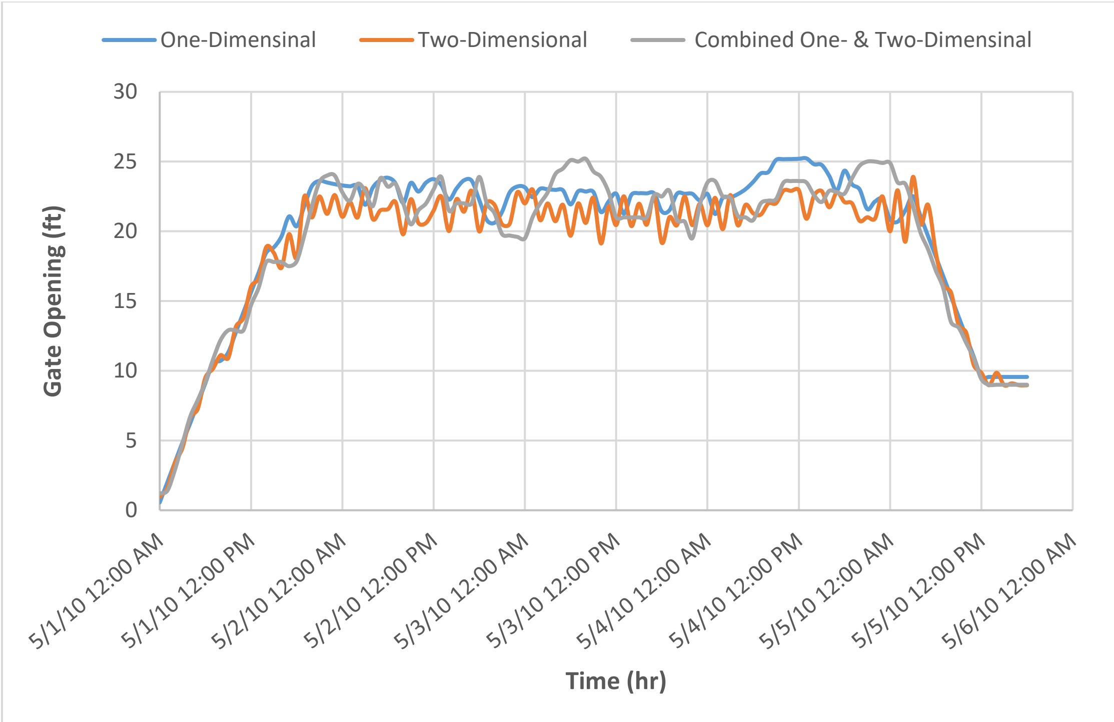

Figure 9.18 Optimized Gates Operation of Old Hickory Dam, May 2010.

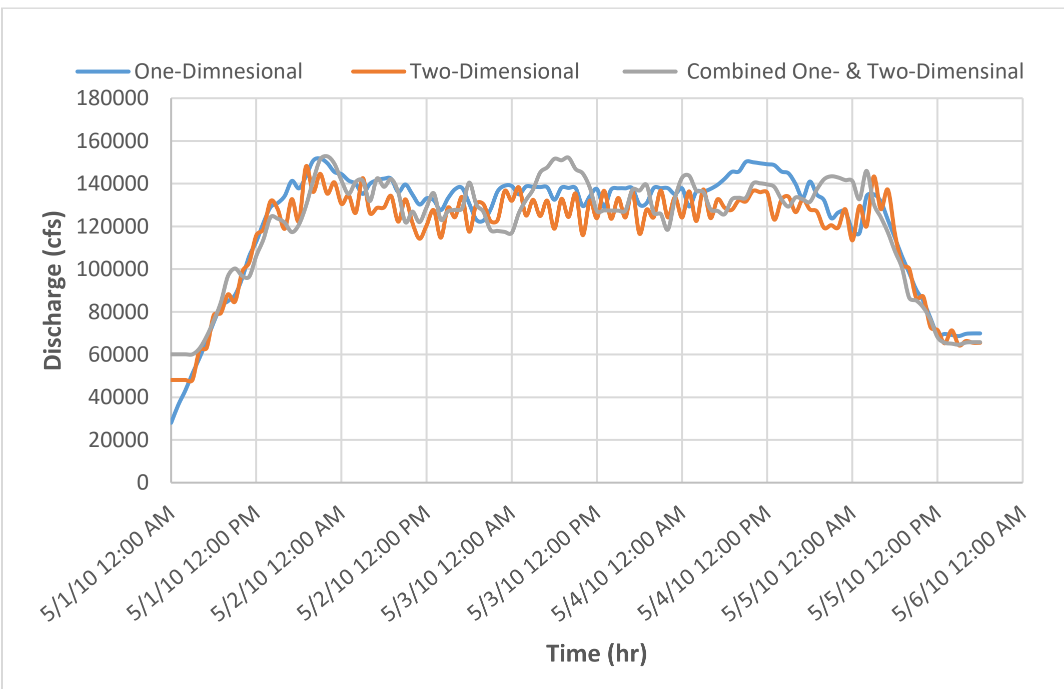

Figure 9.19 Optimized Discharges From Old Hickory Dam for the May 2010 Flood Event.

Woodland gage station at Nashville is considered as the control cell of the optimization-simulation model for both scenarios. It is simulated as a profile line created across the region where the model can read the time series of the passing flow and the stage as well. Figure 9.13 shows the flood stage at the Woodland station.

A comparison is now presented of the resulting optimized operations with the actual operations by the U.S. Army Corps of Engineers during the May 2010 flooding event. Figure 19.20 illustrates the differences between the optimized operations of the Old Hickory flood gates which were all opened the same distance as compared to the actual operations which were all opened the same distance. Obviously, the actual operation of the gates waited too late to open the gates. Figure 19.21 illustrates the optimized and actual releases at Old Hickory Dam during the May 2010 flooding event. Figure 19.22 compares the optimized and simulated flows at Nashville during the May 2010 flooding event.

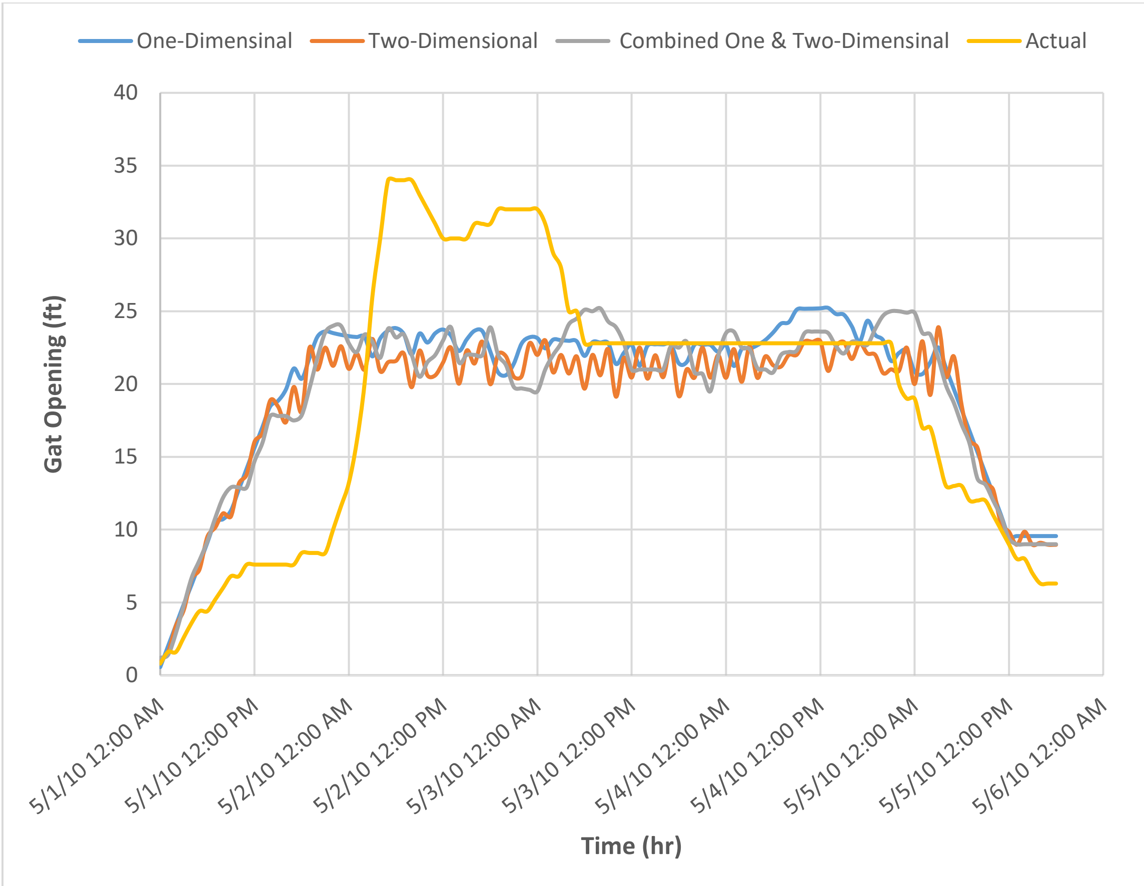

Figure 19.20 Differences Between the Optimized and Actual Operations of the Old Hickory Flood Gate Openings during the May 2010 Flooding Event

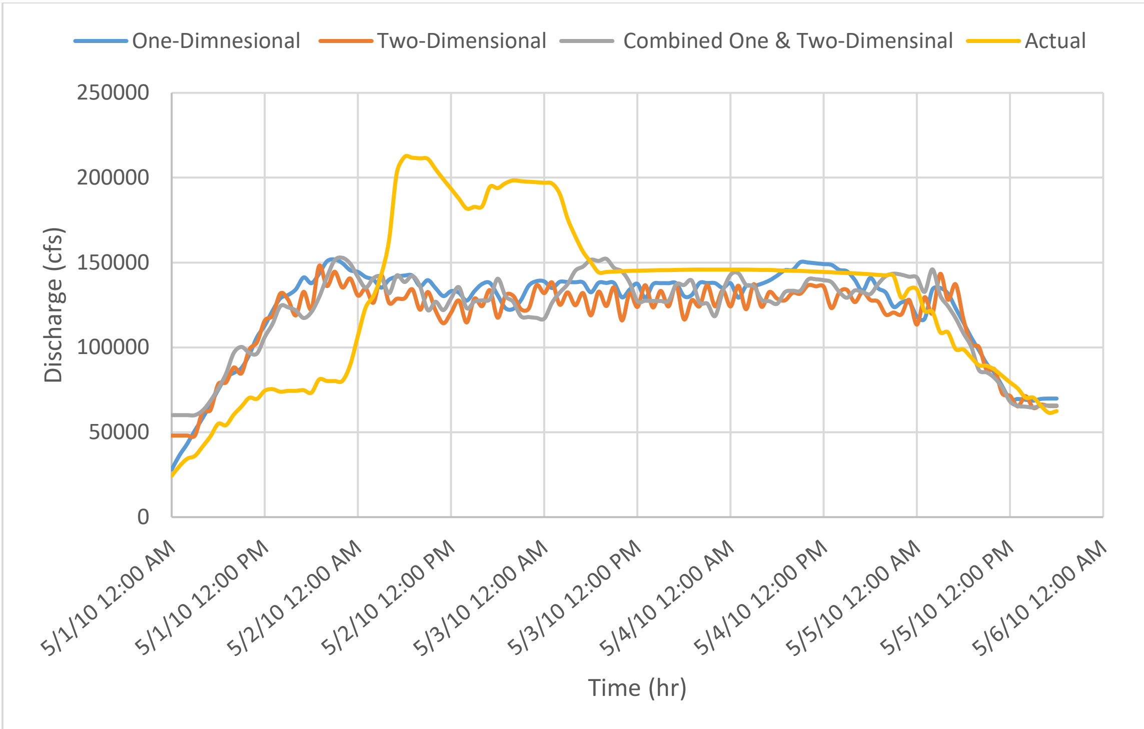

Figure 19.21 Optimized and Actual Releases at Old Hickory Dam during May 2010 Flooding Even

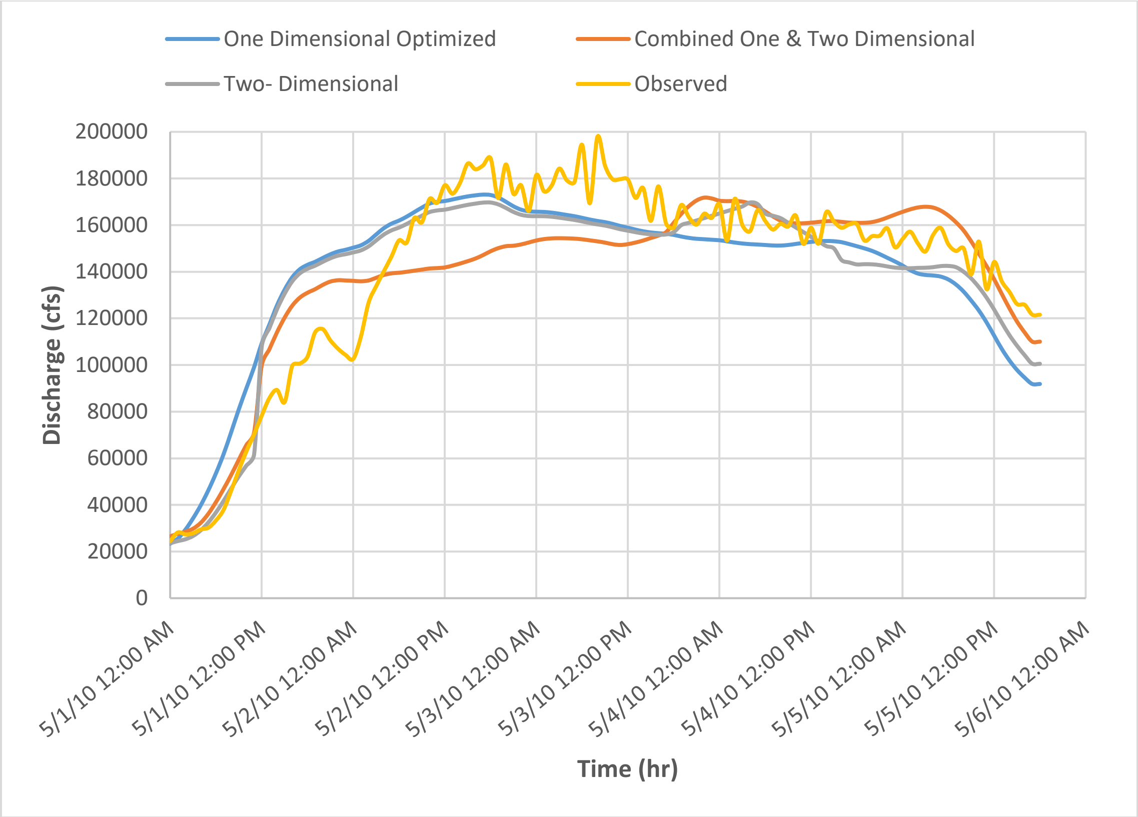

Figure 19.22 Optimized and Simulated Flows at Nashville during the May 2010 Flooding Event

CHAPTER 10 SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS FOR FUTURE RESEARCH

10.1 Summary

An optimization-simulation model has been presented to determine real-time optimal gates operations for a river - reservoir system under severe flood conditions. Reservoir releases schedules before during and after the flood event are necessary to minimize and/or eliminate flooding. The proposed optimization-simulation model consists of five interfaced components: rainfall forecasting model, rainfall-runoff model, one and two-dimensional unsteady flow model, reservoir operation model and an optimization model. Each of these components functions independently of other elements to serve their purpose. The model components communicate through a MATLAB code written for efficient operation.

The one and two-dimensional optimization-simulation model uses the hydrological modeling system HEC-HMS to simulate the rainfall runoff and the newest version of river analysis system HEC-RAS 5.0.6 developed by the U.S. Army Corps of Engineers Hydrologic Engineering Center which has the ability of both one and two-dimensional modeling. The river reaches, and floodplain areas are modeled using the combined one and two-dimensional modeling for both applications. The river reaches were modeled using the one-dimensional and combined and two-dimensional approaches, floodplain areas were modeled using the two-dimensional in both two-dimensional and combined one- and two-dimensional approaches.

Heavy rainfall events are becoming more intense and frequent in different regions in the United States and across the world as well, mostly attributed to climate change. Studies provide valuable evidence of the expectations that extreme rainfall events will continue to increase. Thus, real-time operation of river-reservoir systems is required to avoid and\or to minimize the expected damages that can happen. A measure to enhance and advance the real-time operation of the river-reservoir system is using the two-dimensional modeling. This became possible due to the technology and high speed of computers that helps in reducing the required computation times of two-dimensional simulation. Also, the Hydraulic Engineering Center of the U.S. Army Corps of Engineers facilitated the task of two-dimensional modeling by incorporating this ability to the HEC-RAS to become the first two-dimensional unsteady flow software open to the public for free. Practically, now that two-dimensional modeling is becoming widespread in the HEC-RAS community and attracting the interest of civil engineers due to the reasons explained above.

10.2 Conclusions

The basic conclusions include:

- The research objectives set forth in Section 1.3 of this dissertation have been accomplished.
- A strategy for determining a time series operation for releases through control gates of a river-reservoir system during a flooding event in a real-time fashion has been developed.

- A model has been developed to manage reservoir releases before, during, and after an extraordinary storm event that could cause significant flooding.
- This effort has advanced the methodologies for real-time operation of river-reservoir systems under severe storm conditions.
- The work presented herein adopted different scenarios to simulate the unsteady flow modeling upstream and downstream of a reservoir using one-dimensional, combined one and two- dimensional, and two-dimensional unsteady flow modeling. In One-dimensional simulation, the water surface elevation over each cross section is averaged, thus when a wide flood plain modeled, this approximation of properties is not accurately described as in case of using two-dimensional modeling. So, in many cases, modeling considers one-dimensional flows is difficult to model floodplain flows accurately, as there are numerous directions of water flow on a flat plain. Therefore, the hydraulics of the flood plain needs to be precisely predicted. Usually, modeling flow in a network of channels can be performed using one-dimensional modeling. Also, ate one-dimensional modeling could only determine one resulting water surface elevation at a cross-section. Therefore, the fluctuations across the section will not occur in the model as they would in the case of a real event. However, the one-dimensional analysis can predict good results for river reaches.
- In spite of the extensive studies that many previous researchers have done to analyze floods and flow management, but uncertainties remain about the precise

nature of these changes and flooding problems still occur, causing tremendous devastation of life and property in both the short and long terms.

- One of the most effective measures for flood management is real-time flood forecasting. As a focused activity in the hydro-meteorological sector, flood forecasting is a relatively recent development that might indicate a growing seriousness of flood impacts.
- The process of any hydrologically related forecast is an estimation of the future state of a hydrological event. Like the flow rate, the water volume and level of an area that would be affected or inundated by water and average velocity of flow in a particular region or location of a stream.
- Diffusion of flow in a flood plain includes many issues to be considered, especially in complex topography. During flooding conditions, allocating water stream flow at a particular time can exceed the flood level and then propagate horizontally onto the flood plain in different directions, so it is going to be difficult to model in one direction. Starting with a dry flood plain is another important feature of two-dimensional solutions. Based on the topography of the flooding area, water spreads out in the flood plain in different directions at the beginning of modeling.
- In practice, two-dimensional unsteady flow simulation models are one of the approaches for streamflow and floodplain forecasting as well. For a given set of operation policies, a two-dimensional unsteady flow simulation model can be

used to simulate the flow rates, water surface elevations, and velocities in both X and Y directions at various locations for specified time steps.

- There are various types of one- and two-dimensional unsteady flow models, most of which are commercial models and used in practice.
- Accurate rainfall data in real-time should be available in order to forecast the upcoming inflow to the reservoir and then to decide the outflow from the flood control gates of the reservoir, as the majority of uncertainty and capability to precisely predict the flow and the associated water surface elevations in a river-reservoir system primely because of poor forecasting of rainfall and then under or overestimation of reservoir inflow hydrograph. Thus, real-time flood management needs accurate real-time inflow data to determine how much water should be released from the control facilities. Sometimes inflow data would not be available at the event time so, forecasting the required data for short-term depending on the availability may be required.

10.3 Recommendations for Future Research

10.3.1 Reduction of Number of Simulations of Two-Dimensional Model

The reservoir operation (determination of spillway gate operations) is optimized using the genetic algorithm in MATLAB. The genetic algorithm requires many function evaluations (solutions of the simulator HEC-RAS) to reach an optimal solution.

The application of the optimization – simulation model to the Cumberland River System pointed out this throughout this dissertation. The modeler should present more effort on how to reduce the number of times to call the simulator (in this case HEC-

RAS), because it is the consuming time, to reduce the time of the model running. This will help in making the right decision of the reservoir operation.

The metaheuristic types of optimization approaches have been very valuable in allowing simulation models to be interfaced with optimization. The genetic algorithm approach has become very powerful in allowing the development of optimization – simulation models to be developed. However, these optimization methodologies require a large number of simulations to be performed in many applications. Research needs to be performed to consider other types of optimization approaches that can be interfaced with the HEC-RAS simulator. These optimization approaches include the leap-frog method and others.

10.3.2 Choice of Two-Dimensional Approach

The HEC-RAS program allows the modeler to choose either the Saint-Venant or diffusion-wave equations in two-dimensions to solve the model which are set as the default. In general, the two-dimensional diffusion-wave equations have reduced computation times and have more stable properties than using the full two-dimensional Saint-Venant equations. Even though, a wide range of modeling situations can be precisely modeled with the two-dimensional diffusion-wave equation, the user always has to test if the full Saint-Venant Equations are required for his specific situation by creating another run, then if there will be a significant difference between the two runs plans, the modeler must use the Saint-Venant Equations (full momentum) as it is more accurate. The last, the Hydraulic Engineering Center now is developing a new version of HEC-RAS considering a hydrologic model inside of it, if this version comes out, considering it in the optimization-simulation presented here in this research will subside two components which are the HEC-RAS and HEC-HMS. This saves more time in running the model. Therefore, a study to identify the various types of applications that are more favorable to each of the two types of two-dimensional solution techniques is needed.

10.3.3 Inclusion of Sediment Transport and Erosion into Optimization-Simulation Model

The HEC-RAS model used in the optimization-simulation model has the capability to perform sediment transport and erosion for rivers and flood plains. This component of the modeling system (HEC-RAS) could be coupled with the optimization-simulation model developed in this research to control the sediment transport and movable boundaries resulting from scour and deposition in a river-reservoir system. The optimization-simulation model can be employed to determine the optimum discharges over the system and can be applied whether to a single flood event or to simulate long term trends of scour and deposition in a river reservoir-system.

10.3.4 Inclusion of Water Quality into the Optimization-Simulation Model

The HEC-RAS model used in the optimization-simulation model has the capability to perform water quality analysis of streams, rivers, and reservoirs. This component of the modeling system (HEC-RAS) could be coupled with the optimization-simulation model developed in this research to perform river-reservoir system water quality analysis in a real-time as well for controlling the transportation of several water quality constituents. This can be done through using the optimization-simulation model to determine the optimum release from the reservoir gates while satisficing all the demands, the concentration of the moving water quality constituent and the other constraints.

10.3.5 Use of Optimization-Simulation Model for Developing Operation Manuals

This research would focus on the demonstration of the optimization-simulation model for determining operation rules for river-reservoir systems considering flood operation, sediment transport and erosion control, and water quality operation. Thus, the model can be applied to any river-reservoir system for different scenarios and different rainfall storms frequency to test the system reaction and prepare a flood operation manual and contingency flood plan for the system.

10.3.6 Expand Optimization-Simulation Model for Purposes of Early Flood Warning System

The optimization-simulation model developed herein could be expanded into a framework for use as an early flood warning system. This research work could be very valuable for various groups for the reduction of damages and lives as a result of major flooding events.

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APPENDIX A

SUMMARY OF OLD HICKORY LOCK AND DAM OPERATION

Project: Old Hickory Lock and Dam

Location: Cumberland River Mile 216.2, Davidson and Sumner Counties, Tennessee

The Flood of May 2010 resulted in a peak project headwater of 451.45 feet msl and required an estimated maximum discharge of 212,260 cfs to control reservoir levels. Prior to the Flood of May 2010, based upon National Weather Service rainfall forecasts, the Old Hickory Lock and Dam pool elevation was lowered approximately 0.5 foot. This adjustment was within normal reservoir levels. Beginning at 1200 hours on Saturday, May 1, spillway discharges were initiated at Old Hickory Lock and Dam at a rate of approximately 5,000 cfs/hour until a total flow of 75,000 cfs was reached at 2300 hours on the same day. This discharge was held in an effort to allow local inflows to recede downstream, until 1000 hours on Sunday, May 2, when a series of spillway discharge increases were required as a result of rapidly increasing reservoir elevations. Spillway discharge increases as often as every 15 minutes, and as much as 10,000 cfs each, were necessary to prevent the upstream lock wall from being overtopped. Had the pool climbed 0.55 feet higher it would have overtopped the upstream lock wall, resulting in flooding of the powerhouse, and requiring the complete evacuation of the dam. It has been estimated that such an event would have resulted in a flood crest approximately 4 feet higher in Nashville than was actually experienced.

At 2100 hours on Sunday, May 2, the first spillway discharge reductions were made to manage the crest at Nashville that was forecast at that time to occur at 0100 hours on Monday, May 3. Additional reductions were made throughout the night; however, at 0300 hours on Monday, May 3, spillway discharge increases were again required as large discharges from Cordell Hull and local uncontrolled runoff had pushed the reservoir dangerously high. In an effort to reduce the flood crest through Nashville and protect the lone remaining water treatment plant for the city, spillway discharges were reduced beginning at 1300 hours on Monday, May 3. Between 1300 hours and 1700 hours the total discharge out of Old Hickory Lock and Dam was reduced from 196,500 cfs to 144,200 cfs. These spillway gate settings were held until 0900 hours on Wednesday, May 5 when a series of spillway discharge reductions were made in response to steadily declining inflow to the project. By the end of the day on Thursday, May 6 Old Hickory Lock and Dam was back within its normal operating range.

Flood damages sustained at Old Hickory Lock and Dam are estimated at $11.5M. The debris load, and force of the flood, damaged spillway gates and components, the tail deck crane and collector rails, underwater spillway structures, capstans, handrails, and power cables on the lock and dam and left large amounts of debris in the intake trash screens. Large amounts of debris were deposited on the tail deck and in the left bank tailwater area. The tail deck crane was knocked off of its rails and sustained damage. Severe erosion occurred on the dam embankments and abutments resulting in destabilization of slopes, and on the bluff adjacent to the switchyard, resulting in slope instability which threatens the access road to the power plant as well as the parking lot. Damage occurred at numerous recreation areas and associated facilities; propane and fuel tanks and household chemicals were deposited at various areas across the project. Several campsites were inundated and electrical hookup facilities damaged by water. The nature trail by the dam was impacted as high water washed away the boardwalks and deposited tons of debris. The stream gage on Bledsoe Creek was lost during the event due to the extreme high water.

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