ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

HYDROLOGIC AND HYDRAULIC MODELLING USING HEC-HMS AND HEC-RAS MODELS (CASE STUDY ON BIG AKAKI RIVER BASIN)

A thesis in HYDRAULIC ENGINEERING

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A Thesis

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The undersigned have examined the thesis entitled "**Hydrologic and Hydraulic Modeling Using HEC-HMS and HEC-RAS Models (Case Study on Big Akaki River Basin)"** presented by **Zerihun Wondimu Bekele**, a candidate for the degree of **Master of Science** and hereby certify that it is worthy of acceptance.

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ABSTRACT

Flooding is the most common environmental hazard worldwide. Almost every year, Addis Ababa is affected by floods although their nature and impacts are different from space to space and hydrological nature. This study aims to develop hydrologic and hydraulic models for flood-affected areas of Addis Ababa using modeling tools from Hydrologic Engineering Centre's (i.e. HEC-HMS and HEC-RAS). The big Akaki River basin is considered in the study to analyze the effects of rainfall on surface runoff and peak discharges in the downstream flood plain areas. Additionally, the effect of discharge released from 'Lega Dadi and Dire dams on the peak flow is analyzed and ultimately produce flood inundation levels to assess the flood map in the study areas.

The hydrological and hydraulic modeling is done by dividing the watershed into different sub-catchments. To compute infiltration loss by the SCS CN method which convert excess rainfall to runoff model using Clark unit hydrograph, and channel flow routing accomplished by Muskingum routing method of HEC-HMS model. To examine the accuracy of the simulation, model calibration and validation is performed. Furthermore, the hydraulic model using HEC-RAS is conducted to estimate the flood map of big Akaki river with related flood runoffs within the respective return periods of 25, 50, 100, and 1000 years were analyzed.

The output result from HEC-HMS used in the flood mapping of Big Akaki river for the return period of 25, 50, 100, and 1000 year's events were determined. Without considering peak discharges from both 'Lega Dadi and Dire dams, the values were obtained to be 277.459m^3/s, 362.207 m^3/s, 462.898 m^3/s and 946.851 m^3/s respectively. Taking the discharge released from both dams into account, the corresponding peak discharges are estimated to be 354.05 m^3/s, 429.15 m^3/s, 516.97 m^3/s, and 963.10m^3/s. By considering both dam flow release, the maximum channel flood depths found in the middle big Akaki river is 8.8m, 9.3m 10.1m, 15.5m for the mentioned return periods and the corresponding areas of flooding are 0.0578km2, 0.0641$km^{2,}$ 0.0698km^2, 0.100km^2 respectively. Apart from some extreme events, the magnitude flood-prone area for the case of flood released from both dam simultaneously and separately at different time intervals on the downstream flood plain is identified. The flooding impact inundates large downstream flood plain region during high flood level. Furthermore, numerous people living in Akaki River watershed along the river as well as large investment area, agricultural land, and downstream plains are predominately prone to high flood.

Keywords: DEM, TIN, Flood Modeling; HEC-HMS, HEC-GeoHMS; HEC-RAS, HEC-GeoRAS, RAS-Mapper, Flood Mapping

LIST OF ABBREVIATIONS

AAWSA	Addis Ababa Water and Sewerage Authority
AAiT	Addis Ababa Institute of Technology
BFI	Base Flow Index
CN	Curve Number
DEM	Digital Elevation Model
DTM	Digital Terrain Model
E.G.US	Energy Grade Line at Upstream
EMA	Ethiopia Metrological Agency
EMWEB	Ethiopia Ministry of Water and Energy Bureau
ET	Evapotranspiration
ESA	The land use delineated from European Space Agency
ESRI	Environmental System Research Institute
GIS	Geographical Information System
HEC-GeoHMS	Hydrologic Engineering Centre's Geographic Hydrologic Modeling System
HEC-GeoRAS	Hydrologic Engineering Centre's Geographic River Analysis System
HEC-HMS	Hydrologic Engineering Centre's Hydrologic Modeling System
HEC-RAS	Hydrologic Engineering Centre's River Analysis System
HSG	Hydrologic Soil Group
SCS	Soil Conservation Service
TIN	Triangulated Irregular Network
UCL	University Catholique de Louvain
USACE	United States Army Corps of Engineers
U.S	United State

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1 INTRODUCTION

1.1 Background

Flooding is the most common environmental hazard worldwide. The human interventions together with the effects of global climate change, currently and in the future, flooding is the problem that has been more serious. It occurs more frequently and unpredictably across the globe exacerbating damages in urban and pre-urban areas. According to the World Disaster Report (Muturi, 2010) floods affect a large number of people in different parts of the world. According to the assessment of this report, about 99 million people were affected per year between 2007 and 2008 in the world.

Floods are already having very large impacts on cities and smaller urban centers in many African nations. For instance, the floods in Algiers in 2001 affected a significant number of residents in which around 900 people killed, and 45,000 affected. Ethiopia also facing the same global challenges of climate change effects and challenges associated with hydrologic extremes such as drought and floods. The country experiences two types of floods: flash floods and river floods. Flash floods are the ones formed from excess rains falling on upstream watersheds and gush downstream with massive concentration, speed, and force. Often, they are sudden and appear unnoticed (ELIAS, 2015). For example, the flood event most frequently happed in the country like lower and upper Awash River plain, Omo river basin are prone to flash flood. Similarly, Little Akaki, Kabana, Big Akaki and Dire Dawa river plain face to sever river flood. For example, in 2007 Dire Dawa town face with river flood that cause damage to properties and loss of human life.

Another significant characteristic of flood is its frequency of occurrence. Flood occurrences are highly associated with land use land cover and other climatic factors that commonly linked with hydrologic processes in a river basin. As such, land-use and land-cover changes may have direct impacts on the hydrological cycle that can cause floods, droughts, changes in river and groundwater regimes.

In view of land use change, Addis Ababa (the capital city of Ethiopia) is highly associated with the rapid conversion of land from rural to urban uses more than anywhere else in the country. For the last one hundred seventeen years, it has been noticed that there is an intensive conversion of rural land to urban due to development of buildings, transportation networks, and facilities (airports and highways), recreation areas, and other manmade structures is increased within the town (UNEP, UNESCO, UN-HABITAT, 2003). The demographic feature of Akaki river plain is changed due to the city fast population growth, uncontrolled urbanization and industrialization. The change in demographic feature, has led to increased exposure of communities, agricultural land, and infrastructure to flood hazard. Hence, if the flood is not managed properly, it causes huge damage to animal life, infrastructure, recreation areas, and population utilities. Therefore, sustainable and consistent flood prediction, proper design of control structure and demarcation for flooded area remains a major challenge of the city. This requires the knowledge of the hydrologic and hydraulic model of the river and the characteristics of the entire flow contributing catchment.

Notwithstanding the natural processes in a catchment, the hydrologic and hydraulic modeling need consistent estimates of stream flow from a catchment area. This is required to help policymakers to inform decisions on water planning and management. There are a range of methods available to estimate stream flow from catchments, using observed data wherever possible, or using empirical and statistical techniques, more commonly known as rainfall-runoff models (Khadka & Bhaukajee, et al., 2018).

Rainfall-runoff modeling used to evaluate runoff from precipitation in the catchment and hydraulic modeling to evaluate the magnitude of flood as well as flood inundation map along riverbank. This study will be considering the natural change and geomorphologic characteristics of Big Akaki river catchment as well as the river characteristics across Addis Ababa. To estimate flood magnitude and flood level for inundation map of Akaki, Hechu and Gemeda area, flood simulation and hydraulic model of river with aid of geographic information system (GIS), HEC-HMS, HEC-RAS software will be used in combination with their extension Arc-hydro, HEC-GeoHMS, HEC-GeoRAS.

1.2 Statement of the problem

The hydrologic behavior of river systems draining Addis Ababa city is highly affected by the human-induced impacts. As a result, it is highly challenging to deal with the urban hydrologic characteristics of the main river systems. The present study found to be unprecedented in that the hydrologic behavior of individual streams where analysis will be conducted from both hydraulic and hydrologic characteristics. The city is prone to flood damage during the rainy season and the morphometric characteristics of the main river stream flow network changed every year with the flow in the system. For this reason, the river characteristics across the town require a detail assessment.

The excess flows in water bodies can happen due to several factors, but seasonal heavy rainfall is the main cause of flooding in most parts of the Upper Awash River Basin. More specifically, the problem of river flooding due to excess rainfall in a short time and the following high river discharge is a great concern in the Basin (Gebre SL, 2015). Such issues are more pronounced in densely populated areas of Addis Ababa.

On the other hand, the determination of flood magnitude is more challenging; because the source of flood is attributed to surface runoff, groundwater flow, unexpected dam flow releasing from the upstream and the conveyance structure of drainage into the stream. Often, low lying areas of Addis Ababa city receive high runoff from upper catchments and local rainfall on the floodplain resulting in flood inundation problems.

Akaki River plains situated upstream of Upper Awash Watershed which is susceptible to flood during heavy rainfalls. A recent observation showed that the flood has displaced people, damage agricultural farms around the Gelan condominium, Kebena, Filwuha, Akaki Woreda, Hechu and Gemeda kebeles. The worst flood that occurred in 2017/2018 summer has brought intensive damage to agriculture, infrastructure, goods, properties, and animal life. Therefore, the flood problem of this area is the most critical issue needs detail assessment.

1.3 Objectives of the Study

1.3.1 General

The general objective of the study is to evaluate the hydrologic and hydraulic behavior of Akaki River using available models for flood mapping.

1.3.2 Specific Objectives

The specific objectives of the study are:

- To assess the watershed characteristics of rivers crossing Addis Ababa city;
- To develop the flood map for the affected areas in the big Akaki river basin;
- To simulate flow for the big Akaki river basin using HEC-HMS and HEC-RAS;
- To establish GIS database in combination with hydrologic and hydraulic modeling for the determination of flood risk areas in big Akaki river plain;
- To analyses the flow releases from ‘Lega Dadi and Dire dam to the downstream flood plain areas.

1.4 Significance of the study

The significance of this study will be to update and expand information on stream flow, river characteristics crossing Addis Ababa town and provide updated flood map to water resource managing and planning sector. These help them to have good information about integrated land use planning and manage for flood plain area. In addition to this, the outcome of this study will give the updated information for the downstream dwellers on the effect of peak discharge released from ‘Lega Dadi, Dire dam and sewerage of drainage of the town. The well- informed population for upstream catchment characteristics will be aware for the unexpected flooding in the future.

1.5 Thesis Outline

This thesis report consists of Seven Chapters. The contents of each chapter are organized as follows: In Chapter 1 the background information, problem statement, general and specific objectives, and Significance of the study are discussed. Chapter 2 tries to include the views of different literatures on a key points of the subject matter is presented especially the theories behind the software and models applied in this research. The major factors considered on hydrological and hydraulic modeling are also presented. In Chapter 3, the study area and the modeling input datasets defined in detail. In Chapter 4 the methodologies and procedures used for the development of hydrologic and hydraulic modeling are described in detail. In Chapter 5 both the hydrological & hydraulic modeling, as well as flood inundation map results, are presented and discussed. In Chapter 6 the study results conclusion and implications associated with results as well as the contribution of this research, suggestion related future research issues and recommendations of the study presented in detail. The final chapter, which is including the reference and annex is presented.

2 LITERATURE REVIEW

2.1 Hydrological Processing and Components

The hydrologic cycle is the central focus of hydrology estimating the total amount of water on the earth and in the various processes of the hydrologic cycle. However, quantitative data are scarce, particularly over the oceans, and so the amounts of water in the various components of the country's hydrologic cycle are still not known precisely (Chow, 2010). The study of any hydrological relation will be commenced from the knowledge of the hydrological cycle in detail as it is illustrated in Figure 2.1.

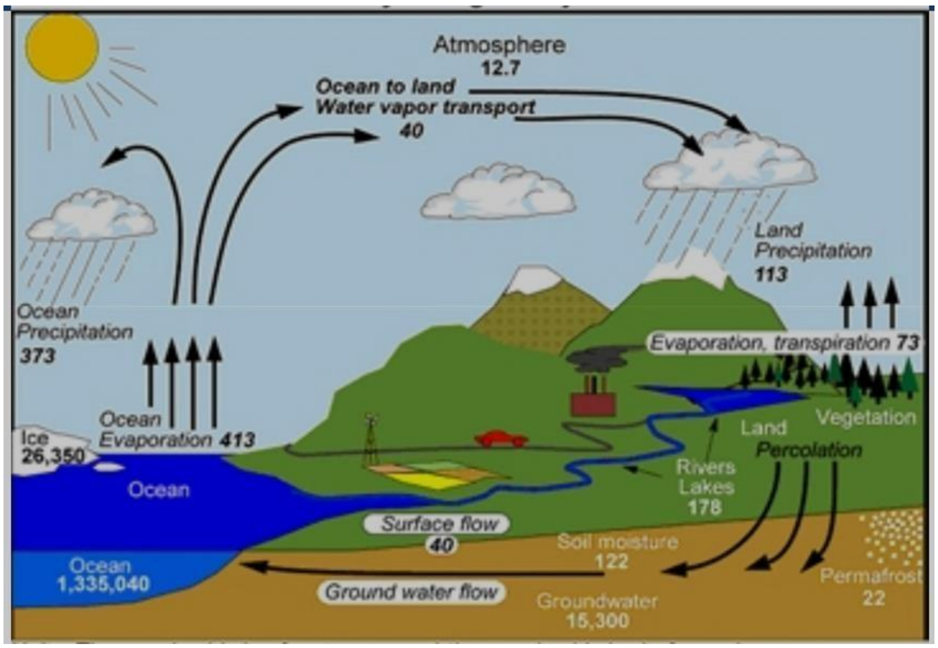


*Figure 2-1 Total Hydrologic Cycle (*www.national geographic.org/.../hydrologic-cycle)

The total precipitation created in the atmosphere to the ground surface. Because, some of them will be intercepted with vegetation leaf, infiltrated into the soil, return to the atmosphere through evaporation and transpiration. The remaining precipitation that is not reached on the land is fall on the body of the ocean. The excess runoff after soil saturation can be change into surface runoff and the infiltrated water through the soil from the groundwater flow. The combination of both groundwater flow and surface water flow is changed into rivers, lakes, and streams. The indicated in the figure above is in thousand cubic Km for storage and thousand cubic km per year for exchanges.

2.2 Hydrologic and Hydraulic Flood Modeling

Flood model is one of the means to understand the behavior of flood in a particular area. Model simulation can provide flood depth and extent. With the increased availability of the computing resources and the development of new models, flood hazard maps can be prepared at a high resolution with better accuracy for preparedness planning. Flood vulnerability map can be also prepared by integrating infrastructure and population data with the flood hazard maps. Flood models are the representation of hydraulic and hydrologic processes in the river channel and flood plain. Accurate representation of the actual processes is of paramount significance in predicting flood extent and depth, especially explaining the transient characteristics of river water flow in the model domain. Determining the variation of flow characteristics in the spatial and temporal resolution enables to design of flood evacuation plan quite efficiently (Zelalem, et al. 2011).

Physically based flood inundation model approaches solve the St. Venant equation using one dimensional (1-D) or two dimensional (2-D) numerical schemes. 1D model assumes change of stage, velocity and discharge along the longitudinal direction. These models are competent with hydrometric observations. In this model cross sections are taken at certain locations along stream wise direction and the geometry is assumed to be constant between the cross sections. Hydraulic energy, pressure forces, flow depth and velocity are assumed constant in the transverse direction and locations of the section may predetermine flow direction and condition.

2.3 Model Selection

Selecting appropriate model is an essential part in any research study. The section of model depends on the required output of the model, availability of input data, prices, availability of the model and model structure. There are different flood modeling tools which have their own distinct model structure and solution procedures. In this Section, the theoretical background of model used for this study is described. The models selected for this research are discussed as follows:

2.3.1 Hydrologic Model (HEC-HMS)

2.3.1.1 Rainfall-Runoff Model

Hydrologic Engineering Center and hydrologic Modeling system embodies all of the physical processes that are involved in the conversion of precipitation to stream flow, as well as physical characteristics of the watershed and atmosphere that influence runoff generation. The use of computer models to simulate the hydrologic system is of major significance in the performance of many flood-runoff analyses. A fundamental problem in simulating hydrologic systems is to employ the appropriate level of detail to represent those components of the system that have a significant influence on the phenomena being modeled. An associated problem is to acquire and interpret information on watershed characteristics (USACE, 1994).This software is used to model to simulate the precipitation-runoff process of branched watershed system. It is designed to be used in a wide range of geographic areas for solving the widest possible range of problems.

The simple schematic representation of the runoff process computer-generated in HEC-HMS is shown in figure 2.2. The different options of mathematical models for hydrological components that conceptually represent watershed behavior are incorporated in this software. The software uses separate model to represent each component of the runoff process like model to compute runoff volume, model of direct runoff or baseflow, channel flow as well as alternative models to account for the cumulative losses. To account the loss from all precipitation to the surface the program include different option of loss accounting method. Some of the loss method includes in the program like Empirical methods, Soil Conservation Service (SCS) curve number (CN), are based on correlating parameters estimated from rainfall-runoff records to factors affecting loss rates such as soil type and surface cover.

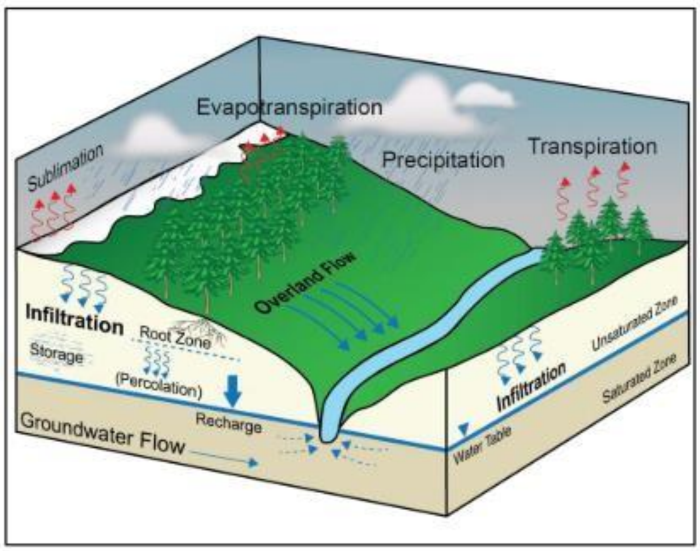


Figure 2-2 HEC_HMS representation of Watershed runoff (Sun, 2015)

The rainfall infiltration or loss process is complex and affected by many factors. Soil properties are important, but the chemistry of the water, biologic activity, soil heterogeneity, and surface cover modify the soil's infiltration capacity. Surface cover and topography also are involved in losses by intercepting, storing, and detaining rainfall. Finally, the dynamics of the rainfall-runoff process are important in determining the volume of rainfall available for direct runoff. Even though excess may be generated at some point in an agricultural or urban area, some of this excess may infiltrate as overland flow traveling to a channel. In forested areas, flow that has infiltrated is a major contributor to direct runoff (USACE, 1994).

2.3.1.2 HEC-HMS Software Components

HEC-HMS model components are used to simulate the hydrologic response in a watershed. HMS model component includes basin models, metrological models, control specifications, and input data. A simulation calculates the precipitation-runoff response in the basin model using a given input from Metrologic model. The control specifications define the time period and the time step of simulation to run. Input data components such as time-series data, paired data, and gridded data are often required as a parameter or boundary condition in basin and Metrologic models. All the input data is prepared using Arc Hydro tools that consists of five components: Network, Drainage, Channels, Hydrography and Time Series:

- ***Network*** component contains a water resources network of streams, rivers and centerlines of water bodies. Its main purpose is to describe the connectivity of water movement through the landscape.
- **Drainage** component defines drainage areas delineated through analysis of land surface topography.
- **Channel** component describes the three-dimensional shape of river and stream channels.
- **Hydrography** component contains base map information on point, line, and area water resource features.
- **Time Series** component describes time-varying water property of the features.

Terrain Preprocessing contains the following functions on Arc hydro tools

- **Data Management**: set the tags for the subjects used in the Terrain Preprocessing menu.
- **DEM Manipulation:** group of functions allowing manipulating the DEM grid (burning, fencing, filling).
- **Flow Direction**: create flow direction grid from a DEM grid.
- **Adjust Flow Direction with Sinks:** create a flow direction grid that forces the cells within the sinks to flow toward the sink point, as well as a sink link grid and sink watershed grid.

- **Adjust Flow Direction in Streams:** Create the stream sloped flow direction grid by replacing the cells in the streams in the input flow direction grids with the values from the stream flow direction grid generated by the function create drainage line structures. stream link grids.
- **Adjust Flow Direction in Lakes:** modify the input flow direction grid within the lakes by forcing the water toward the closest stream within each lake.
- **Flow Accumulation:** create a flow accumulation grid from a flow direction grid.
- **Stream Definition:** create a new grid (stream grid) with cells from a flow accumulation grid that exceeds the user-defined threshold.
- **Stream Segmentation:** create a stream link grid from the stream grid (each link between two stream junction gets a unique identifier).
- **Combine Stream Link and Sink Link:** create a link grid representing both dendritic (Stream link) and deranged (sink link) terrain.

- **Catchment Grid Delineation:** create a catchment grid for a link grid. It identifies areas draining into each link.
- **Catchment Polygon Processing:** create catchment polygons out of the catchment grid.
- **Drainage Line Processing:** create streamlines out of the stream link grid.
- **Adjoint Catchment Processing:** create an ad joint catchment polygon for each catchment in the catchment polygon feature class. Adjoins catchment is total upstream area (if any) draining into a single catchment.
- **Drainage Point Processing:** create a drainage point at the most downstream point in the catchment (center of a grid cell with the largest value in the flow accumulation grid for that catchment).
- **Longest Flow Path for Catchments:** create the longest flow paths for catchments that will be used as preprocessed inputs to speed up the generation of the longest flow path for sub-watersheds.
- **Longest Flow Path for ad joint Catchments:** create the longest flow paths for ad joint catchments that will be used as preprocessed inputs to speed up the generation of the longest flow path for watersheds.
- **Accumulate Shapes:** create aggregated polygon features by merging each input polygon feature with all its upstream polygon features.
- **Slope:** create a slope grid in percent.

2.3.2 Hydrologic Model Description

For the simplified hydrologic cycle has been divided into different components, which are modeled separately. The models included in the software can thus be categorized as follows:

Loss Method: HEC_HMS model that computes the runoff volume and it accounts for the losses that occur during a rainfall event as a result of infiltration and evapotranspiration.

Transform Method, Baseflow Method: Baseflow models are used to simulate slow subsurface flow drainage of water into the channel.

Routing Method: If the analyzed watershed is divided into sub-watersheds, the flow at the outlet of a certain upstream watershed has to be routed through the river channel in the downstream watershed. The models used to simulate this routing process are therefore called routing methods. They account for the geometry and roughness of the relevant river channel (Heimhuber, 2013).

2.3.2.1 SCS Curve Number (CN)

Surface loss is a function of land use and differs greatly between forested, agricultural, and urban areas. According to Viessman et al. (1977), the interception of rainfall by surface cover is greatest for a forest and decreases for agricultural and urban land uses. Some of loss mode

Incorporated in the program include initial and constant rate model, deficit and constant rate model, SCS curve number (CN) loss model, green and Ampt model.

The main factors that determine CN are the hydrologic soil group (HSG), land use land cover, treatment, hydrologic condition, antecedent runoff condition, and impervious areas. On the other hand, infiltration rates of soils vary widely and are affected by subsurface permeability as well as surface intake rates. Soils are classified into four HSG's (A, B, C, and D according to their minimum infiltration rate (ELIAS, 2015).

2.3.2.2 Transform

HEC_HMS models of direct runoff which consider the water that is not infiltrated and evaporated is transformed into surface runoff are called transform method, and the transform method convert the effective rainfall over a watershed into a hydrograph at the outlet of the watershed. It accounts the surface roughness and geometry of the watershed. This type of transformation is a unit hydrograph that represents direct runoff at the outlet of a basin resulting from one unit for example 1 mm of precipitation excess over the basin. The excess runoff occurs at a constant intensity over a specified duration. Depending on the unit hydrograph to generate runoff based on the principle of super positioning as shown by Figure 2.3

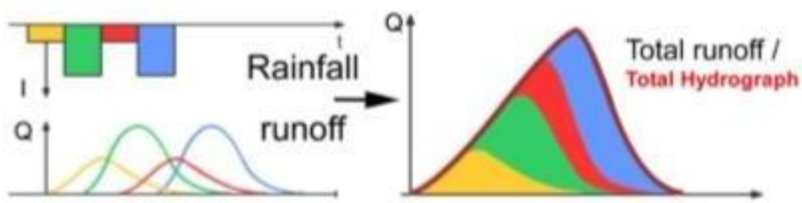


Figure 2-3 Principle of super positioning in the unit hydrograph method (Heimhuber, 2013)

If there is no gauging station in the basin, the unit hydrograph transformation method is used transfer from gaged station to ungagged, in this case, means that there is a stream gauge at the basin outlet for which measurements during historical storms are available, and data from precipitation gauges are available with which hydrographs of basin-average precipitation can be developed for the storms.

2.3.2.3 Routing (Muskingum)

The heart of the HEC-HMS routing model is the fundamental equation of open channel flow such as the momentum equation and the continuity equation. The momentum equation accounts for forces that act on a body of water in an open channel. In simple terms, it equates the sum of gravitational force, pressure force, and friction force to the product of fluid mass and acceleration.

The Muskingum method is a commonly used hydrologic routing method for handling a variable discharge-storage relationship. This method models the storage volume of flooding in a river channel by a combination of wedge and prism storages figure 2.4. During the advance of a flood wave, inflow exceeds outflow, producing a wedge of storage. During the recession, outflow exceeds inflow resulting in a negative wedge. In addition, there is a prism of storage which is formed by a volume of constant cross-section along the length of the prismatic channel. Assuming that the cross-sectional area of the flood flow is directly proportional to the discharge at the section, the volume of prism storage is equal to KQ where K is a proportionality coefficient, and the volume of wedge storage is equal to KX (l - Q), where X is a weighting factor having the range 0 < x < 0.5(Chow, 2010).

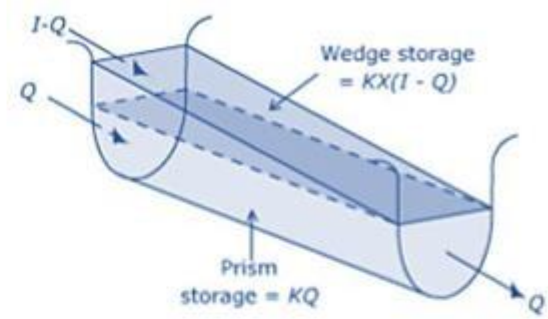


Figure 2-4 Prism and wage storage in channel reach (Chow, 2010)

The total storage is, therefore, the sum of two components,

$$S = KQ + KX(I - Q)$$ *Equation 2-1*

Which can be rearranged to give the storage function for the Muskingum method is shown in equation 2.2.

$$S = K[XI + (1 - X)Q]$$ *Equation 2-2*

Equation 2.2 represents a linear model for routing flow in streams. The value of X depends on the shape of the modeled wedge storage the value of X ranges from 0 for reservoir-type storage to 0.5 for a full wedge. When X = 0, there is no wedge and hence no backwater; this is the case for a level-pool reservoir. The value of storage at time j and j+1 can be written, respectively by

$$S_j = K[XI_j + (1 - X)Q_j]$$ *Equation 2-3*

Generally, the Muskingum model the storage in channel reach can be written by

$$Q_{j+1} = C_1 I_{j+1} + C_2 I_j + C_3 Q_j$$ *Equation 2-4*

Where the coefficient C_1, C_2, and C_3 will be

$$\left.\begin{aligned} C_1 &= \frac{\Delta t - 2KX}{2K(1-X) + \Delta t} \\ C_2 &= \frac{\Delta t + 2KX}{2K(1-X) + \Delta t} \\ C_3 &= \frac{2K(1-X) - \Delta t}{2K(1-X) + \Delta t} \end{aligned}\right\}$$

Equation 2-5

2.3.3 Application and Limitation of the HEC-HMS

HEC-HMS has been applied successfully for more than 30 years and is accepted for many official purposes such as the determination of floodways for the U.S. Federal Emergency Management Agency (Heimhuber et al., 2013). The HEC-HMS model used for estimation of peak flows, hydrograph, stream routing, reservoir and detention routing, watershed studies, watershed impact analysis, and flood insurance study. One of the major advantages of this application is a selection of different hydrologic models that is suitable for different environments and under different conditions. In addition, HEC-HMS includes options for the calibration and optimization of the selected models with measured precipitation and runoff data using different parameters incorporated with the model. With HEC-HMS being a widely used, complete and flexible software solution for the modeling of the rainfall-runoff process, the applicability rather depends on the suitability of the hydrologic models for a given situation rather than on the software itself (Heimhuber, 2013)

However, every simulation has limitations due to the choice of the designer and development of the software. HEC-HMS is used to perform rainfall-runoff modeling but the limitation arises from the aspect of simplifying formulation of model and simplified flow representation. The simplifying model formulation means all the mathematical equation in the program is deterministic this assume that the boundary condition, initial conditions, and the parameters of the model are constant. Additionally, simplifying flow representation is only for the watershed basin model of dendritic stream network that means the stream flow network imaged as a branch of trees. The trunk of the tree represent the mainstream and the branches of the tree represent tributaries of the river. The core idea is stream doesn’t separate into two streams. In the basin model, each hydrologic element have to only one downstream connection so it is not possible to split the downstream outflow into two different stream flows. In general, the branching and loop system stream network doesn’t simulated in the model and it requires the separate hydraulic model for this type of stream network. Based on a combination of the Curve Number model, the Clark Unit Hydrograph model and the Muskingum flow routing model. The major limitation of the Curve Number method is that during the modeling of a storm event of a large duration, the infiltration rate eventually approaches zero (Heimhuber, et al. 2013)). Furthermore, the intensity and duration of the rainfall is neglected in this method so that a 25 mm rainfall in one day, results in the same cumulative loss as a 25 mm rainfall in one hour. Nevertheless, it is a simple, predictable and stable method that is widely accepted for use in the U.S. and abroad (HEC, 2000). Furthermore, the computational procedure used in HEC-HMS introduces a time component to the model by computing amounts of excess precipitation for successive intervals in a storm. Since the CN is the only input parameter for this model, a correct estimation is crucial for the accuracy of the modeling results

The limitation of this type of model for the case of floodplain when the runoff outflow out on side of the bank of the river, flood routing does not account for overbank flow and channel flow separately rather the model combines them into one to rout flood of the channel.

Therefore, to analyses the transition from the main channel to overbank flows, the model must account for varying the conveyance between the main channels and the overbank areas. For one-dimensional flow models, this is normally accomplished by calculating the hydraulic property of the main channel and the overbank areas for separately, then combining them to formulate a composite set of hydraulic relationships. This cannot be accomplished with the kinematic-wave and Muskingum model because the Muskingum model parameters are constant (USACE, 2000).

2.3.4 Hydraulic Model

Under this section, a detailed description of the methodology that is applied to obtain the hydraulic modeling results is included. The river analysis using HEC-RAS developed by the Hydrologic Engineering Center U.S. Army Corps of Engineers River Analysis model tool which is worked with the interface of ArcGIS version 10.4 tools. It is used to perform one-dimensional steady flow, unsteady flow, and calculation of water surface profile, sediment analysis, water quality, and water temperature analysis. The component of one-dimensional river analysis includes steady flow water surface profile computation, unsteady flow simulation, movable boundary sediment transport computation, and water quality computation in which they use the common geometric data representation and common geometric and hydraulic computation routing.

2.3.4.1 HEC-RAS

Reliable assessment and resolution of river hydraulics issues depend on the engineer’s ability to understand and describe, in both written and mathematical forms, the physical processes that govern a river system(USACE, 1993).

2.3.4.2 River Analysis Component

2.3.4.2.1 Steady Flow Water Surface Profile

This component of the modeling system is intended for calculating water surface profiles for steady gradually varied flow. The system can handle the full network of channels, a dendritic system, or a single river reach. The steady flow component is capable of modeling subcritical, supercritical, and mixed flow regime water surface profiles. Steady flow analysis assumes that, although the flow is steady, it can vary in space. (G. Brunner, 2010).As it is noted by (G. W. Brunner, 2016) the 1D steady-state is considered with the limitation. This limitation in 1D steady flow program includes flow is steady, gradually varied, one dimensional and river channels have small slopes, say less than 1:10.

2.3.4.2.2 Boundary Conditions in HEC-RAS

To perform the hydraulic calculation the boundary condition is required. If supercritical flow analysis is going to be performed, then only the downstream boundary is required. If subcritical flow analysis is going to be performed, then only the upstream boundary is required. If the modeler is going to perform a mixed flow regime calculation, both upstream and downstream boundary conditions are required(G. Brunner, 2010).

2.4 Watershed Characteristics

A watershed is the upslope area that contributes flow generally water to a common outlet as concentrated drainage. It can be part of a larger watershed and can also contain smaller watersheds, called sub-basins. The boundaries between watersheds are termed drainage divides. The outlet, or pour point, is the point on the surface at which water flows out of an area. It is the lowest point along the boundary of a watershed.

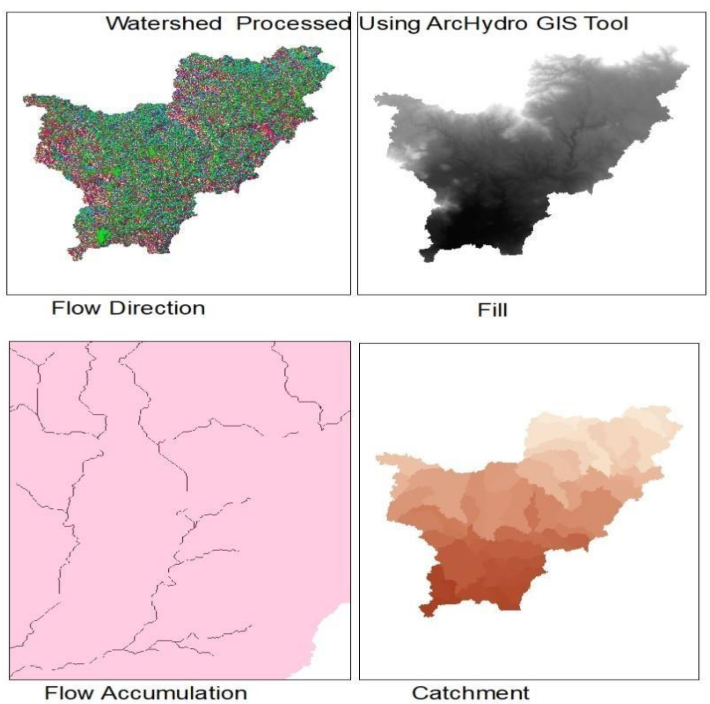


Figure 2-5 Watershed flow directions, flow accumulation, fill DEM and Catchment

The watershed delineation needs to provide the locations to determine the catchment area for outlet determine characteristics of the contributing area and to use a flow accumulation threshold. When the threshold is used to define a watershed, the pour points for the watershed will be the junctions of a stream network derived from flow accumulation. Therefore, a flow accumulation raster must be specified as well as the minimum number of cells that constitute a stream (the threshold value). The watershed boundary, drainage divide and catchments are shown in Figure 2.5. Watershed characteristics include the following;

Basin slope: The rates of change (delta) of the surface in the horizontal (dz/dx) and vertical (dz/dy) directions from the center cell determine the slope.

River slope: It is the rates of change surface in horizontal and vertical directions from one end of drainage segment line to the other ends of the segment.

Basin centroid: The function drainage Centroid generates the centroid of drainage areas as centers of gravity. However, if a center of gravity is not located within its drainage area polygon, the function uses instead as centroid the projection of the center of gravity on the drainage area boundary.

Longest flow path: The function Longest Flow Path for Sub watersheds the longest flow path for selected sub watersheds.

River Length: The function Calculate Length Downstream for edges calculates the length from a network edge to the sink that the edge flows to, and populates the field Length down in that feature class with the calculated value.

2.5 Previous Studies in upper Awash Basin

Even though the Akaki area is characterized as high flooding prone area, there are still few studies applied. The current flood problem and lack of detail studies in the areas show the importance of the in-depth study. The study conducted by Eliyas (2016) is centered on flood mapping and mainly focused on different flood protection structures around the river. Another study is conducted by Misganaw (2016) is considered on urbanization and its effect on surface runoff to develop effect of high rate of growth of the Addis Ababa City on land used land cover that generate runoff for the study area (Akaki river plain). Some researcher study on upper Awash River and their finding is summarized in table 2.2.

Table 2-1 Summary of different Thesis in the upper Awash Basin

No.	Thesis Title	Author/Year	Study Area	Model Used	Key Finding
1	Prediction and Flood Modeling For Kello Town	Admasu Bereded,2017	Kello Town,(16.69km2)	GIS ,HEC-HMS and HEC-RAS	Flooding for Kello town is a result of flash flood originating from rainfall on the Eastern highlands of catchments
2	Urbanization and Its Effect on Surface Runoff	Misganaw Neg,2016	Great Akaki River, Addis Ababa, Ethiopia, (884.63km2)	GIS V10.1,HEC-HMS V. 4.0 and HEC-RAS V.4.1	Effect of high rate of growth of the Addis Ababa City on land used land cover that generate runoff
3	Effect of Sediment Aggradation on the Hydraulic Performance of River Cross Drainage Structures	Natnael Hailu,2016	Mekanisa Bridge on Mekanisa to Gofa Highway Road,(202 km2)	GIS ,HEC-HMS and HEC-RAS	Reduction in hydraulic performance of the Bridge and increase bed level at the Bridge location
4	FLOOD MAPPING	Elias Zeleke, 2015	Bantyeketu River in Addis Ababa	GIS V.10 ,HEC-HMS and HEC-RAS	Flood Mapping and its protection
5	Application of GIS in Cross Drainage Structure	Melat Kaleab,2015	Major drainage structure on Goro-Akaki road,(725.98 km2)	HEC-HMS version 3.2	Application of GIS which easier and better to upgrade the usual trend of hydrological analysis
6	Assessment of Failure on Drainage Structures along the Ethiopian National Railway line of Sebeta-Mieso	Yerosan Abera, 2015	Akaki River Crossing Drainage Structure,(894km2)	GIS ,HEC-HMS and HEC-RAS	Akaki Railway Bridge is hydraulically efficient over its design period

3 Material and Methodology

3.1 General Overview of the Study Area

Awash River basin is the most important river basin in Ethiopia, and covers a total land area of 110,000 km^2. According to (Misganaw, et al., 2016) sited International Livestock Research Institute, this basin serves as home to 10.5 million inhabitants. Akaki river catchment is located in upper Awash River basin bounded with the Southern flank of Intoto ridge (3199m a.s.l.) and expanded in all directions crossing the Addis Ababa city.

There are two out locations are selected on Akaki River near Turunesh Hospital and Abasamuel reservoir. By considering outlet of Aba Samuel reservoir, the watershed extended within the geographical boundary of Easting $38^0$40’00’’E- $39^0$0’0’’E and Northing $8^0$40’00”N -$9^0$14’00”N. Similarly, the study are outlet at Turunesh Hospital cover an area of 1546.22 km^2 and extends up to the geographical location between Easting $38^0$46’00’’E- $39^0$6’0’’E and Northing $8^0$40’00”N -$9^0$14’00”N.

Big Akaki and little Akaki rivers are main two rivers crossing the study area. The main Tributaries on the left side of big Akaki River includes Dangora, Kilinto, Bora, Doro, Fanta, Abeyi and Kersa Doso river. Similarly, on the right side tributaries of the big Akaki River include Tafo, Michire, Bashale, Luke, Kersa and Hanku River. Kotobe and Kurtume are the tributaries of the Kebena River. Moreover, tributaries of Little Akaki River includes: Shagole and Jamo are left tributaries whereas; Jijiga, Malka Rafu, and Harbu River are the right side tributaries.

Additionally, there are four dams Dire, ‘Lega Dadi, Gerfersa, and Abasamuel found in the watershed of the study area. Dire, ‘Lega Dadi, and Gafersa reservoirs are surface water sources for water supply of Addis Ababa city while Abasamuel reservoir is the source of hydropower (the oldest in the nation).

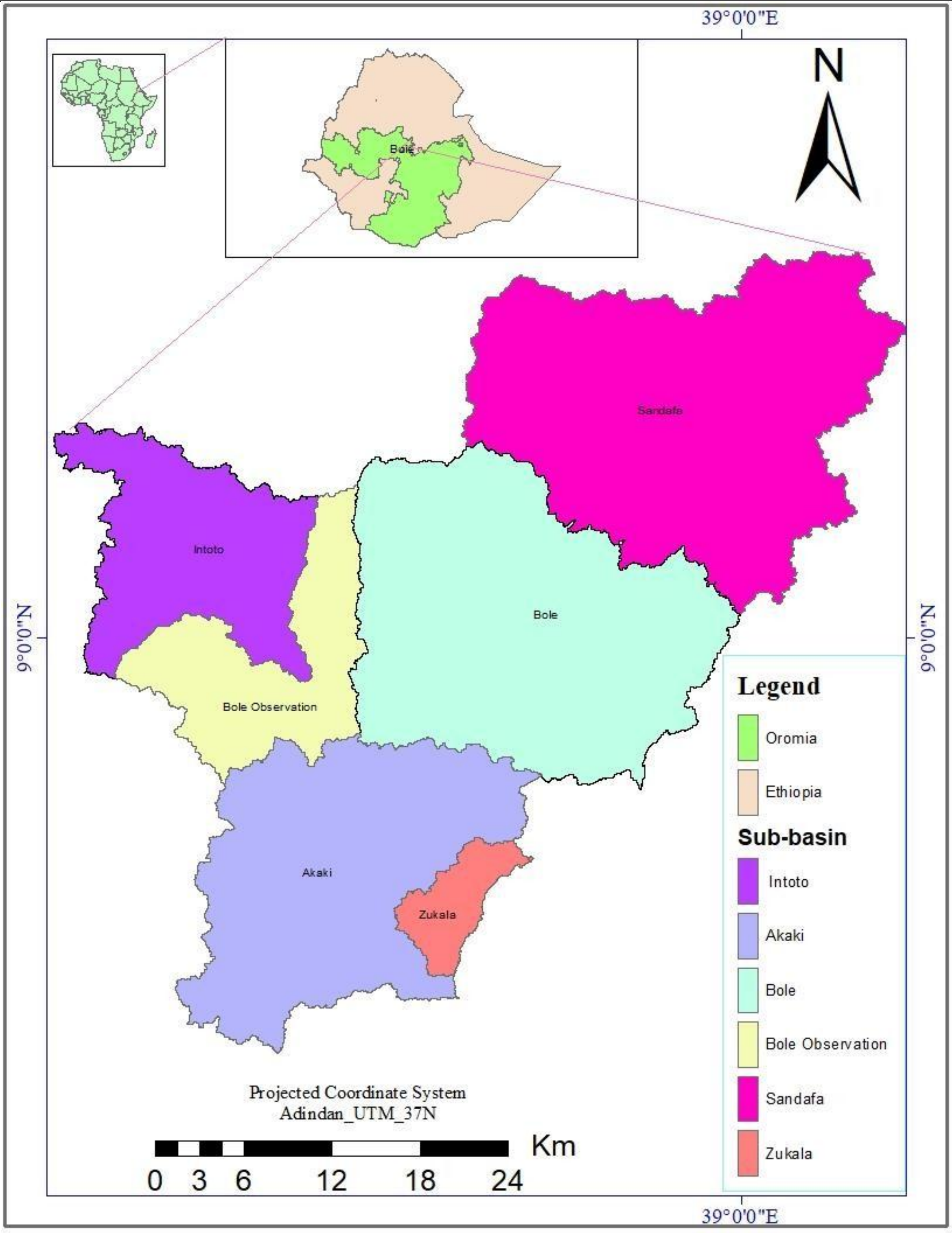


Figure 3-1 The study area for the including the Aba Samuel watershed

3.2 Material and Analysis

3.2.1 Meteorological Data

The metrological data for temperature, rainfall, humidity, wind speed and radiation of sunshine are fully recorded for the catchment of the study are. This study uses data for 32 years and the mean total annual rainfall, mean total annual maximum and minimum temperature and mean total annual evapotranspiration is 100.22mm, 23.65^0C, 10.95^0C and 4.12^0C respectively.

Process of the input data like rainfall and flow is the essence for the output result and it needs to make sure for consistency of data, homogeneity, sufficient, correct, and no missing data of the recorded input data. Error in the data process leads to overestimate or underestimate the simulated date that leads to a large amount in the residual value of between observed and recorded data. Due to this reason, the data correction, checking for consistency and filling of missing data with different techniques. For this study of research, the metrological data is collected from Ethiopia Metrological Agency (EMA) and flow data from the Ethiopia Ministry of Water and Energy Bureau (EMWEB).

The metrological rainfall data for cumulative of two years interval before and after adjusted of five stations in the study area is shown in figure 3.2 all rain data is adjusted by taking the reference rain gage station of A.A Bole observation. This station is selected due to a small percentage of missing data illustrated in table 3.1.

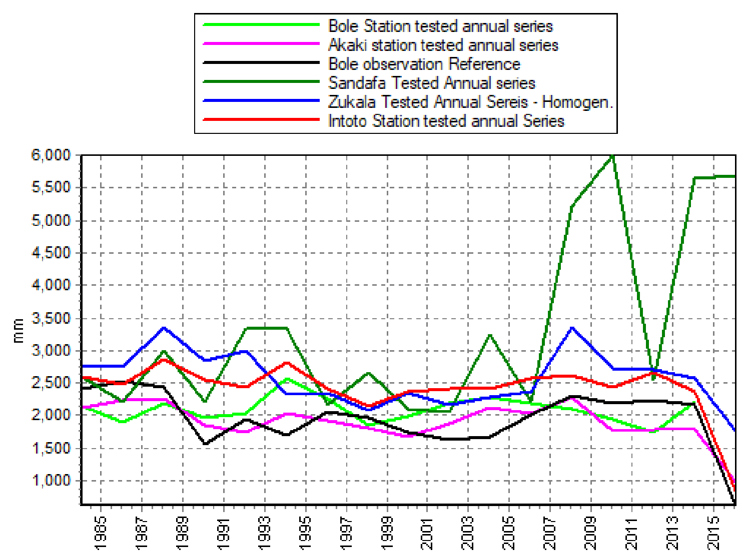


Figure 3-2 Two Years' time step cumulative annual series rainfall of all station before data adjustment for homogeneity

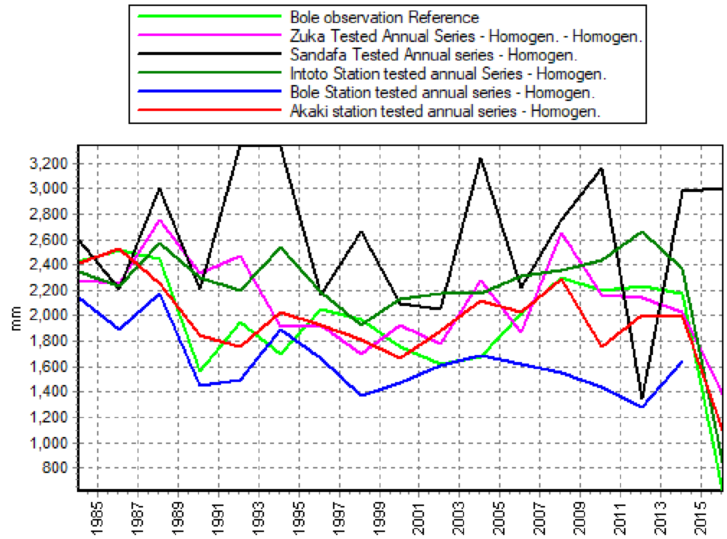


Figure 3-3Two Years' time step cumulative annual series rainfall of all station after data adjustment for homogeneity

The figure 3.2 above indicates the rainfall distribution within the catchment is uniformly distributed and Sandafa rain gage station data is much larger but it is adjusted by A.A Bole observation rain gage data. In this figure, the annual series mean the annual cumulative rainfall and monthly series means the monthly cumulative rainfall of the catchment.

3.2.1.1 Location of Meteorological Data Station

The data collection of station is the essence for the study of hydrology of the specified catchment and the rainfall depth is measured at the gauging station.

The USA National service provides guidelines on the density of rain gages network. This suggests that the minimum number of rain gages N, for a local flood warning network is; $N=A^{0.33}$ in which A is the area of watershed in square miles (USACE, 2000). The total area of Big Akaki river catchment is 1546.22km^2 and according to the equation above the required number of gauging stations is estimated to 49. Although the gaging station is available in the study catchment, some of gaging station has large missed data. For this reason, six gaging stations are selected for this investigation. The geographical location of the metrological data within the catchment is illustrated in Figure.3.3.

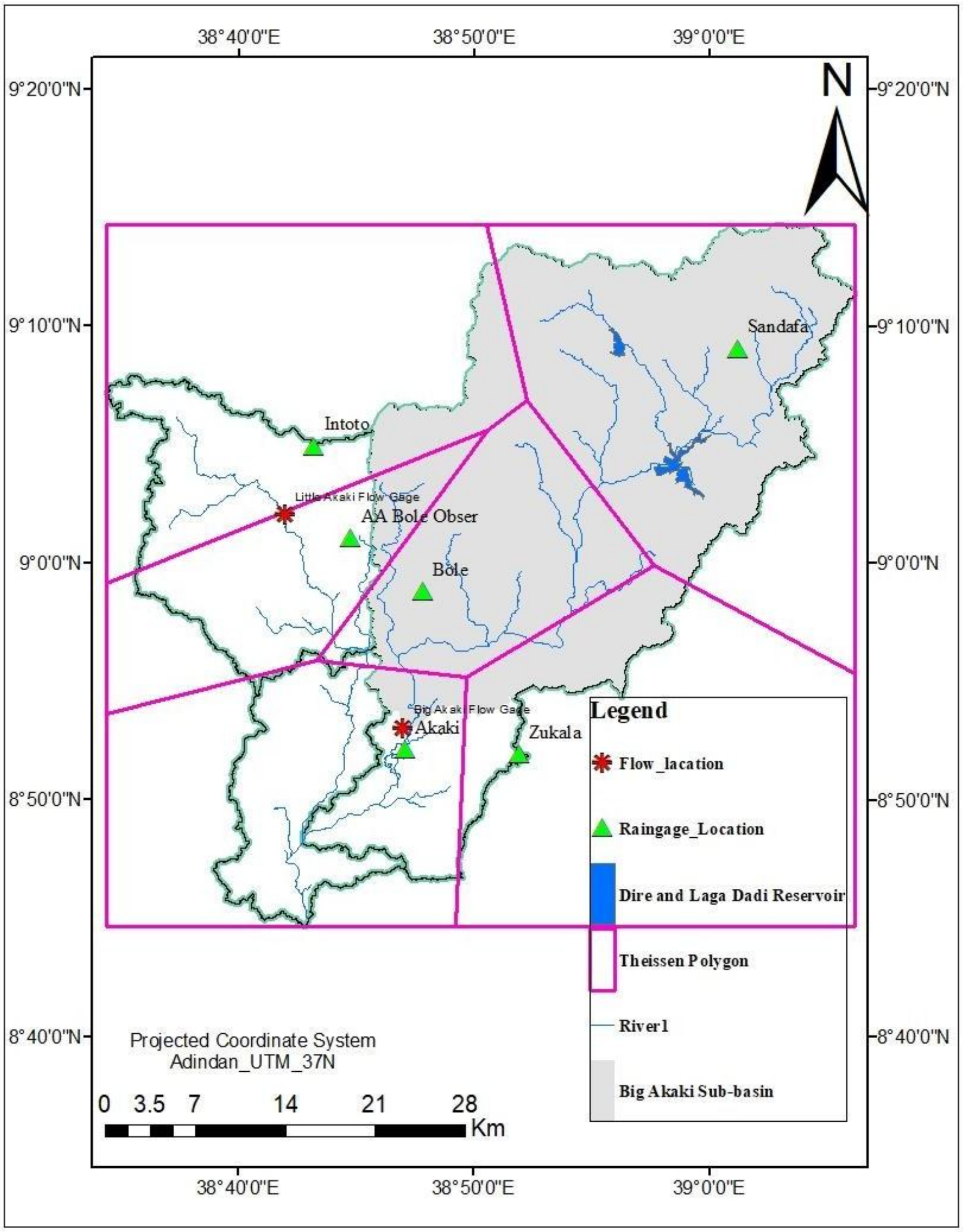


Figure 3-4 Metrological and Flow Gage Location of Big Akaki Watershed

Metrological gage location is sub-divided with the aid of GIS software by Theissen polygon method and the coverage of each station is selected by the percent cover of rain gage. Therefore, Sandafa, A.A Bole observation, Bole, Intoto, Zukala and Akaki rain gage were selected for big Akaki Watershed and similarly big Akaki flow gage station is used in this study.

3.2.2 Filling Missing Precipitation

There are different techniques to estimate the missed precipitation values. Rainfall depth is being recorded continuously; unfortunately, the data may not be homogeneous for the change of gage location due to growth of trees and built up of tall building in the vicinity of the gage site, change of instrument and fault record of the instrument. To minimize the error, filling of missed data using station average, normal ratio method, isohyet map and quadrant method is used. The normal ratio method is used for this research due to the missing percent is less than 10% for all gaging station.

The method is used when the normal annual precipitation of the index stations differ by more than 10% of the missing stations (ELIAS et al., 2015). This is the case for the stations near the study area. The general formula for computing missing precipitation by this method is:

$$P = \frac{1}{n}\sum_{i=1}^{n} \frac{P_{AK}}{P_{Ai}} Pi$$

Where,

P= depth of rainfall required for missing station

n=number of station

P_{XA}= mean annual rainfall depth of required station

P_{Ai}= mean annual rainfall depth of station i.

P_i=rainfall depth of station i

The Percent of missing for all gaging station is shown in the figure 3.5 and table 3.1

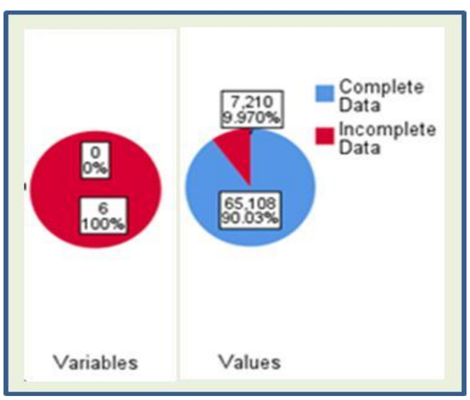


Figure 3-5 Overall Summary of all six station missing Values

The variables in figure 3.4 represent six rainfall gage station and values indicate that total recorded values within 32 years recording period (1985-2017).

Table 3-1 All variable summary

Station Missing Summary					
Station	Missed Number	Percent (%)	Valid Number	Mean	Std. Deviation
Sandafa	2373	19.7	9680	3.197	7.28
Zukala	2236	18.6	9817	3.364	9.40
Intoto	1746	14.5	10307	3.389	6.88
Bole	376	3.1	11677	3.866	6.41
Akaki	363	3.0	11690	2.670	6.53
AA Observatory	116	1.0	11937	3.352	7.30

The recorded values for different rainfall gage station indicated in table 3.1 represents for 32 years recorded time.

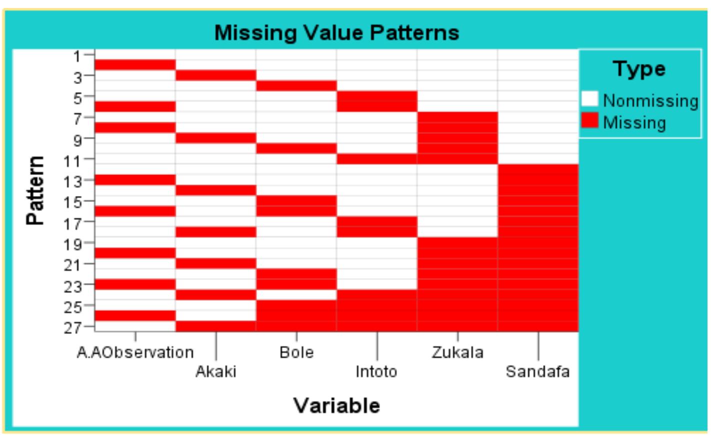


Figure 3-6 The missing value patterns of all station

The data record of our country is very poor in this catchment as it is shown in the Fig 3.5 and 3.6. The pattern in figure 3.6 displays days missed data within all selected years of 1985 to 2017 for the recorded values of rain gage station enable to compare days of non-missing with missing days and therefore, it is essential to fill the missing data.

After all, data processed and missed data is filled using normal ratio method for Addis Ababa Bole observation, Bole station, Akaki, Intoto, Sandafa and Zukala station is illustrated into the following figure.

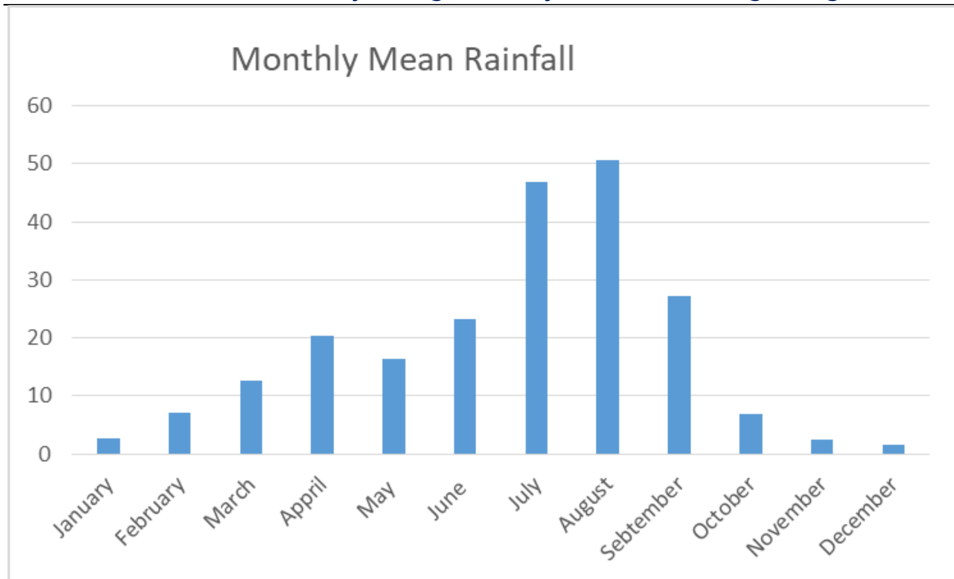


Figure 3-7 Mean Monthly Rainfall for 32 years (1985-2017) data of six station

3.2.3 Checking the Consistency of Data

The data recorded at gaging station and reported to the central recording organization. The recorded data for many years may not consistent due to missing recorded data at gaging station. After a number of years, data may be felt inconsistent and it needs readjustment. For this study, the mass curve method is used for data adjustment. The mass curve Akaki recording station adjusted to Addis Ababa Bole observation is shown in Figure 3.8 and Annex-C shows for the other five.

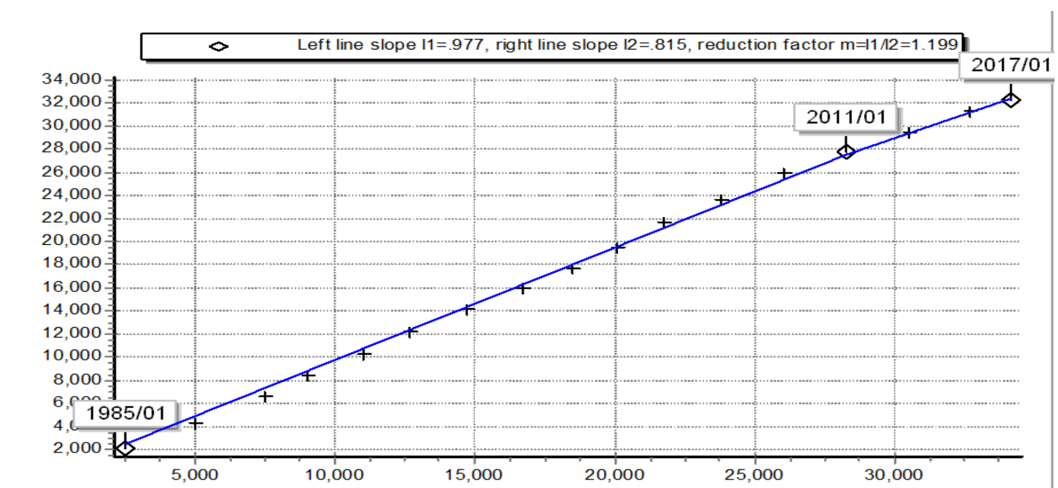


Figure 3-8 Double Mass curve of Akaki station with reference station of Bole Observation station

3.2.4 Topography

The land surface topography of the study area is extracted from ASTER, DEM which covers the entire world at high resolution, 30m and the transitional terrain located at the southern flank of Intoto ridge is considered for this study. The average elevation range between 2043 and 3377 which expanded in all direction is extracted from Digital Elevation Model (DEM). DEM data used for this investigation obtained from USGS Earth explorer web site. Figure 3.9 shows a topographic map of big Akaki river and all river across Addis Ababa Town.

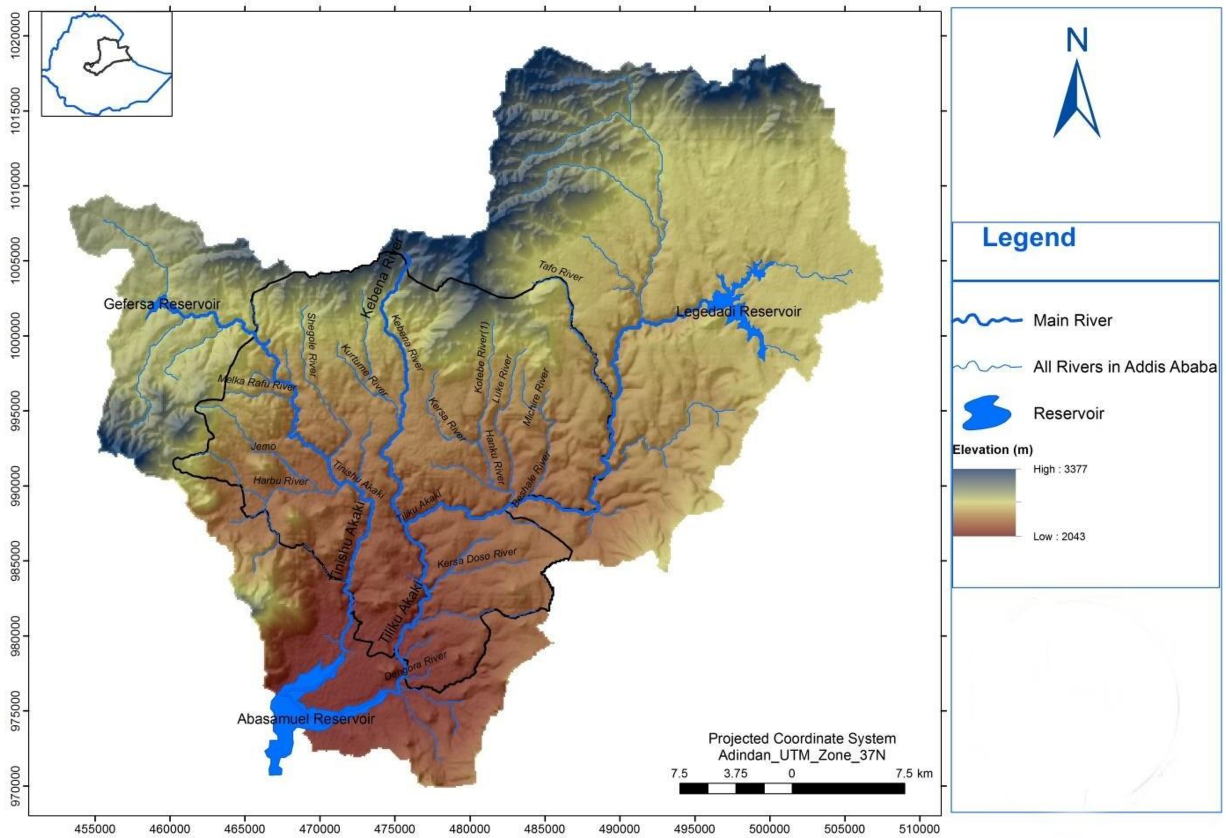


Figure 3-9 Digital Elevation Model Delineated for the Watershed of Study area

3.2.5 Land Use

The land use land cover data for this study is extracted by considering the outlet at Abasamuel reservoir and Turunesh Hospital. This land use is reclassified into forest, agricultural land, buildup area, and open water area. When the data extracted by selecting the outlet at Abasamuel reservoir, the catchment cover 2.58% forest, 82.77% agricultural land, 13.95% buildup area and 0.57% open water body. similarly, big Akaki watershed cover 1.95% tree cover areas, 82.85% agricultural land, 14.66% Built-up areas and 0.4% Open Water. Both watersheds include Dire, Sandafa, Burayu, and Addis Ababa town. This study uses data collected from European Space Agency (ESA) website which is developed land use at the global scale. This classification is more reliable than the land use land cover map prepared by the Ethiopian Ministry of Water and Energy Bureau. Therefore, land use land cover of the catchment is shown in Figure 3.10.

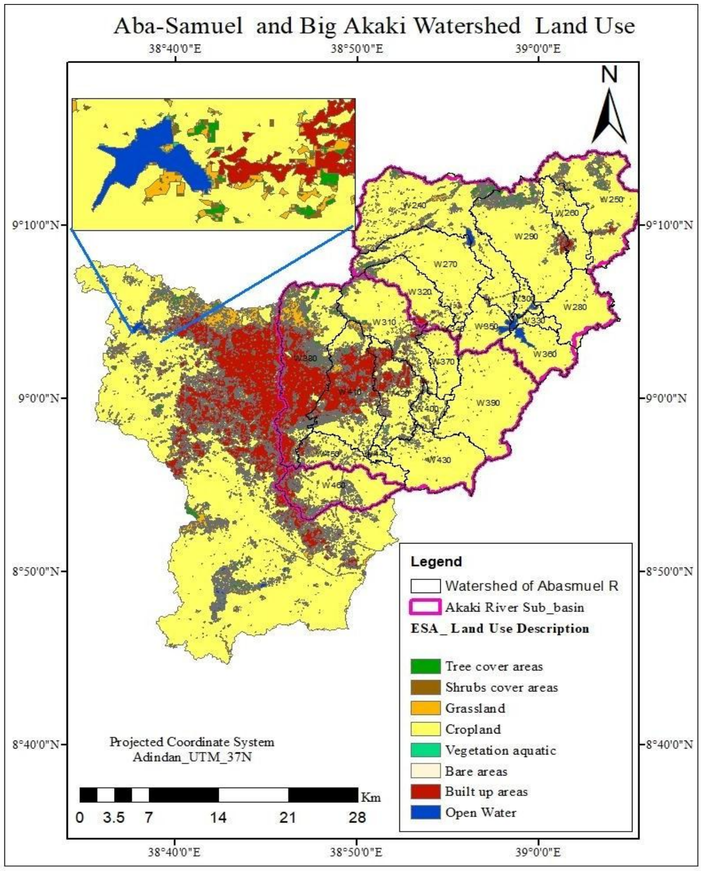


Figure 3-10 The most recent map delineated from the LC map series from the year 2015, at 300 m spatial resolution for global-scale land cover, Source: www.UCL-Geomatics 2017.

3.2.6 Soil and Geology

The geological condition of the study area suggested by different researcher includes Miocene-Pleistocene volcanic succession in the Addis Ababa area from bottom to top are: Alaji basalts, Intoto silicic, Addis Ababa basalts, Nazareth group, and Bofa basalts (UNEP, UNESCO, UN-HABITAT, 2003). Furthermore, the soil data is collected from the Ministry of Water, Mine and Energy Bureau and the soil type indicated in Figure 3.11 according to FAO soil classification.

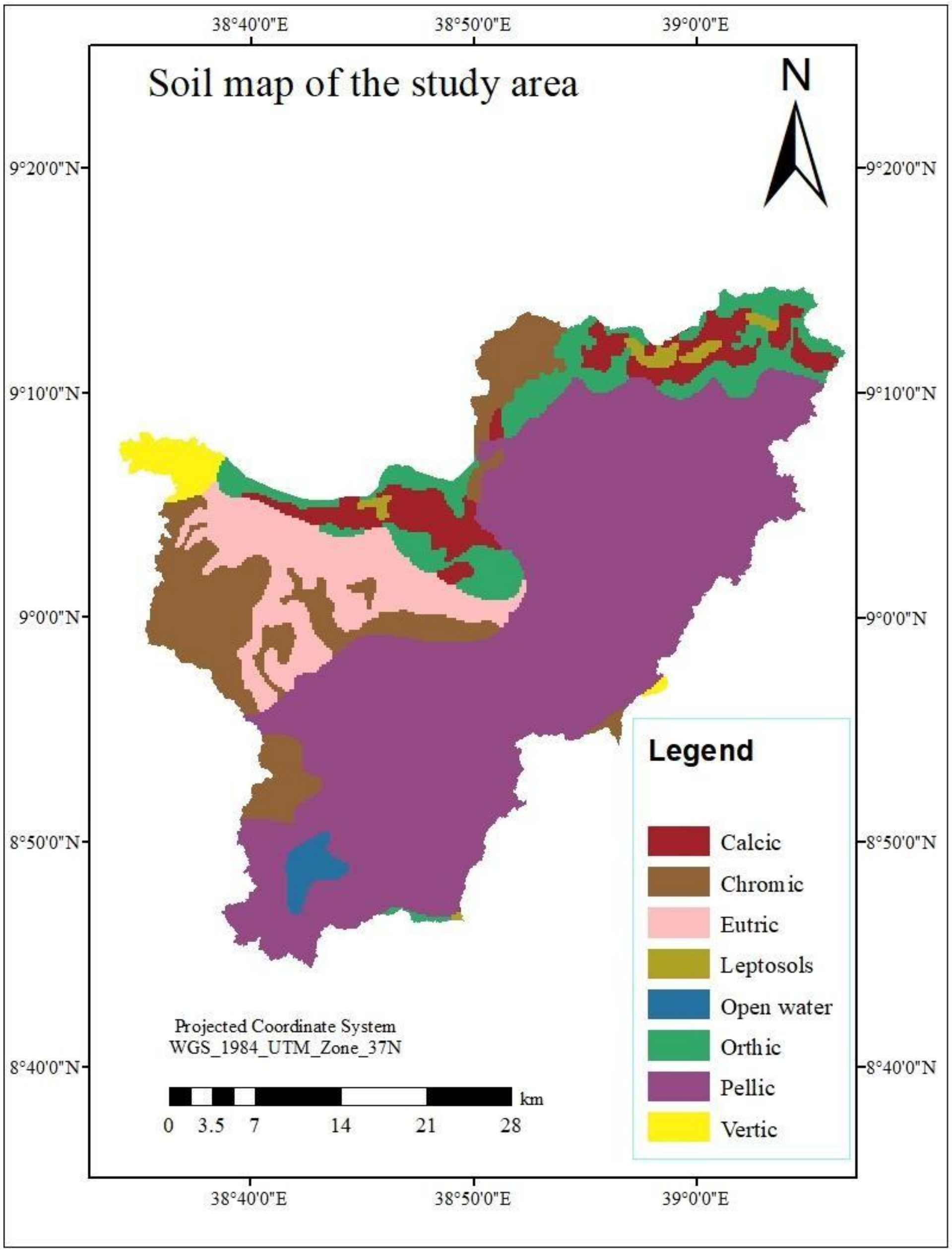


Figure 3-11 The General soil map of the project area.

3.3 METHODOLOGY

3.3.1 Introduction

This section summarizes the development of the hydrologic and hydraulic model of big Akaki watershed and the hydrologic characteristic across Addis Ababa which extends up to Abasamuel reservoir. The hydrologic modeling is using the software developed by the United States Army Corps of Engineers' (USACE) software is Hydrologic Engineering Center's Hydrologic Modeling System (HEC- HMS) Version 4.2.1 to perform the hydrologic modeling. Additionally, the hydraulic modeling if big Akaki river is using the software Hydraulic Engineering Centers of River Analysis System (HEC-RAS version 5.0.4). The software used for hydrologic and hydraulic modeling HEC-HMS and HEC-RAS with interface of ArcMap version 10.4 including their extension Hec-GeoHMS, Archydro, and Hec-GeoRAS. Furthermore, the frequency analysis of different return period is determined using Hydro gnomon version 4.0software.

3.3.2 Watershed Characteristics Across Addis Ababa

The watershed Characteristic is developed with the aid of GIS Version 10.4 and its extension Arc hydro and Hec-GeoHMS. The main two river across Addis Ababa city is little Akaki and big Akaki rivers which stream flow drain to the Abasamuel reservoir. The watershed characteristic includes river length, river slope, basin slope, longest flow path, basin centroid, and centroid longest flow path. The watershed is divided into 72 sub-basin based on drainage area network of $15km^2$ within the basin and locations of selected outlet. The area of network $15km^2$ is selected to obtain river drainage network that much with the existing physical river network. In this watershed, as it is noted by (UNEP, UNESCO, UN-HABITAT, et al. 2003) in 1940 Aba Samuel dam is found that which is built on the Akaki River, 30 km south of Addis Ababa. The dam has a storage capacity of 65 million cubic meters and an annual output of 23 million kilowatt- hr. However, due to siltation and pollution, it is not produce the required amount of electric power. This study considers for big Akaki River by selecting the outlet location around Turunesh (Beijing Hospital). The rainfall data collected from the ministry of water, mine and energy bureau to model the runoff hydrograph at the selected outlet location. Hence, the total length of big Akaki, little Akaki, and Kebena River is 71.55km, 47.55km and 20.51km respectively. Figure 4.1 shows the detail of the watershed characteristics across Addis Ababa city that extended to Abasamuel reservoir.

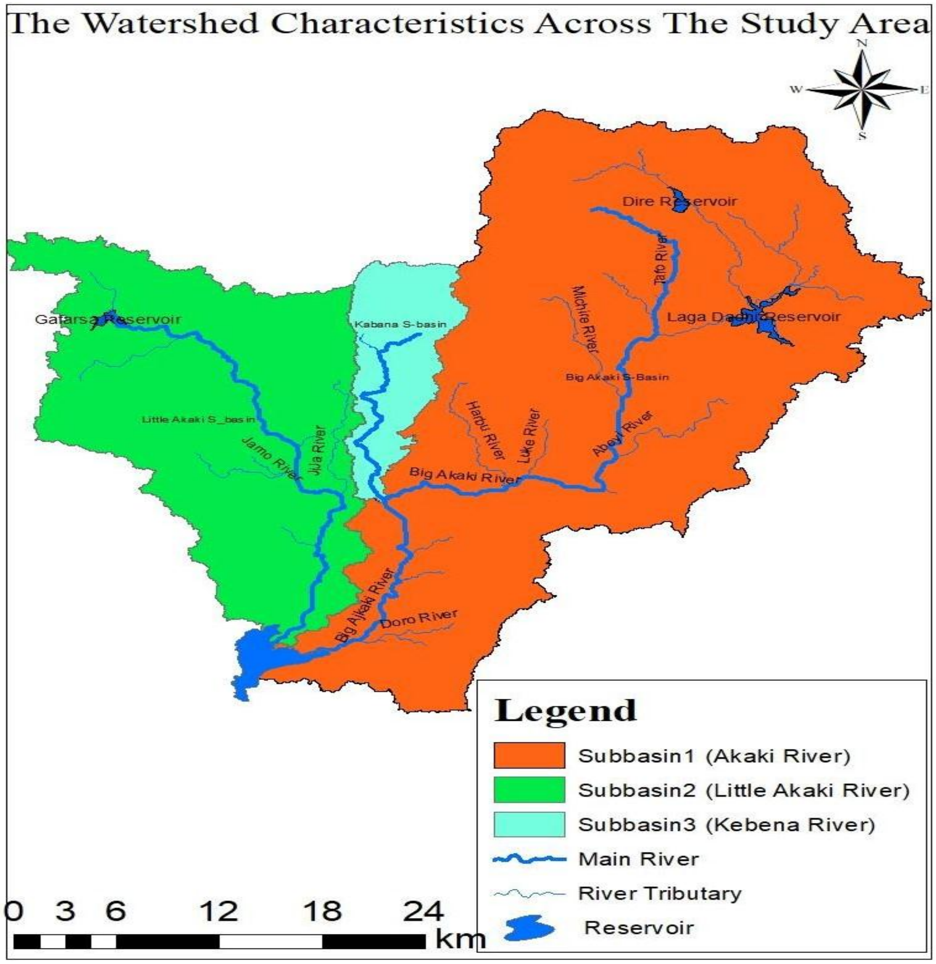


Figure 3-12 Characteristic of watershed outlet point selected at Abasamuel Reservoir

The watershed Delineation is made by selecting outlet at the entrance of the Abasamuel reservoir. All sub-basins delineated by considering drainage area network of 15km^2 and reshape the sub-basin using merging tool bar. As it is shown in figure 4.1 the watershed is divided into three main sub-basins which include big Akaki river, little Akaki river, and Kebena river sub-basin. There are preprocess and post process is included in in arc hydro tools. Preprocess of arc hydro using digital elevation model terrain (DEM) includes DEM manipulation of fill sink, flow direction, flow accumulation, stream definition, stream segmentation, catchment grid delineation, catchment polygon processing, drainage line processing, ad joint catchment processing, drainage point processing, longest flow path of catchment, longest flow path of ad joint catchment and slopes. The watershed post processing includes batch watershed delineation and batch sub-watershed delineation is performed in detail. Furthermore, Gafarsa, ‘Lega Dadi, and Dire reservoir are found in this watershed. The reservoir boundary is generated from land use land cover map which is developed by the European Space Agency (ESA) at the global scale of 300 resolutions. Additionally, the main river and the tributaries are developed with the aid of GIS extension tool arc hydro.

3.3.3 HEC-HMS Model Development

3.3.3.1 Basin Model Creation

The basin model of big Akaki watershed is created by applying Hec-GeoHMS and Arc hydro software with the function of Arc Map in GIS environment. Therefore, the first major step to create this basin model is to delineate the watershed and drainage line network at desired location. For this watershed outlet of the basin is selected at upstream of Akaki Bridge or near Turunesh Beijing Hospital. Before delineate watershed, Digital elevation model (DEM) of 8^0N38^0 E, 8^0N39^0 E, 9^0N38^0 E, and 8^0N39^0 E is merged using mosaic raster data into new raster data set of special analyst tool.

The data preparation for HEC-HMS with the aid of GIS tool extension of terrain preprocessing is discussed below. The first step is to manipulate the DEM data and then Arc Hydro geo-data consists of hydro features is connected to time series. Hydro features describe the physical environment through which water flows, and the time series describe the flow and water quality properties of the water within those features.

Generally, the terrain preprocessing of the study are is illustrated in figure 4.2

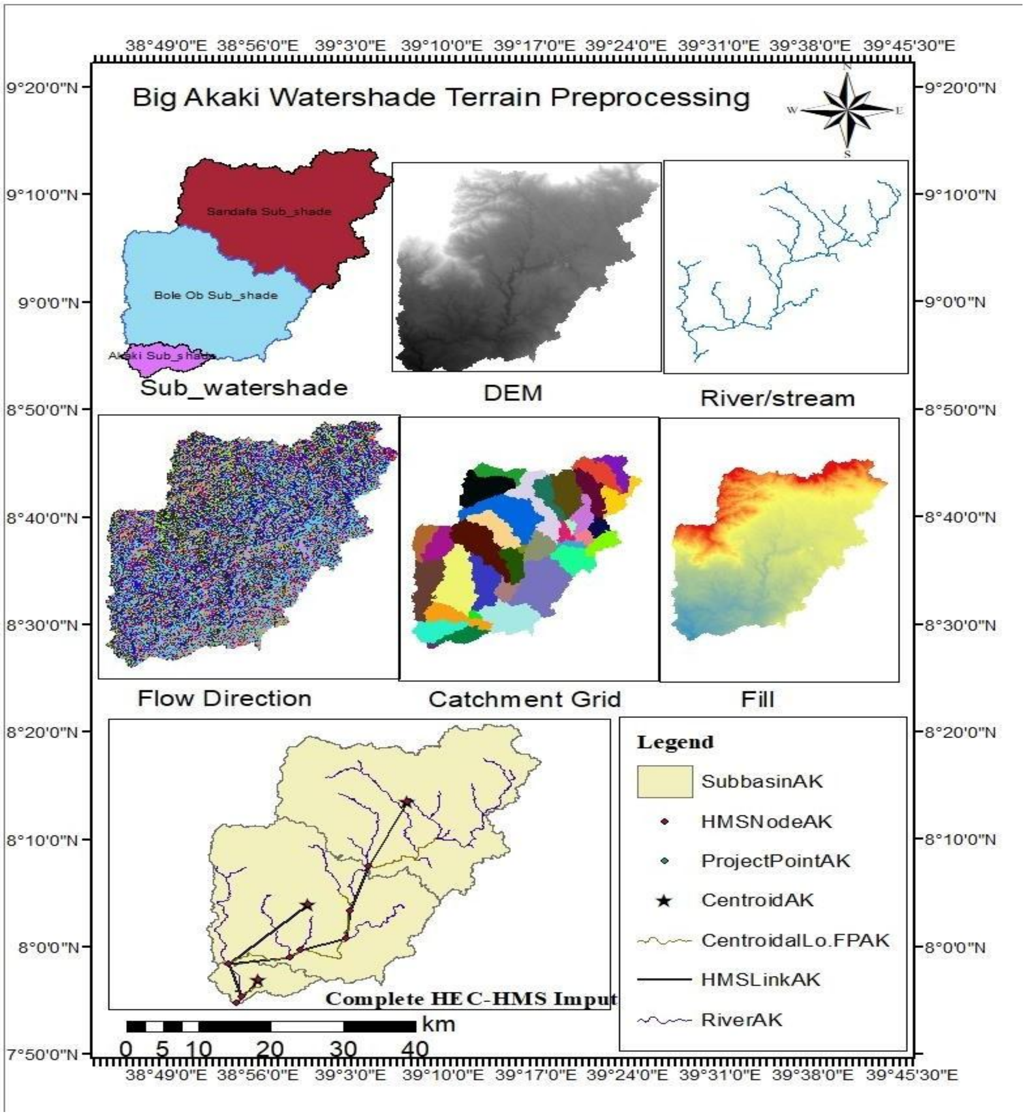


Figure 3-13 Big Akaki Watershed Terrain Preprocessing for HEC-HMS

To generate the completed dataset for the function of HEC-HMS the first step reconditioning of the DEM to adjust the elevation threshold and then filling the hole in surface terrain. As it is shown in the figure 4.2 after filling the DEM flow direction, flow accumulation, stream definition, stream segmentation, catchment grid delineation, catchment polygon processing, drainage line processing, ad joint line processing, drainage point processing, longest flow path for catchment, longest flow path for ad joint catchment and slope of the basin is processed respectively.

After complete terrain processing, Hec-GeoHMS used to export all data from GIS to HMS software. This software is the extension tools of GIS and its main components are preprocessing, project setup, basin processing, characteristic, parameters, HMS, and utility. Furthermore, the connection and background file is exported to the HMS software. Using project setup a project point had to be defined. The project point is defined outlet of the watershed that is intended to be used for the model development and thus has to be placed on a drainage line by carefully zooming required outlet location. Based on the outcomes of the terrain preprocessing and the definition of the project point, HEC-GeoHMS delineates the project area and creates all necessary layer files for this study area. Then all the created data is stored in a new geo database as raster data and vector data files.

The location of the project points that were defined for the different sub watershed (W) is automatically set the Outlet for all sub watershed with the function of the software. This software creates 23 sub-watershed within the Big Akaki river basin and all sub basins were named as W240 to W460. The resulting watershed has a total area of 921.80 km^2 and originally included 23 sub basins. In this catchment, there is six rainfall gaging station is used for all sub basin delineated at the outlet of study area. Each drainage station is assigned for the sub basin using GIS Theissen polygon tool and hence all sub basin with the up or downstream basin is shown in Figure 4.3. Those sub basins are assigned as Sandafa, Bole observation, Bole, Zukala,and Akaki sub watershed. This six sub basin had geometrical calculated area of 699.44km^2, 270.06km^2, 276.54 km^2, 674.01 km^2, 730.69 km^2 and 531.65km^2 respectively. The total model resulted 23 sub basins with a minimum and maximum area of 2 km^2 and 101 km^2 correspondingly.

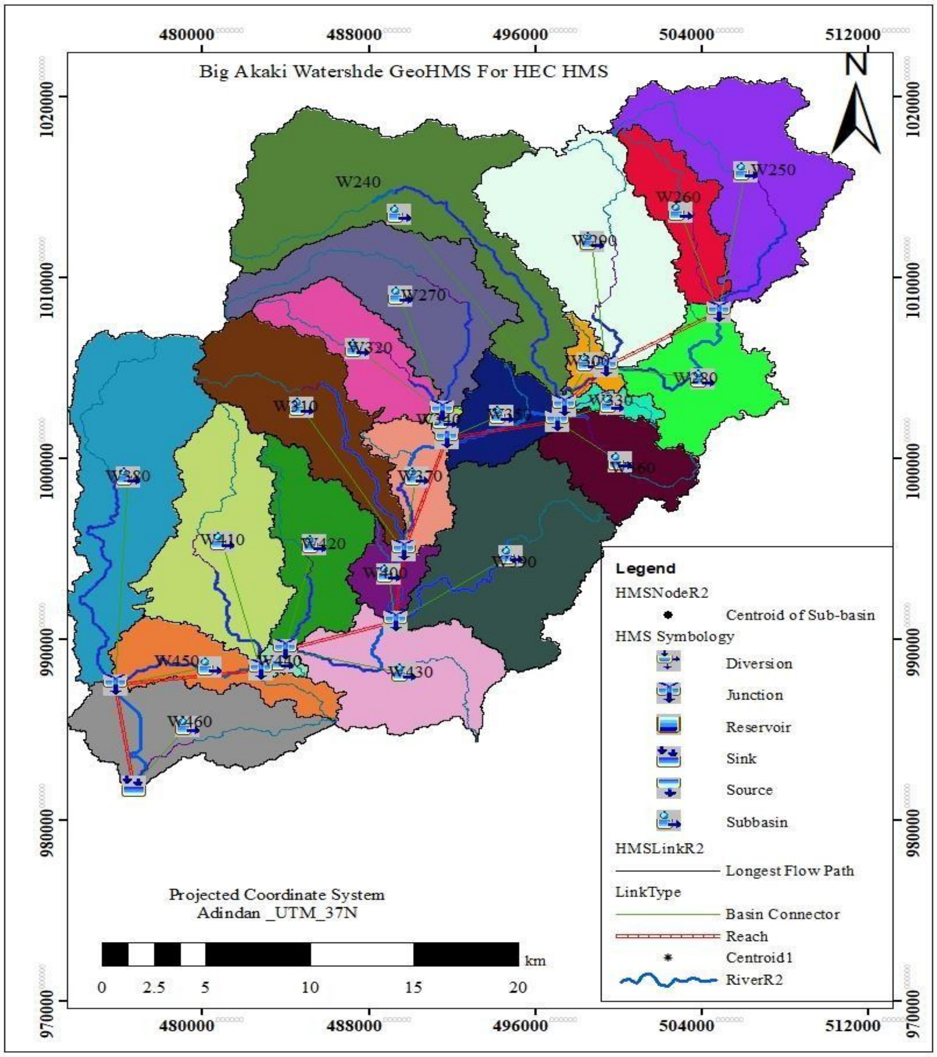


Figure 3-14 Big Akaki watershed delineated by assigning outlet at Akaki river (Turunesh Hospital)

For each of the resulting stream segments and the related sub basins, a serious of physically-based characteristics were computed based on the reconditioned DEM by using the HEC-GeoHMS tools. These characteristics include the lengths and slopes of each river segment as well as the average basin slope and the longest flow path of each sub basin. The resulting data is automatically stored in the attribute table of the river and sub basin layer. Hence in this water shade, there is 23 sub watershed, 11 junction, 9 stream flow, 2 reservoirs and one outlet (sink) point is included.

3.3.4 Hydrological Model

HEC-HMS model is used to simulate the excess runoff using rainfall data for small to the large watershed like those of big Akaki and Abasamuel. The schematic representation of hydrological model is illustrated in the fig 4.4 that summarize the system in the HEC-HMS software.

The simple schematic representation of all the HEC-HMS component is included the Fig.4.4

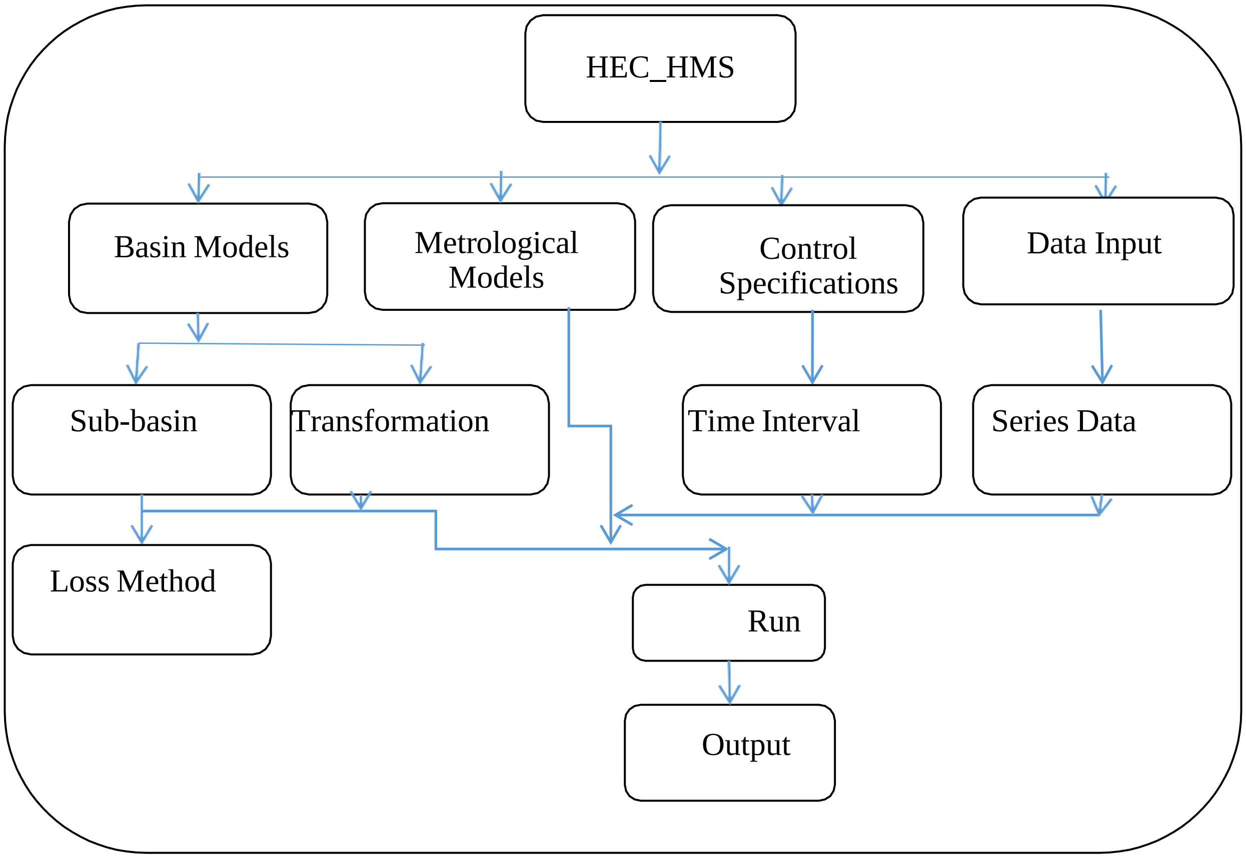


Figure 3-15 components of HEC-HMS

3.3.5 Meteorological model

3.3.5.1 Precipitation

The use of rainfall data is essential and fundamental to the rainfall-runoff process. The rainfall data are the driving force in the relationship. The accuracy of the rainfall data at a point (i.e., at the rain gauge) is extremely significant to all the remaining use of the data. Therefore all rainfall data of the gaging station of Sandafa, Addis Ababa Bole, Bole Observation, Intoto, Zukala and Akaki which is located within 18km radius for each representative sub basin in the study area.

Using Thiessen polygon an area-based weighting scheme, based upon an assumption that the precipitation depth at any point within a watershed is the same as the precipitation depth at the nearest gage in or near the watershed. Thus, it assigns a weight to each gage in proportion to the area of the watershed that is closest to that gage (USACE, 2000).

3.3.5.2 Evapotranspiration

As in the case of infiltration, a well-developed evapotranspiration (ET) theory exists for ideal conditions, conditions where the properties of the soil and the vegetative cover are well defined. However, the theory, as in the case of infiltration, is rarely implemented in a watershed model because the actual field situation deviates significantly from the ideal conditions assumed in the theory. Instead, the theory is used as a basis to develop many parametric methods that attempt to capture the essence of the evapotranspiration process for this reason for this study loss due to evapotranspiration is considered by Hargreaves which need the maximum and minim temperature. The evapotranspiration is calculated using hydro gnomon software 4.1 which capable to determine evapotranspiration using minimum and maximum temperature.

3.3.6 Control specification

Control specifications are one of the main components in a project, even though they do not contain much parameter data in the software. Their principal purpose is to control when simulations start and stop, and what time interval is used in the simulation. The one (1day) time interval used in this study based on the calibrated data.

3.3.7 Time series data

This is also one of the components of the project which is used to feed raw data. There are two methods for entering data into the project the simple method and DSS method is possible. But for this project, the simple method is used for the reason of easiness to use. The data is copy from excel and past in the time series data of the project.

3.3.8 Computing Runoff Volumes with HEC-HMS

HEC-HMS computes runoff volume by computing the volume of water that is intercepted, infiltrated, stored, evaporated, or transpired and subtracting it from the precipitation. Interception and surface storage are intended to represent the surface storage of water by trees or grass, local depressions in the ground surface, cracks and crevices in parking lots or roofs, or a surface area where water is not free to move as overland flow. Infiltration represents the movement of water to areas beneath the land surface. Interception, infiltration, storage, evaporation, and transpiration collectively are referred to in the HEC-HMS program and documentation as losses (USACE, 2000). The loss for the study area of basin calculated by the software using trial and error in the calibration of the loss but the evapotranspiration is calculated as is shown in Section 4 point 4.3.3.2. The initial and constant-rate loss model is considered in the model of runoff volume loss in this paper. Hence, With each model, precipitation loss is found for each computation time interval and is subtracted from the maximum area precipitation depth for that interval. The remaining depth is referred to as precipitation excess.

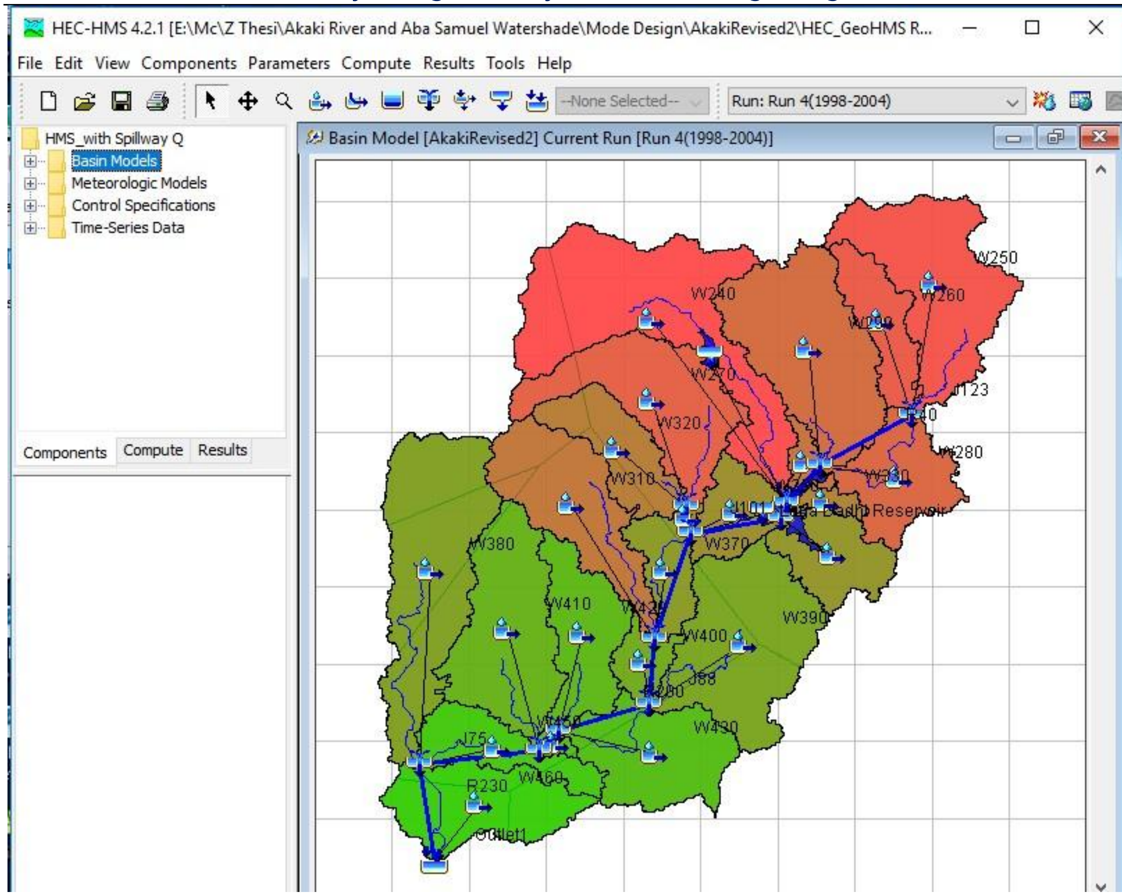


Figure 3-16 HEC-HMS Data setup with the background layer of all subbasin of watershed

All data proccessed in HEC-GeoHMS is imported to HEC_HMS and including the model setup all the drainage network, river center line and shape file data that used as background layer is shown in figure 4.5. Furthermore, each sub basin represented with (W) identified with color-coding; similarly, thick blue line display flood rout reach while tin blue line display river network and the software result is clearly analyzed at the outlet point.

The sub-watershed is sub divided with Theissen polygon using GIS and this polygon used as background as it is shown in figure 4.5 with tin black line. The category of each sub watershed weighted as it is illustrated in table 4.2 and the percent weight is considered into the model to use reference of rain gage.

Table 3-2 Each sub water shade divided for five rain gage station using Theissen polygon and their wattage

Theissen Polygon Name	Total Area of Each Theissen polygon (km2)	Sub-basin Name	Name of Theissen polygon shared	Area of each sub-basin(km2)	Area of Intersection	weightage(%)	Sub-basin Name	Name of Theissen polygon shared	Area of each sub-basin(km2)	Area of Intersection(km2)	Weightage(%)
Sandafa	699.44	W240	101				W370	19			
Bole	276.54		Sandafa		84	12		Bole	18		6.5
Bole obse	270.06		Intoto		17	2.5		Sandafa	1		0.14
Intoto	674.01	W250	68				W380	84			
Akaki	531.65		Sandafa		68	9.7		Bole	28		10.1
Zukala	730.69	W260	25					Intoto	24		3.6
			Sandafa		25	3.6		AA Bole Obser	32		11.8
		W270	59				W390	75			
			Sandafa	51		7.3		Bole	19		6.9
			Intoto	8		1.2		Sandafa	30		4.29
		W280	35					Zukala	26		3.56
			Sandafa	35		5	W400	12			
		W290	76					Bole	12		4.3
			Sandafa	76		10.9	W410	64			
		W300	7					Bole	62		22.4
			Sandafa	7		1		AA Bole Obser	1		0.4
		W310	52				W420	38			
			Bole	39		14.1		Bole	38		13.7
			Intoto	11		1.6	W430	47			
			AA Bole	2		0.7		Bole	7		2.5
		W320	31					Zukala	41		5.61
			Bole	6		2.2	W440	3			
			Sandafa	19		2.72		Bole	3		1.1
			Intoto	6		0.9		Zukala	0.01		0
		W330	6				W450	27			
			Sandafa	6		0.86		Bole	21		7.6
		W340	2					Zukala	6		0.82
			Sandafa	2		0.29	W460	45			
		W350	22					Bole	9		3.3
			Bole	0.01		0		Zukala	13		1.78
			Sandafa	22		3.15		Akaki	23		3.15
		W360	29								
			Sandafa	29		4.15					

3.3.9 Modeling Direct Runoff with HEC-HMS

The process of excess rainfall changed in to direct runoff considered as transformation of precipitation excess into point runoff. The unit hydrograph is developed for the determination of runoff and the unit hydrograph is a well-known, commonly-used empirical model of the relationship of direct runoff to excess precipitation. The known transform method is User-

Specified Unit hydrograph, Synthetic UH, SCS UH Model, Clark's UH model, Modelark model, and Kinematic-wave model incorporated in the software. For this study, we generate runoff using all methods and analyze the result to the actual local flow of the area determined with different researchers on this basin. Based on this analysis the Clark UH is selected for the runoff model of big Akaki sub-basin. Clark's model derives a watershed UH by explicitly representing two critical processes in the transformation of excess precipitation to runoff. This is movement of the excess from its origin throughout the drainage to the watershed outlet, and reduction of the magnitude of the discharge as the excess is stored throughout the watershed. This method requires the properties of the time area histogram and the storage coefficient. These two explicit requirement is determined based on the fit to the goodness of the result.

3.4 Runoff Frequency Analysis

The frequency concept for runoff can be discussed in terms of either the return period or the exceedance probability. we must continually recognize that frequency analysis is a statistical tool that can be applied to any random variable, not just runoff. Statistical frequency analysis is the most commonly used procedure for the analysis of flood data at a gaged location.

For this study, the frequency analysis is estimated using Hydro gnomon hydrologic software to determine peak flow within the return period of 25year, 50year, 100year, and 1000 years. For this, the mathematical model equation of different formula is compared and the best fit from the graph is selected. Accordingly, from figure 3.5 the gamma family Log person III and the Pareto family of L-moments EV3 is selected and the peak flow is determined for the listed return period. Frequency analysis is used to develop the frequency curve used for estimating damage that is related to the peak flow in a stream, maximum peak flows should be selected from the record. Occasionally, it is necessary to select a related variable in place of the one desired. For example, where mean-daily flow records are more complete than the records of peak flows, it may be more desirable to derive a frequency curve of mean-daily flows and then, from the computed curve, derive a peak-flow curve by means of an empirical relation between mean daily flows and peak flows.

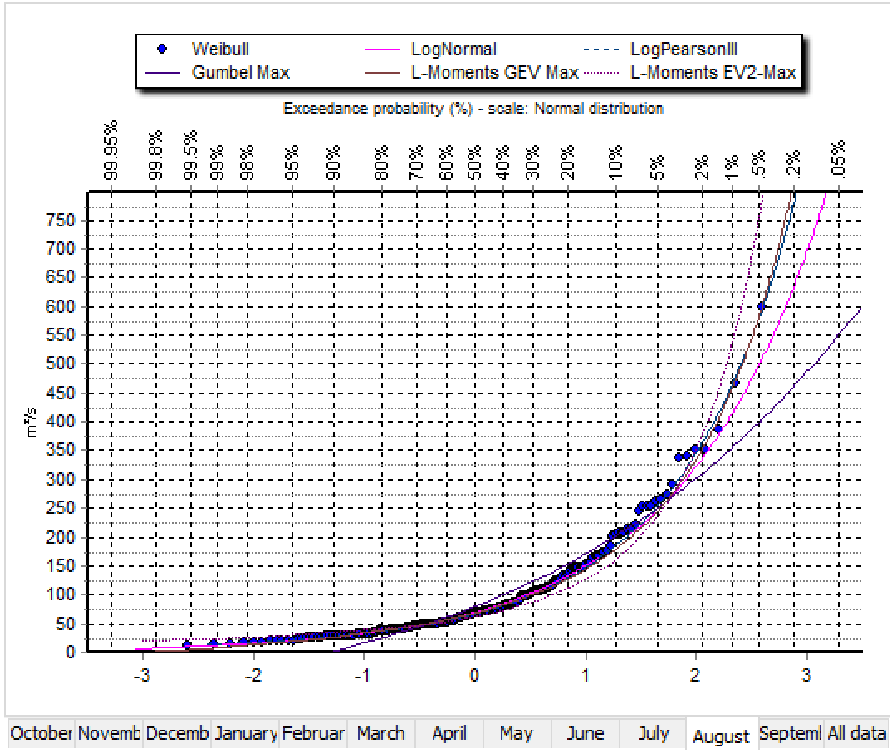


Figure 3-17 The exceedance probability function versus the peak flow

If one wished to design a flood protection work that would be exceeded, on average, only one time every 100 years (one percent chance exceedance), the usual design would be based on the normal standard deviate of 2.326. Notice that there is a 0.5 percent chance that this design level may come from a true curve that would average 22 exceedances per 100 years. On the other side of the curve, instead of the expected one exceedance, there is a 99.5 percent chance that the "true" curve would indicate 0.004 exceedances. Note the large number of exceedances possible on the left side of the curve. This relationship is highly skewed towards the large exceedances because the bound on the right side is zero exceedance. A graph of the number of possible "true" exceedances versus the probability that the true curve exceeds this value.

From the graph developed for the Akaki river frow analysis peak flow is obtained from the simulation analysis of HEC_HMS is about 599.7m^3/s.The exceedance probability of this value from the graph is 5% exceed the peak flow value. Similarly, the peak flow for different return period is shown in the figure 4.7.

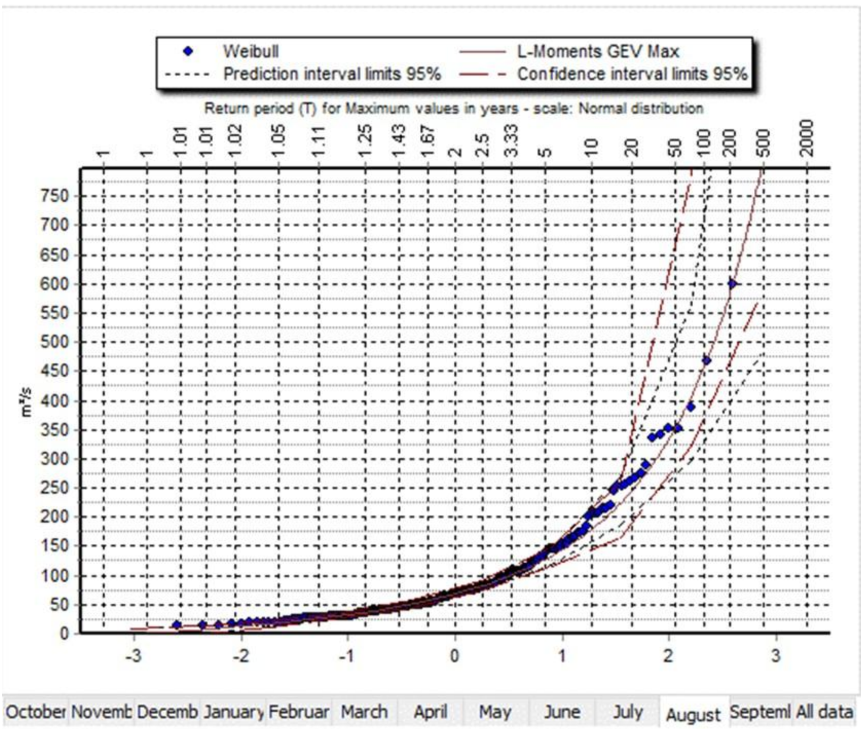


Figure 3-18 The peak discharge of Big Akaki river for different return period

The sample flood frequency curve represents some average estimate of the true population curve. When the flood estimate obtained from a frequency curve is used for design, we recognize that there is a high probability, possible 50% that the true value actually lies above the sample estimate of the T-yr event. This implies that there is a good likelihood that locations will be flooded by a T-yr event even when the design is based on the T-yr estimate obtained from a flood frequency analysis. Due to the possibility of flooding in areas where public facilities have been designed to provide flood protection at a T-yr level of protection, public agencies may wish to reduce the average flooding due to the T-yr event to some percentage well below that provided by the T-yr frequency curve estimate. This is one possible motivation for computing and using a confidence interval on the frequency curve estimate. Either one-sided or two-sided confidence intervals can be developed. A two-sided confidence interval provides both an upper and lower limit. For one-sided confidence intervals, either a lower or an upper limit, but not both, can be constructed. The type of problem will determine which form is used. Therefore in this study, the two-side interval is developed as it is shown in figure 4.7 above. The graph indicated in figure 4.6 and 4.7 is developed using hydro gnomon software version 4.1

3.5 Curve Number Grid development

As described in Section 3, this loss method only requires the definition of a single input parameter, the curve number. The CN depends on the land-use and the hydrologic characteristics of the topsoil and is determined based on tables provided below. The methodology for the determination of the land-use, the hydrologic soil group and the hydrologic condition of the topsoil is presented in the following sections.

3.5.1 Determination of the Curve Number

The hydrologic soil is discussed in Section 2 and the hydrologic soil group used from the table developed by the European Space Agency (ESA) that is developed on a global scale. Accordingly, the curve number grid is developed using the Hec-GeoHMS of Arc Map extension tool. The grid developed using land use land cover and hydrologic oil group. The land use reclassified into four main classes such as forest, open water area, and agricultural land and built-up areas and the look table is prepared for these classes from table 4.2. The grid is the intersection of both hydrologic soil group and land use that enable as to calculate the curve number using the Hec-GeoHMS for all parcel of land in the watershed of the study area. The estimated curve number used in HEC-HMS software to consider the loss in the sub-basin is illustrated as follows.

Table 3-3 Curve Number Lookup table used in the model (Chow, 2010)

Land use Description		Curve Number for hydrologic soil group – Hydrologic soil Group A	B	C	D
Cultivated land	without conservation treatment	72	81	88	91
		62	71	78	81
Pasture or range land	Poor condition	68	79	86	89
	Good condition	39	61	74	80
Meadow	Good condition	30	58	71	78
Wood or forest land	Thin stand, poor cover, no mulch	45	66	77	83
	Good cover	25	55	70	77
Open space, lawns, parks, golf course, cemeteries, etc	Good condition: grass cover with 70% or more of the area	39	61	74	84
	Fair condition: grass cover on 50% to 75% of the area	49	69	79	84
Commercial and business areas 85% impervious		89	92	94	95
Industrial districts 72% impervious		81	88	91	93
Residential					
Average lot size	Average % impervious				
506 Square meter or less	65	77	85	90	92
1012 Square meter	38	61	75	83	87
1349 Square meter	30	57	72	81	86
2023 Square meter	25	54	70	80	85
4047 Square meter	20	51	68	79	84
Paved parking lots, roof, driveways, etc		98	98	98	98
Street and road Paved with curbs and storm sewer		98	98	98	98
Gravel		76	85	89	91
Dirt		72	82	87	89
Initial abstraction is estimated Ia=0.2S, S=(1000/CN)-10					

The cover type is defined based on the comparison of the major plants as listed in the CN table and the observed vegetation cover on site. The land user coverage of the study area is shown in Section 3 under point 3.6. The definition of the CN table is found to most accurately describe the land cover of the representative area. The grid develops as figure 4.8 to determine the curve number of all sub-basin.

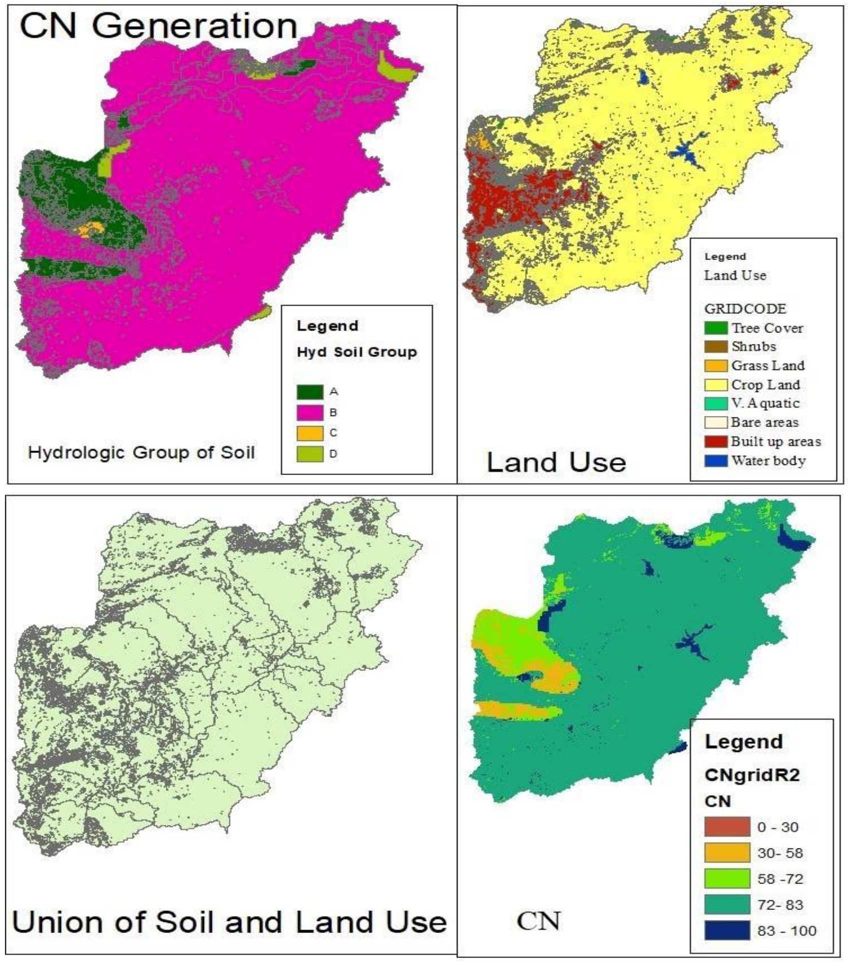


Figure 3-19 The curve number grid preparation using Hec-Geo HMS of Big Akaki Watershed

The generated curve number of the big Akakki river mostly within the curve number of 58 to 72 and 72 to 83 as it is shown in figure 4.8 and using this curve number the initial abstraction of the watershed is determined. The curve number of each sub-basin shown in figure 4.3 extracted from this curve number grid and it is tabulated in table 3.4.

Table 3-4 CN for each sub watershed

Sub-Basin Name	Basin CN	Area_ HMS(km2)		Sub-Basin Name	Basin CN	Area_ HMS(km2)
W240	76.64	101				
W250	76.81	68		W360	78.12	29
W260	76.31	25		W370	76.96	19
W270	76.31	59		W380	65.54	84
W280	76.92	35		W390	77.16	75
W290	76.64	76		W400	77.16	12
W300	78.98	7		W410	68.78	64
W310	76.13	53		W420	73.14	38
W320	76.56	31		W430	76.99	47
W330	80.18	6		W440	77.30	3
W340	77.00	2		W450	76.83	27
W350	78.01	22		W460	76.64	46

3.6 Baseflow separation

In HEC-HMS, the base flow model is applied both at the start of simulation of a storm event and later in the event as the delayed subsurface flow reaches the watershed channels. HEC-HMS includes three alternative models of base flow: Constant, monthly-varying value; Exponential recession model; and Linear-reservoir volume accounting model (USACE, 2000). In this study, the exponential recession model is selected for the base flow model.

The parameters of this model include the initial flow, the recession ratio, and the threshold flow. As noted, the initial flow is an initial condition. For analysis of hypothetical storm runoff, initial flow should be selected as a likely average flow that would occur at the start of the storm runoff. For frequent events, the initial flow might be the average annual flow in the channel. Therefore, the initial flow is calculated using methods for continuous separation generally divide the flow into one quick and one delayed component using an automated time-based separation. The delayed flow component is thought to represent the proportion of flow that originates from groundwater and other delayed sources, by (Gregor, 2010) defined as the baseflow, Qb. Time-series of base flow have been seen as useful as a measure of the dynamic behavior of groundwater in a catchment, whereas the baseflow proportion of the total flow has as an index of the catchment’s ability to store and release water during dry weather. The separation of base flow is using hydro office software BFI and the result is shown in figure 4.9. For the baseflow separation is created by a number of methods like fixed interval, sliding interval and local minimum methods are included with the software. In this study simple interval is used due to assigns the lowest discharge in each interval (N) to all days in that interval starting with the first

Day of the period of record. This method can be visualized as moving a bar 2N* days wide upward until the bar first intersects the hydrograph. The discharge at that point is assigned to all days in the interval. The bar is then moved 2N* days horizontally, and the process is repeated. Accordingly, the value of initial average base flow 8.5m^3/s is determined for the flow duration of 1998 to 2004 years. The recession ratio and threshold flow is obtained during the calibration process.

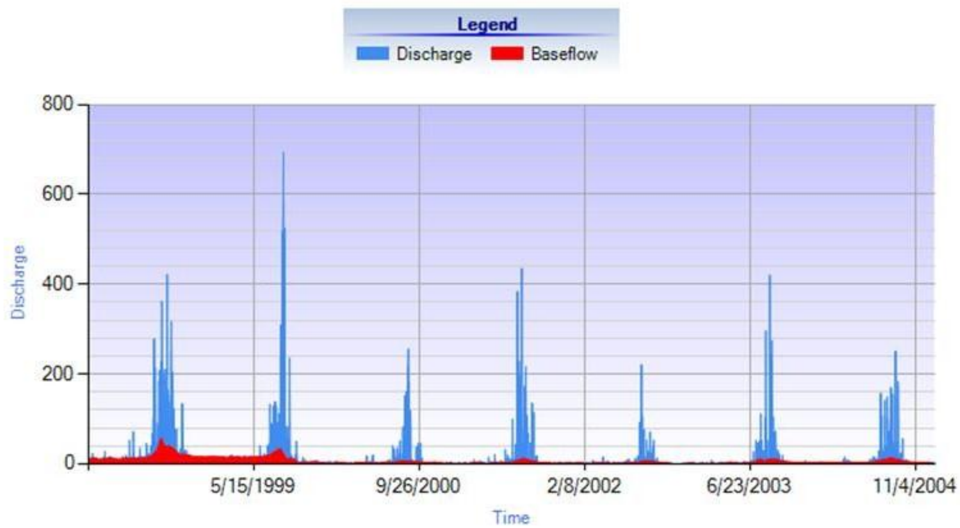


Figure 3-20 Baseflow separation using hydro office BIF

From the figure, the area filled by red color represents the base for each year and the blue color represents the direct flow.

3.7 Model Calibration and Validation of HEC- HMS

3.7.1 Calibration

The Calibration and validation of the models are necessary before using them in research and the calibrated flow values are used for flood map delineation. Model calibration is a systematic process of adjusting model parameter values until model results match acceptably with observed data. The quantitative measure of match value is described by the objective function. In the precipitation-runoff models, this function measures the degree of variation between computed and observed hydrographs. Furthermore, the calibration process finds the optimal parameter values that minimize the objective function. Calibration can either be manual or automated (optimization). Manual calibration relies on the user’s knowledge of basin physical properties and expertise in hydrologic modeling. In the automated calibration model parameters are iteratively adjusted until the value of the selected objective function is minimized. The objective function choices Goodness-of-fit indices in HEC-HMS are described in the following section..

Sum of absolute errors:- This objective function compares each ordinate of the computed hydrograph with the observed, weighting each equally. The index of comparison, in this case, is the difference in the ordinates. However, as differences may be positive or negative, a simple sum would allow positive and negative differences to offset each other. In hydrologic modeling, both positive and negative differences are undesirable, as overestimates and underestimates as equally undesirable. To reflect this, the function sums the absolute differences. Thus, this function implicitly is a measure of fit of the magnitudes of the peaks, volumes, and times of peak of the two hydrographs. If the value of this function equals zero, the fit is perfect: all computed hydrograph ordinates equal exactly the observed values. Of course, this is seldom the case (USACE, 2000).

Sum of squared residuals:- This is a commonly-used objective function for model calibration. It too compares all ordinates but uses the squared differences as the measure of fit. Thus a difference of 10 m^3/sec scores 100 times worse than a difference of 1 m^3/sec. squaring the differences also treats overestimates and underestimates as undesirable. This function too is implicitly a measure of the comparison of the magnitudes of the peaks, volumes, and times of peak of the two hydrographs.

Table 3-5HEC-HMS Objective function for calibration (USACE, 2000)

Criterion	Equation
Sum of absolute errors (Stephenson, 1979)	$Z = \sum_{n=i}^{NQ} \lvert q_o(i) - q_s(i) \rvert$
Sum of squared residuals (Diskin and Simon)	$Z = \sum_{i=n}^{NQ} \left[q_o(i) - q_s(i) \right]^2$
Percent error in peak	$Z = 100 \left\lvert \frac{q_s(peak) - q_o(peak)}{q_o(peak)} \right\rvert$
Peak-weighted root mean square error objective function (USACE, 1998)	$Z = \left\{ \left(\frac{1}{NQ} \right) \sum_{i=1}^{NQ} \left[\left(q_o(i) - q_s(i) \right)^2 \left(\frac{q_o(i) + q_o(mean)}{2 q_o(mean)} \right) \right] \right\}^{\frac{1}{2}}$

Z=objective function; NQ=number of computed hydrograph ordinated; $q_o(i)$= observed flow; $q_s(i)$= calculated flow, computed with a selected set of model parameters; q_o (peak)= observed peak; q_s(Peak)= calculated peak and q_o (mean)= mean of observed flow

Percent error in peak: - This measures only the goodness-of-fit of the computed-hydrograph peak to the observed peak. It quantifies the fit as the absolute value of the difference, expressed as a percentage, thus treating overestimates and underestimates as equally undesirable. It does not reflect errors in volume or peak timing. This objective function is a logical choice if the information needed for designing or planning is limited to peak flow or peak stages. This might be the case for a floodplain management study that seeks to limit development in areas subject to inundation, with flow and stage uniquely related (USACE, 2000).

Peak-weighted root mean square error:-This compares all ordinates, squaring differences, and it weights the squared differences. The weight assigned to each ordinate is proportional to the magnitude of the ordinate. Ordinates greater than the mean of the observed hydrograph are assigned a weight greater than 1.00, and those smaller, a weight less than 1.00. The peak observed ordinate is assigned the maximum weight. The sum of the weighted, squared differences is divided by the number of computed hydrograph ordinates; thus, yielding the mean squared error. Taking the square root yields the root mean squared error. This function is an implicit measure of comparison of the magnitudes of the peaks, volumes, and times of peak of the two hydrographs (USACE, 2000).

The initial values of parameters that are subject to automated calibration are required to start an optimization process. The HEC-HMS model has default hard constraints that limit the range of optimized values within reasonable physical intervals. Values within hard constraints do not cause numeric instabilities or errors in computations. The objective function for this study with HEC-HMS version 4.2 is shown in figure 4.10.

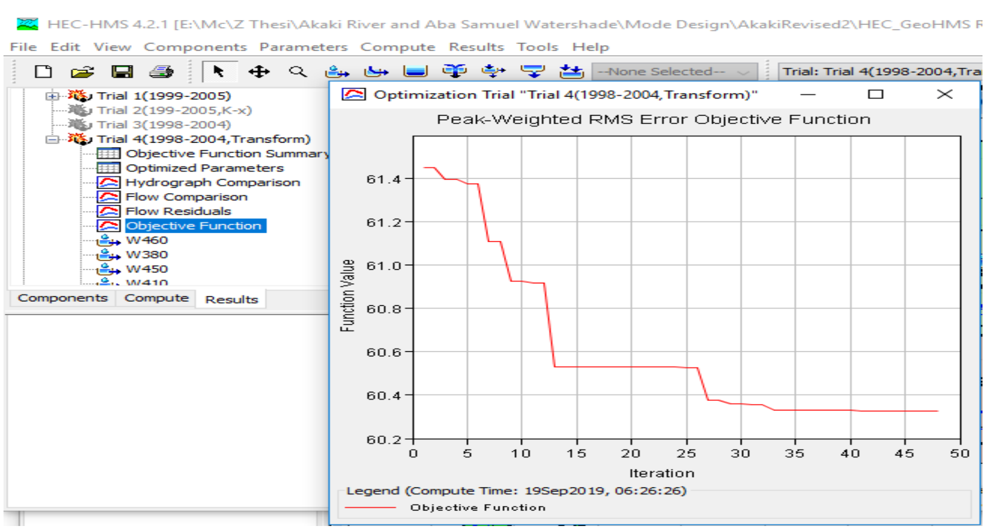


Figure 3-21 Objective function

3.7.1.1 Model efficiency evaluation criteria

As stated earlier, the main goal of calibration in HEC-HMS is to identify reasonable parameters that yield the best fit of computed to observed hydrograph, and this parameter is evaluated with an efficiency model to simulate the rainfall data with the observed flow data hydrograph.

For this study, the evaluation technique is Nash Sutcliffe Efficiency (NSE) and coefficient of determination (R^2) which are widely applicable in hydrologic modeling. The Nash-Sutcliffe Efficiency (NSE) is a normalized statistic that determines the relative magnitude of the residual variance (noise) compared to the measured data variance (Misganaw, et al.2016). The Nash-Sutcliffe simulation efficiency (NSE) indicates how well the plot of observed versus simulated value fits the 1:1 line. If the measured value is the same as all predictions, NSE is 1. If the NES is between 0 and 1, it indicates deviations between measured and predicted values. If NES is negative, predictions are very poor, and the average value of output is a better estimate than the model prediction (ELIAS, et al. 2015). This two evaluation criterial is shown in the following equation.

$$NSE = 1 - \frac{\sum (Q_{obs(t)} - Q_{sim(t)})^2}{\sum (Q_{obs(t)} - \overline{Q_{obs}})^2}$$

Equation 3-1

Where, NSE = Nash and Sutcliffe Efficiency, Qobs = observed value at the i th time interval, Qsim = simulated value at the ith time interval and = mean of the observed discharges.

Whereas the Coefficients of determination (R^2) describe the degree of co-linearity between simulated and measured data (Misganaw, et al.2016).

$$R^2 = \frac{(\sum (Q_{obs(t)} - \overline{Q}_{obs}) \sum (Q_{sim(t)} - \overline{Q}_{sim(t)}))^2}{\sum (Q_{obs(t)} - \overline{Q_{obs}})^2 \sum (Q_{sim(t)} - \overline{Q}_{sim(t)})^2}$$

Equation 3-2

Where, R^2= coefficient of determination, Qobs = observed value at the i^{th} time interval, Qsim= simulated value at the i^{th} time interval $\overline{Q}_{bs(t)}$ mean of observed discharges and $\overline{Q}_{im(t)}$ mean of simulated discharges. In this study, the parameter calibration criteria is based on table 4.4

Table 3-6 Calibration parameter constraints (USACE, 2000)

Model	Parameter	Minimum	Maximum
Initial and constant-rate loss	Initial loss	0 mm	500 mm
	Constant-loss rate	0 mm/hr	300 mm/hr
SCS loss	Initial abstraction	0 mm	500 mm
	Curve number	1	100
Green and Ampt loss	Moisture deficit	0	1
	Hydraulic conductivity	0 mm/mm	250 mm/mm
	Wetting front suction	0 mm	1000 mm
Deficit and constant -rate loss	Initial deficit	0 mm	500 mm
	Maximum deficit	0 mm	500 mm
	Deficit recovery factor	0.1	5
Clark'd UH	Time of concentration	0.1 hr	500 hr
	Storage coefficient	0 hr	150 hr
Synder's UH	Lag	0.1 hr	500 hr
	C_p	0.1	1
Kinematic wave	Manning's n	0	1
Base flow	Initial base flow	0 m^3/s	100000 m^3/s
	Reccession factor	0.000011	-
	Flow -to-peak ratio	0	1
Muskingum routing	K	0.1 hr	150 hr
	X	0	0.5
	Number of steps	1	100
Kinematic wave routing	N-value factor	0.01	10
Lag routing	Lag	0 min	30000 min

The Successful calibration requires an accurate and reliable historical record of both rainfall and stream data. We used six rain gage stations with recorded rainfall data of 32 years from 1985 up to 2017 collected from the Ministry of Water, Mineral and Energy bureau. The 5-year long observation period from June 2008 to May 2013 is selected for the calibration of the continuous

Model (Sun, 2015). Similarly, the calibration period of this study relies on five year rainfall data that occurred between January 1998 up to December 2004 is selected. The calibration time is selected based on the magnitude of peak flow and time of year with low missing flow data. The maximum flow recorded at Akaki flow gage station is occurred in 1999 year with magnitude of 691.3m^3/s.

According to criteria for listed in the table 4.5 calibration parameter constraints, each optimization output is processed. If the output is acceptable, the calibration process is completed, otherwise, the initial optimization parameters were altered and the process repeated. The calibration process started with hydrometric stations that represented outlets of single sub basins. Once these stations were calibrated, hydrometric stations with more than one contributing sub basins followed. In the final stage, individually calibrated sub basins were linked into one model and the calibration finalized. The calibration process is shown in figure 4.11.

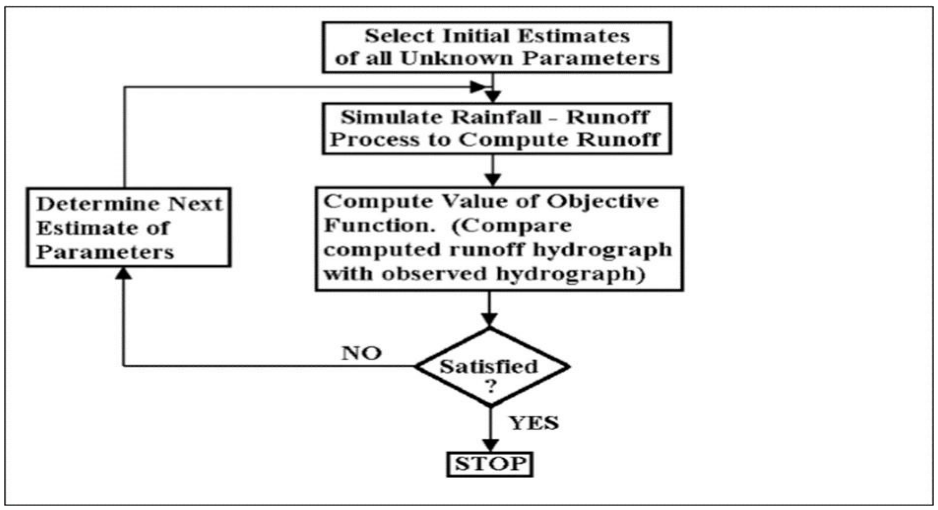


Figure 3-22 HEC-HMS Calibration Process

After the completion of calibrated parameter for the model, the accepted values are used for flow simulation at the outlet location of the study area.

3.7.2 Validation

Validation is a process of testing model ability to simulate observed data without using the calibration data, with acceptable accuracy. During this process, calibrated model parameters are not subject to change, their values are kept constant. The verification output is assessed by flow comparison graphs and the statistical goodness-of-fit measures.

3.8 Hydraulic Modeling

The main objective of the hydraulic model is to estimate the flood map of big Akaki river related to flood runoffs with return periods of 25, 50, 100, and 1000 years. Additionally, the geographic nature of study area shown in figure 4.12 which is vulnerable to flood and it require buffer zone of river bank by developing flood map around the river. The common approach for the delineation of a flood hazard map based on the depth and the velocity of the water in the floodplains predicted by the flow model is found to be inappropriate for the given situation. In order to obtain results that could be incorporated into a flood inundation map, hydraulic models with different cross-section configurations and additional geometric features were developed with HEC-GeoRAS and analyzed with regarding the validity and accuracy of the simulation results. The final modeling approach that is developed based on the preliminary simulations is described in the following section.

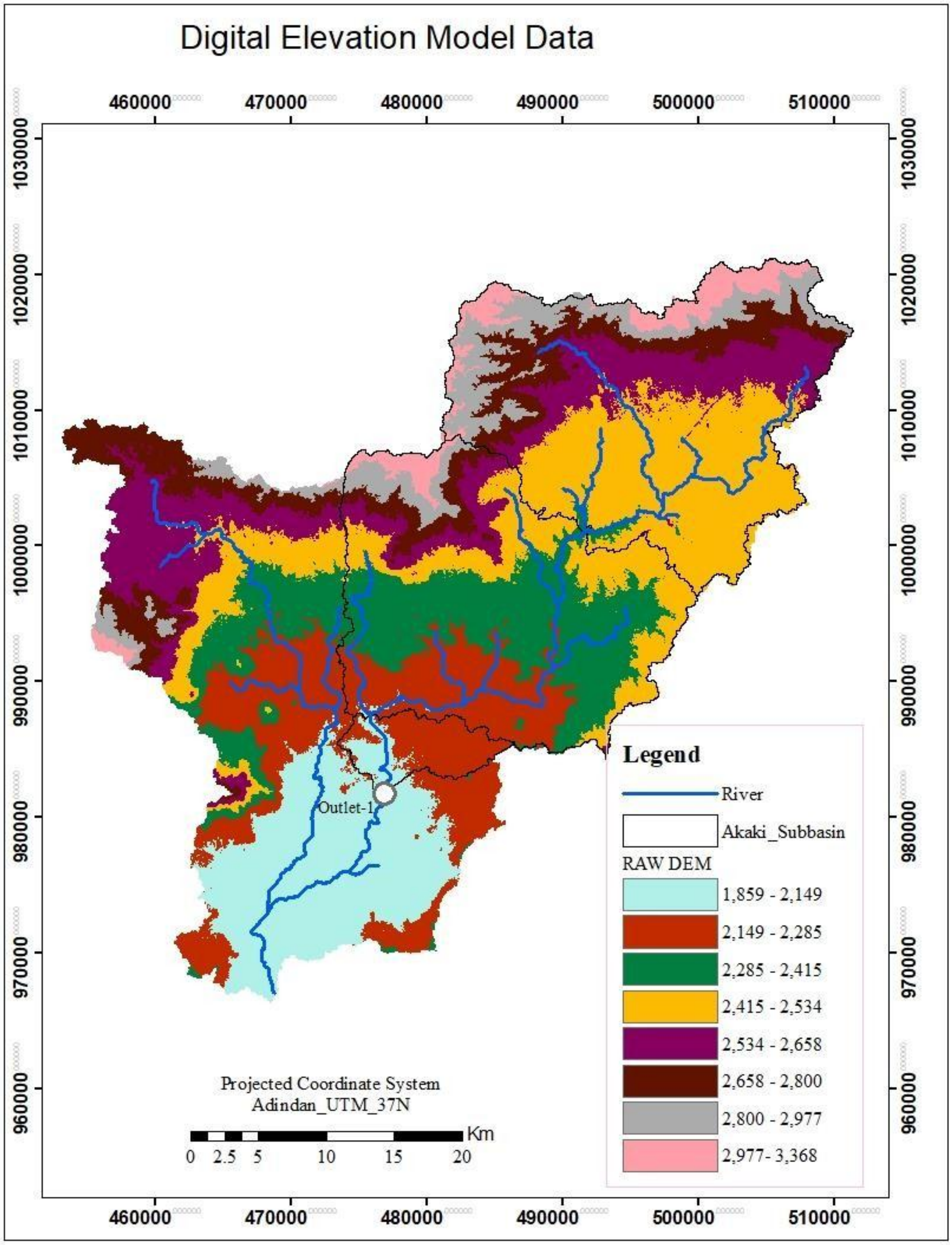


Figure 3-23 The geographical classification of elevation within the study river basin

This figure shows different elevation range of the study area and the range classified 1859m to 2149 m is most vulnerable to flood. The outlet of the big Akaki river basin shown in figure above selected at Turunesh Hospital. Hence, starting from the location of the outlet to the downstream reach length displayed with cyan color is developed for the estimation of flood inundation zone.

3.8.1 HEC-GeoRAS

HEC-GeoRAS is the extension tool of ArcGIS used to perform data preparation on the GIS and import processed data to HEC-RAS. GIS is used Digital Terrain Model (DTM) to process the imported data for further use in HEC-GeoRAS. HEC-GeoRAS uses DTM represented by a triangulated irregular network (TIN) or a GRID. It used to create a series of points, lines, and polygon layers on the GIS layer to develop geometric data for HEC-RAS. Some of the line layer created in the model include centerline of river or stream, left and right bank of river, flow path centerline which is optional and cross-sectional cut lines. Additionally, land use grid, levee alignment, ineffective flow area, blocked obstruction, bridge/culverts, inline structures, lateral structures, and storage areas also incorporated in the model.

The location river centerline, riverbank, flow path, river cross-section and hydraulic structure like bridge is prepared before exported to Hec-RAS. Furthermore, using GIS tools the contour map of study area is converted to DEM and TIN respectively. The detail of figure 4.13 and 3.14 shows completed prepared data before exporting to Hec-RAS.

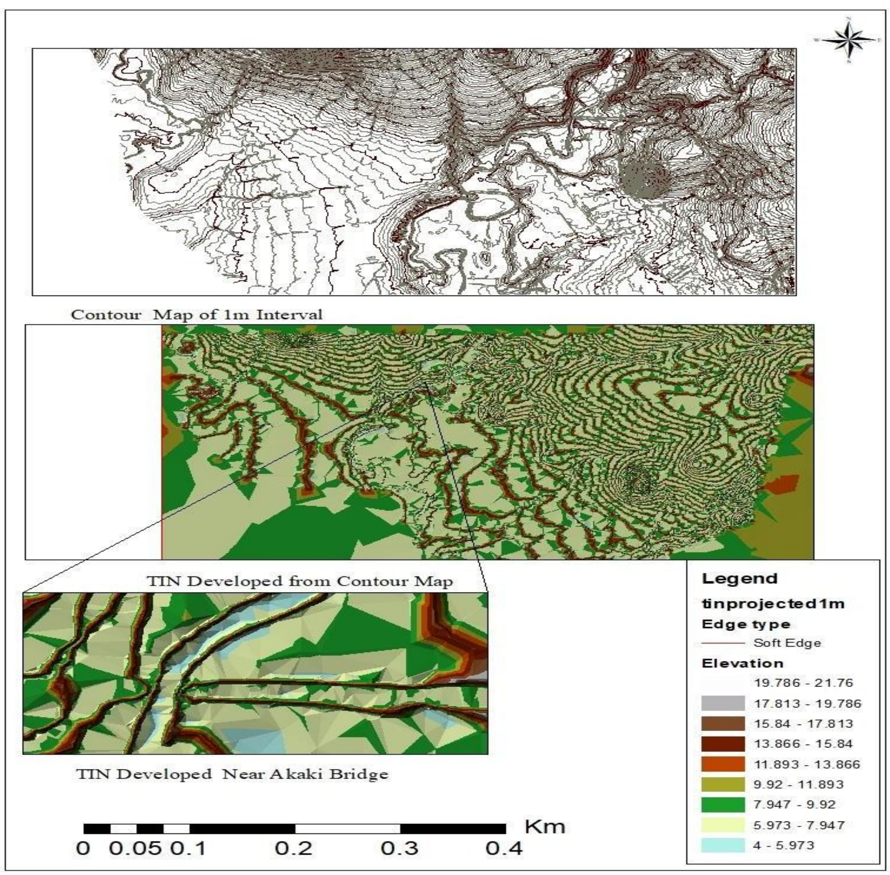


Figure 3-24 Contour map of 1m interval, DEM and TIN of Akaki river

3.8.1.1 Model Extend and DEM Selection

To creation the model geometry it defined the fraction of Akaki river that is relevant for the flood inundation analysis and to choose the topographic dataset for the model development. The topographic dataset is selected as it is shown in figure 4.12. Most frequently the flood risk occurred for the area at a point where bank overtopping and the model had to begin sufficiently upstream of the Akaki bridge extending to upstream of Aba Samuel reservoir. The total reach length considered in this study cover for an extension of 9103m. The pre process of creating model include contour map converted into DEM and generating TIN data to digitizing the river centerline, riverbank, cross-section cut line and flow path of the river.

3.8.1.2 Model Geometry based on TINs

The Cross sections are the essential inputs for the HEC-RAS model. They contain much information, including data on the elevation of the main channel and over banks, bank stations, and the downstream reach lengths. Based on the resulting layers, 332 cross-section, extraction of elevation from from DEM at specified cross-sectional and cross-sectional properties are defined based on points of intersection between the digitized layers. The resulting data is automatically stored in a RAS-GIS export file, which is then imported to HEC-RAS. Although, the location of the bank points for each cross-section can be easily adapted in HEC-RAS, the placement of the cross-sectional cut lines in HEC-GeoRAS is a crucial aspect in the development of the geometric model. Akaki River illustrated in figure 4.14

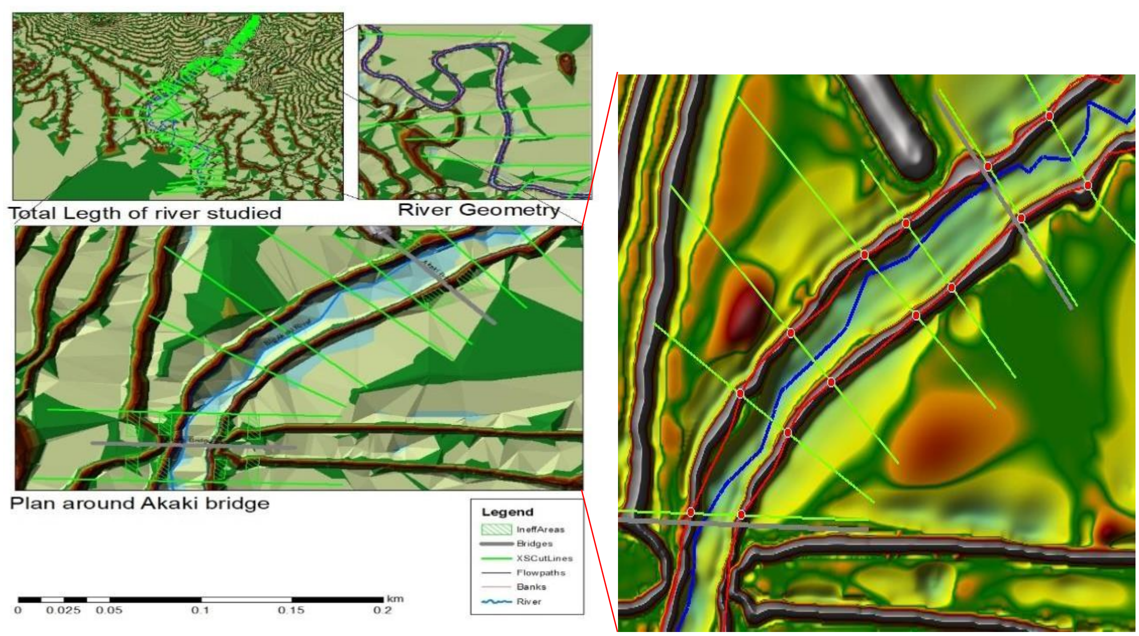


Figure 3-25 The river centerline, flow path, riverbank and bridge location on TIN

Cross section spacing and extend since the cross-sections built the computational nodes for the water surface profile calculations, their configuration as defined in HEC-GeoRAS directly influences the modeling results. As the (Heimhuber, 2013) sited the HEC-RAS Hydraulic

Reference Manual provides the following general guidelines for the placement of the cross-sectional cut lines along a reach (HEC, 2010b). Cross-sections are required at representative locations along the modeled reach and at locations where changes occur in discharge, slope, shape or roughness.

Cross-sections should span across the flow channel and the entire floodplains. At locations where abrupt changes occur in the channel geometry, cross-sections should be placed close enough to describe the changes accurately. Cross-sections should be placed perpendicular to the expected flow paths in the channel as well as in the left and right overbank.

In the modeling process, each cross-section is assumed to be representative for the reach geometry halfway to the next up and downstream cross-section. The appropriate spacing of the cross-sectional cut lines along the modeled reach depends on the channel size and slope as well as on the uniformity of the cross-section shape. Since the spacing of the cross-sections directly influences the stability and accuracy of the modeling solution average spacing of 50m is considered in this river analysis.

As it is shown in figure 4.14 the channel reach lengths are usually measured along the stream thalweg, but they should be measured along a line through the estimated center of mass of the flow if that line differs materially from the stream thalweg. In many cases, computed and estimated n values for overbank and channel flows are based on the same reach lengths for the overbank and channel areas. Defining the channel length based on the low flow channel course assumes that the flow will always follow the channel, even for flood flows. The cross-sectional representation at Akaki Bridge is shown in figure 4.15.

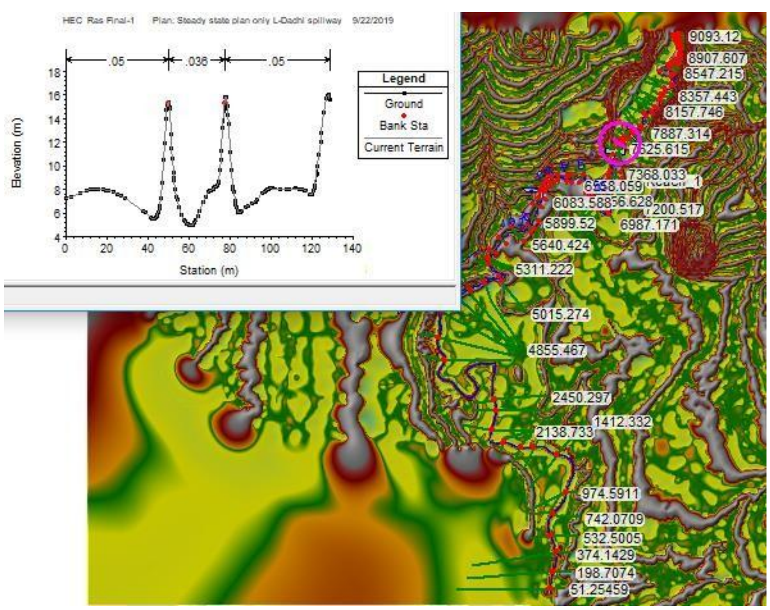


Figure 3-26 Cross section number 7625.14 at new constructed Akaki bridge located 50m upstream of bridge pier

Overbank reach lengths: If overbank flow follows a considerably shorter path than the main channel flow as in the case of a stream that meanders through the floodplain (in other cases it may be longer), and if computed or estimated n values used in the water surface profile computations do not include the effects of over- bank reach lengths, then separate lengths should be measured for overbank and channel areas. Furthermore, from figure 4.14 the green line displays the x-section cut line while blue line, red line, light blue line, pink line display river centerline, path flow line, and riverbank and bridge respectively. In this river model, the three bridge on Akaki River at Turunesh Hospital the, old a bridge, on Construction Bridge and the bridge around new jail is considered. Then after all this process is excited the Hec-geoRas data file is exported to Hec-Ras for the further process of river analysis as discussed in the following section.

3.8.1.3 Manning

Manning's roughness coefficient represents the resistance to flow in the channels and floodplains. The Manning's n-value depends on a number of factors, which include surface roughness, vegetation, and channel irregularities, degree of meander, obstructions, and size and shape of the channel (Sun, et al. 2014). The n values for this Akaki river length is assumed as 0.036 for the middle reach length while for the overbank it is extracted from the land use land cover prepared in the figure 3.10 and the river bank n values are assigned as 0.1, 0.07 and 0.12 for developed medium intensity, cultivated crops, and mixed forest respectively.

3.8.1.4 Expansion and Contraction Coefficients

The changes in cross-section geometry along the reach leads for the energy loss, for this reason, hydraulic computation loss coefficients are applied. This loss resulting from contraction and expansion of flow. The energy loss caused by a transition in channel geometry is calculated by multiplying these coefficients by the absolute difference in velocity head between one cross-section and the next downstream cross-section (HEC, 2010). In this study (G. Brunner, 2010) suggests the HEC- HMS manual for steady-state use values of 0.1 (contraction) and 0.3 (expansion). Based on low simulation runs, the flow in the Akaki River is found to be a subcritical flow regime. Before the beginning of the artificial flow channel, the cross-sections were assigned with a contraction 0.1 and expansion 0.3 is considered.

3.8.1.5 Hydraulic Structure Data Input

Since hydraulic structures such as bridges and culverts can cause energy loss and attenuate hydrographs, it is important to include their effects in the HEC-RAS model. Three bridges incorporated into HEC-RAS model and created two cross-sections to model each hydraulic structure. The one cross-sections include downstream cross-section where the flow has fully expanded, one cross-sections that are located just upstream of the bridge/culvert, and an upstream cross-section before flow starts to contract. Generally, the bridge cross-sections and the upstream and downstream cross-sections should be determined by field investigation. The bridge dimension used in the model is shown in figure 4.16

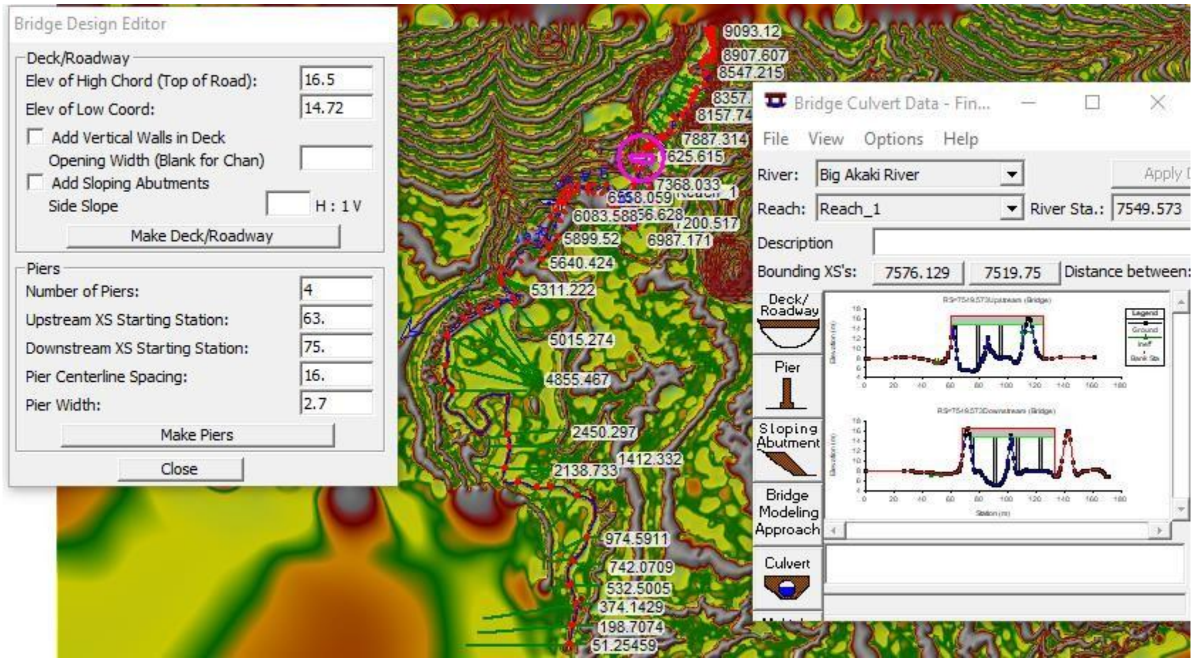


Figure 3-27 Typical bridge geometry in the HEC-RAS model for on construction of Akaki Bridge located around Turunesh Hospital

3.8.1.6 Equations for Basic Profile Calculations

Water surface profiles are computed from one cross-section to the next by solving the Energy equation with an iterative procedure called the standard step method(G. Brunner, 2010). The energy equation is written as follows:

$$Z_2 + Y_2 + \frac{a_2 V_2^2}{2g} = Z_1 + Y_1 + \frac{a_1 V_1^2}{2g} + h_e \qquad \textit{Equation 3-3}$$

Where: Z_1, Z_2 = elevation of the main channel inverts

Y_1 ,Y_2= depth of water at cross-sections

V_1 ,V_2 = average velocities (total discharge/ total flow area)

a_1 , a_2= velocity weighting coefficients

g=gravitational acceleration

h_e =enery head loss

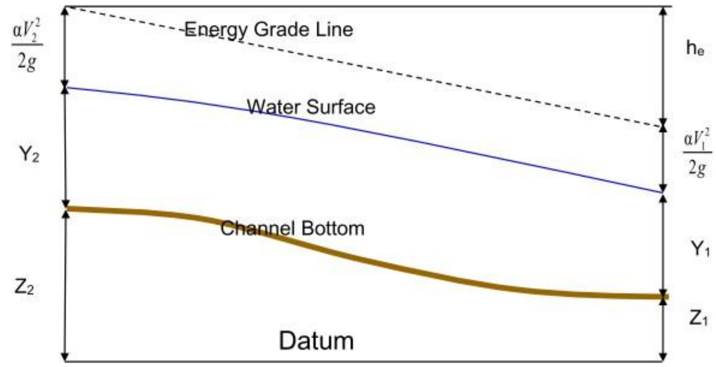


Figure 3-28 Representation term of energy equation (G. Brunner, 2010)

The energy head loss h between two cross-sections is comprised of friction losses and h_e contraction or expansion losses. The equation for the energy head loss is as follows:

$$h_e = L\overline{S}_f + C\left|\frac{a_2V_2^2}{2g} - \frac{a_1V_1^2}{2g}\right| \qquad \textit{Equation 3-4}$$

Where: L = discharge weighted reach length

$\overline{s_f}$ = representative friction slope between two sections

C = expansion or contraction loss coefficient

The distance weighted reach length, L, is calculated as:

$$L = \frac{\overline{Q}_{lob} + \overline{Q}_{ch} + \overline{Q}_{rob}}{Q_{lob} + \overline{Q}_{ch} + Q_{rob}} \qquad \text{Equation } 3-5$$

Where: L_{lob}, L_{ch}, L_{rob}= cross-section reach lengths specified for flow in the left overbank, main channel, and right over bank, respectively

$\overline{Q}_{lob}$, $\overline{Q}_{ch}$, $\overline{Q}_{rob}$ = arithmetic average of the flows between sections for the left overbank, main channel, and right over bank, respectively.

3.8.1.7 Flow Data and Boundary Conditions

The four profiles to be calculated; with the peak flow data for river reach and every profile flow data used is shown in table 4.5.

Table 3-7 Flow data used in Hec-Ras

Return period	25 years	50 years	100 years	1000 years
Q peak	277.459	362.207	462.898	946.851

As described in Section 2.3.3.2.2, boundary conditions for steady-flow simulations need to be defined for the end reach of the model. since the flood hydrograph at the downstream end of the modeled reach is unknown, as it is the case in this study, the normal depth can be used as the downstream boundary condition. Hereby, the water level at the last cross-section is computed for a user-entered friction slope by using Manning´s equation. Since the friction slope is often unknown, it is approximated as the channel bed slope in the area of the last cross-section. The last cross-section average slope is calculated slope of 0.0092.

3.8.1.8 Application and Limitations of HEC-RAS Software

HEC-RAS is the powerful software for the study of river analysis, floodplain mapping,1-D and 2-D flow model in the stream flow, and therefore in this study the flood plain mapping of peak flood for return period of 25, 50, 100 and 1000 years is determined.

The software is selected primarily due to its widespread recognition as a stable and practical flow model, its relative ease of use and the very complete and practical documentation of the model. According to Hu and Walton (2008), HEC-RAS is probably the most widely used 1-D flow model in the world. In both government agencies and private firms in the U.S. and the rest of the world, HEC-RAS is being used extensively for more than 20 years (Brunner, et al., 2010). Due to its widespread use in practical river engineering, the accuracy as well as the limitations of the model have been analyzed in various studies and are well documented in literature (HEC, 2010). Nevertheless, the accuracy of the results highly depends on the experience of the modeler and the general suitability of the model to the given circumstances (G. Brunner, 2010).

The limitation of this software is due to the assumption of the flow regime condition assumed as 1-D flow to calculate the water surface profile. In reality, this condition may not occur for the natural water flow in the stream.

3.8.2 Model summary in HEC-RAS

To perform the hydraulic modeling all the created model geometry files with HEC-GeoRAS which is the interface of RAS-GIS. Then the export file is imported into HEC-RAS, in which the hydraulic modeling be analyzed. Since the accuracy of the DEM generated from the topo map (contour) data is found to be limited for an accurate description of the channel geometry, the extracted cross-sections were adjusted manually in HEC-RAS. This adjustment is only for some cross-section for the location of the river bank only. These areas of adjustment can be seen in Figure 4.14. The completion of the model in HEC-RAS further comprised the assignment of roughness, expansion and contraction coefficients to each cross-section as well as the definition of the computational time step and the boundary conditions for steady-flow calculations.

3.9 Summary of Methodology used in this study

Rainfall-runoff modeling is carried out with the help of HEC-HMS and HEC-GeoHMS which is a hydrological extension in ArcGIS. Furthermore, Hydraulic analysis of the river system is performed with the help of HEC-RAS along with HEC-GeoRAS, an extension tool in ArcGIS. The schematic representation of both software is shown in figure 4.18.

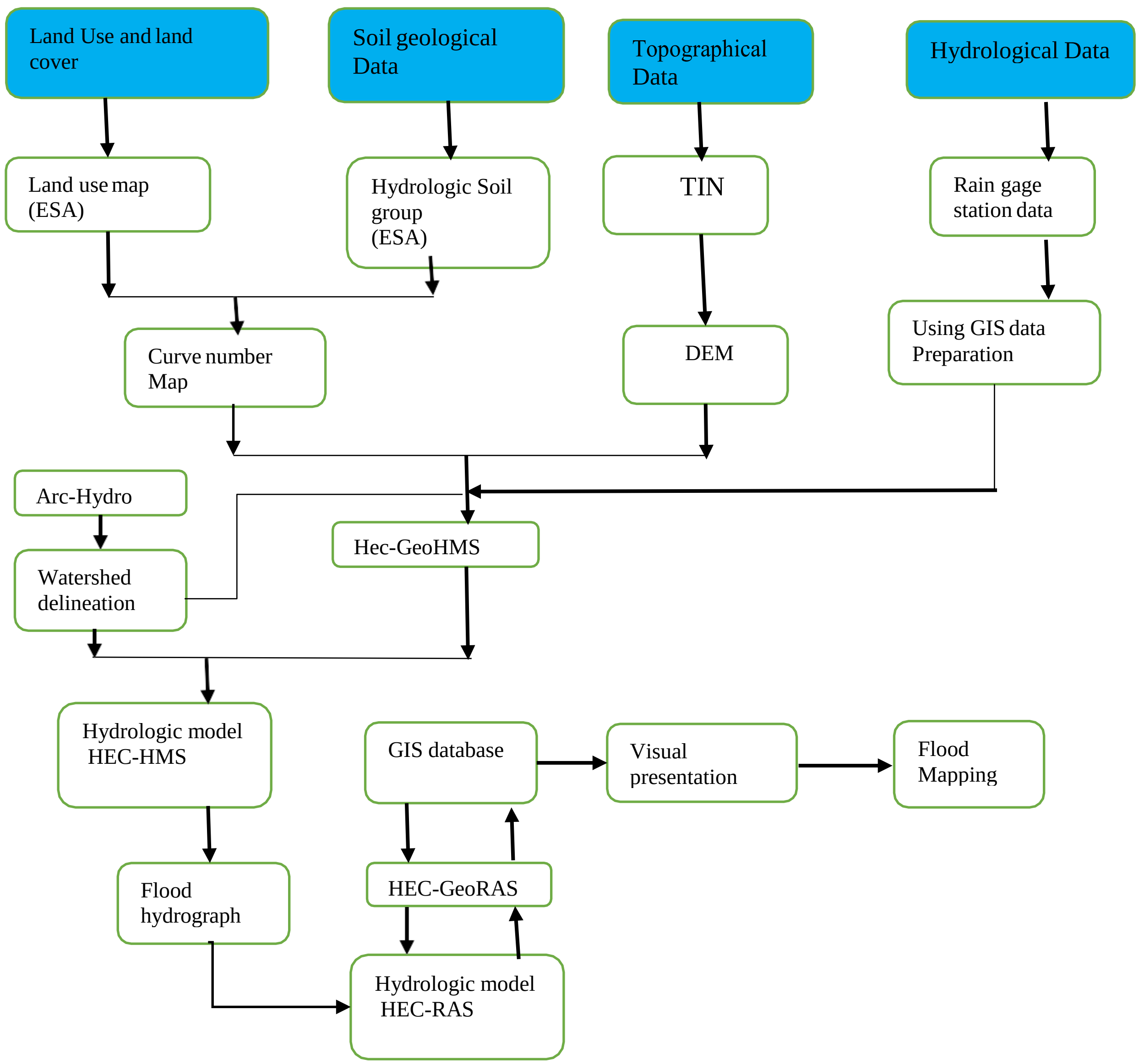


Figure 3-29 Modelling approach Rainfall-Runoff Modelling and Flood Inundation Mapping

4 RESULTS AND DISCUSSION

4.1 Sensitivity Analysis to select the parameter of hydrologic model

In HEC-HMS, the systematic search for the best (optimal) parameter values for loss, transform, base flow and routing is calculated with a different method. The calculated value compared with each value and the sensitive parameter is selected. Accordingly, for loss in all subbasin initial and constant method is selected; for the transform of precipitation into direct runoff by Clark unit hydrograph is considered for selection; for base flow, recession method is selected and similarly for routing of the reach analysis Muskingum method were selected. when performing sensitivity analysis we adjust the initial value for one type of parameter by considering the other parameter constant and run the program then evaluate the result using the unit hydrograph at the outlet point of watershed.

4.2 Calibration and Validation

4.2.1 Calibration for Bib Akaki River Basin

Figures 5.1 and 5.2 are the graphs that compare observed flow to simulated flow for the calibrated years, 1998 to 2004. The black dotted lines denote observed outflow measured at the gauge station, at the respective outlet. The blue solid line denotes the total simulated outflow at that outlet and the blue dashed line denotes the outflow from the upstream reach of the junction.

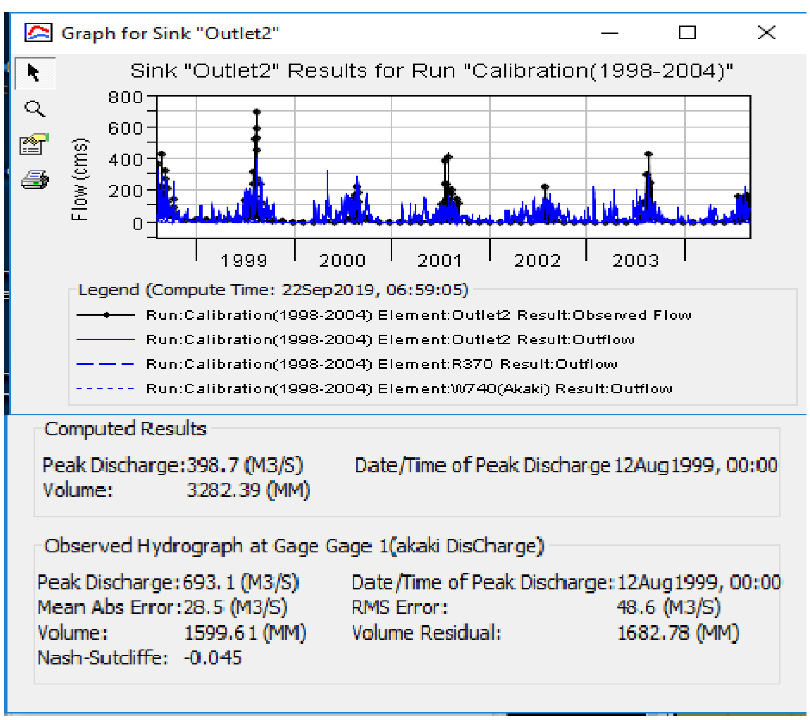


Figure 4-1 Observed and Simulated flow hydrograph at big Akaki river basin and model evaluation result in the calibration process

The calibrations at the outlet of watershed basin result indicate simulated outflow volume is larger than the observed volume. The simulated peak value 57.53% underestimated than observed peak value at the outlet point. But the time to peak 12 August 1999 is similar to the time to peak of observed and simulated. Furthermore, according to the result seen in figure 5.1, the model generate peak flow in dry season (December, January, February,) for year of 1999, 2003 and 2004 this failure may be due to ambiguity in the precipitation data. Apart from this, some important meteorological parameters such as wind speed, relative humidity, and air pressure have not been included in the model whereas wind plays an important role in the hydrological cycle. The computational time step for simulation is set for 1(one) day therefore, the computed outflow data were also for 1(one) day time interval. Similarly, the precipitation and discharge data were daily averaged data. This time interval of input data and the output also created some level of error. In this study, since the model is mainly calibrated by considering the outflow peak value and Nash-Sutcliffe, the calibrated flow pattern poorly represented as it is compared to observed flow at the outlet (sink) as it is seen in Figure 5.1. As it is discussed in section 4.7.1 the Nash-Sutcliffe result -0.045 shows that the correlation between observed and simulated clearly very poor. In the process of calibration different parameter is changed until the satisfactory Nash-Sutcliffe value obtained. The acceptable and good result is reached with the calibrated parameter value listed in table 5.1. This parameter is calibrated using the criteria listed in table 4.4 and according to the process of calibration shown in figure 4.11. Therefore, the acceptable calibrated parameter values were summarized in table 5.1.

Table 4-1 The initial and calibrated values for the watershed of Big Akaki

Initial and constant Loss	**Initial Value for all Akaki sub-watershed**	**Calibrated Value for all Akaki Sub -watershed**
Initial abstraction	0.1	16.04
constant rate	0.1	0.1847
Base flow		
Initial discharge	0.1	8.5033
Recession constant	0.01	0.71
Peak to ration	0.1	0.35
Transform		
Time of concentration (Tc)	0.1	38.87 It average value of all sub basin
Storage coefficient	0	
Muskingum K		
x	0	0.35
k	0.1	The value vary based on Velocity of the Channel Reach
Curve Number	Values extracted from GIS	Values Vary based on antecedent moisture
Impervious area	Values extracted from GIS	The value vary based for Each sub basin

In the calibration process, the initial and constant value starting from 0.1 up to 16.04 is considered until the value isn't changed on the output value. The optimal value is obtained 16.04 for each sub-basin and this can be calibrated by considering other parameters were constant. Similarly, for the remaining parameters like base flow, transformation method and hydraulic routing for Muskingum is calibrated one by one separately. The curve number is used to determine initial abstraction and it is not calibrated in this study. After calibration completed for all parameter the final value listed in table 5.1 are used in the model to generate runoff hydrograph for all sub-watersheds. The HEC-HMS final result that is used for flood mapping is shown in figure 5.2 and table 4.2.

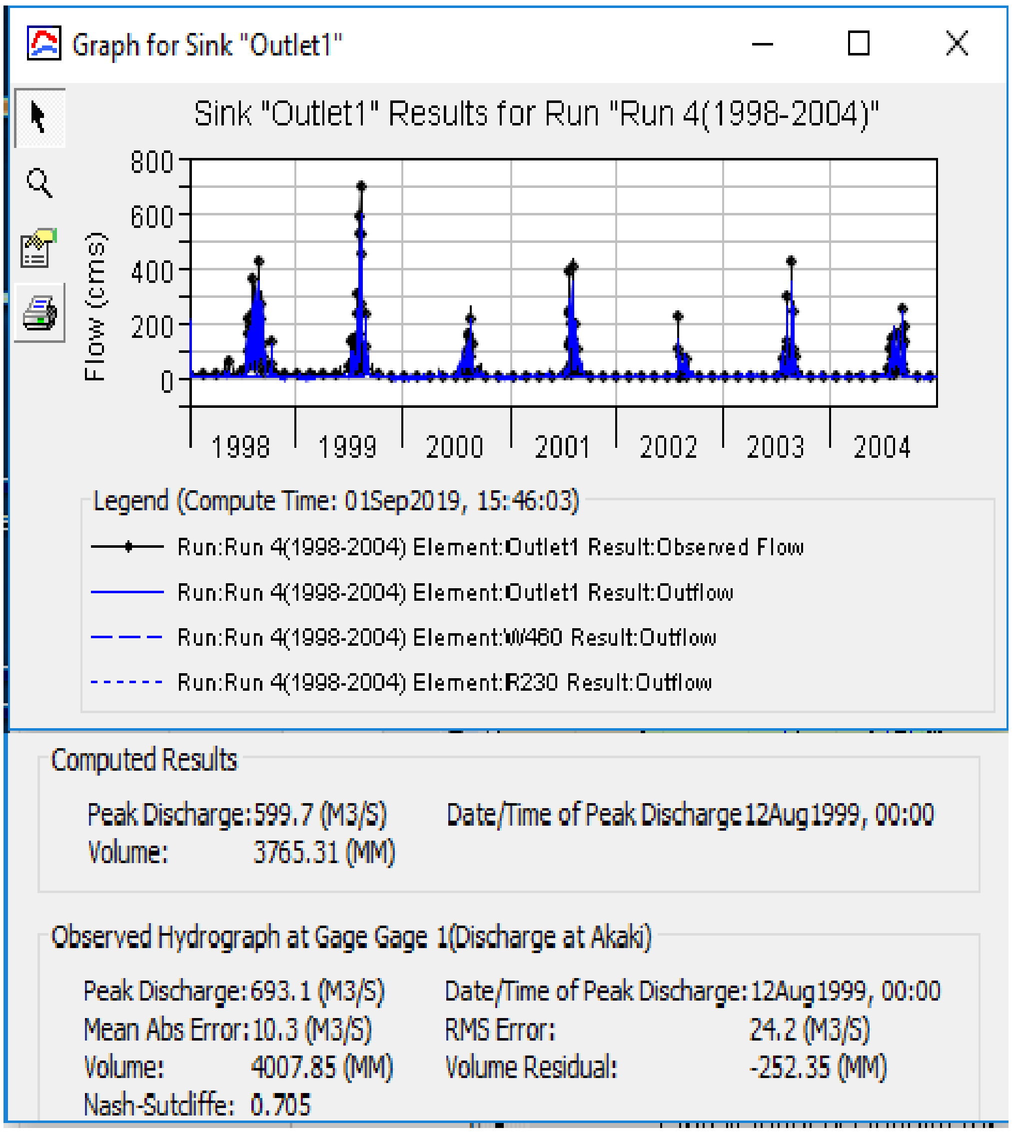
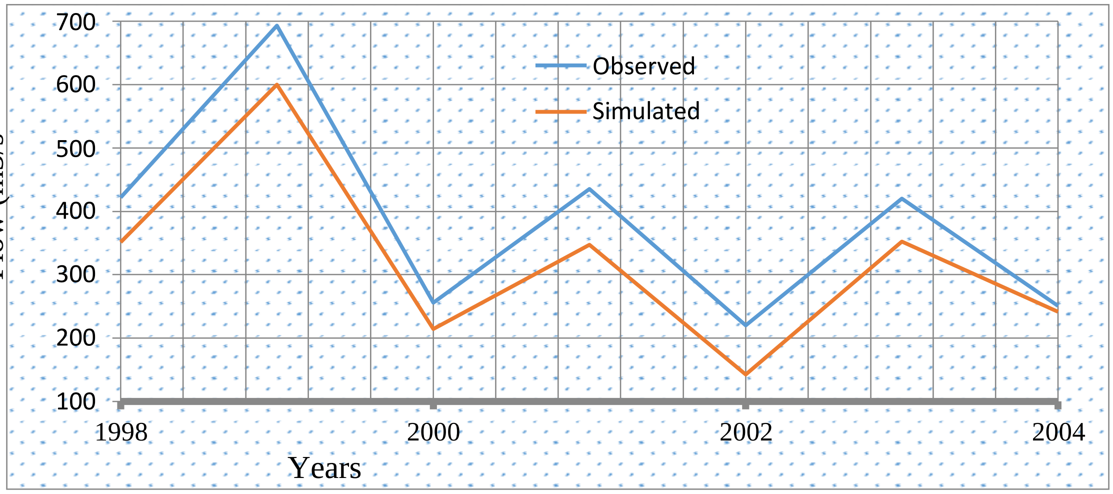


Figure 4-2 Observed and Simulated flow hydrograph at big Akaki river basin and model evaluation result for final

Table 4-2 The HEC-HMS result obtained by considering no release of flow from dam



As discussed in the methodology section for evaluation of model using Nash-Sutcliffe the accepted range is 0-1 if the value approaches 1 the simulated output result is very good. As it is noted by (Misganaw, et al.2016) Nash-Sutcliffe Efficiency (ENS) value ranges between -∞ and 1.0 (1 inclusive), with NSE =1 being the optimal value. Values between 0.0 and 1.0 are generally viewed as acceptable levels of performance, whereas values <0.0 indicates that the mean observed value is a better predictor than the simulated value, which indicates unacceptable performance. Furthermore, the value of coefficient determination R^2 ranges between 0 and 1, this estimate the correlation between simulated and observed values. The value of zero means no correlation at all and one means best fit between observed and simulated values. Generally, values greater than 0.5 are considered acceptable (Khadka & Bhaukajee, et al., 2018).In this study the NSE and R^2 of the model 0.705 and 0.70 respectively. This value falls within the acceptable ranges and the summary of peak discharge and volume is indicated in table 5.2. The flood map is done based on different result obtained from HEC-HMS by setting three scenarios. These scenarios were without considering flow release from lega Dadi reservoir only, Dire reservoir release only, and both spillway released at one time. The result of HEC-HMS for the case no release of flow rom dam and release of flow both dam is tabulated in table 4.3 and table 4.4

Table 4-3 Result summary of observed and simulated without considering discharge released from 'Lega Dadi and Dire Dam spillways

Description	Observed flow value at Akaki Bridge	Simulated flowvalue	Difference	Remark
Peak Discharge(m3/s)	693.1	599.7	93.4	Underestimated
Volume(MM)	4007.85	3765.31	242.54	Underestimated
Time to peak	8/12/1999	8/12/1999	-	Match

Table 4-4 The HEC-HMS result obtained by considering release of flow from both dam

The difference in peak flow and volume shown in table 5.2 indicates that it is an additional source of flow into Akaki River. The Big Akaki River is gauged near Trunesh hospital. The station is equipped with an automatic water level recorder and capable of discharge measurements. This gage record all flow from sewerage as well as surface flow of the catchment. Therefore, in the study area, significant contribution from sewage that passed through the drainage system into the nearby streams and release of lega Dadi, Dire reservoir leads to the variation of peak flow and volume. (UNEP,UNESCO,UN-HABITAT, 2003) study by AAWSA noted that from total water supplied to Addis Ababa about 70% returns as sewage and 60% of the returned flow has an outlet through big Akaki river and the remaining 40% join Little Akaki river. The average supply of water for Addis Ababa from surface reservoir and ground water abstraction is about 163,000 m^3/d. Thus, the contribution of sewage to the runoff in big Akaki and Little Akaki river is 0.79 m^3/s and 0.53 m^3/s respectively. Additional simulation is done by considering the release discharge from both reservoirs and separately as it is indicated in table 5.3. At the time of the dam become full, spillway and bottom outlet of Lega Dadi which equipped with three opening spillways and two opening of bottom outlet release flow for 30minutes. The amount of flow released from both spillway of Dire and lega Dadi is 39 m^3/d and 40 m^3/d respectively. The bottom outlet with a diameter of 900mm also release about 6.4m3/s and from treatment plant continuously 0.079 m^3/s flow discharge is released. Generally, the output result from HEC-HMS used in the flood mapping of big Akaki River is summarized in table 5.3.

Table 4-5 Flow result of HEC-HMS with return period estimated using mathematical analysis

Return period in Years	Without Considering Discharge released from 'Lega Dadi and Dire Dam		By considering Discharge released from both 'Lega Dadi and Dire Dam		By considering Discharge released from Dire Dam only		By considering Discharge released from 'Lega Dadi Dam only	
	Log Pearson III	EV2-Max (L-Momme nts)	Log Pearson III	EV2-Max (L-Momme nts)	Log Pearson III	EV2-Max (L-Momme nts)	Log Pearson III	EV2-Max (L-Momme nts)
25	277.459	263.415	354.046	350.558	312.321	309.825	313.337	310.885
50	362.207	376.408	429.149	429.122	391.789	401.402	392.667	247.015
100	462.898	536.455	516.969	524.509	486.533	519.055	487.202	518.968
1000	946.851	1729.74	953.104	1018.12	932.369	1213.63	952.312	1206.29

As it is discussed in section 4.4 the value listed the peak discharge using gamma family Log Pearson III which is best fit to the Weibull probability distribution of the sample for the return period of 25, 50, 100, and 1000 years event is, 277.459 m^3/s, 362.207 m^3/s, 462.898 m^3/s, and 946.651 m^3/s is determined respectively without considering discharge released from 'Lega Dadi and Dire Dam. Furthermore, the inundation mapping is evaluated using Log Pearson III for three scenarios stated in the following figure.

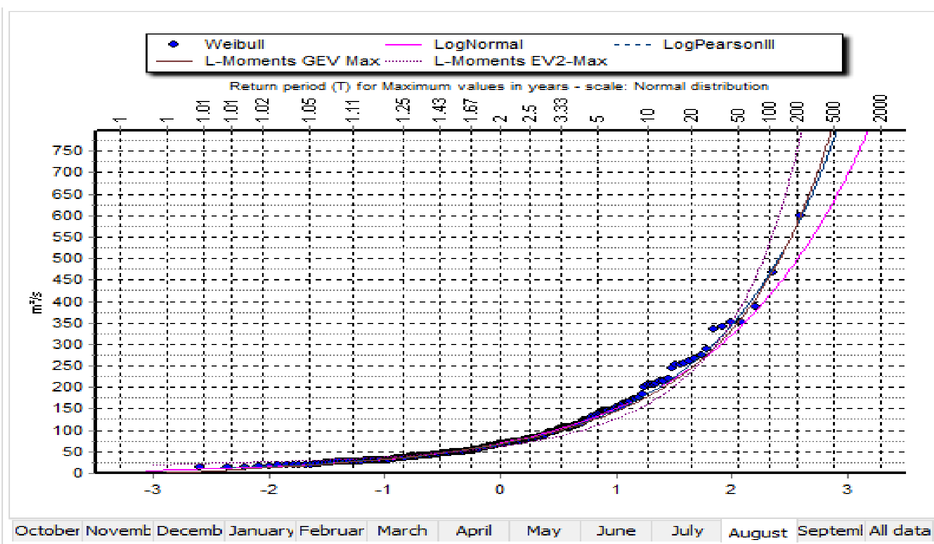


Figure 4-3Return period for maximum values 2000 years

4.2.2 Validation for Big Akaki River Basin

To verify the output of this model, validation is also performed at discharge stations from the year 2005- 2008. This is performed to check if the models were able to predict the runoff at the discharge stations for the period other than calibrated one or not. The resulting graphs for the validation period are shown in the figures 4.4 and table 4.6 below.

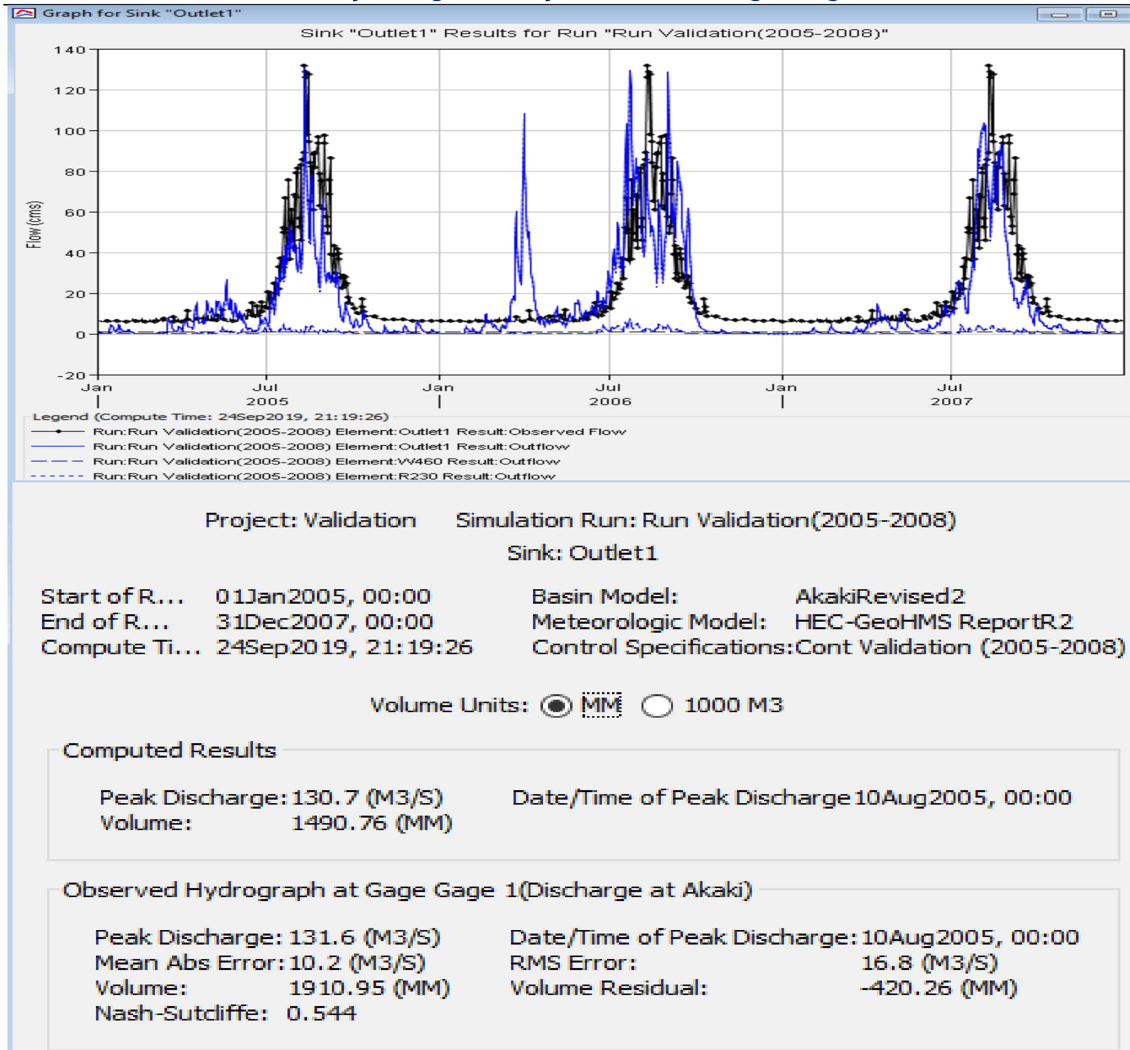
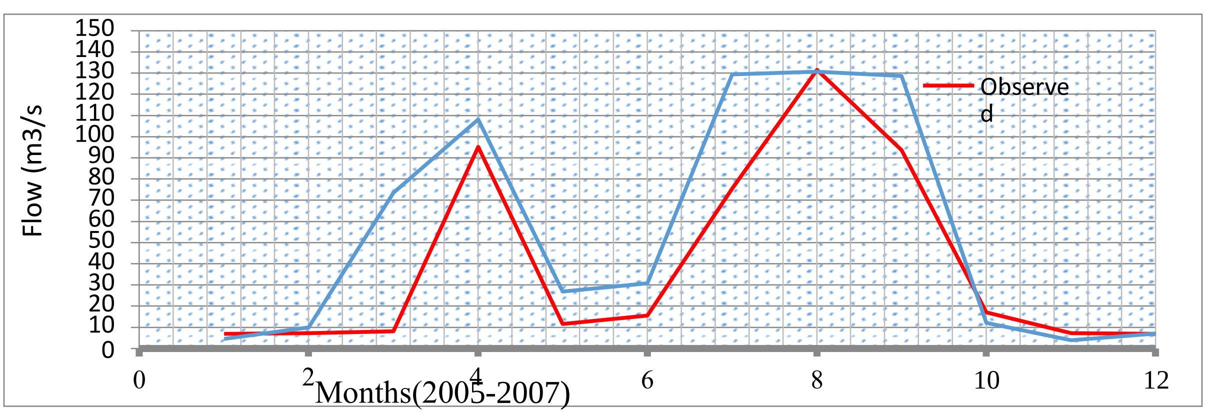


Figure 4-4 HEC-HMS Result for Simulated and observed values in the validated period between 2005 and 2008 years

Table 4-6 Simulated and observed values in the validated period between 2005 and 2007 years



As it can be seen from the figures above, the pattern of simulated and observed flow is almost identical to each year except in February 2006 is much higher than the observed value. The difference is due to the discharge from the sewerage line connected to the river. At the outlet point of the Akaki river basin the peak value error of 0.3 %.and the maximum volume error is about 22 % during validation. But the time to peak for both simulated and observed is on 10 August 2005 which indicates that the time to peak good fit. In the figure 5.4, the black dotted line denotes observed value and the blue line denotes the simulated flow line. The accuracy of the prediction of Nash-Sutcliffe is 0.544 which is fairly satisfactory.

4.3 The result of Watershed Characteristics Across Addis Ababa watershed basin

As stated in section 4.2 the total characteristic across the city is delineated by selecting the outlet at downstream of Aba Samuel reservoir. The total basin area, the main river, and all tributaries river length, area of each sub-basin, area of reservoir within the basin, slope and elevation range of the basin were summarized in table 5.4

Table 4-7 Summary of Watershed characteristics across the Aba Samuel watershed

Description	Length(km)	Area(km2)	Slope range(%)	Elevation Range(m)
Main Akaki River	71.55		0.00028 Up to 0.0173	2045 Up to 2386
Little Akaki River	47.55		0.00014 Up to 0.0195	2036 Up to 2552
Kabana River	20.51		0.01606 Up to 0.0434	2125 Up to 2549
All tributary in Abasamuel Watershed	316.4		-0.521 Up to 0.043	2036 Up to 2671
Akaki River Sub_basin		873.83	0 Up to 73.75	2010 Up to 3248
little Akaki River Sub_basin		804.46	0 Up to 165.404	2011 Up to 3368
Kabana River Sub_basin		84.15	0 Up to 138.28	2098 Up to 3198
Abasamuel Reservoir		17.11	0 Up to 48.08	2010 Up to 2072
Laga Dadhi Reservoir		4.051	0 Up to 129.574	2353 Up to 2578
Gafarsa Riservoir		1.23	0.5761 Up to 83.23	2529 Up to 2664
Dire Reservoir		1.15	0.5761 Up to 108.697	2464 Up to 2607
Total Sub_basin		1785.981	0 Up to 170	2010 Up to 3368

This table is prepared from the GIS result of figure 4.1 shown in section 4.2 and the total area of the watershed is 1785.981km^2. Similarly, the total length of big Akaki River is about 71.55km.The river length, area of sub-basin, area of reservoir, slope and elevation is calculated using GIS software.

4.4 Hydraulic / Flood Inundation Modelling Results

The flood values for different return periods were calculated using the maximum instantaneous flow from station of Akaki River located at Turunesh hospital (Beijing Hospital) which lies upstream of Akaki Bridge. There is only one discharge station through the Akaki river reach recording for a long time. Therefore, flow measured at this location is considered as a suitable

Representative of the outflow occurring in the basin. The inundation map of the big Akaki River due to the 100-year return flood is shown in Figure 5.5. The flow value for this 100-year flood has been obtained from the Log Pearson III distribution method which has been explained in Section 4.4. By considering discharge released from both ‘Lega Dadi and Dire Dam is about 516.969 square kilometers land is flooded. When two dams simultaneously release flow over top of the river bank station at the upstream and downstream of Akaki Bridge at Beijing Hospital including the lower basin area is inundated. The flow calculated for the return period of 25, 50,100 and 1000 is 354.046m^3/s, 429.149m^3/s, 516.969m^3/s, 963.109m^3/s respectively. According to the values described in table 5.3 flood mapping is developed for four scenarios. The flood over top bank station in case of all scenarios, but the magnitude over top depth and length of over top is varied for each return period. The highest value of water depth during the 100-year flood in this river is 10.1 m. The average depth of inundated flood, the inundated area and maximum water level given by HEC-RAS during 25, 50, 100 and 1000-year flood is tabulated in table 5.5

Table 4-8Area inundated and maximum water level in big Akaki River Basin during 25, 50, 100, 1000-year floods by considering flow release from Dire and ‘Lega Dadi Dam

	Upstream of Akaki Bridge(1390m)						
Return period	Q Total	W.S. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Depth
	(m3/s)	(m)	(m/m)	(m/s)	(m2)	(m)	(m)
25	354.05	13.42	0.00264	1.59	21299.18	130.24	1.35
50	429.15	14.06	0.00264	1.69	23760.63	132.24	1.62
100	516.97	14.75	0.00263	1.81	26470.08	134.75	1.92
1000	946.85	14.62	0.00187	1.73	110434.64	318.71	3.64
	Downstream of on construction Akaki Bridge (7713m)						
Return period	Q Total	W.S. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Depth
	(m3/s)	(m)	(m/m)	(m/s)	(m2)	(m)	(m)
25	354.05	10.97	0.0011	1.30	36504.65	297.82	1.74
50	429.15	11.32	0.0011	1.39	40294.70	300.00	2.01
100	516.97	11.70	0.0012	1.51	43348.90	304.51	2.21
1000	946.85	13.15	0.0013	1.8658	59850.20	313.70	3.71

The maximum flooding zone, depth of inundated flood overlay with the TIN data generated from contour map of the study area as shown in figure 5.5 for the return period of 25, 50, 100 and 1000

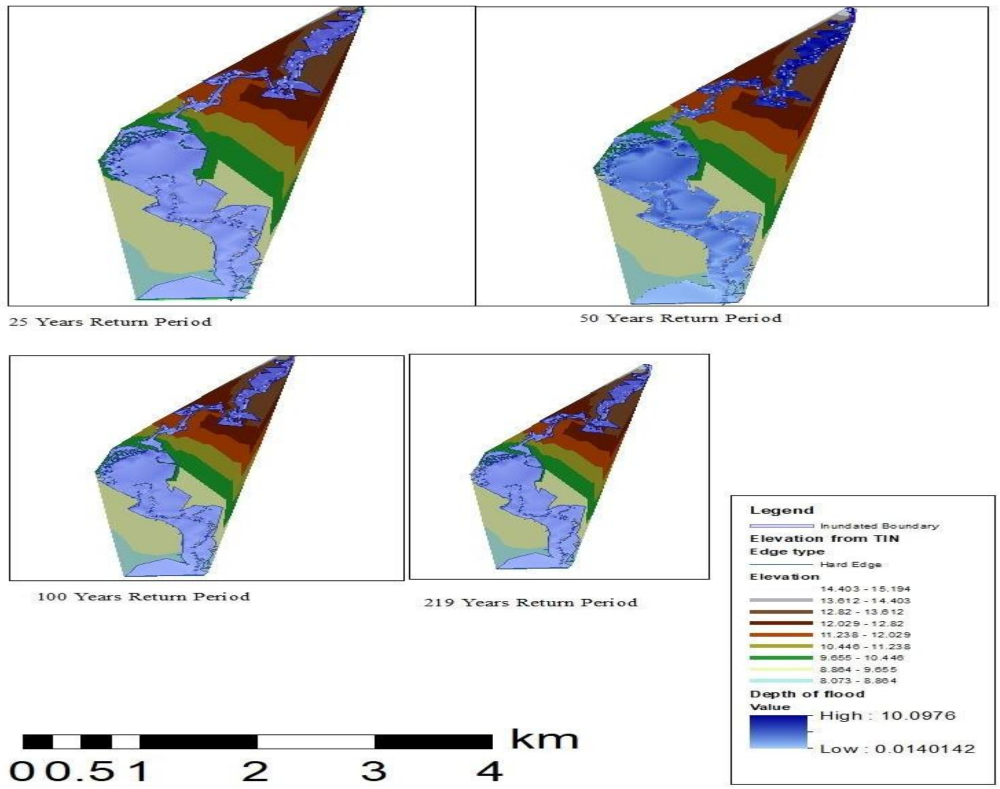


Figure 4-5 Flood Map when both dam released simultaneously

There are many settlement areas especially at the downstream of Akaki river basin. When both dam releases flow simultaneously for the return period of 25, 50, 100, 1000 years, the respective total area of 57803.83 m^2, 64055.33 m^2, 69818.98 m^2, and 170284.8m^2 lands near river is at the risk of flood. Furthermore, as it is illustrated in table 5.5 for the length of 1.39km from upstream of on construction Akaki Bridge may safer for the average flood depth of 1.96m for the 100 years peak flood in the basin. Therefore, at the time of peak maximum flood for the return period 25, 50,100 and 1000 years may cause large damage in this area. The on construction Jail around Galan condominium also affected by the peak maximum flood within the return period of 25, 50, 100 and 1000 having the average flood inundated depth of 1.97, 2.14,2.32 and 2.68 m respectively. Therefore, this area will need great attention for all floods coming within different return periods. Some major inundated areas in the basin have been marked with the red line as it indicated in Figure 5.6 and their detailed view with exposure areas to flood.

Table 4-9 Flooding area and overtop river cross section depth

Return Period	Without Considering Dam		By Considering Both Dam		By Considering only 'Lega Dadi Dam		By Considering only Dire Dam	
	Area(m2)	Average flood Depth (m)	Area(m2)	Average flood Depth (m)	Area(m2)	Average flood Depth (m)	Area(m2)	Average flood Depth (m)
25	51516.99	1.85	57803.83	1.54	54215.06	1.44	54215.06	1.43
50	59012.76	2.06	64055.33	1.81	61069.23	1.73	61069.23	1.74
100	67093.31	2.29	69818.98	2.07	68196.50	2.07	67599.83	2.02
1000	170284.84	4.78	100438.51	4.87	99863.51	4.83	99182.31	3.61

This table show flooding area and over toped flood depth for the return period of 25, 50,100 and 1000 by considering different case release of discharge from dam.

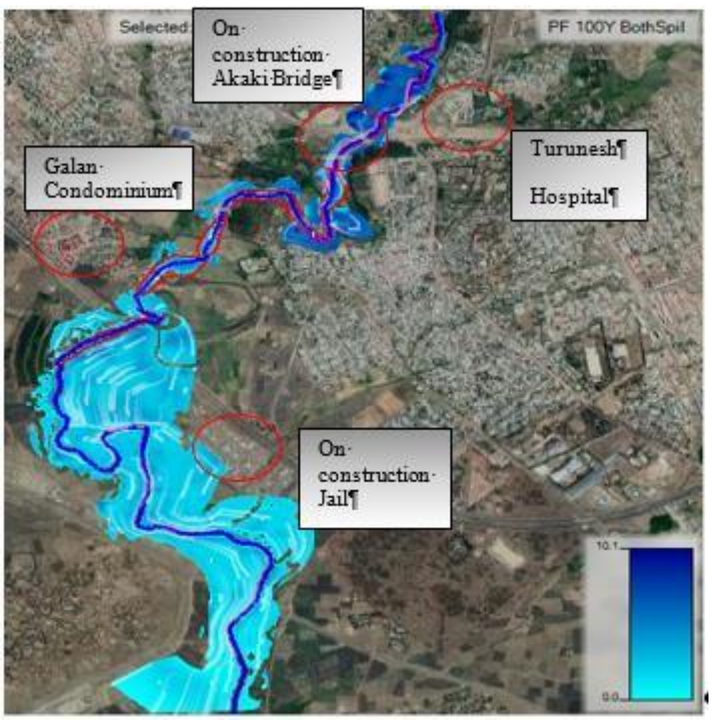


Figure 4-6 Flood mapping result by considering both 'Lega Dadi and Dire dam release simultaneously for 100 year return period

Some cross-sections of the river indicated in figure 5.7 assist to visualize the effect of flood in the banks. In addition to the 100-year flood level, 25 50 and 1000-year flood levels are also shown in the figures 5.8 to see how the water level varies with different flood values. The effect of flood on both banks of the river is almost equal in area. The geo-referenced view of flood map of big Akaki river basin at the specified river cross section is shown in in the following figure.

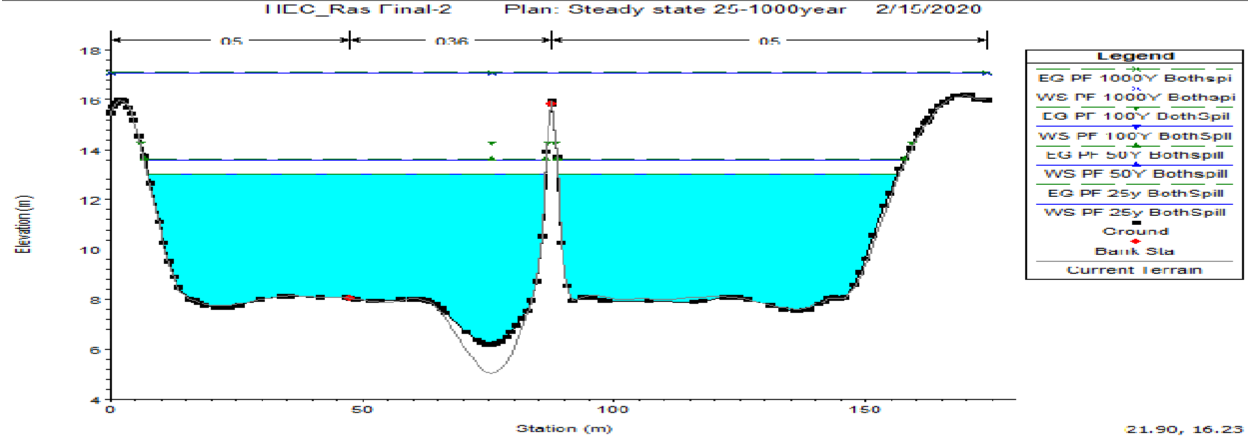


Figure 4-7 Maximum flood in return period 25, 50,100,1000-year with Water level at Akaki Bridge River at Location 115m downstream of bridge cross section number 7455.87

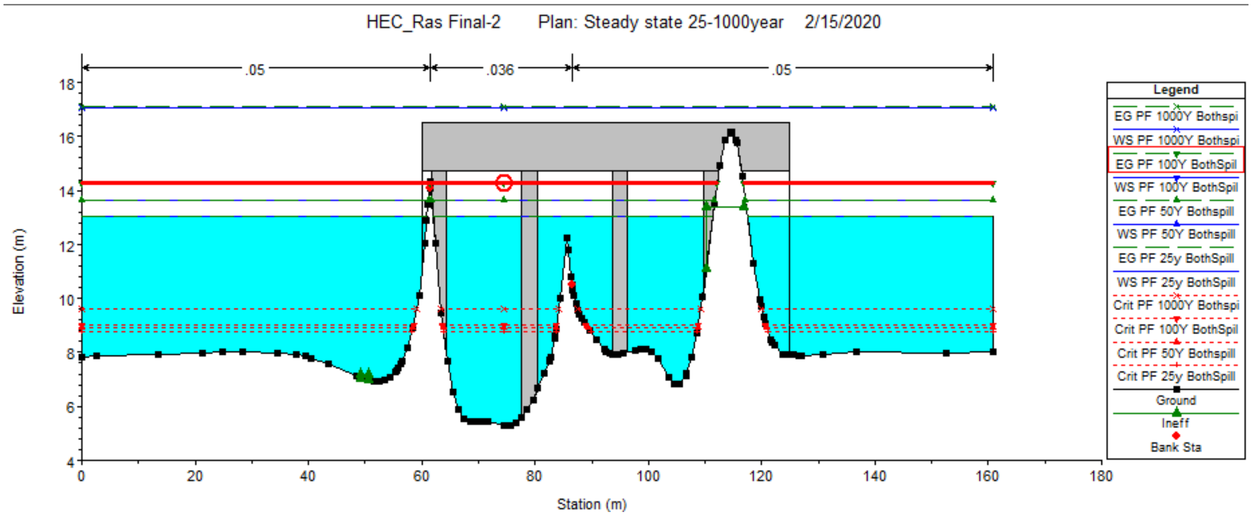


Figure 4-8 Maximum flood in return period 25, 50, 100, 1000-year with Water level at Akaki Bridge River at Location on construction bridge axis

According to the result shown in figure 5.8 if the peak maximum flood comes with the release of both Dire and ‘Lega Dadi dam simultaneously for the return period of 25, 50 and 100 years the bridge can pass the flood safely but for the case of 1000 years return period the bridge could not pass safely. Similarly, as it is displayed in figure 5.8 the right bank of the river section, the flood over top the bank for all return period. Figure 5.7 indicate the left-back of the river overtopped. The HEC-RAS result for three bridge tabulated in table 5.6 and the three bridge represent with cross-section number 7749.981 (Akaki old bridge at the upstream of on construction bridge near Turunesh hospital), cross-section 7549.573 (on construction bridge near Turunesh Hospital) and cross-section 6154.573 (bridge near Galan condominium on highway) is identified.

Table 4-10 Result of three bridge on big Akaki river two of them located near around Turunesh hospital and one cross highway near Galan condominium

HEC-RAS Plan: plan25-100yr River: Big Akaki River Reach: Reach_1

Reach	River Sta	Profile	E.G. US. (m)	Min El Prs (m)	BR Open Area (m2)	Prs O WS (m)	Q Total (m3/s)	Min El Weir Flow (m)	Q Weir (m3/s)	Delta EG (m)	E
Reach_1	7749.981	PF 25y BothSpill	13.05	14.80	153.34		354.05	5.93		0.01	
Reach_1	7749.981	PF 50Y Bothspill	13.67	14.80	153.34		429.15	5.93		0.01	
Reach_1	7749.981	PF 100Y BothSpil	14.33	14.80	153.34		516.97	5.93		0.01	
Reach_1	7749.981	PF 1000Y Bothspi	17.16	14.80	153.34		963.10	5.93		0.02	
Reach_1	7549.573	PF 25y BothSpill	13.03	14.72	335.50		354.05	6.93		0.00	
Reach_1	7549.573	PF 50Y Bothspill	13.64	14.72	335.50		429.15	6.93		0.00	
Reach_1	7549.573	PF 100Y BothSpil	14.30	14.72	335.50		516.97	6.93		0.00	
Reach_1	7549.573	PF 1000Y Bothspi	17.11	14.72	335.50		963.10	6.93		0.01	
Reach_1	6154.782	PF 25y BothSpill	11.96	17.00	512.77		354.05	7.67		0.00	
Reach_1	6154.782	PF 50Y Bothspill	12.43	17.00	512.77		429.15	7.67		0.00	
Reach_1	6154.782	PF 100Y BothSpil	12.93	17.00	512.77		516.97	7.67		0.00	
Reach_1	6154.782	PF 1000Y Bothspi	15.10	17.00	512.77		963.10	7.67		0.00	

4.5 Flood Inundation Modelling Results comparison four scenarios

The potential flood extent maps in this research have been presented for the case of flood inundations for peak flow with no release of flow from ‘Lega Dadi and Dire dam, simultaneous release of flow from both dam, the flow released from only ‘Lega Dadi dam and the flow released from Dire dam only is analyzed. Therefore, the important issues with regard to flood mapping like flood depth and flood width (extent) which are found from the HEC-RAS result is tabulated in table 5.7. The flooded prone areas are identified for the 25, 50, 100 and 1000-year flood frequency storm. Based on this study, the number of flood prone cross-section located on Akaki River having reach length of 9103m and total 332 cross-sections is identified. The number of cross-section affected by flood during the flow released from both dams, only ‘Lega Dadi, only Dire dam and no release from the dam are 200,193,192, 190 respectively. Over toped number of cross section is obtained by counting from flood inundation map. The maximum flood average depth along the flooding area, the maximum flood extent (width) and the flooded area for 25, 50, 100, and 1000 year return periods tabulated in table 5.7. Generally, the maximum flood prone area by considering both dam releases are 0.0578km^2, 0.0641km^{2,} 0.0698km^2, 0.1km^2 respectively.

Table 4-11 The flood inundation comparison for three scenarios summarized fro table annex A-4 up to A-7

Without Considering Dam		By Considering Both Dam		By Considering only 'Lega Dadi Dam		By Considering only Dire Dam	
Area(m2)	Average flood Depth (m)	Area(m2)	Average flood Depth (m)	Area(m2)	Average flood Depth (m)	Area(m2)	Average flood Depth (m)
51517.0	1.9	57803.8	1.5	54215.1	1.4	54215.1	1.4
59012.8	2.1	64055.3	1.8	61069.2	1.7	61069.2	1.7
67093.3	2.3	69819.0	2.1	68196.5	2.1	67599.8	2.0
170284.8	4.8	100438.5	4.9	99863.5	4.8	99182.3	3.6

Similarly, in addition to the above point, other hydraulic parameters are extracted from HEC-RAS outputs. These parameters extracted from the main river centerline are left bank length, right bank length, flows channel, channel velocity, flow area, different elevation (water surface, minimum channel, critical water surface, and energy grade level elevations), and velocity head and channel Froude number. Area of inundation is calculated from average cross section over each the bank of river. The outputs are available in the form of both graphical and tabular. This detail report is attached in annex A-13

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusion

In this study, hydrological and hydraulic models of big Akaki river basins with different climatic and geographical characteristics were developed. The hydrological model using HEC-HMS is developed to study the effect of rainfall and flow release from the dam on surface runoff and peak discharges at the study area. Then flood inundation maps produced using HEC-RAS that enable to study the flood extent and its characteristics. At the outlet location of the basin, the case study of hydrologic and hydraulic modeling is developed by considering four scenarios at the upstream of the selected out location.

To develop hydrological modeling first basin model of big Akaki watershed is created by applying the Hec-GeoHMS and Arc hydro software with the function of Arc Map in a GIS environment. Then data preparation for HEC-HMS with the aid of GIS tool extension and terrain preprocessing is manipulated on DEM data. The main input data used for pre- hydrological modeling are land use, soil hydrological group, daily rainfall, and evapotranspiration. After the pre-hydrological modeling completed HEC-HMS version 4.2.1 project is used. Finally, HEC-HMS computes peak runoff by computing the volume of water that is intercepted, infiltrated, stored, evaporated, or transpired and subtracting it from the precipitation.

The first reason for hydrologic modeling in this study is to show the effect of flow release from Dire and 'Lega Dadi dam lies in the watershed of the big Akaki river basin. The magnitude flood prone area due to dam structure on the downstream flood plain is identified. In this study, the magnitude of flooded area without considering the release of flow from both dams, by considering simultaneous release flow from both dam, release of flow from Dire dam and 'Lega Dadi dam separately is analyzed in detail. Therefore, the releases of flow from both dam have significant change on peak flow and flow volume on the result of runoff modeled by HEC-HMS software.

The second reason need for the hydrologic model is flood mapping. Using the result obtained from HEC-HMS the statistical frequency analysis which is the most commonly used procedure for the analysis of flood data at a gaged location is considered to predict the flood within the return period. Accordingly, peak discharge without considering the release of flow from 'Lega Dadi and Dire dam is 277.459 m^3/s, 362.207 m^3/s, 462.898 m^3/s, 946.851m^3/s for respective return period of 25,50,100 and 1000 is calculated. Additionally, according to the result stated in the annex A-5 by considering flow release from both dam the peak discharge of 354.05 m^3/s, 429.15 m^3/s, 516.97 m^3/s, and 963.10m^3/s is obtained. This result found from Hydro gnomon software. Then the result is used for further hydraulic analysis and flood map generation.

To performed the hydraulic modeling all the created model geometry files with HEC-GeoRAS which is the interface of RAS-GIS and the .exported file from HEC-GeoRAS is imported into HEC-RAS version 5.0.4, in which the hydraulic modeling be analyzed. The main components of the River Analysis System (RAS) geometry are, flow data entry, plan for run the model, RAS-Mapper and result graph data. After all hydraulic parameters were analyzed the result connected with RAS Mapper and exported to GIS for flood mapping. Finally, flood maps were developed using RAS Mapper, ArcGIS, and GIS extension tool HEC-GeoRAS. Flood depth and its extent for each return period is visualize in RAS mapper and HEC-GeoRAS.The maximum channel flood depths found in the middle big Akaki rive is 8.8, 9.3, 10.1 and 15.5 m for 25,50, 100 and 1000 return period. This result obtained by considering the simultaneous release of flow from 'Lega Dadi and Dire dam. Similarly, the average depth of overtopping flood for each return period is 2 m and an area of flooding 0.0578km^2, 0.0641km^{2,} 0.0698km^2, and 0.1km^2 is obtained respectively. Although the magnitude of the flooded area varied with small amounts, it is significant to analyze the risk on the downstream flood plain. The average top width of flood over the bank is 136.34m for each return period of 25, 50,100 and 1000 years.

Impacts of flood are seen in both the catchments with larger effects in big Akaki River basin. An area which is prone to flood risk is Timkete Behar near Turunesh hospital, upstream and downstream of Akaki Bridge, compound of newly constructed Jail and some of the agricultural land near Galan condominium is identified.

Additionally, this study focus to assess the watershed characteristic developed with the aid of GIS and its extension Arc hydro and Hec-GeoHMS for the watershed across Addis Ababa by considering the outlet at Abasamuel reservoir. The main two river across the town is little Akaki and big Akaki river which is not dendritic stream flows into Abasamuel reservoir. Aba Samuel watershed covers an area of 1785.981km^2. Some of the river within this watershed are big Akaki, little Akaki, Kabana river and other tributaries with length 71.55km, 47.55km,20.51 and 316.4km is determined respectively. The remaining watershed characteristics are shown in section 5.3.

5.2 Recommendations

The Addis Ababa city with fast population growth of the town, uncontrolled Urbanization and industrialization causes changing of demographic features within the river flood plain has led to increased exposure of communities, agricultural land, and infrastructure to flood hazard. If the flood is not managed properly, it causes huge damage to human and animal life, infrastructure, recreation areas, and population utilities. In this study the magnitude of flooded area without considering the release of flow from both dams in the upstream, by considering simultaneous release of flow from both dam, separate release of flow from Dire and 'Lega Dadi dam is analyzed in detail. The result of all this for scenarios indicate that the flood plain in the study area may suffer from flood hazard. Different construction works near the bank of river and floor of the valley will have to done beyond 136.34m from the bank of the river.

Although, this research is conducted by using limited data the following recommendations are made;

- The average 2m flood depth above the critical water surface which leads to topping of river bank need great attention for the area of Timkete Behar near Turunesh hospital, upstream and downstream of Akaki Bridge, compound of newly constructed Jail and some of the agricultural land near Galan condominium.
- Like other large cities of the world, the land in Addis Ababa is more or less built up with impervious materials like corrugated iron roof, asphalt or compacted gravel roads, drainage system, airfields, car parks, recreational areas, and other man-made impermeable structures. These human-induced features significantly increase the amount and movement of water in the streams crossing the city. This study did not consider the water quality in the hydrologic model, the water quality analysis will need research for further study.
- According to this study, when the peak maximum flood come for the return period of 100 year on construction Akaki bridge will be flooded. The maximum depth of flood within the middle of the river is 10.1m and additional study will be required for flood control structures.
- Due to the rapid growth of the population and expansion of the city from year to year, there is a serious shortage of housing, community-built their home in buffer zone. mitigating and integrated management mechanism of land use and land use planning have been required
- The entire river across Aba Samuel watershed will not have enough flow gage station and the Akaki flow gage equipped with an automatic water level recorder is not functional. Therefore, enough flow gages should have to install on the river to record flow level used for further studies.
- The land use delineated from the European Space Agency (ESA) that is developed at the global scale for the resolution of 300m. This classification is more reliable than the land use land cover map prepared by the Ethiopian Ministry of Water and Energy Bureau for further studies.

6 Reference

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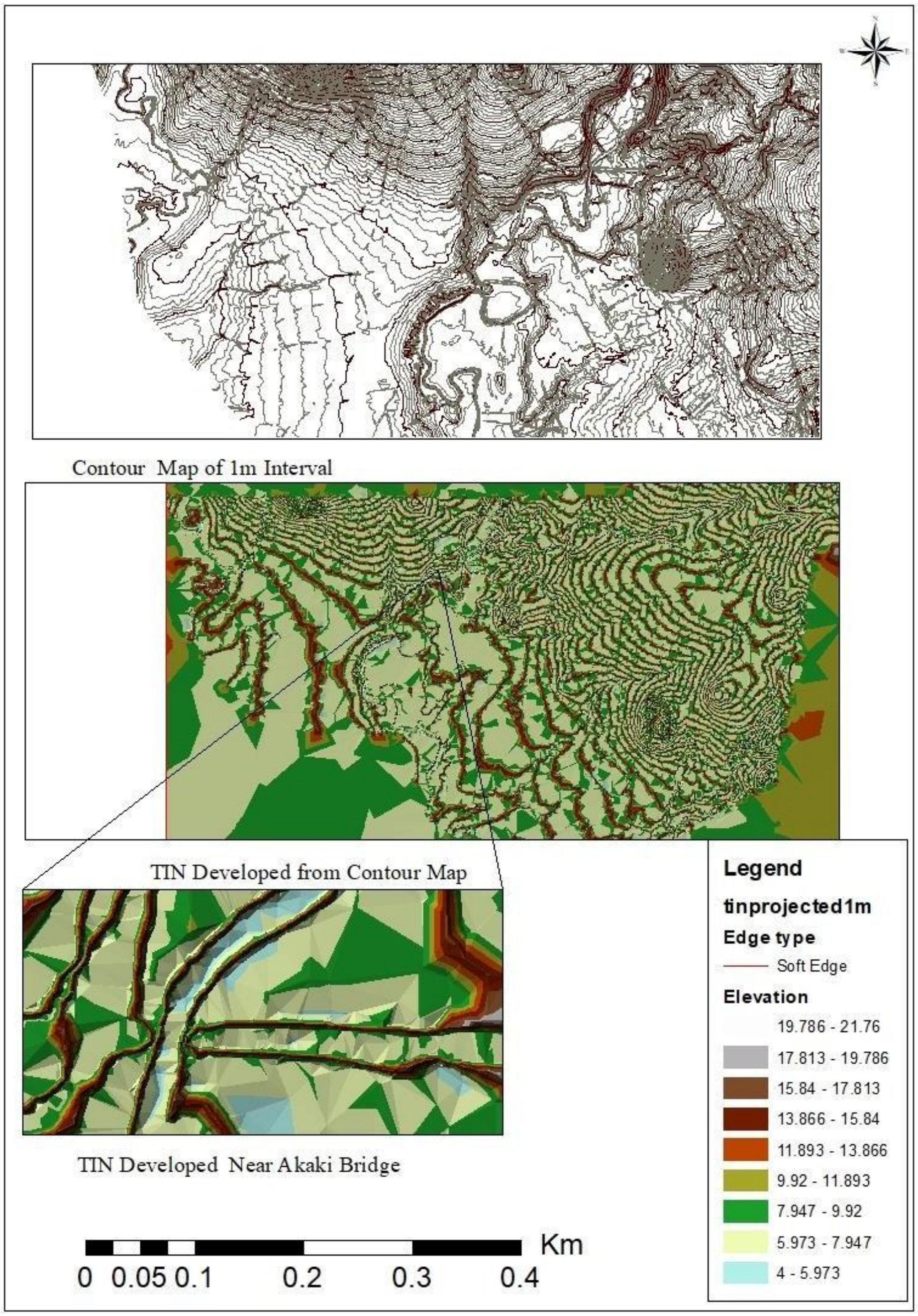
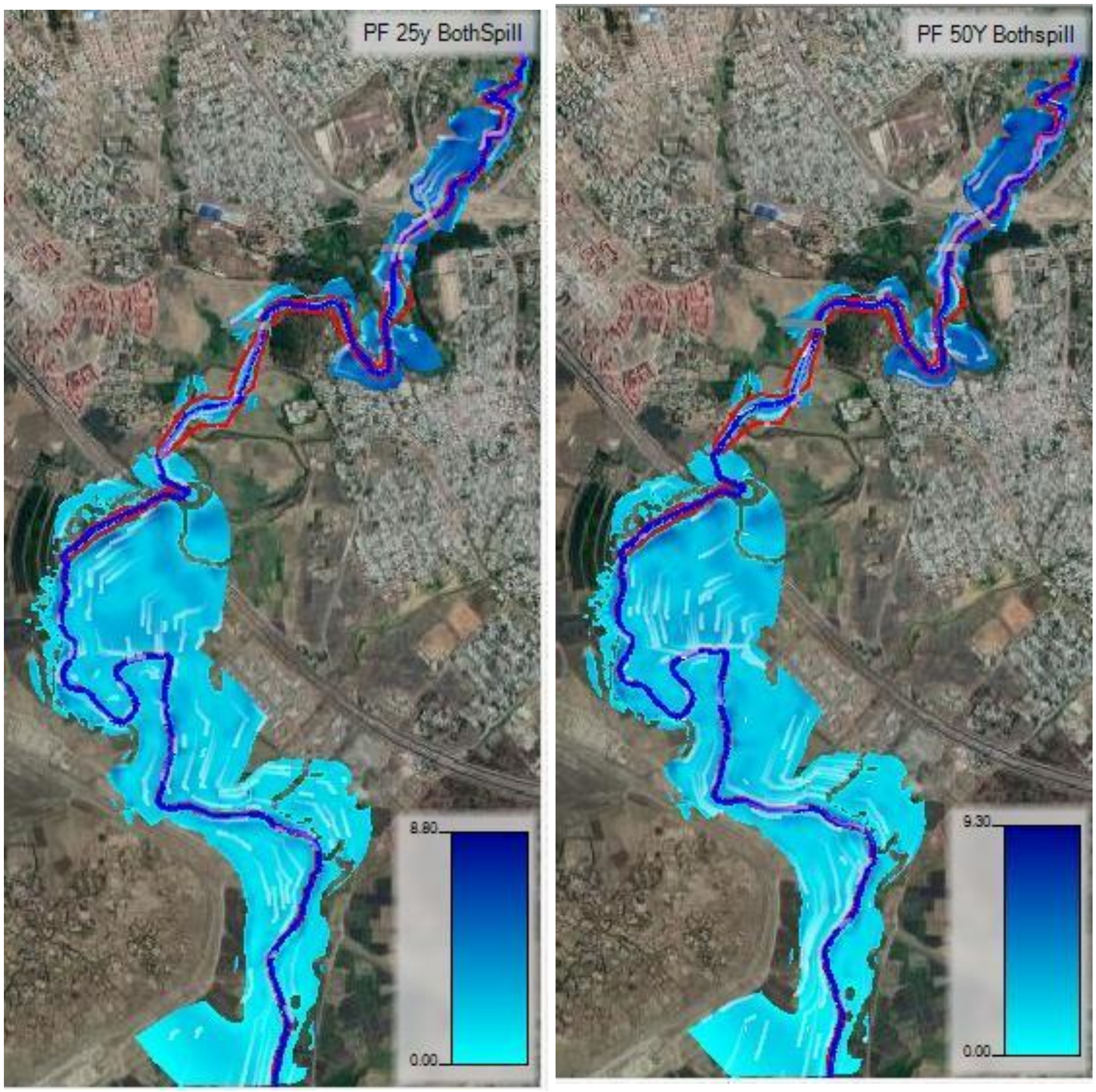
Annex

Annex-A

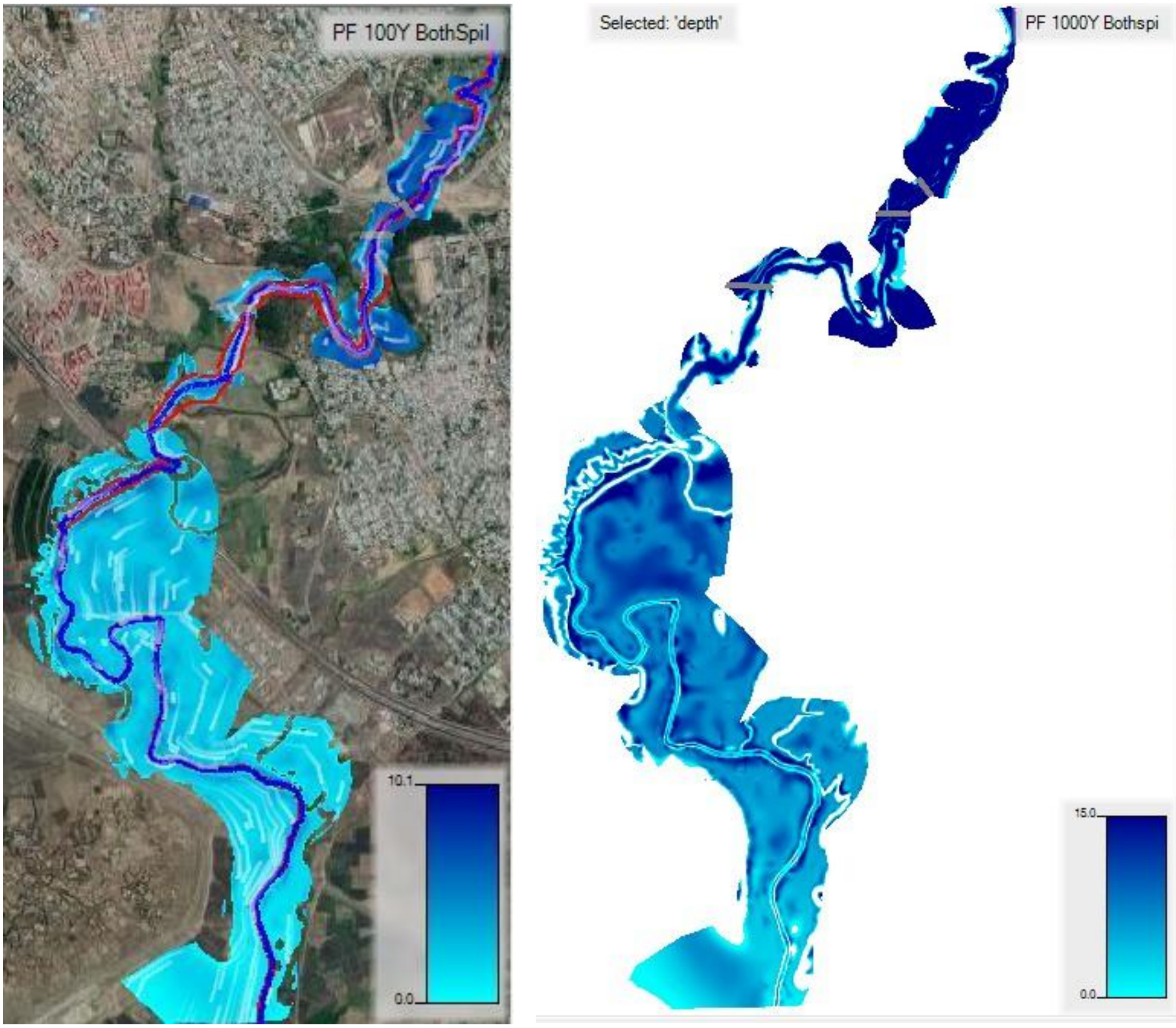
A-1 TIN Map Developed from Contour map 1m interval for HEC-RAS Model



A-2 Flood map for case of flood released from both Dire and 'Lega Dadi Dam Simultaneously for 25, 50 years return period



A-3 Flood map for case of flood released from both Dire and 'Lega Dadi Dam Simultaneously for 100, 1000 years return period



A-4 Flood inundation map result without considering the flow release from dam in upstream of outlet

Upstream of Akaki Bridge(1390m)							
Return period	Q Total	W.S. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Depth
	(m3/s)	(m)	(m/m)	(m/s)	(m2)	(m)	(m)
25	277.46	12.82	0.00200	1.35	18695.62	134.48	1.79
50	362.21	13.60	0.00210	1.49	21615.95	137.22	1.95
100	462.90	14.42	0.00218	1.63	24797.26	140.38	2.13
1000	946.85	17.57	0.00238	2.16	38134.01	144.67	5.85
Downstream of on construction Akaki Bridge (7713m)							
Return period	Q Total	W.S. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Depth
	(m3/s)	(m)	(m/m)	(m/s)	(m2)	(m)	(m)
25	277.46	10.58	0.0011	1.15	32821.37	293.01	1.92
50	362.21	11.01	0.0011	1.27	37396.81	295.99	2.18
100	462.90	11.47	0.0012	1.40	42296.05	299.11	2.44
1000	946.85	13.15	0.0013	1.87	59850.20	313.70	3.71

A-5 Flood inundation map result by considering the flow release from 'Lega Dadi and Dire dam in upstream of outlet

Upstream of Akaki Bridge(1390m)						
Q Total	W.S. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Depth
(m3/s)	(m)	(m/m)	(m/s)	(m2)	(m)	(m)
354.05	13.42	0.00264	1.59	21299.18	130.24	1.35
429.15	14.06	0.00264	1.69	23760.63	132.24	1.62
516.97	14.75	0.00263	1.81	26470.08	134.75	1.92
963.1	17.67	0.00237	2.17	38533.95	144.92	5.92
Downstream of on construction Akaki Bridge (7713m)						
Q Total	W.S. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Depth
(m3/s)	(m)	(m/m)	(m/s)	(m2)	(m)	(m)
354.05	10.97	0.0011	1.30	36504.65	297.82	1.74
429.15	11.32	0.0011	1.39	40294.70	300.00	2.01
516.97	11.70	0.0012	1.51	43348.90	304.51	2.21
963.1	13.28	0.0013	1.8560	61904.5600	311.3292	3.81

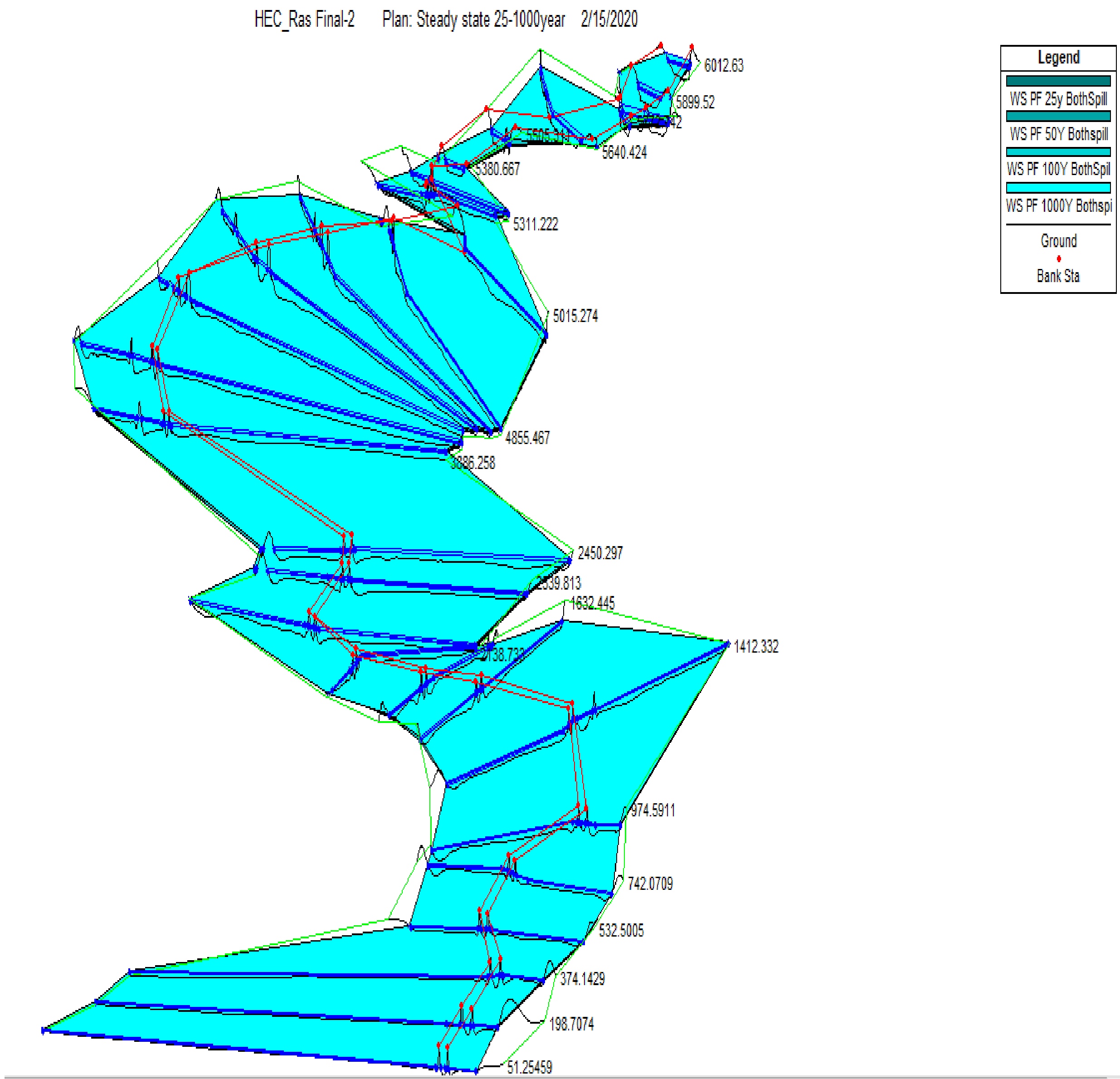
A-6 Flood inundation map result by considering the flow release from 'Lega Dadi in upstream of outlet

Upstream of Akaki Bridge(1390m)						
Q Total	W.S. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Depth
(m3/s)	(m)	(m/m)	(m/s)	(m2)	(m)	(m)
313.34	13.05567	0.002641	1.526	19904.78	129.205	1.273
392.67	13.76033	0.002636	1.646333	22582.63	131.2257	1.567333
487.2	14.52467	0.002636	1.772333	25566.39	133.9227	1.899
952.31	17.91133	0.002378	2.162333	38268.57	144.7473	5.871
Downstream of on construction Akaki Bridge (7713m)						
Q Total	W.S. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Depth
(m3/s)	(m)	(m/m)	(m/s)	(m2)	(m)	(m)
313.34	10.77245	0.001065	1.235472	34310.28	296.6783	1.597358
392.67	11.15698	0.001122	1.34717	38486.6	298.8281	1.895283
487.2	11.64038	0.001007	1.415769	42630.11	290.5613	2.234423
952.31	13.50642	0.001311	1.849434	61522.16	311.1132	3.788113

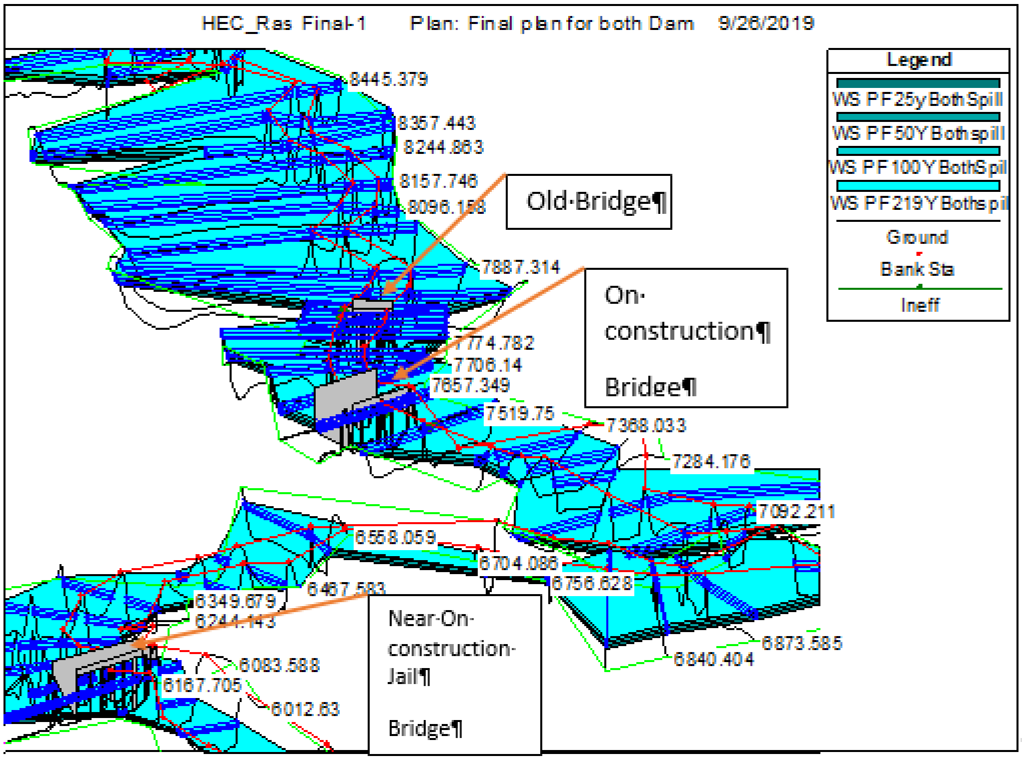
A-7 Flood inundation map result by considering the flow release from Dire dam in upstream of outlet

	Upstream of Akaki Bridge(1390m)						
Return period	Q Total	W.S. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Depth
	(m3/s)	(m)	(m/m)	(m/s)	(m2)	(m)	(m)
25	313.34	13.06	0.00264	1.53	19904.78	129.21	1.27
50	392.67	13.76	0.00264	1.65	22582.63	131.23	1.57
100	487.20	14.52	0.00264	1.77	25566.39	133.92	1.90
1000	953.10	14.55	0.00183	1.70426	111798.02	325.87625	3.58
	Downstream of on construction Akaki Bridge (7713m)						
Return period	Q Total	W.S. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Depth
	(m3/s)	(m)	(m/m)	(m/s)	(m2)	(m)	(m)
25	313.34	10.77	0.0011	1.24	34310.28	296.68	1.60
50	392.67	11.16	0.0011	1.35	38486.60	298.83	1.90
100	487.20	11.53	0.0012	1.47	42033.44	303.89	2.14
1000	953.10	13.10	0.0014	1.89	58718.11	316.57	3.65

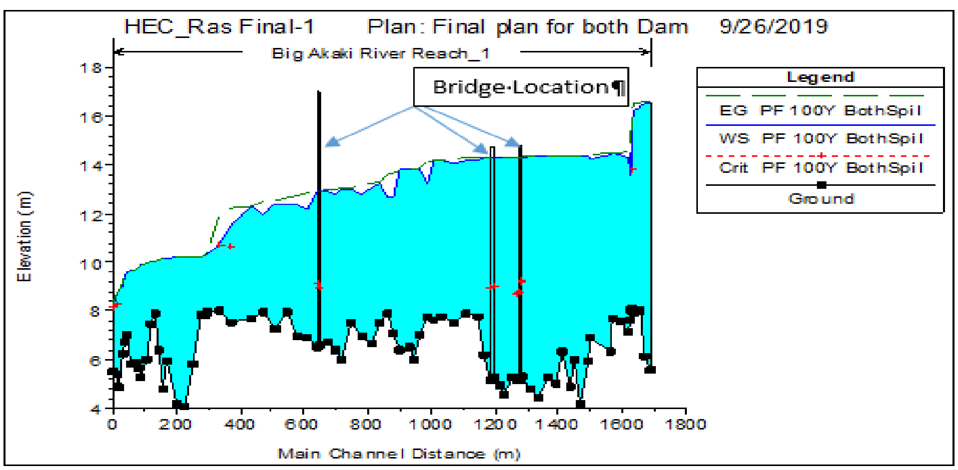
A-8 x, y, z perspective view of flow around all reach for 25, 50,100, 1000 year return period



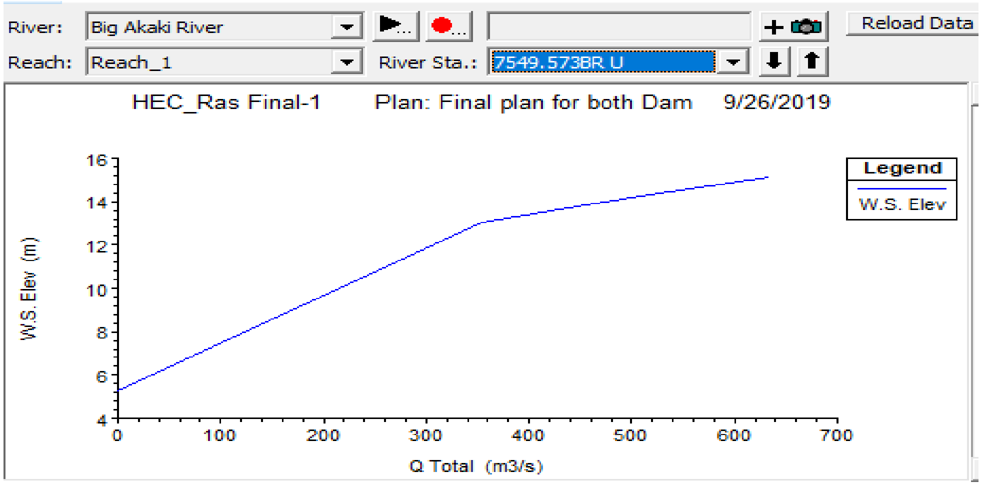
A-9 x, y, z Perspective view of flow around three bridge for 25, 50,100, 1000 year return period



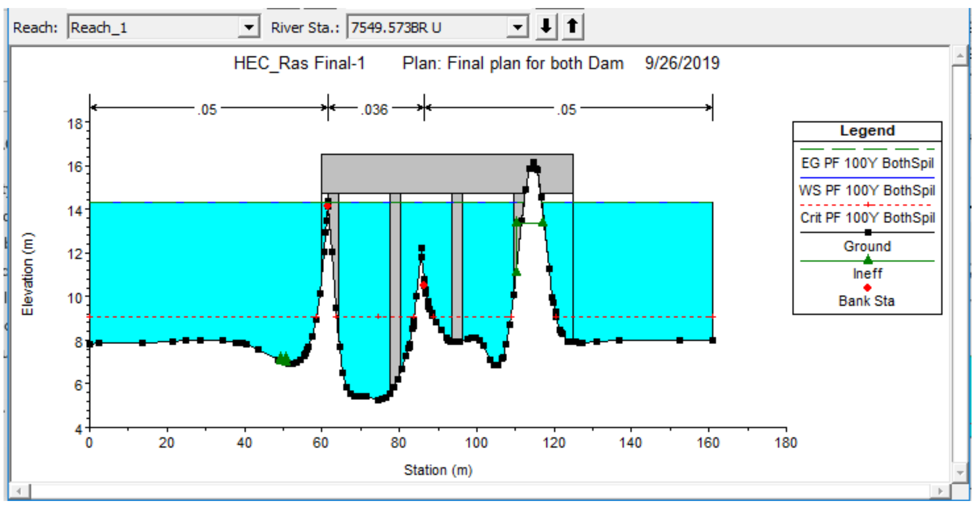
A-10 Energy grade line, water surface a critical water surface profile of main channel for both dam release flow for 100 year return period



A-11 Rating Curve at Akaki Bridge 100 years return period



A-12 Akaki Bridge by considering both dam release for 100 years return period



A-13 HEC-RAS Result for Steady-state simulation by considering both dam release flow simultaneously

HEC-RAS Plan: Final plan Both dam River: Big Akaki River Reach: Reach_1 Profile: PF 100Y BothSpil

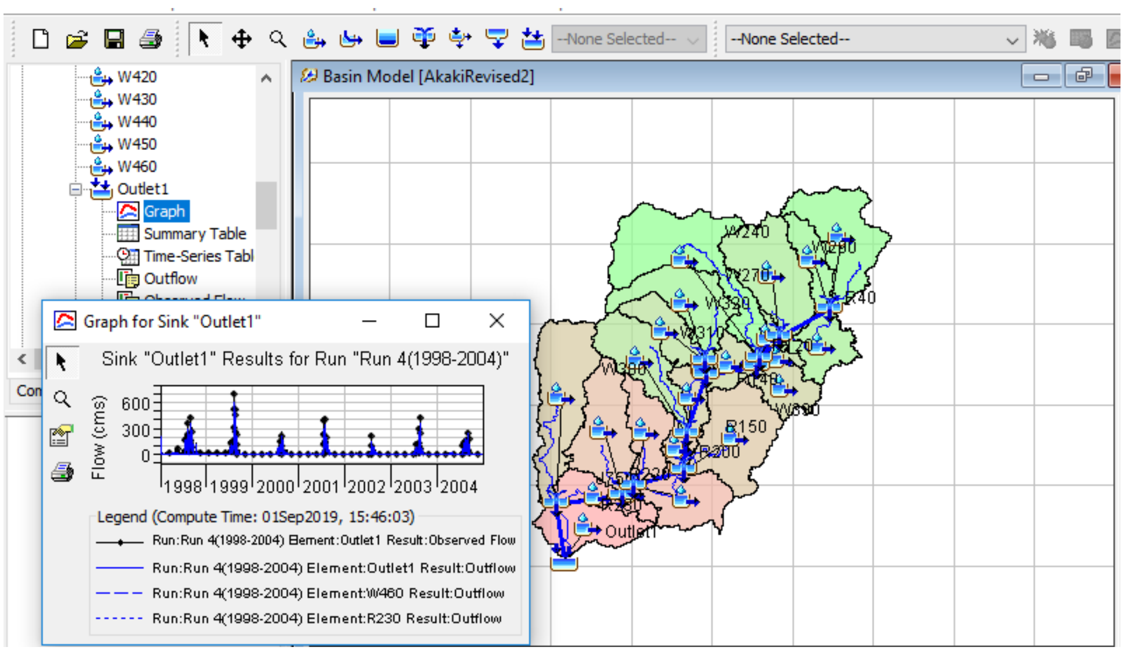
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Reach_1	9093.12	PF 100Y BothSpil	516.97	5.55	16.55		16.61	0.000123	1.11	661.54	84.96	0.12
Reach_1	9076.03	PF 100Y BothSpil	516.97	6.12	16.48		16.60	0.000315	1.62	443.38	61.67	0.18
Reach_1	9061.953	PF 100Y BothSpil	516.97	7.99	16.41		16.59	0.000828	2.19	364.53	58.05	0.29
Reach_1	9052.95	PF 100Y BothSpil	516.97	7.99	16.31		16.56	0.001597	2.76	312.45	53.37	0.38
Reach_1	9045.11	PF 100Y BothSpil	516.97	8.00	16.26		16.52	0.001893	2.88	287.82	48.03	0.40
Reach_1	9015.544	PF 100Y BothSpil	516.97	7.98	16.20		16.48	0.003116	3.20	254.14	39.27	0.45
Reach_1	8984.802	PF 100Y BothSpil	516.97	8.09	15.89		16.39	0.003247	2.47	167.43	27.72	0.36
Reach_1	8953.16	PF 100Y BothSpil	516.97	7.99	13.78	13.78	16.08	0.039348	8.65	92.89	20.77	1.43
Reach_1	8907.607	PF 100Y BothSpil	516.97	7.99	14.46		15.13	0.014328	4.83	150.80	28.48	0.78
Reach_1	8862.708	PF 100Y BothSpil	516.97	7.62	13.59		14.89	0.010475	5.89	133.22	30.18	0.88
Reach_1	8820.239	PF 100Y BothSpil	516.97	7.14	14.32		14.56	0.000848	2.26	288.36	52.82	0.31
Reach_1	8787.353	PF 100Y BothSpil	516.97	7.59	14.42		14.50	0.000239	1.39	481.74	73.09	0.18
Reach_1	8749.498	PF 100Y BothSpil	516.97	7.68	14.46		14.48	0.000126	0.86	1263.67	191.09	0.11
Reach_1	8667.173	PF 100Y BothSpil	516.97	6.32	14.45		14.47	0.000072	0.79	999.45	150.90	0.10
Reach_1	8547.215	PF 100Y BothSpil	516.97	6.90	14.24		14.44	0.000924	2.27	334.45	61.72	0.32
Reach_1	8445.379	PF 100Y BothSpil	516.97	5.93	14.37		14.37	0.000071	0.60	1696.92	261.82	0.08
Reach_1	8357.443	PF 100Y BothSpil	516.97	4.17	14.36		14.37	0.000043	0.58	1955.95	290.11	0.07
Reach_1	8295.271	PF 100Y BothSpil	516.97	5.99	14.35		14.36	0.000133	0.88	1150.90	161.83	0.11
Reach_1	8244.863	PF 100Y BothSpil	516.97	4.91	14.35		14.36	0.000068	0.64	1678.60	259.55	0.08
Reach_1	8157.746	PF 100Y BothSpil	516.97	6.33	14.34		14.35	0.000094	0.71	1486.75	242.83	0.09
Reach_1	8096.158	PF 100Y BothSpil	516.97	5.03	14.34		14.35	0.000081	0.77	1454.12	231.75	0.09
Reach_1	8022.108	PF 100Y BothSpil	516.97	5.23	14.33		14.34	0.000065	0.68	1591.58	231.07	0.08
Reach_1	7887.314	PF 100Y BothSpil	516.97	4.44	14.33		14.34	0.000018	0.36	1804.10	264.07	0.04
Reach_1	7816.769	PF 100Y BothSpil	516.97	4.80	14.33		14.34	0.000015	0.35	2049.04	303.82	0.04
Reach_1	7774.782	PF 100Y BothSpil	516.97	5.29	14.28	9.25	14.33	0.000319	1.43	691.43	103.13	0.17
Reach_1	7749.981		Bridge									
Reach_1	7729.94	PF 100Y BothSpil	516.97	5.10	14.29	8.72	14.32	0.000095	0.88	755.80	109.85	0.10
Reach_1	7706.14	PF 100Y BothSpil	516.97	5.26	14.30		14.31	0.000044	0.60	1138.36	172.12	0.07
Reach_1	7657.349	PF 100Y BothSpil	516.97	4.57	14.28		14.31	0.000154	1.06	991.74	150.54	0.12
Reach_1	7625.615	PF 100Y BothSpil	516.97	4.96	14.28		14.30	0.000094	0.80	807.83	121.67	0.10
Reach_1	7576.129	PF 100Y BothSpil	516.97	5.28	14.28	9.05	14.30	0.000067	0.69	981.09	156.16	0.08
Reach_1	7549.573		Bridge									
Reach_1	7519.75	PF 100Y BothSpil	516.97	5.12	14.28	8.94	14.30	0.000062	0.66	1025.38	162.18	0.08
Reach_1	7455.87	PF 100Y BothSpil	516.97	6.19	14.27		14.29	0.000068	0.75	901.02	151.89	0.09
Reach_1	7455.87	PF 100Y BothSpil	516.97	6.19	14.27		14.29	0.000068	0.75	901.02	151.89	0.09
Reach_1	7368.033	PF 100Y BothSpil	516.97	7.74	14.19		14.28	0.000320	1.37	401.58	87.12	0.20
Reach_1	7326.326	PF 100Y BothSpil	516.97	7.91	14.16		14.27	0.000337	1.41	369.08	80.60	0.21
Reach_1	7284.176	PF 100Y BothSpil	516.97	7.54	14.05		14.24	0.000615	1.94	266.19	51.70	0.27
Reach_1	7200.517	PF 100Y BothSpil	516.97	7.77	14.18		14.18	0.000011	0.26	2335.84	397.23	0.03
Reach_1	7125.012	PF 100Y BothSpil	516.97	7.64	14.18		14.18	0.000012	0.26	2238.28	355.42	0.03
Reach_1	7092.211	PF 100Y BothSpil	516.97	7.76	13.21		14.09	0.003711	4.15	124.67	26.12	0.61
Reach_1	6987.171	PF 100Y BothSpil	516.97	7.04	13.80		13.83	0.000124	0.84	676.23	102.99	0.11
Reach_1	6873.585	PF 100Y BothSpil	516.97	6.04	13.80		13.82	0.000082	0.75	865.67	141.20	0.10
Reach_1	6840.404	PF 100Y BothSpil	516.97	6.48	13.80		13.81	0.000078	0.71	916.73	159.82	0.09
Reach_1	6803.909	PF 100Y BothSpil	516.97	6.40	13.80		13.81	0.000049	0.53	1137.75	193.31	0.07
Reach_1	6756.628	PF 100Y BothSpil	516.97	7.10	12.69		13.69	0.004444	4.45	116.27	26.13	0.67
Reach_1	6704.086	PF 100Y BothSpil	516.97	7.87	12.66		13.59	0.004403	4.27	121.16	30.18	0.68
Reach_1	6558.059	PF 100Y BothSpil	516.97	7.50	13.25		13.29	0.000164	0.94	657.13	129.18	0.13
Reach_1	6467.583	PF 100Y BothSpil	516.97	6.62	13.01		13.25	0.000885	2.20	256.30	59.50	0.32
Reach_1	6423.305	PF 100Y BothSpil	516.97	6.97	12.78		13.19	0.001593	2.87	189.01	46.98	0.42
Reach_1	6349.679	PF 100Y BothSpil	516.97	7.51	12.96		13.06	0.000513	1.60	385.84	85.65	0.23
Reach_1	6296.364	PF 100Y BothSpil	516.97	6.01	12.97		13.04	0.000344	1.32	475.08	105.62	0.20
Reach_1	6244.143	PF 100Y BothSpil	516.97	6.40	12.82		13.01	0.000937	2.17	369.63	83.73	0.33
Reach_1	6204.329	PF 100Y BothSpil	516.97	6.70	12.90		12.95	0.000305	1.21	538.52	119.74	0.19
Reach_1	6167.705	PF 100Y BothSpil	516.97	6.55	12.91	8.94	12.93	0.000150	0.91	763.39	166.75	0.13

Reach_1	7455.87	PF 100Y BothSpil	516.97	6.19	14.27		14.29	0.000068	0.75	901.02	151.89	0.09
Reach_1	7368.033	PF 100Y BothSpil	516.97	7.74	14.19		14.28	0.000320	1.37	401.58	87.12	0.20
Reach_1	7326.326	PF 100Y BothSpil	516.97	7.91	14.16		14.27	0.000337	1.41	369.08	80.60	0.21
Reach_1	7284.176	PF 100Y BothSpil	516.97	7.54	14.05		14.24	0.000615	1.94	266.19	51.70	0.27
Reach_1	7200.517	PF 100Y BothSpil	516.97	7.77	14.18		14.18	0.000011	0.26	2335.84	397.23	0.03
Reach_1	7125.012	PF 100Y BothSpil	516.97	7.64	14.18		14.18	0.000012	0.26	2238.28	355.42	0.03
Reach_1	7092.211	PF 100Y BothSpil	516.97	7.76	13.21		14.09	0.003711	4.15	124.67	26.12	0.61
Reach_1	6987.171	PF 100Y BothSpil	516.97	7.04	13.80		13.83	0.000124	0.84	676.23	102.99	0.11
Reach_1	6873.585	PF 100Y BothSpil	516.97	6.04	13.80		13.82	0.000082	0.75	865.67	141.20	0.10
Reach_1	6840.404	PF 100Y BothSpil	516.97	6.48	13.80		13.81	0.000078	0.71	916.73	159.82	0.09
Reach_1	6803.909	PF 100Y BothSpil	516.97	6.40	13.80		13.81	0.000049	0.53	1137.75	193.31	0.07
Reach_1	6756.628	PF 100Y BothSpil	516.97	7.10	12.69		13.69	0.004444	4.45	116.27	26.13	0.67
Reach_1	6704.086	PF 100Y BothSpil	516.97	7.87	12.66		13.59	0.004403	4.27	121.16	30.18	0.68
Reach_1	6558.059	PF 100Y BothSpil	516.97	7.50	13.25		13.29	0.000164	0.94	657.13	129.18	0.13
Reach_1	6467.583	PF 100Y BothSpil	516.97	6.62	13.01		13.25	0.000885	2.20	256.30	59.50	0.32
Reach_1	6423.305	PF 100Y BothSpil	516.97	6.97	12.78		13.19	0.001593	2.87	189.01	46.98	0.42
Reach_1	6349.679	PF 100Y BothSpil	516.97	7.51	12.96		13.06	0.000513	1.60	385.84	85.65	0.23
Reach_1	6296.364	PF 100Y BothSpil	516.97	6.01	12.97		13.04	0.000344	1.32	475.08	105.62	0.20
Reach_1	6244.143	PF 100Y BothSpil	516.97	6.40	12.82		13.01	0.000937	2.17	369.63	83.73	0.33
Reach_1	6204.329	PF 100Y BothSpil	516.97	6.70	12.90		12.95	0.000305	1.21	538.52	119.74	0.19
Reach_1	6167.705	PF 100Y BothSpil	516.97	6.55	12.91	8.94	12.93	0.000150	0.91	763.39	166.75	0.13
Reach_1	6154.782		Bridge									
Reach_1	6141.933	PF 100Y BothSpil	516.97	6.54	12.91		12.93	0.000135	0.88	780.61	166.30	0.12
Reach_1	6083.588	PF 100Y BothSpil	516.97	6.87	12.19		12.85	0.002908	3.62	144.70	36.07	0.55
Reach_1	6012.63	PF 100Y BothSpil	516.97	6.98	12.34		12.70	0.001608	2.65	195.04	49.69	0.43
Reach_1	5899.52	PF 100Y BothSpil	516.97	7.98	12.33		12.63	0.001444	2.44	211.80	58.20	0.41
Reach_1	5816.029	PF 100Y BothSpil	516.97	7.28	12.44		12.55	0.000551	1.61	373.14	91.73	0.25
Reach_1	5772.742	PF 100Y BothSpil	516.97	7.98	11.95		12.46	0.002561	3.17	163.18	44.51	0.53
Reach_1	5640.424	PF 100Y BothSpil	516.97	7.73	12.28		12.30	0.000087	0.65	921.08	244.12	0.10
Reach_1	5505.311	PF 100Y BothSpil	516.97	7.49	11.56	10.67	12.21	0.004308	3.58	144.46	45.30	0.64
Reach_1	5380.667	PF 100Y BothSpil	516.97	8.00	10.73	10.73	11.92	0.009839	4.82	108.05	46.68	1.00
Reach_1	5311.222	PF 100Y BothSpil	516.97	7.98	10.34		10.39	0.000621	1.06	565.08	227.04	0.24
Reach_1	5261.613	PF 100Y BothSpil	516.97	7.84	10.25		10.29	0.000681	0.73	576.60	250.14	0.18
Reach_1	5015.274	PF 100Y BothSpil	516.97	7.86	10.23		10.24	0.000130	0.44	1075.22	358.41	0.11

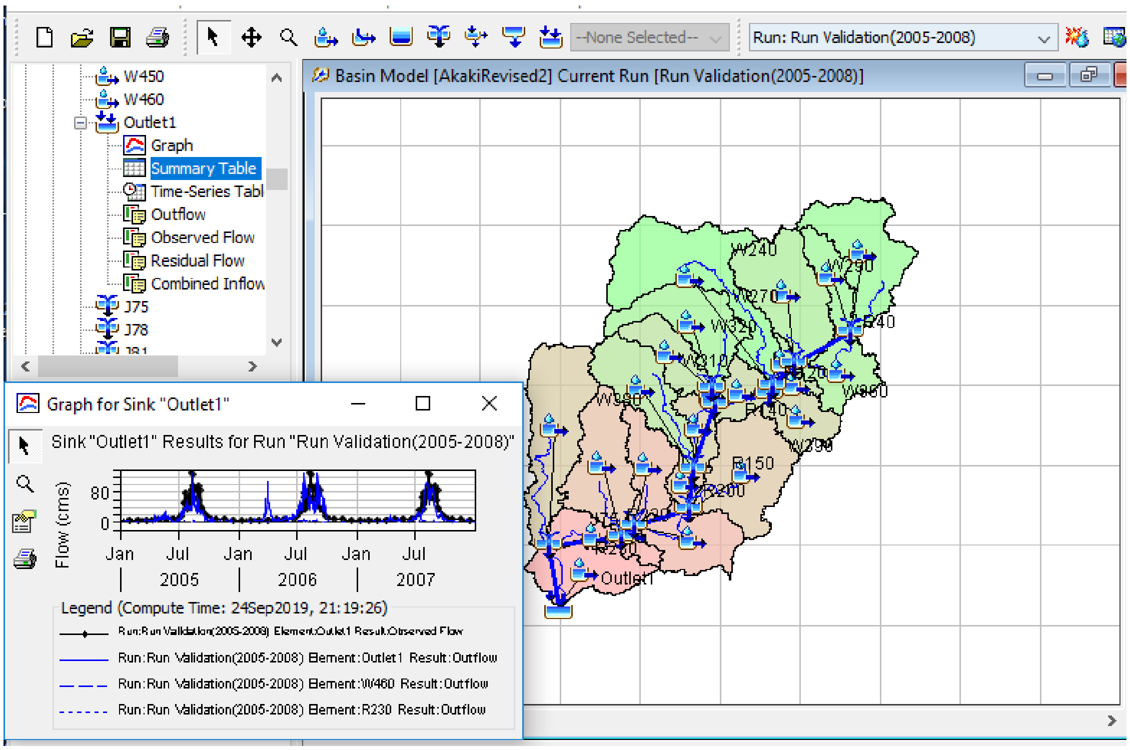
Reach_1	6154.782		Bridge									
Reach_1	6141.933	PF 100Y BothSpil	516.97	6.54	12.91		12.93	0.000135	0.88	780.61	166.30	0.12
Reach_1	6083.588	PF 100Y BothSpil	516.97	6.87	12.19		12.85	0.002908	3.62	144.70	36.07	0.55
Reach_1	6012.63	PF 100Y BothSpil	516.97	6.98	12.34		12.70	0.001608	2.65	195.04	49.69	0.43
Reach_1	5899.52	PF 100Y BothSpil	516.97	7.98	12.33		12.63	0.001444	2.44	211.80	58.20	0.41
Reach_1	5816.029	PF 100Y BothSpil	516.97	7.28	12.44		12.55	0.000551	1.61	373.14	91.73	0.25
Reach_1	5772.742	PF 100Y BothSpil	516.97	7.98	11.95		12.46	0.002561	3.17	163.18	44.51	0.53
Reach_1	5640.424	PF 100Y BothSpil	516.97	7.73	12.28		12.30	0.000087	0.65	921.08	244.12	0.10
Reach_1	5505.311	PF 100Y BothSpil	516.97	7.49	11.56	10.67	12.21	0.004308	3.58	144.46	45.30	0.64
Reach_1	5380.667	PF 100Y BothSpil	516.97	8.00	10.73	10.73	11.92	0.009839	4.82	108.05	46.68	1.00
Reach_1	5311.222	PF 100Y BothSpil	516.97	7.98	10.34		10.39	0.000621	1.06	565.08	227.04	0.24
Reach_1	5261.613	PF 100Y BothSpil	516.97	7.84	10.25		10.29	0.000681	0.73	576.60	250.14	0.18
Reach_1	5015.274	PF 100Y BothSpil	516.97	7.86	10.23		10.24	0.000130	0.44	1075.22	358.41	0.11
Reach_1	4855.467	PF 100Y BothSpil	516.97	5.80	10.23		10.23	0.000032	0.28	2124.39	677.98	0.05
Reach_1	4705.24	PF 100Y BothSpil	516.97	4.05	10.21		10.21	0.000034	0.38	2228.35	820.48	0.06
Reach_1	4570.888	PF 100Y BothSpil	516.97	4.16	10.19		10.20	0.000034	0.38	2239.67	839.94	0.06
Reach_1	4379.548	PF 100Y BothSpil	516.97	5.96	10.16		10.17	0.000060	0.38	1827.17	747.38	0.07
Reach_1	4108.303	PF 100Y BothSpil	516.97	4.81	10.14		10.14	0.000029	0.24	2347.02	804.87	0.04
Reach_1	3886.258	PF 100Y BothSpil	516.97	6.37	10.11		10.12	0.000070	0.33	1708.45	699.51	0.07
Reach_1	2450.297	PF 100Y BothSpil	516.97	7.92	10.05		10.06	0.000103	0.38	1417.96	585.79	0.09
Reach_1	2339.813	PF 100Y BothSpil	516.97	7.43	10.01		10.02	0.000131	0.43	1253.64	515.66	0.10
Reach_1	2138.733	PF 100Y BothSpil	516.97	5.98	9.98		9.99	0.000090	0.41	1471.15	579.29	0.08
Reach_1	1900.367	PF 100Y BothSpil	516.97	5.23	9.90		9.93	0.000624	1.28	654.35	337.55	0.22
Reach_1	1742.427	PF 100Y BothSpil	516.97	5.64	9.75		9.79	0.000755	1.26	584.78	291.63	0.24
Reach_1	1632.445	PF 100Y BothSpil	516.97	5.88	9.71		9.72	0.000231	0.73	1009.29	465.10	0.14
Reach_1	1412.332	PF 100Y BothSpil	516.97	5.85	9.67		9.68	0.000127	0.51	1424.58	708.68	0.10
Reach_1	974.5911	PF 100Y BothSpil	516.97	6.99	9.57		9.59	0.000460	0.80	757.94	378.16	0.19
Reach_1	742.0709	PF 100Y BothSpil	516.97	6.71	9.43		9.48	0.001156	1.29	568.87	370.68	0.30
Reach_1	532.5005	PF 100Y BothSpil	516.97	6.26	9.04		9.12	0.002768	2.02	415.74	335.24	0.47
Reach_1	374.1429	PF 100Y BothSpil	516.97	4.91	8.83		8.86	0.000928	1.47	811.31	812.63	0.29
Reach_1	198.7074	PF 100Y BothSpil	516.97	5.41	8.67	8.24	8.70	0.001288	1.51	732.16	789.74	0.33
Reach_1	51.25459	PF 100Y BothSpil	516.97	5.53	8.18	8.17	8.34	0.009203	3.65	407.75	855.07	0.86

Annex-B

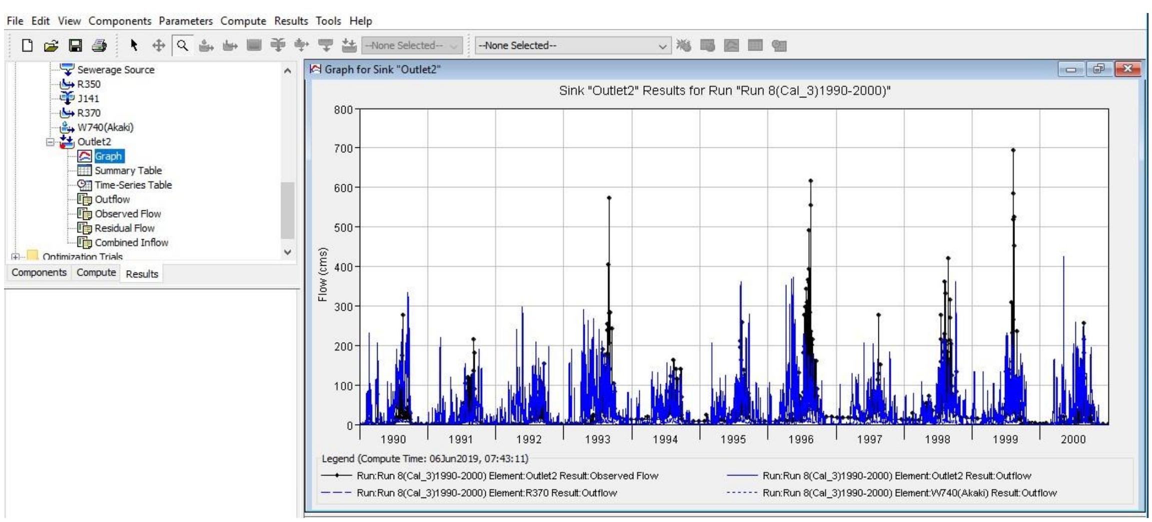
B-1 Graphical HEC-HMS Result after calibration completed for five years



B-2 Graphical HEC-HMS Result after Validation for Two years

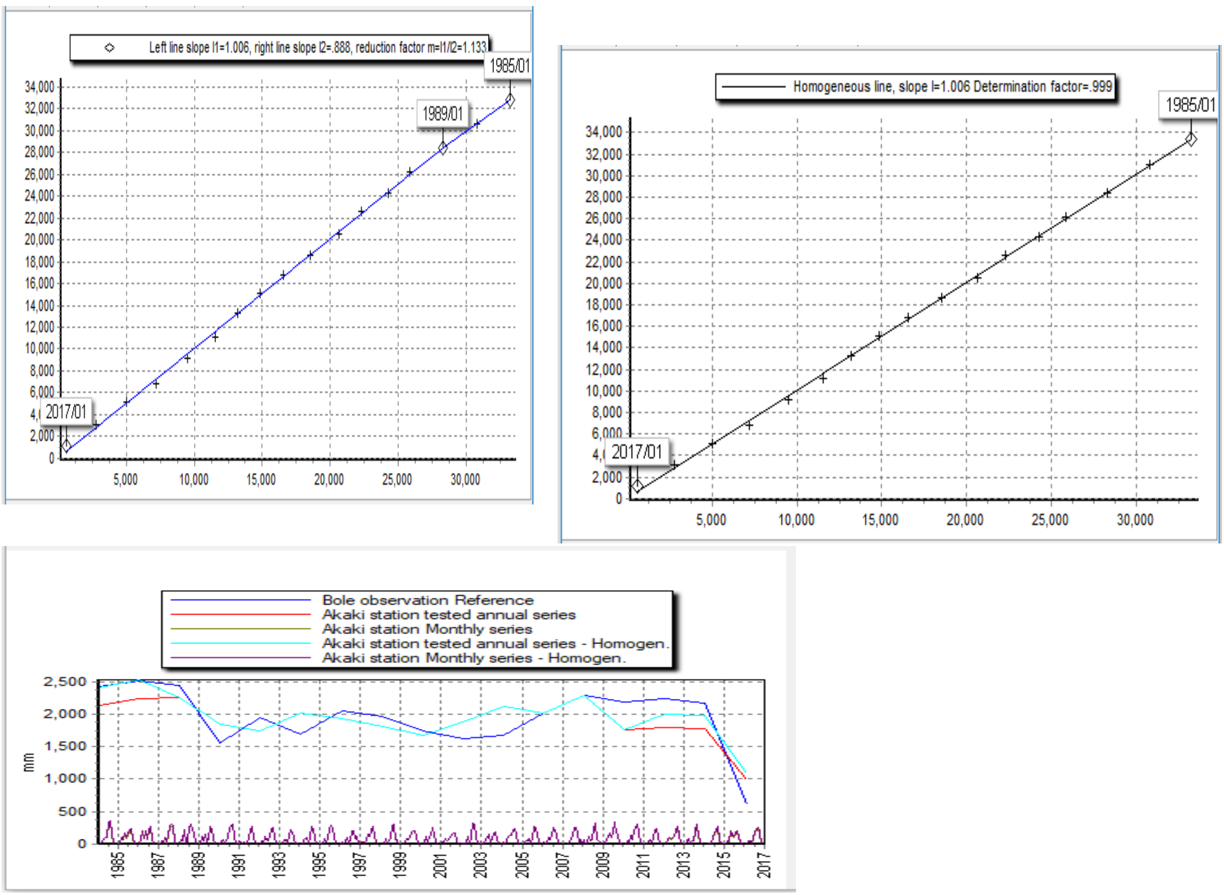


B-3 Graphical representation in calibration process for 1980 up to 2000

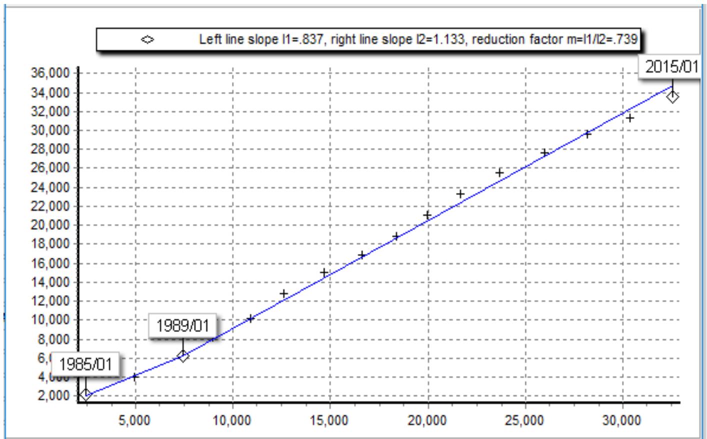


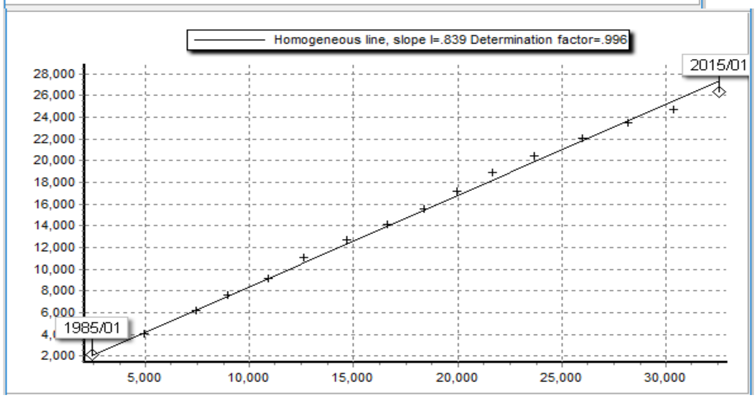
Annex-C

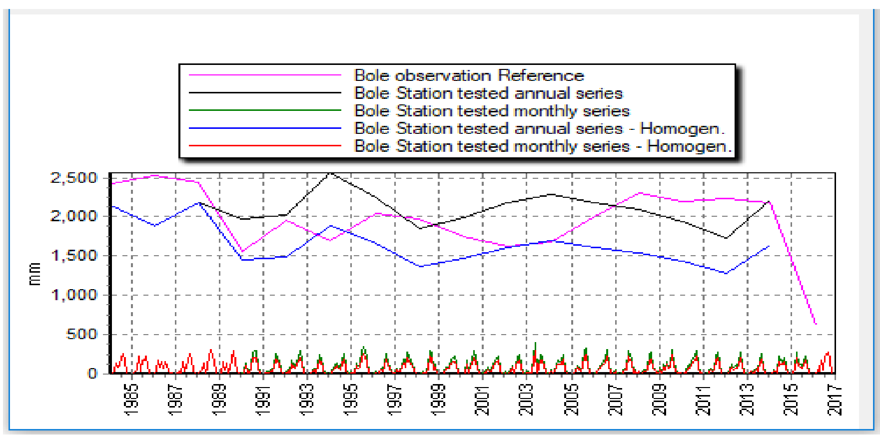
C-1 Double Mass curve of Akaki station with reference station of Bole Observation station before after adjusted and Bole Observation station and Akaki homogeneity test summary



C-2 Double Mass curve of Bole station with reference station of Bole Observation station before, after adjusted and Bole Observation Station and AA Bole station Homogeneity test summary







C-3 Double Mass curve of Intoto station with reference station of Bole Observation station before, after adjustment and Bole Observation station and Intoto station homogeneity test summary