ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

Flood Damage Analysis Using HEC-FDA Software (In case of Upper Awash Sebata-Awas Woreda)

A Thesis in Hydraulic Engineering

By Webeshet Hailu

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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 Damage Analysis Using HEC-FDA Software (In case of Upper Awash Sebata-Awas Woreda**' presented by **Webeshet Hailu**, a candidate for the degree of **Master of Science** and hereby certify that it is worthy of acceptance.

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ABSTRACT

Flood occurs with high frequency in Sebata-Awas Woreda and cause tremendous damage to cultivated land inundation and silting, displacing people, life and other properties every year. The study describes technical approach in flood Damage analysis, with objective of detail study environmentally feasible flood protection plan and flood damage management.

Flood frequency for different return period done, by selecting appropriate probability distribution based on goodness fit test. Statistical analysis i.e. outlier, independent and stationary and filling missing data performed for flow observed at Awash Belo gauge station. Contour of 1m interval from (Digital Elevation Model) DEM 30m prepared to determine depth damage function. Flood inundation mapping in ArcMap and field survey done to estimate property loss. Triangulated Irregular Network (TIN) prepared from contour in ArcMap with GeoRAS extension to set layout for both canal and river. However, the cross section entered from field data. Manning number and Boundary condition estimated on field. Water surface elevation computed by HEC-RAS model.

Observed data are independent and have no outlier. Flood frequency estimated by Gumbel (Extreme I) method 42.36 m3 /s, 49.01 m3 /s, 53.41 m3 /s, 58.97 m3 /s, 63.10 m3 /s, 67.20 m3 /s, 72.59 m3 /s and 76.66m3 /s discharges for 2, 5, 10, 25, 50, 100, 250 and 500 year respectively. These floods used for water surface profile computation. Expected and Equivalent, annual damage amount reduced for different exceedance probability estimated. Total economic loss 69423.77 $ damage expected annually without project. By providing canal we can reduce loss to 5572.80 $ s per annum and reduce ton of sediment. Canal loss which very low per households. Inland flood protection and dyke integrated to Canal nullifies (0 $) expected damage per annum. Equivalent annual damage report shows without project, Canal and Integration of Canal with dyke and Inland flood protection are 69423.77 $, 5572.80 $ and 0 $ respectively through project life 50 years. Expected annual damage and Equivalent annual damage HEC-FDA model statics shows that flood protection plan is required.

Keywords: Sebeta-Awas, Flood Protection Structure, Flood frequency, Expected annual damage, Equivalent annual damage

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ABBREVIATION

ANOVA	Analysis of Variance
CBE	Canal Bed Elevation
DEM	Digital Elevation Model
DPPA	Disaster Prevention and Preparedness Agency
DTM	Digital Terrain Model
EAD	Expected Annual Damage
EAP	Annual Exceedance Probability
ESRI	Environmental System Institution
GPS	Global Positioning System
GVF	Gradually Varied Flow
GIS	Geographical Information System
HEC-RAS	Hydrologic Engineering Center River Analysis System
HEC-FDA	Hydrologic Engineering Center Flood Damage Analysis
IPPC	Intergovernmental Panel on climate Change
IFRCS	International Federation Red Cross society
JICA	Japan International Cooperation Agency
NAVSTAR	Navigation Sattelite Timing and Ranging
RAS	River Analysis System
TIN	Triangulated Irregular Network
UNDP	United Nation Development Program
UNDRO	Office of United Naions Disaster Relief Coodinater
USAC	US Army Corps Engineering
2D	Two Dimensional

CHAPTER 1 INTRODUCTION

1.1 Back Ground

River is very useful natural resource play great roll for human existence. Almost all-ancient civilization born on river. As human expanded to every continent's follow river corridors. River has not benefit only but also damage to life and property.

Flood damage is loss of life and property due to occurrence potentially damaging flood Event of ascertain magnitude with given time[1]. Throughout human history river flooding is induce damage on properties and live[2]. From globally recorded disaster, flood disaster holds 43.4% that is highest of all disaster. This disaster related to climate change. The major victims of flood damage are poor people of developing countries[3]. Africa one of developing continents which affected by high frequency of flood and drought.

Our country Ethiopia globally known for its rivers. The annual flow from its rivers amounts 122 billion m^3 all of this generated within its boarder and goes across to other countries[4]. Ethiopia topography is mountainous and lowland deserts, which shape its rainfall and river. As river flow, from mountainous to lowland causes bed load deposition, changes its courses (meandering) and flooding. The country experiences two type of flood, flash flood and river floods. "A flash flood are defined as those flood events where the raise in water is either during or within the a few hour of the rainfall that produces the rise and river flood in contrast flash flood, typically unfold over days or even month "[1]. The main cause of river over flow (river flood) through the country is due to heavy rainfall during summer. The major flood susceptible area through the country are flood plain along upper, middle and downstream Awash River, eastern and southeastern lowland along Wabeshebele River and Genale-Dawa Rivers, south western lowland along rivers Baro, Gilo and Akobo River, downstream of Omo river, flood plain along Lake Tana, Gumera, Rib and Megech rivers[5].

Awash River which set up journey from central high lands to east direction with area of 113000km^2 induce high flood damage along its course from upstream to downstream[4]. According to Woreda Emergency Preparedness Office, in Sebata-Awas woreda alone about 9,678,679 Birr (Ethiopia birr) was damaged by 2017 summer flood. The river has flooded a wide area of land and has displaced about 2500 people in 2006. At time about 750 households host in Tafki camp[5]. The problem of flood damage is particularly is prominent in Awash Belo kebele where riverbed gradient is very low. Natural resource degradation imposed by human aggravate flooding and meandering deposition in area of flood plain low river gradient. These social and economic losses of local people obligate states to implement fast, and socially cost effective structural as well as nonstructural counter measure[6]. Nonstructural measures are land use management, early warning system based on hydrological and metrological forecasts, a system flood insurance. Structural measure includes dykes, cannel ditches, modifying river path etc. In this study structural measure provided based on collected and forecasted flow data.

1.2 Statement of Problem

The major problem of study area is flood damage. Southwest shoa lies near the source of Awash River. As River emerges from central highlands to Shoan flood plain, it has flooded a wide area. In southwest Shoa of Oromia region alone, it displaced many people, damage to cultivated land and loss life every year. Due to high, increased flood frequency every year, many hectares cultivated land which harvested by local people currently out of services. This result in social and economic crises. Siltation result in change of river course and damaged to cropland. The detail of this study provides input the most comfortable and applicable flood protection structure to overcome this problem.

1.3 Objective of Study

General Objective

The main objective of the study is to conduct flood damage analysis and select the best flood damage management plan at **Sebata- Awas flood plain**.

Specific Objectives

- To compute flood frequency and its inundations area using HEC-RAS model and ArcMap.
- To estimate expected annual damage, Equivalent annual damage and Expected annual exceedance probability (AEP) measures the chance of a flood occurring in any given year and corresponding damage using HEC-FDA model.
- Analysis of Economic loss and gain for different plan (damage reduced and distributed for different plan).

1.4 Limitation of Study

The limitation during the course of this research was the lack of continuous flow data. Flow data had been collected contain missed data. Missed data limitation gaps were filled by appropriate missed data estimation methodology discussed in chapter 3.

HEC-RAS model assume fixed canal geometry, which sometimes is far from real world. Assumption steady, one-dimensional model also which not always valid when we compare real world. These mean, accurate results depend up on a quality of Channel and Canal cross section data. To overcome model related limitation cross section levelling was done at each 100m station along the river.

CHAPTER 2 LITERATURE REVIEW

2.1 Flooding

Flooding has been defined as" a water over flowing into the land that is usually dry"[1]. Flood produce damage through the power of moving water and through the deposition of dirt and debris when the floodwater finally recedes. Flooding include several river activities that causes a damage i.e., inundation of flood plains and adjacent terraces, bank cutting, river channel shifting, and debris torrents during normally high discharge[7].

2.1.1 Flood Control and Management

Hydrological information plays a major role in flood risk management. Flood risk management can be subdivided into two different parts [6]. In narrow sense, flood risk management describes the process of managing an existing flood risk situation. In a wider sense, it includes the planning of activities, which will reduce flood risk. This planning directed to an optimization or improvement of existing system. A system has not necessarily to be constructional system. The installation of early warning system based on metrological and hydrological forecasts a system flood insurance, legal system or legal system specifying building codes could also reduce flood risk at different levels[8].

Flood control can divided into ("6") categories[9]

- To increase the river discharge capacity
- To reduce or control the peak discharge of flood
- To prevent inland flood
- To prevent bank collapse and harmful degradation of river bed
- To prevent obstruction against river flow, maintain or conserve the good condition of the river in order to keep the flow uninterrupted.

2.1.1.1 *To Increase discharges Capacity*

- By levee or dike
- By widening of water way or River
- By dredging or Excavation

- Combination of above

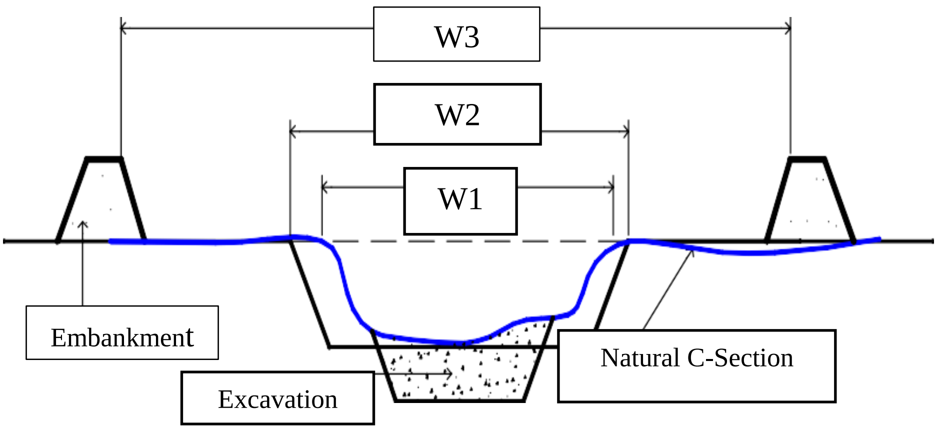

Figure 2-1 Flood protection plan Source,[9]

2.1.1.2 *To Reduce or Control the Peak discharge of Flood*

We reduce peak flood and it is time to peak by constructing dam. Dam reduce much flood.

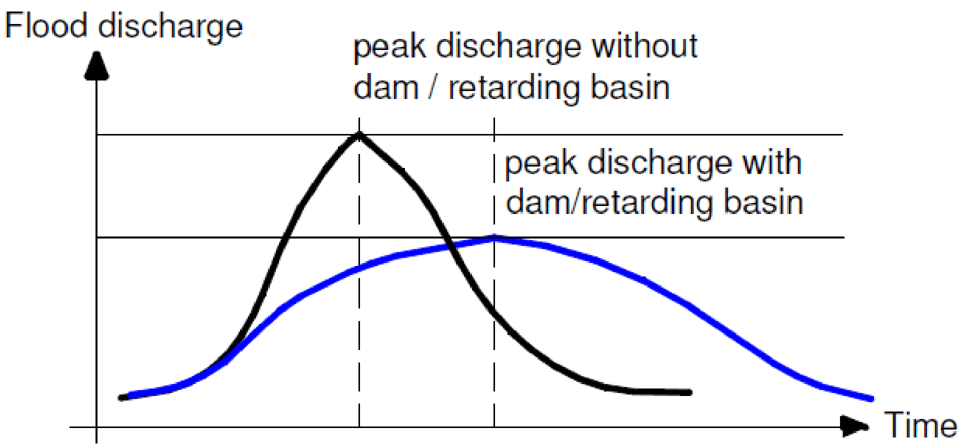

Figure 2-2 Dam as flood Protection plan Source [9]

2.1.1.3 *To prevent Inland flood*

Inland flood is flood caused rainfall due to which not drained by gravity due to the high-water stage of the river. Inland flooding prevented by lateral improvement i.e. (storm drain, drainage main, open channel, ditches etc.), tributary improvement and pumping station.

2.1.1.4 *To Prevent Bank Collapse and Harmful Degradation of River Bed*

There is various method used to prevent bank collapse and harmful degradation of riverbed such as revetment, spur dike, change of waterway or cut of channel and by ground sill.

2.1.1.5 *To Prevent Obstruction Against River Flow, Maintain or Conserve the Good Condition of The River in Order to Keep the Flow Uninterrupted.*

Improving land use and sediment control and regular maintenance channel by excavation and dredging can prevent this problem.

2.2 Flood Frequency Analysis

The primary objectives of frequency analysis is to relate the magnitude of extreme event to their frequency of occurrence using probability distribution[10]. Historical record of observed data in river system analyzed by frequency analysis. For best frequency analysis, data must be independent and identically distributed[11]. In practice, the true probability distribution of data at site or region is unknown. The assumption that data in given system arise from given from a single parent distribution may be questionable when data from large watersheds analyzed. However, for the analysis to be practical use, simpler distribution often used to characterize the relation between flood magnitudes and their frequencies. Among the most widely used probability distribution functions for hydrological analysis, related with flood frequency analysis, are [12]:

- Two and three parameters Lognormal (LN2 and LN3)
- General Pareto
- Log-Pearson Type III (LP-III)
- Extreme Value Type I (EVI)
- Extreme Value Type II (EVII)
- General Extreme Value (GEV)

There are various methods is used to determine parent distribution. The well-known method are Goodness of fit test, conventional moment's method and probability weighted method[11].

2.2.1 Conventional Moment and Moment Ratio diagram Methods

Moment about the origin or about the mean are used to characterize probability distributions. Moment are expected values of power a random variable. For a distribution with a probability density function f(x), the r[th] moment about the origin is given by[13].

$$\mu'_r = \int x^r f(x)dx \qquad \mu'_1 = \mu = mean \tag{2.2.1}$$

The central moment μ_r are computed by equation 2.2.1

$$\mu_r = \int (x-\mu'_1)^r f(x)dx \qquad \mu_1 = 0 \tag{2.2.2}$$

When' r '1.2…

It can be proved that the relationship between μ_r and μ_r ' is given by 2.2.3

$$\mu_r = \sum_{j=0}^{r} \binom{r}{j} \mu'_{r-j} (-\mu_1')^j \tag{2.2.3}$$

$$\mu_r' = \sum_{j=0}^{r} \binom{r}{j} \mu_{r-j} (-\mu_1')^j \tag{2.2.4}$$

Sample moment m $_1$ and m $_r$ calculated by the following equation.

$$m'_r = \frac{1}{n}\sum x^r, m'_1 = \bar{x} = \text{Sample mean} \tag{2.2.5}$$

$$m_r = \frac{1}{n}\sum (x_1 - \bar{x})^r, m_1 = 0 \tag{2.2.6}$$

For given distribution conventional moments expressed as function of the parameter of distributions. Conventionally the higher order moments expressed as function of lower order moments. For two parameter distribution μ_3 expressed as function μ_2 [11]

$$C_S = \mu_3 / \mu_2^{\frac{3}{2}} \tag{2.2.7}$$

$$C_V = \mu_2 / \mu'^{\frac{1}{2}}_1 \qquad 2.2.8$$

$$C_K = \mu_4 / \mu_2^2 \qquad 2.2.9$$

Where: C_S , C_V and C_K skewness, variant Coefficient and Kurtosis Respectively.

2.2.2 Probability Weighted Moment and L-Moment

Probability weighted moment (PWMs) were defined by for random variate x with cumulative distribution function F(X) and quantile function x(s) [14]:

$$M_{p,r,s} = E[x^p F^r (1-F)^S = \int_0^1 [x(F)]^p F^r (1-F)^s dF \qquad 2.2.10$$

Certain linear combination of probability-weighted moments directly interpreted as measure of location, scale, and shape of probability distribution. L-moments are defined by in terms of PWMs α and β [15]

$$\lambda_{r+1} = (-1)^r \sum_{k=0}^{r} P^*_{r,k} \alpha_k = \sum_{k=0}^{r} P_{r,k}{}^* \beta_k \qquad 2.2.11$$

Where: $P_{r,k}{}^* = (-1)^{r-k} \binom{r}{k}\binom{r+k}{k}$ 2.2.12

K= 0, 1, 2… r and

α and β probability weight moment

L-moment ratio which are analogous to conventional moment ratio are define by [11]

$$\tau = \lambda_2 / \lambda_1 \qquad 2.2.13$$

$$\tau_r = \lambda_r / \lambda_2 , r \geq 3 \qquad 2.2.14$$

Where λ_1 measure of location τ is measure of scale and dispersion, τ_3 is measure of skewness and τ_4 is measure of kurtosis. It can be shown that that for “greater than or equal to 3, the absolute value of τ_r is less than one. Furthermore, if $x \geq 0$ almost surely, then τ , the LC_v of x satisfies $0 < \tau < 1$. This boundedness of L-moment ratio is an advantage because it easier to interpret a measure such as τ_3 which is constrained to lie within interval (-1, 1) than conventional skewness coefficient which can take arbitrarily large values. Analogous to conventional moment ratio diagrams based on relationships L-Moment ratio. The relation between measure of skewness and kurtosis shows appropriate parent distribution[14].

2.2.3 The Goodness of Fit Test

The choice of distribution used in flood frequency analysis has been a topic of interest for long time[11]. There several methods used for selection of parent distribution, which best fits. The best-fit parent distribution also selected chi square test, Kolmogorov Smirnoff test and Anderson darling[16],

2.2.4 Test on Hydrologic Data

Two basic assumption in flood frequency analysis are independence and stationary of the data series[11]. In addition, the assumption that the data come from the same distribution (homogeneity) made. There several data test done in order get appropriate data represent real world [13]. These are Independent and Stationary test, Outlier test, Homogeneity test, consistency test and etc.

2.2.5 Estimation of Missing River flow Data

There is various method are used for filling river flow data. These are regression and Recession method. Each selected method of filling missing data universally best. Every method has own advantage and disadvantage[17]. When we selecting each method there is criteria. For regression methods, the selection of independent and dependent variable based on correlation. Catherine, J Salim and S. Kazumba [18] had done Estimation of Missing River Flow Data for Hydrologic Analysis in the Case of Great Ruaha River Catchment Lusajo by regression.

2.3 Assessment and Analysis of Flood

Flood is common in southwest Shoa. The problem includes riverbank erosion, silting, loss to life, loss to agricultural land and people displaced from their home every year. In new millennia, it is obvious that risk from hydrological extreme increase. There are two main types flood damage: tangible and intangible[6]. Any damage that measured in monitory values is tangible damage, while damage that not directly measured in monetary terms is considered is intangible. Tangible damage further divided into direct and indirect damage. Direct damage happens to items such as building or inventory items by contact with or submersion in water. In contrast, indirect damage caused by interruption of physical or economic networks such as traffic flow disruption and individual income loss as well as economic loss due to business downtime. Damage due to flooding is depend on many flood parameter such as flood depth, duration, the year of flooding and the use of flood warning system[19].

There are two basic methods for conducting flood damage estimations[6]. One approach is to perform a thorough questionnaire survey of the affected population and properties to estimate the incurred loss. Post-flood damage surveys the most reliable way to predict flood damage[7]. The other way is by using stage damage functions. Computation of stage damage function easily computed by using HEC-FDA software [20]. This point of view is fueled by evidence both from recent changes in the frequency and severity of floods as well as droughts and outputs from climate models, which predict increases in hydrological variability[21]. Dawit Sisay [22] Has prepared flood inundation map using HEC-RAS for flood peak of different return period for upper awash flood plain. However, this study limited flood frequency prediction not set structural measure flood protection. This study includes structural measures evaluation.

2.4 Tool for Flood Damage Analysis

There are various computer programs available for flood damage estimate and flood risk prediction. These are software produced by USA army corps such as HEC-FDA, HEC-GEORAS, HEC-GEOHMS, ARCGIS and HAZUS[23].

2.4.1 Hydrologic Engineering Center Flood Damage Analysis (HEC-FDA)

HEC-FDA is powerful tool to perform an integrated hydrologic engineering and economic analysis during the formulation and evaluation of flood risk analysis. It allows you to perform plan formulation and evaluation for flood damage studies. It includes risk analysis methods that follow federal and corps of engineer policy regulations (ER 1105-2-100 and ER 1105-2-201) [19].

Both economic flood damage and hydrologic engineering analysis are performed using a consistent configuration: streams, damage reaches plans and analysis year. The computer program performs three function. These are stage discharge function, exceedance probability function and stage damage function. These three function required to estimate expected annual damage and equivalent annual damage numerically by Monte Carlo analysis [19]. Study on Analysis HEC-FDA sensitivity and uncertainty analysis shows that depending on the time and money spent in the collection and preparation of the input data sets, relatively lesser or greater accuracy can be achieved in EAD computation[24].

2.4.1.1 *HEC-FDA Component*

FDA is an integrated system of software that designed for interactive use. The program consists of a graphical user interface (GUI), hydrologic engineering and economics components, database and management capabilities, graphics and reporting facilities. The interface designed to make the program easy and efficient to use. The interface provides the following functions[19]:

- File management
- Data entry and editing
- Data selection and assignment
- Hydrologic and economic analyses
- Tabular and graphical displays of input and output data
- Reporting facilities
- On-line help

2.4.2 Hydrologic Engineering Center River Analysis System (HEC-RAS)

HEC-RAS is computer program developed by hydrologic engineering center (HEC) of USA army Corps of engineers. Hydrologic engineering centers River system Analysis (HEC-RAS) Supports steady and unsteady flow water surface profile computation, sediment transport computation and water quality analysis. The program in addition has tool for performing inundation mapping directly inside the software. The HEC-RAS software successor the old HEC-2 river hydraulic package that was one dimensional, steady flow water surface profiles program[25]. The first version of HEC-RAS released 1995. Since then several update have been developed 1.1, 1.2… and now version 5.07 available[26].

2.4.2.1 *Components of HEC-RAS*

HEC-RAS consists of several individual components that linked to one another through the graphical user interface RAS.EXE. HEC-RAS employs the concept of a project to describe a stream system. The graphical user interface used to create, edit and manage project files. Projects created and edited within the graphical interface. Steady- state hydraulic simulations conducted using the steady-state flow model SNET.EXE. Unsteady flow computations carried out by the UNET.EXE model. The user interface will prepare the data for use with the appropriate steady or unsteady flow model, which then performs numerical computations. The hydraulic results are stored in an output file, which then read by the user interface. Using tabular and graphical reporting features available within the user interface, the results of the hydraulic simulation examined[19].

2.4.2.2 *Steady Flow Surface Profile*

This type of modelling used for computation steady and gradually varied flow water surface profiles. The steady flow modelling is powerful tool for computing subcritical, critical, super critical and mixed flow regime of water surface profile. The model executes instructions based on solution of one-dimensional energy equation. Energy losses evaluated by friction, contraction, and expansion coefficient. The momentum equation utilized in situation where the water surface profile is rapidly varied[26].

2.4.2.3 *Gradually Varied Steady Flow*

A steady non-uniform flow in a prismatic channel with gradual changes in its water surface elevation is termed as *gradually varied flow* (GVF). In a GVF, the velocity varies along the channel and consequently the bed slope, water surface slope, and energy slope will all differ from each other. Regions of high curvature excluded in the analysis of this flow[27].

2.4.2.4 *Energy Equation*

Water surface profile are computed from one cross section to the next by solving the energy equation with an iterative procedure[25]. In addition, water kinetic Energy determined by this principle. According to this equation, the total energy at a downstream section differs from the total energy at the upstream section by an amount equal to the loss of energy between the section[27].

$$E_1 = E_2 + \text{Loss} \qquad 2.4.1$$

Where: E is Specific Energy

In gradually varied flow, water surface in general varies in longitudinal direction along river course.

$$E = Z + y + \alpha \frac{V^2}{2g} \qquad 2.4.2$$

Where: Z is datum

y is Slope

α is kinetic energy factor

V is Velocity

g is Gravitational acceleration

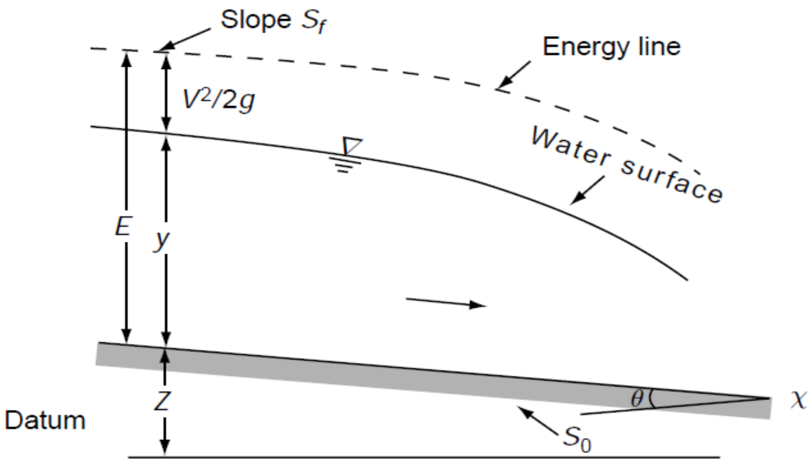

Figure 2-3 Energy Equation Source [27]

2.4.2.5 *Conveyance Computation*

The determination of total conveyance and the velocity coefficient for a cross section require that flow sub-divided into units for which the velocity is uniformly distributed. The approach used in HEC-RAS is to subdivide flow in over bank areas using input cross section n-value break points as basis for subdivision. Conveyance calculated within each subdivision forms the following Manning equation[26][27].

$$Q = KS^{1/2} \qquad 2.4.3$$

Where: $K = \frac{1}{n} AR^{2/3}$ 2.4.3

K is Conveyance for sub-division

n is Manning 's roughness coefficient for sub-division

A is flow area for sub-division

R is hydraulic radius for sub-division

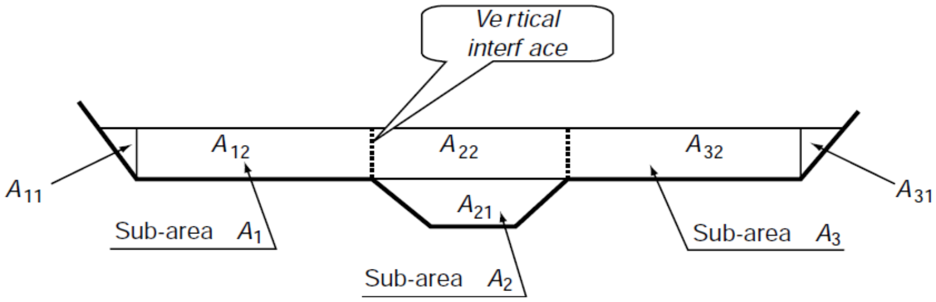

Figure 2-4 Hydraulic Area Source[25]

2.4.2.6 *Unsteady Flow Data*

This type of flow modeling primary developed for subcritical flow regime computation. Unsteady flow data files contain flow hydrograph at upstream boundaries; starting flow condition and downstream boundary conditions[26].

2.4.3 Hydrologic Engineering System GIS Based River Analysis (HEC-GEORAS)

ESRI develop ArcMap extension for purpose of Geo-spatial data analysis for HEC-RAS. The extension provide data from digital terrain model (DTM) required by HEC-RAS i.e. geometric data, Manning constant and export to HEC-RAS for analysis. Result from HEC-RAS imported arc map by extension for analysis and mapping. HEC-RAS Model result inundation depth and boundaries mapped by this extension. HEC-GEORAS 10.5 require ArcMap 10.5.[28]

2.4.4 Geographical Information System (GIS)

New millennia come with born of geographic information system engineering which make task easy in water resource engineering. GIS defined as computer system capable of assembling, storing, manipulating, and displaying geographically referenced information[29].

2.4.5 Global Positioning System (GPS)

The Global positioning System (GPS) is space-based radio navigation system, which managed for the government of United States by U.S. Air Force (USAF), the system operator. The radio navigation system of assessing object on Earth surface. GPS works globally anywhere 24 hour a day. GPS satellite revolves twice a day in very precise orbit and transmit signal information to Earth. A GPS receiver locked on signal of at least three satellite to compute a 2D position latitude and longitude [30]. Old GPS 5m error but currently developed mobile phone loaded software precise 3m radius.

CHAPTER 3 MATERIALS AND METHODS

3.1 Study Area

3.1.1 Location of Study Area

The Awash River bear near Ginch and flows in northeastern direction through northern Extension of Rift valley to eventually discharge Lake Abe near Djibouti boarder. The study area is in upper Awash River part, which found near its sources. Sebata–Awas woreda flood susceptible area found about 50Km southwest of Addis Ababa along Awash River flood plain on the Addis-Jima highway. The River gage station found at the boundary of rural town called Awash Belo (38.41^0, E and 8.85^0, N) with catchment Area: 2568.8 sq. km.

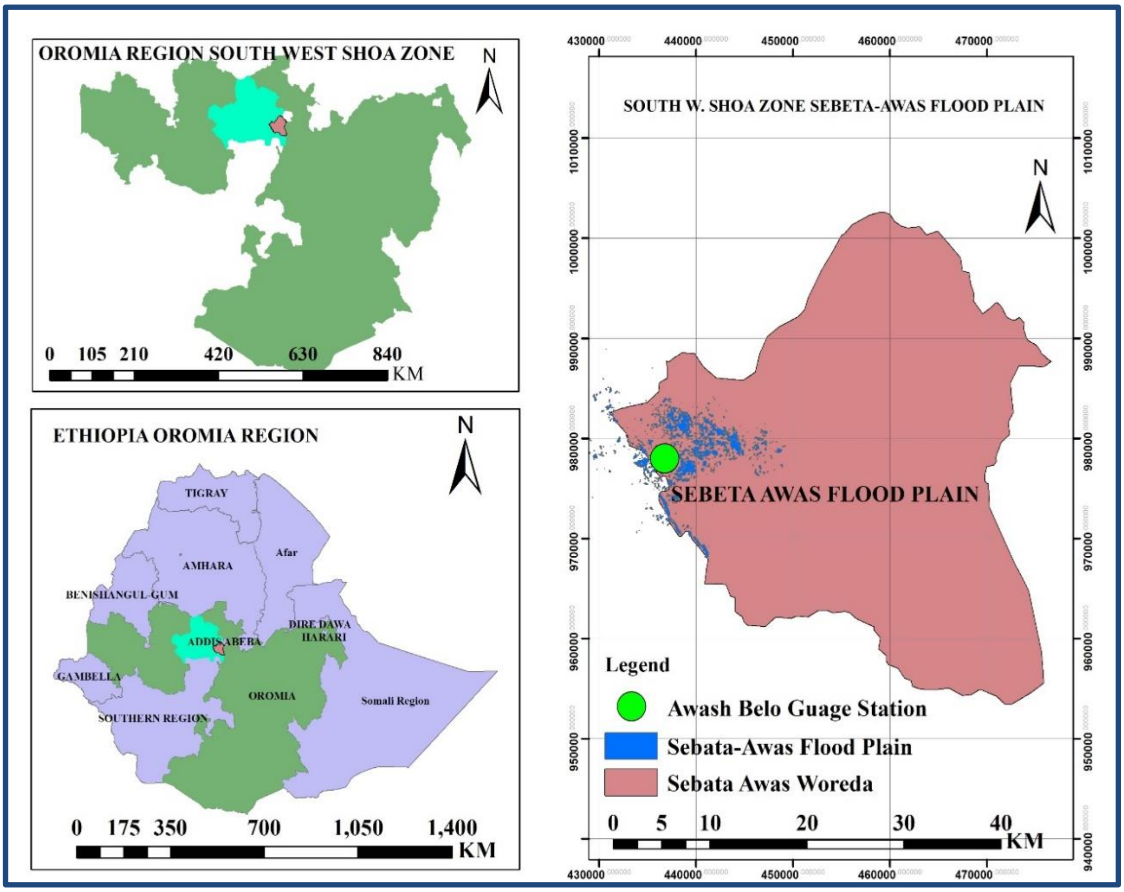

Figure 3-1 Location of study Area

3.1.2 Hydrological and Metrological Characteristics

Climate characteristic of study area is Weinadega type Ethiopia climate. Average temperature of study area varies from 15^0C to 25^0C With Africa equator Sunshine and the average rainfall varies from 700mm to 800mm. The study area receives highest rainfall during summer (Kiremt).

3.1.3 Land use Land Cover

Most of study area occupied by manmade forest and cultivated lands. The main crops are grown in study area are Wheat, Teff, Chickpeas, Lentils, Corn, Sorghum, Maize and vetch (Guaya). According to the woreda Agricultural and rural development office, 87.2% of land devoted the Agriculture, 4.2% is pasture, 2.9 is forest, and 1.86% is reserved for industrial establishment, lakes and other bodies of water cover 1.68% and built land cover 1.28.

3.1.4 Topography and Geology

The altitude of the woreda ranges from 1700 meters about 3385metres above sea level to. The altitude of flood plain varies 2030 meter to 2070 meter above sea level and slope of flood plain was approximately 100% flat area or gentle slope. Small fold characterizes the study area (cyclin and anti-cyclin). Soil type found in the area vertosols type, which always occur gently sloping or moderately steep plateaus. The area found near foot country central highlands with gentle slope have alluvium washed from mountain.

3.1.5 Socio-Economic Condition

Majority of flood plain inhabitant livelihood depend on agriculture and business. On plain along the river, there is a lot of traditional irrigation and livestock production practiced. The high way passes through study area connect south zone of country to capital city of country. Being vicinity to country capital accessible for highway, study area has great opportunity for business.

3.2 Data Collection

3.2.1 Material Used in Collection of Data

There are several tangible tool and software or computer program used for this operation. These are river cross section and distance measuring devices such as optical level, stadia (level road) and tape. Global positioning system (GPS) also used to determine average elevation of landscape above sea level. The computer programs used for this study are HEC-RAS, HEC-FDA, HEC-GeoRAS, Easy fit, Excel, ArcGIS and Google Earth.

3.2.2 Collection of Hydraulic and Flow Data

The annual instantaneous daily flows for 32-years had been collected from Ministry of Water, Irrigation and Energy. Gauging station was located at Awash Below rural town. Rivers flows along the edge of town. The river cross section data were collected from the site through field survey using optical level, stadia and tape. Cross section had been taken at each 100m station for about 10km along the river (Appendix C map:2). Manning roughness was estimated through field survey by studying channel and flood plain condition.

3.2.3 Socio-Economic Flood Damage Data

Socio-Economic flood damage data were collected from Woreda Emergency Preparedness office. Structural and content damage depth was estimated based on field survey by measuring height damage value. Data related to Agriculture type, Average yield per hectare and percent of crop to be cultivated were collected from Woreda rural development Office.

3.2.4 Topographic Map

Digital Elevation Model thirty by thirty meter (DEM 30) was collected from ministry of Water Irrigation and Energy survey department. HEC-GeoRAS TIN, Flood inundation mapping and damage depth estimation were done by using this topographic map.

3.2.5 Secondary Data Sources

Related publications and books were consulted from Addis Ababa institute of technology library. Other damage related reports such as Ethiopian Red Cross Report, Ethiopia

Disaster Prevention and Preparedness Agency and global disaster report were reviewed source in this study.

3.3 Data Organization and Preparation

3.3.1 Estimation of Missed Data

Flow data had been collected from ministry Water, Irrigation and Energy contain missed data. The missed data were filled with statistically acceptable values. The methodology used in filling missed river flow data are linear regression, multiple regression and recession [18]. In this study, annual instantaneous daily flows were estimated by linear regression method. Linear regression was selected based on correlation coefficient.

3.3.1.1 *Pearson correlation coefficient (r)*

Pearson's correlation coefficient is a measure of the strength of the linear relationship between two variables. The accuracy of linear regression is measured by value r. Estimation of missed data by regression acceptable if only if correlation (r) is above 0.6 [18]. Given paired measurements (x_1, y_1), (x_2, y_2) . . . (x_n, y_n), the Pearson correlation coefficient is given by:

$$r=\frac{\sum_{i=1}^{n}(x_i-\bar{x})(y_i-\bar{y})}{\sqrt{\sum_{i=1}^{n}(x_i-\bar{x})^2\sum_{i=1}^{n}(y_i-\bar{y})^2}} \qquad 3.3.1$$

Where $\bar{x}$ and $\bar{y}$ are the sample mean of x_1, x_2….x_n and y_1, y_2…..y_n, respectively

3.3.1.2 *Linear regression analysis*

If x and y are two related variables, then linear regression analysis helps us to predict the value of y for a given value of x expressed by:

$$y=\beta x+\alpha \qquad 3.3.2$$

Where: β is the slope and α is y-intercept.

3.3.2 Test of Independence and Stationarity

The basic assumption in statistical flood frequency analysis are independent and stationarity of the data series. For given size N, the Wald-Wolfowitz method used to test for independent of data set and test for existence of trend in it. For given data set x_1, x_2, x_3 … x_N then determinant factor R estimated below[11].

$$R = \sum_{i=1}^{N-1} x_i x_{i+1} + x_1 x_N \qquad 3.3.3$$

when the element of the samples is independent, R follow normal distribution with mean and variance given below.

$$\overline{R} = \frac{S_1^{\,2} - S_2}{N-1} \qquad 3.3.4$$

$$\operatorname{var}(R) = \frac{S_2^{\,2} - S_4}{N-1} - \overline{R}^2 + \frac{S_1^{\,4} - 4S_1S_2 + 4S_1S_3 + S_2^{\,2} - 2S_4}{(N-1)(N-2)} \qquad 3.3.5$$

Where: $\overline{R}$ is mean determinant R

var(R) is variance of determinant R.

S_r is n[th] moment about the origin R

The statistic $t = \frac{R - \overline{R}}{\operatorname{var}(R)^{\frac{1}{2}}}$ approximately normally distributed with mean Zero and variance Unity and used to test the hypothesis of independence at significance level α by comparing statics ‘t’ with standard normal variate $t_{\frac{\alpha}{2}}$ corresponding to probability of Exceedance $\frac{\alpha}{2}$.

3.3.3 Test for Outliers

An outlier is an observation that deviate significantly from the bulk of data, which may be due to errors in data collection or recording [31]. Outlier test was done by Grubbs and Beck method. Boundary of higher and lower outlier given below.

$$X_H = \exp(\bar{x} + K_N S) \qquad 3.3.6$$

$$X_L = \exp(\bar{x} - K_N S) \qquad 3.3.7$$

Where: $\bar{x}$ is mean of natural logarithm of sample

S is standard deviation of natural logarithm of sample

X_H is higher boundary, above this boundary observed data considered as outlier

X_L is Lower boundary, below this boundary observed flow considered outlier

K_N is (G-B) Constant is function of observed event

$$K_N = -3.6201 + 6.28446N^{1/4} - 2.49835N^{1/2} + 0.491436N^{3/4} - 0.037911 \quad 3.3.8$$

Where: N are numbers of observed data

3.4 Flood Frequency Analysis

Hydrologic data series used for study are; complete duration data series, a partial duration series an Extreme value series. In these thesis extreme value series used because the study of extreme hydrologic event involves selection of sequence of the largest or smallest observation from set of data[10]. Annual instantaneous maximum daily flow data were taken. Among the most widely used probability distribution functions for hydrological analysis, related with flood frequency analysis, are: [12]

- Two and three parameters Lognormal (LN2 and LN3)
- General Pareto
- Log-Pearson Type III (LP-III)
- Extreme Value Type I (EVI)
- Extreme Value Type II (EVII)

3.4.1 Selection of Parent Distribution

The choice of distributions to be used in flood frequency analysis determine accuracy of peak flow predicted[32]. The most popular parent distribution selection methods are goodness of fit test, Kolmogorov-Smirnov test and L-moment Ratio diagram[11]. Based on statistics of parent distribution selection: Gumbel, General Pareto and Log Pearson distributions were selected.

3.4.1.1 *Goodness of Fit Test*

Goodness of fit test is very powerful tool to select a best-fit distribution. Among most popular goodness of fit test: *Chi-Square*, *Anderson Darling* and *Kolmogorov-Smirnov Test*. All three type of goodness fit Test computed by easy fit software and ready for parent distribution selection. Goodness fit test method of parent distribution selection involves comparing theoretical and observed events[11].

$$\chi^2 = \sum_{j=1}^{k} \frac{(O_j - E_j)}{E_j} \qquad 3.4.1$$

Where: O_j is observed events in the class interval j,

E_j is the number of events that expected from theoretical distribution.

3.4.1.2 *Kolmogorov-Smirnov Test*

Kolmogorov-Smirnov test, the sample distribution function $F_N(x)$ from completely specified continuous hypothetical distribution function $F_0(x)$ compared in test[11]. The test statistic D is defined in Eq. 3.4.2

$$D_N = \max\left|F_N(x) - F_0(x)\right| \qquad 3.4.2$$

Where: $F_N(x)$ is N_j/N, Nj is cumulative number of sample events at class limit j.

$F_o(x)$ is 1/k, 2/k, … where k is number of class interval

3.4.1.3 *L-Moment Ratio*

Probability weighted moments and L-moments are used in selection and evaluation of parent distribution. L-moments are derived from probability weighted moments [11].

Probability weighted moment α_k (k =0, 1,2 and 3) are described as follows.

$$\alpha_0 = \overline{X} \qquad 3.4.3$$

$$\alpha_1 = \sum_{i=1}^{n-1} \frac{(n-i)xi}{n(n-1)} \qquad 3.4.4$$

$$\alpha_2 = \sum_{n=1}^{n=2} \frac{(n-i)(n-i-1)xi}{n(n-1)(n-2)} \quad 3.4.5$$

$$\alpha_3 = \sum_{i=1}^{n=3} \frac{(n-i)(n-i-1)(n-i-2)xi}{n(n-1)(n-2)(n-3)} \quad 3.4.6$$

Where: $x, \overline{X}$ and n are event, mean of event and number of events respectively.

L-moments are linear combination of probability weighted moments. Selection of parent distribution is based on ratio of L-moments.

L-moment, λ_r (r =1,2,3 and 4) are given by:

$$\lambda_1 = \alpha_0 \quad 3.4.7$$

$$\lambda_2 = \alpha_0 - 2\alpha_1 \quad 3.4.8$$

$$\lambda_3 = \alpha_0 - 6\alpha_1 + 6\alpha_2 \quad 3.4.9$$

$$\lambda 4 = \alpha_0 - 12\alpha_1 + 30\alpha_2 - 20\alpha_3 \quad 3.4.10$$

Where: α_k probability weight moment (k =0, 1,2 and 3)

L-Moment ratio for τ and τ_r are given by equation (3.4.11) were used in selection parent distribution.

$$\tau_1 = \frac{\lambda_2}{\lambda 1} \text{ And } \tau_r = \frac{\lambda_r}{\lambda_2} \text{ for } x \geq 3, \quad 3.4.11$$

3.4.2 General Pareto Distribution

The generalized Pareto probability distribution define observed events, cumulative density function and probability density function as follows[11].

$$x = \varepsilon + \frac{\alpha}{k}[1\text{-}(1\text{-}F)^k] \quad 3.4.12$$

$$F(x) = 1 - \left[1 - \frac{k}{\alpha}(x - \varepsilon)\right]^{1/k} \quad 3.4.13$$

$$f(x) = \frac{1}{\alpha}\left[1 - \frac{k}{\alpha}(x-\varepsilon)\right]^{1/k-1} \qquad 3.4.14$$

Where: x is Events

$f(x)$ is Probability density

$F(x)$ is Cumulative density

α , ε and k are constant

The variable x takes the value in range $\varepsilon \le x < \infty$ for $k \le 0$ and $\varepsilon \le x \le \varepsilon + \frac{\alpha}{k}$ for $k > 0$.

Parameter for General Pareto distribution are determined from population mean, standard deviation and skewness.

$$\mu_1^{'} = \int_{\varepsilon}^{\infty} \frac{x}{\alpha}\left[1 - \frac{k}{\alpha}(x-\varepsilon)\right]_{dx}^{1/k-1} \qquad 3.4.15$$

$$\mu_1^{'} = \varepsilon + \frac{\alpha}{1+k} \qquad 3.4.16$$

$$\mu_2 = \frac{\alpha^2}{(1+k)^2(1+2k)} \qquad 3.4.17$$

$$C_s = \frac{2(1-k)(1+2k)^{1/2}}{1+3k} \qquad 3.4.18$$

Where: $\mu_1^{'}, \mu_2$ and C_s mean, standard deviation and skewness respectively.

Parameters are determined by numerically solving equation (3.4.15 to 3.4.18). Quantile estimation of flow and corresponding frequency factor for different return period given by:

$$x_T = \varepsilon + \frac{\alpha}{k}[1 - T^k] \qquad 3.4.19$$

$$K_T = \frac{(1+2k)^{1/2}}{k}[(1+k)(1-T^{-k}) - k] \qquad 3.4.20$$

Where x_T is Quantile (peak flow at any return period)

K_T is Frequency factor

T is Return period

3.4.3 Gumbel Method

This extreme value distribution introduced by Gumbel (1941) and commonly known as Gumbel distribution. It is one of most widely used probability distribution function for extreme values in hydrologic and Metrologic studies for prediction flood peaks, maximum rainfalls, maximum wind speed, etc.[33]

$$x_T = x_{av} + k_{Tr} S_{n-1} \qquad 3.4.21$$

Where: x_T is Maximum flood peak discharge

x_{av} is Average value of discharge

S_{n-1} is Standard deviation of sample size

$$k_{Tr} = -\frac{\sqrt{6}}{\pi}\{0.5772 + \ln[\ln(\frac{Tr}{Tr-1})]\} \qquad 3.4.22$$

Where: Tr is return period

Quantile confidence interval given by:

$$\mathrm{x}_{1/2} = x_T \pm f(c) S_e \qquad 3.4.23$$

$S_e = b\frac{\sigma_{n-1}}{\sqrt{N}}$ Probable error

$$b = \sqrt{1 + 1.3k + 1.1k^2}$$

Where: k, σ_{n-1}, N and $f(c)$: frequency factor, sample standard deviation, number of observed events and tabulated confidence probability function respectively. Where, $f(c)$ was determined from table. The confidence interval indicates the limits about the calculated value between which the true value can be said to be lie with specific probability based on sampling errors only.

3.4.4 Log-Pearson III Distribution

If the variable log (x) assumed to have a Pearson III distribution, then distribution of variable x is log-Pearson distribution. This distribution is extensively used USA for project sponsored by USA government[33]. If x is variate of random hydrologic series, the series of Z variates are determined below.

$$y = \log x \tag{3.4.23}$$

$$K_{Tr} = z + (z^2 - 1)k + \frac{1}{3}(z^3 - 6z)k^2 - (z^2 - 1)k^3 + zk^4 + \frac{1}{3}k^5 \tag{3.4.24}$$

$$k = \frac{C_S}{6} \tag{3.4.25}$$

$$z = w - \frac{2.515517 + 0.802853w + 0.010328w^2}{1 + 1.432788w + 0.189269w^2 + 0.001308w^3} \tag{3.4.26}$$

$$w = [\ln(\frac{1}{p^2})]^{1/2} \tag{3.4.27}$$

$$Z = \bar{y} + \sigma_{sy} K_{Tr} \tag{3.4.28}$$

$$X_T = ant\log(Z) \tag{3.4.29}$$

Where X_T is Quantile (peak flow a given return period)

P is 1/T (Inverse of return period)

C_S is Skewness

K_{Tr} is Frequency factor

z and w are constant, they are function of return period

3.5 Flood Protection Structure

There various river engineering works that are used either individually or in combination to provide flood protection and reduce flood damage along the river reaches[9]. Most popular flood protection structure are Canal (River improvement), Dyke, Dam, Floodway and Retarding basin

Floodway is aimed at diverting floodwater to the sea, lake or another main river from the existing river. A retarding basin has the same function as a dam used in mountainous area. The flood peak discharge is reduced and stored in the reservoir and later released so as to reduce the peak discharge in the downstream[9].

In this study, based on topography of study area Canal and Dyke were selected. The topography of the study area is characterized by meandering river and gentle slope (approximately flat), which requires river improvement to increase the flow capacity of the existing river channel. River improvement includes widening, dredging or excavation and dike construction[9].

3.5.1 Canal

Major flood damages occurred in study area are due to overtopping, silting and changing of river way through a crop field. Hydraulic study of this flood protection canal must be considering all these entire problems. The non-silting and non-scoring canal profile is required. Non-silting and non-scoring canal slope and canal cross-section are given below [34].

$$S = \frac{n^2 m^{2.02}}{1.338 Q^{0.02}} [0.258 + (15 + 6.44Q)^{-0.417}]^{0.6135} \quad 3.5.1$$

$$\frac{B}{D} = (15 + 6.44Q)^{0.382} \quad 3.5.2$$

$$D = [\frac{1.818Q}{(\frac{B}{D} + 0.5)m}]^{0.3788} \quad 3.5.3$$

Where: S is Slope

n is Manning number

m is Critical velocity ratio (function of soil type)

Q is Discharge

B is canal width

D is canal depth

Longitudinal Canal bed elevation determined from longitudinal slope and horizontal distance along river at any station.

3.5.1.1 *Canal Freeboard*

The freeboard of a Canals is an allowance in height, and it is the function of design flood discharges.

Table 3-1 Recommended Freeboard For Canal Source [34]

No	Q(CUM)	Freeboard (m)
1	1 to 5	0.5
2	5 to 10	0.6
3	10 to 30	0.75
4	30 to 150	0.9

3.5.1.2 *Critical velocity and Critical Velocity Ratio*

Critical velocity is the mean velocity which will just make the channel free from silting and scouring. The velocity is based on the depth of the water in the channel. The general form of critical velocity is as follow[34]:

$$V_0 = 0.546D^{0.64} \qquad 3.5.4$$

Where: V_0 is Critical velocity

D is Full supply Depth

0.546 and 0.64 are Constant

$$V = 0.546mD^{0.64} \qquad 3.5.5$$

Where: V is actual Velocity

m: critical velocity ratio which equal to actual velocity (V) divided by critical velocity (Vo), Critical velocity ratio is depend on soil type.

Table 3-2 Critical velocity ratio for different soil type source[34]**.**

No	Type of soil	Critical velocity ratio(M)
1	silt	0.7
2	sandy silt	1
3	sandy silt, a little courser	1.1
4	sandy, loamy silt	1.2
5	debris of hard soil	1.3

3.5.1.3 *Manning Roughness for Canal*

Values of the roughness coefficient “n “ may be assigned for conditions that exist at the time of a specific flow event, for average conditions over a range in stage, or for anticipated conditions at the time of a future event[35].

Table 3-3 Recommended Manning Roughness for Canal Source[34]

`No	Q(CUM)	n
1	14 to 140	0.025
2	140 to 180	0.0225
3	above 280	0.02

3.5.2 Dyke

Levee or flood wall were designed for 50-year to 100-year peak flood[33]. HEC-FDA requires to specify levee size and failure characteristics and interior versus exterior stage relationships associated with the levee[19]. Currently, no levee structure constructed in the study area. Therefore, no failure characteristics or probability failure was determined. Levee size, interior stage and exterior stage were determined from computed water surface profile and average ground level.

3.5.2.1 *Dyke Height*

The height of a dike is based on the design flood level with a required freeboard added to it. In this case, flood water levels should be calculated and consider the longitudinal gradient of provisional design flood level.

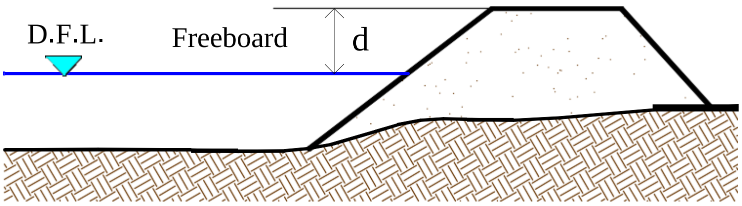

Figure 3-2 Dyke Height Source [9]

$$\text{Dike height} = \text{Design flood level} + \text{Freeboard} \tag{3.5.6}$$

3.5.2.2 *Dyke Freeboard*

The freeboard of a dike is an allowance in height and it is the function of design flood discharge.

Table 3-4 Minimum Required Freeboard for Dyke Source [9]

Design Flood Discharges (Cumecs)	Free Boards (m)
< 200	0.6
200-500	0.8
500-2000	1.0
2000-5000	1.2
5000-10,000	1.5
>10,000	2.0

3.5.2.3 *Dyke Crest*

The crest width of a dike shall be in accordance with the design flood discharge, and shall not be less than the value given in Table below

Table 3-5 Recommended Crest width for Dyke Source [9]

Design Flood Discharges (Cumecs)	Crest width (m)
<500	3
500-2000	4
2000-5000	5
5000-10,000	6
>10,000	7

3.5.2.4 *Dyke Slope*

The slope of a dike shall have a gentle gradient of 2:1 (Horizontal to Vertical) or less. The slope is decided based on the dike's body. A gradient steeper than 2:1 is generally not preferable in view of the stability conditions of the slope face[9].

3.6 Steady flow and Flood Damage Analysis

For this study, the general method adopted for steady flow and flood damage analysis basically consist of seven steps:

i. Organization and Preparation of discharges, cross section, Manning number and boundary condition data.
ii. Preparation of TIN (Triangulated Irregular Network) in ArcMap for GeoRAS processing.
iii. GeoRAS Pre-processing to generate HEC-RAS Import file.
iv. Cross section estimated by GeoRAS replaced by field data in HEC-RAS.
v. Running of HEC-RAS to calculate water surface profiles.
vi. From HEC-RAS results, flood inundation mapping and flood Damage estimation.
vii. From imported Water surface profile and manually organized flood damage, HEC-FDA compute expected annual damage, equivalent annual damage and project performance.

3.6.1 HEC RAS Steady Flow Analysis

This is the major part of the model where simulation is done. The import file created by HEC-GeoRAS was imported in Geometric Data Editor interface within HEC-RAS. All the required modification, editing was done at this stage. The flood discharge for different return periods were entered in steady flow data. Reach boundary conditions and Manning Roughness were also entered in this window. Then, water surface profiles were calculated in steady flow analysis window. Water surface profiles were computed from one cross section to the next by solving the energy equation. The flow data were entered in the steady flow data editor for eight return periods as 2-year,5-year,10-year, 25-year 50-year, 100-

year, 250-year and 500-year. Boundary condition was defined as normal depth for both upstream and downstream side.

3.6.1.1 *Preparation of TIN (Triangulated Irregular Network)*

TIN generated in ArcMap from DEM (Digital Elevation Model) for GeoRAS processing. River cross section that have been estimated from TIN, derived from 30 DEM has very poor quality. To overcome such wrong estimation river cross section data derived from TIN was replaced by field measured data in HEC-RAS model.

3.6.1.2 *River System Manning's Roughness Coefficient*

The roughness of a surface affects the characteristics of runoff, whether the water is on the surface of the watershed or in the channel. With respect to the hydrologic cycle, the roughness of the surface retards the flow. For overland flow, increased roughness delays the runoff and increase the potential for infiltration. Reduced velocities associated with increased roughness should also decrease the amount of erosion. The manning roughness for manmade channel or canal reviewed from literature[26]. The general effects of roughness on flow in a channel are similar to those for overland flow. There are various method available for manning number estimation in channel and flood plain [36]. These are:

- Field observation
- Photo of Calibrated stream
- Published documents
- Formulas
- Calibrated to observed profile

Calibrated to observed water surface profile simple and best approach, however due to luck rating curve it is impossible to apply. In this thesis, field observation or commonly known Cowan method is used to estimate Manning roughness coefficients[36]. Equivalent manning roughness as function of channel and flood plain condition.

$$n = (n_b + n_1 + n_2 + n_3 + n_4)m \qquad 3.6.1$$

Where: n is Equivalent manning number

n_b is Manning roughness number bare soil

n_1 is Channel and flood plain irregularities correction

n_2 is Channel and flood plain cross section correction

n_3 is Channel and Flood plain Irregularities correction

n_4 is Vegetation Cover correction

m is Meandering effect correction

Table 3-6 Manning Roughness for Channel Source[37]**.**

	Channel Condition	"n"	Example
Degree of Irregularity (n_1)	Smooth	0.000	Compares to the smoothest channel attainable in a given bed material.
	Minor	0.001-0.005	Compares to carefully dredged channels in good condition but having slightly eroded or scoured side slopes.
	Moderate	0.006-0.010	Compares to dredged channels having moderate to considerable bed roughness and moderately sloughed or eroded side slope
	Severe	0.011-0.020	Badly sloughed or scalloped banks of natural streams; badly eroded or sloughed sides of canals or drainage channels; unshaped, jagged, and irregular surfaces of canal in rock
Variation in Channel Cross Section (n_2)	Gradual	0.000	Size and shape of channel cross sections change gradually.
	Alternative occasionally	0.001-0.005	Large and small cross sections alternate occasionally, or occasionally the main flow occasionally shifts from side to side Owing to changes in cross-sectional shape.
	Alternative Frequently	0.01-0.015	Large and small cross sections alternate frequently, or frequently the main flow frequently shifts from side to side owing to changes in cross-sectional shape.
Effect of Obstruction	Negligible	0.000-0.004	A few scattered obstructions, which include debris

(n_3)			Negligible deposits, stumps, exposed roots, logs, piers, or isolated boulders, that occupy less than 5 percent of the cross-sectional area
	Minor	0.005-0.015	Obstructions occupy less than 15 percent of the cross-sectional area and the spacing between obstructions are such that the sphere of influence around one obstruction Minor does not extend to the sphere of influence around another obstruction. Smaller adjustments are used for curved smooth-surfaced objects than are used for sharp edge irregular objects
	Appreciable	0.02-0.03	Obstructions occupy from 15 to 50 percent of the cross-sectional area or the space between obstructions is Appreciable small enough to cause the effects of several obstructions to be additive, there by blocking an equivalent part of a cross section.
	Severe	0.04-0.05	Obstruction occupy more than 50 percent of the cross- sectional area or the space between obstructions is Small enough to cause turbulence across most of the cross section.
Amount of vegetation (n_4)	Small	0.002-0.01	Dense growths of flexible turf grass, such as Bermuda, or weeds growing where the average depth of flow is at least two times the height of the vegetation; supple tree seedlings such as willow, cottonwood, arrowed, or salt cedar growing where the average depth of flow is at least three times the height of the vegetation.
	Medium	0.010-0.025	Turf grass growing where the average depth of flow is from one to two times the height of the vegetation moderately dense steamy grass , weeds, or tree seedlings growing where the average depth of flow is from two to three times the height of the vegetation: brushy, moderately dense vegetation, similar to l- to 2-year-old willow trees in the dormant season, growing along the banks and no significant

			vegetation along the channel bottoms where the hydraulic radius exceeds 2 feet.
	Large	0.025-0.050	grass growing where the average depth of flow is about equal to the height of vegetation: 8- to lo-year old willow or cottonwood trees inter grown with some weeds and brush (none of the vegetation in foliage) where the hydraulic radius exceeds 2 feet; bushy willows about 1 year old inter grown with some weeds alongside slopes (all vegetation in full foliage) and no significant vegetation along channel bottoms where the hydraulic radius is greater than 2 feet.
	Very large	0.05-0.10	Turf grass growing where the average depth of flow is less than half the height of the vegetation: bushy willow Very large trees about 1 year old inter grown with weeds alongside slopes (all vegetation in full foliage) or dense cattails growing along channel bottom; trees inter grown with weeds and brush (all vegetation in full foliage).
Degree of Meandering (m)	Minor	1.0	Ratio of channel length to valley length 1.0 to 1.2
	Appreciable	1.15	Ratio of the channel length to valley length is 1.2 to1.5.
	Sever	1.30	Ratio of the channel length to valley length is greater Than 1.5.

Table 3-7 Manning Roughness for Flood plain Source [37]**.**

	Flood Plain Condition	"n"	Example
Degree of Irregularity(n_1)	Smooth	0.000	Compares to the smoothest, flattest flood plain attainable in a given bed material.
	Minor	0.001-0.005	Is a flood plain with minor irregularity in shape. A few rises and dips or sloughs may be visible on the flood plain.
	Moderate	0.006-0.010	Has more rises and dips. Sloughs and hummocks ay occur

	Sever	0.011-0.02	The flood plain is very irregular in shape. Many rises and dips or sloughs are visible. Irregular ground surfaces in pastureland and furrows perpendicular to the flow are also included.
Variation of Floodplain Cross Section (n_2)	Not Applicable	0.000	
Effective of Obstruction (n_3)	Negligible	0.000-0.004	A few scattered obstructions, which include debris deposits, stumps, exposed roots, logs, or isolated boulders, occupy less than 5 percent of the cross-sectional area.
	Minor	0.005-0.019	Obstructions occupy less than 15 percent of the cross-sectional area
	Sever	0.02-0.03	Obstructions occupy from 15 to 50 percent of the Cross-sectional area.
Amount of Vegetation (n_4)	Small	0.001-0.010	Dense growth of flexible turf grass, such as Bermuda, or weeds growing where the average depth of flow is at least two times the height of the vegetation: or supple tree seedlings such as willow, cottonwood, arrow weed, or salt cedar growing where the average depth of flow is at least three times the height of
	Medium	0.011-0.025	Turf grass growing where the average depth of flow is from one to two times the height of the vegetation; or moderately dense steamy grass, weeds, or tree seedlings growing where the average depth of flow is from two to three times the height of the vegetation; brushy, moderately dense vegetation, similar to l- to 2-year-old willow trees in the dormant season
	Large	0.025-0.05	Surf grass growing where the average depth of flow is about equal to the height of vegetation: or 8- to lo-year-old willow or cottonwood trees inter grown with some weeds and brush (none of the vegetation in foliage) where the hydraulic radius exceeds 2 ft.; or mature row crops such as small vegetables; or mature field

			crops where depth of flow is at least twice the height of the vegetation.
	Very large	0.05-0.10	Surf grass growing where the average depth of flow is less than half the height of the vegetation: or moderate to dense brush: or heavy stand of timber with few down trees and little undergrowth with depth of flow below branches: or mature field crops where depth of flow is less than height of the vegetation.
	Extreme	0.1-0.2	Dense bushy willow, mesquite, and salt cedar (all vegetation in full foliage): or heavy stand of timber, few down trees, depth of flow reaching branches.
Degree of Meander (m)		1.0	Not applicable

3.6.1.3 *Boundary Condition*

Boundary conditions are necessary to establish the starting water surface at the ends of the river system upstream and downstream. A starting water surface is necessary in order for the program to begin the calculation. In a subcritical flow regime, Boundary conditions are only necessary at the downstream ends of the river system. If a supercritical flow regime is going to be calculated, Boundary conditions are only necessary at the upstream ends of the river system. If the mixed flow regime calculation is going to be made, then Boundary condition must be entering at all ends of the river system. According to the model, there are four types of boundary conditions[25]. These are:

- Known water surface elevation
- Normal depth
- Critical depth
- Rating curve

Known water surface elevation had been applied at junction of reservoir and river. Rating curve and critical depth applied in area where water elevation and it’s discharge available. Since rating curve haven’t been developed yet for most our rivers, it is not applied. Normal depth selected, upstream and downstream channel bed slope required to estimate normal depth.

3.6.2 HEC-FDA Flood Damage Analysis

All flood damage analysis results i.e. expected annual damage, equivalent annual damage and project performance are computed by HEC-FDA program. This program compute three functions from imported water surface profile and manually organized depth damage estimation. These functions are discharge exceedance function, depth exceedance function and aggregate depth damage function. All these functions used by HEC-FDA for flood damage analysis[19].

3.6.2.1 *Study of HEC-FDA Configuration*

Study configuration contains the data describing the physical layout for the study area and plan definition for analysis. Data includes streams, damage reaches, plans and analysis years[19].

3.6.2.1.1 Streams

Streams include various water bodies such as rivers, streams, ditches, canals, bayous, lakes, ponds, etc. Streams are defined for the study area, therefore common for all plans and analyses year [19]. A study may include one or more streams. In thesis only one stream defined.

3.6.2.1.2 Damage Reaches

Damage reaches are specific flood susceptible geographical areas within a floodplain[19]. They are used to define consistent data for plan evaluations and to aggregate structure and other potential flood inundation damage information by stage of flooding. In this study, damage reaches stretch from 2000m river station to 10400m river station.

3.6.2.1.3 Plans

A plan may represent the with or without project conditions[19]. The with-project condition plan consists of one or more flood damage reduction measures and actions. For this research without project (without flood protection structure), Canal as flood protection and integration of Canal and Dyke are studied.

3.6.2.1.4 Analysis Years

Analysis years define damage and project performance information for specific time periods during the project life, such as the base year or most likely future year[19]. Usually analysis year 30 year. In this study analysis year 2019 to 2049 accepted.

3.6.2.1.5 Price Index

Price index used to calculate time value of money. It updates economic damage values at given analysis year[19]. Ethiopian commercial bank interest rate 7% in march used as price index.

3.6.2.2 *Flood Damage Economics*

Flood damage economics is where you perform data entry and computations to produce stage-damage functions with uncertainty for flood damage reduction [19].

3.6.2.2.1 Damage Category

Damage categories are used to consolidate large numbers of structures into specific categories with similar characteristics for analysis and reports[19]. Typical damage categories include: residential, commercial, industrial, open space, and public facilities. In this study, Ethiopian rural shelter (residential damage) considered.

3.6.2.2.2 Structural Occupancy Type

A structure occupancy type describes a class of structures. Data entered for a structure occupancy type is applied to all the structures assigned to that structure occupancy type[19]. In this works, we have two villages based on first floor variation. First floor of structure above sea level estimated from google earth and counter derived from DEM. Structural occupancy depth damage estimated by randomly selected structure (house) and registering damage at given height from first floor elevation.

3.6.2.2.3 Structural Inventory

Structure inventories are performed to develop a record of the attributes of unique or groups of structures relevant to flood damage analysis[19]. The information is used to compute an aggregated stage-damage function by damage category at the damage reach index location station.

3.6.2.2.4 Structural Cost

Cost of structure damaged i.e. cost maintenance badly damaged mud hut and clearing silt were estimated by randomly selecting structure (house) and registering damage in current USA dollar. Damage depth estimated from height of structure.

3.6.2.2.5 Content Cost

This include cost of damaged family food, animal food and house material perished by flood. Data collected from Woreda emergency preparedness office.

3.6.2.2.6 Other Cost

Other cost is damage happened to cultivated land and landscape. The most damage reported every year Agricultural loss. Amount of land inundated in hectare estimated by ArcMap from water surface profile.

$$C_T(\$) = Y * X * (\%cultivated) * A_T \qquad 3.6.2$$

Where: C_T *($)* is damage cost of cultivated land at given elevation

A_T is land inundated due to rise of water in River (in Hectare).

Y is yield in kuntal or 100 kg yield per Hectare

X is $\left(\frac{\cos t(\$)}{kuntal}\right)$ Cost of one Kuntal (100kg cultivated land yield) dollars

% Cultivated is percent of different cereal sown.

3.6.2.3 *Expected and Equivalent Annual Damage Analysis*

Monte Carlo simulation is used in HEC-FDA to derive the expected annual damage (EAD) corresponding to a particular plan or analysis year for a damage reach. EAD is the mean damage obtained by integrating the damage exceedance probability curve for the damage reach. The inclusion of uncertainty for these variables requires that a numerical integration approach to be applied[19].

$$EAD = \int_{0}^{\infty} Df(D)dD \sim \sum_{i=1}^{i=N} D_i \Delta_p \qquad 3.6.3$$

Where: EAD is Expected Annual Damage

Δ_p is probability of damage

Df is damage exceedance probability

D is annual damage

N is number of intervals

Equivalent annual damage is flood damages associated with a plan are calculated in average annual equivalent terms. The procedures discount the expected annual damage stream to the beginning of the period of analysis or the base year.

3.6.2.4 *Project performance*

Plan and damage reach project performance analyses are based on target standards defined for without-project conditions for the study. There are three different cases for determining the target[19]:

i. For reaches without levees, the target is based on an estimate of the stage at which significant damage begins for the without condition,
ii. For reaches with a levee that have no geotechnical failure, the target is the top of levee stage, and
iii. For reaches with a levee that have geotechnical failure, the target stage is based on both the annual exceedance probability and the probability of failure.

Experience at HEC has shown that a 5 percent residual damage associated with the 0.01 exceedance probability event is normally a good target stage and was adopted as the FDA default target stage.

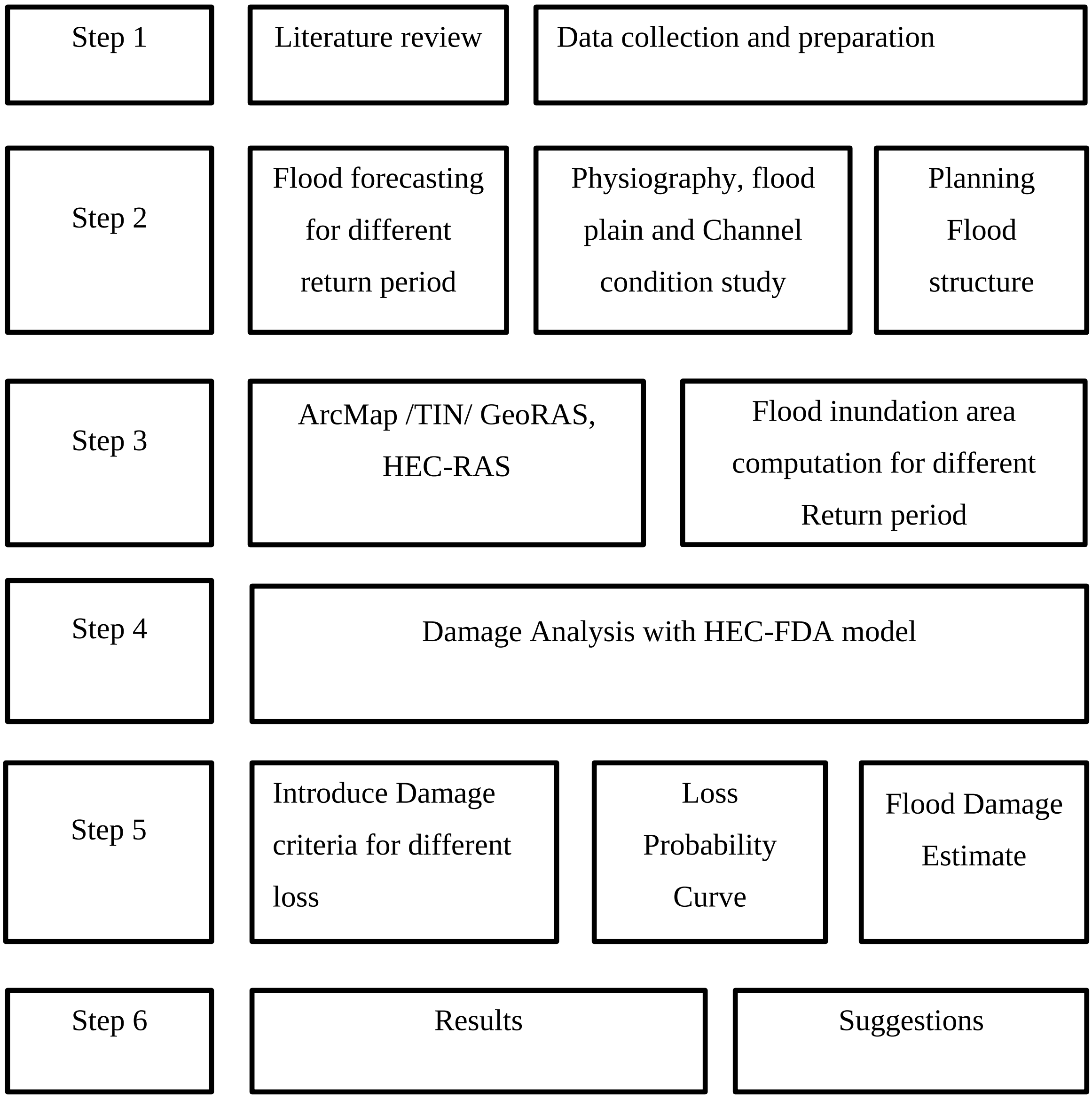

Figure 3-3 Summary of Methodology

CHAPTER 4 RESULTS AND DISCUSSION

4.1 Result of Missed Data Estimation

4.1.1 Pearson Correlation Coefficient (r)

Pearson's correlation coefficient is a measure of the strength of the linear relationship between two variables[18]. The value of correlation coefficient ranges between -1 and 1. If the two variables are in perfect linear relationship, the correlation coefficient will be either 1 or -1. The sign depends on whether the variables are positively or negatively related. The correlation coefficient is 0 if there is no linear relationship between the variables. Estimation of missed data by regression acceptable if only if correlation (r) is above 0.6. In this work, relation between instantaneous annual maximum daily flow(y) and year(x) are directly proportional with correlation coefficient 0.93 (from equation 3.3.1). This shows that precise and accurate estimation.

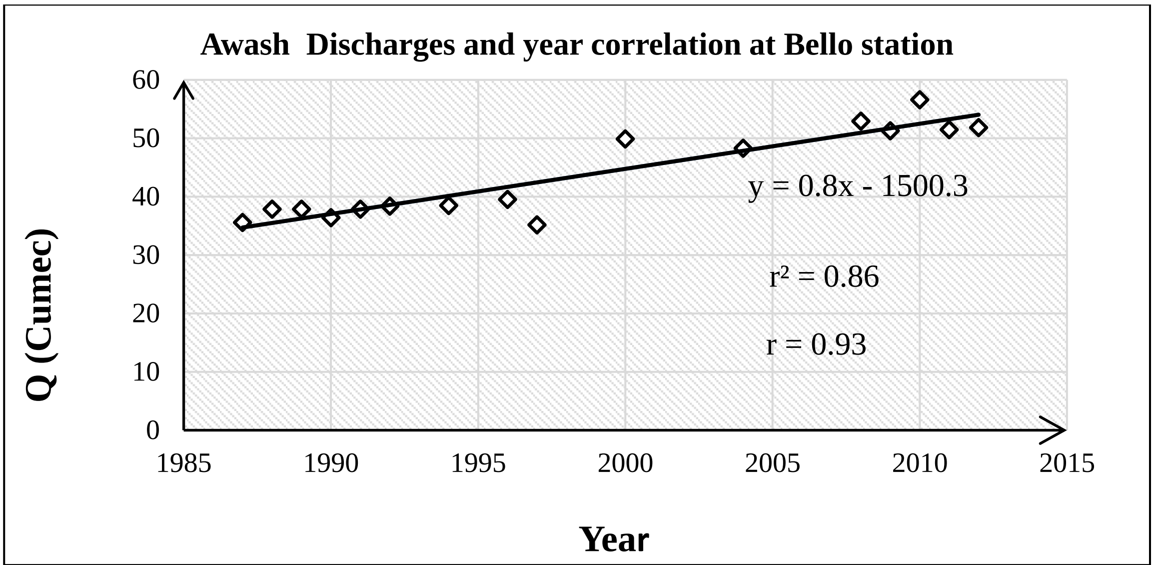

Figure 4-1 Pearson Correlation Coefficient for year and discharges.

4.1.2 Regression Equation for Estimation of Missed Data

Regression equation for estimation of missed data was done by using excel or spread sheet by analysis of variances (ANOVA). The intercept and slope for boundary of 95% confidence level was tabulated in Table 4-1. Percent of error (P-Value) and degree of variability (F) are very low or approach zero, which mean estimation is accurate.

$$y=\beta x+\alpha$$

Where: β is 0.8 Slope

α is -1500.3 Intercept

y is discharges

x is year

Table 4-1 Intercept and slope for median, lower and upper 95%

	Intercept (α)	Variable (β)
Coefficients	-1500.3	0.8
Lower 95%	-1864.3	0.6
Upper 95%	-1136.3	0.95
Standard Error	169.7	0.1
Significant F	3E-07	3E-07
P-value	4.2E-07	3E-07

Maximum annual daily instantaneous missed flow data were estimated and filled by this regression equation. Flows belongs to 1983, 1984, 1985 and 2013 years were missed or not recorded. The maximum monthly flow data for years 1986, 1993, 2001 and 2002 were missed. The daily instantaneous flow data for years 1998, 1999, 2005 and 2006 were lost (Appendix A Table 1).

The maximum annual daily instantaneous flow for years 1983, 1984, 1985, 1986 and 2013 were estimated to 31.6, 32.4, 33.2, 33.9 and 54.8 Cumecs respectively. For years those have maximum monthly missed data 1986, 1993, 2001, and 2002 their annual daily instantaneous flows had been estimated to 33.9, 39.4, 45.5 and 46.3 Cumecs respectively. In same way for years those have maximum daily missed data i.e 1998, 1999, 2005 and 2006 their missed flows were estimated to 43.2, 44, 48.6 and 49.4 Cumecs respectively (Appendix A Table 3).

4.2 Result of Data Test

4.2.1 Independence and Stationery Test

Wald Wolfowitz method was used to test independence and stationary of data series. The “t” was calculated from equation 3.3.3 through 3.3.5 and it was compared with standard normal variate at 95% significance level. Hypothesis of independence and stationary was accepted only if the absolute value of the calculated ‘t’ is less than the tabulated ‘t’ at 95% significance level. Otherwise, it will be rejected. The test shows that “t” calculated was 0.05 and it was less than “t” tabulated which is 2.01 (Appendix A Table 5). This indicates that the hypothesis (tentative explanation) of independent and stationary was accepted.

4.2.2 Outlier Test

An outlier is an observation that deviates significantly from the bulk of the data, which may be due to errors in data collection, or recording. Outlier test was done by Grubbs and Beck method. The results of test show that there were no outliers. Boundary of outlier was determined by equation 3.3.6 to 3.3.8. Maximum and minimum Observed bulk of data were considered as maximum and minimum outliers, only if above 67.6 Cumecs. and below 27.3 Cumecs respectively (Appendix A Table 6). All of the observed flow data are falls between 27.3cumecs to 67.6 Cumecs.

4.3 Result of Flood Frequency Study

From goodness fit test statistics three frequencies distribution function were selected (Appendix A Table 10). These are General Pareto, Gumbel or General Extreme I and Log Pearson III. By using L-moment Wakeby distribution (modified General Pareto) was best fit (Appendix A Table 8 and 9). These three flood frequencies were used widely in hydrology in flood peak forecasting. Choosing a proper quantile is a crucial interest for the quality of the results of ongoing analyses. A too low discharges will lead to a great number of floods, which likely violates the requirements of the statistical model [38]. This will result in a major bias. Historical fact show that flood damage had happened due to extreme (highest) flood event. Therefore, highest flood was estimated by extreme value I

(Gumbel method) used in this study. Peak flows were estimated for different return period by selected parent distribution by using equation (3.4.12 to 3.4.29) summarized in table 4.2.

Table 4-2 Flood Frequency for different return period

	General Pareto		Gumbel Method		Log Pearson III	
Return period	K tr	Q (Cum)	K tr	Q(cum)	K tr	Q(cum)
2	-0.13	44.47	-0.16	42.36	-0.019	42.83
5	0.062	44.99	0.72	49.01	0.84	49.72
10	0.27	45.56	1.30	53.41	1.29	53.86
25	0.68	46.71	2.04	58.98	1.79	58.74
50	1.14	47.95	2.59	63.10	2.11	62.17
100	1.78	49.68	3.14	67.2	2.41	65.46
250	3.01	53.04	3.86	72.59	2.77	69.68
500	4.37	56.75	4.4	76.66	3.02	72.79

The value of the variate for a given period X_T determined by Gumbel's method it may have error due to limited data used, an estimate confidence limit required. The confidence

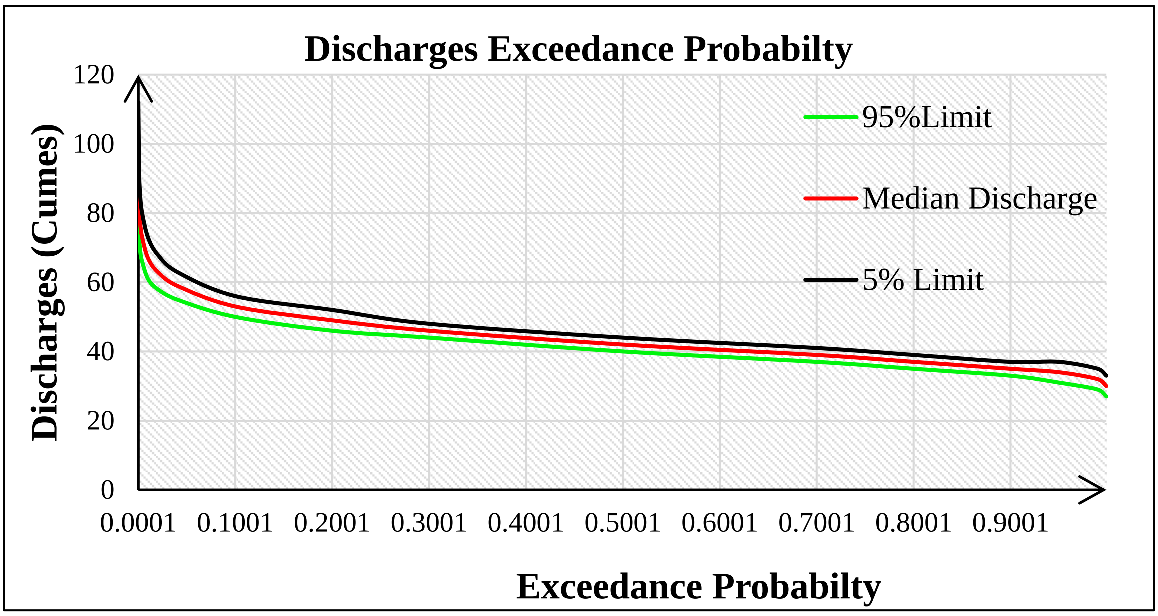

Figure 4-2 Extreme I (Gumbel) Discharge Exceedance Probability

interval indicates the limits about the calculated value between which the value lies with specific probability based on sampling error. It is the function of frequency factor and standard error. It was determined by equation (3.4.23).

4.4 Result of Flood Protection Structure Setup

4.4.1 Canal Profile

The miscellaneous Hydraulic structure such as wear, aqueducts, cross drainage works, flood embankment or levee were designed for 50 to 100 years flood frequency[33]. Flood frequency for 50 years was estimated by Gumbel method is 63.1 Cumecs. Non-scoring and non-silting slope by equation 3.5.1, for given discharges (63.1 Cumecs), for Manning roughness ($n = 0.025$, table 3.3) and critical velocity ratio ($m = 1$, table 3.2) was computed to 0.000221. Width (B) and depth (D) by equation 3.5.2 and 3.5.3 were estimated to 25m and 2.5m respectively. By providing 1 m freeboard (table 3.1), canal has 32m top width. Earthen unlined side slope H : V, 1:1 Slope was provided[34]. Longitudinal bed of canal was adjusted by multiplying longitudinal slope with station (horizontal distance).

4.4.2 Assigning Dyke to Plan

Dyke was added to river and flood protection structure in HEC-FDA model[19]. Assigning a dyke to river in this particular topography is not economical. Damped water was increased in all direction; therefore, it is not appropriate for dyke site. Assigning dyke on Canal is appropriate decision to reduce amount of cut depth. Under Levee Features, you specify levee size, failure characteristics and interior versus exterior stage relationships associated with the levee.

4.4.2.1 *Dyke Height*

The height of a dike was computed by adding 100-years flood level to a required freeboard (equation 3.5.6). Level of 100-year flood is 2040.5m above sea level in Canal. Recommended freeboard for 67.2 Cumecs was 0.6m (Table 3.4). Top dyke height was 2041.1m above sea level. The height of dyke above average ground level is 2m. Average ground level is about 2039 above sea level.

4.4.2.2 *Dyke failure characteristic*

Dyke failure characteristics was determined from type of levee and structural stability. Dyke failure characteristics are two types, these are levee with probability of geotechnical failure and levee without probability of geotechnical failure. To estimate failure characteristic, dyke or levee had been constructed a year ago or detail design levee required. The scope this study only limited to estimation damage reduced by providing levee(dyke). In this study, dyke(levee) without probability of geotechnical failure was considered.

4.4.2.3 *Dyke Water Stage*

The water surface stage in the Canal or exterior side of the levee determined from water surface profile computed from HEC-RAS (Appendix A Table 14 & Appendix B figure 10). Interior water stage was happened due to inland flooding. Inland flooding was caused by localized torrential rain which could not be drained by gravity due to high water stage in the river. Inland flooding could be prevented by: storm drain, drainage main, open canals and ditches. Interior stage was estimated from inundation mapping considering no damage to Agricultural land by using inland flood protection (Table 4.7).

Table 4-3 External and Internal water level in proposed Dyke

Exterior Stage (m)	Interior stage (m)
2039.60	2038.9
2039.70	2038.9
2039.80	2038.9
2039.90	2038.9
2040.00	2038.9
2040.10	2038.9
2040.20	2038.9
2040.30	2038.9
2040.40	2038.9
2040.50	2038.9

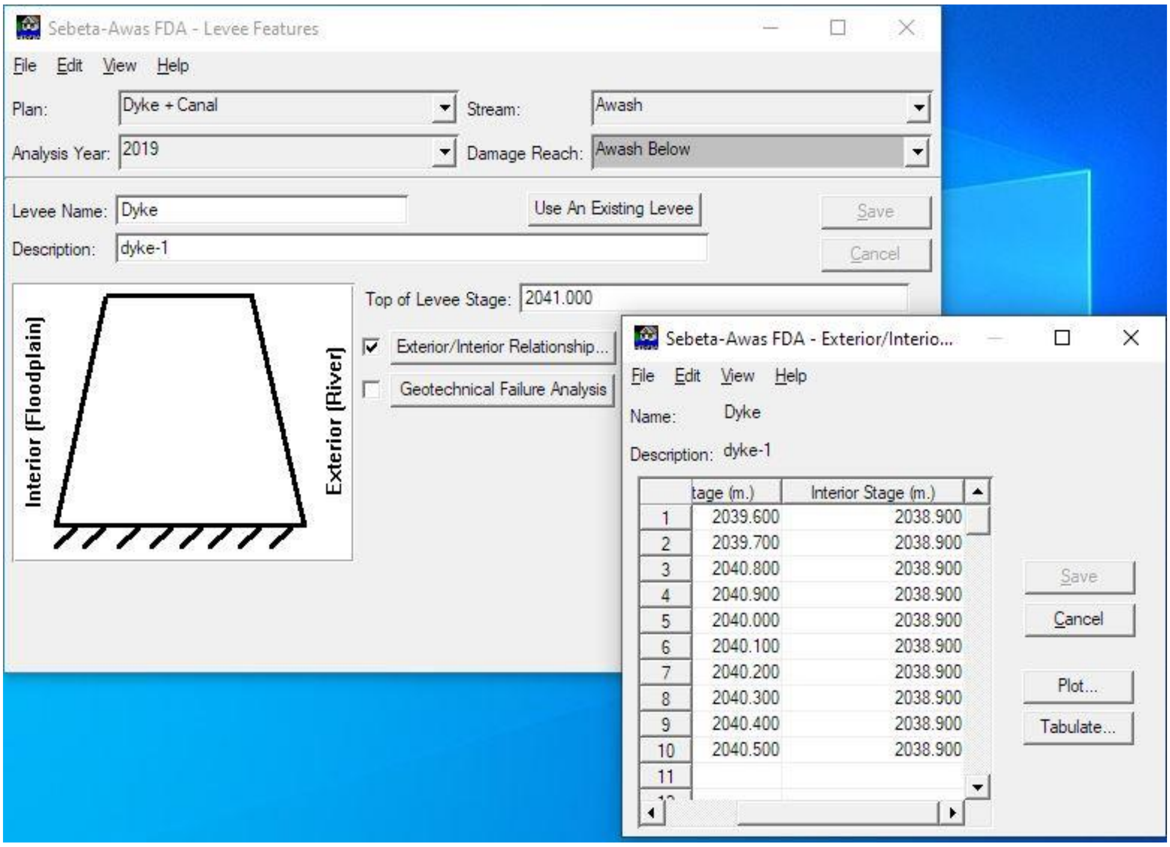

Figure 4-3 Dyke Assignment to Canal in Model

4.4.3 River and Canal Layout

If there is a problem on the existing land use and flow disruption because of sharp meandering, the cut of channel shall be considered [9]. Several route set by combining the portion of existing river use and the portion new river excavation or proposed canal layout. TIN (Triangulated Irregular Network) was prepared for GeoRAS in ArcMap, in order to locate geographically both river and canal. The layout of canal and centerline of river were planned by HEC-GeoRas shown in figure 4.4 below.

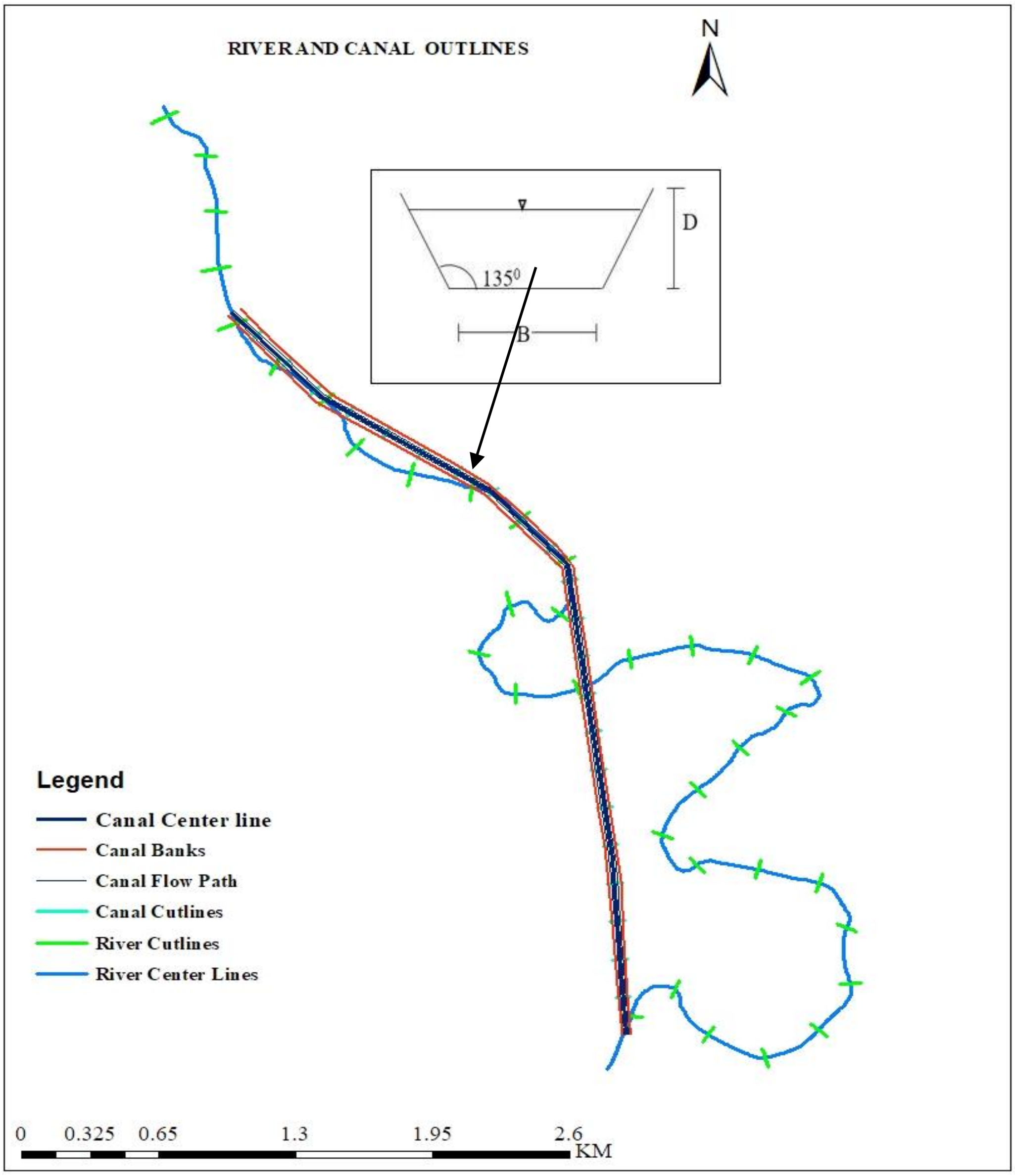

Figure 4-4 River and Canal Layout

4.5 Result of Steady Flow Analysis

4.5.1 River Cross Section

River cross sections were taken on field survey at each 100m along a river for about 10 km. Also, River cross section was interpolated in HEC-RAS model between each station or at 50m.

4.5.2 Boundary Condition Estimation

For this thesis normal depth had been selected for boundary condition. A normal depth was calculated from upstream and downstream bed slope of the channel bottom. Expected flow type in river was subcritical which characterized by high depth low velocity. For subcritical flow boundary condition was provided at downstream end only. Natural river boundary condition the last (600m) riverbed elevation at end of meandering lowered by (0.20m), which mean it has (0.00033) slope. In non-silting and non-scoring Canal mixed (subcritical and super critical) flow type was expected. For mixed flow type boundary condition must be provided at both upstream and downstream end. For canal, the boundary condition was determined by equation (3.5.1) and it has (0.000221) slope at both upstream and downstream end.

4.5.3 Manning Roughness Preparation

Manning number for was river system determined by studying Channel and flood plain condition (Table 3.6 and Table 3.7 and Appendix B figure 1 to 5). The study area flood plain and channel condition with their corresponding Manning roughness by using equation 3.6.1 were summarized below in table 4.4 and table 4.5.

Table 4-4 Manning Roughness Estimation for River (Channel)

Manning roughness (n)	Channel Condition	Recommended	Estimated (n) for Channel
Manning of natural soil (n_b)	Natural bare soil	0.025	0.025
Degree of Irregularities (n1)	Moderate	0.006-0.01	0.008
Variation in Channel cross section (n2	Frequent cross section variation	0.01-0.015	0.0125
Obstruction (n3)	Negligible	0.00-0.004	0.002
Amount of Vegetation (n4)	Small Medium Large	0.002-0.01 0.01-0.025 0.025-0.05	0.006 0.0175 0.0375 Average = 0.023
Meandering Effect (m)	Meandering effect. Ratio of river length to vector distance is more than (1.5) i.e.(10551m) to (5723m).	1.3	1.3
Equivalent (n)	-	-	0.08814

Table 4-5 Manning Roughness Estimation for Flood Plain

Manning Roughness	Flood plain Condition		Estimated (n) for flood plain
Manning of natural soil	Natural bare soil	0.025	0.025
Degree of Irregularities (n1):	Flood plain highly irregular	0.011-0.020	0.0155
Variation in Channel cross section (n2)	Is not measurable,	-	Therefore, not applicable
Obstruction (n3)	Sever type flood plain	0.02-0.030	0.025
Amount of Vegetation (n4):	Small Medium Large Very large Extreme	0.001-0.01 0.0111-0.025 0.025-0.05 0.05-0.1 0.01-0.2	0.0055 0.08 0.0375 0.075 0.15 Average = 0.0572
Meandering Effect (m)	Has value 1 not applicable	1	1
Equivalent (n)	-	-	0.1227

4.5.4 Water Surface Profile

The water surface profiles for different return periods, i.e. 2, 5, 10, 25, 50, 100, 250 and 500 years were computed by HEC-RAS shown in figure below. Water surface profile varies from 2046m to 2048m above sea level in a river. Water surface were limited below 2041m by constructing Canal (Appendix B Figure 9 to 10).

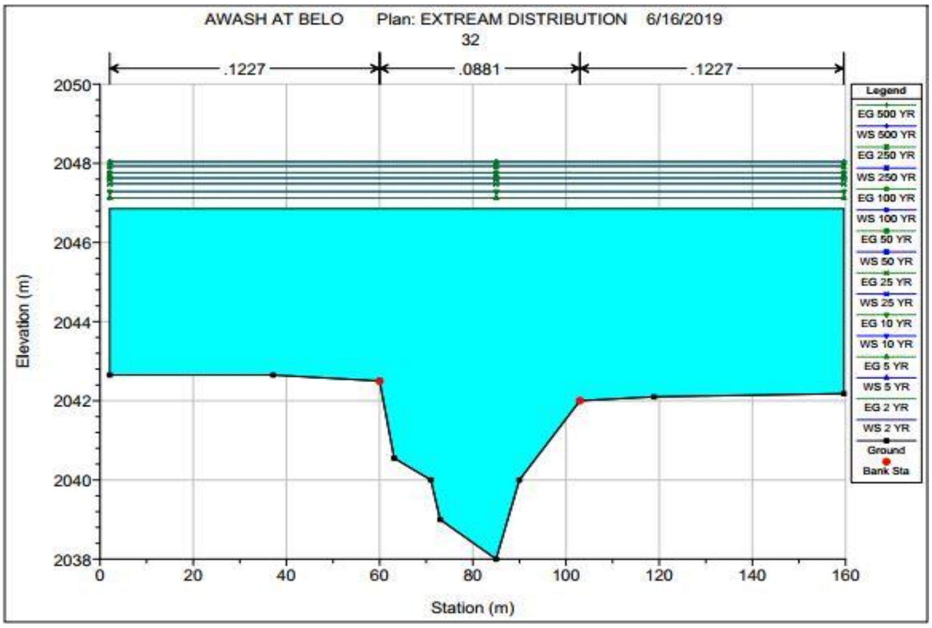

Figure 4-5 Water Surface Cross Sectional View in Channel

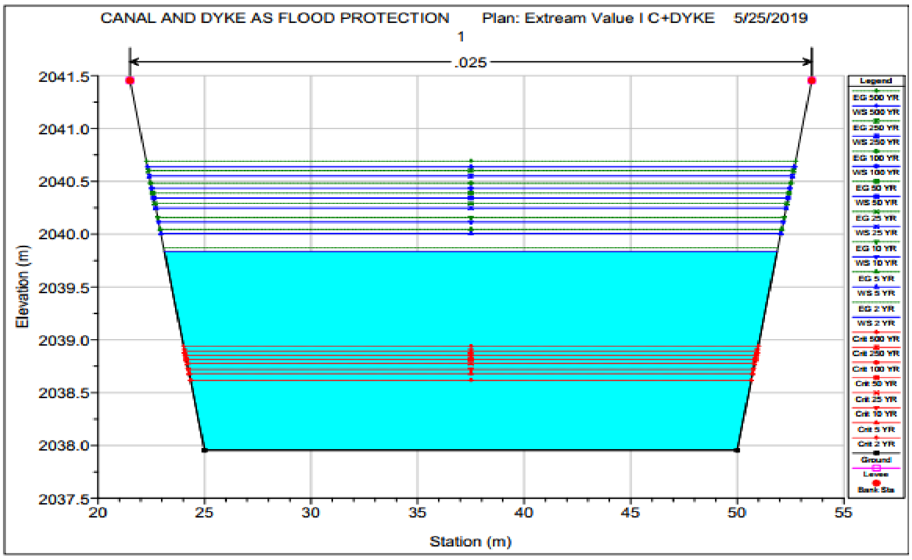

Figure 4-6 Water Surface Cross Sectional View in Canal

4.6 Flood Damage Analysis

4.6.1 Structural Occupancy Type Data Preparation

A structure occupancy type describes a class of structures which same average economic damage. Data entered for a structure occupancy type is applied to all the structures assigned to that structure occupancy type. Based on ground level variation all structures were categorized into Village 1 and Village 2. Ground level for total 492 houses were estimated from 1m contour interval tabulated in table below.

Table 4-6 Ground Level Estimation for Structural Occupancy

Type of Occupancy	Number of Structure	Structure ground Level Interval(m)	Average ground level(m)	Ground level Standard deviation(m)
Village 1	307	2045-2050	2047.5	3.56
Village 2	185	2048-2049	2048.5	0.49

The standard deviation in meters is the uncertainty in the first-floor stage estimate of a particular structure occupancy type. Theses value were estimated by counting structures (houses) between consecutive given major contour interval.

Figure 4-7 Contour Used for Structure Ground Level and its Standard Deviation Estimation.

4.6.1.1 *Structural Damage Estimate*

It is the Cost of maintenances of badly damage mad hut and silt clearing. Cost of structures damaged were estimated by randomly selecting structure (house) and registering damage in current USA dollar. Average structural damage of 9 house was estimated to 85.7 $. Data are used to compute expected and equivalent Annual damage in HEC-FDA.

4.6.1.2 *Content Cost Estimate*

Cost of damaged family food, animal food and house material perished by flood were estimated to 717.2$. Data were collected from Sebata-Awas woreda emergency preparedness office tabulated in (Appendix A Table 13).

4.6.1.3 *Other Cost*

Other cost is a damage happen to agricultural land and land scape. The figure below shows that damped (ponded) water behind Addis Ababa to Jimma road. The gray matter (ponded water) shown on figure gradually increased to fill cultivated land and displace people of Awash Belo and Bonde located below.

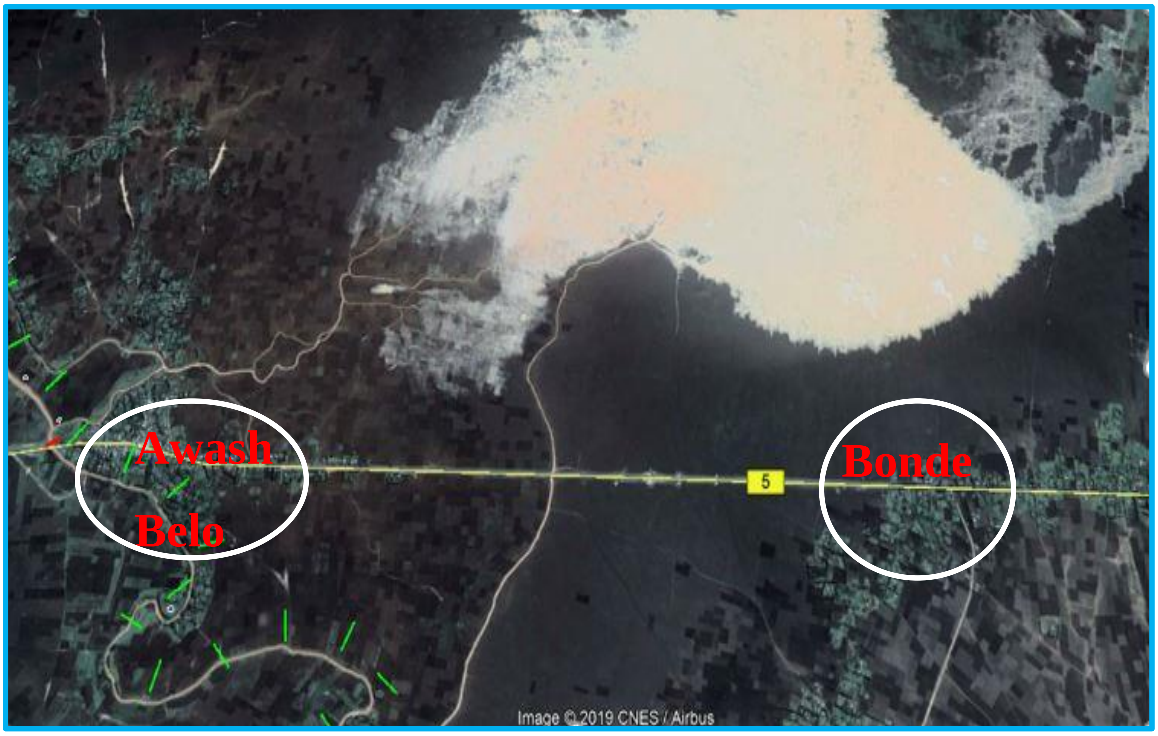

Figure 4-8 Water ponded (Damped) on Agricultural land

Damage to agricultural land was estimated from amount of land inundated in hectare by equation 3.6.2 with information tabulated in (Appendix A Table 12). It is observed that the flood levels for dry season, peak of 2-years, 5-years ,10-years, 25-years ,50-years 100-years, 250-varies and 500-years vary from 2039m to 2048m above sea level.

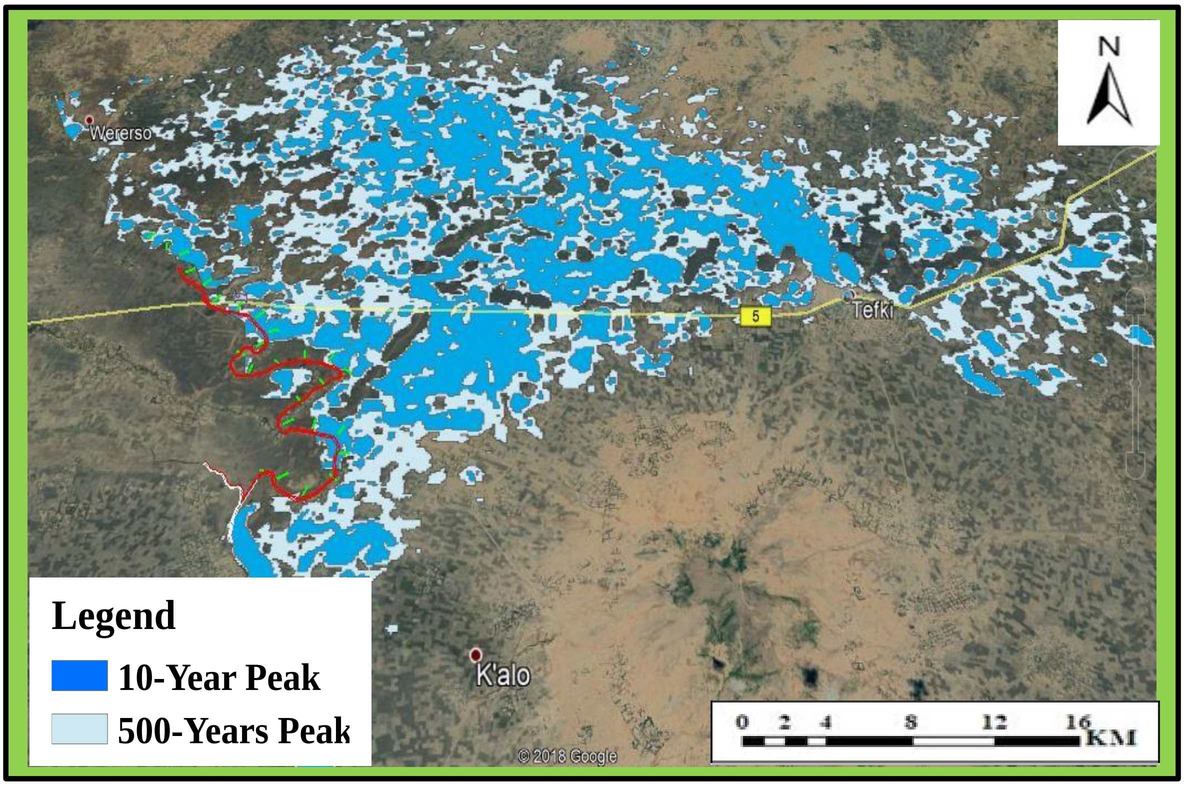

Figure 4-9 Flood Inundation Map for Different Return Period Peak Discharges

The area was inundated by 10-year and 500-year return period nearly same. This shows that flood was occurred due to average gradient of flood plain not due to peak flood. Amount of area have been submerged below given elevation above sea level was estimated by spatial analyst tool in ArcMap. Cultivated land loss associated with inundated land estimated by equation (3.6.2) were summarized below in table 4.7.

Table 4-7 Cultivated land damage (Other cost) Associated with Flooded area

Elevation(m)	Area (HA)	Damage ($)	% Damage ($)
2039	483.7	0.0	0.0
2040	666.8	1435605.1	8.7
2041	1006.6	2167001.5	13.1
2042	1377.8	2966244.3	17.9
2043	1861.6	4007664.5	24.2
2044	2475.8	5330003.0	32.3
2045	3237.6	6970092.0	42.2
2046	5269.9	11345316.6	68.6
2047	5269.9	11345316.6	68.6
2048	6441.9	13868427.7	83.9
2049	7677.6	16528753.5	100.0

4.6.1.4 *Content to Structure Value Ratio*

The content to structure value ratio is the numeric value, that represents the content value divided by the structure value for a particular structure occupancy type. It is used to estimate total content value if the structure inventory does not include content value information. In this study, for both structural occupancy type 8.4 was estimated.

4.6.1.5 *Other to Structure Value Ratio*

The other to structure value ratio is the numeric value, that represents the maximum other value divided by the maximum structure value for a particular structure occupancy type. Damage estimated in agricultural land at 2049m above sea level was about 16528753.5$ (Table 4.7). Average damage per 492 households was 33595 $. From this data, Other to structure ratio for both structural occupancies was 391.9.

4.6.1.6 *Depth Percent Damage Estimation*

Depth-percent damage functions define the percent of the structure damaged for a range of flood stages at a structure. The percent-damage is multiplied by the structure value to get a unique depth-damage function at the structure. The zero depth is assumed to coincide with stage (elevation) of the first floor. Structural depth damage was estimated from height of structure. Average height of house above the ground measured to 3.5m. Cost of maintaining badly damaged house and silt clearing directly increasing with height of structure. Content depth percent damage was estimated by measuring height and position of damage value in house (Appendix B figure 6 to 8). Other depth percent damage was estimated by spatial analysis tool in ArcMap. Summary of percent depth damage for both occupancy type was tabulated below.

Table 4-8 Depth Percent Damage for Both Village 1 and Village 2

Structural cost		Content cost		Other cost	
Depth	%Cost	Depth	% Cost	Depth	%Cost
-1	0	-1	0	-7.5	0.00
0	0	0	0	-6.5	8.7
1.5	33	1	33	-5.5	13.1
2.5	66	2	66	-4.5	17.9
3.5	99	3	99	-3.5	24.2
				-2.5	32.2
				-1.5	42.2
				-0.5	68.4
				0.5	83.5
				1.5	95

4.6.2 HEC-FDA Damage Analysis Result

Once computations are finished, HEC-FDA provides three major groups of reports are expected annual damage, Equivalent Annual Damage and the Project performance. These reports display the expected annual damage and equivalent annual damage for the without and with-project conditions for all plans for a selected analysis year

4.6.2.1 *Expected Annual Damage*

EAD is the probability weighted average of all possible peak annual damages, and is also termed the mean. Expected annual damage estimated from damage-exceedance probability function. The numerical integration was necessary because the damage-exceedance probability function is not defined by a continuous analytic function making an analytic integration impossible. Numerical Monte Carlo Simulation summarized in (Appendix B figure 13)

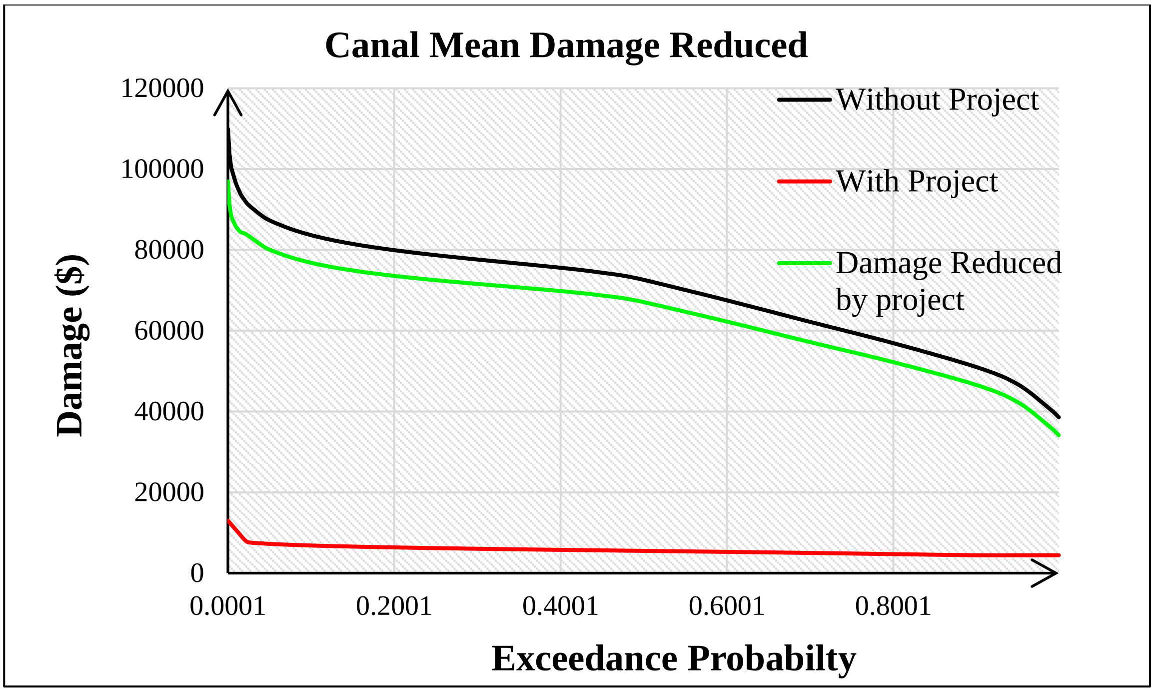

Figure 4-10 Damage Exceedance Probability Used in Computation Expected Annual Damage for Canal

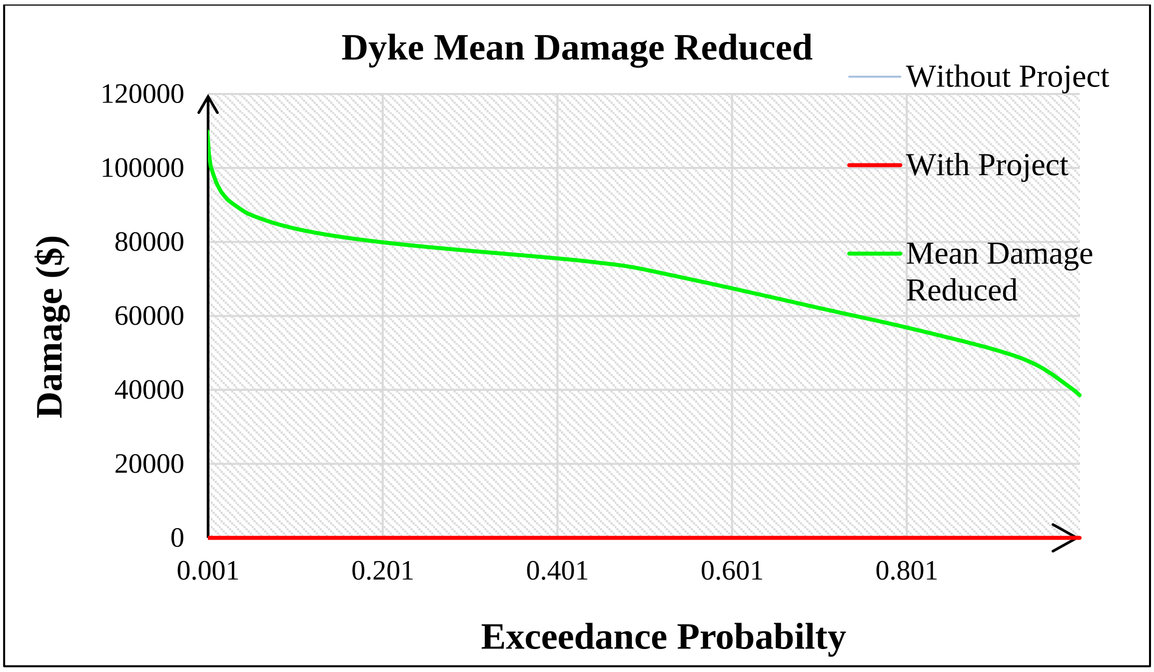

Figure 4-11 Damage Exceedance Probability Used Computation Expected Annual Damage for Dyke

Damage exceedance function was derived by HEC-FDA from imported water surface profile and estimated damage depth function. As explained in (section 3.6.2.1.4) flood damage analysis study was done for 30-years. Result for current year (2019) most likely feature year (2049) was equal. This shows that flood happen due average slope of landscape not due peak flood. Adoption of Canal reduce chance of exceedance of damage below 10,000 $. By integrating Dyke to Canal, HEC-FDA statistics shows that damage exceedance was reduced to zero.

Expected annual damage were estimated to 69423. 8$, 5572.8 and 0$ for river (without project), Canal and integration of Dyke into Canal respectively. It also displays the distribution of expected annual damage reduced by plan (flood protection structure) in terms of the probability that the damage reduced exceeds a value for the probabilities of 0.75, 0.50, and 0.25 computed in table below.

Table 4-9 Expected Annual Damage and Probability Damage Reduced and Distributed for Different flood protection Plan

Plan Name	Plan Description	Expected Annual Damage ($)			Probability Damage Reduced Exceeds Indicated Values		
		Total Without Project	Total with Project	Damage Reduced	0.75	0.50	0.25
Without	Without project condition Default	69423.8	69423.8	0.0	0.00	0.00	0.00
Canal	Canal as Protection	69423.8	5572.8	63851	62487.5	63882.1	65274.5
Dyke & Canal	Apply Dyke on Canal	69423.77	0.00	69423.8	67477.2	69424.8	71378.2

4.6.2.2 *Equivalent Annual Damage*

The flood damage associate with a plan calculated in average equivalent terms. The procedure discount the expected annual damage stream to beginning of period of analysis or base year. Future year damage values are linearly interpreted between the base and most likely future year conditions and assumed constant from most likely future year to the end of analysis year. The analysis period (project life) is the period of time over which the plan has significant beneficial or adverse effects. It is normally 50 years and is not to exceed 100 years [19]. For 50-years analysis period and 7% Ethiopian commercial bank interest rate expected annual damage not changed or expected and equivalent annual damage are equal. This shows that flood level is not much different for 30-years and 50-years discharges.

Table 4-10 Equivalent Annual Damage Reduced and Distributed by Plans (Damage in $'s) Analysis.

Plan Name	Plan Description	Equivalent Annual Damage			Probability Damage Reduced Exceeds Indicated Values		
		Total Without Project	Total with Project	Damage Reduced	0.75	0.50	0.25
Without	Without project condition Default	69423.8	69423.8	0.00	0.00	0.00	0.00
Canal	Canal as Protection	69423.8	5572.8	63851	62487.5	63882.1	65274.5
Dyke & Canal	Apply Dyke on Canal	69423.8	0.00	69423.8	67477.2	69424.8	71378.2

4.6.2.3 *Project performance*

The project performance reports display information about the hydrologic and hydraulic performance of a flood protection structure plan. One of objective flood protection structure planning is identifying target stage.

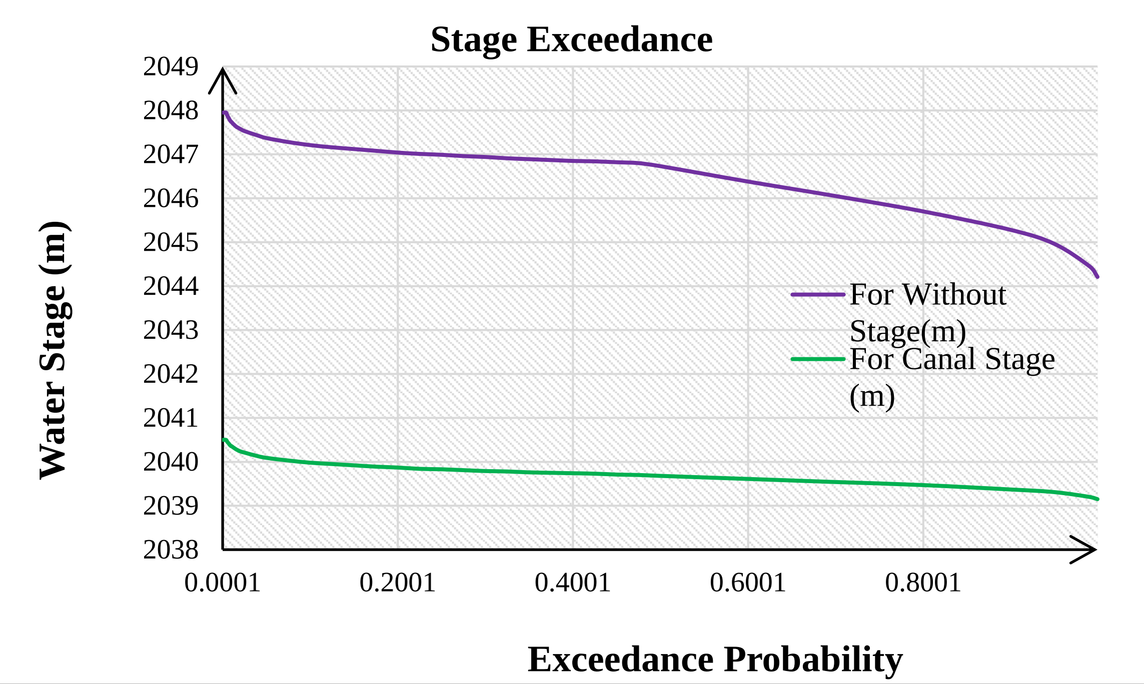

Figure 4-12 Water Stage Exceedance for Different Probability for River and Canal

The target stage is based on an estimate of the stage at which significant damage begins. By default, the criterion is set to 5% of the total damage for the 1% flood event (Appendix: B figure:12). Stage corresponding this predefined event were estimated in table below.

Table 4-11 Target Stage for (5%) Residual Damage (0.01) Exceedance of Peak Discharges

Plan	Stream Name	Damage Reach Name	Target Stage
Without	Awash	Awash Below	2043.40
Canal	Awash	Awash Below	2043.40
Dyke + Canal	Awash	Awash Below	2041.00 L

Long-Term Risk is probability of the target stage being exceeded in a 10-years, 30-years, and 50- years period. Estimated long term risk was tabulated in table below.

Table 4-12 Long Term Risk of Target Stages

Plan	Target Stage	Target Stage Annual Exceedance Probability		Long-Term Risk (years)		
			Expected	10	30	50
Without	2043.40	0.9990	0.9977	1.0000	1.0000	1.0000
Canal	2043.40	0.0001	0.0000	0.0000	0.0000	0.0000
Dyke + Canal	2041.00 L	0.0001	0.0004	0.0043	0.0128	0.0213

Conditional Non-Exceedance Probability by Events is the chance of containing the specific 0.10, 0.04, 0.02, 0.01, 0.004, and 0.002 exceedance probability event within the target stage, should that event occur.

Table 4-13 Conditional non-exceedance probability of target stages

Plan	Target Stage	Conditional Non-Exceedance Probability by Events					
		10%	4%	2%	1%	.4%	.2%
Without	2043.4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Canal	2043.4	1.0000	0.9987	0.9750	0.8829	0.6279	0.4050
Dyke + Canal	2041. L	1.0000	0.9987	0.9751	0.8830	0.6277	0.4052

CHAPTER 5 CONCLUSIONS AND RECCOMENDATIONS

5.1 Conculusion

This thesis presents a systematic approach in flood planning and flood damage management. The major model used in this method ArcGIS, HEC-GEORAS, HEC-RAS, HEC-FDA, Google Earth and Easy Curve fit.

Flood plain inundation mapping, one dimensional steady flow analysis and flood damage forecasting and flood plan evaluation is powerful tools to provide more efficient, effective and standardized results and save times and resources. HEC-FDA provide a new perspective for flood damage estimation and flood structure evaluation

Based on HEC-FDA output, flood protection plan selection for this particular Geographical area the appropriate plan is Canal, combination of canal and dyke and inland flood protection structure. Flood damage forecasting and flood plan evaluation can help decision maker to identify appropriate and comfortable flood protection plan.

The study also assessed flood protection plan with relation to the return period of floods and their water depth. Expected annul damage and Equivalent Annual damage shows by providing canal, combination of Canal and Dyke, inland flood protection structure, etc., we can reduce a lot of damage loss. Without flood protection plan, expected damage annually is about 69423.77 $. By using single canal, we reduce silt problem, disaster to cultivated land, evacuating and resettlement of many household every year. Currently no flood protection plan constructed there. With increasing exceedance of damage-induce flood aggravated by climate change flood protection plan is required.

5.2 Recommendation

This study was conducted under major constraint of limited data availability. Therefore the following recommendation are made for the further studies in future.

- **Flow data** there are many daily, monthly and yearly data are missing. Flow data have not recorded since 2012. This require attention for further future study.
- **Economic depth percent** in country like USA, Economic depth function that represent every flood susceptible area developed. Our country has no such data. This also require attention of higher education institution.
- **Quality Study** this study done under limited budget and time. Quality of prediction increased or decreased based on time budget allocated for study.

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APPENDIX A TABLES

Table 1. Observed Flow at Belo Gauge Station Source, (Ministry of Water, Irrigation and Electricity in Cumecs)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1983.0	-	-	-	-	-	-	-	-	-	-	-	-
1984.0	-	-	-	-	-	-	-	-	-	-	-	-
1985.0	-	-	-	-	-	-	-	-	-	-	-	-
1986.0	-	-	-	-	-	-	-	-	-	8.8	1.4	0.7
1987.0	0.5	0.4	7.3	12.9	15.0	20.8	27.8	35.6	23.9	6.3	1.3	0.8
1988.0	0.6	1.0	0.7	1.3	0.6	5.3	33.5	37.8	36.0	21.6	2.5	1.2
1989.0	1.0	2.5	3.3	5.8	2.0	4.0	37.8	37.4	37.8	13.4	2.4	3.0
1990.0	1.1	4.5	7.8	7.5	1.4	9.2	33.3	36.0	36.4	19.7	2.3	1.2
1991.0	-	1.2	3.2	0.9	1.7	6.5	33.7	37.7	37.8	10.0	1.8	1.1
1992.0	1.0	2.9	1.8	7.5	3.6	13.5	29.6	38.4	35.8	25.7	3.0	1.5
1993.0	-	1.9	1.1	13.1	16.3	18.0	35.2	-	-	-	-	1.9
1994.0	1.3	0.9	1.3	1.6	2.5	10.3	34.3	35.4	38.5	19.4	2.9	1.4
1995.0	1.0	0.9	1.0	9.0	4.7	10.9	32.9	35.6	34.5	8.4	1.7	1.2
1996.0	2.4	0.8	8.2	11.2	9.9	24.8	36.9	39.5	34.5	15.5	2.4	1.4
1997.0	1.9	1.4	0.8	1.8	1.1	12.1	34.4	35.2	22.0	11.5	7.7	1.7
1998.0	2.7	0.6	7.0	4.8	5.0	28.7	49.3	-	49.5	36.1	5.0	1.9
1999.0	1.5	1.8	1.3	0.6	2.8	11.0	45.9	-	45.4	44.3	5.2	1.3
2000.0	0.7	0.5	0.4	1.3	2.8	10.5	34.9	49.9	42.8	41.4	5.6	1.6
2001.0	-	1.0	2.4	2.3	4.1	29.1	-	-	-	-	-	-
2002.0	2.9	0.9	0.9	1.4	0.8	-	-	-	-	-	-	-
2003.0	-	-	0.9	6.1	1.0	8.5	-	-	47.2	22.5	1.0	0.6
2004.0	0.7	0.5	0.7	7.1	3.6	28.9	40.5	48.3	47.1	13.5	1.4	0.7
2005.0	0.3	0.0	0.8	7.8	8.2	23.4	48.3	-	42.7	18.3	1.3	0.7
2006.0	0.4	1.2	2.5	11.1	14.1	15.4	-	-	-	11.4	2.5	-
2007.0	-	-	-	-	8.0	43.8	-	-	50.0	32.5	1.6	0.7
2008.0	0.5	0.4	0.4	2.2	2.9	12.0	49.3	52.7	52.9	9.4	13.5	1.0

2009.0	0.8	0.7	0.3	0.5	0.3	4.7	43.2	51.3	48.6	-	-	-
2010.0	0.5	2.0	2.8	7.7	8.0	29.8	49.9	48.5	56.6	15.3	1.6	0.8
2011.0	0.9	0.3	1.0	0.3	4.8	21.4	48.6	51.5	46.6	17.8	2.9	0.9
2012.0	0.2	0.1	0.2	1.4	4.1	8.9	46.0	51.8	-	-	-	-
2013.0	-	-	-	-	-	-	-	-	-	-	-	-

Table 2. Maximum Annual daily instantaneous Flow Used in Filling missed flow data studies

year	1987	1988	1989	1990	1991	1992	1994	1996
Flow in(cum)	35.6	37.8	37.8	36.4	37.8	38.4	38.5	39.5
year	1997	2000	2004	2008	2009	2010	2011	2012
Flow in (cum)	35.2	49.9	48.3	52.9	51.3	56.6	51.5	51.8

Table 3 Maximum Annual Daily Instantaneous Flow Used in Flood Frequency Analysis

year	1983	1984	1985	1986	1987	1988	1989	1990
Flow in (cum)	31.6	32.4	33.2	33.9	35.6	37.8	37.8	36.4
year	1991	1992	1993	1994	1995	1996	1997	1998
Flow in (cum)	37.8	38.4	39.4	38.5	40.9	39.5	35.2	43.2
year	1999	2000	2001	2002	2003	2004	2005	2006
Flow in (cum)	44.0	49.9	45.5	46.3	47.1	48.3	48.6	49.4
year	2007	2008	2009	2010	2011	2012	2013	2014
Flow in (cum)	50.2	52.9	51.3	56.6	51.5	51.8	54.8	55.6

Table 4 Independent and Stationary Test

	xi	xi+1	xi*xi+1	S2	S3	S4
1	31.6	32.4	1024.8	147.9	-1888.9	24950.7
2	32.4	33.2	1074.9	129.5	-1546.3	19107.1
3	33.2	33.9	1126.2	112.2	-1247.8	14355.6
4	33.9	35.6	1207.4	96.2	-990.5	10550.8
5	35.6	37.8	1345.8	66.6	-571.0	5061.7
6	37.8	37.8	1431.9	34.2	-210.4	1337.4
7	37.8	36.4	1377.0	34.2	-210.4	1337.4
8	36.4	37.8	1377.0	53.7	-412.9	3285.1
9	37.8	38.4	1451.6	34.2	-210.4	1337.4
10	38.4	39.4	1509.7	28.3	-158.3	915.3
11	39.4	38.5	1513.8	18.6	-84.3	394.8
12	38.5	40.9	1573.2	27.2	-149.0	844.1
13	40.9	39.5	1616.2	7.5	-21.7	64.6
14	39.5	35.2	1389.3	17.2	-75.0	338.0
15	35.2	43.2	1519.4	73.6	-662.7	6173.9
16	43.2	44.0	1901.1	0.2	-0.1	0.0
17	44.0	49.9	2193.5	0.2	0.1	0.0
18	49.9	45.5	2270.5	40.5	270.5	1869.4
19	45.5	46.3	2108.6	3.9	8.0	17.0
20	46.3	47.1	2180.1	7.6	21.8	65.2
21	47.1	48.3	2273.1	12.5	46.4	178.1
22	48.3	48.6	2347.7	22.6	113.1	584.3
23	48.6	49.4	2401.9	26.1	139.7	774.4
24	49.4	50.2	2478.2	34.7	214.5	1372.2
25	50.2	52.9	2654.2	44.6	312.2	2263.3
26	52.9	51.3	2713.1	89.4	886.8	9104.3
27	51.3	56.6	2901.4	60.9	499.6	4235.6
28	56.6	51.5	2911.6	173.8	2405.7	34445.5
29	51.5	51.8	2666.9	63.8	535.3	4644.3
30	51.8	54.8	2840.1	69.8	612.0	5552.4
31	54.8	55.6	3045.9	129.6	1548.9	19151.0
32	55.6	31.6	1757.9	148.1	1891.9	25004.3
sum(R)			62183.8	1809.4	1066.8	199315.0

Table 5 Independent and Stationary Test's Summery

S_1	S_2	S_3	S_4	R	$\overline{R}$	Var (R)	t
1395.2	1809.4	1066.8	199315.0	62183.8	62734.6	123718593	-0.05

Table 6 Outlier Test's Summery

Year	Flow (Cumecs)	Ln Flow (natural logarithmic of flow)
1983	31.6	3.5
1984	32.4	3.5
1985	33.2	3.5
1986	33.9	3.5
1987	35.6	3.6
1988	37.8	3.6
1989	37.8	3.6
1990	36.4	3.6
1991	37.8	3.6
1992	38.4	3.6
1993	39.4	3.7
1994	38.5	3.6
1995	40.9	3.7
1996	39.5	3.7
1997	35.2	3.6
1998	43.2	3.8
1999	44.0	3.8
2000	49.9	3.9
2001	45.5	3.8
2002	46.3	3.8
2003	47.1	3.9
2004	48.3	3.9
2005	48.6	3.9
2006	49.4	3.9
2007	50.2	3.9
2008	52.9	4.0

2009	51.3	3.9
2010	56.6	4.0
2011	51.5	3.9
2012	51.8	3.9
2013	54.8	4.0
2014	55.6	4.0
Summary of Variables		
	MEAN	3.8
	STAND.DV	0.2
	K_N	2.6
	Upper boundary	67.6
	Lower Boundary	27.3

Table 7 Probability Weight Moment

i	X(Flow)	α1	α2	α3	α4
1.0	31.6	1.0	1.0	1.0	1.0
2.0	32.4	1.0	0.9	0.9	0.9
3.0	33.2	1.0	0.9	0.8	0.8
4.0	33.9	1.0	0.9	0.8	0.7
5.0	35.6	1.0	0.8	0.7	0.6
6.0	37.8	1.0	0.8	0.7	0.6
7.0	37.8	1.0	0.8	0.6	0.5
8.0	36.4	0.9	0.7	0.5	0.4
9.0	37.8	0.9	0.6	0.5	0.3
10.0	38.4	0.9	0.6	0.4	0.3
11.0	39.4	0.8	0.6	0.4	0.2
12.0	38.5	0.8	0.5	0.3	0.2
13.0	40.9	0.8	0.5	0.3	0.2
14.0	39.5	0.7	0.4	0.2	0.1
15.0	35.2	0.6	0.3	0.2	0.1
16.0	43.2	0.7	0.3	0.2	0.1
17.0	44.0	0.7	0.3	0.1	0.1
18.0	49.9	0.7	0.3	0.1	0.0
19.0	45.5	0.6	0.2	0.1	0.0

20.0	46.3	0.6	0.2	0.1	0.0
21.0	47.1	0.5	0.2	0.1	0.0
22.0	48.3	0.5	0.1	0.0	0.0
23.0	48.6	0.4	0.1	0.0	0.0
24.0	49.4	0.4	0.1	0.0	0.0
25.0	50.2	0.4	0.1	0.0	0.0
26.0	52.9	0.3	0.1	0.0	0.0
27.0	51.3	0.3	0.0	0.0	0.0
28.0	56.6	0.2	0.0	0.0	0.0
29.0	51.5	0.2	0.0	0.0	0.0
30.0	51.8	0.1	0.0	0.0	0.0
31.0	54.8	0.1	0.0	0.0	0.0
32.0	55.6	0.0	0.0	0.0	0.0
sum		19.7	12.4	9.0	7.1

Table 8 Computed Probability weight moment, L-moment L- moment Ratio

α_1	α_2	α_3	α_4	λ_1	λ_2	λ_3	λ_4	τ	τ_3	τ_4
19.7	12.4	9.0	7.1	43.6	-4.3	0.08	0.14	-0.1	-0.02	-0.03

Table 9 τ_3 and τ_4 Relation to predict best fit

Parent Distribution	L Moment Ratio Relationship
1. Uniform	$\tau_3 = 0,\ \tau_4 = 0$
2.Exponential	$\tau_3 = 1/3,\ \tau_4 = 1/6$
3. Gumbel (EV1 (2))	$\tau_3 = 0.1699,\ \tau_4 = 0.1504$
4. Logistic	$\tau_3 = 0,\ \tau_4 = 1/6$
5. Normal	$\tau_3 = 0,\ \tau_4 = 0.1226$
6. Generalized Pareto	$\tau_4 = \tau_3 (1 + 5\tau_3)/(5 + \tau_3)$
7. Generalized Logistic	$\tau_4 = (1 + 5\tau_3)/6$ or $\tau_4 = 0.16667\tau_3 + 0.83333$
8. Generalized Extreme Value	$\tau_4 = 0.10701 + 0.11090\tau_3 + 0.84838\tau_3^2 - 0.06669\tau_3^3 + 0.00567\tau_3^4 - 0.04208\tau_3^5 + 0.03763\tau_3^6$
9. Gamma and Pearson III	$\tau_4 = 0.1223 + 0.30115\tau_3^2 + 0.95812\tau_3^4 - 0.57488\tau_3^6 + 0.19383\tau_3^8$
10. Lognormal (two and three parameters)	$\tau_4 = 0.12282 + 0.77518\tau_3^2 + 0.12279\tau_3^4 - 0.13638\tau_3^6 + 0.11368\tau_3^8$
11. Wakeby lower bounds	$\tau_4 = -0.0737 + 0.14443\tau_3 + 1.03879\tau_3^2 - 0.14602\tau_3^3 + 0.03357\tau_3^4$

Table 10 Goodness Fit Test Easy Fit Program Results

	Distribution	Kolmogorov-Simirnov statics	rank	Anderson Darling statics	rank	Chi square statics	rank
1	Beta	0.1	2	1.2	43	1.0	2
2	Burr	0.2	44	0.6	15	3.7	15
3	Burr (4P)	0.1	29	0.7	21	4.2	30
4	Cauchy	0.2	39	1.2	45	2.8	6
5	Chi-Squared	0.1	8	0.7	33	4.3	35
6	Chi-Squared (2P)	0.1	28	0.7	25	5.9	44
7	Dagum	0.1	24	0.7	31	3.1	11
8	Dagum (4P)	0.5	55	18.2	58	28.4	52
9	Erlang	0.2	41	0.8	36	6.3	47
10	Erlang (3P)	0.1	36	0.8	37	3.9	21
11	Error	0.1	6	0.2	2	3.7	16
12	Error Function	1.0	60	562.8	60	N/A	
13	Exponential	0.5	56	10.1	55	44.7	54
14	Exponential (2P)	0.2	48	2.5	49	3.1	10
15	Fatigue Life	0.1	11	0.7	20	4.1	28
16	Fatigue Life (3P)	0.1	12	0.7	19	4.1	27
17	Frechet	0.2	37	0.8	35	4.0	24
18	Frechet (3P)	0.1	30	0.7	28	3.7	14
19	Gamma	0.1	21	0.6	12	4.1	29
20	Gamma (3P)	0.1	25	0.6	17	3.9	20
21	Gen. Extreme Value	0.1	16	0.5	5	3.6	12
22	Gen. Gamma	0.1	27	0.7	18	4.2	34
23	Gen. Gamma (4P)	0.2	45	4.9	52	N/A	

24	Gen. Pareto	0.1	3	0.2	1	3.0	9
25	Gumbel Max	0.2	38	1.1	42	6.4	48
26	Gumbel Min	0.2	49	1.3	46	6.7	49
27	Hyper secant	0.2	47	1.3	47	8.1	50
28	Inv. Gaussian	0.2	42	0.7	34	2.8	8
29	Inv. Gaussian (3P)	0.2	40	0.7	32	2.8	7
30	Johnson SB	0.1	1	0.3	4	1.0	1
31	Kumaraswamy	0.1	33	1.2	44	2.2	5
32	Laplace	0.2	50	1.8	48	10.4	51
33	Levy	0.6	57	13.6	56	91.3	56
34	Levy (2P)	0.4	54	4.0	50	5.1	38
35	Log-Gamma	0.1	10	0.6	13	4.0	23
36	Log-Logistic	0.1	31	0.7	29	5.2	40
37	Log-Logistic (3P)	0.1	23	0.7	30	5.2	39
38	Log-Pearson 3	0.1	15	0.6	7	4.1	25
39	Logistic	0.2	46	1.0	41	6.1	46
40	Lognormal	0.1	13	0.7	23	4.1	26
41	Lognormal (3P)	0.1	26	0.7	22	4.2	32
42	Nakagami	0.1	34	0.6	16	3.8	18
43	Normal	0.1	35	0.6	11	3.8	19
44	Pareto	0.2	51	4.1	51	5.8	43
45	Pareto 2	0.6	58	15.6	57	49.8	55
46	Pearson 5	0.1	20	0.7	27	5.6	41
47	Pearson 5 (3P)	0.1	22	0.7	24	4.2	31
48	Pearson 6	0.1	19	0.7	26	5.7	42
49	Pearson 6 (4P)	0.3	53	8.0	54	N/A	
50	Pert	0.1	17	0.6	9	4.8	37

51	Power Function	0.1	4	1.0	40	1.4	4
52	Rayleigh	0.3	52	5.4	53	34.0	53
53	Rayleigh (2P)	0.1	14	0.6	10	6.0	45
54	Reciprocal	0.1	7	0.9	39	1.3	3
55	Rice	0.1	9	0.5	6	3.9	22
56	Student's t	1.0	59	224.6	59	59526.0	57
57	Triangular	0.2	43	0.9	38	3.7	13
58	Uniform	0.1	5	0.3	3	3.7	17
59	Weibull	0.1	32	0.6	14	4.7	36
60	Weibull (3P)	0.1	18	0.6	8	4.2	33

Table 11 Flood plain cultivated land (Woreda rural development office)

KEBELE NM	TOTAL (HA)	TEFF	WHEAT	BARLY	CORN	BEAN	PEA	CHICPEA	LENTIL	(VETCH)
A/ Bal'oo	750	35	35	0	0	0	0	450	150	80
W/yekkaa	1560	412	600	40	49	37	12	235	108	67
Bondee	2100	380	979	22	30	50	0	454	115	70
N/Tafki	1225	266	310	0	31	26	0	367	151	74
D/Maanyo	1600	512	873	0	20	25	0	43	82	45
H/Jilaa	1020	220	355	190	15	65	45	24	70	36
G/Guddaa	2060	612	975	75	54	80	34	85	105	40
Boolee	820	180	320	142	13	43	30	45	27	20
SUM	11135	2617	4447	469	212	326	121	1703	808	432

Table 12 Percent of cereals Cultivated and Yield and yield in $, Source (Woreda rural development office and Ethiopian agricultural Market)

Title	% Sowed	100KG/HA	100KG IN $
Teff	24%	25	98.2
Wheat	40%	40	82.1
Barley	4%	30	85.7
Corn	2%	50	64.3
Bean	3%	25	85.7
Pea	1%	25	71.4
Post flood	26.40%	22	83.3

Table 13 Flood Damage Estimated 2017 Summer Flood (Woreda Emergency preparedness office)

Total Households	492
Human Food Damage	2511137 ET BIR
Wear Damage	1838900 ET BIR
Animal Food Damage	3000000 ET BIR
Home Material Damage	2328642 ET BIR

Table14 100-year Water Surface Elevation in Canal

Reach	River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)
UPPER REACH	200	100 YR	67.20	2037.06	2039.55
UPPER REACH	399.9999	100 YR	67.20	2037.11	2039.59
UPPER REACH	599.9999	100 YR	67.20	2037.15	2039.63
UPPER REACH	800	100 YR	67.20	2037.20	2039.68
UPPER REACH	1000	100 YR	67.20	2037.24	2039.72
UPPER REACH	1200	100 YR	67.20	2037.29	2039.77
UPPER REACH	1400	100 YR	67.20	2037.33	2039.81
UPPER REACH	1600	100 YR	67.20	2037.38	2039.86
UPPER REACH	1800	100 YR	67.20	2037.42	2039.90
UPPER REACH	2000	100 YR	67.20	2037.47	2039.95
UPPER REACH	2200	100 YR	67.20	2037.51	2039.99
UPPER REACH	2400	100 YR	67.20	2037.55	2040.03
UPPER REACH	2600	100 YR	67.20	2037.60	2040.08
UPPER REACH	2800	100 YR	67.20	2037.64	2040.12
UPPER REACH	3000	100 YR	67.20	2037.69	2040.17
UPPER REACH	3200	100 YR	67.20	2037.73	2040.21
UPPER REACH	3400	100 YR	67.20	2037.78	2040.26
UPPER REACH	3600	100 YR	67.20	2037.82	2040.30
UPPER REACH	3800	100 YR	67.20	2037.87	2040.35
UPPER REACH	4000	100 YR	67.20	2037.91	2040.39
UPPER REACH	4200	100 YR	67.20	2037.96	2040.43
UPPER REACH	4400	100 YR	67.20	2038.00	2040.48

APPENDIX B FIGURE

Figure 1 Channel and Flood Plain Condition

Figure 2 Channel Condition

Figure 3 Channel Condition

Figure 4 Channel and Flood plain Condition

Figure 5 Flood plain and Channel Condition

Figure 6 Contents (Rise 1.5 m above First floor and 2.5m above ground)

Figure 7 Contents (Human Food 2.30m high above first floor)

Figure 8 Contents (Animal Food 3.2m High)

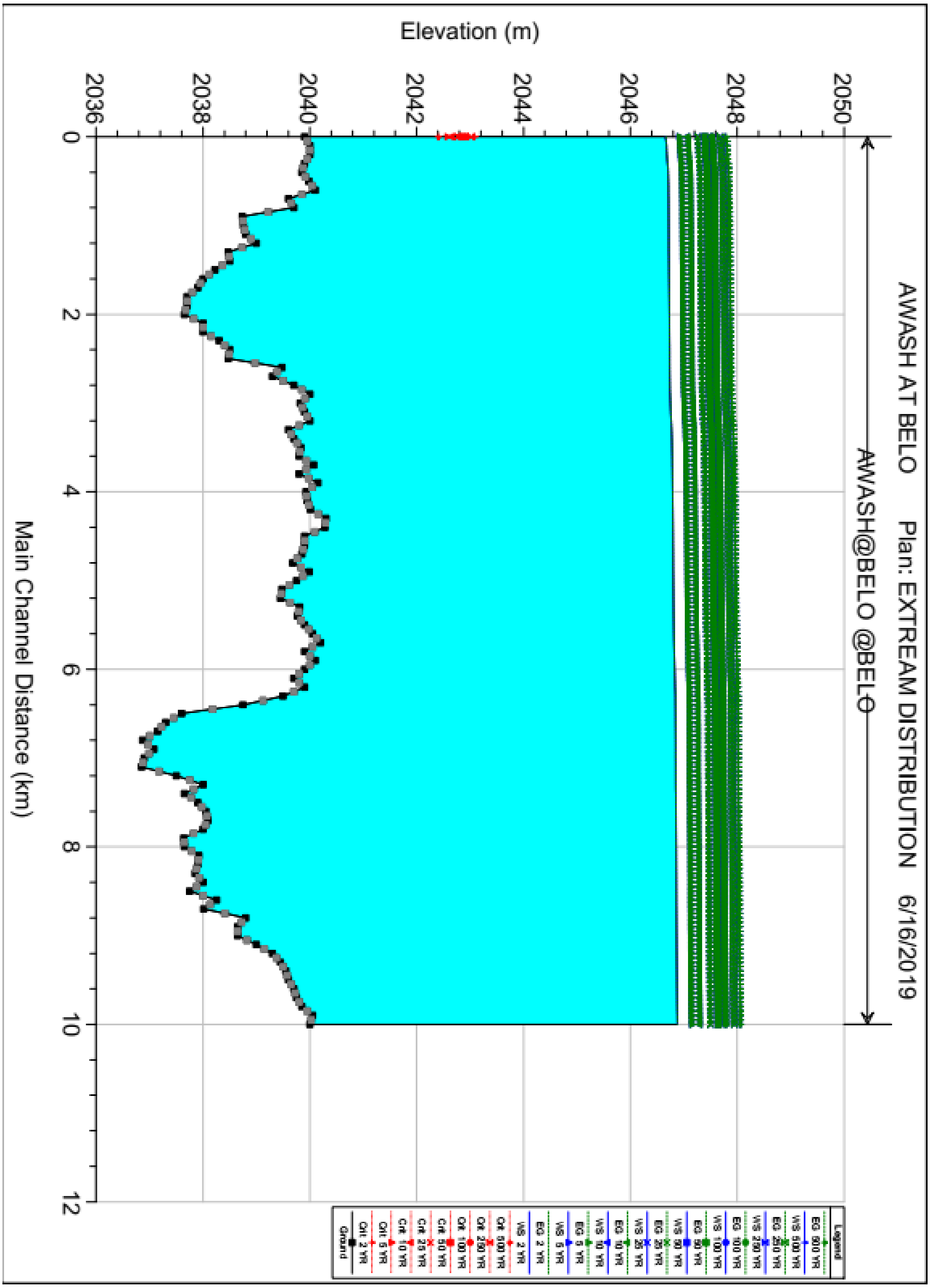


Figure 9 Water surface Profile in Channel

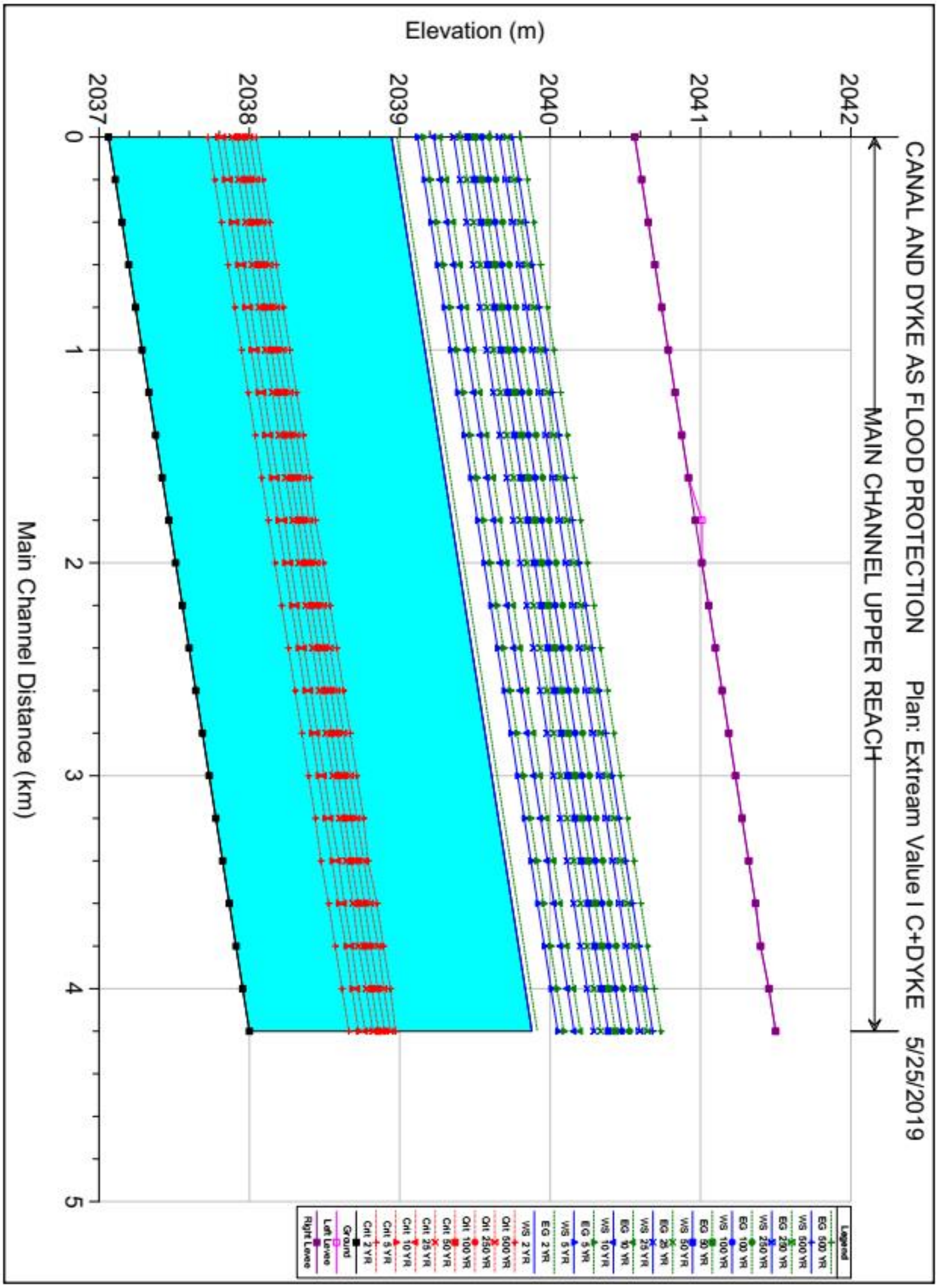


Figure 10 Water surface Profile in Canal

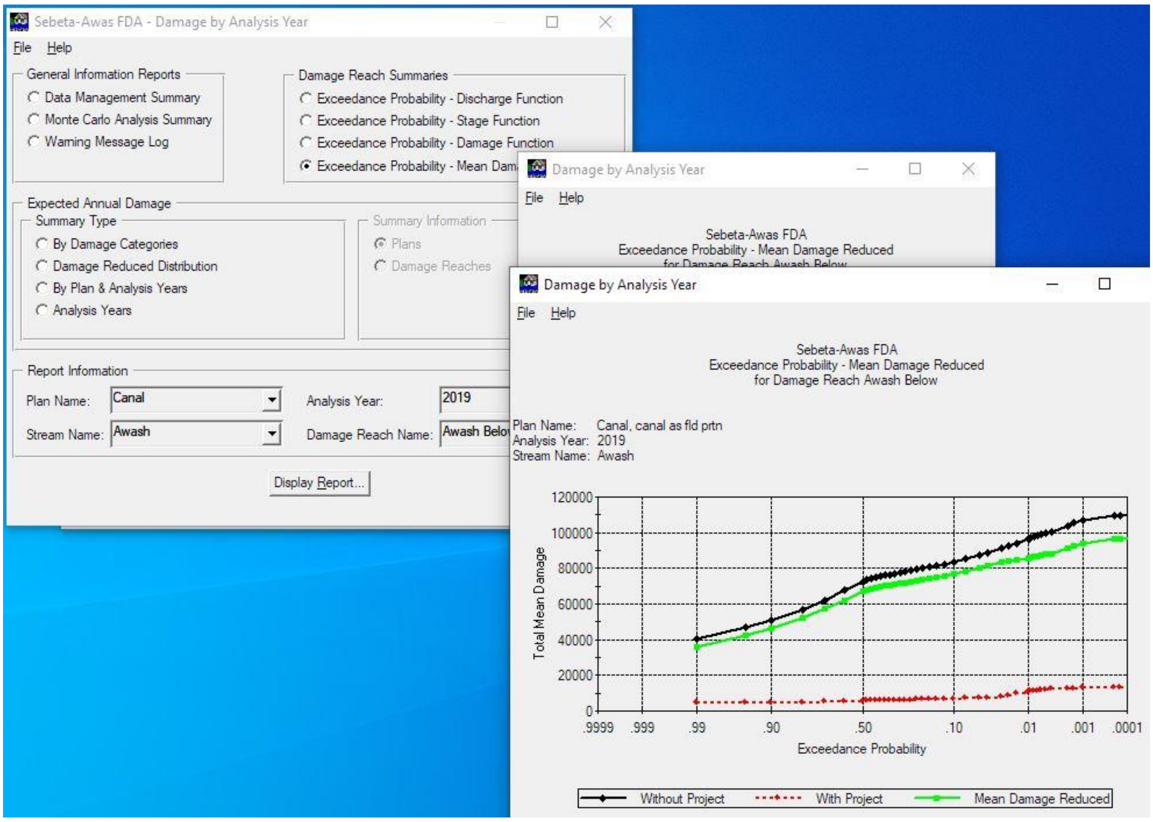


Figure 11 Exceedance Probability Model Out Put Same with Figure 4.11

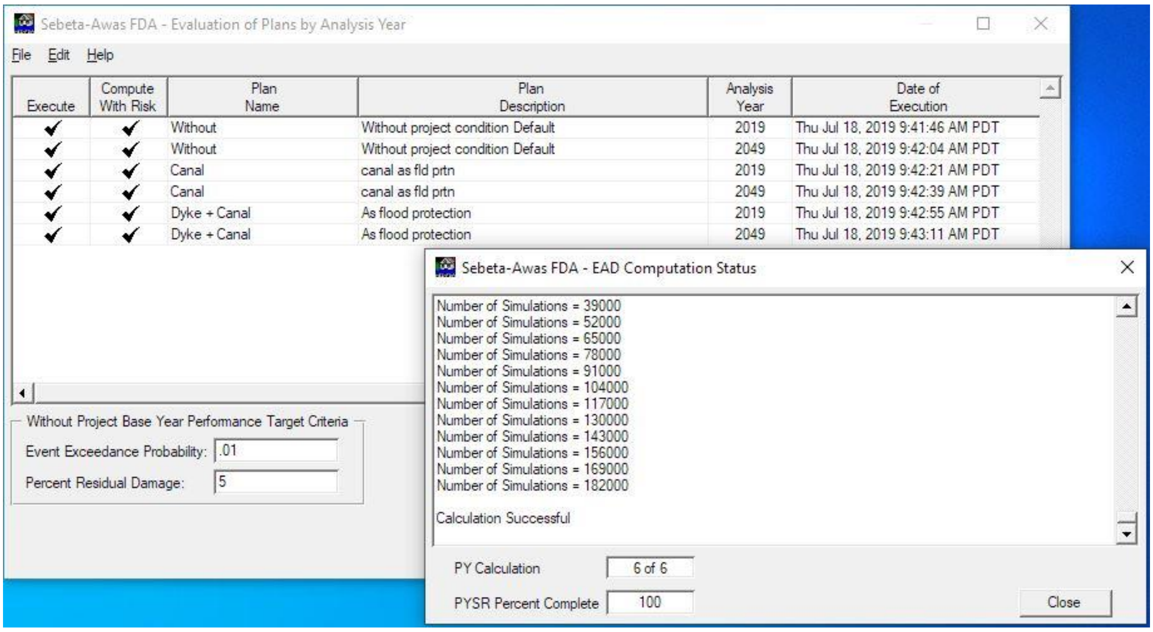


Figure 12 Project performance for 0.01 Exceedance Probability and 5% Damage

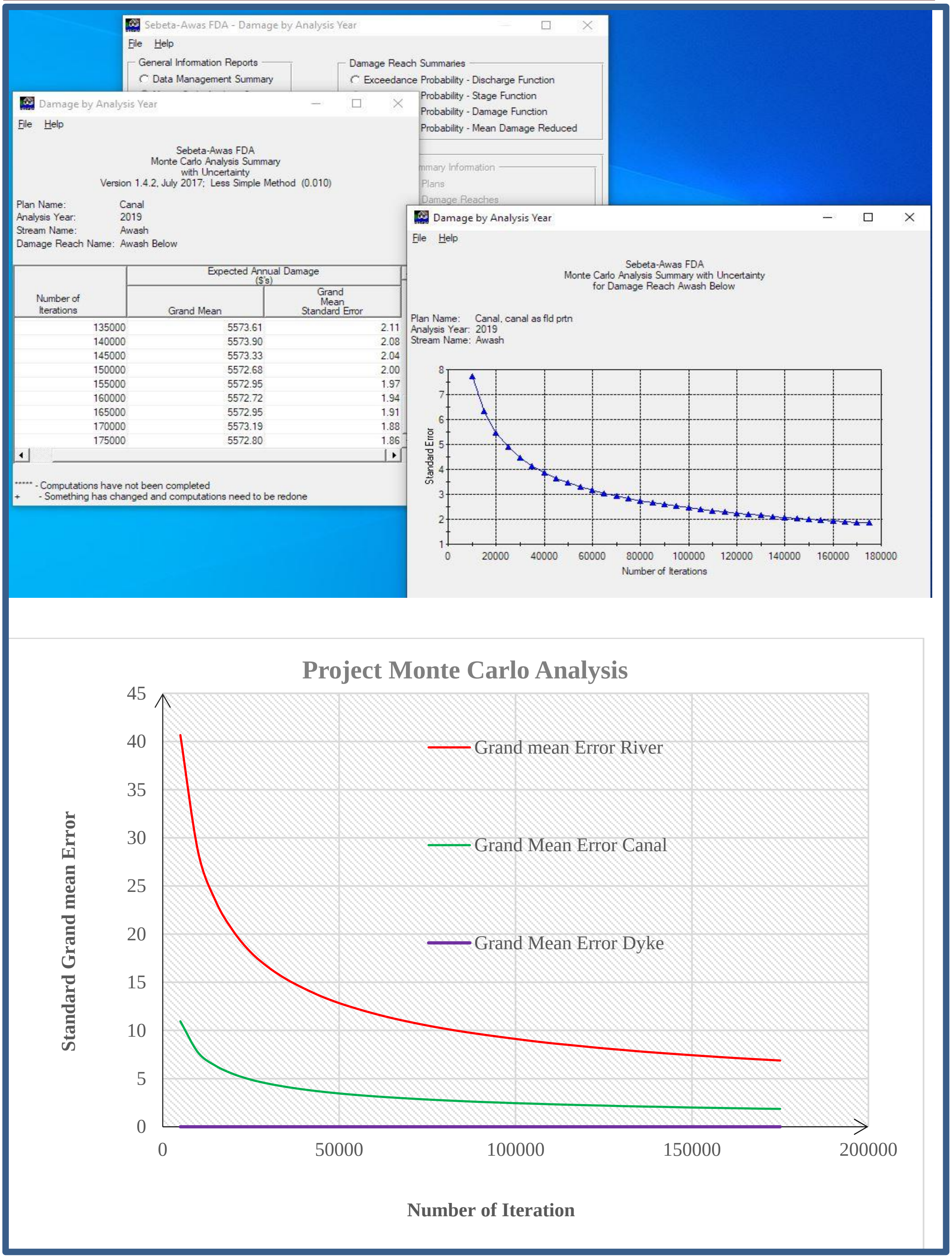
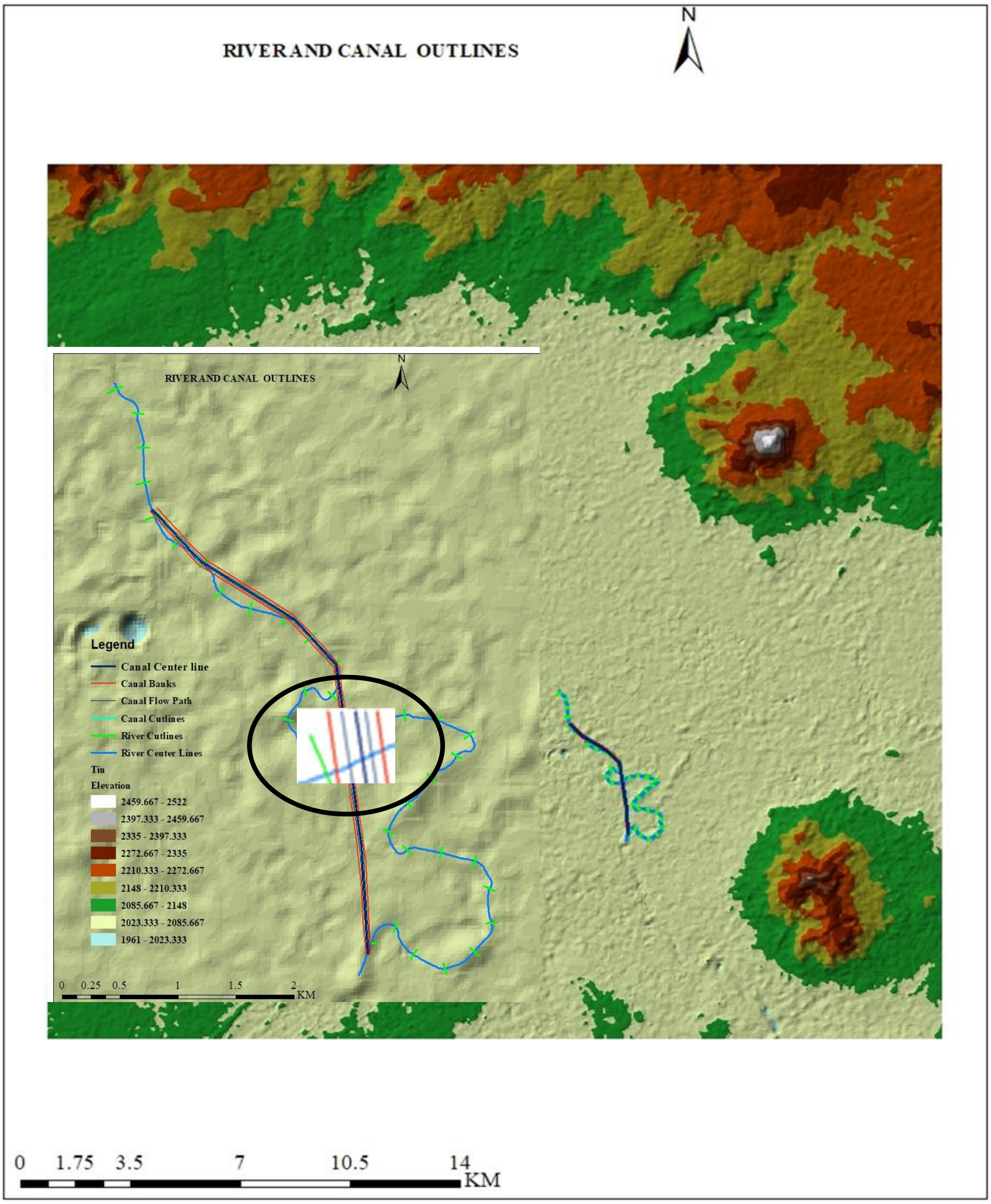
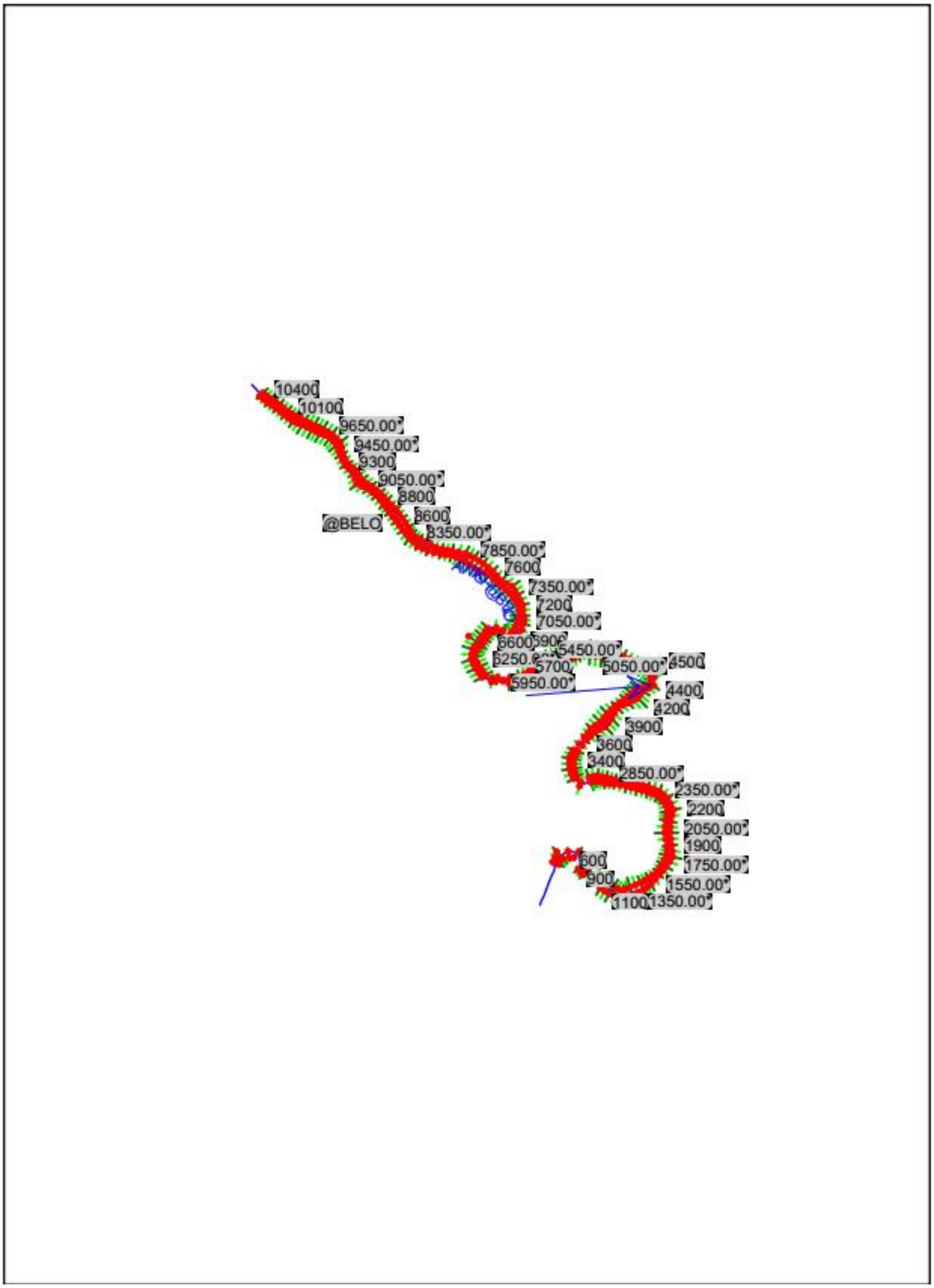


Figure 13 Monte Carlo Analysis Summery

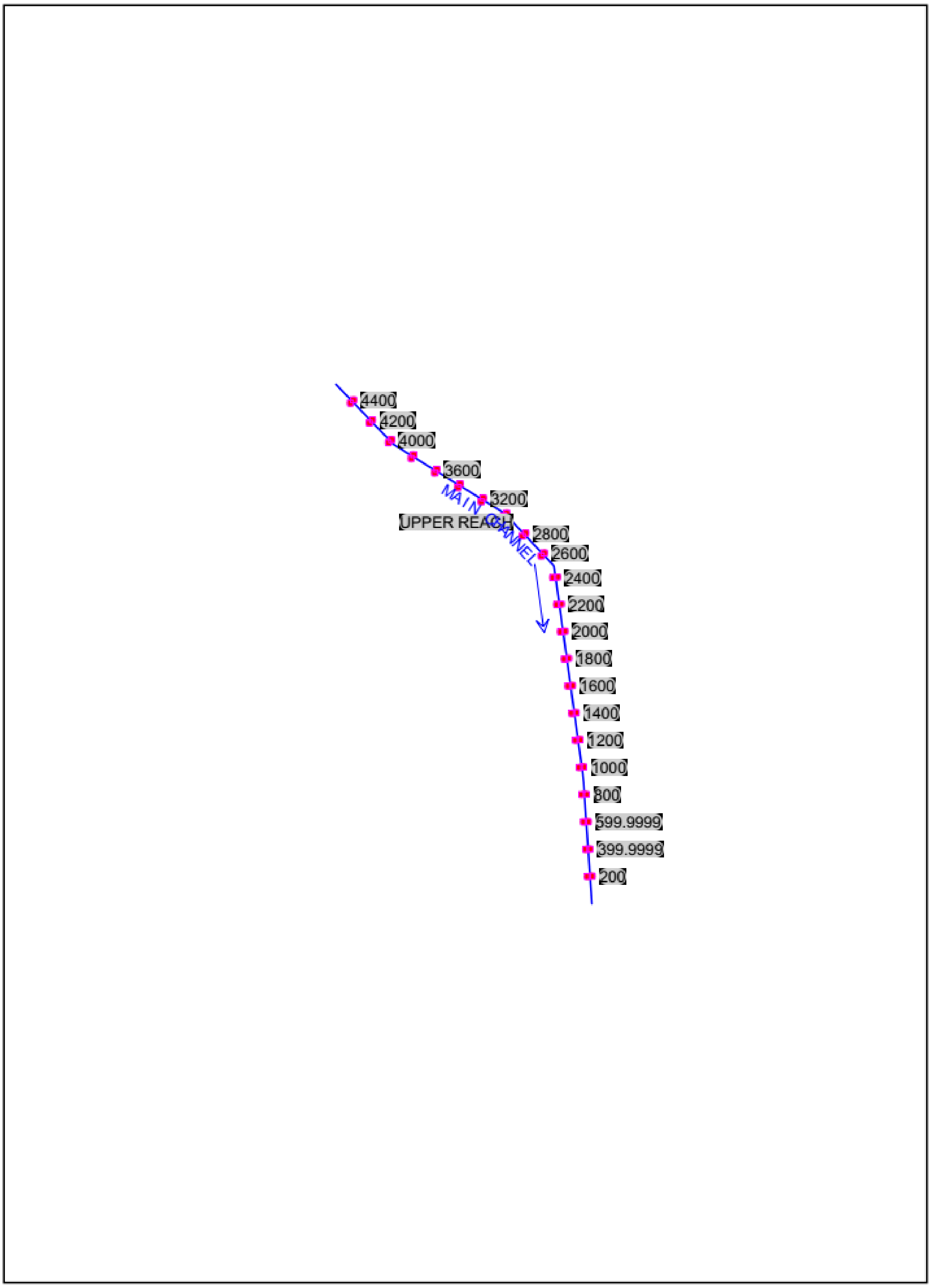
APPENDIX C MAPS



Maps 1 Canal and Channel Outlines



Map 2 HEC - RAS Channel Geometric Outlines



Map 3 HEC-RAS Canal Geometric Outlines