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SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

"Flood Mapping and Mitigation Measures for Upper Awash River"

A Thesis in Hydraulic Engineering

By

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A Thesis

Submitted in Partial Fulfillment of the Requirements for the Degree of Master of Science

The undersigned have examined the thesis entitled '**Flood mapping and mitigation measures for upper Awash River**' presented by **Yonata Belina**, a candidate for the degree of **Master of Science,** and hereby certify that it is worthy of acceptance.

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I confirm that the research work titled "**Flood mapping and mitigation measures for upper Awash River**" is my work. The work has not been presented elsewhere. Where material has been used from other sources it has been properly acknowledged.

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ABSTRACT

This study focuses mainly on the flood mapping and proposes mitigation measures by using the application of arc-Gis based hydrologic model (HEC-HMS) for estimation of peak discharge and hydraulic modeling (HEC-RAS) for estimation of water surface profile for different return periods. The DEM (30m*30m) of the study area was used to extract the physical characteristic of a watershed using Geo-Spatial Hydrologic modeling extension HEC-GeoHMS. The satellite data analysis was used in the calibration and validation of the Awash Belo flooded area. The six indices were selected in water area extraction from Landsat-7 Geotiff like NDWI, AWEI, NDVI, NDMI, WRI, and NDMI. Among them, NDWI was the best fit with simulated flow and used for both calibration and validation for events of 9/3/2001 and 8/25/2001 respectively.

Finally, different mitigation alternatives were proposed to take corrective measures for Awash Belo flood-damaged area. The appropriate mitigation measure for the Belo flood plain was selected based on three main criteria after their design like flood risk, Environmental impact, and balance of the two criteria. Even if, it was difficult to decide the efficient alternative because some alternative contradicts each other, the multi-criteria decision analyses were used. The two-stage channel modification was best in the risk-oriented decision but least in environmentally oriented decisions. The one-stage channel modification was good in environmental impact-oriented whereas least preferred in risk-oriented. The levee in combination with channel modification was good in environmental oriented but, less in risk-oriented. The balance of all environmental and flood risk gives higher priority for levee construction and less priority for channel modification.

The peak discharge developed at upstream flood plain was 82.8 and 100 m^3/s for 50 and 100 years respectively. The area inundated by this peak was 500.6 ha and 505 ha for 50 and 100 years return period respectively without any protection measure. The area falls to 52.27 and 57.28 hectares when the levee construction mitigation measure was applied for 50 and 100 years return period. Scenario analysis confirms the superiority of levee construction to other proposed alternatives with the highest rated value of 0.733 as well as strong weakness of two-stage channel modification with the least preferred rated value of 0. Although increasing the size of the channel or decreasing its roughness can lead to a reduction in flood level because of additional channel capacity, channel modification could also have a negative effect such as increasing inflow velocity and stream power. Therefore, from the different alternatives, the construction of a levee has been proposed as the best alternative remedial measure to protect the flood from the damaged area.

Keywords: HEC-HMS, HEC-RAS, HEC-GeoHMS.HEC, NDWI, AWEI, WRI

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LIST OF ABREVATIONS

DEM: Digital Elevation Model

GIS: Geographic Information System

HEC-RAS: Hydraulic Engineering Center for River Analysis System

HEC-GeoRAS: Hydraulic Engineering Center River geometry analysis system.

NMA: National Metrological Agency

TIN: Triangular Irregular Network

USACE: United States Army Corps of Engineers

USGS: United States Geological Survey

DPPA: Disaster Prevention and Preparedness Agency

PMF: Probable maximum flood.

SCS-UH: Soil Conservation Service Unit hydrography.

MoWR: Ministry of water resource.

NDWI: Normalized Difference Water Index

NSE: Nash-Sutcliffe coefficient of efficient

NDVI: Normalized Difference Vegetation Index

GeoHMS: Geospatial Hydrological Modeling

HEC-HMS: Hydraulic Engineering Center-Hydrological Modeling System.

IDF: Intensity Duration Frequency

CN: Curve Number

NDMI: Normalized Difference Moisture Index

TOPSIS: Technique of Order Preference by similarity to Ideal Solution.

MCDA: Multicriteria Decision Analysis.

AWEI: Automated Water Extraction Index

WRI: Water Ratio Index

CHAPTER 1 INTRODUCTION

1.1 Background

Human existence involves exposure to many hazards. Since the beginning of natural civilization disasters, such as floods and earthquakes have threatened humanity. With technological progress, new technologies and corresponding hazards were introduced. Since the industrial revolution, technical hazards, such as industrial accidents, and airplane crashes also disrupt society regularly [1].

Topographically, Ethiopia is both mountainous and lowland country. It is composed of nine major river basins, in which the drainage systems of which originate from the centrally situated highlands and make their way down to the peripheral or outlying lowlands. Especially during the rainy season (June-September), the major perennial rivers as well as their numerous tributaries forming the country's drainage systems carry their peak discharges[2].

A flood occurs when the surplus flows exceeding the carrying capacities of the stream. This excess flows results in an over flanking of banks and spread over the area and become damage to life and properties. The flood might have resulted from excess rain, rising groundwater dam failure, etc.

Ethiopia experiences two types of floods, flash floods, and river floods. Most of the floods in the country occur due to river overflow following extended rainfall causing inundation of areas along riverbanks in lowland plains. Among the major river, flood-prone areas are parts of Oromia and Afar regions lying along the upper, middle, and downstream plains of the Awash River; parts of Somali Region along the Wabi shebelle, Genale, and Dawa rivers; low-lying areas of Gambella along the Baro, Gilo, and Akobo rivers; downstream areas along the Omo River in SNNPR and the extensive floodplains surrounding Lake Tana and the banks of Gumara, Rib and Megech rivers in Amhara [3].

Flash floods, which occur in lowland areas when excessive rainfalls in the highlands, are also frequent in central and western Tigray, North and South Wollo, West Gojjam and Oromia zones (Amhara), North and West Shewa (Oromia), Wolayita, Hadiya, Guraghe, and Sidama zones (SNNPR) and Dire Dawa and Jijiga Towns (DPPA, 2013 Flood Alert). Such flash floods are characterized by sudden onset with little lead time for early warning and often resulting in huge damages.

Because of flooding in these regions, severe damage to agricultural field and humans has been occurring and causing damage especially in Illu, Sebeta Hawas and Ejerie, Wereda of Oromia, Wolayta mainly Humbo wereda of South Region, Dire Dawa Administrative Council, Gambella Region, Dalocha Wereda of Afar Region, Bahir Dar of Amhara Region (DPPA,2013 Flood Alert). In this regard, it is not only those areas which according to NMA (2013) forecasts and foreign media warnings get direct heavy rains that are affected by flooding but also the eastern low-lying areas of the country which become the victims of the flooding.

The rains have caused most rivers to swell and overflow or breach their courses, submerging the surrounding floodplains, which are mostly located in the outlying pastoralist regions of the country.

1.2 Statement of the problem

Flood is one of the widespread natural disasters and results in loss of life and properties in the world. This natural disaster is not exceptional for Ethiopia because it affects many lives and livelihoods in some parts of the country where there is flood vulnerability. The flooding in Ethiopia is directly linked with the natural topography of the country. The drainage lines of Ethiopia start from the highland mountains of the country and make their way towards the lowlands. Therefore, when there is heavy rain especially during rainy seasons, the excess flood formed to make their ways toward the lowlands of the country and results in damage to population and properties. The problem of river flooding in Ethiopia is getting more and more acute due to human intervention in the fragile highland areas at an ever-increasing scale (UNOCHA, 2006). The worst scenario of flood in Ethiopia occurred in the summer of 2006 as the result of prolonged and intensive rainfall which resulted in flash floods and overflow of river and dams affecting199,900 people in eight regions of Ethiopia resulting in loss of lives, damage of properties and destruction of livelihoods of tens of thousands of people. According to a preliminary document, the overall resources needed to withstand the disaster, both for emergency relief and rehabilitation was estimated to be US$60,907,574 [40].

Ethiopia's eastern Oromia region was the worst hit in the giant Horn of Africa nation, with 97,000 people affected, of which 37,000 lost their homes (UNOCHA, 2006). The floods swamp large areas of cropland in this region. According to the 2006 Flash Appeal (Joint Government and Humanitarian Partners,2006), a total of 524,400 remain vulnerable to flood disaster throughout the country of which 199,900 people are affected by the flood disaster in various areas, in Oromia Region 61,300 people are vulnerable and 21,900 were affected by the 2006 flood event.

Awash River is one of the rivers which cause flooding at Ilu, Sebeta, and Ejerie floodplain. The river conveys high runoff from upper catchments and local rainfall on the floodplain to resulting in flooding problems. Flooding due to outbursts of banks of Awash River and overflow from the Koka reservoir led to the displacement of 40,000 people in Wonji and Metehara in1996 (DPPA,2006). Additionally, flooding from the upper basin of the Awash River affected 14 peasant associations (PAs) in Illu, Sebeta Hawas, and Ejere woredas of the West Shewa zone. The flood was reported to have affected a total of 14,790 people out of which 2052 people were displaced and forced to live in temporary shelters. The Illu-Sebeta and Ejerie floodplain is affected by 4,506 hectares of different crops that have been damaged and livestock disease outbreaks are anticipated by floods as it is located at a lower level in the river [4].

The specific issue of this thesis to compute flood inundation area for different return periods and propose mitigation measures to protect the flooded area from flooding based on the computed magnitude of flood surface profile and suitability of topography at Belo flood plains.

1.3 Objectives of the study

1.3.1 General objective

The main objective of this study is to map flooded areas and propose mitigation measures for the upper Awash River flood plain.

1.3.2 Specific objectives

- To predict the chance of flooding at a particular cross-section.
- To simulate flood profiles at the study reach.
- To map flooded areas for the various return period.
- To develop optimal mitigation measures

1.4 Research questions

What is the efficient mitigation measure for the study area?

What is the flooded area for different return periods?

What is the adequacy of the existing section to carry a flood of various magnitude?

What is the highest flood peak discharge for different return periods?

1.5 Outline of the thesis

This thesis is structured as follows. In chapter 1, introduction on the influence of flooding in general and flooding in the upper Awash River basin in particular; and the necessities of doing on proposing mitigations there was introduced. Additionally, the research objective, research questions, and thesis outline are presented in this chapter. In chapter 2, Flood peak estimation using hydrologic modeling (HEC-HMS) and Water surface generation using hydraulic modeling (HEC-RAS) are introduced. Besides, the structural and non-structural flood mitigation measures are described. In chapter 3, different hydrological data analyses like filling a gap, consistency checkup, outlier's tests, and areal rainfall estimation is presented. In chapter 4, the research method and methodology used for addressing the objectives are presented. In Chapter 5, the result obtained using the methods are discussed. In chapter 6, the research conclusions and recommendations are introduced.

1.6 Significance of the study

The study plays a vital role to the following concerns: As the traditional way of flood forecasting system are causing many hazards on the community, as a result, this study will be dedicated to flood forecasting for habitant of Ilu flooded area in Awash River Basin using HEC-HMS model for 2,5,10,25,50 and 100 years return period. Also, this thesis very significant to the following issues:

- A flood inundation map generated in this study is used as flood hazard zones identifying tools in planning and developing any water-related projects along the Awash River course crossing the area.
- Flood prone areas delineated at different return period also used to warn habitant of Ilu wereda in the coming 2, 5,10,25,50, and 100-years.
- The results obtained in this thesis works can also be inputs for other studies like social and economic impacts of flood damage that helps in knowing how the community was suffered by the flooding.
- Again the output of this work can also be a benchmark in knowing the impending flood from the upstream habitats to take measures for the downstream habitat in the next return period.
- Control flooding during important farming periods by applying mitigation measures.
- Future flood-related researches in the study area

CHAPTER 2 LITRETURE REVIEW

2.1 Rainfall-runoff process, runoff hydrograph, and peak discharge

Rainfall-runoff process

The runoff generated by a watershed depends on the amount of precipitation occurring over a watershed. The rainfall hyetograph can be separated into three time-dependent parts: initial abstraction, losses, and rainfall excess [6]. Only rainfall excess generate to the runoff hydrograph**.**

When the rainfall on the ground, losses take place before it results in runoff. The **initial abstraction** is precipitation that occurs before the start of the runoff. Interception is the part of rainfall captured temporarily by leaves and stems. The portion of rainfall that enters the soil is known as infiltration. The surface depression storage is the part of rainfall that is stored by the puddles, lakes, and ponds. Evaporation is the portion of water that is evaporated from soils and open water bodies. The initial abstraction is that part of precipitation, which has no contribution to the direct runoff. The value of initial abstraction depends on the type of soil of the drainage area being studied, land use, and land cover. When the initial abstraction is satisfied, the ***excess rainfall*** become appears on the surface as direct runoff. Even if the direct runoff is starting the Losses never stop and this is called the loss function. A UH can be used as a transfer function to transform rainfall excess into a direct runoff. A UH is the hydrograph that results from 1 inch or 1mm of excess rainfall falling uniformly over the watershed at a uniform rate during the specified period [6].

Runoff hydrograph

The runoff hydrography is a time change of river discharge at a specified stream cross-section for a particular storm. The runoff hydrograph can be separated into two parts direct runoff, and baseflow [6].

Peak discharge

Peak discharge estimates are often needed at ungauged sites where no observed flood data are available.

The peak flood flow is the maximum expected flow at a certain location for a given frequency. According to [7], peak flood flows depend on the catchment area, the slope of the main channel, the basin shape factor, the hydrologic region, and the return period.

Potential extreme peak discharges are estimates of the highest peak discharges expected to occur at a certain location and, according to[7], are explained mostly by the area of the corresponding catchment and by the hydrologic region where the catchment is located[8].

2.2 Hydrologic and HEC-HMS modeling

Hydrological modeling is the use of physical or mathematical techniques to simulate the hydrologic cycle and its effect on watersheds. Hydrologic models are simplified, conceptual representations of a part of the hydrologic cycle. They are primarily used for hydrologic prediction and for understanding hydrologic processes. (Wikipedia, the free encyclopedia) .Hydrologic models attempt to simulate the rainfall-runoff process to tell us **"how much water, how often."** They use rainfall information or simulations to provide runoff characteristics including peak flow, flood hydrograph, and flow frequencies[9]**.**
HEC-HMS consists of separate models of the major hydrological processes and transports. It consists of runoff volume models, models of direct runoff (overland flow and interflow), base flow models; channel flow models.HEC-HMS gives flexibility to the user by providing each component with a suite of models. The user can choose a suitable combination of models depending on the availability of data, the purpose of modeling, and the required spatial and temporal scales.

2.2.1 Initial and constant loss method

The Initial and constant loss method is a mature model that has been used successfully in hundreds of studies throughout the USA. Due to the lack of direct physical relationship of parameter and watershed properties this model is more appropriate for the Awash Belo study area because of insufficient direct measured parameters in the catchment.Unlike, of, of SCS curve number loss method the predicted values are following classical unsaturated flow theory in this model which made it adaptable to the Awash Belo Study area [10].

2.2.2 SCS-UH transform method

The SCS adopted a unit hydrography model; this model is added in HEC-HMS. This unit hydrography is generated from gaged Rainfall and runoff from small agricultural areas.[11] and the [12] explain unit hydrography in detail. The basic Concepts and Equations behind this model is a non-dimensional, single-peaked UH. This dimensionless UH, expresses the UH discharge, Ut, as a ratio to the UH peak discharge, Up, for any time t, a fraction of Tp, the time to UH peak. This equation shows the relation between peak unit hydrography and peak time or the time when the peak discharge is coming.

$$Up = C\ A\ /\ Tp \text{...} \textbf{Equation 2.1}$$

In which A = watershed area; and C = unit conversion. The time to peak is equaled with half of the excess rainfall duration plus basin lag time: $Tp = \frac{\Delta t}{2} + tlag$ in which Δt = surplus precipitation time; and t_{lag} = the basin lag time. [Δt, that is less than 29% of **t** lag must be used [13]. When the lag time is given, HEC-HMS solves Equation above to compute the time of peak and peak discharge for unit hydrography. With Up and Tp known, the UH can be found from the dimensionless form, which is included in HEC-HMS.

2.2.3 Monthly constant base flow method

This is best estimated, empirically with measurement of channel flow when storm runoff is not occurring but this might be difficult with a limited period of the thesis. In absence of such records, field inspection may help to establish the average flow.

2.2.4 Lag routing method

The Lag method is a special case of other routing methods. The outflow hydrography in this case is inflow hydrography. The flows are not attenuated, so the shape is not changed. This model is widely used, especially in the urban drainage channel. Mathematically the downstream hydrography is computed as follows.

$$Ot = \begin{cases} I_t & t < lag \\ I_{t-lag} & t \geq lag \end{cases} \text{...} \textbf{ Equation 2.2}$$

Where O_t=out flow hydrography ordinate at time t; I_t inflow hydrography ordinate at time t; and lag=time by which inflow ordinate are to be lagged [14].

2.3 HEC-RAS hydraulic modeling

From much hydrologic software, HEC-RAS (Hydrologic Engineering Center - River Analysis System) is a good choice for 1D food plain modeling[15]. Its ArcGIS application companion HEC-GeoRAS makes it easier to gather physical data required by the model from a high-resolution DEM [19].

2.3.1 Governing equation of the model

Energy equation

For open channel flow, the energy head has three main components like of head due to channel bottom datum, head from flowing water, and heads from the velocity of flowing water [16]:

$$H = y + z + \propto \frac{V^2}{2g}$$ **Equation 2.3**

Where H = Energy head (m)

z = Reduced level for channel bottom (m)

$\propto$ =Velocity variation coefficient

The energy head is the sum of the potential head, velocity head, and pressure head. The change in energy head between the two cross-sections is head loss:

$$H1 = H2 + hL$$ **Equation 2.4**

$$h_L = H1 - H2$$ **Equation 2.5**

Where H_1 = energy head at cross-section 1 (m)

H_2 = energy head at cross-section 2 (m)

h_L = energy head loss (m)

The head loss between the adjacent cross-sections is the sum of friction head loss and flow contraction/expansion head loss. The friction losses arise from shear stress between the flowing water and channel bed.

$$h_f = LS_f$$ **Equation 2.6**

Where: h_f = friction head loss (m)

L = Discharge Weighted distance between adjacent cross-sections (m) Contraction/expansion head losses arise as a result of the formation of eddies wherever there is a contraction or expansion of the channel.

$$ho = C\left[\left(\frac{\propto 2V2^2}{2g}\right) - \left(\frac{\propto 1V1^2}{2g}\right)\right]$$ **Equation 2.7**

Where: ho = contraction or expansion head loss (m)

C = contraction or expansion coefficient.

2.3.2 HEC-RAS parameters

There are different parameters used in this model for hydraulic analysis of river geometry and its flood plain.

These parameters are used to generate a series of channel cross-sections along the direction of the stream. This channel cross-section is divided into segments of the left floodway, main channel, and right floodway.

At every cross-section, HEC-RAS assumes that energy is constant and that the velocity vector is perpendicular to that cross-section[16].

2.3.3 Water surface profile estimation

For steady, gradually varied flow, the primary procedure for computing water surface profiles between cross-sections is called the **standard step method.**

The procedure is summarized below[16].

(i) Assume a water surface elevation at the upstream or downstream cross-section based on flow type.

(ii) The area, hydraulic radius, and velocity based on the cross section profile should compute.

(iii) Estimate the velocity head and conveyance for each cross-section.

(iv) Estimate the required losses for both contraction/expansion and friction losses with corresponding parameters.

(v) Apply the Energy equation for the water surface elevation at the upstream Cross-section or downstream cross-section.

(vi) Evaluate/Compare/ the estimated water surface profile with the one assumed in step (i).

(vii) Do again Step (i) through (vi) up to the assumed value and estimated water surface elevations are within accepted tolerance.

2.4 Floodplain mapping

Flooding, like a major natural disaster, affects many parts of the world including developed countries. Due to this natural disaster, billions of dollars in infrastructure and property damages and hundreds of human lives are lost each year.

The floodway is the channel of a stream plus any adjacent flood.

Two types of floodplain inundation maps, flood-prone areas, and flood hazard maps have been used.

2.4.1 Flood prone area maps

This map shows areas likely to be flooded by their proximity to a river, stream, bay, ocean, or other watercourses as determined from readily available information.

2.4.2 Flood hazard maps

This map shows the extent of inundation as determined from a thorough technical study of flooding at a given location. Flood hazard maps are commonly used in floodplain information reports and require updating when changes have occurred in the channels, on the floodplains, and in upstream areas. These changes include structural modifications and channel or floodplain modifications in upstream areas. Development of new buildings on the floodplain, obstructions, or other land-use changes can affect the stream discharges, water surface elevation, and flow velocities, thereby changing the elevation profile defining the floodplain.

2.5 Flood mitigation measures

Mitigation is defined as a continued action taken to minimize or eliminate risk to property or humans. Mitigation is the least visible concept, but it plays an important role in protecting communities from disaster. The decision made about the mitigation plan will affect the safety of an area in the future when faced with disaster.

Types of flood mitigation

The main purpose of flood mitigation is to minimize or prevent problems raised from flood disasters in the flooded area. There are two main types of mitigation measures[17]. Those are:

2.5.1 Structural mitigation Measures

This is used to control flood by using engineering structures like a levee, Dykes, dam, Diversion channel, channel modification, etc.[18].

2.5.2 Non-structural mitigation measures

This is not structural measures rather it focuses on policy and legal-related works. It includes activities like land management. These methods can simply be applied by improving the performance of the channel.

2.6 Calibration and validation of the HEC-HMS model

The successful application of the hydrologic watershed model depends upon how well the model is calibrated which in turn depends on the technical capability of the hydrological model as well as the quality of the input data. HEC-HMS watershed model is calibrated for the event-based simulation. The objective of the model calibration is to match observed simulated runoff volumes, runoff peaks, and timing of hydrographs with the observed ones.

2.6.1 Comparison criteria for calibration and validation

The model must be evaluated on the extent of its accuracy, consistency, and adaptability. A forecast efficiency criterion is therefore necessary to judge the performance of the model. Assessing the performance of a hydrologic model [19] requires subjective and/or objective estimates of the closeness of the simulated behavior of the model to observations. The performance of the HEC-HMS model was evaluated subjectively by following the basic approach of assessing model efficiency by visual inspection. It enabled examining systematic behavior, over- or under prediction, and dynamic behavior, timing, rising limb, and recession curve, of the simulated and observed hydrographs visually during calibration and validation of the model. In model calibration, the model parameters have to be adjusted until the observed natural system output and the simulated model output show an acceptable level of agreement. The goodness of fit is always evaluated through an objective function that is selected based on several criteria. These criteria should be selected properly to evaluate different aspects of the hydrograph. In this study the criteria that were considered in selecting the objective function are as follows:

Nash-Sutcliffe efficiency: The Nash-Sutcliffe efficiency (NSE) is used to evaluate the overall agreement of the shape of the simulated and observed hydrograph. NSE measures the efficiency of the model by relating the goodness of fit of the simulated data to the variance of the measured data.

This objective function can vary between 1 and $-\infty$ and performs best when a value of 1 is generated. Besides, due to frequent use of this objective function, it is known that when values between 0.60 and 0.80 are generated, the model performs reasonably well. Values between 0.80 and 0.90 indicate that the model performs very well and values between 0.90 and 1 indicate that the model performs extremely well[19].

Coefficient of determination

Coefficient of determination (r^2): it expresses the measure of how good trends in the measured data are reproduced by the simulated results over a specified period and for a specified time step. The range of values for r^2 is 1.0 (best) to 0.0.

2.7 Awash master plan

To ensure orderly, effective, and economical progress in development, it should form the start be carried through following a "Master Plan" which, embracing the three phases mentioned in the table below should be laid down as soon as the further investigations of all kinds, recommended, have made sufficient progress. This Master Plan should be flexible in application, and design of details; it should be periodically reviewed and whenever necessary revised, in the light of further knowledge and experience. Once the necessary further surveys and investigations have resolved the present remaining uncertainties affecting water control and irrigation development, it will be possible to prepare a "Master Plan", at least in its main outlines, for the whole Awash Valley, in such a form as to fit into the national development plan for Ethiopia and make the optimum possible contribution to the country's economy. In this, of course, the financing of development will have to be taken into account, with a realistic schedule of investment[20].

Within the framework of the master plan, the timing of the successive stages can be flexible, to be suited to circumstances. If the Imperial Government wishes to develop to the full the potential resources of the Awash Valley most effectively and beneficially possible, two things are particularly essential. In the first place, it will be necessary to prepare a "Master Plan" of development, in contrast to the somewhat haphazard methods of development which have been applied hitherto. ©tis plan will ensure that each item of development is designed to fit in properly with others in the coordinated whole, giving the most efficient use of the resources involved and the funds invested, which are national assets of the country as a whole. Further, it will make sure that the development of any item in the earlier stages will not in any way cause difficulties in the subsequent development of other items at later stages [20].

Table 2-1: Proposed development of irrigation by projects and stages [20]

Project	Present Area	First Stage		Second Stage		Third Stage	
		Additional	Total	Additional	Total	Additional	Total
UPPER VALLEY							
Wonji, Genet, and	6,600	0	6,600	0	6,600	0	6,600
others Nuri, Eva	50	5,350	5,400	0	5,400	0	5,400
Total Upper Valley	6 650	5,350	12,000	0	12,000	0	12,000
MIDDLE VALLEY							
Abadir – Metehara	850	9,650	10,500	0	10,500	0	10,500
Kesem - Kebena	650	1,350	2,000	0	2,000	15,550	17,550
Melki Sedi	0	6,000	6,000	2,550	0,550	0	8,550
Amibara - Angelele	50	950	1,000	15650	16,650	0	16,650
Bolhamo	0	0	0	2,500	2,500	6400	
							8,900
Karo Gala	0	0	0	10,650	10,650	12,150	22800 22,800
	1550	17,950	19,500	31,350	50,850	34,100	84,950
Lower plains							
Dubti	4,000	5,050	9,050*	0	9,050	0	9,050
Small riverside	0	0	0	0	0	3,700	3,700
areas	500	10,450	10,950*	5,400	16,350	0	16,350
Dit Bahri	11,600	300	11,900	14,000	25,900	0	25,900
Asayita Delta etc.	0	0	0	0	0	11,300	11,300
Old Awash							
Total Lower Plains	16,100	15,800	31,900	19,400	51,300	15,000	66,300
Grand Total	24,300	39,100	63,400	50,750	114,150	49,100	163,250

2.8 Previous studies performed in the upper awash basin

The influence of the rainfall intensity on the observed river discharges is also recognized in other studies performed in the Upper Awash catchment (Israel, 2011;[21];[22]) and flood risk damage analysis was done by ([23];[24]).

2.8.1 Rainfall-runoff model using SCS numbers study at upper Awash River

In [25] a water balance for the catchment upstream of Melka Hombole is made with the use of a rainfall-runoff model. There are three methods used to find a rainfall-runoff relation. One relates rainfall to a retention parameter and runoff; the other relates the retention parameter to the Soil Conservation Service Curve Number. The retention parameter is the maximum potential difference between rainfall and runoff from the time that the storm begins, used in the daily curve number calculation. Two methods of calculating the optimal retention parameter are used. One lets the retention parameter vary with the soil moisture and the other lets the retention parameter vary with the plant evaporation. After validation and calibration of the model, all three methods show other results for the water balance of the watershed, although predicted streamflow shows agreement between the methods. The modeled components of the water balance are shown in the Figure below. All three methods show that the majority of the water balance component is evapotranspiration. The difference between the calculated components of baseflow and surface runoff in the two methods that use the curve number is remarkable. The method that uses plant evaporation (middle graph) calculated 24% of the precipitation to be surface runoff and 2% base flow. While the method using soil moisture finds a 3% surface runoff and 21% base flow (right graph). The water that drains from the shallow groundwater bodies in the catchment feeds the base flow component. [25] Found two papers that back up both of these scenarios. A water balance derived from a lumped conceptual rainfall-runoff model of the same area from Moreda (1999) the base flow accounted for 2% and lateral flow 13%. This agrees with the plant evaporation method that finds base flow as a relatively small contributor. Another study by Chekol (2006) that also uses the soil moisture method finds base flow as the dominant contributor to the water balance with a percentage of 9.8% versus 6.8% for surface runoff[26].

2.8.2 Flood prediction model and soil properties study at upper Awash River

All the studies performed in the study area recognize the influence of vertisols on the rainfall-runoff relation. The floodplains that are the main focus of this survey are built up from fine-grained sediments. In these fine-grained deposits, the vertisols develop. In the dry season large deep cracks develop in the clay-rich soils due to the shrinkage. During the wet season, the soil will become saturated and will have low hydraulic conductivity [22]. [22] also simulated streamflow in the Upper Awash catchment to predict floods. He found over- and underestimations for the small and main rainy seasons. The reasons his model does not match the observed streamflow data are caused by the absence of certain data. Soil properties play an important role in the rainfall-runoff relationship and need to be investigated in more detail. The study did divide the catchment into six sub-basins. Often the sub-basins have one rainfall and one discharge station. The extrapolation of the rainfall data from one station per basin can introduce some inaccuracies for simulation of the rainfall-runoff relation ([25] 2011; [22].

2.8.3 Flood prediction using remote sensing study at upper Awash River

[21] Developed a model to predict floods in the Upper Awash catchment. The results of the LISFLOOD model were compared with observed data and calibrated by trial and error. For the collection of soil moisture and evapotranspiration in the studied area satellite data from several sources was used.

This study also finds certain model parameters that influence base flow and certain parameters that influence quick flow or surface runoff. The parameters controlling the flow from the upper to the lower groundwater zone, and the parameter controlling the amount and timing of the outflow from the lower groundwater reservoir, influence the base flow. Whereas the parameters controlling flows influenced by the soil saturation and the parameters are controlling flows of the upper groundwater zones control the surface runoff. With the model, a prediction of flooding of the flood-prone areas is made using a standard precipitation index and a topographical wetness index. Two periods are found to have a high standard precipitation index and excess rainfall in those periods would cause flooding. The periods are between June 13 - 20 and July 27 - August 04.

2.8.4 Flood risk analysis study at upper Awash River

[23] Developed flood frequency analyses for prediction of future flood from Awash Belo gaging station streamflow. This way of flood forecasting will completely result in inaccuracy because he didn’t consider different basin physical parameters. Additionally, it might be difficult to obtain a reasonable result from prediction due to no calibration and validation of the inputs and outputs.[23]didn’t consider risk analyses for resources like structural and content. (Dawit, 2015) only considered with risk related to agricultural resources. Generally, there was no calibration and validation for both hydraulic and

hydrologic analyses/results. Although in this paper he didn't propose any mitigation measure within all result of flood hazard for study area.

2.8.5 Flood damage analyses study at upper Awash River

[24] Developed flood frequency analyses for prediction of future flood from Awash Belo gaging station streamflow. This way of flood forecasting will completely result in inaccuracy because he didn't consider different basin physical parameters. Additionally, it might be difficult to obtain a reasonable result from prediction due to no calibration and validation of the inputs and outputs. Although, as he tries to recommend the area there was no sufficient hydrological data for most of the season, therefore with all this insufficient data it is difficult to do flood frequency analyses. This technique requires a detailed hydrological quality test and sufficient data length. The same as Dawit (2015), his paper didn't have any calibration and validation.

His model included one dimensional, steady flow and with this assumption, it might be difficult to do flood damage analyses rather unsteady flow analysis is best.

CHAPTER 3 MATERIAL AND METHODS

3.1 Description of the study area

Awash Basin is the largest catchment in Ethiopia by area (110000km^2) and average annual runoff (4.6BM^3). The total length of the main course is some 1200, km and is the principal stream of an endorheic drainage basin covering parts of Oromia, Somalia, Amhara, and Afar region [21]. The geographic location of Awash River Basins is between 7‘53’N latitudes and 37‘57’E and 43‘25’E of longitudes [29].

The area is undertaken for this research only covers 2613.75 km^2 of the upstream part of the basins. The study area is located about 51 km to 55 km southwest of the capital Addis Ababa and surrounded by Welmera and Burayu Weredas in the North, Akaki Wereda in the East, Kersana Malima, Tulubolo, Illu and Sebeta Weredas in the south, and Ejere Weredas in the West[23].

The flood-prone areas, particularly in the study area, are around Jigdu Mida and Tulu Mangero Keble’s adjacent to Illu Wereda and Awash Bello and Welena Eka Keble’s adjacent to Sebeta Hawas Wereda and Dibu from Ejerie Wereda situated in Southwest and West Shewa zone, which is located 38 24" 59.9' Latitude and 8 51" 59' Longitude [23].

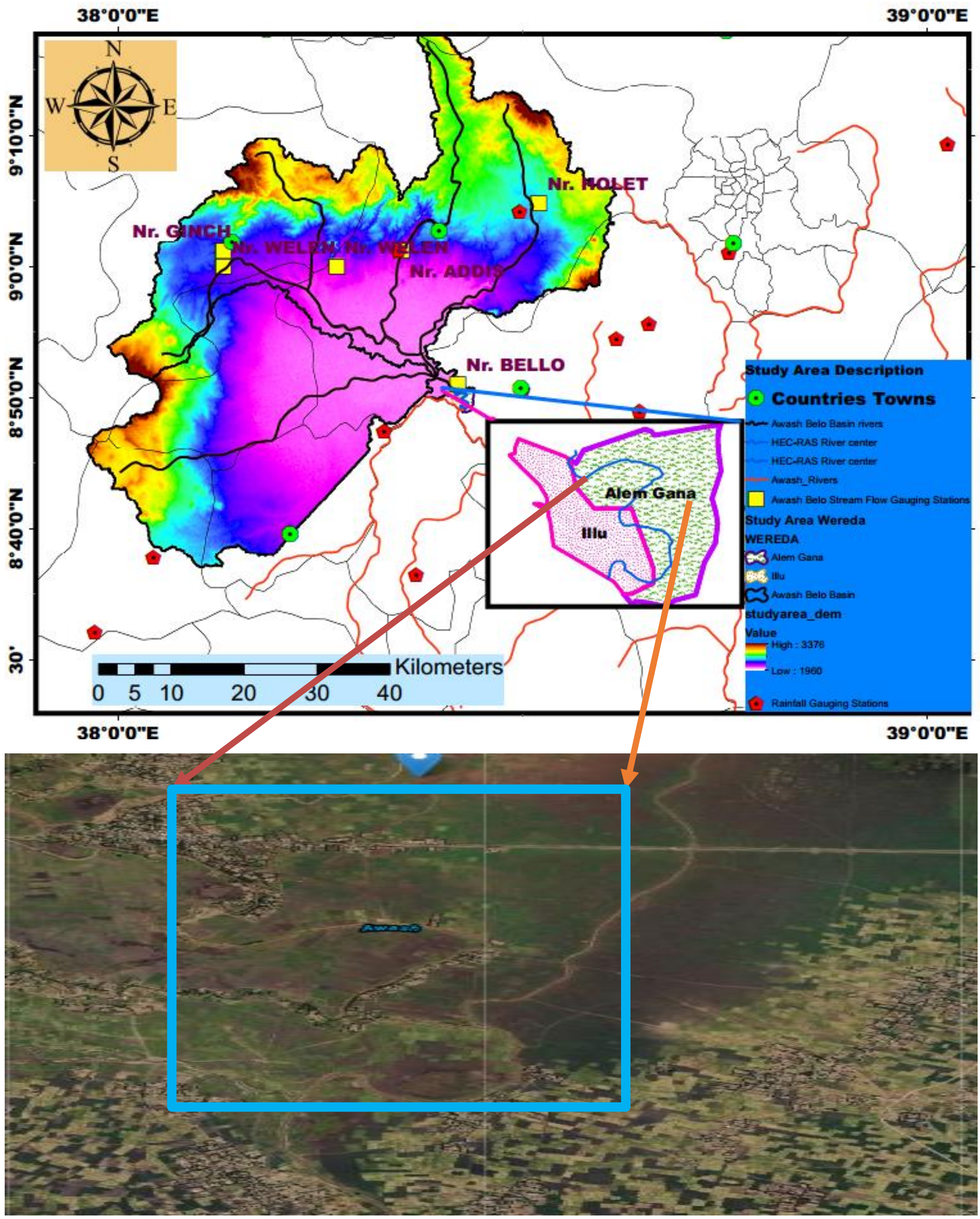


Figure 3-1: Location of the study area

3.1.1 Hydrological and metrological characteristic of the study area

The climate characteristic of the study area is the Woina Dega type of Ethiopian climate[30].

The average temperature of the study area varies from 15^0C to 25^0C with Africa equator sunshine. The study area receives the highest rainfall during the summer season (June to September). The spatial distribution of rainfall of the study area is explained as follows. This is described as the variations of rainfall of the awash basin from the start of the basin with the highest rainfall to the end of the basin with the lowest rainfall at the lowland of the basins. This described as well the spatial distributions of rainfall and anyone can simply visualize where the highest and lowest record exists. The spatial distributions of the basin were not uniform from the upstream highland to downstream low land of the basins.

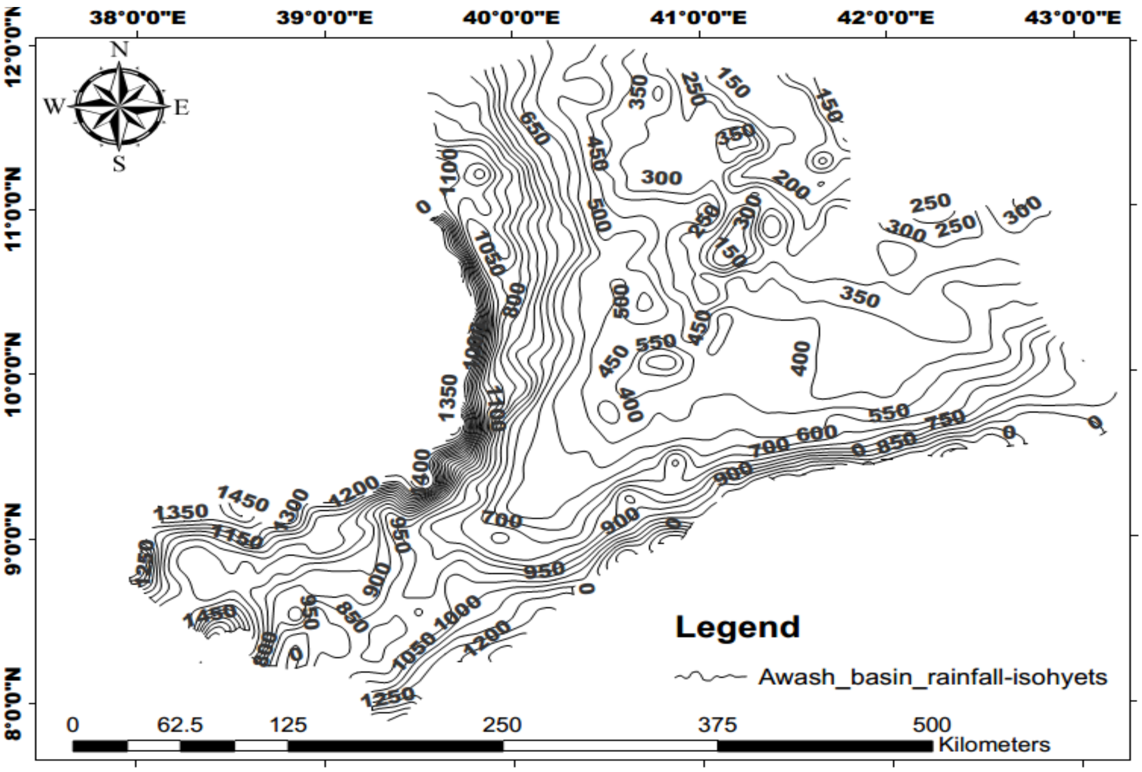


Figure 3-2: Rainfall isohyets for all Awash Basin

3.1.2 Land use land cover

The catchment is densely populated and intensively cultivated by smallholders with crop and livestock production was the main farming system. Most of the areas are occupied by Cropland with shrubland and herbaceous wetland. The main crops grown in the study area are Wheat, Teff, Chickpeas, Lentils, Corn, Sorghum, Maize, and Vetch. According to Wereda Agricultural and Rural Development office,87.2% of land devoted the

Agricultural 4.2% is Pasture,2.9 is forest,1.86% is reserved for the industrial establishment, lake and other bodies of water cover 1.68%, and built land cover 1.28.

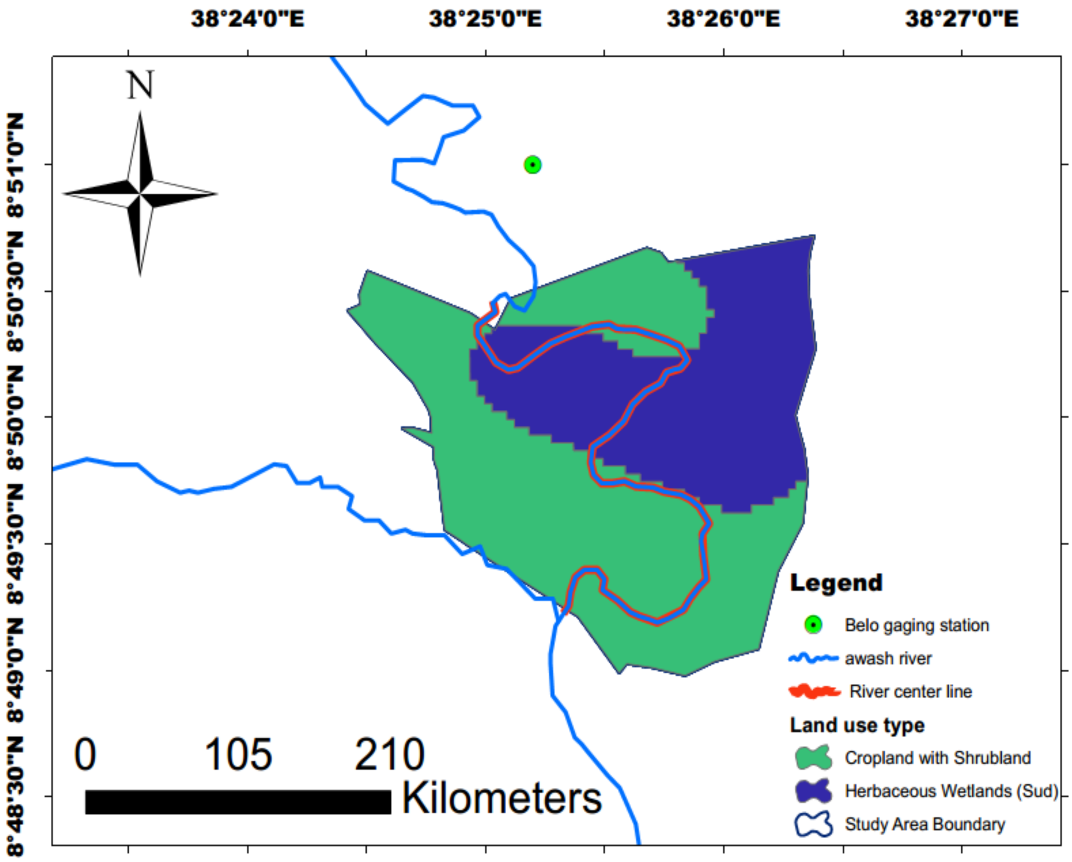


Figure 3-3: Land use type for the study area

3.1.3 The soil of the study area

The soil plays a great role, especially in curve number generation. The parameters of the curve number generated from the soil and land use of the study area were also used to estimate initial abstraction that we use in the initial and constant loss method as initial parameter estimation. There are two types of soil in the study area those soils are chromic cambisols and pellic vertisols. The dominant soil type in the study area is pellic vertisols.

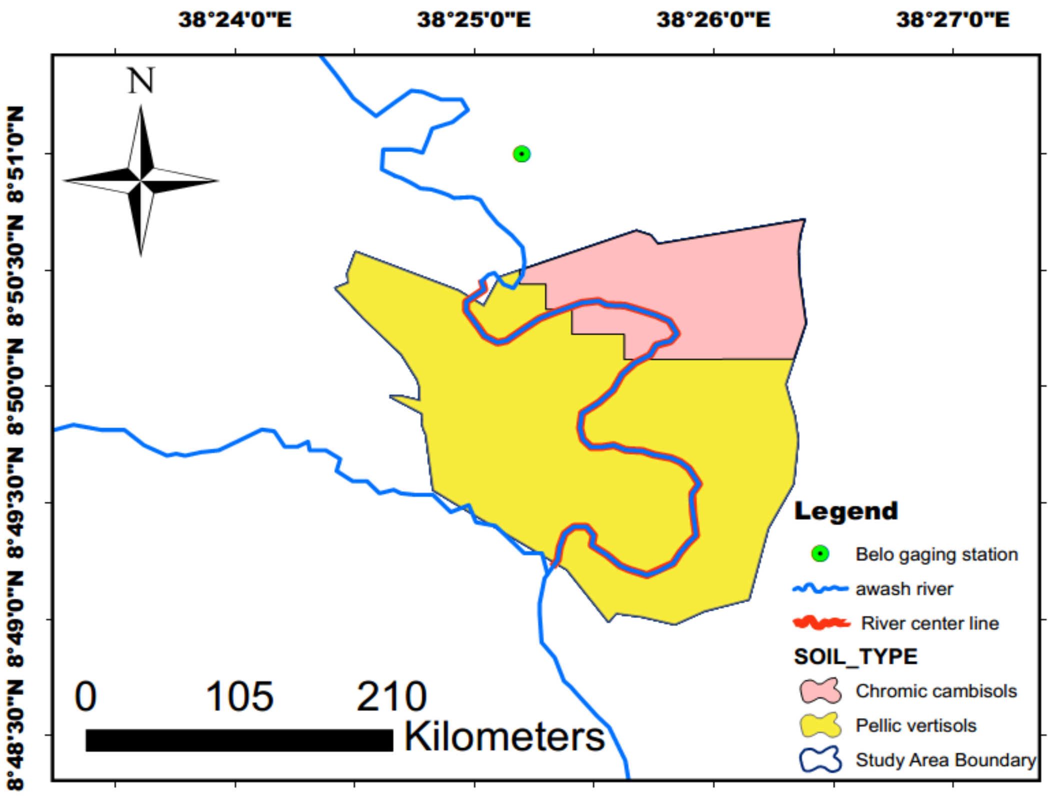


Figure 3-4: Belo flood Plain area land use

3.1.4 Geology of the study area

The geology of the Awash basin is categorized into five as follows and also there is a small portion of fault area at the border of the catchment. Particularly the study areas are covered with lower part code (NQtb) is alkaline basalt and Trachyte and most upper part code (Q) is sand, silt, clay, diatomite, and beach sand geology.

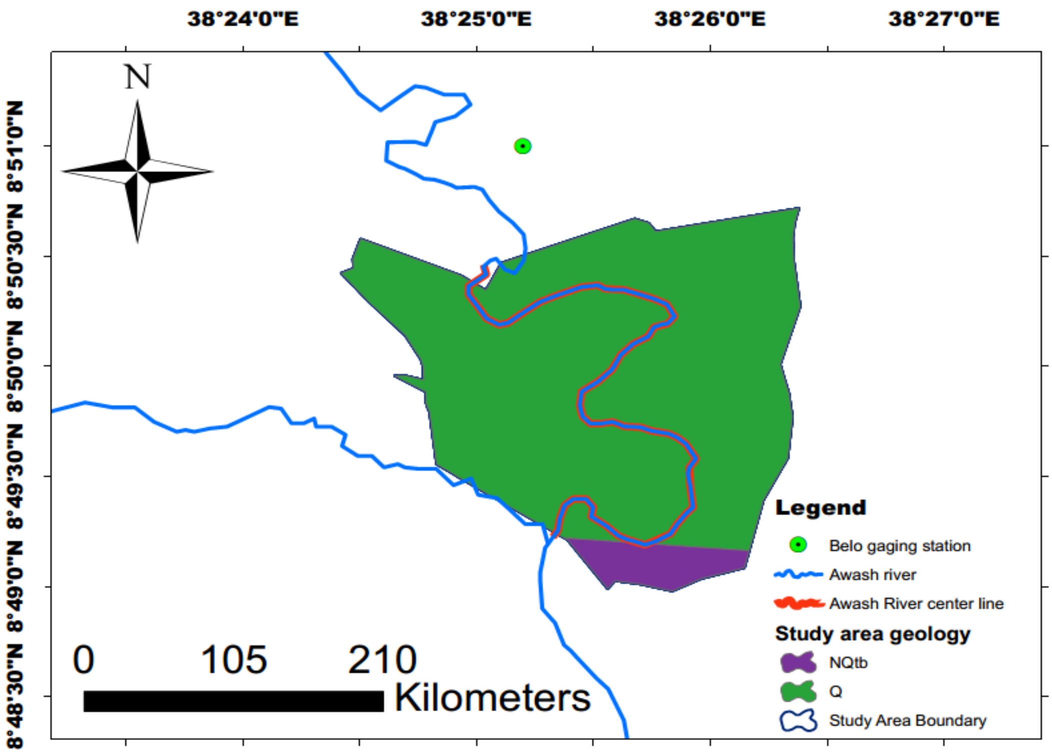


Figure 3-5: Belo flood Plain area geology

3.1.1 The topography of the study area

The topography of the area is undulated or rugged with a difference in altitude of a minimum of 1590m to the maximum altitude of 3612m. The figure below shows the DEM of the study area.

The variations in elevation between the highest and lowest altitude result from high runoff generation which suspects the study area for flood disaster effect.

The slope of the flood plain was approximated to 100 % flat area or gentle slope.

3.2 Material and data used in this research

3.2.1 Data used and sources

This section described the general category of data, purposes of each data, specific data, and their sources for the Awash Belo basin and downstream flood plain. The data described below requires quality tests and detailed analyses to be used for model processing and performance evaluation.

Table 3-1: the data used in this study were summarized as follow

Category of Data	Specific Data Name	Sources	Purposes
Hydrological Data	Gauged flow	MoWR	• For Calibration and Validation
Metrological Data	Rainfall Data	NMA	• For Direct Runoff Processing
Topographic Data	Digital Elevation Model (DEM 30m) Digital Elevation Model (DEM 12.5 m)	USGS Alaska University Facility	• To extract basin characteristics in HEC-HMS. • Flood Channel geometry extraction
River Networks	River for Basin	Ethio GIS	• To identify Study Area flood plain, etc.
Soil Data	Soil for Basin	Ethio GIS	• For Curve Number generation.
Land use and Land Cover	Land use and Land Cover for Basin	Ethio GIS	• For Curve Number generation and Manning Estimation.
River Cross Section	River Cross Section for Flood Plain Reach	Field surveying During Field Inspection	• To Develop River Geometry for HEC-RAS Model
Satellite Data	Landsat-7 and 8 Geotiff	USGS	• For HEC-RAS calibration and validation
IDF Data	IDF of study Area	Ethiopian Road Authority	• Inputs for frequency storm metrological methods.

3.2.2 Software used in this study area

This section includes different software and interfaces with their purposes and sources. The listed software and interface in these sections were played a great role in this thesis based on their importance, purpose, and availability.

Table 3-2: Software used in this Study

Name	Purpose	Source
ArcGIS 10.3	• To prepare river geometry • To map flooded area • To delineate study Area • For pre and Post-processing of HEC-RAS input and Output Data	Student Trial Licenses
HEC-RAS 5.0	• Steady Flow Simulation. • Water Surface Profile Generation.	https://www.usace.army.mil/software/hec- ras/downloads.aspx
HEC-Geo-HMS 9.2	• Study Area Delineation. • Thiessen polygon Development. • Curve Number Generation. • HEC-HMS Project Development. • Metrology Development. • Basin Development etc.	https://www.usace.army.mil/software/hec-geo-hms/downloads.aspx
HEC-GeoRAS	• For pre and post-processing in flood map.	https://www.usace.army.mil/software/hec-geo-ras/downloads.aspx
HEC-HMS 3.5	• For Hydrologic and Metrologic Data Processing. • To Calibrate and validate simulated and Observed Flow Data.	https://www.usace.army.mil/software/hec-hms/downloads.aspx
Mendeley Desktop and E-Draw	• To arrange all journals used in this Thesis. • To insert Citation for all Journals.	https://www.mendeley.com/download-desktop/
Global Mapper	• For 3D analysis	Online web

3.2.3 The equipment used for field surveying data collection

The equipment's like tape, staff gage, and geographic position system (GPS) was used in this study area to collect river cross-section at different sections of the river where there is variation in slope and discharge.

3.3 Data quality controlling

3.3.1 Meteorological data availability and processing

Before beginning any hydrological analysis it is important to make sure that data are homogenous, correct, sufficient, and complete with no missing values. Errors resulting from a lack of appropriate data processing are serious because they lead to bias in the final answers, (Vedula, 2005). Generally, data should be appropriately adjusted for inconsistency, corrected for errors, extended for insufficient, and filled for missing using different techniques.

A well understanding of hydro-meteorological conditions of the study area is one of the responsibilities of water resource management. For this study work Meteorological, Hydrological, and Digital Elevation Model (DEM) data set was undertaken for Awash Belo catchment and the corresponding floodplain area.

Table 3-3: Metrological station used in the study area

Stations Name	Longitude in (deg.)	Latitude in (deg.)	Elevation in (m)	Data used in Years (1990-2005)	Class
Busa	38.11	8.83	2200	1990-2005	4
Enselale	38.41	8.93	2285	1990-2005	4
Ginchi	38.11	9.03	2290	1990-2005	4
Kimoye	38.35	9.03	2115	1990-2005	3
Teji	38.36	8.8	2110	1990-2005	4
Tulu Bolo	38.21	8.66	2100	1990-2005	3

Table 3-4: Awash Belo Gauging station

Station Name	Longitude (deg)	Latitude (deg)	Year of Data Used	Station Number
Nr.Belo	38.42	8.85	1990-2005	031020

3.3.2 Hydrological and metrological gaging stations used for the study area

This section describes the location of rainfall and hydrologic gaging stations that was used in both hydraulic and hydrologic modeling. Most of the Metrologic stations used in this study are class-4 type, which is recording only rainfall. Those stations are located either in the sub-basin or nearest to the study basin as figure 3.1.

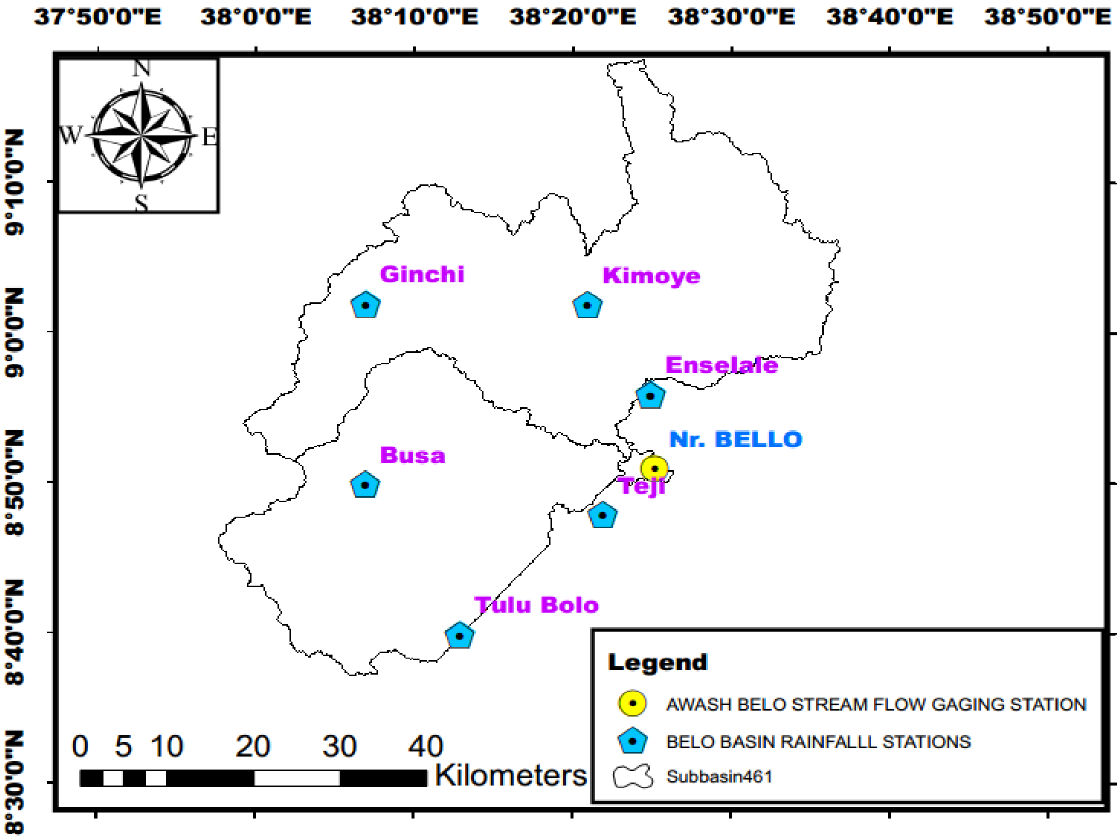


Figure 3-6: Hydrologic and Metrologic gaging stations location

These graphs play a great role in the understanding of the study area's rainfall history. It helps as in visualization of the existence of outliers and how the trends of rainfall in the basins follow each other. Although, we can simply understand where there is inconsistent data recorded and which rainfall stations contribute more rainfall.

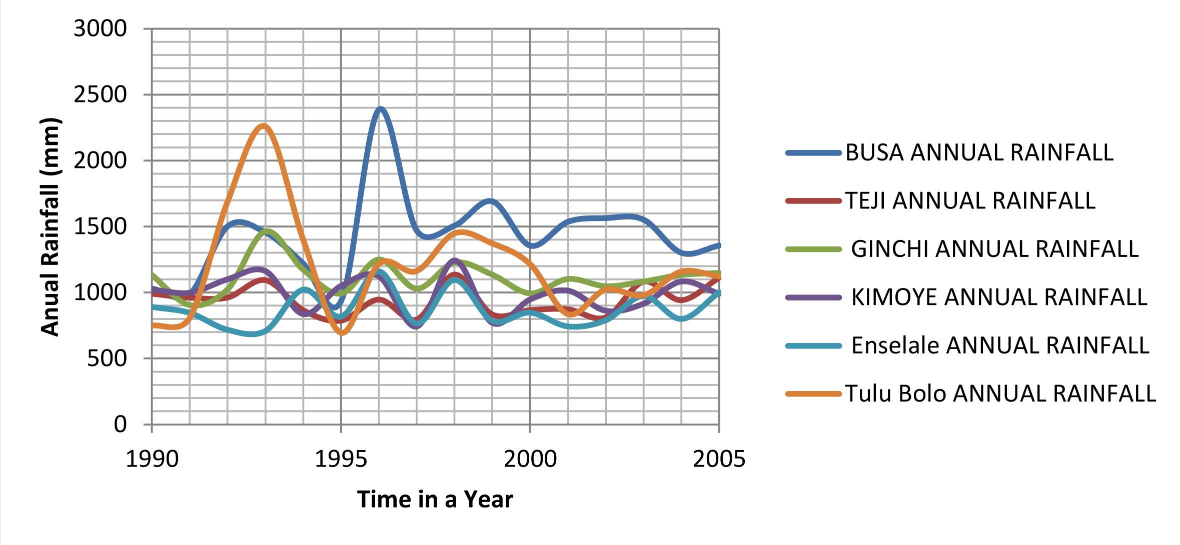


Figure 3-7: Time sequences of annual rainfall for Belo basin in (1990-2005)

3.3.3 Rainfall data quality tests

3.3.3.1 Estimating missing precipitation

This was the first step that was proceeded to know the possibility of filling of missing rainfall Data. From this concept, the missing percentage for all gaging stations is less than 30%, which shows the possibility of filling missing data as follows.

$$\text{Percentage of missing Data} = \frac{\textit{Number of missing obsevation}}{\textit{Total number of observation}} * 100\% \quad \text{...... Equation 3.1}$$

Table 3-5: Rainfall Missing Percentage for different stations

Rainfall Gaging Stations	Ginchi	Teji	Tulu Bolo	Busa	Kimoye	Enselale
Number of Observations	5844	5844	5844	5844	5844	5844
Number of Missing	2	0	222	184	1	35
Percentage of Missing (%)	0.034	0	3.799	3.148	0.017	0.598

Therefore, it was decided to proceed to fill the study stations missing rainfall. Several methods have been proposed for estimating missing precipitation. The station average method is the easy one. The normal ratio and quadrant method provide a weighted mean, with the former biasing the weights on mean annual precipitation at each gauge and the later having weights that depend on the distance between gauges where recorded data are available and the point where the value is required. The normal ratio method is used in this research paper. The method is used when the normal annual precipitation of the index stations differ by more than 10% of the missing stations. This is the case for the stations near the study area. The general formula for computing missing precipitation by this method is:

For the case of this study, it was checked for normal annual rainfall of missing station about more than 80% of it was differ by more than 10% from the normal annual rainfall of neighboring stations and this made why the normal ratio was selected as compared with other. The general formula for computing missing precipitation by this method is:

$$PX = \frac{NX}{3}\left[\frac{P1}{N1} + \frac{P2}{N2} + \frac{P3}{N3}\right] \quad \textbf{Equation 3.2}$$

Where

PX=precipitation for the station with missing records

P1, P2, and P3 are adjacent stations precipitation value

N1, N2, N3 are the long-term mean annual precipitation values at respective stations and 3 is the number of stations surrounding the station x.

Generally, six gaging stations were used for the Awash Belo catchment. Those stations were Tulu Bolo, Teji, Busa, Kimoye, Enselale, and Ginchi but, six of them didn't use for filling the missed stations because of their less correlation as they are far away from the missed station. For example, when filling for Tulu Bolo, Teji and Busa three of them was index/neighboring/ stations while for the rest of three stations like Ginchi, Kimoye, and Enselale they were used as index stations for each other.

3.3.3.2 Consistency and homogeneity for rainfall

Filling of missing data is one problem that hydrologist needs to address. The second problem is data inconsistency where the characteristic of records is changed with time[27]. A consistent record is one where the characteristics of the record have not changed with time. Data inconsistency may arise from: Change place for gage stations, exposure, instrumentation, or problem arises from lack of real-time and place observations. To overcome the problem of inconsistency a technique most widely applied called a double mass curve is used. Double-Mass Curve (DMC) analysis is a graphical method for identifying or adjusting inconsistencies in a station record by comparing its time trend with those of other stations nearby (Shaw, 1988).

Here accumulated mean annual values at the station in the study area are plotted against those of nearby reliable stations or groups of stations. An abrupt change in the slope of the Double-Mass Curve plot suggests some change not related to climatic variables, and adjustment should be made to the data based on the ratio of the slopes of the two segments.

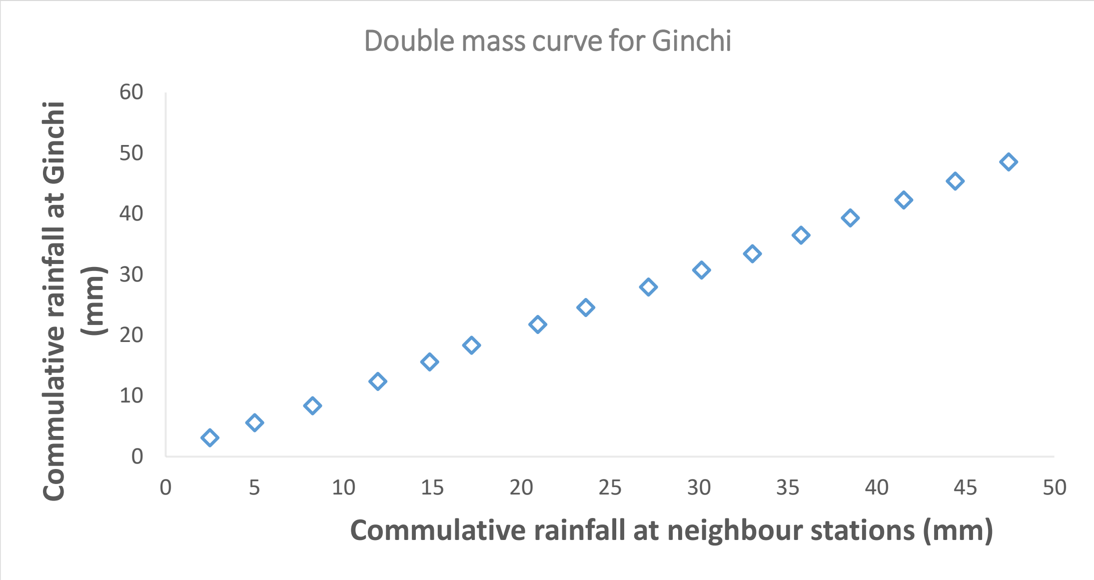


Figure 3-8: Double mass curve for Ginchi station

The entire of the station was consistent with a small error bar and there was less slope variation which is less than 10% and no need for correction at all.

(Slope of original-corrected Slope)/original slope*100 was less than 10% which is not recommended to take correction for slope.

3.3.3.3 Outlier test for rainfall

The Water Resources Council method recommends that adjustments be made for outliers. Outliers are data points that depart significantly from the trend of the remaining data. The retention or deletion of these outliers can significantly affect the magnitude of statistical parameters computed from the data, especially for small samples. Procedures for treating outliers require judgment involving both mathematical and hydrologic considerations. According to the Water Resources Council (1981), if the station skew is greater than +0.4, tests for high outliers are considered first; if the station skew is less than - 0.4, tests for low outliers are considered first. Where the station skew is between ±0.4, tests for both high and low outliers should be applied before eliminating any outliers from the data set.

The following frequency equation can be used to detect high outliers:

Yh = y + Knsy

Where: Yh is the high outlier threshold in log units and Kn is given from sample size n.

The Kn values are used in one-sided tests that detect outliers at the 10-percent level of significance in normally distributed data. If the logarithms of the values in a sample are greater than Yh in the above equation, then they are considered high outliers. Flood peaks considered high outliers should be compared with historic flood data and flood information at nearby sites. Historic flood data comprise information on unusually extreme events outside of the systematic record. According to the Water Resources Council (1981) if the information is available that indicates a high outlier is a maximum over an extended period, the outlier is treated as historic flood data and excluded from the analysis. If useful historic information is not available to compare to high outliers, then the outliers should be retained as part of the systematic record.

A similar equation can be used to detect low outliers:

Y1 = y-Key

Flood peaks considered low outliers are deleted from the record and a conditional probability adjustment described by the Water Resources Council (1981) can be applied.

As outliers affect the statistics, especially higher-order statistics is recommended to test for outliers and eliminating or taking correction on by studying historical data for the study area [28]. In this research, the highest and lowest value was calculated based on principles of water resource councils 1981, and the corrective measure was taken based on historical data records of the study.

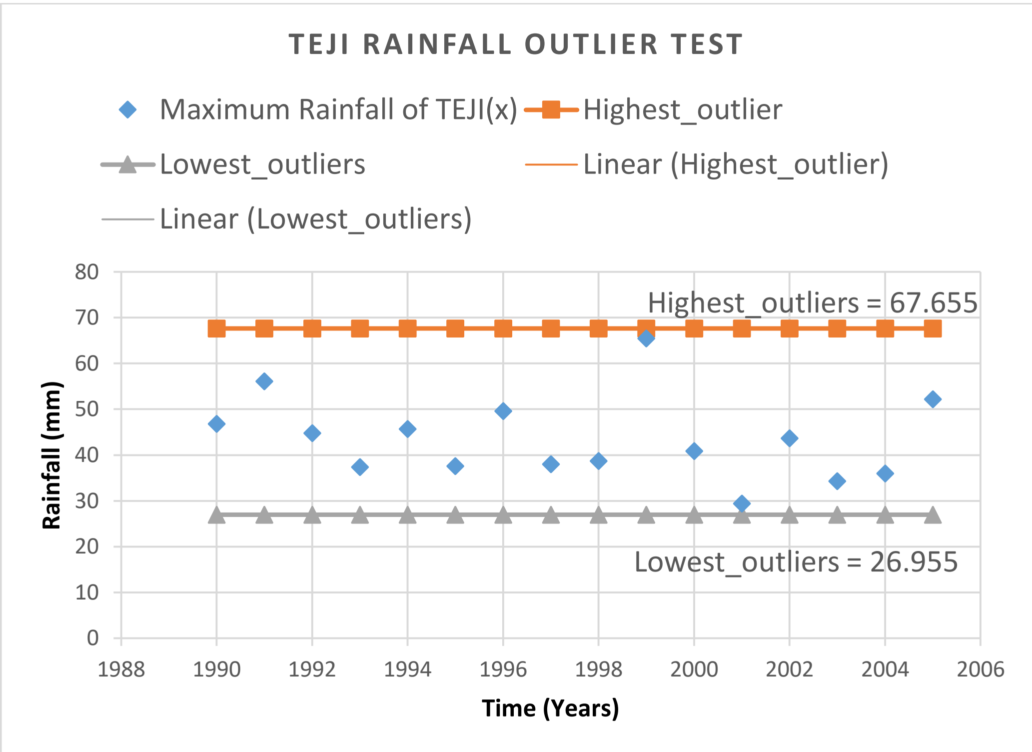


Figure 3-9: Checking outlier for Rainfall data

3.3.4 Areal precipitation

For analyses involving areas larger than a few square miles, it may be necessary to make estimates of areal rainfall depths over sub-watershed areas since a rain gauge records rainfall at a geographical point.

There are many methods available to determine the areal rainfall over the catchments from the rain gauge measurement: Arithmetic Mean, Theissen Polygon, Isohyet, GridPoint, Percent Normal, Hypsometric, etc. The choice of these methods requires consideration for the quality and nature of the data, and the importance, use, and required precision of the result.

A Theissen polygon method is the most widely used method compared to others and was used in this research paper. In this method, weights are given to all the measuring gauges based on their areal coverage of the watershed, thus eliminating the discrepancies in their spacing over the basins. All the stations in and around the basins are considered and a linear variation in the precipitation between two gauge stations is assumed.

In this procedure, areas and lines between adjacent stations were drawn on a map of the area by using Arc GIS 10.3 software with Arc toolbox extension.

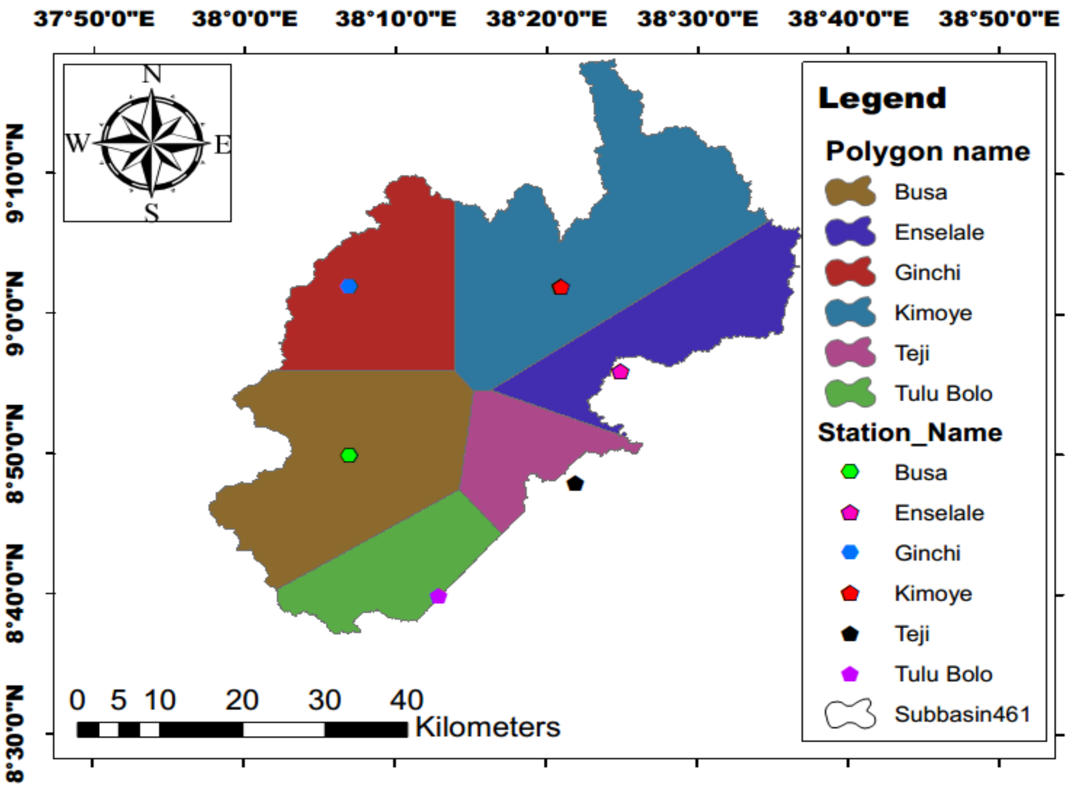


Figure 3-10: Theissen polygons created from stations in Awash Belo Catchments

The perpendicular bisector of these lines forms a pattern of polygons with one station in each polygon. The area with which each station is taken represents the area of its polygon, and this area is used as a factor for weighting the station precipitation. The contribution of the rainfall from each gauging station is limited by its weighting factor.

3.3.5 Flow data filling and quality test for the study area

The daily discharge has some missing values for a certain length of time and gap-filling was done using a regression equation developed to represent the study area. The gaging stations are located near Awash Belo where the downstream end is considered flood-prone to the study area.

3.3.5.1 Flow data filling using regression equation for Belo gaging station

Missing flow data records for the sub-basin was filled by developing correlation stations with missing data and any of the neighbor stations with the same features and common data periods. Consistency and extension of flow data are analyzed by the regression technique. The correlation equations used for Awash Belo gauging station in terms of Asgori, Addis Alem, and Holota which shows good correlation is expressed below for every month of (1990-2005) years.

Table 3-6: Missing data filling for streamflow

Month	Regression equations for each month for specified stations	Correlation
January	Q@Awash Belo=1.1-0.46*Q@Addis Alem	0.65
February	Q@Awash Belo=0.42+2.2*Q@Asgori	0.81
March	Q@Awash Belo =0.65+1.6*Q@Asgori	0.6
April	Q@Awash Belo=0.74+2.7*Q@AddisAlem	0.73
May	Q@Awash Belo=0.4+2.44*Q@Addis Alem	0.87
June	Q@Awash Belo=3.4+3.3*Q@AddisAlem	0.63
July	Q@Awash Belo=19.9+0.8*Q@Addis Alem	0.58
August	Jully&September Equations	Half
September	Q@Awash Belo=17.1+1.42*AdisAlem	0.73
October	Q@Awash Belo=0.68+8.3*Q@Holota	0.88
November	Q@Awash Belo=2.6-4.32*Q2Asgori	0.8
December	Q@Awash Belo=1.15-0.4*Q@AdisAlem	0.65

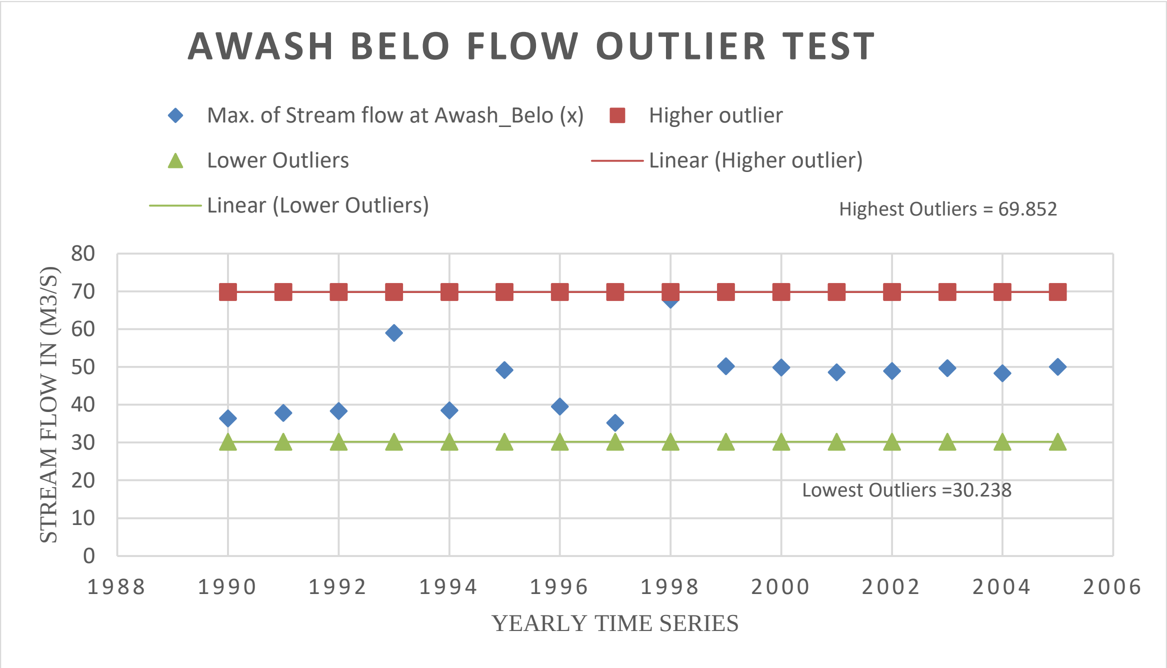
3.3.5.2 Streamflow outlier Tests

There was no outlier for streamflow for this specified period at Awash Belo station. The outliers test result shows the data points at the study area were located between the highest value 69.85 and the lowest value 30.2.

Table 3-7: Outliers estimation using water resource council 1981 technique

Year	Max. of Streamflow at Awash Belo (x)	Higher outlier	Lower Outliers
1990	36.389	69.8	30.2
1991	37.84	69.8	30.2
1992	38.361	69.8	30.2
1993	59.02205756	69.8	30.2
1994	38.466	69.8	30.2
1995	49.14011078	69.8	30.2
1996	39.516	69.8	30.2
1997	35.157	69.8	30.2
1998	67.75577921	69.8	30.2
1999	50.216	69.8	30.2
2000	49.863	69.8	30.2
2001	48.56395857	69.8	30.2
2002	48.92603776	69.8	30.2
2003	49.686	69.8	30.2
2004	48.283	69.8	30.2
2005	50.039	69.8	30.2
	YT=Yavg+Kn*STAD	1.8	69.8
	Yl=Yavg-Kn*STAD	1.48	30.2

Table 3-8: Outlier test for streamflow at Awash Belo gauge station



3.4 The procedure followed for this research

This section explains the methods and procedures applied in this research

- Terrain preprocessing using DEM, ARC-GIS and Arc-Hydro tool for preparation of spatial hydrographic features to be used as an input for HEC-GeoHMS.
- Curve Number (CN) Grid generation using land use and soil data of the Awash Belo basin.
- HEC-GeoHMS.HEC data processing for Watershed delineation and the generation of the Basin Model file and importing it into HEC-HMS.
- Generating peak flow, time to peak, the volume of discharge, and the runoff hydrographs on the study area using the HEC-HMS Hydrological Model by employing storm precipitation depth of the Study area.
- Field survey to obtain data to reach cross-section for preparation of geometric data to be used in the HEC-RAS model.
- Running steady-state simulations in HEC-RAS for the computation of water surface profile by employing discharge from HEC-HMS Model result for 2, 5,10,25,50, and 100 years return period.
- HEC-RAS post-processing in GeoRAS extension for flood inundation mapping using water surface profile generated on the above step.
- Calibrate and validate flood inundation area using selected events from the Landsat 7 image, NDWI.
- Proposed different flood protection alternatives for the study area to select the most efficient alternative for the Belo flood plain.
- Select the best alternatives for the study area by considering the criteria like inundation area, environment-related problem, and balance of the two using Multicriteria Decision Making (MCDA).

3.4.1 Research framework

This section includes the whole study framework at the Awash Belo basin and downstream flood plains. It contains detailed descriptions to represents this study in the next section. The step listed in this framework is only representing the general steps that were passed through and didn't include minor steps undertaken in methods due to the wideness of the study.

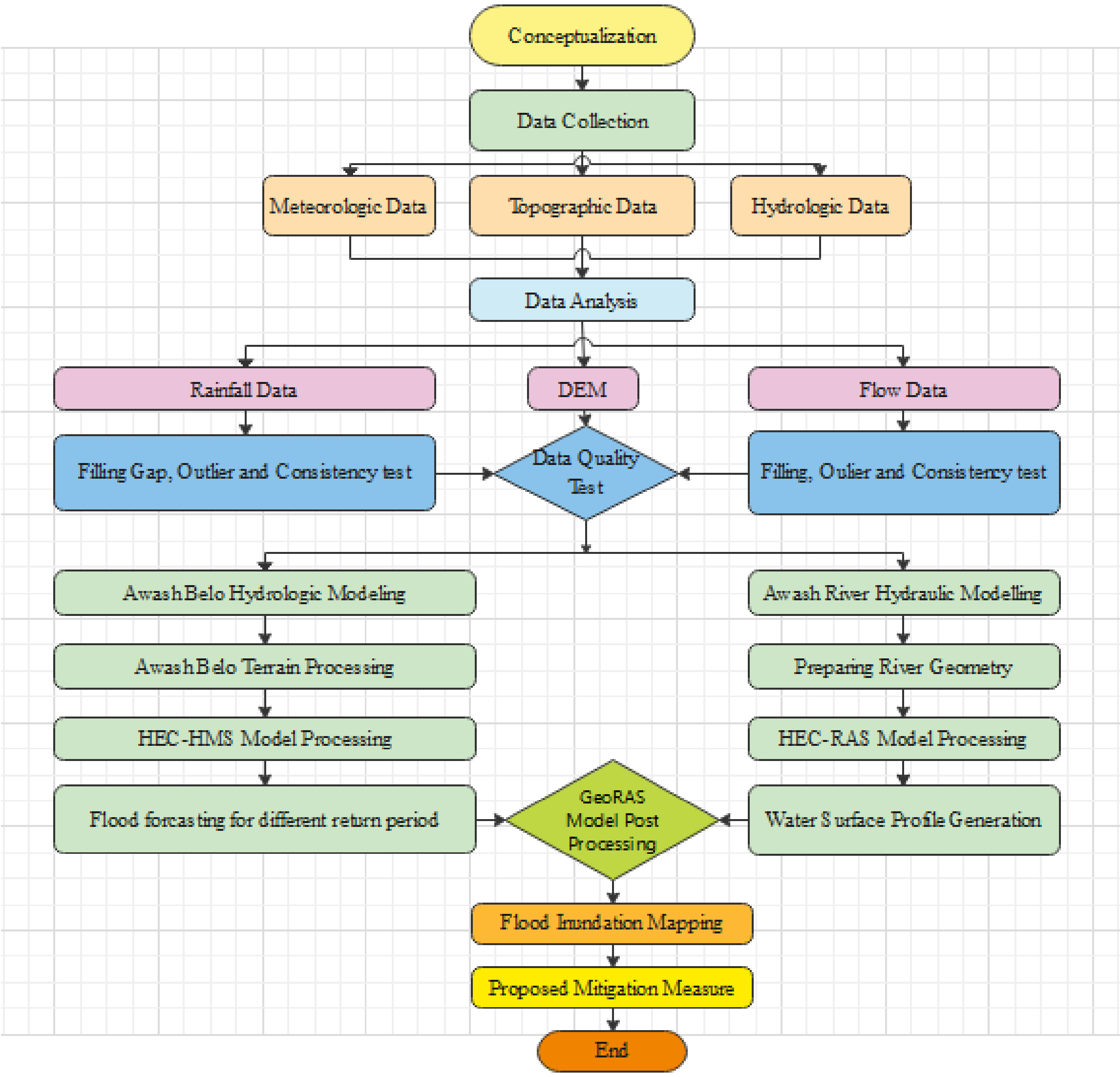


Figure 3-11: Study area conceptual frame work flow chart

3.4.2 Terrain preprocessing

This is the first step in doing any watershed-related works. This model works first requires different basin input parameters that were developed from the study area basin using Gis-extension HEC-GeoHMS like stream slope, river length, sub-basin area, stream network density, etc.

3.4.3 Creating CN grid for initial model parameters

In this study, HEC-GeoHMS was used to create the curve number grid. HEC-GeoHMS uses the merged feature class soil and land use, and the lookup table (CNLookUp) to create the curve number grid. In this study, these values were used for only the initial input of the HEC-HMS model before optimization and the model used the final calibrated model that can fit the objective function.

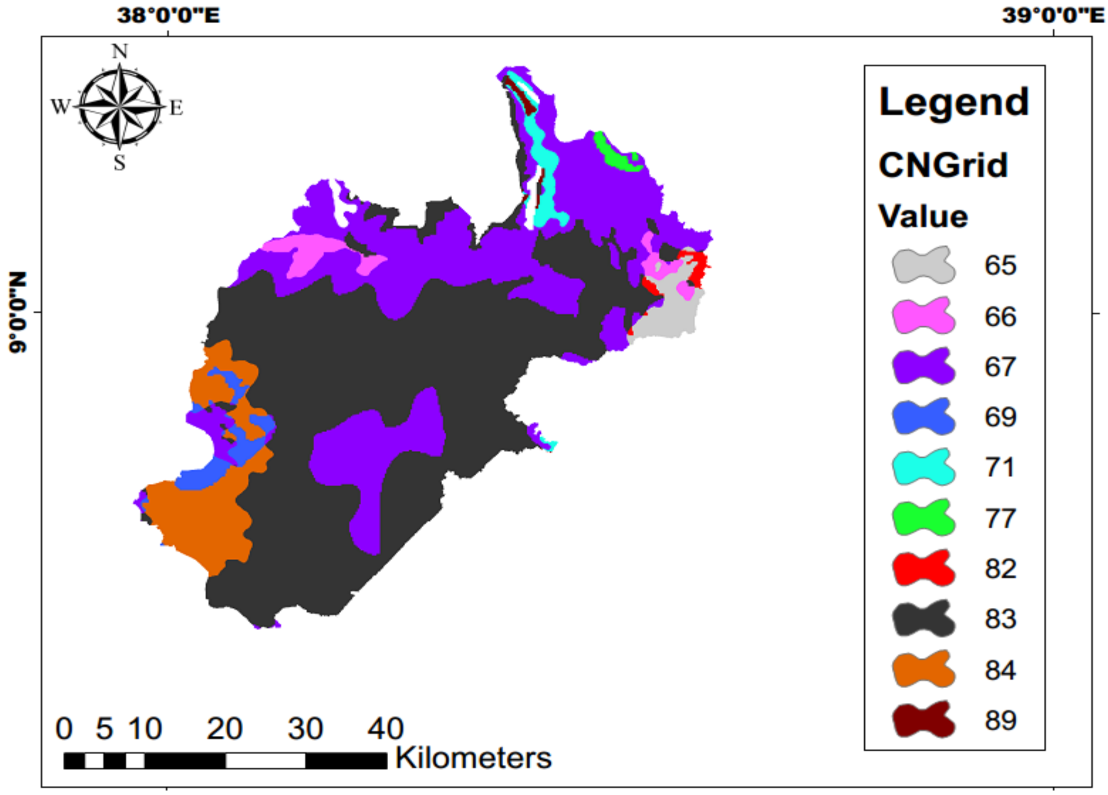


Figure 3-12: Curve number grid of the catchment

3.5 HEC-HMS basin model development

The HMS model consists of three main model components: basin model, Metrologic model, and control specification. Among them, the basin model consists of different catchment characteristics such as river reach, basins area, junction, outlets, etc. This study basin model consists of R20, W40, W50, W60, and outlet where W and R stand for sub-basins and reach respectively. There are different sub-basins elements for the transformation of excess rainfall to runoff, runoff to excess rainfall, routing, and base flow methods. In this study, the listed below are applied based on their purpose and suitability.

3.5.1 Initial and constant loss method for Awash Belo basin

Unlike, of SCS curve number loss method the predicted value is following classical unsaturated flow theory in this model, which made it most adaptable to the Awash Belo.

In the Initial and constant loss method, each sub-basin requires the following parameters**.**

Initial loss

The value of Initial Abstraction for the corresponding Curve Number value is calculated using the following formula

I_a=0.2*S……………………………………………………………….Equation 4.1

$S = \frac{25400-254*CN}{CN}$……………………………………………………..……..Equation 4.2

Curve number

The average curve number for sub-basins is already computed using HEC- GeoHMS and this value is imported during the exporting of the Basin Model to HEC-HMS for initial parameter inputs before optimization.

Percent of imperviousness

Percent of imperviousness for the entire sub-basin was tried to be computed by taking the ratio of the area of the watershed covered by houses to the total area of the subwatershed but since the majority of study catchment is covered by agricultural land the percentage of impervious is less and neglected.

Constant loss rate

The soil condition of the Awash Belo basin is dominated by type D soil, which means soil that swell significantly when wet, heavy plastic clays, and certain saline. For such soil, the Loss rate is within the range of (0-1.25) mm/hr.[13].Because the model parameters are not measured directly the initial parameter is best estimated by calibration.

3.5.2 The SCS unit hydrograph transform method for Awash Belo basin

The SCS unit hydrograph method was used for this study because the required parameter can simply be extracted from catchment characteristics using HEC-GEO-HMS for initial estimations. Not only that, but it was also a well-applicable model on the different catchments of the other country like the USA. Therefore this model was generated good results as compared with other methods for the Awash Belo basin.

The SCS unit hydrograph method requires only one parameter for each sub-basin: lag time between rainfall and runoff in the sub-basin. The lag time is already computed in the previous section using the HEC- GeoHMS Model and imported with the Basin Model. The parameter that is specified here is time to peak and the program will compute the time of concentration and peak flow to rescale the SCS Dimensionless unit hydrography this is then used to compute direct runoff for the catchment. These values were not used directly for model simulation rather it was extracted from basins for initial model parameters

3.5.3 Lag routing method for Awash Belo basins

The Lag routing method was selected for the Awash Belo basin. Since the study catchment is small, there is less storage influence and there is no attenuation of discharge along the reach but there is a time change of hydrography along the reach (Translation). Not only that, but the reach length of the Belo flood plain is also short and made it suitable for Lag method Routing. The slope of the channel is also greater than 0.002 which shows the suitability of any routing methods for the study area. The hydrography of the upstream is less varied from downstream or Awash Belo gaging station from observed data except for variation in time which shows the most suitability of using lag routing as compared with other methods.

3.5.4 Monthly constant base flow method for Awash Belo basin

The monthly constant base flow method was selected for this study because of the availability of observed flow for estimation of minimum flow during fair weather conditions and storm runoff is not occurring from historical data[13]. The values of this may vary monthly but the same for this study case. The minimum values of streamflow before calibration during there weren’t storm runoff occurring were taken. By considering those above criteria the values of 0.1 and 0.2 m^3/s were used for sub-basins W50 and W60, and W40 respectively in streamflow estimation.

3.6 Metrological model

3.6.1 Weight metrological method for Awash Belo basin

The gage weight method is a meteorological method used in the meteorological model to produce runoff from given timely precipitation data. The method requires the following inputs: sub-basin, gage precipitation, and gage weights (temporal and spatial)[10]. Using the Theissen polygon, the stations used to represent each sub-basin are tabulated here.

Table 3-9: Spatial and temporal weight for gage weight metrological model

Sub-basin/station		Spatial/depth weight	Time/seasonal weight
1	Teji	0.003	1
	Busa	0.046	1
	Kimoye	0.493	1
	Enselale	0.227	1
	Ginchi	0.230	1
2	Teji	0.173	1
	Busa	0.522	1
	Kimoye	0.004	1
	Enselale	0.003	1
	Ginchi	0.052	1
	Tulu Bolo	0.247	1
3	Teji	0.851	1
	Enselale	0.149	1

3.7 HEC-HMS Model calibration, validation, and performance of the basin

3.7.1 Model calibration

The calibration uses observed hydro-meteorological data in a systematic search for parameters that yield the best fit of the computed results to the observed runoff.
The search is often referred to as optimization.

To compare a computed hydrograph to an observed hydrograph, the program computes an index of the goodness-of-fit.
The algorithm included in the program search for the model parameters that yield the best value of an index, also known as objective function [10]. Out of four objective functions in HMS, Peak-weighted root means the square error is selected for Awash Belo basins.

This function is an implicit measure of comparison of the magnitudes of the peaks, volumes, and times of peak of the two hydrographs. Nelder and Mead Algorithm method of search parameter that minimizes the value of the objective function is used for this study. This algorithm relies on a simple direct search.

A total of 16 years of historical data from 1990 to 2005 was used, and out of this (1990-2001) was used for calibration at the Awash Belo basin. Manual and automatic calibration was used for the optimization of observed and simulated flow data using the initial parameter from watershed characteristics. Calibration was done by taking initial parameters generated from HEC-GeoHMS and Arc Hydro for the catchment. With the initial parameters, the calibration was then processed until the simulated value resembles the observed data.

3.7.2 Model validation

The model validation is the process of testing the model's ability to simulate observed data without using other data used in calibration with acceptable accuracy. In the validation process, the values of the parameters optimized during calibration should be kept constant without any change. The quantitative measure of the match is again the degree of variation between computed and observed hydrography. The data selected for validation was (2001-2005) for the selected watersheds of the Awash Belo.

3.7.3 Model efficiency/performance

For the Awash Belo catchment, model simulation has been evaluated using efficiency criteria such as coefficient of determination (R^2) and [Nash and Sutcliff (E_{NS}), 1970]. Both coefficients above were used to measure the similarity of observed and simulated flow for a specified time step. The range of values for R^2 is 1.0 (best) to 0.0. The statistical index of modeling efficiency (E_{NS}) values range from 1.0 (best) to negative infinity[19].

A) Nash-Sutcliffe Efficiency, NSE

The NSE is used to evaluate the overall agreement of the shape of simulated and observed hydrograph during calibration and validation for the Awash Belo basin.
[31]recommended for monthly time step that NSE values between 0.75 and 1 are very good and NSE values between 0.65 and 0.75 are good.

$$NSE = 1 - \frac{\sum_{i=1}^{n}(oi - si)^2}{\sum_{i=1}^{n}(oi - \bar{o})^2} \text{..................................} \textbf{Equation 3.3}$$

Where oi is observed flow at the i^{th} period is simulated flow at the i^{th} period

O_{avg} is the mean of observed flow.

B) Coefficient of determination, R^2

It expresses how good trends in measured data are reproduced by the simulated result over a specified period of calibration and validation. It is calculated as:

$$R^2 = \frac{[\sum_{i=1}^{n}(Qs - \overline{QS})]^2}{[\sum_{i=1}^{n}(Qs - \overline{QS})]^2[\sum_{i=1}^{n}(Qo - \overline{Qo})]^2} \text{...} \textbf{Equation 3.4}$$

[31]recommended for monthly time steps that R^2 values between 0.75 and 1 are very good and R^2 values between 0.65 and 0.75 are good.

3.8 HEC-HMS flood forecasting and data processing for Awash Belo basin

3.8.1 Intensity frequency duration for the study area

With the input from HEC-GeoHMS and some edition from the main HEC-HMS, the model is simulated for a rainfall intensity of 2, 10, 25, 50, and 100 year return periods. The storm frequency intensity equation of the study area was taken from the IDF equation for similar rainfall regions of Ethiopia[32]. This equation was well developed for the study area with a high-performance value. The strongness of this equation is the ability to estimate rainfall depth for any time as compared with graphs developed by ERA for different rainfall regions of the countries.

$$I = \frac{758.1 * Tr^{0.22}}{(Td+8.36)^{0.82}}$$ **IDF equation of study area ………..…Equation 3.5**

Table 3-10: IDF table for the study area

Duration (hr.)	2	5	10	25	50	100	200
1	27.63	33.8	39.37	47.9	56.09	65.34	76.1
2	32.96	40.32	46.96	57.46	66.92	77.94	90.78
3	36.09	44.16	51.44	62.91	73.29	85.38	100.38
6	41.65	50.94	59.34	72.6	84.54	98.52	114.72
12	47.64	58.2	67.8	83.04	96.72	112.6	131.16
24	54.2	66.24	77.28	94.512	110.08	128.16	149.28

3.8.2 Meteorological model for flood forecasting at the study area

After having an established basin model, the Metrologic model is created. The Rainfall data required to simulate a watershed are stored in the Metrologic model. From the different options of the method, the Frequency Storm type of precipitation was selected for the Awash Belo basin. The frequency storm method is used in the model to produce a frequency storm from a given statistical precipitation data. The Method requires the rainfall depth for storm duration of 24 hours as input data from the IDF table above. As Rainfall duration increase the cumulative rainfall depth increase but the average intensity over duration decrease because severe rainfall cannot be sustained for very long.Once the precipitation type is identified the next step is to enter data.

3.8.3 Defining control specification for flood forecasting at the study area

This section requires information related to when the storm starts and ends. Besides this, it requires a storm time step. In the Belo basin, the duration is arbitrary; long enough to depict the runoff a 1-day storm, but the 6 hour time interval is part of the basin file model set up and should remain fixed for this Awash Belo Model. The time interval is computed as follows. The smallest Lag time in the watershed is 1322 minutes for sub-watershed W60.So the time interval for control specification will be always less than 0.29*1322<6.38 hours values of 1hr. as the taken time of interval for control specification. The time interval should also less than the maximum intensity interval to capture the flow without missing peak at a specified time.

3.9 HEC-GeoRAS extension

According to the [14], HEC-GeoRAS is a set of ArcGIS tools specifically designed to process geospatial data for use with the (HEC-RAS). The HEC-GeoRAS was used in this study for pre and post-processing of the HEC-RAS model. This extension was used especially in river geometry preparation by using ras layers like digitization of stream centerline, banks, flow path, and cross-sections. The river geometry extracted from this extension was used as input parameters for HEC-RAS geometric data. Additionally, this interface was used in HEC-RAS post-processing for flood map generation or delineation using ras mapper tools.

3.9.1 Geometric data processing for Belo flood plain

First of all the Awash main river and the study area should be digitized or extracted from DEM 12.5 m and overlay to the awash river polylines from the Google earth map shapefile to get the appropriate feature class. Next, some correcting measurements like overlapping with the DEM map of the study should be done to get appropriate river geometry of the Belo flood plain. The geometric preprocessing of the Belo flood plain involves digitization river characteristics and having their attribute table which in turn used for HEC-RAS model inputs. Because of the topographic feature of the study area as flat terrain and the whole as cultivation, the only performing activities of the geometric data layer here is creating and digitizing the main river, the bank stations, flow paths, and cross-sections across the cultivated area.

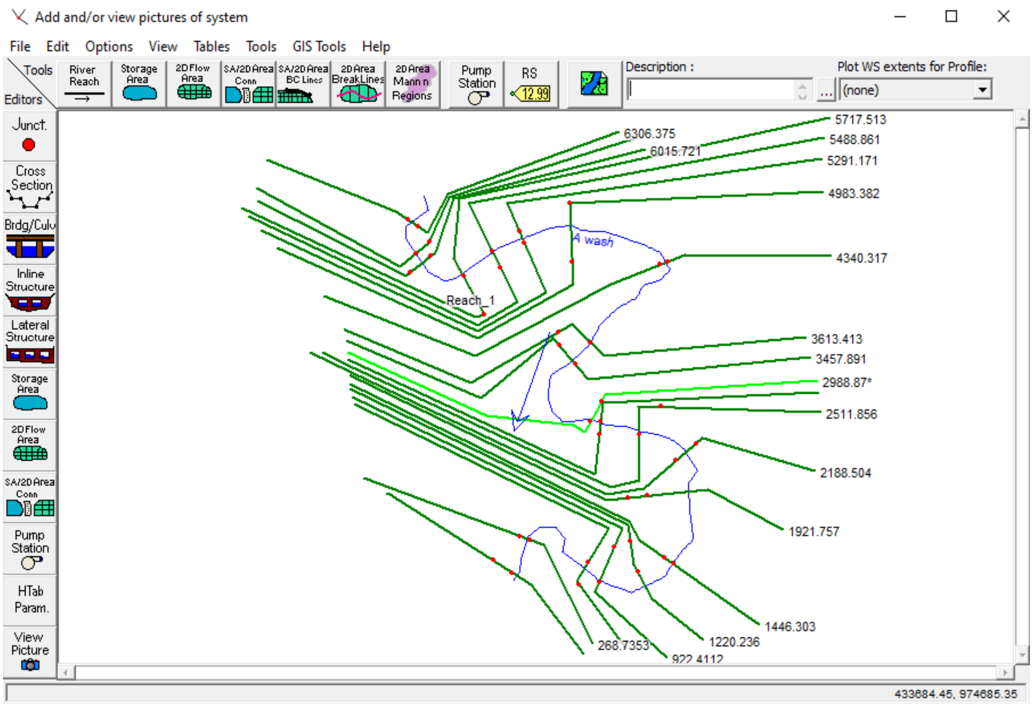


Figure 3-13: Geometric profile exported from HEC-GeoRAS in HEC-RAS

3.10 HEC-RAS model, calibration, and validation for Belo flood plains

The HEC-RAS model was used for Awash Belo flood plains to generate water surface profiles for different flow scenarios. Water surface profiles were used by the GeoRAS model to delineate the flooded area by using the Water surface TIN and Digital terrain model of Belo flood plains. The model was intended for a steady flow water surface profile generation. The model is capable of modeling all flow regimes but, in this study mixed flow regime was used.

In these models, three main components of the data set were used. Those are geometry data, flow data, and plan data. The geometric data contains information related to reach, cross-sections, manning, and expansion, and contraction coefficient. Flow data used for entering steady flow data at upstream of flood plains. The plan contains information pertinent to the working plan.

Manning's roughness coefficient estimation for initial model parameter

The roughness of the surface affects both watershed surface runoff and channel flow.

The roughness of the surface or channel results in flow retardation. In overland flow, the increase in the roughness of the surface results in flow dilatation of flow which decreases the velocity of flow and results increase the potential infiltration. The decreased velocity due to the roughness of the surface results in decreases in the amount of erosion. The roughness effects on the channel flow are the same as that of surface overland flow.

It is a necessary input in floodplain delineation, also, several methods for estimating the timing of runoff use n as input and used in the design of stable channel systems.

The “n” values were assigned based on the characteristics of the upper Awash River channel and floodplain area. Each characteristic was given n-value based on table values for Similar conditions Cowan [33] and engineering judgment using field survey photos of the study area.

Expansion and contraction coefficient

According to [14] the contraction and expansion coefficient for the natural channel which has a gradual change in cross-section is estimated to be 0.1 and 0.3 respectively. By understanding this concept, since the Awash river is a natural channel, it has the same value as stated above. Even if Awash river is not straight but it meanders nature with irregular shape in cross-section and when coming to its longitudinal profile, it is gradually varied. Therefore, to perform flow calculation in the channel, 0.1 was used for the contraction coefficient whereas 0.3 was used for the expansion coefficient.

Entering and editing flow data

The Awash Belo flood plains have only one reach. Therefore, the maximum flow forecasted for different return periods from the HEC-HMS model was used at the most upstream location of the Belo flood plain. The flow that is entered at upstream of the flood plain is assumed to be steady or no change of flow with time for specified reach since there is no other reach. For carrying out the analysis here, the peak flood having 2, 5, 10, 25, 50, and 100 year return period of flow was used.

Boundary condition

The boundary conditions are necessary to establish the starting water surface either at the start or at the end of the reach for the program to start water surface computation. This boundary condition depends on the flow regime of the study area critical, sub-critical, super-critical, and mixed flow. In this study mixed flow regime was used therefore boundary condition was taken at both upstream and downstream of the reach. The boundary condition is the initial water depth that the program uses to start water surface computation by iterative standard step method. There are four boundary conditions in this program known water depth, rating curve, critical depth, and normal depth. Among them, the normal depth for mixed flow regime was selected for this study because boundary condition was required at both upstream and downstream. The reason, for the selection of normal depth, was since the bed slope of the channel at both ends is less varied, the upstream and downstream end water surface profile is approximately uniform. A normal depth should be calculated for the upstream and downstream end profile of the study area. The slope of the channel bottom was 0.001 and 0.002 at upstream and downstream respectively.

Steady flow simulation for Belo flood plain

The simulation of flow requires information pertinent to the type of flow and the regime of the flood. In this study, steady flow simulation was selected for one reach flood plain because of the requirement of only instant maximum forecasted flood for levee as a structural mitigation design. Hydraulic structures are designed with maximum forecasted streamflow. Therefore, the levee is one of the hydraulic structure designed with the maximum flood. Additionally, in this case, the mixed flow regime was selected because it starts flow calculation from the upstream and downstream ends of the river system.

3.10.1 Calibration of the HEC-RAS model using Landsat-7 image of 9/3/2001 event

Indices were chosen for calibration

This section includes an application of different indices to the satellite image out of which the best matching to the HEC-RAS, of different return period extent, could be chosen for calibration purposes. Several previous studies have revealed that not all indices that were proposed to be used to generate GIS, raster layer depicting water-land separation, generate the same result[34]. Therefore, six indices were chosen which are calculated based on RGB composites, created in GIS software, combining different spectral bands. The indices are as follows: Normalized Difference Water Index (NDWI), Modified NDWI (MNDWI), Normalized Difference Moisture Index (NDMI), Water Ratio Index (WRI), Normalized Difference Vegetation Index (NDVI), and Automated Water Extraction Index (AWEI)[34]. Comparing the surface areas extracted from each index to the area of the different return periods HEC-RAS flood band, provided a quantitative means of determining the usability of the indices and selecting the best-resembled water delineated area from indices with HEC-RAS simulated flood area for calibration and validation[34]. The best matching to the HEC-RAS simulated flood map was chosen from all indices for calibration and validation. Among all indices, the NDWI was the best match with HEC-RAS simulated flood map in terms of the magnitude of the area it covers and the two event flood overlay for Belo flood plain.

There are different formulas applied for those raised above indices as follow[34]:-

1. Automated water extraction index (AWEI)

AWEI=4*(Green-MIR)-(0.25*NIR+2.75*SWIR)

Where MIR: Middle Infrared for Landsat 7 Geotiff

Green: Band 2 MIR: Band 5 SWIR: Short wave infrared

NIR: Band 4 Red: Band 3

SWIR: Band 7 NIR: Near Infrared

2. Water ratio index (WRI)

WRI= (Green +Red)/ (NIR+MIR)

= (Band2+Band3)/ (Band4+Band5) for Landsat 7 Geotiff

3. Modified normalized difference water index (MNDWI)

MNDWI= (Green-MIR)/ (Green+MIR)

= (Band 2-Band 5)/ (Band 2+ Band 5) for Landsat 7 Geotiff

4. Normalized difference water index (NDWI)

NDWI= (Green –NIR)/ (Green+NIR)

= (Band2-Band4)/ (Band2+Band4) for Landsat 7 Geotiff

5. Normalized difference vegetation index (NDVI)

NDVI= (NIR-Red)/ (NIR+Red)

= (Band4-Band3)/ (Band4+Band3) for Landsat 7 Geotiff

6. Normalized difference moisture index (NDMI)

NDMI= (NIR-MIR)/ (NIR+MIR)

= (Band4+Band5)/ (Band4+Band5) for Landsat 7 Geotiff.

The best indices used in calibration for flood map event of 03/09/2001

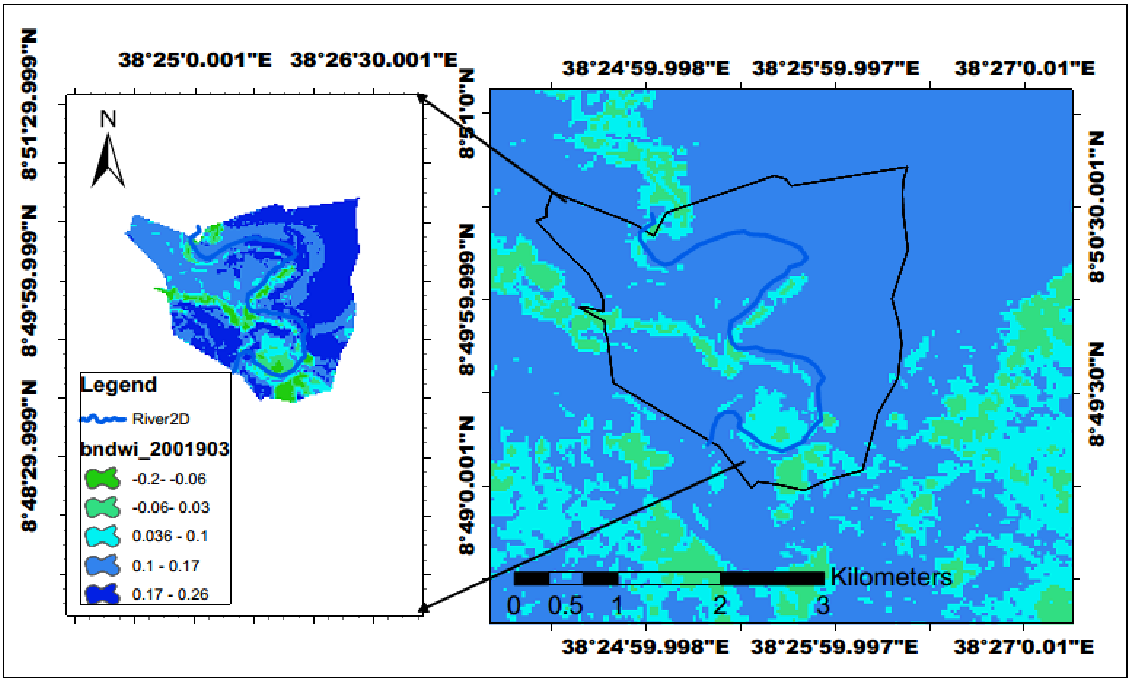


Figure 3-14: Water body extraction from Landsat-7image of 03/09/2001 for calibration

The best indices applied for a validation event

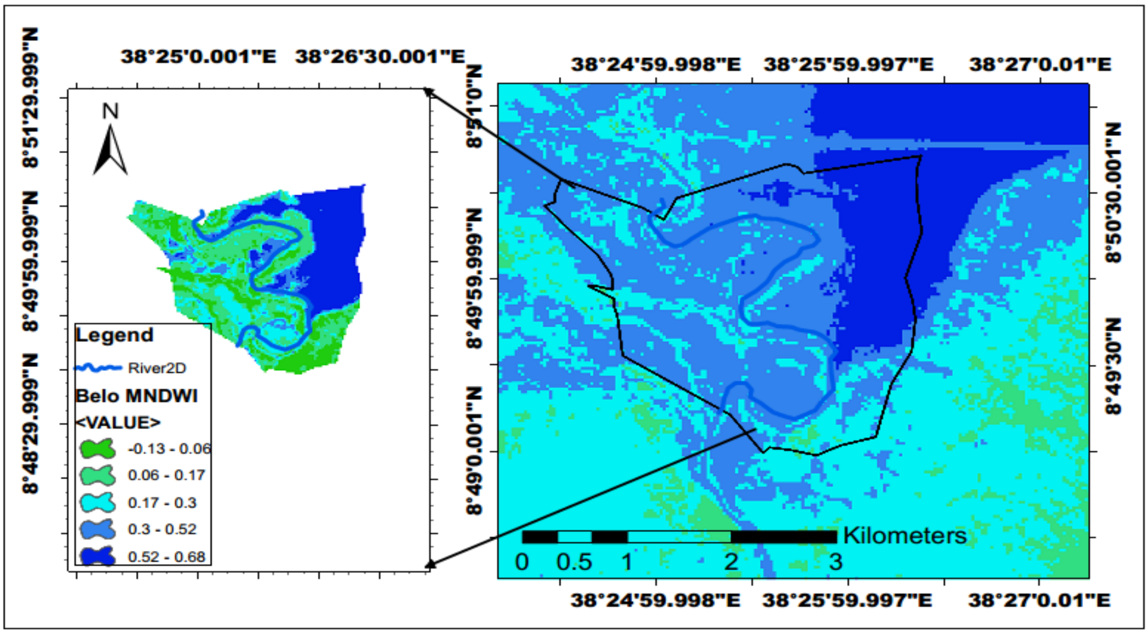


Figure 3-15: water extraction from Landsat-7 image of 25/08/2001 for validation

If all data are not available for calibration and validation, we can follow options like remote sensing analysis of the event and needs surveying the historical event. In such cases, we are forced to use trial and error by changing the coefficient of the manning number for calibration until simulated flood map and Satellite image flood map become resembled and remote sensing analysis for validation[35].

3.10.2 Validation of the HEC-RAS model using Landsat-7 image of 8/25/2001

In this thesis works, the satellite imagery of good resolution for flood event (August 25, 2001) was downloaded (LANDSAT 7 Geotiff image of Band-2 and Band-4 for analysis of the Normalized Difference Water Index).Normalized Difference Water Index used for both calibration and Validation in which events of (September 03, 2001, and August 25, 2001) were compared with the HEC-RAS flood extent (September 03, 2001, and August 25, 2001) respectively. Normalized Difference Water Index (NDWI) is used for water body analysis. The index uses Green (Band 2) and near infra-red (Band-4) bands of remote sensing images.

Normalized difference vegetation index (NDVI) in 2020

NDVI:-is a measure of the degree of the greenness of in vegetation cover of the land surface (Rouse et al., 1974).

$$NDVI=\frac{NIR-RED}{NIR+RED}$$

In Landsat-8 the NIR and RED bands are designated as band 5 and band 4 respectively.

The range of NDVI is between -1 and 1. The more the values approach 1 the more greenness of vegetation is considered. The main consideration of this value in this research was to use NDVI output as a supportive idea for inundation mapping calibration. From NDVI of the area, we understand that the less the greenness area was covered by the more water as compared with the more greenness area and vice versa. As we understand from satellite data analyzed below the degree of the greenness of the study area less because of the range of NDVI (0.05976-0.2923) far away from 1 (fully greened area) which shows the area is more covered by the water-based on NDWI build up above for calibration purpose.

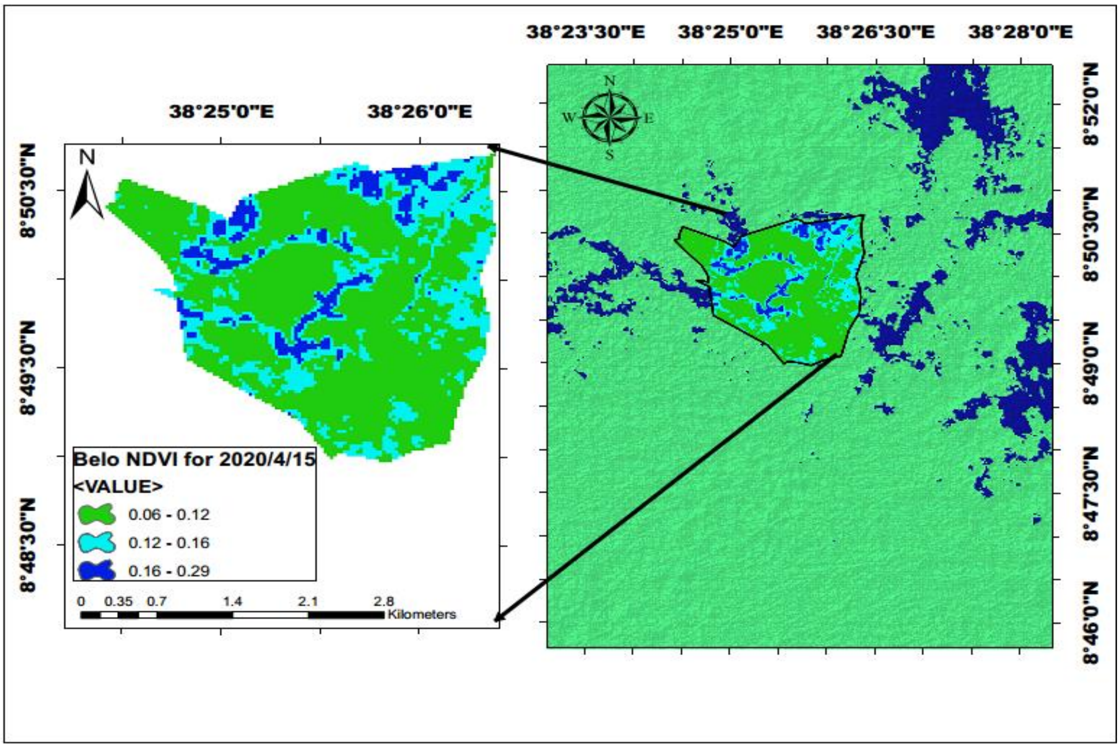


Figure 3-16: NDVI for Landsat 8 on April 15, 2020

3.11 Post-RAS processing of geometric data

After successful simulation of HEC-RAS, the exported HEC-RAS results to ArcGIS to post-processing of them, as a result, flood inundation mapping should be produced for the different return periods to propose mitigation measure.

Bounding polygon

These define the study area based on cross-section extension across the river. It identifies where flood inundation should be analyzed.

3.12 Proposed mitigation measures for Belo flood plains

General

Flood mitigation refers to flood correction measures taken to reduce the effect of flooding on life and property. The main components of flood mitigations are water surface profiles of the study area because it can show us where to focus to take action. The flood mitigations of the study area were taken to reduce the physical extent of the flooding for saving humans and agricultural properties at Belo flood plain. From the flood elevation profile of the study area, the flood is out of the banks for most of the flow scenarios for 50 and 100 years. The mitigation measures taken at Belo were to keep the flood extent within a bank. These measures were taken by considering flood magnitude and Belo flood plain reach characteristics. Among structural and non-structural mitigation measures, the structural measure was applied for this study. Different structural measures can be taken to reduce the effects of flooding:

- Dams.
- Retention Ponds.
- Flow Diversions.
- Pump houses.
- Deepening/Widening/Straightening of the river.
- Levees/Embankment/Bunds.

Non-structural methods of flood controls and protection are:

- ❖ Policies, Guide Lines, and Laws.
- ❖ Controlled development in flood plains.
- ❖ Flood Forecasting and Warning.
- ❖ Flood Proofing.
- ❖ Relocation of Population.

In the upper Awash Belo flood plain, four structural mitigation alternatives solutions have been proposed. These alternatives are:

1. Two-Stage Channel Widening and Degrading modification system (Alternative-1).
2. One stage Channel Widening and Degrading modification system (Alternative-2).
3. Using the above three alternatives at an appropriate location (Alternative-3)
4. Levee Construction along the river reach (Alternative-4);

3.12.1 Two-stage channel widening and degrading modification system

This alternative was done at two stages; both first and second stages had different width and degrading depth based on the construction stability and hydraulics of the flow at the different cross-section of the reach. The river was with appreciably meandered and requires well safety consideration regarding the erodibility and sediment deposition to be confirmed by the environment. The first excavation was done with varying top width of the channel in between 173 m to 911m and the second excavation was done after finishing of first stage excavation with the top width in between 262m and 892m to obtain compound channel with the increased conveyance. Although the degradation depth corresponding to the first stage excavation was defined at varying depth as of the top width within the range of 0.61m to 3.38m for first stage excavation and 0.24 to 2.69m for the second stage.

3.12.2 One stage channel widening and degrading modification system

The one-stage channel widening and degrading modification system are almost the same as that of the two-stage alternative except, the one-stage had one design width with one degrading depth. The shape of this alternative was approaching rectangular whereas trapezoidal for a two-stage river modification system or alternative one above. The bottom elevation for this modification alternative was defined at bed elevation of each cross-section. The width of excavation corresponding to this model was varying from cross-section to cross-section with value in between 173m to 911m based on hydraulics and stability of the Chanel. This alternative might be unstable in rectangular design but with defined low depth of excavation, it is not much suspected of the instability of channel. From field inspection, most of the material of excavation was deposited sediment material. The velocity corresponding to this channel was suitable but higher than that of the compound channel due to less conveyance area.

3.12.3 Using the two alternatives with levee construction at upstream reach

This method was somewhat boring in compromising the levee structure with that of channel modification at the transition between the two structures. Even if it was difficult in construction, the material of the levee construction was easily accessible from channel modification excavation. In this thesis, it’s recommended to use excavated soil during channel modification for levee construction. This shows that the cost of transportation of levee construction material is most efficient as compared with the other alternative that was used for this study. The Levee was constructed for both right and left of the bank where there was over flanking of flood during simulation time for the upstream river station (6306.375) to downstream river station (4340.317). The Channel modification starts at the end of the downstream levee defined above to the end of the flood plain reach (41.70034). The dimension of all geometry for both levee and channel modification alternatives as defined above. The alignment of the levee was selected at upstream of the flood plain where the channel is less meandered as compared with downstream reach based on the suitability of construction of levee along the river. The velocity at upstream is less as compared with the downstream reach, therefore it is suitable for the construction of levee at upstream and the influence of the increase of velocity with the construction of levee had not to result in instability of the channel as compared with downstream reach.

3.12.4 Flood protection by Levee

Design of levees general

A levee is described as an embankment extending parallel to the river course and designed to protect the area behind it from overflow by floodwaters. Building levees along a stream as a shield against high water level has been one of the most ancient means of flood protection. For this study, it is proposed to construct levee in areas where flood depth above the banks of the river. For the safe design of levees, the following characteristics of the levees are determined.

Height of the levees

When levees are built along the main channel, a compound cross-section is obtained, the form of which depends on the distance of the levees from the stream bank.

The same discharge could be carried between higher levees built close to the riverbank, and low levees placed away from the river. Except for some special cases, the distance from the riverbanks and thus the heights to the levees is generally based upon economics, safety, and hydraulics, and freeboard consideration. The main purpose of the freeboard is to provide for additional safety against exceptional flood events. It should be sufficient to allow for settlement and to contain wind-driven waves. This additional height is 0.30-0.4m [36]. The design water levels were based on the design discharge of a return period of 100yrs. From the HEC-RAS result of water surface profile analysis for a 100-year return period, the flood water level above the banks has been determined. Adding the freeboard of 0.3 m on this flood depth above the banks, the height of the levees was computed and presented for both left and right banks of the river in the result section of this thesis. The levees should be placed at a fair distance from the main channel to avoid erosion along the reach. The inter-levee distance should remain more or less constant along the reach [36].

3.13 Decision analysis for selection of optimal proposed mitigation alternatives

A multi-criteria decision analysis method, TOPSIS ([37] and;[38]), was used to find optimal flood mitigation alternatives by evaluating with different criteria. In this method, the optimal alternative could be found based on the shortest distance from an ideal solution, and the furthest distance from anti-ideal solutions. As [39], showing a decision matrix is two dimensional (m*n) matrix for which A1, A2, A3,………Am are m times alternatives among which decision-makers have to choose, and C1, C2,…….Cn are n decision criteria in which flood mitigation alternatives are compared based.

X_{mn} indicates the performance rating of alternative A_m concerning criterion C_n. The relative importance of each criterion is usually given by a set of weights that are normalized to sum one. In the case of n criteria, a weight set is:

$$W = (W1, W2, W3, \ldots\ldots, Wn \ldots \text{Equation 3.6)}$$

$$\text{When } \sum_{j}^{n} Wj = 1$$

Then, the normalized value X_{ij} is calculated as:

$$r_{ij} = \frac{Xij}{\sqrt{\sum_{i=1}^{m} Xij^{\wedge}2}}, i=1,2,3,\ldots\ldots,m; j=1,2,3,\ldots\ldots,n \ldots\ldots \text{Equation 3.7}$$

The weighted normalized value Vij was calculated as:

$$V_{ij} = W_j X_{ij}, i=1,2,3\ldots,m; j=1,2,3\ldots\ldots,n \ldots\ldots \text{Equation 3.8}$$

Where W_j was the weight of the j^{th} criterion and $\sum_{j}^{n} Wj = 1$.the ideal A*and A'anti-ideal solution could be determined as:

$$A^* = \{(max_i V_{ij} \mid j \epsilon J), (min_i V_{ij} \mid j \epsilon j') \mid i=1, 2\ldots\ldots, m\} \ldots\ldots \text{Equation 3.9}$$

$$= \{V_1^*, V_2^*, V_3^*, \ldots\ldots V_J^*, \ldots\ldots, V_n^*\}$$

$$A' = \{(max_i V_{ij} \mid j \epsilon J), (min_i V_{ij} \mid j \epsilon j') \mid i=1, 2, \ldots\ldots, m\} \ldots\ldots \text{Equation 3.10}$$

$$= \{V_1', V_2', V_3', \ldots\ldots V_J', \ldots\ldots, V_n'\}$$

Where J={j=1,2,…….,n│j} is associated with positive criteria, and J'={j=1,2,…………,n│j} is associated with negative criteria. The separation of each alternative from the ideal and anti-ideal solution could be measured.

$$S_I^* = \sqrt{\sum_{j=1}^{n}(Vij - v*j)^2} \quad \text{, i=1, 2,………, m………Equation 3.11}$$

$$S_I' = \sqrt{\sum_{j=1}^{n}(Vij - v'j)^2} \quad \text{, i=1, 2,………..., m………Equation 3.12}$$

At the final stage, the relative closeness of A_i concerning A* defined as:

$$C_i^* = \frac{Si'}{(Si' + Si*)}, 0 < C_i^* < 1, i = 1,2, \ldots, m \text{……..Equation 3.13}$$

The two decision criteria and sub-criteria involved in the comparison of four flood mitigation alternatives including flood risk (Inundation area and Flood level) and environmental impact (flow velocity and stream power). For all decision criteria and sub-criteria, a set of weight should be assigned to reflect the importance of that particular decision variable regarding the above equation. In this research, the researcher was assigned weight for all criteria based on their importance. A different weight was given for each criterion to identify the sensitivity of criteria to weight. To identify how different weights of the criteria affect the alternatives, scenario-based decisions should be used. To say the proposed alternatives are optimal different criteria should be applied. For this study, three decision-making scenarios were considered for comparison. The first scenario was flooding risk-oriented criteria which include flood inundation area and flood level sub-criteria.In these scenarios, higher weights were given for both flood inundation area and flood level. The second scenario was an environmental impact-oriented scenario which includes both stream power and velocity as sub-criterion.In this scenario, higher weights were given for both stream power and stream velocity. Higher priorities were given for environmental issues like erosion and bank cutting at river bend and downstream as the result of mitigation implementation. The final scenario was a balanced decision criterion. In this scenario, equal weights were given for all criteria which are 0.25 wights for both flood risk and environmental impact.

Table 3-11: Weight of all decision-making criteria based on different scenario

Decision Criteria		Decision Making Scenarios for flood Risk mitigation		
Criteria	Sub-Criteria	Risk-Oriented	Environmental-Oriented	Balance
Flood Risk	Inundation area (ha)	0.4	0.1	0.25
	Flood Level (m)	0.4	0.1	0.25
Environmental Impact	Flow Velocity (m/s)	0.1	0.4	0.25
	Stream Power (N.m/s)	0.1	0.4	0.25

CHAPTER 4 RESULT AND DISCUSSION

4.1 HEC-HMS Model calibration, validation, and result

The main purpose of using HEC-HMS in this study was to estimate the different frequency discharge using frequency storm. The result of this calibrated and validated model was used in water surface profile generation. The generated water surface profile in turn was used to generate flood inundation mapping. The calibrated and validated flood map in HEC-GeoHMS was used to propose a mitigation measure.

4.1.1 Flow calibration at Awash Belo gaging Station

The HEC-HMS program was selected for the current study due to its versatility, capability for flow generation, automatic parameter optimization, and its connection with GIS through HEC- Geo-HMS. The systematic search for the optimum parameter is carried out using automatic HEC- HMS Model parameter optimization from the model compute toolbar regarding observed flow at Awash Belo station.

Model parameters were calibrated manually followed by HEC-HMS automatic optimization until the satisfactory agreement between simulated and observed flow was obtained. The model goodness of fit and the model performance was evaluated after adjusting the parameters. Flow calibration was made by using eleven years (1990-2001) hydro-meteorological data with an equal length of time (11 years). During these periods, the simulated monthly and daily flow matched well with the observed monthly and daily flow. The first performance evaluation Nash and Sutcliffe Model Efficiency (NSE) [Nash and Sutcliffe, 1970] for monthly streamflow calibration is 0.8. The second performance evaluation result that is Pearson's Coefficient of Determination (R^2) gives 0.8 and 0.88 for daily and monthly calibration respectively. Hence, HEC-HMS can predict the water potential of the basin. However, the model slightly overestimates the peak flow in most of the simulation periods. The results are summarized in the graphical form as follows.

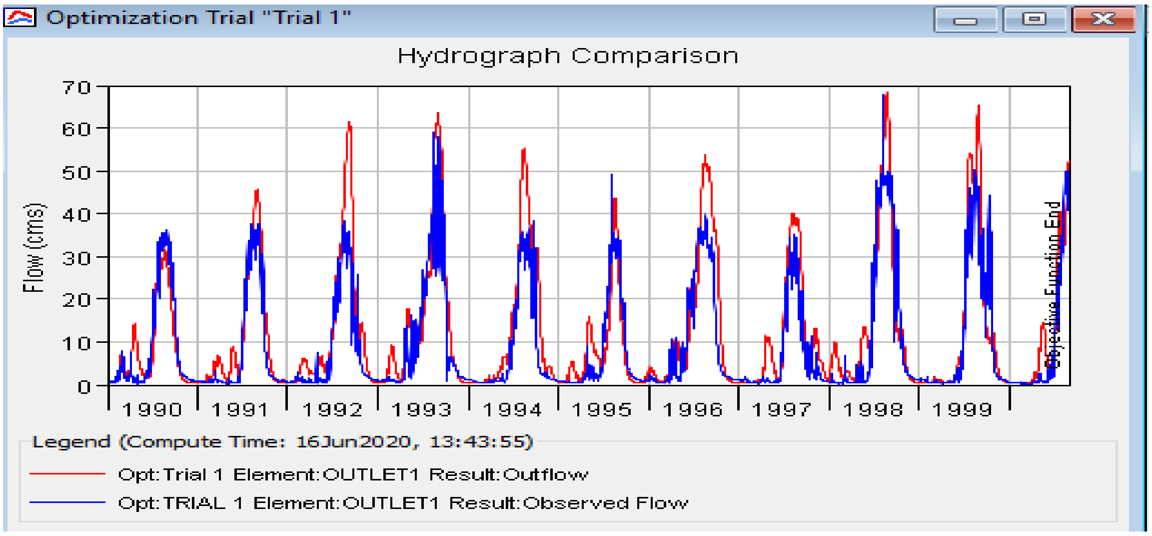


Figure 4-1: Calibrated model output daily time series

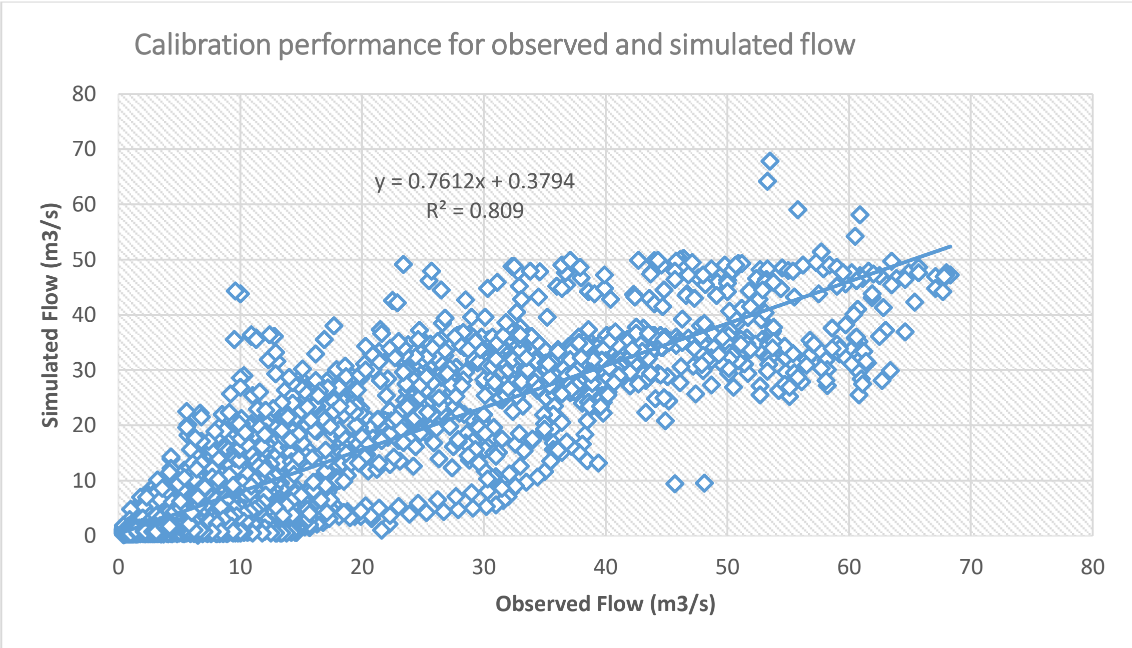


Figure 4-2: Daily Comparison between observed and simulated flow

4.1.2 Flow validation at Awash Belo basin

The model performance is validated accordingly. For the HEC-HMS model, there are a total of 16 years taken for both calibration and validation. The validation time was selected as the last five years of the total (2001-2005). Accordingly, a good match between measured and simulated flow was obtained in the validation period (R^2 = 0.75 and NSE = 0.7). The model captures the peak flow and simulated flow follows the pattern of the observed flow.

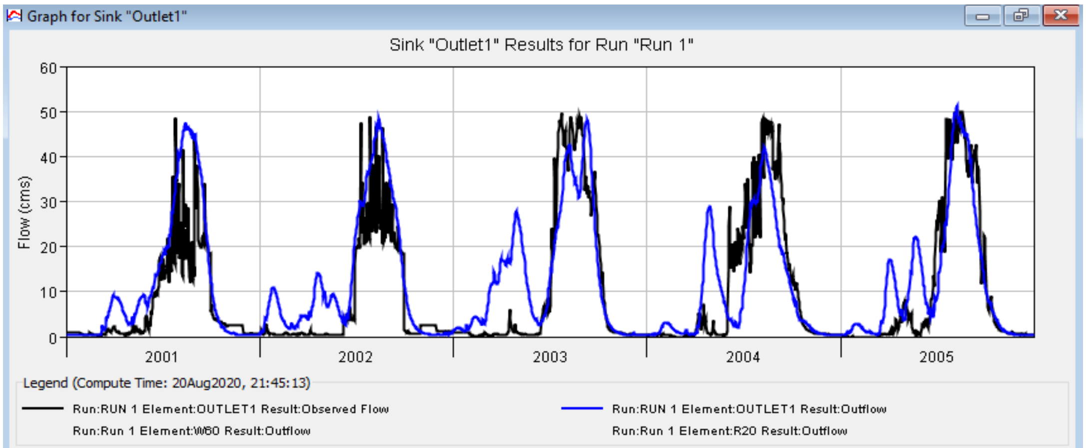


Figure 4-3: Daily validated flow for five years

4.2 Optimized parameter for Awash Belo basin

The optimized parameter is the parameter obtained after calibration and validation of observed and simulated flow with the required performance.

The values of the optimized parameters here were used in flood frequency storm estimation. The initial parameter used in the table below was extracted from the catchment characteristic using HEC-GEO-HMS and used as an initial parameter before calibration. The optimized parameter in the table below was also adjusted values of initial parameters through calibration.

Table 4-1: Optimized parameters of HEC-HMS for Awash Belo gaging station

Component	Parameter	Unit	Initial	Optimized
W40	Subbasin Lag	MIN	30000	30000
W50	Subbasin Lag	MIN	25774	25774
W60	Subbasin Lag	MIN	1321.2	1322
W40	Constant Rate	MM/HR	0.5	0.95327
W50	Constant Rate	MM/HR	0.33	0.30268
W60	Constant Rate	MM/HR	0.4	0.0050758
W40	Initial Loss	MM	16.8404	17.298
W50	Initial Loss	MM	12.7112	12.454
W60	Initial Loss	MM	14.6985237	15.269
Reach, R20	Lag	MIN	1200	501.97

4.3 Modeling by frequency storm method

These methods play great roles in the estimation of stream runoff from different return period storm.

4.3.1 The output of HEC-HMS by frequency storm

The model can produce and generate values for different flow conditions (return periods). After the whole above input parameters, was filled the flow values are found accordingly. The result table HEC-HMS output shows that the minimum peak flow for the Awash River occurred at 2 year return period for 24-hour storm duration and the maximum obtained with a 100-year frequency storm for the 24-hour duration. The value being 30 m^3/s and $100m^3/s$ for 2 years and 100-year frequency respectively was obtained at the minimum and maximum return period from selected reoccurrence interval.

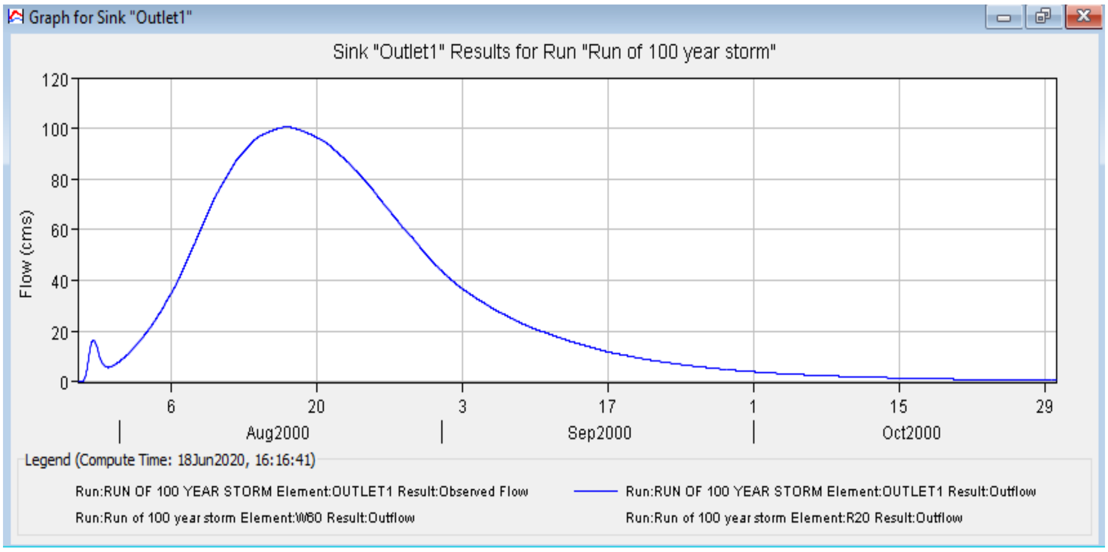


Figure 4-4: 100 Year flow hydrography of the Awash River

4.4 Geometrical pre-processing

This study area is under the influence of cloud cover which results in less quality of DEM and Satellite data even with a high resolution of (12.5 m DEM). Therefore it was difficult to extract river geometry without using alternatives. To overcome this problem the four comparison techniques with field-collected river cross-section were applied to obtain realistic river geometry especially stream channel for model simulation.

1. River cross-section obtained during the field inspection

There was no recorded historical surveying data for the study area. Therefore, to overcome this problem the river geometry data was collected during field inspection by the researcher. The data collected during this time might be insufficient to represent the river geometry. Due to that the alternative listed below were analyzed to obtain a realistic stream channel by comparing and contrast the whole alternative obtained from satellite and (12.5 m*12.5 m) DEM of the study area. An observation and field measurement shows that the stream channel shape and size was assumed to be uniform. The total length of the flood plain is 6 kilometers which show as the possibility of river cross-section collection due to the short distance of the reach. Therefore the data was collected at different ten river stations with 800 m and above distance from the confluence of Teji and Awash rivers.

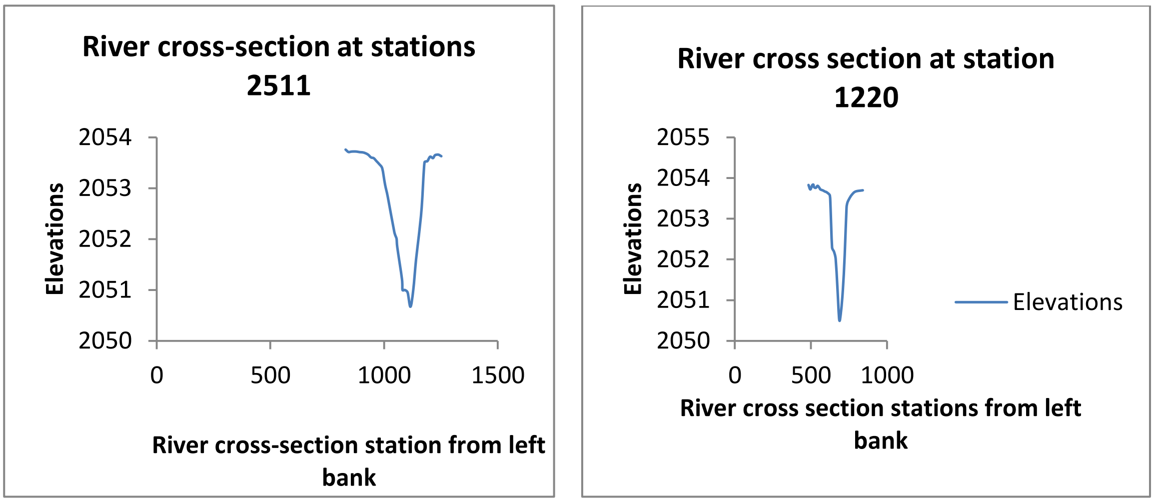


Figure 4-5: Some of the samples of river cross-sections taken from the field

2. Satellite data for stream channel geometry extraction

The different bands of Landsat 7 Geotiff were applied for study flood plain to separate land use and water body to obtain clear channel by image composition for all bands. This alternative was less reliable for channel size extraction rather the shape of the channel.

Landsat-8 river cross-section extraction

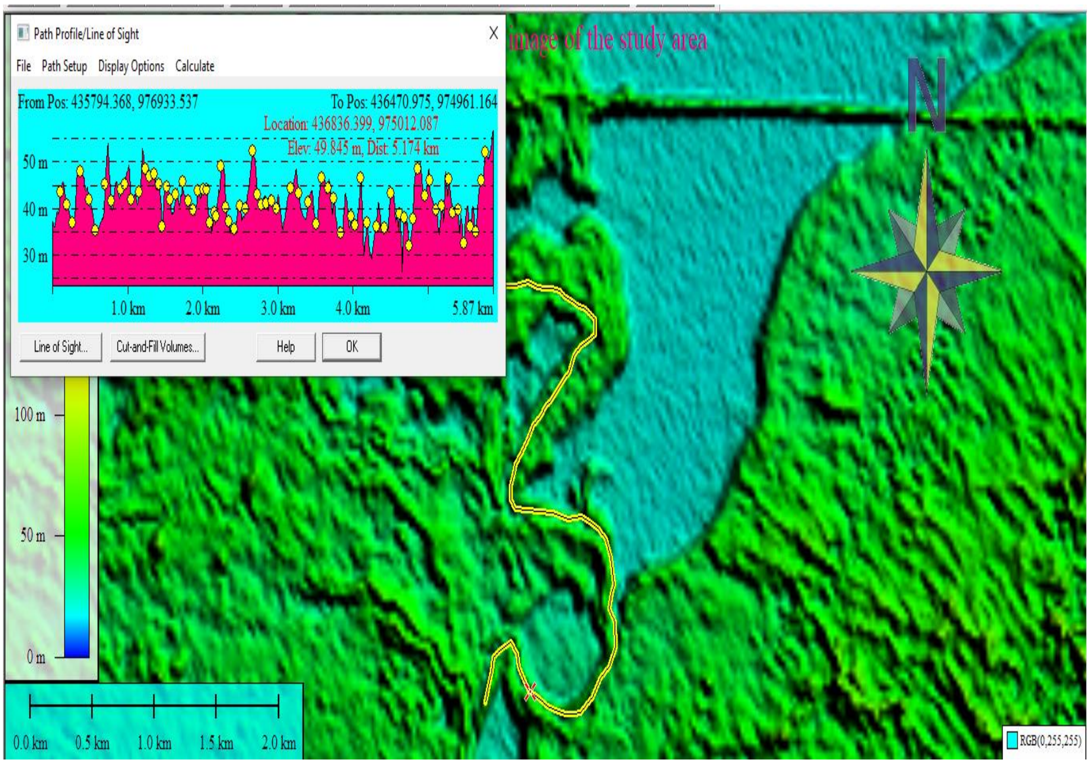


Figure 4-6: Flood Channel Profile for steady reach

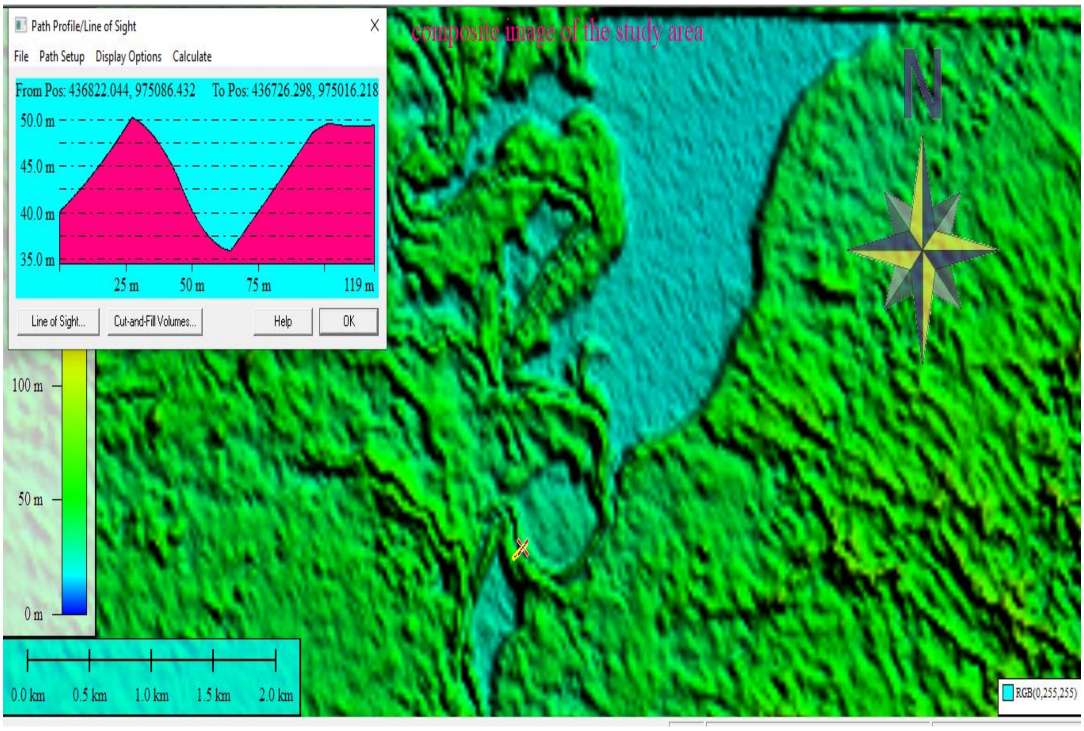
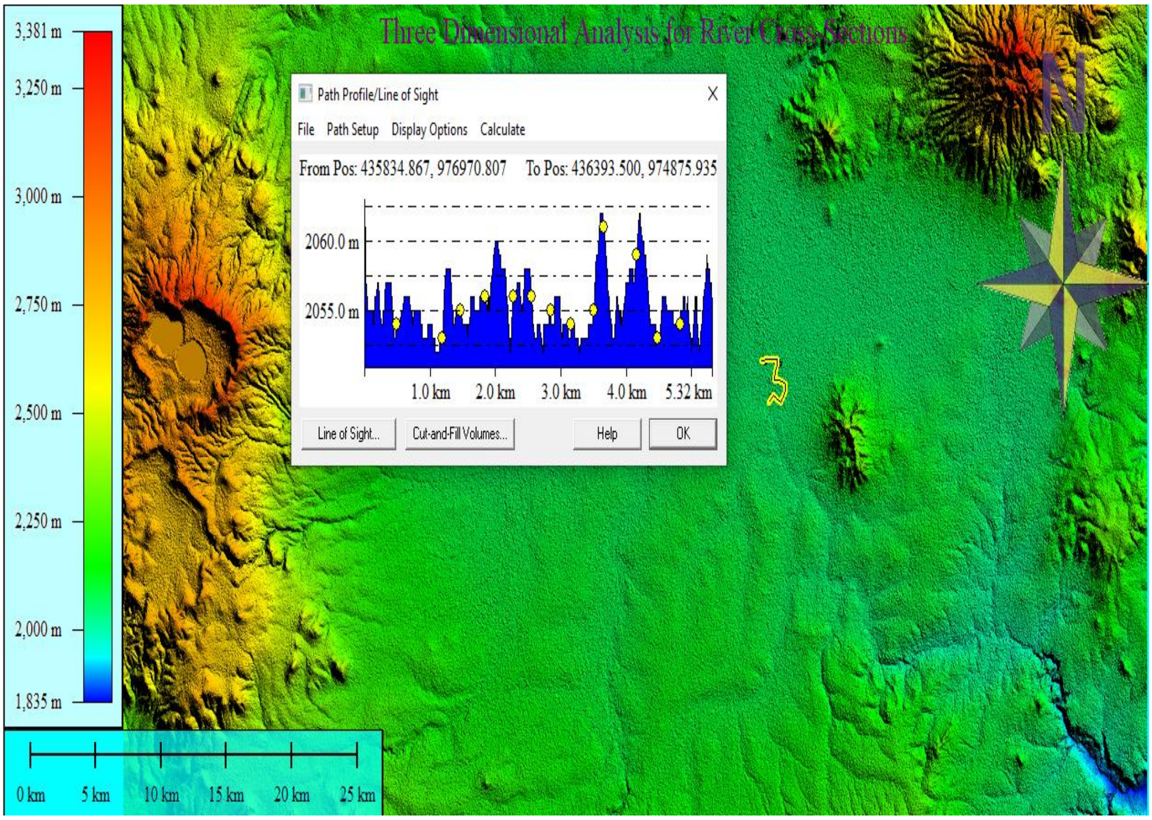


Figure 4-7: River cross-section sample was taken at 800m above interval

3.3D analysis for (12.5 m*12.5 m) DEM for river geometry extraction

This alternative uses the DEM and Satellite data and interpolates line at a specified station which results in river cross-section and longitudinal profile.



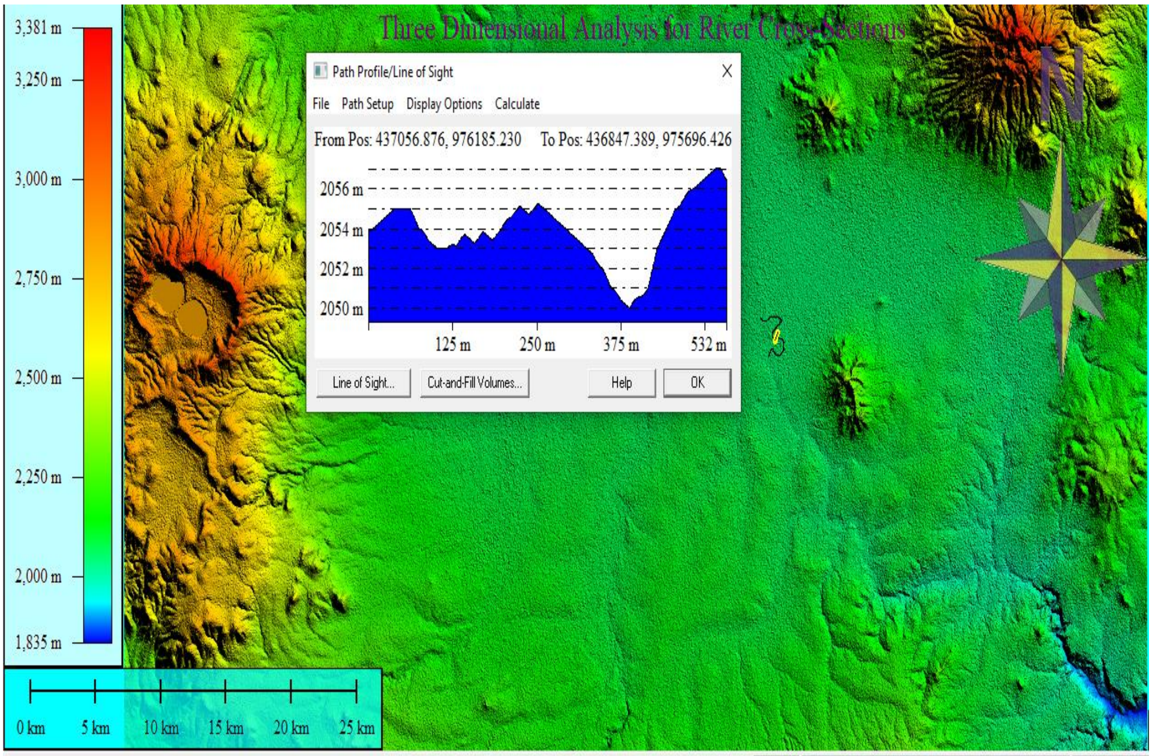


Figure 4-8: River cross-section sample was taken at 800m above interval from 3D analysis

4. Google earth river cross-section

This alternative clearly shows the channel and topography of a steady area. River cross-section at the study area was collected by using a path profile and cross-section profile drawing tool. This technique was not much really because it was difficult to obtain a stream channel bed level. After all, it was difficult to obtain a variation of elevation at a different cross-sectional station.

Hint: All the above four techniques except field-collected data were not directly used for the model but they used me to extract river profile and compare and contrast them with field results. During the field measurement, it was difficult to have all elevation within a short intermediate distance due to the inaccessibility of banks but possible to have stations at in an accessible from those above alternatives by compromising with field-collected data.

4.5 HEC-GEORAS river geometry analyzes

4.5.1 Stream centerline

The river and reach network is represented by the stream centerline layer. The network is created on a reach basis, starting from the upstream end and working downstream following the Awash River channel. Each reach is comprised of a river name and reach name.

4.5.2 Bank lines

The Bank lines layer defines the main channel flow from flow in the over banks. Bank station will be assigned to each cross-section based on the intersection of the bank lines with the cut lines. It is often more efficient to skip this layer and complete the data in HEC-RAS using the graphical cross-section editor tools.

In general, the bank lines will serve three purposes,

- Assign bank stations,
- Calculate overbank flow paths, and
- Serve as the distinction between channel and floodplain roughness.

4.5.3 Flow paths centerlines

The flow path centerlines identify the hydraulics flow path in the, left overbank, the main channel, and right overbank by identifying the center of mass of flow in each region. Further creating the flow path centerlines layer will assist you in properly laying out the cross-sections cut lines.

4.5.4 Cross-section cut lines

The cross-section is the main input parameter for the HEC-RAS model. It is used to extract elevation across the river by following the river length. Cross-sections play a great role in developing an attribute table for the HEC-GeoRAS model like right and left bank station at the intersection with bank lines.

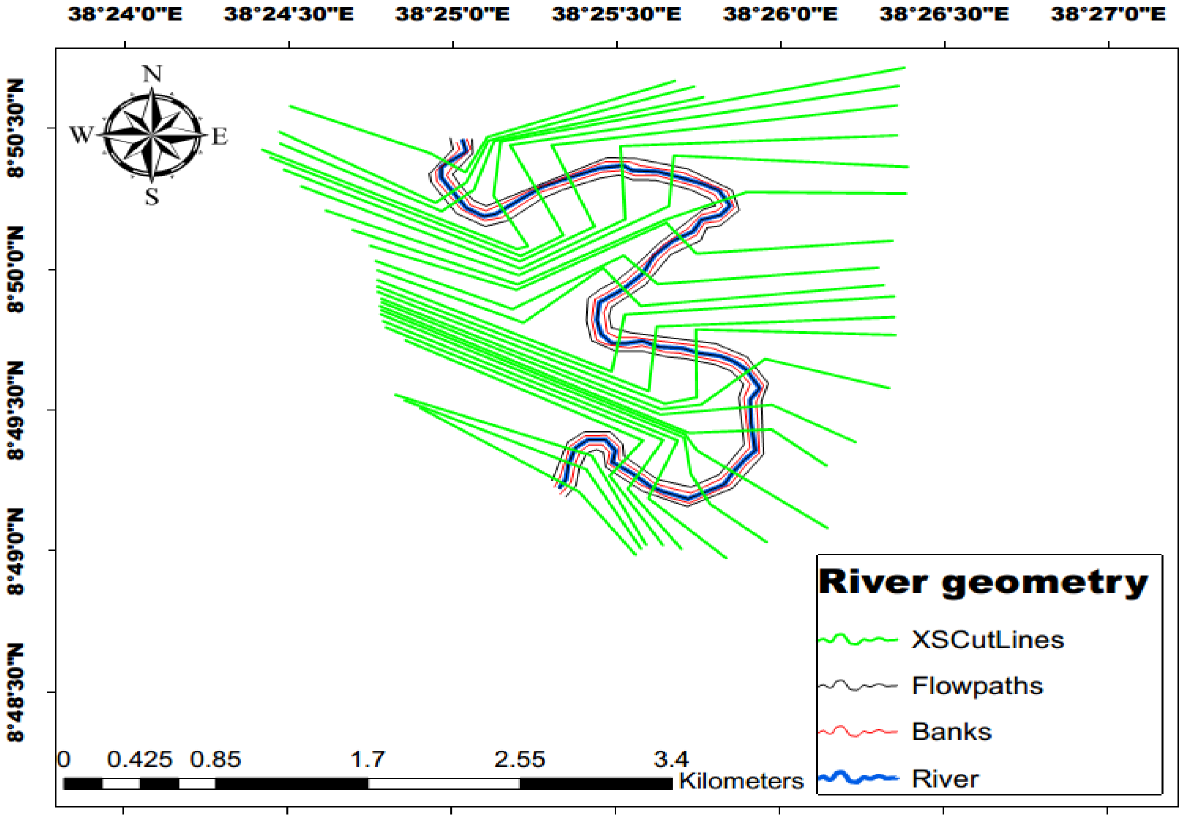


Figure 4-9: - Geometric pre-processing result in the Belo flood plain

4.6 Best indices for calibration and validation of the HEC-RAS model using Landsat-7 image

Among all indices the NDWI was the best match with HEC-RAS simulated flood map in terms of the magnitude of the area it covers and the two event flood overlay (satellite and simulated) from figure 5-10.

4.6.1 Landsat-7 image and simulated flood map overlay for calibration event of 03/09/2001

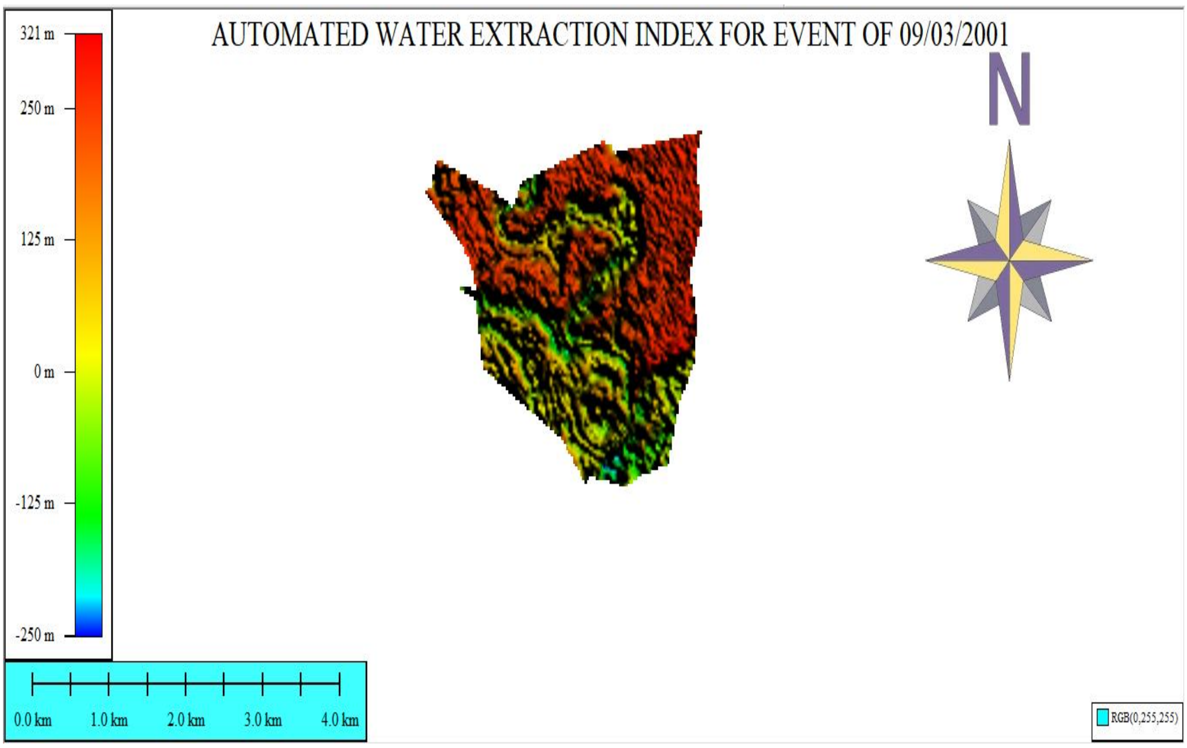


Figure 4-10: Belo Automated water extraction index from Landsat-7 event

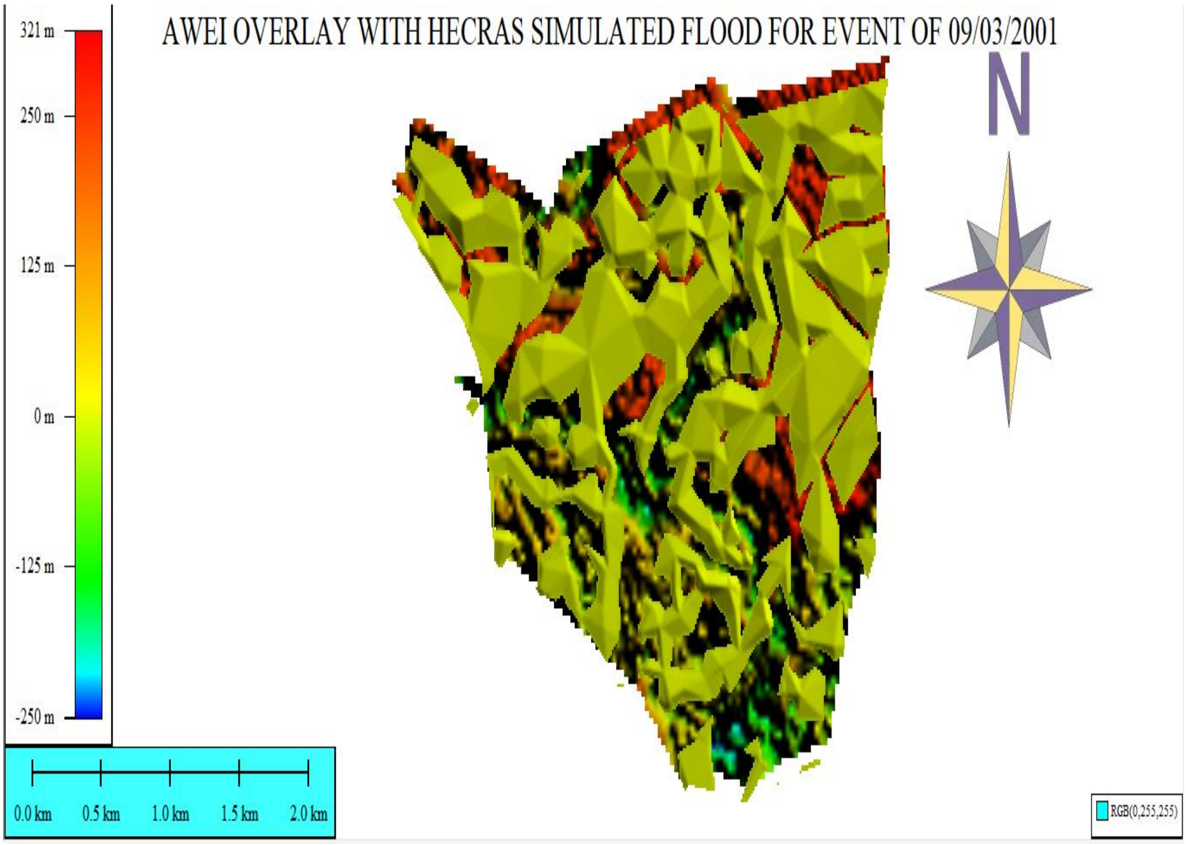


Figure 4-11: Belo AWEI overlay with a simulated flood map of 09/03/2001

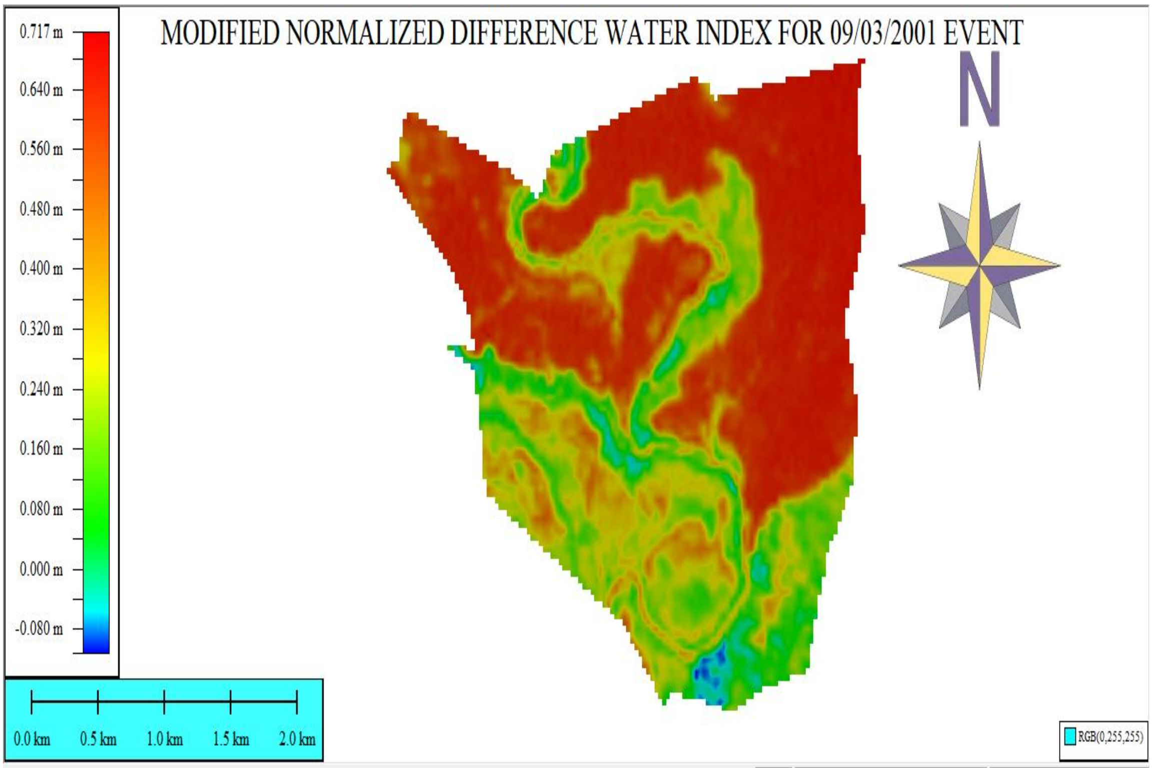


Figure 4-12: Belo modified normalized difference water index **from Landsat-7 event**

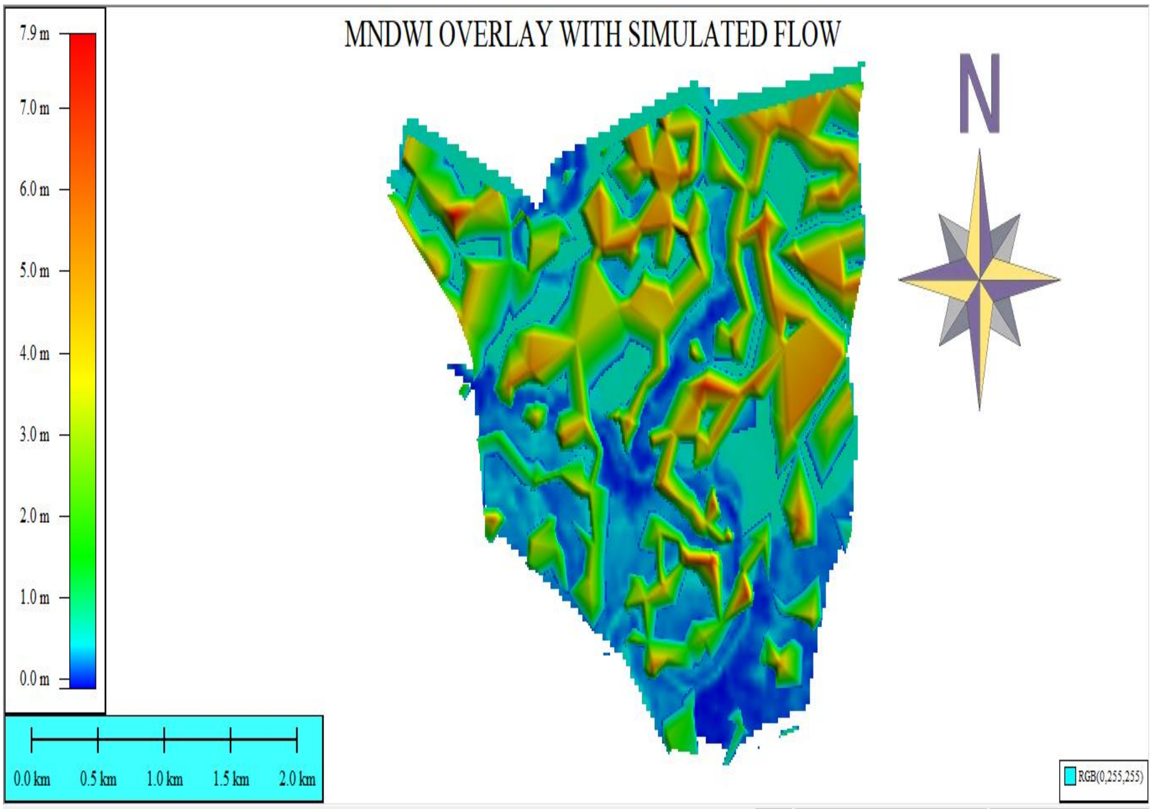


Figure 4-13: Belo MNDWI overlay with flood map of 09/03/2001

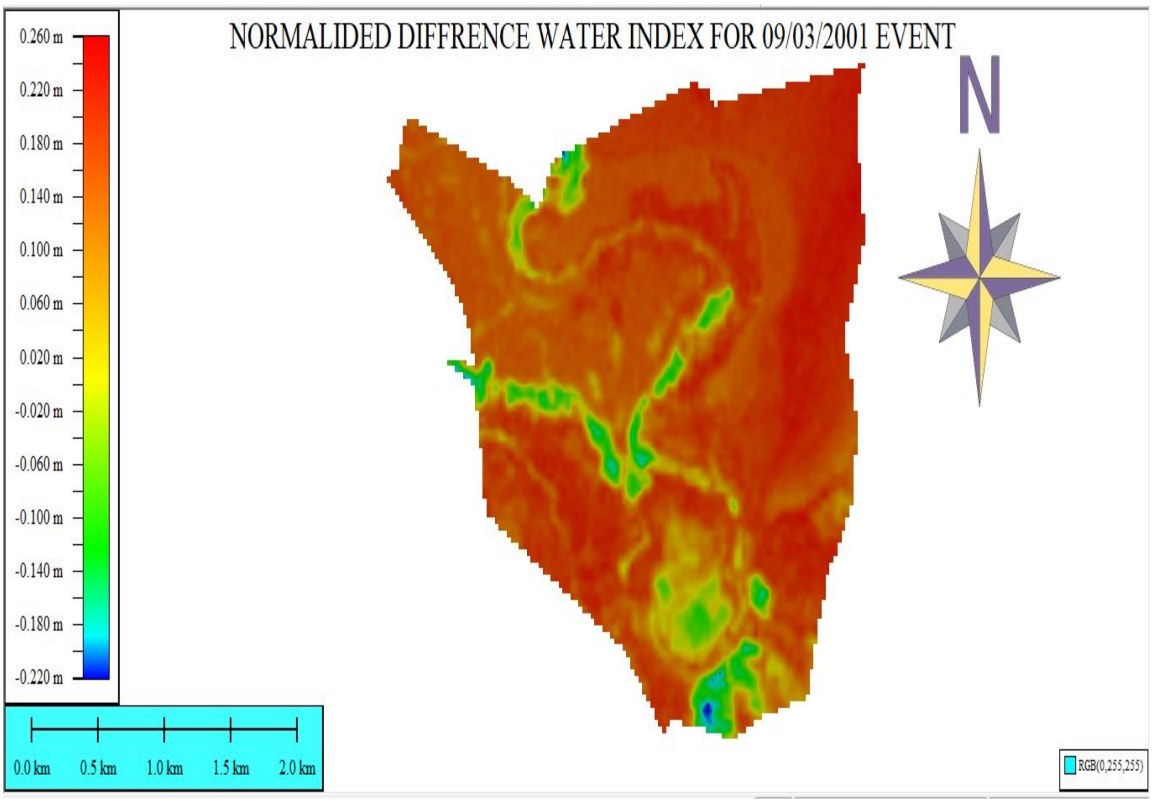


Figure 4-14: Belo normalized difference water index **from Landsat-7**

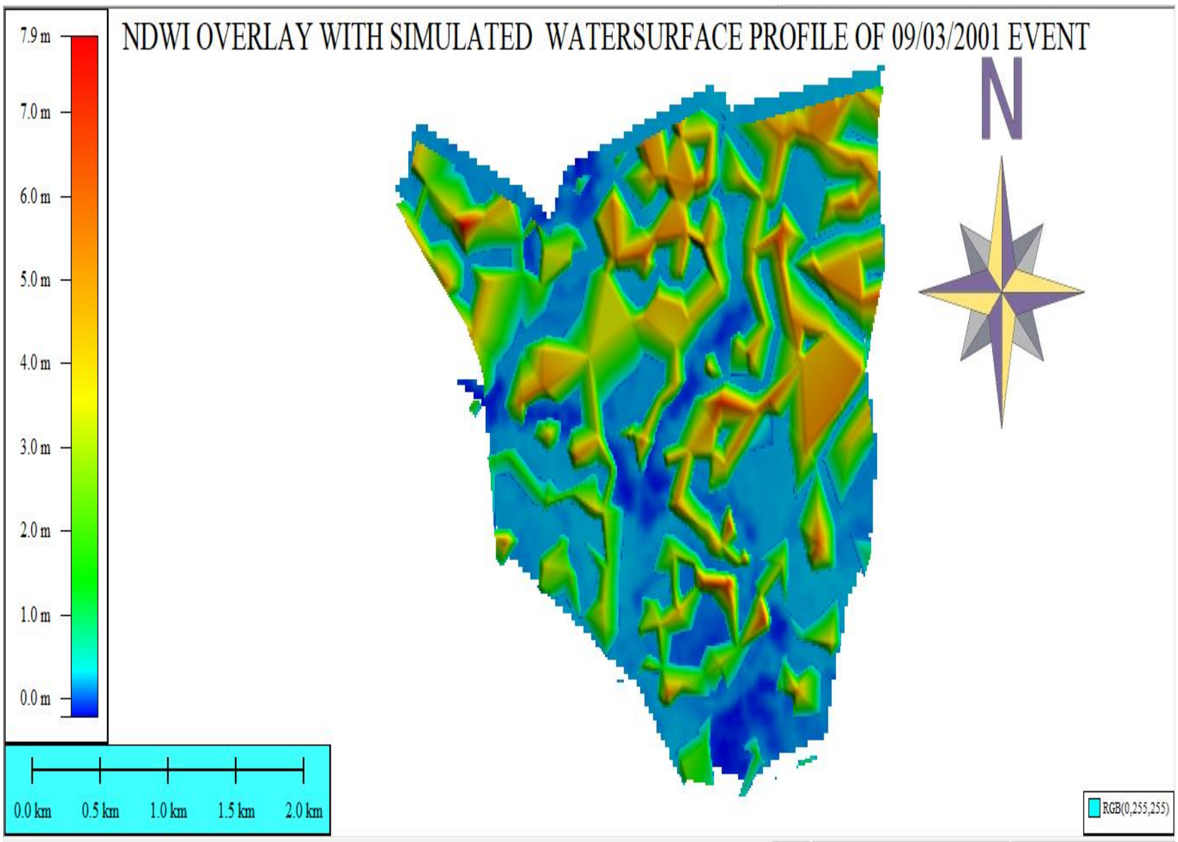


Figure 4-15: Belo NDWI overlay with a simulated flood map of 09/03/2001

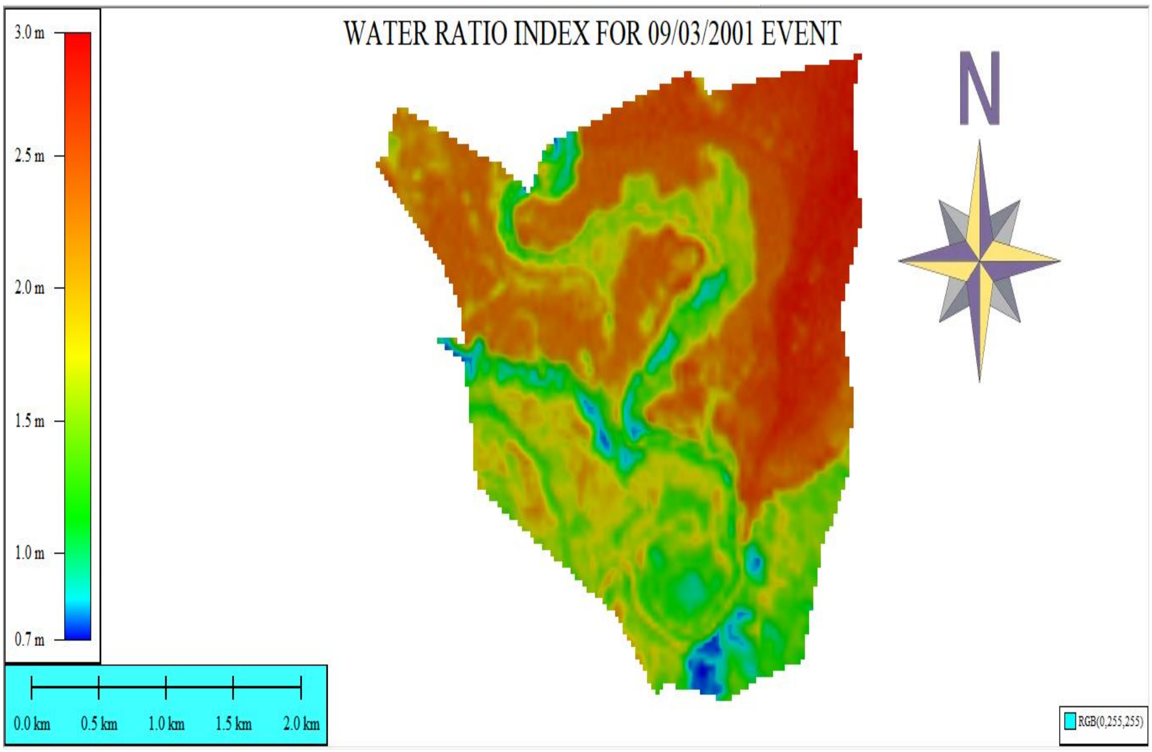


Figure 4-16: Belo normalized difference water index **from Landsat-7**

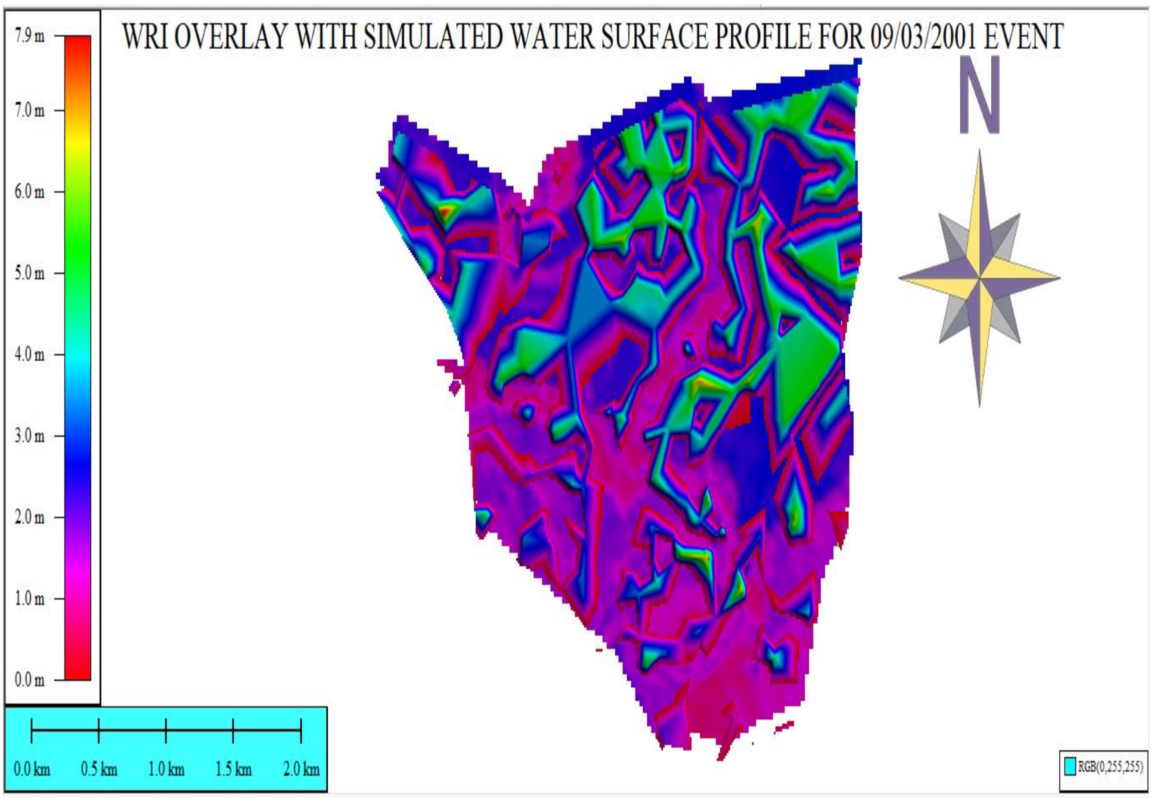
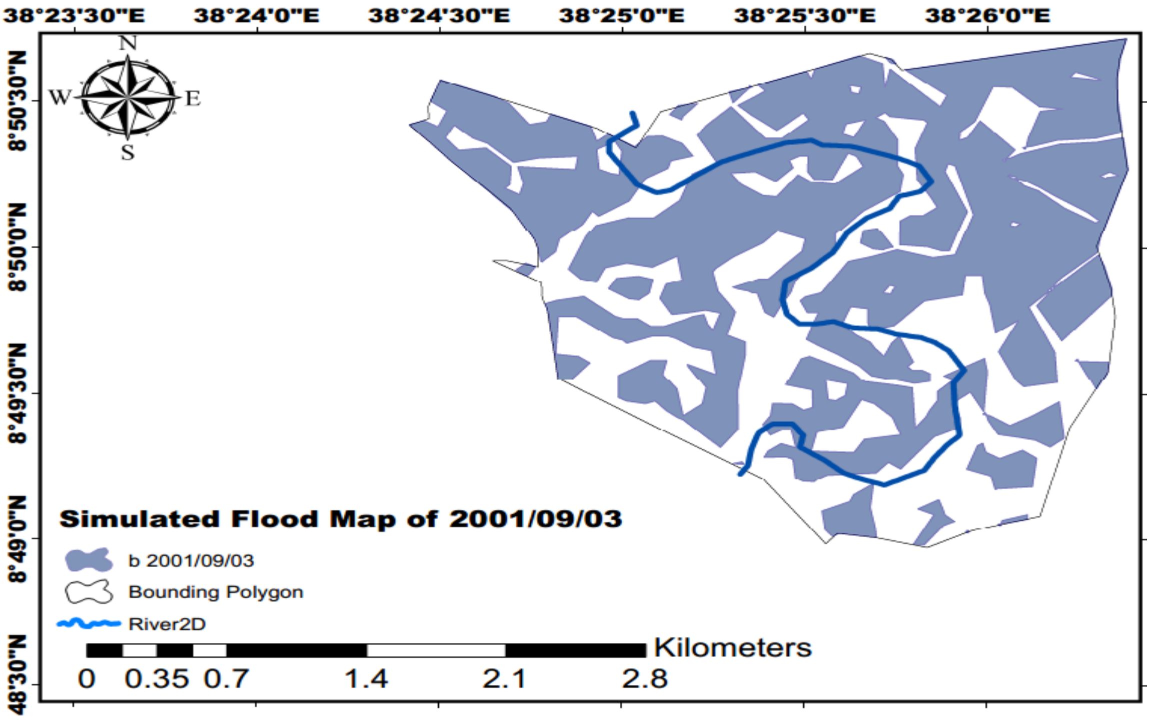


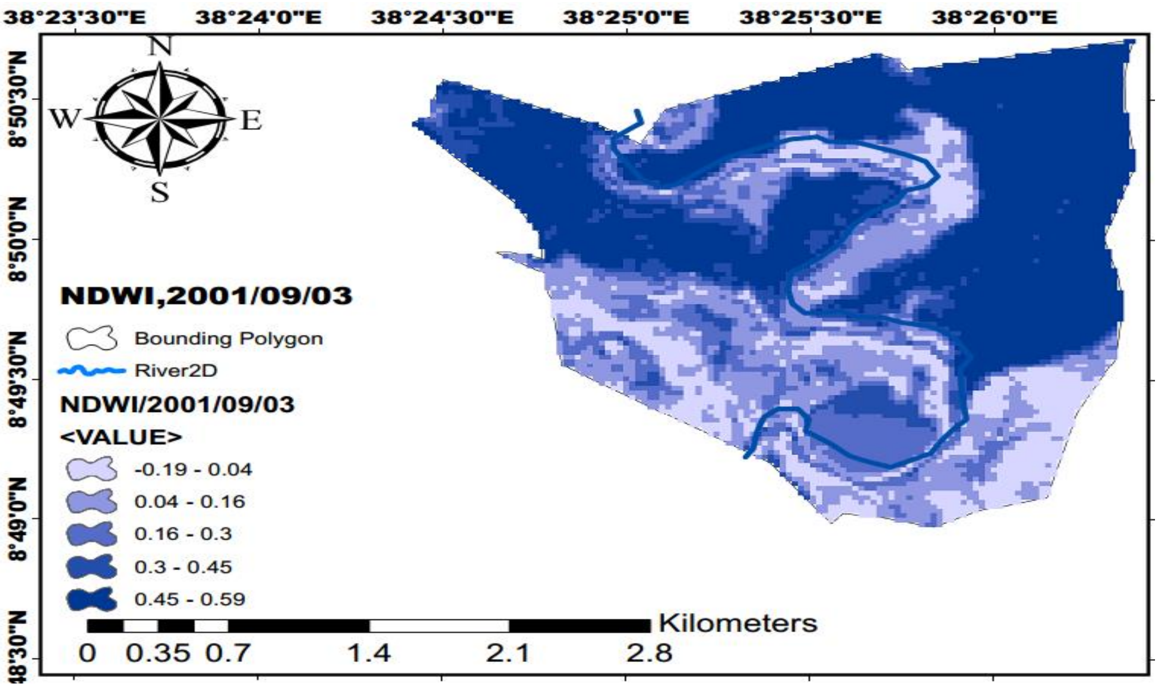
Figure 4-17: Belo WRI overlay with a simulated flood map of 09/03/2001 event

4.6.2 Calibration of HEC-RAS using the event of September 3, 2001

In this thesis works the two logics were applied. The result of the inundation map from HEC-RAS was calibrated using a flood event of (September 03, 2001) by changing the manning coefficient roughness until the NDWI of the satellite image for the above event resembles each other as follow.



A) Inundated flood extent of simulated

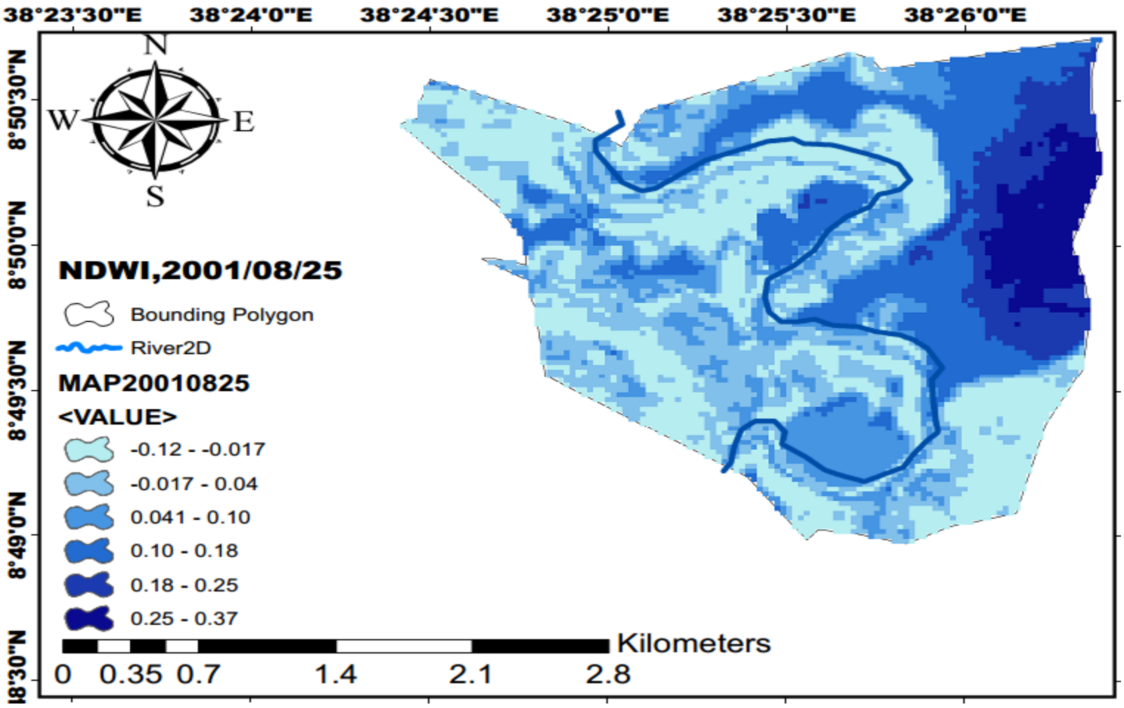


B) Image of Flood Extent NDWI

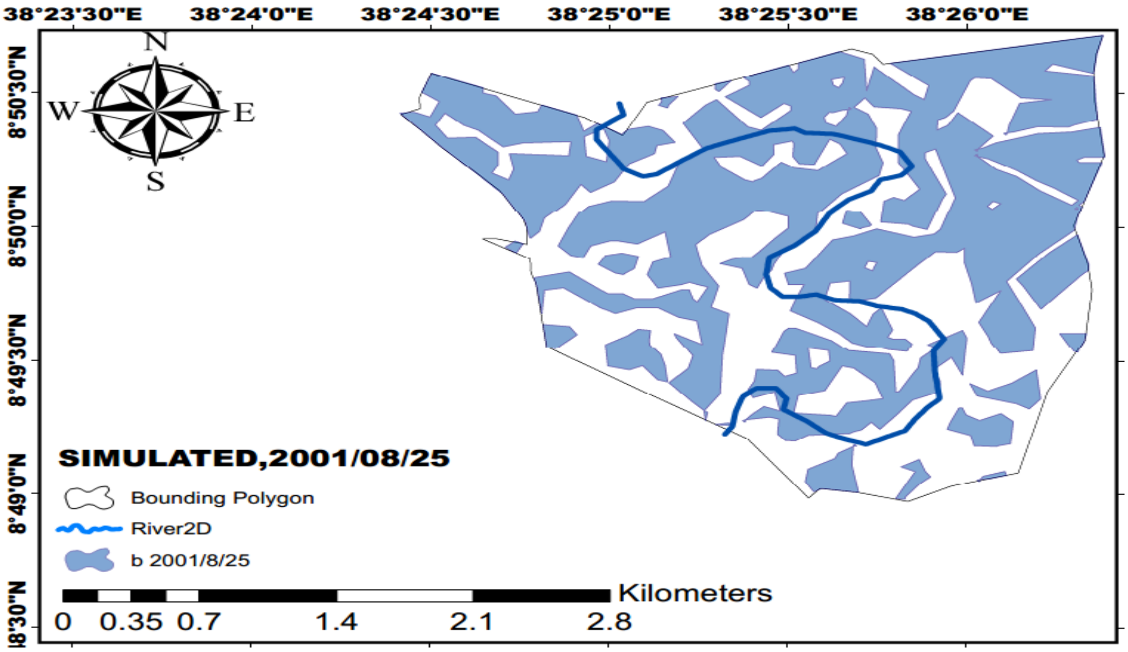
Figure 4-18: Comparison of simulated inundation map and flood extent

4.6.3 Validation of HEC-RAS using the event of August 25, 2001

The NDWI can develop water information efficiently in most cases as compared with different indices above. It is sensitive to build-up land and results in over-estimated water bodies. Inundated flood extent of the same year of the study areas are approximately near to each other and this indicates that the inundated flood extent obtained from HEC-RAS was promised and therefore it is validated according to the NDWI flood map of flood extent during the event according to [31].



A) Image of flood extent NDWI



B) Inundated flood extent of simulated

Figure 4-19: Comparison of simulated flood and satellite Image for validation

Based on trial and error, the model runs a manning roughness coefficient of 0.042, and 0.035 produced the best fit against the satellite data for the river main channel and flood plain respectively. The flood depth from the optimal manning's in relative agreement with all NDWI flood delineated ones.

4.7 HEC-RAS outputs

4.7.1 Cross-sectional view

The 2 and 5 year return period floods 30.50 m^3/s and 41.5 m^3/s respectively have not brought significant change because of the depth of flood corresponding to this discharge is less as compared with other 50 and 100-year water depth at the specified cross-section. However, the 50 and 100 year return period floods have higher magnitudes of 82.8 and 100 m^3/s, respectively; these discharges fill the whole cross section and inundate the adjacent floodplains.

4.7.2 Water surface profile

The water surface profiles of the different return periods, i .e.2, 5, 10, 25, 50, and 100 years, was calculated for this study. This is basic for Ras post-processing in the delineation of the flooded area. The TIN that was used in the flood plain delineation was taken or developed based on the result of the water surface profile of the Belo flood plain.

4.8 Floodplain delineation using raster

Floodplain delineation used the Water surface TIN and terrain model to calculate the floodplain boundary and inundation depth. This is done by comparing both the water surface TIN and the Digital terrain model of the area. First, the water surface TIN is subtracted from the digital terrain model of Belo flood plain and the positive difference shows a flooded area whereas the negative difference shows a dry or ineffective place. After the inundation map was created, a check on the quality of the inundation polygon was made. The ability to judge the quality of terrain and flood inundation polygon comes with the knowledge of the study area, satellite data analysis, and experience. The quality of the inundation polygon for this study was also done through satellite image comparison for the selected event during calibration and validation.

The entire study (both dry and wet) area was 44.95km^2 or 4495.02 hectares. From the entire study area, 500.6 ha and 505ha portion was covered by a flood of 50 and 100 year return periods respectively.

4.9 Detail design output for different four mitigations alternatives

4.9.1 Detail design result for two-stage channel modification (Alternative-1)

Table 4-2: Alternative-1 ground design output

River station	Minimum channel level	Left side station	Maximum Left Elevation	Right side station	Maximum Right Elevation	Maximum Elevation from bed level	2nd stage width
6306.375	2054.11	1203.93	2055.11	1623	2055.1	1	419.07
6081.194	2053	1318	2054.49	1769	2054.45	1.49	451
6015.721	2053	1371	2054.52	1931	2054.5	1.52	560
5717.513	2052	2575	2054.43	3321	2054.43	2.43	746
5488.861	2052	2275	2054.22	3104	2054.16	2.22	829
5291.171	2052.54	1621	2054.34	2513	2054.34	1.8	892
4983.382	2052.57	1363	2054.45	2018	2054.43	1.88	655
4340.317	2052.19	811	2054.42	1192	2054.42	2.23	381
3613.413	2051.54	1189	2054.23	1741	2054.22	2.69	552
3457.891	2052.99	1351	2054.39	1708	2054.39	1.4	357
3106.13*	2053.5	1338	2054.4	1757	2054.36	0.9	419
2754.359	2054	1266	2054.71	1616	2054.7	0.71	350
2511.856	2050.63	841	2052.29	1464	2052.25	1.66	623
2188.504	2053.67	650	2054.02	1097	2054.03	0.35	447
1921.757	2051.54	795	2053.26	1159	2053.23	1.72	364
1446.303	2053.01	486	2053.9	808	2053.8	0.89	322
1220.236	2050.5	407	2053.6	878	2053.03	3.1	471
922.4112	2052.44	562	2053.66	1001	2053.55	1.22	439
802.3994	2051.32	378	2053.03	729	2053.03	1.71	351
268.7353	2052.73	676	2052.97	938	2052.97	0.24	262
41.70034	2048.46	602	2048.95	903	2048.94	0.49	301

Table 4-3: Water Profile for two-stage channel widening

River Station	Q Total	Minimum Channel Elevation	W.S. Elevation	E.G. Elevation	E.G. Slope	Channel velocity	Flow Area	Top Width
	m3/s	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)
6306.38	100	2054.1	2054.57	2054.59	0.001508	0.55	163.22	358.79
6081.19	100	2053	2054.56	2054.56	0.000033	0.18	570.27	474.17
6015.72	100	2053	2054.56	2054.56	0.00002	0.14	709.85	567.26
5717.51	100	2052	2054.56	2054.56	0.000002	0.06	1595.9	780.77
5488.86	100	2052	2054.56	2054.56	0.000001	0.05	1881.21	915.71
5291.17	100	2052.5	2054.56	2054.56	0.000002	0.06	1698.85	1029.84
4983.38	100	2052.6	2054.56	2054.56	0.000005	0.08	1178.77	723.71
4340.32	100	2052.2	2054.55	2054.55	0.000008	0.12	835.12	432.52
3613.41	100	2051.5	2054.55	2054.55	0.000002	0.06	1497.78	600.48
3457.89	100	2053	2054.55	2054.55	0.000036	0.19	511.87	418.44
3106.13	100	2053.5	2054.53	2054.53	0.000148	0.29	355.13	458.29
2754.36	100	2054	2054.24	2054.36	0.021052	1.31	66.68	289.46
2511.86	100	2050.6	2054	2054	0.000002	0.07	2604.08	3239.25
2188.5	100	2053.7	2053.88	2053.98	0.021642	1.23	71.12	349.46
1921.76	100	2051.5	2053.42	2053.42	0.000021	0.16	607.6	400.33
1446.3	100	2053	2053.25	2053.37	0.019679	1.29	65.43	271.6
1220.24	100	2050.5	2053.22	2053.22	0.000004	0.09	1076.41	521.59
922.411	100	2052.4	2053.2	2053.21	0.000332	0.36	278.55	460.65
802.399	100	2051.3	2053.2	2053.2	0.000027	0.19	556.16	409.38
268.735	100	2052.7	2053.02	2053.14	0.020629	1.51	66.34	290.07
41.7003	100	2048.5	2048.96	2048.99	0.002151	0.68	134.74	305.5

4.9.2 Detail design result for one stage channel modification (Alternative-2)

The result shown in the tables shows a detailed description of one stage channel widening and degrading. From this table, it is easy to understand the layout of mitigation taken to the field. The non-uniformity of the mitigation width and depth developed from the natural variation of stream and topography of the area. In channel modification, I recommended that no need for modifications for the bed of the channel, and the width of the modification is between natural river banks for all cross-sections to be stable and economic.

Table 4-4: Alternative-2 ground design output

River Station	Bed level	Max.right of bank excavation level	Max.Left of bank excavation level	Depth of Excavation of left and right banks	Right bank excavation station	Left bank excavation station	Max. entire xsc's Excavation width
6306.375	2054.11	2055.7	2055.7	1.39	1621.62	1255.1	266.5
6081.194	2053	2055	2055.5	2.3	1766.58	1326.4	340.16
6015.721	2053	2055.6	2055.6	2.4	1884.29	1407.5	376.78
5717.513	2052	2055.49	2055.5	3.3	3298	2604	594
5488.861	2052	2055.56	2055.58	3.38	3081.61	2318.8	662.81
5291.171	2052.54	2055.05	2055.13	2.39	2604.42	1662	842.42
4983.382	2052.57	2055.51	2055.68	2.91	2040.29	1398	542.29
4340.317	2052.19	2055.49	2055.6	3.21	1228	804	324
3613.413	2051.54	2055.46	2055.49	3.75	1749.13	1211.6	437.49
3457.891	2052.99	2055.49	2055.61	2.42	1777	1340	337
3106.13	2053.52	2055.34	2055.34	1.62	1738.8	1351.6	287.16
2754.359	2054	2055.29	2055.46	1.26	1633.91	1292.8	241.12
2511.856	2050.63	2053.78	2053.87	3.04	1718.78	706.84	911.94
2188.504	2053.67	2054.4	2054.55	0.68	1060	679	281
1921.757	2051.54	2054.43	2054.43	2.69	1144	813	231
1446.303	2053.01	2054.5	2054.59	1.38	791	506	185
1220.236	2050.5	2053.67	2053.77	3.07	852.58	436	316.58
922.4112	2052.44	2053.71	2053.74	1.1	971.57	598	273.57
802.3994	2051.32	2053.76	2053.87	2.35	707.64	408.64	199
268.7353	2052.73	2053.41	2053.54	0.61	918	689.6	128.4
41.70034	2048.46	2050.77	2050.09	1.43	889	616	173

Table 4-5: Water profile for one stage channel widening and degrading

River Station	Q Total	Minimum Channel Elevation	W.S. Elevation	E.G. Elevation	E.G. Slope	Channel velocity	Flow Area	Top Width
	(m3/s)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)
6306.375	100	2054.11	2054.52	2054.55	0.002175	0.61	145.8	355.87
6081.194	100	2053	2054.52	2054.53	0.00002	0.14	624.08	421.07
6015.721	100	2053	2054.52	2054.52	0.000018	0.13	685.44	464.34
5717.513	100	2052	2054.52	2054.52	0.000002	0.06	1593.14	670.52
5488.861	100	2052	2054.52	2054.52	0.000001	0.05	1821.73	747.92
5291.171	100	2052.54	2054.52	2054.52	0.000002	0.05	1646.83	852.23
4983.382	100	2052.57	2054.52	2054.52	0.000004	0.08	1144.5	615.65
4340.317	100	2052.19	2054.52	2054.52	0.000007	0.11	861.18	399.47
3613.413	100	2051.54	2054.52	2054.52	0.000001	0.06	1498.69	525.19
3457.891	100	2052.99	2054.51	2054.52	0.00003	0.17	522.4	378.68
3106.13	100	2053.52	2054.49	2054.5	0.00011	0.24	349.77	368.41
2754.359	100	2054	2054.24	2054.36	0.020658	1.3	66.87	287.48
2511.856	100	2050.63	2054	2054	0.000001	0.04	3723.45	3239.44
2188.504	100	2053.67	2053.88	2053.99	0.021515	1.23	70.83	344.18
1921.757	100	2051.54	2053.41	2053.41	0.000017	0.15	597.19	325.57
1446.303	100	2053.01	2053.25	2053.37	0.020068	1.3	64.96	270.72
1220.236	100	2050.5	2053.21	2053.21	0.000003	0.08	1054.23	409.06
922.4112	100	2052.44	2053.2	2053.21	0.000267	0.32	272.56	365.22
802.3994	100	2051.32	2053.2	2053.2	0.000021	0.16	538.98	295.28
268.7353	100	2052.73	2053.01	2053.15	0.019224	1.41	60.87	220.72
41.70034	100	2048.46	2048.95	2048.98	0.002158	0.68	128.96	269.88

4.9.3 Levee design result

For this particular research, the distance between the levee toe and channel has been chosen at suitable channel side character by considering hydraulics of flow, safety, and economies as an above table for both left and right banks.

In this particular case, it was proposed to construct homogenous levees with locally available material and with a height mentioned in the above tables. Detail of the riverbed and bank material investigation should be made to select the construction material that the levee should be constructed from. Data for the levee was entered together with the cross-section into the HEC-RAS model and the alignment, position, and height of the levee is observed during the simulation of the model.

A levee or a dyke is mainly used for flood protection by controlling the river and not by training the river.

The alignment of levees should follow the normal meandering pattern of the river. The retirement of the levees has to be governed by technical as well as economical and political considerations because the land falling within the levees is either to be acquired by the government or remains susceptible to floods. The levees are many times, pitched on the upstream side (i.e. water side). Launching apron may also be provided if the bank or levee is close to the main river channel. In this study area since the levee position is far away from rivers, it is not required to use a Launching apron rather it's recommended to pitch the riverside slope of the levee for safety purposes.

Design of levee Section

Levees are just like earthen dams with the difference that they are very long, come in operation discontinuously, and for a short time, and have limited possibilities for selection of their alignment along favorable geological strata.

Their sections should be designed in such a way as to keep the seepage gradient inside the body of the embankment by at least one meter from below the top surface of the embankment. The normal value of usually adopted seepage gradient varies between 4: 1 to 6: 1 (i.e. H: V) depending upon the character of the soil which may - necessitate riverside slopes varying between 2: 1 to 5: 1, landside slopes between 2: 1 to 7: 1, and top width between 2.5 to 10 m. The top level of the levee should be decided by leaving a sufficient freeboard varying between 0.3 to 1.5 m above the high flood level. Based on these criteria it’s recommended to use 3:1 and 4:1 for riverside and landside slope respectively. The top width for this structure is recommended to 6m for ease in assessability and to keep seepage lines inside the sections. The freeboard of 0.3m was used to compensate for unexpected flood and wave effect for safety purposes.

Table 4-6: Height and lengths of levee provided for the left bank

Left levee design				
River Station	Levee Length (m)	Left Levee Elevetion+Free Board (m)	Levee Bottom Elevation (m)	Levee Height (m)
6306.375-6081.194	137.66	2056.24	2055.66	0.58
6081.194-6015721	65.56	2056.17	2055.5	0.67
6015.721-5717.513	202.19	2056.18	2055.6	0.58
5717.513-5488.861	230.27	2056.16	2055.54	0.62
5488.861-5291.171	198.12	2056.18	2055.54	0.64
5291.171-4983.382	328.79	2056.18	2054.67	1.51
4983.382-4340.317	706.72	2056.17	2055.26	0.91
4340.317-3613.413	484.06	2056.1	2055.48	0.62
3613.413-3457.891	161.37	2056.08	2055.39	0.69
3457.891-3106.13	258.86	2056.07	2055.51	0.56
3106.13 -2754.359	258.86	2055.97	2055.34	0.63
2754.359-2511.856	230.58	2055.56	2055.17	0.39
2511.856-2188.504	383.78	2055.22	2053.57	1.65
2188.504-1921.757	286.3	2055.07	2054.52	0.55
1921.757-1446.303	510.17	2055.13	2054.42	0.71
1446.303-1220.236	249.03	2054.69	2054.3	0.39
1220.236-922.4112	342.84	2054.26	2053.64	0.62
922.4112-802.3994	126.08	2054.08	2053.73	0.35
802.3994-268.7353	385.08	2054.09	2053.6	0.49
268.7353-41.70034	240.94	2053.85	2053.39	0.46
41.70034-end	68.54	2050.58	2050.12	0.46

Table 4-7: left levee station location from the left of bank station

	Left Levee Distance From Main Channel	
Left Bank Stations (m)	Left Levee Stations (m)	Distance From Channel To Left levee Toe (m)
1435.3	1425.83	9.47
1487.54	1445.01	42.53
1547.91	1544.76	3.15
2789	2711	78
2530.12	2514.92	15.2
2135.93	2127.02	8.91
1548.86	1534.53	14.33
1003.63	901.11	102.52
1345.62	1306.91	38.71
1486.3	1476.98	9.32
1465.51	1450.34	15.17
1444.72	1429.24	15.48
979.66	958.9	20.76
762.49	755.14	7.35
899.26	895.14	4.12
638.82	567.77	71.05
577.4	447.57	129.83
648.24	641.94	6.3
470.55	468.03	2.52
745.29	733.16	12.13
655.21	638.36	16.85

Table 4-8: Height and lengths of levee provided for the right bank

River Station (m)	Levee Length (m)	Right Levee Elevation (m)	Levee Bottom Elevation (m)	Levee Height (m)	Levee Height plus Free Board (m)
6306.375-6081.194	294.59	2056.03	2055.71	0.32	0.52
6081.194-6015721	65.29	2055.97	2055.51	0.46	0.66
6015.721-5717.513	388.42	2055.98	2055.39	0.59	0.79
5717.513-5488.861	225.79	2055.96	2053.88	2.08	2.28
5488.861-5291.171	194.19	2055.98	2055.19	0.79	0.99
5291.171-4983.382	286.59	2055.98	2054.6	1.38	1.58
4983.382-4340.317	555.78	2055.98	2055.23	0.75	0.95
4340.317-3613.413	303.55	2055.97	2055.39	0.58	0.78
3613.413-3457.891	149.31	2055.95	2055.45	0.5	0.7
3457.891-3106.13	419.085	2055.85	2055.54	0.31	0.51
3106.13 -2754.359	419.085	2055.81	2055.36	0.45	0.65
2754.359-2511.856	246.55	2055.36	2054.68	0.68	0.88
2511.856-2188.504	269.32	2055.01	2053.74	1.27	1.47
2188.504-1921.757	207	2054.81	2054.53	0.28	0.48
1921.757-1446.303	425.32	2054.94	2054.36	0.58	0.78
1446.303-1220.236	194.96	2054.48	2054.33	0.15	0.35
1220.236-922.4112	233.96	2054.04	2053.59	0.45	0.65
922.4112-802.3994	111.14	2053.93	2053.78	0.15	0.35
802.3994-268.7353	617.46	2053.89	2053.6	0.29	0.49
268.7353-41.70034	210.84	2053.65	2053.34	0.31	0.51
41.70034-end	16.45	2050.46	2050.26	0.2	0.4

Table 4-9: Right levee distance from right bank stations

	Right Levee Distance From Main Channel	
Right Bank Station (m)	Right Levee Station (m)	Distance From Channel To Right Levee Toe (m)
1515.15	1568.63	53.48
1597.79	1631.71	33.92
1751.84	1751.84	0
3047.96	3060.53	12.57
2642.01	2642.8	0.79
2214.08	2250.64	36.56
1892.79	1892.79	0
1053.62	1062.23	8.61
1601.48	1602.3	0.82
1631.09	1631.09	0
1574.295	1602.17	27.875
1517.5	1517.5	0
1288.94	1289	0.06
920.15	938.93	18.78
1019.66	1023.02	3.36
700.25	746.8	46.55
769.04	769.82	0.78
872.08	874.68	2.6
632.71	634.27	1.56
807.53	807.53	0
790.55	841.6	51.05
790.55	841.6	51.05

4.10 Flood inundation analysis for mitigation alternatives

The result of the HEC-HMS model showing the magnitude of 50 and 100-year peak flows was used as a base to run for different alternatives after model calibration was done at the base condition.

These magnitudes were used to develop the model for different flow mitigation alternatives.

Flood mitigation alternatives were placed at an appropriate location along the Belo flood plain in HEC-RAS for different return periods. Delineation of flood extent and depth within the flood plain were conducted by the integration of the HEC-RAS and Gis extensions. Flood inundation extents including different outputs in all cross-sections were extracted after running the model for each alternative.

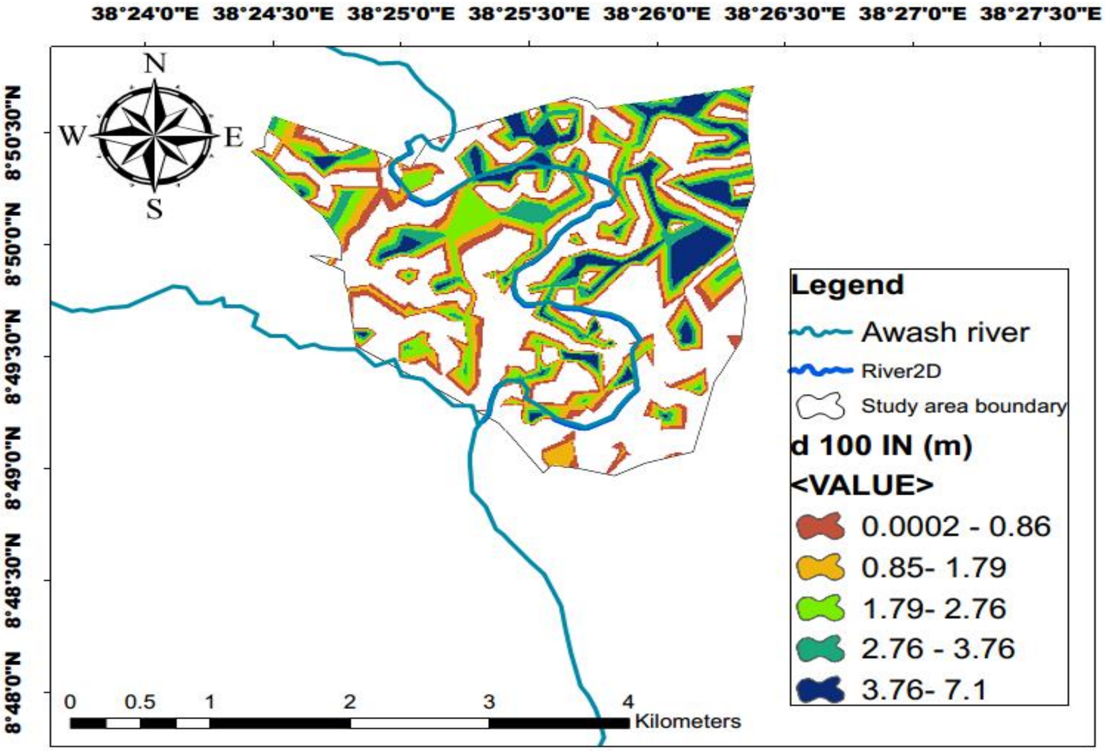


Figure 4-20: Channel modification in two-stage flood map for 100 year return period (Alternative-1)

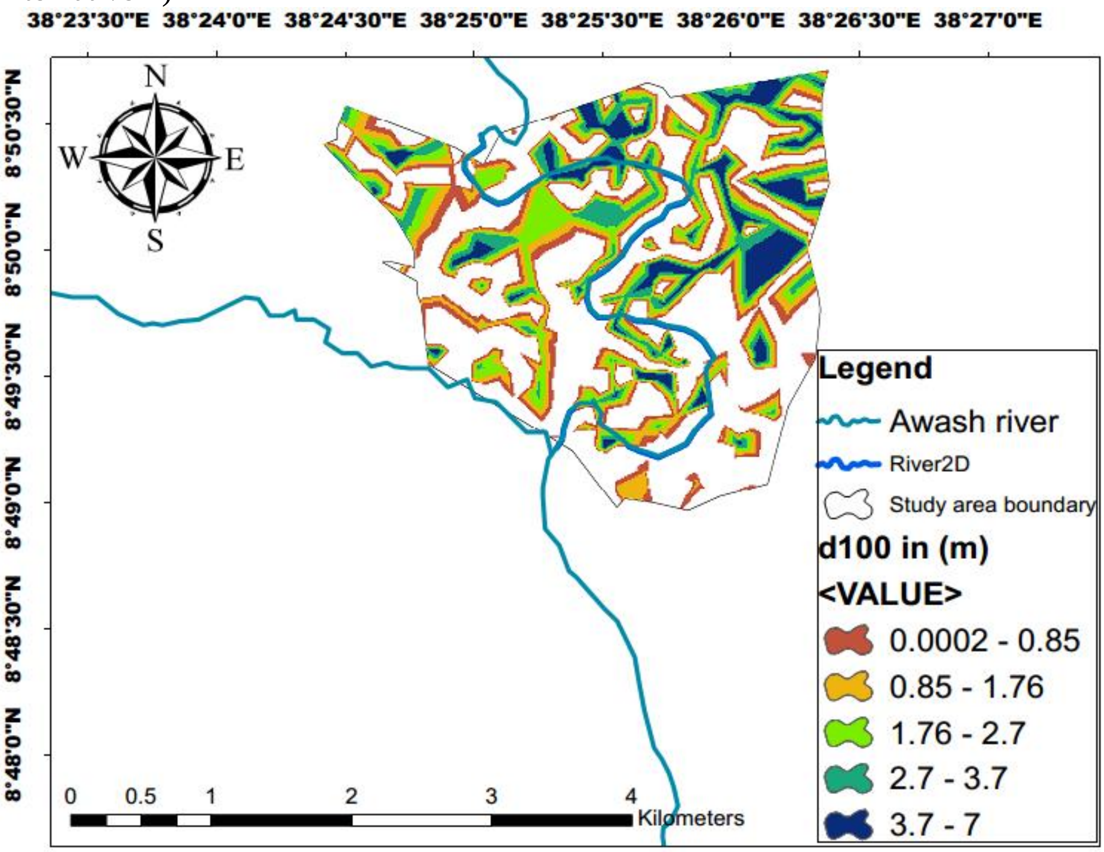


Figure 4-21: Channel modification in one stage flood map for 100 years (Alternative-2)

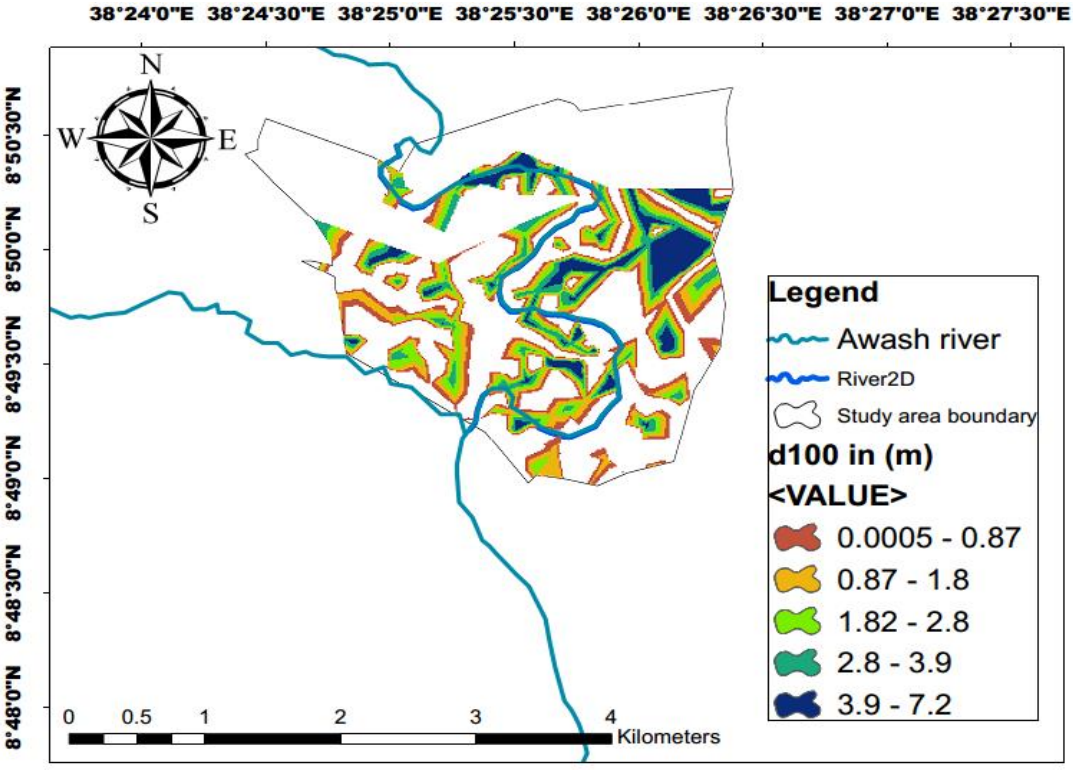


Figure 4-22: Channel modification with levee construction flood map of 100 years

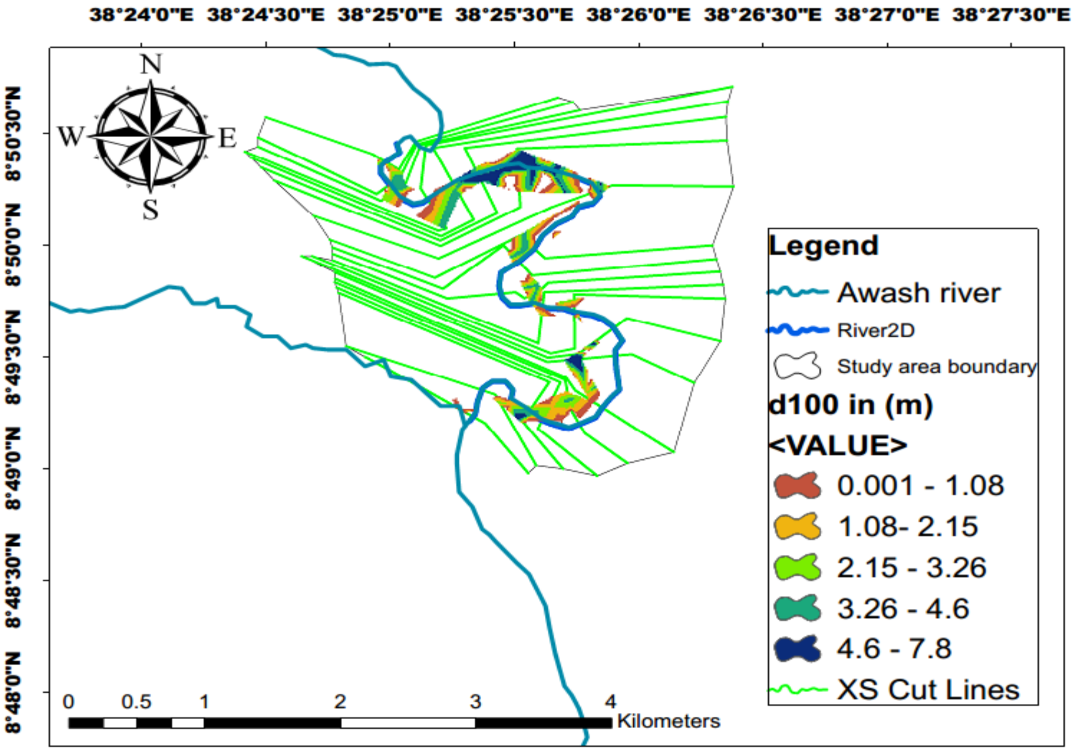


Figure 4-23: Levee flood map for 100 years (Alternative-4)

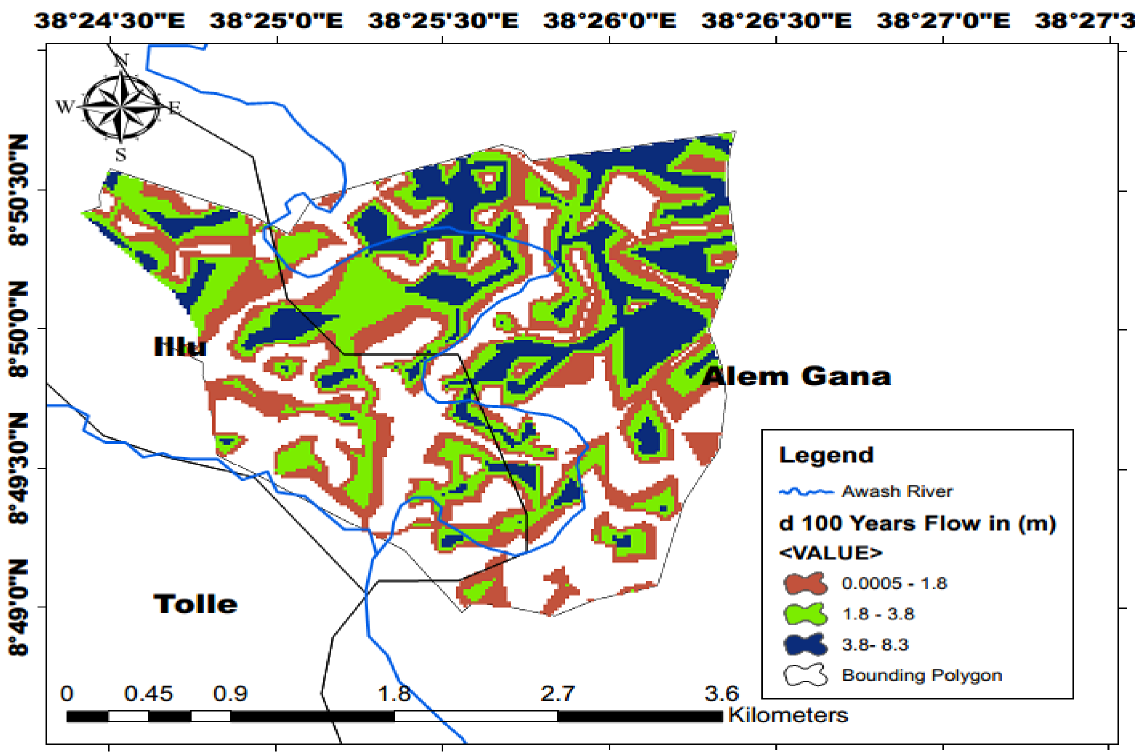


Figure 4-24: Flood map and depth for 100-year flood without mitigation measures

4.11 Comparison of flood mitigation alternatives

All proposed flood mitigation alternatives had different flood extents. The evaluated result of all measures could be summarized as follows in table 5.11. Flood inundation area, flow velocity, flood level, and stream power at two control points along the Awash River system were extracted based on floodplain inundation analysis for 50 and 100 years associated with four proposed alternatives using HEC-RAS simulation. The total area inundated by flood after two-stage channel modification was 379.3 ha and 386.2 ha for 50 and 100 year return period. Although, the total area inundated by flood after one stage channel modification was 377.5 ha and 383.66 ha for 50 and 100 years return period respectively. The one-stage channel modification saved 123.1 ha and 121.34 ha for 50 and 100 years respectively at flood plain. The area inundated by flood after channel modification and levee construction for some portion of the reach was 260.2 ha and 266 ha for 50 and 100 year return periods. This alternative reduced total flood inundated area before this protection was taken from 500.6 and 505 ha to the above values for 50 and 100 years respectively. This protection saved a large area as compared with the other alternatives except for the levee case. The area inundated by the flood due to the construction of the levee was 52.27 ha for 50 year return period and 57.28 ha for 100 year return period. The levee constructed along the flood plain reach reduces the area inundated by the flood before levee construction by 10% and 11.3% for 50 and 100 year return periods respectively, from the entire flooded area of 505 ha. The 1.1% and 1.27% of the entire study areas (4495 ha) are covered by the flood of 50 and 100 year return period respectively due to levee constructions. This alternative saved a large area as compared with the other three above. Even if the construction of levee is expensive as compared with the three alternatives, it is efficient in the reduction of the flood disaster area. The water depth varies from 0 to 7.9m for the area due to the constriction influence of the levee. Without taking any flood mitigation measure (baseline), total the inundation area was 500.6 ha and 505 ha under 50 and 100 years respectively.

Table 4-10: Area inundated for different alternative

Return period (year)	Flood Mitigation Measures				
	Alt.1	Alt.2	Alt.3	Alt.4	Baseline
50	379.3	377.5	260.2	52.27	500.6
100	386.2	383.66	266	57.28	505

Where Alt.1: Two-Stage channel widening and degrading (Trapezoidal)

Alt.2: One stage channel Widening and degrading (Rectangular)

Alt.3: Combination of the levee with the above two alternatives

Alt.4: Levee Construction

The effect on flood reduction by the different alternatives was presented in table 5.12.Alternative1 and alternative 2 had better performance in terms of flood level reduction than other the other alternative on the upstream and downstream control section. However, baseline and levee construction had the highest flood level as compared with others at the selected cross-section. There was no major improvement in flood level reduction using the construction of levee in comparison with the baseline. A comparison between all alternatives in terms of change in flow velocity under 50 and a 100-year flood was presented below at two selected control points. There was a variation of velocity among different alternatives depending on local flow geometry and hydraulic condition of river flow. As shown in table 5.12 alternative 1 and 2 causes a significant decrease in velocity on the downstream cross-section in comparison with the baseline. Alternative 1 performs better than other options in terms of flow velocity. However, alternatives 3 and 4 had the highest downstream velocity which could accelerate downstream riverbank erosion. Table 5.12 also shows a simple comparison of stream power change for the upstream control section and downstream cross-section for each alternative under 50 and 100-year floods. Generally, alternatives 1 and 2 shows better results for stream power on upstream and downstream control points respectively

Table 4-11: Comparisons of hydraulics parameter for proposed flood mitigation

Hydraulic parameters	Return period (year)	Control cross-sections	Flood Mitigation Measures				
			Alt.1	Alt.2	Alt.3	Alt.4	Base Line
Flood Level (m)	50	1 2	2054.498 2048.913	2054.453 2048.9	2055.39 2049.489	2055.563 2049.552	2055.743 2049.996
	100	1 2	2054.569 2048.963	2054.521 2048.951	2055.553 2049.576	2055.949 2049.997	2055.778 2050.173
Flow Velocity (m/s)	50	1 2	0.645 0.703	0.733 0.767	1.68 0.582	1.633 2.287	0.181 0.988
	100	1 2	0.656 0.76	0.736 0.8	1.652 0.712	0.391 0.998	0.202 0.125
Stream Power (N.m/s)	50	1 2	4.433 5.732	6.756 7.124	89.648 3.575	80.332 254.942	0.153 17.62
	100	1 2	4.415 6.967	6.441 7.924	83.392 5.912	1.094 17.614	0.219 15.228

From table 5.12 above we understood that there was a conflict between the criteria which made it must the application of multi-criteria decision analyses to take accurate decision analyses. Therefore, the application of TOPSIS is a multi-criterion decision method that was discussed in the next section for 100-year flood profiles.

4.11.1 Scenario-based flood mitigation analysis

TOPSIS, a multi-criterion decision analyses method was used for the selection of the optimized mitigation alternatives. The results of decision analyses based on different scenarios criteria weighting were shown in the table below. Only those criteria for a 100-year flood in two control points selected were considered for decision analysis. A scenario-based analysis was conducted to reflect the influence of each decision criterion in making decision analyses. In this research, the effects of both flood risk and environmental impacts were put into consideration. The first scenarios were based on the flood risk that includes both flood level and flood inundation area. In this scenario higher weight with twice as others were used.

Table 4-12: Priority rating and rank of proposed flood mitigation alternatives

Flood Mitigation Alternatives	Decision-making scenarios for flood risk mitigation		
	Risk-oriented	Environmental-oriented	Balance
Alt.1[Two stage channel modification]	0.331 [2]	0.000 [4]	0.000 [4]
Alt.2[One stage channel modification]	0.324 [3]	0.621 [2]	0.614 [2]
Alt.3[Levee with channel modification]	0.304 [4]	0.501 [3]	0.549 [3]
Alt.4[Levee Construction]	0.999 [1]*	0.776 [1]*	0.733 [1]*

*in the bracket shows the rank of each flood mitigation alternatives

For the first scenario decision criteria in which the highest percentage was given for flood risk, which includes flood inundation area and flood level the construction of levee along the selected Awash Belo flood plain was appropriate mitigations as compared with others. The levee construction was the highest-rated alternatives (0.999) as analyzed under TOPSIS in annex sections. Although, in these scenarios, alternative 3 was the least preferred alternatives for flood protection whereas alternatives 1 and 2 follow levee constructions respectively.

In the second scenario, a higher weight was assigned to environmental impact criteria, which included flow velocity and stream power at the upstream river cross-section and downstream control point. In this case, levee construction [0.776] and one stage channel modifications [0.621] were the two more preferred options respectively.

The third scenario was considered as the balance between all decision criteria and sub-criteria. In this scenario, the levee construction was the highest rated (0.733) alternatives as compared with others. In all scenarios, the two-stage channel modification was the least preferred mitigation measure.

Scenario analysis confirms the superiority of levee construction to other proposed alternatives as well as the strong weakness of two-stage channel modification. Although increasing the size or depth of the channel or decreasing its roughness can lead to a reduction in flood level because of additional channel capacity, channel modification could also have a negative effect such as increasing inflow velocity and stream power.

As Wyzga, (1993) stated, an increase in flow velocity and stream power owing to river modification results in a progressive out washing finer grains from bed material, which causes negative impact near the area adjoining the river and downstream. From this synthetic assessment, we can conclude that the levee construction plan is superior to the other three flood mitigation alternatives. As a result when taking the risk and environmental impact consideration into account levee construction was beneficial in protecting the cropland area and contributes less sedimentation and increasing downstream power and flow velocity. Different criteria weighting scenarios enable us to take into consideration all of the evaluation criteria such as the least environmental impact as well as effectiveness in reducing the flood hazard in agricultural areas.

CHAPTER 5 CONCLUSION AND RECOMMENDATION

5.1 Conclusion

The Awash River is one of the rivers which cause flooding at Illu and Sebeta/Alamgana flood plain. The river conveys high runoff from upper catchments and local rainfall on the flood plain to resulting in flood problems. This study work used the hydraulic and hydrologic modeling methods in solving this problem by applying HEC-HMS and HEC-RAS respectively with their GIS extension and analyses of satellite image data. The HEC-HMS program was selected for this study to attain flood generation and flood forecasting due to its versatility, capability for flow generation, automatic parameter optimization, and its extension with HEC Geo HMS. The HEC-HMS model was calibrated and validated on both daily and monthly time series concerning Awash Belo gauged discharges using a time series data of 16 years from 1990-2005. The calibration and validation of HEC-HMS show the model performs reasonably to develop flood prediction. The model performance criteria result in shows that R^2=0.88 and NSE=0.8 for calibration, and R^2=0.75 and NSE=0.7 for validation. Therefore, HEC-HMS can predict flood for flood mapping and to take corrective measures.

The hydraulic model was also calibrated and validated using flood events recorded on September 3, 2001, and August 25, 2001, respectively. Above 95% overlapping satellite image of the specified event with model-simulated flood inundation was done. Therefore, the result can be used to take mitigation measures for the area.

Most of the cross-sections of the river are vulnerable to flooding under its low topography for the flood of 2, 5,10,25,50, and 100 years return period. Besides this, the water surface profile of the study reach was less varied at the upstream section of flood plains and more varied at downstream sections of the study flood plains. The total flooded area for 100 years return period was 505 ha without any mitigation measures was taken.

The different four flood mitigation alternatives were selected, compared, and modeled for the study area in reducing the flood risk along the selected flood reach. The first step was channel modification and levee design for all alternatives by the selected 100 years flood peaks. The second step of this was to map flooded areas due to the taken mitigation measure for all alternatives and shows different flood levels, velocity, and stream power.

The third step was to select the most appropriate alternatives based on the multi decision criteria approach (TOPSIS) from the four alternatives. The impacts of taken alternatives were evaluated in terms of flood risk reduction and environmental impacts under 50 and 100-year flood inundation areas and depths. The Multicriteria decision-making result of this study shows that the best alternatives measure to answer the flood problem along the selected Belo plain agricultural area was to construct levee as structural flood mitigation measures with a high rated value of 0.73. The other alternatives such as one and two-stage channel modifications with different dimensions failed to be optimal for the study area. The result of this study shows that channel modification activities change the upstream and downstream bring about the possibility of channel degradation downstream because of an increase in flow velocity. Such negative impacts of channel modification support the superiority of levee construction. The evaluation of flood mitigation measures performed in this thesis focuses on flood risk reduction and environmental objective rather than economic objective.

5.2 Recommendation

This study was conceptualized with steady, one-dimensional, and gradually varied flow simulation. Therefore, several other simulations like unsteady, two-dimensional, and gradually varied flow could also be modifying the hydraulic modeling result. It is also possible to use different methods in the HEC-HMS model which might increase the model accuracy further. For future studies in the area, these recommendations could be considered. Integrated watershed management activities (afforestation, soil, and water conservation activities) should be done to minimize the flood capacity in the catchment. Detention and retention of flood peaks should be exercised well in the catchment to reduce flood inundated areas. The water profile of the study area could be updated more frequently due to the continuous changing of hydrologic, hydraulic conditions, and land use in the basin. An emergency action plan should be developed for the study area residents and properties and a Seasonal based emergency alarm should be planned for the study area. Economic analysis for taken mitigation measures should be done for future study.

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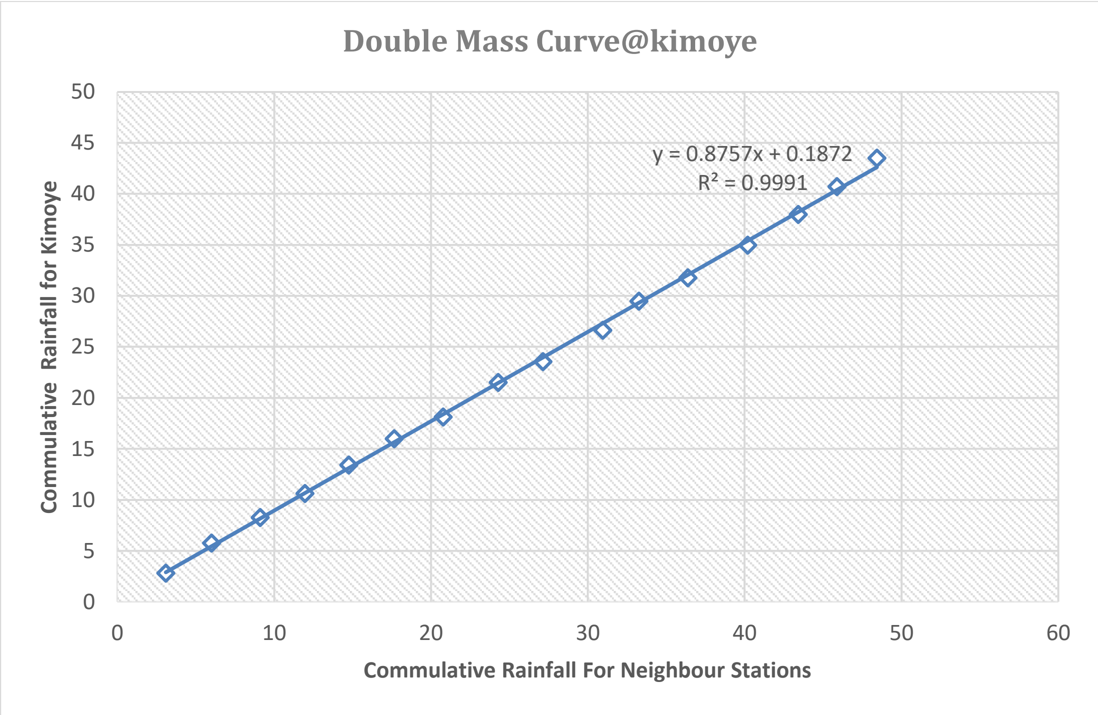
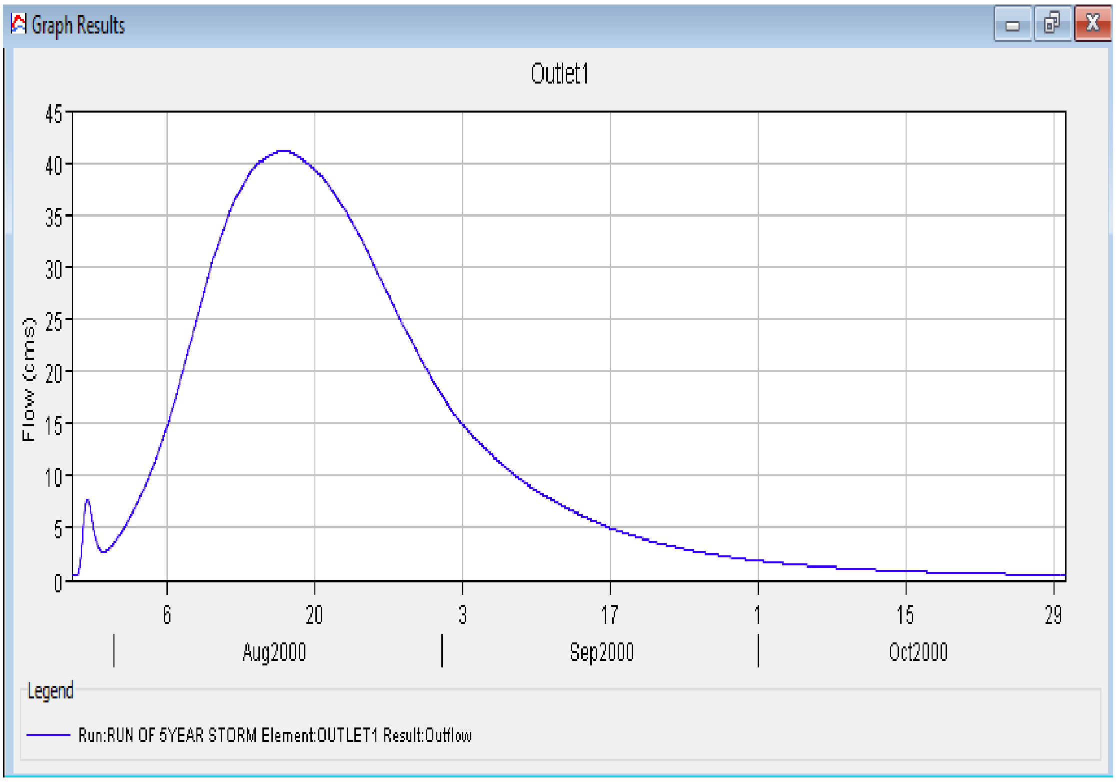
ANNEXES

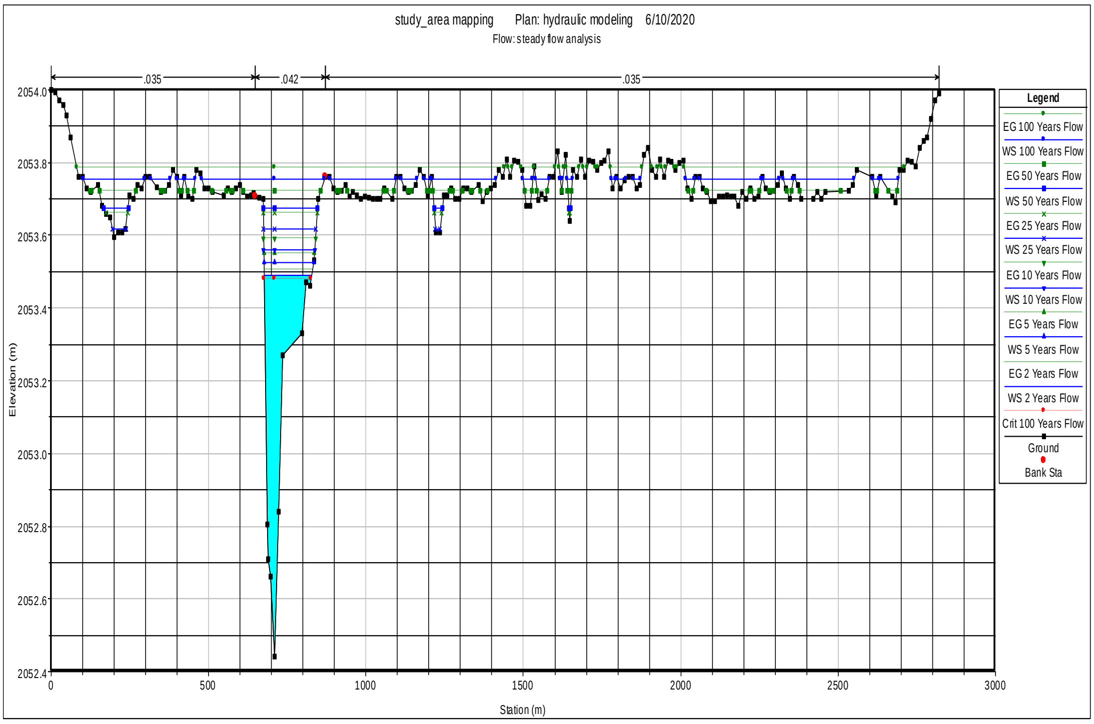
Appendices A: Double Mass Curve of Kimoye



Appendices (B) 5 Year Storm Runoff Hydrography



Appendices (C) HEC-RAS output, River cross-section at 922.4112 River station



Appendices (D). Decision Analyses Technique of order preference by similarity to ideal solution. Calculate the relative closeness to the ideal solution Ci*=(S')/(S'+Si*)

	Ci*=(S')/(S'+Si*)	Rank	Status	Ci*=(S')/(S'+Si*)	Rank	Status	Ci*=(S')/(S'+Si*)	Rank	Status
Alternative-1	0.331	2		0.000	4		0	4	
Alternative-2	0.324	3		0.621	2		0.613559257	2	
Alternative-3	0.304	4		0.501	3		0.548601011	3	
Alternative-4	0.999	1	Best	0.776	1	Best	0.733428784	1	Best