Development of a HEC-HMS Model to Inform River Gauge Placement for a Flood Early Warning System in Uganda

by

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Bachelor of Science in Civil Engineering
University of Michigan, 2012

SUBMITTED TO THE DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF

MASTER OF ENGINEERING IN CIVIL AND ENVIRONMENTAL ENGINEERING
AT THE
MASSACHUSETTS INSTITUTE OF TECHNOLOGY

JUNE 2014

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Abstract

Communities in the downstream region of the Manafwa River Basin in eastern Uganda experience floods caused by heavy precipitation upstream. The Massachusetts Institute of Technology (MIT) has partnered with the Red Cross to develop a Flood Early Warning System (FEWS) that will alert downstream communities of imminent flooding. The first step in the development of the FEWS is to determine if the placement of a river gauge upstream from the existing gauge at Busiu Bridge will be capable of providing an early flood warning. A hydrologic model was developed using HEC-HMS software to determine if this warning is feasible and, if so, to facilitate the optimum placement of a gauge. The HEC-HMS model relates precipitation upstream to river flow downstream. Using an historical precipitation event, the model was calibrated to accurately predict the peak hydrograph caused by the precipitation event. The historical storm is characterized by precipitation evenly distributed over the entire watershed that produced a widespread rise in river height, as opposed to a defined flood wave that moves downstream. This storm served the purpose of calibrating the model, and the analysis of this storm concluded that a gauge upstream of Busiu Bridge will not provide flood warning for a storm characterized by precipitation evenly distributed over the watershed.

The calibrated model was then used to predict the watershed response to a theoretical storm that is characterized by precipitation concentrated upstream. This upstream precipitation event is more likely to produce an upstream flood wave, and is common in the Manafwa River Basin. It was found that, for a storm with precipitation concentrated upstream, an upstream river gauge could be used to provide a flood warning. This study shows that the ability of an upstream river gauge to issue flood warnings is sensitive to the nature of a storm. The developed model produces hydrographs that can be used in a downstream hydraulic model to determine the optimum location for a river gauge in the Manafwa watershed, and the river conditions that would warrant a flood warning.

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Acknowledgements

I could not have written this thesis without the tremendous amount of advice, support, and encouragement I received from many people.

Foremost, I would like to express my deepest gratitude to my advisor Rick Schuhmann for his continuous guidance, support, patience, and friendship throughout the year. His commitment to teaching me how to learn has been an instrumental part of my education at MIT. I appreciate the value he placed on the learning experience, in addition to focusing on the technical results. He taught me the importance of questioning data and assumptions, helping me grow as an engineer by providing a framework to use in approaching problems.

I would also like to thank my teammates Joyce Cheung and Bill Finney for their support on this project, and the friendship that we have developed along the way. I could not have asked for better companions to spend hours with during back-seat car rides in Africa – The Chicken-Place and natural massages were well worth it!

I would like to thank Julie Arrighi for the opportunity to collaborate on such a rewarding project with the Red Cross, and for her continued support throughout the year. She led a productive, fun, and memorable trip to Uganda, and her hospitality is much appreciated. I also want to thank Leo Mwebembezi and Frank Kigozi of the Uganda Ministry of Water and Environment for their assistance and support on the project. Our field work in Uganda would not have been possible without the help from Tumwa, Geoffrey, and Nasa of the Uganda Red Cross, as well as all the other individuals that assisted us, to whom I am grateful for sharing their wonderful country with me.

I am thankful for the advice and technical expertise provided by Yohannes Gebretsadik and Fidele Bingwa throughout the year, and I appreciate their eagerness to volunteer their time.

I would like to thank the other M.Eng advisors and staff for their support and advice throughout the year, and especially my fellow M.Eng classmates for a year that was more fun and rewarding than I could have imagined.

Lastly, I would like to thank my family. My mother for her continuous support and encouragement, my father for his guidance, reassurance, and critiques, and my brother for keeping me grounded through this process, as well as his valuable input on this work.
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1 Introduction

1.1 Project Purpose and Approach

The Manafwa River Basin in Eastern Uganda experiences floods that damage downstream communities. The floods occur when heavy rainfall in the mountainous upstream region flows downstream, overflowing the river's banks in the plains of the Butaleja District. In addition to destroying buildings and crops, contaminating water supplies, and creating food shortages, the floods damage and cut off key roadways, making it difficult for the Red Cross to provide relief. In order for the Red Cross to mobilize ahead of flood events and provide warning for downstream communities, they have partnered with the Massachusetts Institute of Technology (MIT) to develop a Flood Early Warning System (FEWS). The intent of the FEWS is to warn the Red Cross and downstream communities of an approaching flood, facilitating mobilization ahead of the event and reducing potential damage.

The first step in developing the FEWS is to determine if a river gauge installed in the upstream region of the Manafwa Basin will be capable of providing a flood warning for downstream communities, and if so, under what circumstances. Currently there is one river gauge in the downstream region at Busiu Bridge. Figure 1 shows the location of the Manafwa River, the mountainous upstream region (Mt. Elgon), the flooding area of interest (Butaleja), and the region where a river gauge could be placed.

![Figure 1 - Location of Flooding and Potential River Gauge](image-url)
A river gauge is used in flood warning to relate river conditions upstream to future conditions downstream. The optimum location of a gauge must strike a balance between the length of warning lead time that the gauge is capable of providing and the accuracy of the warning provided. A location further upstream corresponds to an earlier, but less accurate warning, and a location downstream corresponds to a later, but more accurate warning. To address these conflicting factors and determine if there is an optimum location for a river gauge that will provide a flood warning, the relationship between rainfall, runoff, river routing, and water level rise must be understood. To understand these relationships and leverage them in the FEWS, hydrologic and hydraulic models are created.

A hydrologic model relates precipitation from a storm event upstream to the resultant flow in the river downstream. The model output is a hydrograph, which is a graph of the river flow over time at a specified location. Hydrographs are generated at different locations along the river and are used to analyze how the river responds to different types of precipitation events. Through this analysis, it is determined under what conditions an upstream gauge is capable of providing a flood warning. To analyze flooding, the hydrograph at a location just upstream of the flooding area of interest is input to a hydraulic model. The hydraulic model relates river flow to water level rise based on the geometric profile of the river bed. The hydraulic model uses the modeled hydrograph to calculate water levels downstream and determine when flooding occurs. By relating the time when overbank flow begins downstream in Butaleja to a flow rate upstream in the Mt. Elgon region, an optimum location for the upstream river gauge can be determined.

The first step in the development of the FEWS is to create the hydrologic model. The accuracy of the hydrologic model depends on how closely it represents the physical characteristics of the Manafwa watershed. This paper focuses on the development of the hydrologic model to accurately represent the watershed’s response to a precipitation event.

### 1.2 Applicability of Hydrologic Models for Flood Early Warning

It is common to use a hydrologic model to create input for a hydraulic model in flood analysis (US Army Corps of Engineers, 2008). In the United States, one of the most commonly used and highly applicable hydrologic models is the Hydrologic Engineering Center Hydrologic Model Simulation (HEC-HMS). This model was developed by the United States Army Corps of Engineers (USACE) and is well suited for application in the United States because of the abundant amount of input data that exist for most domestic watersheds. There are two main classifications of data needed for HEC-HMS: 1) physical watershed characteristic data and 2) observed time-series data.
Physical watershed characteristic data, including elevation, land use and land cover, and soil data maps are readily available in the United States. The United States Geological Survey (USGS) provides ten meter Digital Elevation Maps (DEM) for all of the United States except Alaska, as well as three meter resolution DEMs for large portions of the mainland 48 States (USGS, 2006); the high resolution of these maps (3 or 10 meters) should be noted. Land use maps can be obtained from a number of sources, including the USGS through The National Land Cover Database, which provides 30 meter resolution maps of thematic class, percent impervious surface, and percent tree canopy cover (Homer, et al., 2012). Soil data maps are provided by the United States Department of Agriculture (USDA) Natural Resources Conservation Service, which operates the Web Soil Survey, containing soil maps and data for more than 95% of the nation’s counties (USDA, 2013). Together, these maps provide the basis for the development of the physical basin model as discussed in Section 1.6.2.

The two forms of observed time-series data, precipitation and river flow, are also easily obtained in the United States. The National Oceanic and Atmospheric Administration’s (NOAA) National Climatic Data Center (NCDC) provides historical precipitation data. The data are collected from a number of sources, including satellite and ground stations. Historical hourly precipitation data are available from more than 7,000 gauging stations in the United States, dating back to 1900 (NCDC, 2012). The USGS National Water Information System can provide historical river flow data for approximately 1.5 million gauges across the United States, recorded at intervals ranging from 5 - 60 minutes (USGS, 2011). Again, the high resolution of both precipitation and flow data should be noted. The multiple data sets described here, and similar sources available in the United States, result in a sufficient amount of data, at relatively high resolution and accuracy, to develop and apply HEC-HMS across the nation.

Outside of the United States, HEC-HMS is often applied in flood analysis and has been relatively successful. In Jordan, a study compared the performance of HEC-HMS to another hydrologic model for a single rain event and found that the calibrated HEC-HMS hydrograph fit well with observed data (Abushandi & Merkel, 2013). In Nepal, a HEC-HMS model was developed for flood forecasting that resulted in a predicted peak discharge of 98% of the observed value. The study also suggested that the HEC-HMS approach could be applied to other basins (Kafle, et al., 2010). A literature search resulted in multiple flood related studies that apply HEC-HMS, both within and outside of the United States.

The majority of the HEC-HMS modeling studies were conducted in watersheds in which data availability is comparable to that in the United States. In Jordan, historical precipitation data were available from a combination of satellite measurements and hourly land-based rain gauges. Hourly river flow data were collected from a gauge (Abushandi & Merkel, 2013). In Nepal, 24 land-based rain gauges were available for use in the study, and river flow data were
provided by the local Department of Hydrology and Meteorology. A DEM generated with data gathered by the Survey Department of Nepal was also available (Kafle, et al., 2010). These and other studies demonstrate that HEC-HMS is a valuable tool for flood analysis, when sufficient quantities of high resolution data are available.

1.3 Challenges in Developing a Flood Warning System for the Manafwa Basin

In the Manafwa basin, there is a paucity of both physical watershed characteristic data and observed time-series data. Existing physical characteristic data include 1) a 30 meter resolution DEM; 2) a digital land use map obtained from the United Nations Food and Agriculture Organization (FAO) Africover program; and 3) a digital soil map from the Harmonized World Soil Database. The land use map and soil map were used to develop a curve number map (Bingwa, 2013). The curve number describes a surface’s potential for generating runoff. As Bingwa indicates, the land use map was created in 2001 and, given population dynamics, it is likely that land use has changed since that time; however, this is currently the best source available. There is only one river gauge in the basin, and although data collected there are relatively reliable, the data are recorded daily, a relatively low resolution. There are no other locations within the watershed where river flow data are collected. Rain gauges also exist in the watershed, but none of the gauges have historically reliable records; thus, historical satellite data are used to estimate precipitation, as described in Section 2.2.1. This paucity of data, in quantity, resolution, and relevance, creates the following challenges in developing the hydrologic model.

1.3.1 Model Confidence

The model is fundamentally dependent on precipitation data. As explained in Section 2.2.1, historical satellite precipitation is only an estimate of actual precipitation. The data therefore may not be representative of the actual precipitation that occurred. Another important input for the model is the curve number map, which is dependent on the 2001 land use map. Because a more recent land use assessment is not available, it is likely that the curve number map does not exactly reflect current conditions of the watershed in 2014. These issues decrease confidence in the initial model results.

1.3.2 Calibration Confidence

The limited quantity and resolution of data decreases confidence in the calibration method. As explained in Section 4.2, the model is calibrated by using the observed hydrograph at Busiu Bridge that resulted from an historical precipitation event. After precipitation data are entered into the model, the model performs water balance calculations to generate a modeled hydrograph, and then adjusts watershed parameters in an attempt to reproduce the observed
hydrograph; therefore, corresponding historical precipitation and river flow data are needed to run, and calibrate the model respectively.

To support determining the optimum location for the river gauge, the model must produce results at 15 minute intervals, meaning that the modeled hydrograph will have a flow value every 15 minutes (see Section 3.2.5). Unfortunately, the resolution of the historical hydrograph used for calibration is daily. This difference in resolution is problematic for the calibration procedure because the model is forced to compare 96 simulated values to a single observed value each day, as shown in equation (1.0).

$$\frac{4 \text{ intervals}}{\text{hour}} \times 24 \frac{\text{hours}}{\text{day}} = 96 \frac{\text{intervals}}{\text{day}}$$

Because it is unlikely that the actual river flow was constant for an entire day, a large majority of the 96 simulated values are not compared to the actual flow rate that occurred at the associated times.

Because there is only one location where historical flow rate data were collected, there is only one point in the entire watershed that can be used for calibration. This creates a number of constraints, including:

- The inability to calibrate the model at multiple locations
- The inability to calibrate river routing parameters (to do so, flow data are needed both upstream and downstream of a specific river segment)
- The inability to fine-tune parameters in individual subbasins during the calibration process

As a result, there are few options for calibrating the model, as well as decreased confidence in the calibrated parameters.

### 1.3.3 Analytic Capabilities

The limited quantity of data available for the Manafwa watershed restricts employing the full analytic capabilities of the model. If observed flow was available at more locations, the model could be used to investigate a wider range of results. The existing river gauge is located on the main Manafwa tributary and only allows for calibration of this tributary, upstream of the gauged location. If other tributaries feeding the Manafwa River were gauged, the analysis area could be expanded. Additionally, if gauges were located further downstream, closer to the boundary with the hydraulic model, the hydrologic model would be able to more accurately account for overland flow directly adjacent to the flooding area. Lastly, more precipitation and
flow data could provide a greater variety of storms for use in calibration; currently, only one usable storm was identified as satisfactory for use in model calibration (see Section 2.2.3).

1.3.4 Credibility of Approach Employed

Notwithstanding the challenges discussed above, the available data are sufficient to develop a reliable model for the purpose of this study. This is because, at this stage in the development of the FEWS, the goal is solely to determine the general response of the watershed to a precipitation event. The model will not be linked with real-time data to predict the exact flow rate in the river during or preceding a storm event. Furthermore, future storms will not be identical to the historical storm used for model calibration; thus, replicating the exact 15 minute interval response is not necessary.

The existing data are adequate to derive this general relationship between precipitation and river flow. Although the existing river gauge is located upstream of the downstream boundary of the HEC-HMS model, it is only eight kilometers upstream, equating to about one tenth of the total river length in the HEC-HMS model region. Additionally, about 93% percent of the modeled area drains through the river gauge location; the river gauge is therefore well-placed to facilitate model calibration. Furthermore, as detailed in Section 4.2.2, the model produces accurate results based on the single usable storm event that was identified. The satellite precipitation data appear to agree with the flow rate data for this storm, and thus a land-based rain gauge is not necessary. Although more river and rain gauges and higher resolution maps might increase model accuracy, the calibration process is designed to account for and correct these inaccuracies. The curve number is adjusted during the calibration process, which can correct for the outdated land use map. Because the end goal of this study is to predict the general magnitude and timing of a flood, an approximate hydrograph is more than sufficient.

1.4 Prior Work in the Manafwa Basin

This is the second year (2013-2014) of the FEWS project after its commencement in September of 2012. This year’s work builds upon the work completed last year (2012-2013) by three MIT students in the Master of Engineering (M.Eng) program: Fidele Bingwa, Francesca Cecinati, and Yan Ma. Bingwa developed a curve number map for the Manafwa basin as input for the hydrologic model and investigated the effect of land use changes on flooding potential (Bingwa, 2013). Cecinati looked at precipitation occurring over the Manafwa basin as well as larger climate patterns and their potential in flood prediction (Cecinati, 2013). Ma developed preliminary versions of the hydrologic and hydraulic models and investigated which precipitation loss method is best suited for this study (Ma, 2013).

This paper largely follows Ma’s work on the development of the hydrologic model (HEC-HMS). Although the model developed in 2012-2013 provided a generally informative foundation from
which to build upon, the model results could not be accurately validated because of a lack of reliable calibration data. In this study the model is re-created to more accurately reflect the Manafwa basin and the specific goals of the study. This year’s model differs in three primary aspects:

1. Refined Watershed Conceptual Model
   - The 2012-2013 model is not well aligned with this year’s flooding area of interest. Ma’s hydrologic model covered the majority of the Manafwa basin, including all three tributaries of the river and the flooding area to be analyzed by the hydraulic model this year. Ma’s model is shown below in Figure 2 (Original HEC-HMS Model Region).

The output of the HEC-HMS model should be at a point on the river directly upstream of the flooding area of interest. Additionally, it is now known that there is only one river gauge available for calibration (located at Busiu Bridge, just upstream of the flooding area of interest). Results calculated from a hydrologic model that extends well downstream of the calibration point would be difficult to verify with a high level of confidence; therefore, this year’s model does not include these downstream areas of the watershed. This year’s modeled region lies entirely within subbasin W_150 from Ma’s model. The new modeled region is shown in Figure 3.
This model region conceptually agrees with the purpose of the study and the availability of data that exist.

2. Subbasin Delineation

- When the model was developed in 2012-2013 it was unclear, specifically, what data the HEC-HMS model would need to output to support the placement of a river gauge. It is now clear that the HEC-HMS model must produce simulated hydrographs at specific locations where there is a possibility to place the river gauge; the locations were identified in 2014 through consultations with representatives from the Red Cross (Cheung, 2014). This objective was not accommodated in Ma’s model because these locations had not yet been established, and as explained in Section 3.1.2, a hydrograph can only be produced at subbasin outlets. The model created in this study specifically aligns subbasin outlets with potential river gauge locations.

3. Flow Rate Data for Calibration

- In 2013 the Uganda Ministry of Water and Development provided an Excel data file (lacking dimensions) that was represented as reflecting the daily water level of the river (i.e. stage) at the Busiu Bridge. Bingwa (2013) subsequently attempted to convert this stage data to volumetric flow rate using the Gauckler-Manning formula, under the assumption that the data were in fact representative of river stage. The resulting calculated flow rates were for the
most part not physically possible, making model calibration impossible. During multiple subsequent communications with the Ministry, as recently as December of 2013, the Excel data were represented as daily stages. Rating curves subsequently supplied by the Ministry in fall 2013 with which to convert the stage data to flow resulted in much of the stage data unable to be converted using the rating curves (i.e. the data were outside the rating curve domain) and many converted values outside the realm of physical probability. It was not until the site visit to Uganda in January 2014 that it was learned that the data in fact represent flow rates at Busiu Bridge and not stage. The orders of magnitude difference between stage and flow rate explains the large discrepancy between the observed and modeled hydrographs in Ma’s work. In the calibration performed in this study, the data are correctly represented.

To reconcile these differences, the hydrologic model is recreated in this study based on a new understanding of old data, the incorporation of new data, and a better understanding of the Manafwa basin in general.

1.5 Importance of Documentation

This paper is a foundation document, supporting future use of the model and continued development of the FEWS. It records in detail the decisions made in developing the HEC-HMS model and the reasoning behind them (from this point on, “model” will refer to the HEC-HMS model unless specified otherwise). The paper describes relevant aspects of the study area and the data that are compiled and used in the model. It then describes how the different calculation methods and their associated parameters are selected and derived. Lastly, it explains how the model is calibrated for use in the FEWS and provides an analysis of the modeled results.

1.5.1 Future Development of the Model

Communication of the model in its currently existing form is essential for any future development. Subsequent advancement of the FEWS requires an understanding of how the model methods and parameters were chosen. As described in Chapter 6, there are a number of ways in which the model may be improved. Understanding how it was created will facilitate these improvements. This understanding will also help subsequent users analyze results and understand anomalies, as well as modify the model as needed for a specific use.

1.5.2 Understanding of Model by Stakeholders

An important aspect of a FEWS is the level of confidence users have in its warnings. For a warning to warrant a response, those in the affected area must believe the warning is real. To
contribute to this confidence, the model must be well understood by its stakeholders. In the case of the Manafwa, the primary stakeholder is the Red Cross. The Red Cross must understand how the model works in order to put it to use. Additionally, there is potential for the FEWS to work in conjunction with warning systems operated by the Ugandan government. If this is the case, sufficient knowledge of the model will be necessary to link the two systems.

1.5.3 Application of the Approach to Other Watersheds

If the FEWS is successful in the Manafwa Basin, the approach can be applied to other watersheds. To determine if this approach is applicable to other watersheds, an understanding of the model is necessary. Then, to implement the approach, a description of how the model was developed will provide guidance in creating a new model for a different watershed.

1.6 Software Description

1.6.1 HEC-GeoHMS

To determine the physical watershed parameters required by the HEC-HMS model, the USACE Geospatial Hydrologic Modeling Extension (HEC-GeoHMS) software is used. The software takes advantage of ArcGIS to delineate the watershed and then deduce various parameters. The input for HEC-GeoHMS is the 30 meter resolution DEM file. For a complete description of the software, the reader is referred to the HEC-GeoHMS User’s Manual (US Army Corps of Engineers, 2013).

HEC-GeoHMS is selected for this study because of its ease of use with HEC-HMS. Although there are other watershed delineation tools, HEC-GeoHMS was developed specifically to use in conjunction with HEC-HMS.

1.6.2 HEC-HMS

HEC-HMS is a precipitation based runoff and routing modeling system used to develop and analyze a watershed’s hydrologic relationships. The basic function of the model is to receive precipitation as an input, determine what volume of this precipitation infiltrates the ground versus what volume becomes overland runoff, to route this overland runoff towards the river, and finally to route the flow down the river to determine the total flow downstream. The model is capable of performing both event and continuous simulations. Because this study is an analysis of flash floods, the focus of this study is on event simulation. The model allows the user to choose from a variety of calculation methods to compute each part of the hydrologic process (US Army Corps of Engineers, 2010). For the purpose of this study, the model is conceptualized as shown in Figure 4.
The modeling system has two primary components. Within each component are different processes in the hydrologic cycle. To represent each hydrologic process, there are a variety of calculation methods to choose from. Finally, for each calculation method, specific input parameters are required.

The foundation of the model is the physical basin map imported from HEC-GeoHMS. This is a graphical representation of the different elements of the watershed, including subbasins, river reaches, and junctions. The basin map is shown below in Figure 5.
Parameters are associated with each subbasin and reach element in the basin map. These parameters are derived from the physical watershed characteristic data presented in Section 2.1. For each element, the user selects calculation methods to represent each process in the hydrologic cycle. The first process calculates the volume of precipitation that becomes runoff (precipitation loss method). The second process routes this runoff overland to the river reach (overland flow method). Twelve different precipitation loss methods and seven different overland flow methods are available for use. After calculating the total runoff produced in each subbasin, the reach routing process moves the water downstream through each reach. There are six different reach routing methods available for use. The choice of method is governed by the available data, the intended use of the model, and the modeler’s knowledge of the specific watershed. The reader is referred to the HEC-HMS User Manual (US Army Corps of Engineers, 2010) and Technical Reference Manual (US Army Corps of Engineers, 2010) for a complete description of each method. The methods selected in this study are presented in Section 3.2.

The meteorological component is the second major element of the HMS model. This part of the model is responsible for defining the meteorological conditions that the basin model experiences. It is composed of the evapotranspiration, snowmelt, and precipitation processes. Evapotranspiration and snowmelt are not used in this analysis: evapotranspiration is not significant in event simulation and the Manafwa Basin is equatorial and does not experience significant snowmelt, even at the highest elevations of Mt. Elgon. The precipitation process describes the precipitation event that the basin experiences. There are seven different precipitation methods to choose from and the most applicable method is chosen based on how
historical data are provided. The selected method in this study is presented in Section 3.2. The reader is again referred to the HEC-HMS User Manual and Technical Reference Manual for a complete description of each method.

To apply the HEC-HMS software to this study, model input data are required. Chapter 2 provides a description of the watershed and the data sets that are available within it, including physical watershed characteristic data, precipitation data, and river flow data. In addition to explaining how the data sets are derived, Chapter 2 provides a discussion of their applicability to this study. It concludes with an explanation of how the specific storm event used in this study is selected.
2 Study Area and Data Sets

The Manafwa basin is located in eastern Uganda, just north of the equator. Its most upstream segments begin on the slopes of Mt. Elgon, a large (~4,000 m) extinct shield volcano. The landscape then rapidly transitions to low lying (~1,000 m) plains, through which the Manafwa River meanders on its way to Lake Kyoga. The lake and the Manafwa basin are part of the Nile river basin through their connection with the White Nile. The key characteristics of the watershed with respect to flooding are the steep slopes of Mt. Elgon and the low lying plains below, which together with a heavy precipitation event, create a high risk for flash floods.

2.1 Physical Watershed Characteristic Data

Data describing the physical characteristics of the watershed are needed for model development. The two data sets used in the model are:

1. The DEM, used to delineate the watershed, define river segments, and calculate runoff and routing parameters. A 30 meter DEM of the Manafwa watershed region was obtained from NASA (Ma, 2013).

1. The Curve Number Map, used to calculate precipitation loss over the watershed. Bingwa developed a curve number map of the Manafwa watershed (Bingwa, 2013); a version of the curve number map, trimmed to the HEC-HMS model region, is shown in Figure 6 below:
2.2 Time-Series Data

As described in Section 1.3.2 and further in Section 4.2, historical data are necessary for model calibration; specifically, historical precipitation and the corresponding river flow. Because the goal of this study is to model flooding, historical data that correspond to a flood event are required. The watershed’s response to precipitation may be different depending on the magnitude and duration of the event, as well as the hydrologic state of the watershed in advance of a storm. Using known flooding events to calibrate the model ensures that calibrated parameters are representative of the watershed during a flood event. The data are entered into the model as time-series data, with each value corresponding to a specific date and time.

2.2.1 Precipitation Data

The precipitation data used in the model are obtained from satellite estimates, specifically the Tropical Rainfall Measuring Mission (TRMM), a joint mission between NASA and its Japanese counterpart to monitor rainfall (NASA, 2014). The data can be downloaded for specific square grid cells. These squares have sides of length 0.25 degrees longitude and 0.25 degrees latitude. Every part of the Earth’s surface within each grid cell is assumed to experience the precipitation associated with that grid cell. An overlay of the grid cells relevant to the Manafwa model region can be seen below in Figure 7.

![Figure 7 - TRMM Grid Cells Overlaid on the Manafwa Region](image)

The three labeled grids cells, NE, S, and SE are used to define precipitation for the HEC-HMS model because these are the cells that lie over the HEC-HMS model region. The daily precipitation values for these three grid cells were downloaded from TRMM and organized in a
tabular form. One value is provided for each cell for each day; this value represents the total amount of precipitation that fell over each point in the grid cell during that day. Thus, every piece of land in each cell is assumed to experience the same amount of precipitation. See (Finney, 2014) for a more detailed explanation of TRMM data and how it was extracted.

It is important to note that the TRMM data are only an estimation of rainfall based on cloud density and atmospheric moisture measured by a satellite; these data are not the actual measured rainfall. It is unlikely that the entire grid cell experiences the same exact rainfall at the same exact time; rainfall is likely more concentrated in some areas than others. In addition to the degree of inaccuracy in the location of precipitation, there is also inaccuracy in the amount of precipitation. The most common approach to determine the actual amount of precipitation is the use of a land based rain gauge. Although gauges do exist in the watershed, their data are unreliable and inaccurate (Finney, 2014).

2.2.2 River Flow Data

The river flow data used in the model are obtained from a river gauge at Busiu Bridge.

The gauge at Busiu Bridge measures the height of water in the river (stage); the stage can either be read through an automatic system or manually. Because of ongoing technical difficulties with the automatic system, the gauge is currently read manually twice each day and the values are averaged to determine the water level in the river for that day.
A rating curve is used to relate river stage to the flow rate at Busiu Bridge. The rating curves for Busiu Bridge are shown below in Figure 10.

The rating curves are derived from an historical set of stage and flow rate measurements taken at Busiu Bridge over a varied set of flow conditions. Each point on the graph represents a specific measurement. The measurements are then plotted and a curve is fit to provide an equation to estimate intermediate stage heights. The three curves are valid for three different time periods; because the riverbed geometry changes over time, the curves must be updated to reflect the new geometry. For this study, data corresponding to curve C are used, and the equation is shown as an insert in Figure 10.

2.2.3 Storm Event Selection

To select the dates of specific storms for use in model calibration, periods of increased precipitation and river flow are identified. This is done by graphing precipitation over time and flow rate over time and selecting peaks in the graphs, with the assumption that a peak in either measurement will generally correspond to a peak in the other. Although increased precipitation is the cause of a flooding event and increased flow rate is the effect, the flow rate data are used to identify storms for two reasons: 1) The flow rate data are more reliable than the precipitation data and 2) although a storm is necessary for a flood, not every storm will
cause a flood. In contrast, when there is large increase in the river flow, it is likely that a flood will occur.

To determine which peaks in the river flow data correspond to a flood, the following criteria are considered:

**Table 1 - Storm Selection Criteria**

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Relevance for Storm Selection</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. <strong>Storm Magnitude</strong></td>
<td>Selected storms must be intense enough to cause flooding. Although there is currently insufficient information linking specific river flow records with the exact timing of a corresponding flood, it is assumed that only the highest peaks caused historical floods.</td>
</tr>
</tbody>
</table>
| 2. **Storm Definition and Length** | a. The storm must be discernible from surrounding data to model it as an event.  
                                | b. Preceding and antecedent river flow must be low enough that base flow can be assumed during these periods, and for simplicity antecedent soil moisture neglected.                             |
| 3. **Storm Date**               | The storm should be relatively recent, at least within rating curve C (Figure 10). This ensures that the model is an accurate representation of the current geometry of the river.                                                       |
| 4. **Realistic Data**           | Because the flow data are manually recorded, there may be error in the values. It is important that each value of the hydrograph makes physical sense (i.e. no extreme and unexplainable rise or fall in the hydrograph).                             |
| 5. **Accurate Corresponding TRMM Data** | The corresponding precipitation data should align well with the hydrograph. This may not always be the case because of the inaccuracy in TRMM estimates.                                                                 |

In January 2014, the Uganda Ministry of Water and Environment provided flow data at Busiu Bridge from 1948 – 2013. Data were first trimmed to the most recent rating curve, which is applicable beginning in 2001 (Criterion 3).
Because the greatest peaks appear after 2006, the data were again trimmed to assist in selecting specific storms for calibration consideration.

At this resolution it is possible to select individual peaks that represent significant storms (Criteria 1 & 2). Five initial storms were selected; both the date and magnitude are shown in Figure 12. The storms are evaluated individually to determine applicability.
Figure 13 - Hydrographs at Busiu Bridge – Selected Storms

The June 2007 and February 2010 storms were excluded because they do not meet the criteria presented in Table 1. The peak flow rate of the June 2007 storm is not high enough to ensure a flood occurred (Criterion 1). Additionally, both storms exhibit an unrealistic drop in flow rate during the peak, which is likely an error in the data (Criterion 4).

The November 2006, May 2010, and December 2011 storms were then compared to the corresponding TRMM precipitation (Criterion 5).
The November 2006 precipitation and river flow data agree well. The increase in flow is near, but slightly lagged behind, the increase in precipitation. This lag is expected because the water is slowed down as it flows downstream on its way to Busiu Bridge. In the May 2010 storm, the increased flow does not agree with the TRMM measured precipitation data. There is no observed increase in precipitation that could have produced this observed increase in flow. Because there is more confidence in flow data than in TRMM data, it is likely that the TRMM satellite did not accurately capture the precipitation event. In the December 2011 storm, there
are peaks in both precipitation and flow, but the peaks do not correspond with each other. The precipitation is well ahead of the increased flow, implying that the lag time of the watershed is on the order of magnitude of days, as opposed to hours. A lag time on the order of magnitude of days is unlikely given the size and characteristics of the watershed. Again, the error is most likely attributed to inaccurate TRMM estimates.

For a storm to be useful in model calibration, the measured precipitation data must match the measured hydrograph. This is because the calibration process compares this measured hydrograph to the hydrograph generated by the model; it then adjusts model parameters to correct for any differences between the two hydrographs. The May 2010 and December 2011 measured hydrographs cannot be attributed to the corresponding historical precipitation data. Therefore, the model will not be able to use this historical precipitation data to generate a simulated hydrograph that matches the measured hydrograph. As a result, the May 2010 and December 2011 storms are not used in the model; instead, the November 2006 storm is used because the flow and precipitation data for this storm are well aligned.

The large magnitude of flow during this November 2006 storm indicates that a flood likely occurred. Historical records also indicate that a flood occurred in the Butaleja district between October and December of 2006, affecting 4,000 people (Cecinati, 2013). This provides assurance that the elevated precipitation and flow rate did in fact cause a flooding event.
3 Model Development

This chapter explains how the HEC-HMS model is created. It describes how parameters are derived and then applied to the different calculation methods used in the model.

3.1 Utilization of HEC-GeoHMS in Model Development

3.1.1 Defining the Model Outlet Point

HEC-GeoHMS is used to delineate the watershed and determine physical characteristic parameters. The first step in delineation is to determine an outlet point. The outlet point is the most downstream point along the Manafwa River that is analyzed by the model; all hydrologically contributing area upstream of this point is included in the modeled region.

The outlet point was chosen based on two criteria: 1) proximity to the Red Cross flooding area of interest and 2) proximity to the calibration point at Busiu Bridge. Because precipitation that falls downstream of the outlet point is not accounted for in the model, it is beneficial to place the hydrologic model outlet at the upstream boundary of the flooding area, where the hydraulic model begins (hydraulic model inlet). If the hydrologic model outlet is disconnected from the hydraulic model inlet (i.e. the hydrologic model outlet is placed significantly upstream of the hydraulic model inlet), then the hydrograph input to the hydraulic model might be underestimated. The proximity of the outlet to the calibration point located further upstream is also important. This is because any contributing flow downstream of the calibration point cannot be verified by historically observed data; therefore, the parameters in this area cannot be calibrated with a high degree of confidence. This decrease in calibration confidence is recognized, but as explained in Section 1.3.4, this area only accounts for a small portion of the overall contributing area; hence the impact on overall model accuracy is minimal. Figure 15 shows the placement of the outlet point.
After selecting this outlet point in HEC-GeoHMS, the software uses the DEM to determine the bounds of the modeled region; this area defines the modeled watershed.

3.1.2 Defining Subbasins

Once the model region is defined, it is necessary to define subbasins within this region. Subbasin delineation in this study is based on possible river gauge locations. Because runoff volume is calculated on a subbasin scale, a hydrograph can only be produced at the outlet of each subbasin and at river junctions. A hydrograph is needed at each potential location to determine the optimum river gauge location; see Cheung’s 2014 work for an analysis of the potential river gauge locations. These locations are used to define subbasin outlet points. This results in six subbasins as shown in Figure 16.
Figure 16 - HEC-HMS Model Region and Subbasins

The outlet points of subbasins 6, 5, 4, and 3 are all potential river gauge locations. The outlet point of subbasin 2 is the Busiu Bridge calibration point. The outlet of subbasin 1 is the terminal outlet of the HMS model and produces the hydrograph that supports downstream flood modeling.

3.1.3 Deriving Physical Watershed Parameters

After defining the watershed subbasins, HEC-GeoHMS is used to calculate the physical parameters associated with each subbasin and river reach used in the HMS model. These parameters are:

- Subbasin Area
- River Reach Slope
- River Reach Length
- Length from subbasin outlet to a point nearest the subbasin centroid

The calculations are based on the DEM file; a more detailed description can be found in the HEC-GeoHMS User’s Manual (US Army Corps of Engineers, 2013) and an explanation of how the parameters are used can be found in Section 3.2.

3.2 HEC-HMS Methods and Parameters

As described in Section 1.6.2, the model uses different calculation methods to describe each process of the hydrologic response of the watershed, and each method has associated input
parameters. Figure 17 is a graphical representation of the HEC-HMS processes, methods, and associated parameters used in this study. Base flow, canopy storage, surface depression storage, and channel loss were neglected in this study and do not appear in the analysis (see Section 6.1 for a brief explanation).

The following sections describe the methods chosen for each process and their associated data requirements.

### 3.2.1 Defining Precipitation

As described in Section 2.2.1, historical precipitation data are available in the form of TRMM grid cells. To determine the amount of precipitation that falls over each subbasin, the Gauge Weights Method is used. This method allows the modeler to apply TRMM precipitation to the different subbasins weighted by the area of the subbasin that lies within each TRMM cell. These areas are shown as percentages in Figure 18.
Using these percentages, or “gauge weights”, the total daily precipitation for each subbasin is calculated. For example, the precipitation applied to the area in subbasin 3 for any given day is 76% of the SE TRMM cell value + 24% of the S TRMM cell value. This total, in mm/day, is assumed to fall over the entire subbasin.

### 3.2.2 Defining Precipitation Loss in a Subbasin

The goal of the precipitation loss process is to determine what percentage of precipitation infiltrates through the ground and what percentage becomes runoff, contributing to river flow. For this study, the SCS Curve Number method was selected. The method is popular for ungauged watersheds (Bedient, et al., 2008) and was selected primarily because the parameters it requires are available. The method was developed by the Soil Conservation Service (SCS) and uses soil cover, land use, and antecedent soil moisture to determine...
precipitation excess (US Army Corps of Engineers, 2010). The method requires three input parameters:

1. **The Curve Number (CN)** is the principle parameter of the SCS Curve Number Method and is estimated as a function of land use and soil type. The average curve numbers for each subbasin are derived from the curve number map and are shown in Table 3.

<table>
<thead>
<tr>
<th>Subbasin 1</th>
<th>Subbasin 2</th>
<th>Subbasin 3</th>
<th>Subbasin 4</th>
<th>Subbasin 5</th>
<th>Subbasin 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Curve Number</td>
<td>80</td>
<td>74</td>
<td>78</td>
<td>73</td>
<td>72</td>
</tr>
</tbody>
</table>

2. **Initial Abstraction (I_a)** defines the maximum amount of precipitation absorbed by the ground before runoff begins to occur. The initial abstraction is calculated as a fraction of the potential maximum retention (S), which is the maximum total amount of precipitation that can be absorbed by the ground, and is a function of the CN. The relationships between curve number, potential maximum retention, and initial abstraction in SI units are shown in equations (2.0) and (3.0) below.

   \[
   S = \frac{25400}{CN} - 254 \quad [SI \ Units] 
   \]

   \[
   I_a = 0.2 \times S 
   \]

   These relationships were determined empirically by the SCS from the analysis of many watersheds (US Army Corps of Engineers, 2010). Using the curve numbers shown in Table 3, the initial abstraction for each subbasin is calculated and shown in Table 4 below.

<table>
<thead>
<tr>
<th>Subbasin 1</th>
<th>Subbasin 2</th>
<th>Subbasin 3</th>
<th>Subbasin 4</th>
<th>Subbasin 5</th>
<th>Subbasin 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Abstraction (mm)</td>
<td>12.70</td>
<td>17.85</td>
<td>14.33</td>
<td>18.79</td>
<td>19.76</td>
</tr>
</tbody>
</table>

3. **Percent Impervious** defines the percentage of the subbasin’s surface that is considered impervious. Bingwa found that percent impervious values for the Manafwa Basin in 2012 could range from 0.85 – 2.13 %; a value of 1% impervious was used for this study in each subbasin.

Using these three parameters, the model calculates the volume of runoff in each time step.
3.2.3 Defining Overland Flow in a Subbasin

The overland flow process describes how the volume of excess precipitation is transformed to runoff at a specific point (termed the Transform Method in HEC-HMS). In this study, the SCS Unit Hydrograph method is chosen. This well-established empirical method is based on the analysis of many studies conducted in agricultural watersheds in the United States (Bedient, et al., 2008). From these studies, a relationship was derived relating the magnitude and time of the peak hydrograph to the lag time and area of each subbasin. The area of each subbasin is calculated in HEC-GeoHMS and shown in Table 5 below.

<table>
<thead>
<tr>
<th>Subbasin</th>
<th>Subbasin 1</th>
<th>Subbasin 2</th>
<th>Subbasin 3</th>
<th>Subbasin 4</th>
<th>Subbasin 5</th>
<th>Subbasin 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area (km²)</td>
<td>34.938</td>
<td>58.474</td>
<td>78.047</td>
<td>107.04</td>
<td>59.201</td>
<td>178.25</td>
</tr>
</tbody>
</table>

The lag time is defined as the time between the centroid of excess precipitation and the peak of the resultant hydrograph (Bedient, et al., 2008). Lag time can be calculated through a variety of methods; two common methods are the SCS Unit Hydrograph Method and the Snyder Method.

**Method 1 - SCS Unit Hydrograph:**

\[
t_{Lag} = \frac{l^{0.8} \cdot (S + 1)^{0.7}}{1900 y^{0.5}}
\]

Where:
- \( t_{Lag} \) = Basin Lag Time (hrs)
- \( l \) = length from subbasin outlet to divide along longest drainage path (ft)
- \( y \) = Subbasin slope (%)
- \( S = \frac{1000}{CN} - 10 \) (in)
- \( CN \) = Average curve number for subbasin

This results in the lag time as a function of curve number:

\[
t_{Lag} = \frac{l^{0.8} \cdot (\frac{1000}{CN} - 10 + 1)^{0.7}}{1900 y^{0.5}}
\]

**Method 2 - Snyder Method:**

\[
t_{Lag} = C_t (L/L_c)^{0.3}
\]

Where:
- \( t_{Lag} \) = Basin Lag Time (hrs)
- \( C_t \) = Basin coefficient (Not a physical parameter. Usually ranges from 1.8 – 2.2)
- \( L \) = Length along main stream from subbasin outlet to subbasin divide (mi)
- \( L_c \) = Length along main stream to the point nearest the subbasin centroid (mi)
Although the SCS method for calculating lag time was developed as part of the SCS Unit Hydrograph Method for overland flow, it is not well suited for this study. This is because of its dependence on the curve number; during the calibration process, the model adjusts the curve number to approximate the observed hydrograph (see Section 4.2). Such interdependence in the model (between lag time and curve number) can make it difficult to obtain a true estimate of either parameter. Using the Snyder method allows the lag time to be calculated independent of other model parameters. This is explained in more detail in the HEC-HMS Technical Reference Manual and Hydrology and Floodplain Analysis (Bedient, et al., 2008).

To calculate lag time using the Snyder Method, \( C_t \), \( L \), and \( L_c \) must be calculated. The recommended range of \( C_t \) is between 1.8 and 2.2 (Bedient, et al., 2008), with lower values corresponding to steeper slopes. Accordingly, lower \( C_t \) values are applied to subbasins located further upstream in the mountainous region. The \( C_t \) designated for each subbasin is shown in Table 6.

<table>
<thead>
<tr>
<th>Subbasin</th>
<th>( C_t )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.20</td>
</tr>
<tr>
<td>2</td>
<td>2.12</td>
</tr>
<tr>
<td>3</td>
<td>2.04</td>
</tr>
<tr>
<td>4</td>
<td>1.96</td>
</tr>
<tr>
<td>5</td>
<td>1.88</td>
</tr>
<tr>
<td>6</td>
<td>1.8</td>
</tr>
</tbody>
</table>

To find \( L \), the length along the main river from subbasin outlet to divide, HEC-GeoHMS is used. The software segments the main river into lengths that span each watershed.
The length of each individual segment is measured, which is equivalent to $L$ for that subbasin. The $L$ for subbasin 6 was calculated separately because the main river was not defined to stretch into this most upstream subbasin. To calculate $L$ for subbasin 6, the HEC-GeoHMS Longest Flow Path Tool was used. This tool measures the length of the longest possible flow path in each subbasin, not necessarily along the main river.

For subbasin 6, this longest flow path length was used as $L$. The $L$ value designated for each subbasin is shown in Table 7.
HEC-GeoHMS was also used to determine $L_c$, the length along the main river from the subbasin’s outlet to a point nearest the centroid of the subbasin. The software provides three methods for determining the centroid (US Army Corps of Engineers, 2013):

1. The Center of Gravity Method places the centroid at the center of gravity of the subbasin. If this location is outside of the subbasin, the centroid is placed on the closest boundary.
2. The 50% Area Method places the centroid along the main river at the point where 50% of the contributing area is accounted for.
3. The Longest Flow Path Method places the centroid at the midpoint of the longest flow path in the subbasin.

The results of these three methods are shown below in Figure 21.

For all subbasins except subbasin 4, the Center of Gravity Method provides an accurate estimate for the centroid because the centroid is placed near the middle of the basin and not

---

**Table 7 - Subbasin $L$ Values Used in HMS Model**

<table>
<thead>
<tr>
<th>Subbasin</th>
<th>$L$ (m)</th>
<th>$L$ (mi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8,766</td>
<td>5.45</td>
</tr>
<tr>
<td>2</td>
<td>10,082</td>
<td>6.26</td>
</tr>
<tr>
<td>3</td>
<td>10,390</td>
<td>6.46</td>
</tr>
<tr>
<td>4</td>
<td>4,237</td>
<td>2.63</td>
</tr>
<tr>
<td>5</td>
<td>5,337</td>
<td>3.32</td>
</tr>
<tr>
<td>6</td>
<td>29,760</td>
<td>18.49</td>
</tr>
</tbody>
</table>
on a boundary. For subbasin 4, the 50% Area Method was chosen instead. The chosen centroid locations used in the calculation of $L_c$ are shown in Figure 22.

$L_c$ is then calculated by the software as the length along the main river from the subbasin’s outlet to the point nearest to the subbasin centroid location; those lengths are shown in Figure 23.

The values of the length $L_c$ for each subbasin shown in Figure 23 are tabulated in Table 8.
Table 8 - Subbasin $L_c$ Values Used in HMS Model

<table>
<thead>
<tr>
<th>Subbasin</th>
<th>$L_c$ (m)</th>
<th>$L_c$ (mi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5,961</td>
<td>3.70</td>
</tr>
<tr>
<td>2</td>
<td>8,217</td>
<td>5.11</td>
</tr>
<tr>
<td>3</td>
<td>8,475</td>
<td>5.27</td>
</tr>
<tr>
<td>4</td>
<td>2,212</td>
<td>1.37</td>
</tr>
<tr>
<td>5</td>
<td>4,929</td>
<td>3.06</td>
</tr>
<tr>
<td>6</td>
<td>13,277</td>
<td>8.25</td>
</tr>
</tbody>
</table>

Using equation (6.0) and the values of $C_t$, $L$, and $L_c$ derived from HEC-GeoHMS, the lag time for each subbasin is determined and is tabulated below in Table 9.

Table 9 - Subbasin Lag Time Values Used in HMS Model

<table>
<thead>
<tr>
<th>Subbasin</th>
<th>$C_t$</th>
<th>$L$ (mi)</th>
<th>$L_c$ (mi)</th>
<th>Lag (hr)</th>
<th>Lag (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.20</td>
<td>5.45</td>
<td>3.70</td>
<td>5.42</td>
<td>325</td>
</tr>
<tr>
<td>2</td>
<td>2.12</td>
<td>6.26</td>
<td>5.11</td>
<td>6.00</td>
<td>360</td>
</tr>
<tr>
<td>3</td>
<td>2.04</td>
<td>6.46</td>
<td>5.27</td>
<td>5.88</td>
<td>353</td>
</tr>
<tr>
<td>4</td>
<td>1.96</td>
<td>2.63</td>
<td>1.37</td>
<td>2.88</td>
<td>173</td>
</tr>
<tr>
<td>5</td>
<td>1.88</td>
<td>3.32</td>
<td>3.06</td>
<td>3.77</td>
<td>226</td>
</tr>
<tr>
<td>6</td>
<td>1.8</td>
<td>18.49</td>
<td>8.25</td>
<td>8.13</td>
<td>488</td>
</tr>
</tbody>
</table>

These lag times are used by the model to calculate the time required for excess precipitation in each subbasin to flow overland into the Manafwa River. The model contributes all overland flow from each subbasin into the Manafwa River at a single point, at the outlet of each subbasin.

3.2.4 Defining Reach Routing

Each segment of the Manafwa River is represented by a “reach” in the model. The reach routing process converts a hydrograph at the upstream boundary of the subbasin to a resultant hydrograph at the downstream boundary of the subbasin for each reach, accounting for gains and losses (energy and mass) experienced as the river travels through that particular subbasin. The change in the shape of a hydrograph within a reach as it moves downstream is dependent on river channel geometry and the roughness of the channel surface. These factors determine the degree of energy loss; a wide channel and smooth surface causes little energy loss whereas a constricted channel with a rough surface can cause significant energy loss. Similarly, a steep slope will accelerate flow whereas a gradual slope will decelerate flow. To account for these factors, two routing methods are considered, the Muskingum Method and the Muskingum Cunge Method.
Both methods are well-established and relatively applicable for this study. The parameter requirements for each method, as well as a description of how each parameter is derived, are shown in Table 10.

<table>
<thead>
<tr>
<th>Muskingum Cunge Method</th>
<th>Muskingum Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>• River Length</td>
<td>• Muskingum K</td>
</tr>
<tr>
<td>• Measured from DEM</td>
<td>• Travel time of flood wave through river segment</td>
</tr>
<tr>
<td>• Slope</td>
<td>• Estimate based on field measured velocity</td>
</tr>
<tr>
<td>• Measured from DEM</td>
<td>• Muskingum X</td>
</tr>
<tr>
<td>• Manning's Roughness Coefficient (n)</td>
<td>• Weighting factor based on expected flow</td>
</tr>
<tr>
<td>• Estimated based on site observations</td>
<td>• attenuation</td>
</tr>
<tr>
<td>• Channel Geometry</td>
<td>• Estimated based on knowledge of</td>
</tr>
<tr>
<td>• Estimated based on site observations</td>
<td>• watershed</td>
</tr>
</tbody>
</table>

The major difference between the two methods is that the Muskingum Cunge method depends on physically observed site-specific parameters of the river whereas the Muskingum method depends on parameters derived from empirical relationships associated with rivers in general. Additionally, the assumptions made in the Muskingum method are often violated in natural channels (US Army Corps of Engineers, 2010). The Muskingum Cunge method is therefore chosen for this study, and the associated parameters are derived using physical watershed characteristic data.

The river reach length and slope are both derived from HEC-GeoHMS. Manning’s roughness and channel geometry are based on observations of the river channel during the January 2014 site visit. The Manning’s \(n\) value is estimated from tables in literature which provide a description of a river channel and the associated \(n\) value. Table 11 shows data from Chow (1959) as represented in Bedient (2008):
Table 11 - Manning's Roughness Coefficient $n$

<table>
<thead>
<tr>
<th>Type of Channel and Description</th>
<th>Minimum</th>
<th>Normal</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Natural Streams (top width at flood stage &lt; 100 ft)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A. Streams on plain</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Clean, straight, full stage, no rifts or deep pools</td>
<td>0.025</td>
<td>0.030</td>
<td>0.033</td>
</tr>
<tr>
<td>2. Same as above, but more stones and weeds</td>
<td>0.030</td>
<td>0.035</td>
<td>0.040</td>
</tr>
<tr>
<td>3. Clean, Winding, some pools and shoals</td>
<td>0.033</td>
<td>0.040</td>
<td>0.045</td>
</tr>
<tr>
<td>4. Same as above, but some weeds and stones</td>
<td>0.035</td>
<td>0.045</td>
<td>0.050</td>
</tr>
<tr>
<td>5. Same as above, lower stages, more effective slopes and sections</td>
<td>0.040</td>
<td>0.048</td>
<td>0.055</td>
</tr>
<tr>
<td>6. Same as 4, but more stones</td>
<td>0.045</td>
<td>0.050</td>
<td>0.060</td>
</tr>
<tr>
<td>7. Sluggish reaches, weedy, deep pools</td>
<td>0.050</td>
<td>0.070</td>
<td>0.080</td>
</tr>
<tr>
<td>8. Very weedy reaches, deep pools or floodways</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>with heavy loads of timber and underbrush</td>
<td>0.075</td>
<td>0.100</td>
<td>0.150</td>
</tr>
<tr>
<td>B. Mountain streams, no vegetation in channel, banks</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>usually steep, trees and brush along banks submerged</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>in high stages</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Bottom: gravels, cobbles, and few boulders</td>
<td>0.030</td>
<td>0.040</td>
<td>0.050</td>
</tr>
<tr>
<td>2. Bottom: cobbles with large boulders</td>
<td>0.040</td>
<td>0.050</td>
<td>0.070</td>
</tr>
</tbody>
</table>

To use Table 11, a description of the river must be available. Figure 24 displays photos of the river at different points in the modeled region taken in January 2014.
A higher Manning’s $n$ corresponds to a greater resistance to flow. It is assumed that there is less resistance to flow further upstream because the steeper slopes would prevent pools from forming and vegetation from being entrained. It is also assumed that there is greater resistance to flow as the water level rises. The photos in Figure 24 show that if the water level increases significantly (as in the case of a flood event), brush, vegetation, and even trees would be in the flow path of the river. This water level rise will result in a temporary change (increase) in the value of $n$, which must be considered.

The pictures in Figure 24 show a relatively low water level when compared to that of a flooding event. At the stage shown in these pictures, the Manning’s $n$ is estimated to range from 0.03 upstream to 0.04 downstream (A.1 – A.3 in Table 11). In a flooding event, the water level, and therefore Manning’s $n$, will be higher. During the 2014 trip to Uganda, community members stated that the water level reaches well into the tree line during large floods and even overtops bridges, and that trees and other vegetation are uprooted and carried down the river (Uganda Community Members, 2014). It is estimated that during such an event, the Manning’s $n$ ranges from 0.1 upstream to 0.13 downstream (A.8 in Table 11).
In a storm event, the water level, and consequently Manning’s $n$, will vary based on the flow in the river; specifically, peak flows would be in the range of 0.1 – 0.13 and low flows would be in the range of 0.03 – 0.04. Unfortunately, the Muskingum Cunge method does not have the capacity to adjust the Manning’s $n$ as the flow rate changes. As a result, only one of the Manning’s $n$ ranges can be used. As explained in Section 4.2.2, the goal of the model is to approximate the peak of the hydrograph, and because the peak of the hydrograph is likely to correspond to a high water level and a flooding event, the higher range of Manning’s $n$ is used in the model: 0.1 – 0.13.

The channel geometry in the Muskingum Cunge method is defined by the cross sectional shape of the reach; the options are: circle, eight point, rectangle, trapezoid, or triangle. The Manafwa River is a meandering river that contains curved sections and straight sections, which are most closely approximated by triangular and trapezoidal channel profiles respectively (Lancaster & Bras, 2002). The two river profiles measured during the 2013 site visit support this relationship; a triangular profile was measured at a curved section downstream of Busiu Bridge and a trapezoidal profile was measured at a straight section upstream of Busiu Bridge (Ma, 2013). The channel profile is highly variable over the length of the river such that no single shape will accurately represent an entire river reach. Because much of the river is represented by curves, the profile for the entire river was assumed to be triangular. In HEC-HMS, the triangle is defined by its side slope: units of width per one unit of vertical distance (US Army Corps of Engineers, 2010). An estimate of side slope for the Manafwa was made based on cross section measurements from the 2014 site visits. The side slopes for these cross sections are shown in Table 12, and are generally consistent with the cross sections defined in 2013.

<table>
<thead>
<tr>
<th>Cross Section Location</th>
<th>Side Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>XS-9</td>
<td>18.44</td>
</tr>
<tr>
<td>XS-8</td>
<td>6.69</td>
</tr>
<tr>
<td>XS-31B</td>
<td>5.44</td>
</tr>
<tr>
<td>XS-31S</td>
<td>5.9</td>
</tr>
<tr>
<td>XS-1A</td>
<td>3.67</td>
</tr>
<tr>
<td>XS- Upstream Headworks</td>
<td>6.29</td>
</tr>
</tbody>
</table>

**Table 12 - 2014 Cross Section Side Slopes**

The average side slope is 7.74 units of width per one unit of depth. These cross sections were measured downstream of the HEC-HMS model region (see Cheung 2014 for cross section details), and therefore may not be representative of the upstream segments in the HEC-HMS region. To account for this difference, it is assumed that river lengths further upstream have a smaller side slope; this is attributed to steeper terrain upstream and increasing river width.
downstream. Under this assumption, the side slope is varied in the HEC-HMS model from a value of 7 for the most downstream segment to a value of 5 for the most upstream segment. A summary of the selected Muskingum Cunge parameters is shown in Table 13 below:

<table>
<thead>
<tr>
<th>Table 13 - Muskingum Cunge Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reach</td>
</tr>
<tr>
<td>-------</td>
</tr>
<tr>
<td>Reach 1</td>
</tr>
<tr>
<td>Reach 2</td>
</tr>
<tr>
<td>Reach 3</td>
</tr>
<tr>
<td>Reach 4</td>
</tr>
<tr>
<td>Reach 5</td>
</tr>
</tbody>
</table>

3.2.5 Defining a Simulation Time Step

After the HEC-HMS methods and parameters are selected, the model computes results at a specified simulation time step. The simulation time step is the time increment between succeeding sets of calculations in the model and therefore controls the resolution of results. The time step can range from one day to one minute, and its length must balance the effect on calibration (Section 1.3.2) with the resolution of results needed for the purpose of the study. A longer time step will result in fewer estimated values during calibration but lower resolution, whereas a shorter time step will result in more estimated values but greater resolution. It should also be noted that the use of the SCS Unit Hydrograph method for overland flow calculations requires that the simulation time step is less than 29% of the lag time for each subbasin (US Army Corps of Engineers, 2010).

For the model to be useful in determining the optimum river gauge location, it must be able to produce hydrographs with a sufficiently high time-resolution. The lag time of the basin is on the scale of hours, not days, meaning a flood wave will move through the basin in a matter of hours. In order to see this flood wave move through the basin in simulated hydrographs, it is helpful to produce simulated hydrographs with a resolution that is greater than hourly. Unfortunately, as explained in Section 1.3.2 equation (1), this higher resolution affects the calibration because the model must compare many simulated values to a single observed value. Although this effect on calibration is of concern, it is unavoidable because low resolution hydrographs are not useful for this study. For this reason, a simulation time step of 15 minutes is selected for use in the model.
After defining all necessary input parameters and determining a useful simulation time step, the model can be used to simulate a storm event and analyze the watershed response to precipitation. Chapter 4 presents the simulation results, describes how the model is calibrated to better represent the Manafwa watershed, and applies the calibrated model to study the watershed’s response to a theoretical precipitation event.
4 Model Simulation, Calibration, and Application Results

This chapter details model results using the parameters as defined in Chapter 3. Because many of these parameters are estimates of actual values, it is expected that the simulated results will not exactly match the observed results. To correct for differences between simulated and observed results, the calibration process is used to adjust model parameters, in an attempt to better replicate the observed hydrograph. With these newly defined (i.e. calibrated) parameters, the model is used to study the November 2006 storm, as well as a theoretical storm, in an effort to inform decisions on the FEWS.

4.1 Model Simulation

The simulation is run using the methods and parameters defined in Chapter 3. To determine the accuracy of model results, the simulated hydrograph at Busiu Bridge is compared to the historically observed hydrograph at Busiu Bridge. The two hydrographs are shown below in Figure 25.

As explained in Section 1.3.2, the resolution of the observed hydrograph (1 day) is less than that required of the modeled hydrograph. This makes it difficult to compare the two hydrographs at each 15 minute time step. Nevertheless, the simulated results can be assessed by comparing the general shape of the hydrographs, as well as the magnitude and timing of the peak flow.

In analyzing a flood event, the most important aspect of the hydrograph is the peak flow, because the peak flow corresponds to the maximum downstream flooding. In contrast, peaks that are significantly less than the maximum may correspond to increased water levels, but not necessarily a flood event. The simulated hydrograph in Figure 25 generally represents the shape and peak of the observed hydrograph, although the simulated peak flow is greater than
the observed peak flow rate. This deviation between simulated and observed peak flow could be because:

a) The observed peak flow underrepresents the actual peak flow that day. Recall that observed flows are based on two stage measurements, one in the morning and one in the afternoon. These two stage measurements are averaged and recorded to represent the stage for the entire day. The flow rate for that day is calculated based on the rating curve, which translates stage into flow rate. In reality, at some point during that day, the stage could have been higher than the average measured value, which may have resulted in a flow rate that matches the simulated peak flow.

b) Estimated watershed parameters allowed more simulated runoff into the river than was actually experienced.

For the purpose of this modeling exercise, it must be assumed that the measured hydrograph is an accurate representation of the peak flow that occurred that day. If the simulated hydrograph can be adjusted to match the peak of the observed hydrograph, the model can better facilitate placement of the river gauge. To do so, model parameters are adjusted through the calibration process.

4.2 Model Calibration

4.2.1 Calibration Process

Model calibration is the process of adjusting physical watershed characteristic input parameters to create a simulated hydrograph that matches an observed hydrograph as closely as possible. The matching of the hydrographs provides assurance to the modeler that the model is in fact an accurate representation of the watershed. Using initial parameter estimates, the model relies on an objective function to quantitatively measure the goodness-of-fit between the two hydrographs; the smaller the absolute value of the objective function, the more identical the hydrographs. The process is an iterative one, in which many simulations are run, each using a different set of parameters, in an attempt to find the specific set of parameters that produces the least discrepancy between hydrographs (US Army Corps of Engineers, 2010).

HEC-HMS offers seven different objective functions to measure the goodness-of-fit. The objective functions used in this study are (1) the Percent Error in Peak and (2) the Peak-Weighted Root Mean Square (RMS) Error method. These functions are selected because they give greater weight to matching the peak of the hydrograph. The Percent Error in Peak method only measures the fit between the magnitude of the simulated and observed hydrographs, it does not account for errors in volume or peak timing. The Peak-Weighted RMS Error method also focuses solely on the peak of the hydrograph; however, it takes into account the volume and timing of the peak as well. More information on the objective functions available can be
found in the HEC-HMS Technical Reference Manual (US Army Corps of Engineers, 2010). To minimize the two objective functions, two methods are available: 1) the Univariate Gradient Method and 2) the Nelder and Mead method. The major difference between the two methods is that the Univariate method adjusts one parameter at a time whereas the Nelder method attempts to adjust multiple parameters simultaneously (US Army Corps of Engineers, 2010).

Not all parameters are adjusted through the calibration process. As discussed in Section 1.3.2, it is difficult to estimate river reach parameters without known hydrographs at both ends of the reach. Additionally, care must be taken in adjusting individual parameters that have an effect on each other. For this reason, the only parameters selected are those that can be adjusted across the entire watershed by a single scale factor; these parameters are the curve number and the initial abstraction.

With two objective functions and two methods for minimizing the objective function, four different calibration procedures are used. For all calibration procedures, the objective function tolerance is set at 0.02 and the maximum number of iterations is set at 50.

### 4.2.2 Calibration Results

Results from the four calibration procedures are shown below in Figure 26.

![Figure 26 - Calibration Results](image)

Figure 26 shows that the shape of the calibrated hydrograph is primarily dependent on the objective function (Percent Error in Peak versus Peak-Weighted RMS Error). When the objective function is held constant, only slight changes are seen by changing the minimization method. The Percent Error in Peak function more closely approximates the magnitude of the peak, whereas the Peak-Weighted RMS Error function more closely approximates the overall
volume of the hydrograph. Because the goal of the calibration process in this study is to approximate the peak of the hydrograph, the Percent Error in Peak objective function is chosen. Table 14 below shows results from the two calibration procedures that use the Percent Error in Peak objective function.

<table>
<thead>
<tr>
<th>Table 14 - Parameter Scaling Factors for Percent Error in Peak Method</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Curve Number Scale Factor</strong></td>
</tr>
<tr>
<td>Univariate Minimization of Percent Error in Peak</td>
</tr>
<tr>
<td>Nelder Minimization of Percent Error in Peak</td>
</tr>
</tbody>
</table>

As discussed in Section 4.2.1, the Univariate method only adjusts the curve number in an attempt to replicate the hydrograph, whereas the Nelder method simultaneously adjusts the curve number and initial abstraction. Because an adjustment to the initial abstraction is expected, and also results in a lower objective function value, the Nelder minimization of the Percent Error in Peak objective function was chosen to calibrate the model. The low objective function value, in combination with the resemblance of the peak of the calibrated hydrograph to the simulated hydrograph, indicate that the calibrated model is capable of representing the relationship between precipitation and observed peak river flow for the November 2006 storm. The scaling factors determined through calibration are applied to the original model parameters as defined in Section 3.2. The original parameters and calibrated parameters are shown below Table 15.

<table>
<thead>
<tr>
<th>Table 15 - Original and Calibrated Model Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Subbasin</strong></td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
<tr>
<td>4</td>
</tr>
<tr>
<td>5</td>
</tr>
<tr>
<td>6</td>
</tr>
</tbody>
</table>

The calibration hydrographs at each possible river gauge location for the November 2006 storm can be seen in Figure 27.
As expected, the magnitude of flow increases at locations further downstream as more water reaches the river. The time of the peak flow for the November 2006 storm event at each location is shown below in Figure 28.

<table>
<thead>
<tr>
<th>River Location</th>
<th>Time of Peak</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kato</td>
<td>16:15</td>
</tr>
<tr>
<td>Buwagogo</td>
<td>14:45</td>
</tr>
<tr>
<td>Pasa</td>
<td>13:30</td>
</tr>
<tr>
<td>Shikoye</td>
<td>14:45</td>
</tr>
<tr>
<td>Busiu Bridge</td>
<td>15:15</td>
</tr>
</tbody>
</table>

The November 2006 storm did not produce a peak flow that traveled downstream through the entire watershed (a flood wave). Instead, the peak flow occurred first at Pasa, in the middle of the watershed, and moved downstream from that point. This is because precipitation was relatively evenly distributed across the watershed, both in magnitude and in time (Figure 14); at
any given time, each subbasin experienced about the same precipitation intensity. This evenly distributed rainfall caused a rise in river height along the entire length of the river because each subbasin contributed flow, and therefore did not result in a single upstream peak moving downstream.

4.3 Modeling an Upstream Precipitation Event

An upstream river gauge is more useful for flood warning when precipitation events occur upstream of the gauge and create a flood wave, as opposed to a precipitation event concentrated downstream, or a slow-rising flood caused by a widespread and/or gradual precipitation event. The speed of the flood wave (primarily a function of river slope and channel resistance) must be slow enough such that there is sufficient time between upstream observation and downstream effect within which to issue a warning. To determine if there is potential to use an upstream river gauge to issue flood warnings in the Manafwa watershed, a precipitation event that is more likely to produce a flood wave is modeled.

This theoretical precipitation event applies the same volume of precipitation to the watershed as the November 2006 storm, but concentrates all of the precipitation upstream in subbasin 6. This event is representative of flooding events described by community members in the Butaleja District, who explained that, when the river floods, it is often clear and sunny downstream in Butaleja, but raining upstream on Mt. Elgon (Uganda Community Members, 2014). Such an event is more likely to create a flood wave because the flow originates upstream, resulting in a constant volume of water being transported downstream. This upstream precipitation event was input to the calibrated model and the results are shown below in Figure 29.

![Hydrographs at Potential River Gauge Locations - Upstream Precipitation Event](image)

**Figure 29 - Hydrographs for Upstream Precipitation Event: 11/13 – 12/12**

For this storm event, the flow rate at each location is approximately the same because all flow originates upstream. The magnitude of flow over the entire hydrograph is larger than that of
the November 2006 storm even though the same total volume of precipitation is used; this is because the precipitation is concentrated over a smaller area, resulting in less initial abstraction, and therefore more runoff. Figure 30 below shows the same hydrograph at a magnified scale between the dates of 11/18 and 11/28; the insert in the top left corner of this figure shows the portion of the hydrograph in Figure 29 that is magnified.

Figure 30 - Hydrographs for Upstream Precipitation Event: 11/18 – 11/28

Figure 31 shows the hydrograph for a 2.5 day period, starting at midday on 11/21 and ending on 11/24; the insert in the top right corner of this figure shows the portion of the hydrograph in Figure 30 that is magnified.

Figure 31 - Hydrographs for Upstream Precipitation Event: 11/21 – 11/24

At these scales it is possible to see that the peak moves from Kato to the outlet over time, indicating that a flood wave moves down the river. In general, during the rising leg of each peak on the hydrograph, the flow rate at any given time is higher upstream than it is downstream. This is expected because the upstream precipitation has not yet reached the
downstream locations. After each peak, the flow at any given time is greater downstream than upstream because the bulk of the flow has already moved through each specific location. The first of these phenomena, higher flow rates upstream during the rising leg of a peak, create an opportunity for flood warning. This is because a period of time exists during which the flow rate is increasing upstream (indicative of a flood) but there is no apparent sign of the same increase in flow downstream. A flood warning consists of communicating to downstream communities (1) the occurrence and magnitude of an elevated upstream flow and (2) the rate of flow increase as represented by the slope of the rising leg of the hydrograph.

To illustrate this, the overall peak of the hydrograph in Figure 31 considered. At 23:00 on 11/21, the flow rate at the outlet is relatively low, and more importantly, exhibits a near-zero slope. An observer at this location would not have a reason to believe a flood is approaching because they do not observe elevated water levels or increasing water levels. In contrast, the hydrograph at Kato, the most upstream location, shows a significantly higher flow rate during the same hour, and, more importantly, exhibits a steep slope. The steep slope indicates that the flow is increasing and will continue to rise at Kato. Because Kato is upstream along the same river, it is likely that these conditions (elevated and increasing flow) will soon occur at the downstream location. In this situation, the river conditions upstream at Kato can be used to predict future river conditions downstream. If the flood wave moves slow enough that there is sufficient time between when the conditions experienced at Kato are experienced at the outlet, a flood warning could be issued from Kato for downstream communities.

To determine the amount of warning lead time available, the time at which the peak flow reaches each location is used to calculate the flood wave travel time and velocity for each reach, as shown in Figure 32.
The calculated travel times suggest that, for this theoretical storm, a river gauge placed upstream in the Manafwa watershed could be used to provide a flood warning downstream. In this case, the maximum warning lead time available is eight hours, calculated as the sum of the travel times for each reach between the outlet and Kato.

The velocities are calculated by using the time it takes for the peak flow to travel down each reach. These velocities therefore represent the velocity of the river during high flows and may not be representative of the velocity of the river during low flows. As discussed in Section 3.2.4, the channel roughness experienced by the flow, represented by Manning’s $n$, is greater during flood flows than during low flows. Because there is not a second river gauge to use to calculate the observed flow velocity between two points on the river, the modeled velocities of reaches 1 through 5 are compared to velocities that were measured at Busiu Bridge between 2005 and 2010. These measured velocities are shown in Table 16.
The measured velocities range between 0.24 m/s and 1.07 m/s, with an average of 0.62 m/s. Busiu Bridge is between Reach 1 and Reach 2, which have modeled velocities of 1.0 m/s and 1.2 m/s, respectively (Figure 32). The modeled velocities are on the higher end of the range of velocities measured at Busiu Bridge (Table 16), but because the modeled velocities are representative of peak flows, it is expected that they should best approximate the upper end of the measured velocities at Busiu Bridge, which they do.

It should be noted that the computed travel times represent the travel time of the peak of the hydrograph. As discussed above, the rising leg of the peak hydrograph would instead be used to provide a flood warning. To do so, a specific flow rate and change in hydrograph slope upstream that equates to a subsequent flood downstream need to be specified. The implications and limitations of the modeled upstream precipitation event are discussed in Section 5.4.
5 Discussion

This study uses available watershed characteristic data to create a hydrologic model of the Manafwa watershed. To ensure that the model accurately represents the hydrologic response of the watershed, the model is calibrated using a storm that occurred in November 2006. By adjusting the simulated hydrograph to match the observed hydrograph, model parameters are refined to more accurately represent the watershed. This calibrated model is then used to analyze a theoretical storm event to determine the applicability of an upstream river gauge to provide flood warnings. Finally, a conceptual explanation of how an upstream gauge can provide a flood warning for such a storm event is provided. The limitations and implications of this analysis are discussed below.

5.1 Applicability of Calibrated Model Parameters

The parameters derived through the calibration process are specific to the November 2006 storm. These parameters may or may not be representative of the watershed’s response to a different storm event. To determine the applicability of the parameters to a variety of storm events, additional historical data are needed. With this data, the model could be run with the same calibrated parameters, and the resultant hydrographs could be compared to measured hydrographs to determine the degree of agreement. If the simulated and observed hydrographs for a variety of storm events do not consistently agree, model parameters may need to be adjusted. Unfortunately, this analysis is not possible with currently available data. Nevertheless, because the parameters used in this analysis represent physical characteristics of the watershed, they are likely suitable under most conditions and creating a model with a slightly different set of parameters is not anticipated to provide significantly different results.

5.2 Utility of a Theoretical Storm

As described in Section 4.3, the nature of the November 2006 storm does not lend itself well to flood prediction using an upstream river gauge. To determine if an upstream river gauge is useful to provide a flood warning for a different type of precipitation event, a theoretical storm is created. The intent of this theoretical event is to create precipitation concentrated in the most upstream region of the basin, which is more representative of the frequent storms described by residents. To simulate this event using the model, precipitation is restricted to the most upstream subbasin of the watershed (subbasin 6). This exaggerates the upstream nature of the storm because, although the model applies precipitation over the entire upstream subbasin, the model collects and routes the runoff into the river at a single location. In reality, the entire volume of water would not enter the river at one location. Rather, the volume of water in the river would increase at consecutive points along the river’s length, moving downstream within the subbasin. The model is incapable of representing this situation unless
precipitation is placed in other subbasins, or subbasin 6 is divided into smaller subbasins. Because all flow enters the river at a single location, the magnitude of the flood wave is constant as it moves downstream. Although the magnitude of the flood wave might not be constant in reality, the model is useful and facilitates the analysis of an upstream precipitation event.

5.3 Suitability of an Upstream River Gauge for Flood Warning

The ability of an upstream river gauge to provide a flood warning downstream is dependent on the characteristics of the watershed and the nature of the precipitation event. In general, a watershed that is a good candidate for the use of an upstream river gauge is one that prevents a flood wave from moving too quickly downstream. River characteristics that lead to this condition include a relatively long length between upstream and downstream locations, a relatively gradual slope, and relatively high channel roughness. In general, a precipitation event that creates a flood which can be predicted with the use of an upstream river gauge will be characterized by precipitation that is concentrated upstream. If the majority of precipitation is concentrated upstream of the gauge, the peak flow will pass through the gauge before it reaches a downstream location. When this type of event is paired with the characteristic watershed described above, the peak flow created upstream may move downstream slow enough that a warning can be issued from the upstream location. Table 17 below provides a simplified explanation of the conditions necessary for an upstream river gauge to be useful in issuing flood warnings.

<table>
<thead>
<tr>
<th>Location of Majority Rainfall</th>
<th>Flow Resistance (High Roughness and Gradual Slope)</th>
<th>Flood Warning Available?</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upstream</td>
<td>High</td>
<td>Yes</td>
</tr>
<tr>
<td>Downstream</td>
<td>Low</td>
<td>No</td>
</tr>
<tr>
<td>Upstream</td>
<td>Low</td>
<td>No</td>
</tr>
<tr>
<td>Downstream</td>
<td>High</td>
<td>No</td>
</tr>
</tbody>
</table>

An upstream gauge is therefore not necessarily useful in providing a flood warning for every watershed or precipitation event.

In the Manafwa watershed, the potential for a river gauge to provide a flood warning depends on the type of precipitation event. As discussed in Section 4.3, the November 2006 storm did not produce a flood wave upstream that could have been predicted by an upstream gauge. For that flood, the existing gauge downstream at Busiu Bridge would have been as valuable as an
upstream gauge. In contrast, the theoretical upstream precipitation event produced a flood wave that traveled slow enough to lend itself to a flood warning from an upstream gauge.

5.4 Application of the HEC-HMS Model to the Flood Early Warning System

This study determined that, following certain types of storm events (i.e. precipitation concentrated upstream), an upstream river gauge can be used to provide a flood warning in the Butaleja District of the Manafwa watershed. To determine the optimum location for the river gauge, modeled hydrographs can be used to relate the river condition upstream to subsequent flooding downstream, as demonstrated in the conceptual example in Section 4.3. The goal is to identify a specific point on a hydrograph upstream that, when reached, will result in subsequent overbank flooding to commence downstream, at a later point in time. To create a robust warning, this upstream condition must consistently result in a flood downstream. This requires additional analysis that is not covered in this study, but is initiated in Cheung, 2014. A simplified summary of Cheung’s 2014 work to determine the optimum location for a river gauge follows.

A hydraulic model uses the geometric profile of the river, in the downstream Butaleja district, to relate river flow to water level rise. Using this relationship, the specific flow rate that causes overbank flow can be specified. Overbank flow is the flow that causes the water level in the river to be higher than the river banks, creating a flood. The time it takes for this specified flow to travel from an upstream location down to Butaleja can be calculated from the sum of the lag times of the river reaches. The total travel time between an upstream location and Butaleja is the warning lead time that would be provided by a gauge at that location. To determine an optimum location for the river gauge, the warning lead time must be balanced with other factors such as construction feasibility and ability to operate and maintain the gauge. Through this process, an optimum location for a river gauge can be determined (Cheung 2014).
6 Recommendations and Conclusion

This study documents how the HEC-HMS model is developed and used to determine whether an upstream river gauge can provide a flood warning in the Manafwa watershed. Model parameters are defined and the model is calibrated to accurately predict the watershed’s response to a storm event. In the calibration, emphasis is placed on the ability to estimate the peak flow rate. After the model is calibrated, a theoretical storm is used to show that a river gauge placed upstream of the existing gauge at Busiu Bridge could provide enhanced flood warning to the Red Cross under certain circumstances. There are several improvements to the modeling effort that may be useful in subsequent studies.

6.1 Model Improvements

The accuracy of the model could be improved by expanding the hydrologic processes that it simulates and by obtaining more accurate model parameters. Because the intended use of the model is to provide a general estimate of the peak hydrograph and not to estimate the exact flow rate that occurs at every time step, the model was simplified by excluding hydrologic processes that are unlikely to have a significant effect on model results. In addition, during the calibration process, the Curve Number is used as a surrogate parameter to account for any changes in flow that these multiple processes may cause. Nevertheless, inclusion of these processes may improve model results. The processes that could be included are listed below with the expected improvement that they could provide:

- River Base Flow
  - Would result in a more realistic hydrograph during low flows
  - Would facilitate an analysis of the watershed’s hydrologic budget
  - Will not have a large effect on the peak hydrograph

- Storage and Evaporation
  - Would be useful if the model is used for continuous modeling
  - Would facilitate an analysis of the watershed’s hydrologic budget
  - Often not a significant factor in event modeling (US Army Corps of Engineers, 2010)

The following data sets could be used to obtain a better estimate of physical watershed parameters:

- An updated land use map would facilitate a more accurate estimation of Initial Abstraction and Curve Number
- Additional upstream river profiles would facilitate a more accurate estimation of channel geometry for reach routing
The hydrologic processes and data sets described above could improve model accuracy and should be included in future versions of the model.

6.2 Increased Watershed Monitoring

Model calibration is a fundamental step in developing a reliable model because it provides assurance that the model produces accurate results. To make the model more robust under a variety of conditions, model parameters should be further refined through the use of additional historical storms. Similar to the November 2006 storm, if corresponding precipitation and flow data exist, the precipitation data can be input to the model and the simulated results compared to the observed results. Through this process, the applicability of model parameters to a variety of conditions can be evaluated. It may be determined that no single set of parameters is applicable to all storms. If this is the case, multiple sets of parameters that are applicable to different types of storms can be developed for use in the model.

In addition to having more sets of historical data, data of higher accuracy and resolution would also improve model results. As described in Section 2.2.1, the TRMM satellite data are an estimate of precipitation and do not represent the true amount of precipitation that occurred. Higher accuracy and resolution precipitation data would improve the calibration procedure. Additionally, as explained in Section 4.1, higher resolution historical hydrographs would provide a more comprehensive comparison to the high-resolution simulated hydrographs during model calibration.

Lastly, river monitoring at additional locations in the watershed would facilitate the calibration of reach routing parameters. To calibrate river reach parameters, there must be at least two locations of observed data along the reach, which is not currently available.

To provide this additional watershed data, a greater number of rain and river gauges would need to be installed throughout the watershed. Additionally, the gauges would need to be capable of providing high resolution, accurate, and continuously reliable results.

6.3 Future Application

As more data becomes available and model improvements are made, a greater variety of storms can be simulated. The hydrographs produced from these simulations can be used to refine the river conditions under which a flood warning should be issued, and inform decisions on FEWS. In addition to facilitating the development of the FEWS, the model has potential applications in land use planning. As shown in Bingwa’s 2013 work, the effect of land use change on flooding can be forecasted.
Finally, this study can be used as a foundation for a similar approach in other watersheds. The study provides a guide for how to develop a hydrologic model and presents an example of how the model can be used to inform decisions on flood early warning systems.
Works Cited


