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SCHOOL OF GRADUATE STUDIES

School of Civil and Environmental Engineering

Hydraulic Engineering Stream

Flood Risk Analysis in Illu Floodplain, Upper Awash River Basin, Ethiopia

By

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A Thesis Submitted to the School of Graduate Studies of Addis Ababa University in Partial Fulfillment of the Requirements for the Degree of Master of Science in Hydraulic Engineering.

December, 2015

CERTIFICATION

I, the undersigned, certify that I read and hereby recommend for the acceptance by the Addis Ababa University a dissertation entitled: Flood Risk Analysis in Illu Floodplain, Upper Awash River Basin, Ethiopia in partial fulfillment of a degree of Masters of Science in Hydraulic Engineering Stream.

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Date:---------------------------------

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Date of Defense: **December, 08, 2015**

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ABSTRACT

A flood has negative effect to human beings and properties even if it has positive effect. Because less attention is paid for risk, it should be analyzed and quantified properly. The Upper Awash River and flooding remain inseparable for long time in cultivated area. The objective of this study is to quantify and estimate the risk of flooding on crop production in the study area. The basic assumptions in statistical flood frequency analysis are the independence and stationarity of the data series and that the data come from the same distribution. Homogeneity and stationarity tests at different significance levels were made using Wald-Wolfowitz and Mann-Whitney tests. Using the method of moments of parameter estimation technique, the time series of the flow data was fitted to extreme value type one (EV I). The flood hazard mapping was done through HEC-GEORAS and HEC-RAS software's. Finally, by comparing the amount of crop yield produced previously from a hectare of land and the expected crop yield from the inundated area the flood damage could be estimated. Thus indicates that, improved Awash Bello stream flow gauge station which measures flow data appropriately for the entire period i.e. it properly records the flow data without any disturbance from 1960 to 2010. The flow data of Awash Bello s station was found to be independent, homogenous, stationary and no outlier at 5% significance level. The quantile estimates of the 2, 5, 50, 100, and 500 year return periods for the site were also found to be 39.30, 50.38, 73.86, 80.69 and 96.46 m^3/s, respectively. These floods inundated croplands having sizes of 1,959.50, 2,107.38, 2,299.16, 2,318.84, 2,354.06 hectares, respectively. These corresponded to losses of 44,088.66, 47,415.96, 51,731.03, 52,173.84, 52,966.25 quintals of crop, respectively. This study has shown that the upper most part of the cultivated is more inundated than the middle and lower parts. The areas affected by the flooding are found closer to the Awash River. Therefore, the affected areas is to be free of any agricultural activities, infrastructure development, investment and residence of people in order to avoid the risk of flooding in the area especially closer to the Awash River

Keywords: *Hec-Ras, Hec-GeoRAS, Flood hazard map and Crop production.*

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List of Abbreviations

amsl: Above Mean Sea Level

DEM: Digital Elevation Model

DPPA: Disaster Prevention and Preparedness Agency

DTM: Digital Terrain Model

ESRI: Environmental System Research Institute

EVI: Extreme value Type I

FHM: Flood Hazard Mapping

FDC: Flow Duration Curve

GEV: General Extreme value

GIS: Geographic Information System

GMM: Generalized Method of Moments

GUI: Graphical User Interface

HEC-RAS: Hydraulic Engineering Center for River Analysis System

MLM: Maximum Likelihood Method

MOM: Method of Moments

M-W: Mann-Whitney

NASA: National Astronomy and Space Science

NMA: National Metrological Agency

NWS: National Weather Service

PDF: Probability Density Function

PWM: Probability Weighted Moments Method

SRTM: Shuttle Radar Topography Mission

TIN: Triangular Irregular Network

USACE: United States Army Corps of Engineers

USGS: United States Geological Survey

W-W: Wald-Wolfowitz

1. INTRODUCTION

1.1 Background

Human existence involves exposure to many hazards. Since the beginning of natural civilization disasters, such as floods and earthquakes have threatened humanity. With technological progress, new technologies and corresponding hazards were introduced. Since the industrial revolution, technical hazards, such as industrial accidents, train derailments, tunnel fires and airplane crashes also disrupt society on a regular basis (Bernstein, 1997).

Topographically, Ethiopia is both mountainous and lowland country. It is composed of nine major river basins, the drainage systems of which originate from the centrally situated highlands and make their way down to the peripheral or outlying lowlands. Especially during the rainy season (June-September), the major perennial rivers as well as their numerous tributaries forming the country"s drainage systems carry their peak discharges (Joint Government and Humanitarian Partners, 2006).

River flooding is a natural process and part of the hydrological cycle of rainfall, surface and groundwater flow and storage. Floods occur whenever the capacity of the natural or manmade drainage system is unable to handle the volume of water generated by rainfall.

The country experiences two types of floods, flash floods and river floods. Most floods in the country occur as a result of river overflow following extended rainfall causing inundation of areas along riverbanks in lowland plains. Among the major river flood prone areas are parts of Oromia and Afar regions lying along the upper, middle and downstream plains of the Awash River; parts of Somali Region along the Wabi shebelle, Genale and Dawa rivers; low-lying areas of Gambella along the Baro, Gilo and Akobo rivers; downstream areas along the Omo River in SNNPR and the extensive floodplains surrounding Lake Tana and the banks of Gumara, Rib and Megech rivers in Amhara. (DPPA, 2013 Flood Alert).

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

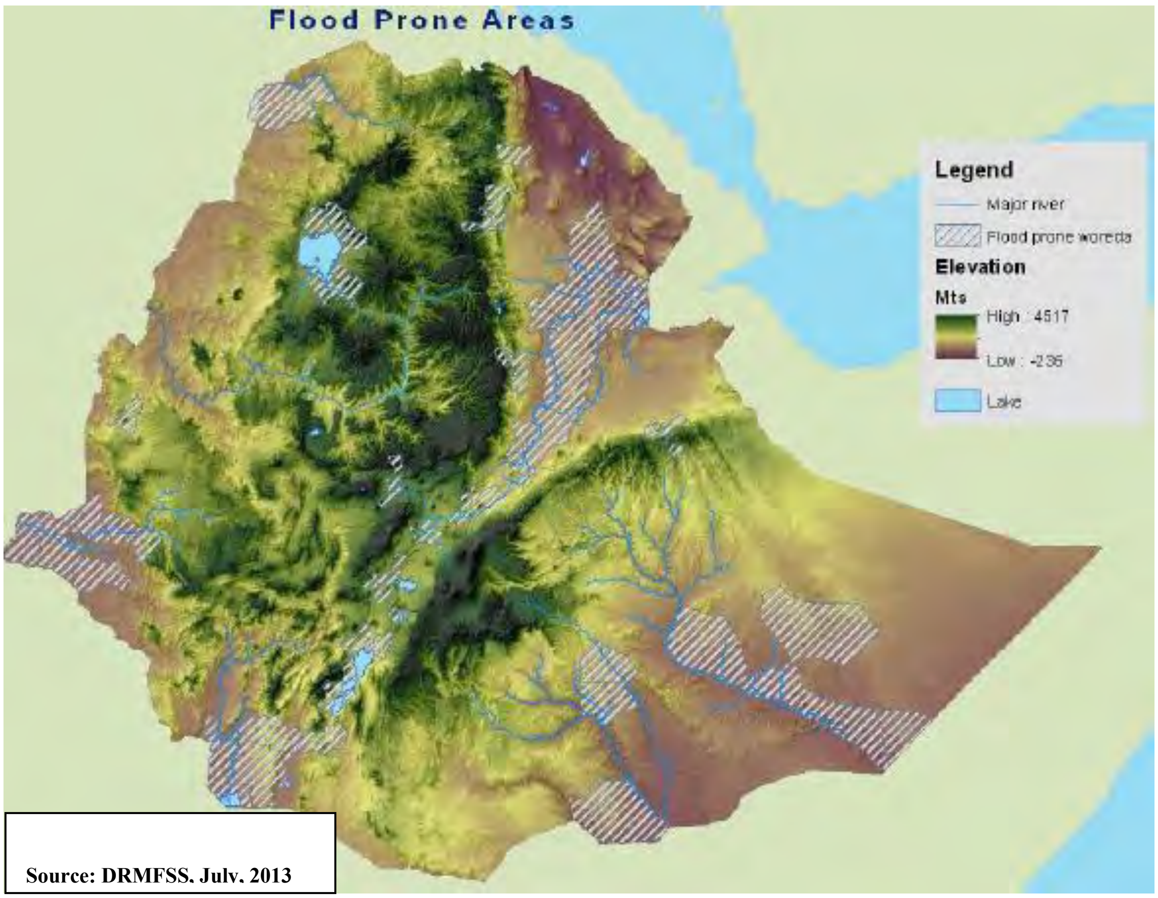


Figure 1.1:- Flood Prone Areas

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

The rains have caused most rivers to swell and overflow or breach their courses, submerging the surrounding floodplains, which are mostly located in the outlying pastoralist regions of the country. It is evident that the problem of river flooding in Ethiopia is getting more and more acute due to human intervention in the fragile highland areas at an ever-increasing scale (UNOCHA, 2006)
Ethiopia's eastern Oromia region was the worst hit in the giant Horn of Africa nation, with 97,000 people affected, of which 37,000 lost their homes (UNOCHA, 2006). The floods swamp large areas of cropland in this region. According to the 2006 Flash Appeal (Joint Government and Humanitarian Partners,2006) a total of 524,400 remain vulnerable to flood disaster throughout the country of which 199,900 people are affected by flood disaster in various areas, in Oromia Region 61,300 people are vulnerable and 21,900 were affected by the 2006 flood event.

A major objective in water management is to see that excess water from extreme flood events is controlled so as to minimize distress and hardship to the population and damage to the environment. Both the river engineer and the water resources engineer need the skills of the hydrologist to evaluate flood flows. The information requirements of the previous vary according to the nature of the river channel and the adjoining floodplain; data on floods for the water resources engineer, apart from managing reservoir storages, are primarily needed for the design of reservoir spillways (Brooks, 2003).

1.2 Statement of the problem

Flood is probably the most devastating, widespread and frequent natural hazard of the world. Flood is one of the major natural hazards in Ethiopia which affects lives and livelihoods in parts of the country. Flooding in Ethiopia is mainly linked with the topography of the highland mountains and lowland plains with natural drainage systems formed by the principal river basins. This problem is more acute in highland areas like Ethiopia under strong environmental degradation due to population pressure.

Although flood events are not new to Ethiopia, the country, in its main rainy season, has been susceptible by quite unprecedented flooding of abnormal magnitude and damage. Apparently, this is, for the large part due to heavy rains falling for long days on the upstream highlands, high sediment concentration in the Awash River resulting in silt deposition, which aggravates the flooding problem by reducing the capacity of the channel to pass flood water downstream.

Awash River is one of the rivers which cause flooding at Illu, Sebeta and Ejerie floodplain. The river conveys high runoff from upper catchments and local rainfall on the floodplain to resulting in flooding problems. The Illu-Sebeta and Ejerie floodplain is affected of 4,506 hectare of different crops have been damaged and livestock disease outbreaks are anticipated by floods as it is located at lower level in the river. (Joint Government and Humanitarian Partners, 2006)

The specific issue of this thesis is the magnitude and impacts of flooding in Illu, Sebeta and Ejerie floodplains located in the upper most catchment of Awash River

1.3 Research questions

What is the magnitude of flood frequency for different return periods?

What is the output of the HEC-RAS model of flood profile of the study area?

What is the delineation of the flood prone area for different water surface profile?

What are the results of flood risk for different return periods?

1.4 Objectives of the study

1.4.1 General objective

The main objective of the study is to contribute to flood risk management in Illu floodplain of upper Awash River Basin.

1.4.2 Specific objectives

- To conduct flood frequency analysis.
- To simulate flood profiles of the Upper Awash River at the study site using HEC-RAS model
- To delineate flood-prone areas.
- To conduct flood risk assessment.

1.5 Scope of the study

The study focused only on flooding of Awash River particularly at risk areas starting from Jigdu Mida and Tulu Mangero Kebele at Illu Wereda and Awash Bello and Welena Eka Kebele at Sebeta Hawas Wereda and Dibu Kebele at Ejerie Wereda. Across the two banks of the river, there is only crop production; therefore, whenever flooding occurs at this site, the only property at risk is cultivated area. Therefore, this thesis also included analysis of the effects of flooding on crop production. The cost of crop damage due to flooding was not determined because time value of money is highly variable. So the effect of flooding on crop was estimated based on the change in production.

1.6 Significance of the study

The study contributes to

- Reduction of the risk of direct flood damage to crops and property.
- Control flooding during important farming periods by applying mitigation measures
- Future flood-related researches in the study area

2. DESCRIPTION OF THE STUDY AREA

2.1 Location of the study area

The Awash River Basin is the fourth largest catchment in Ethiopia by area. The Awash River rises near Ginchi and flows in a north easterly direction through the northern extension of the Rift Valley to eventually discharge into Lake Abe near the Djibouti boarder, a distance of some 1,200km.

The uplands are further divided into the Upper Basin or the Koka catchment upstream of Koka Dam, the Eastern Catchment and the Western Catchment based on location in the basin. The Valleys consist of the Upper and Middle Valley areas and the plain demarcates the Lower Awash.

The study area is found between 50 km to 56 km southwest of the capital Addis Ababa and surrounded by Welmera and Burayu Weredas in the North, Akaki Wereda in the East, Kersana Malima, Tole bolo, Illu and Sebeta Weredas in the south and Ejere Weredas in the West.

The flood prone area particularly in the study area are around Jigdu Mida and Tulu Mangero Kebel‟s adjacent to Illu Wereda and Awash Bello and Welena Eka Kebel‟s adjacent to Sebeta Hawas Wereda and Dibu from Ejerie Wereda situated in South west and West shewa zone, which is located 38^{0} 24`` 59.9` Latitude and $8^{0}$ 51`` 59` Longitude, and has total area of 174,919 hectares. Therefore, these flood risk areas are subject to inundation for long periods.

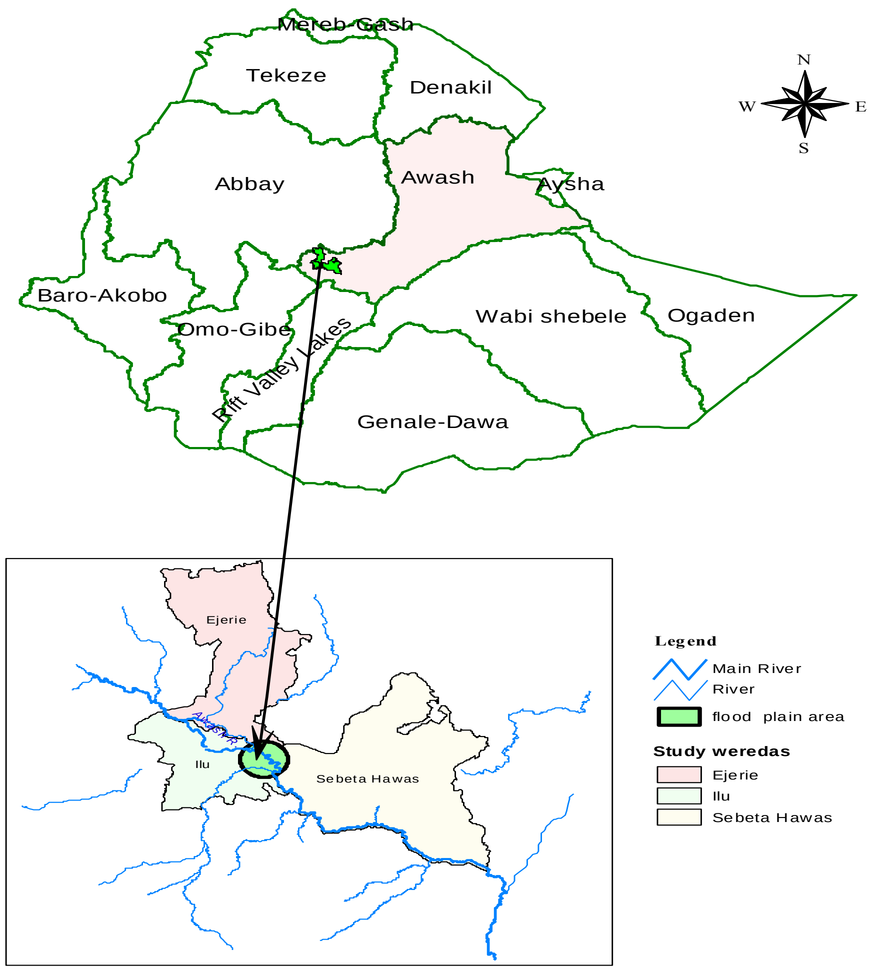


Figure 2.1: - Location Map of the study area

2.2 Topography and Geology

The elevation of the catchment ranges from 2,060m to 2,100m a.s.l the slope of the floodplain was approximates 100 % flat area or gentle slope. The geological condition of the catchment most upper part code **(Q)** is Sand, Silt, Clay, Diatomite, and Beach Sand and the lower part code **(NQtb)** is Alkaline basalt and Trachyte.

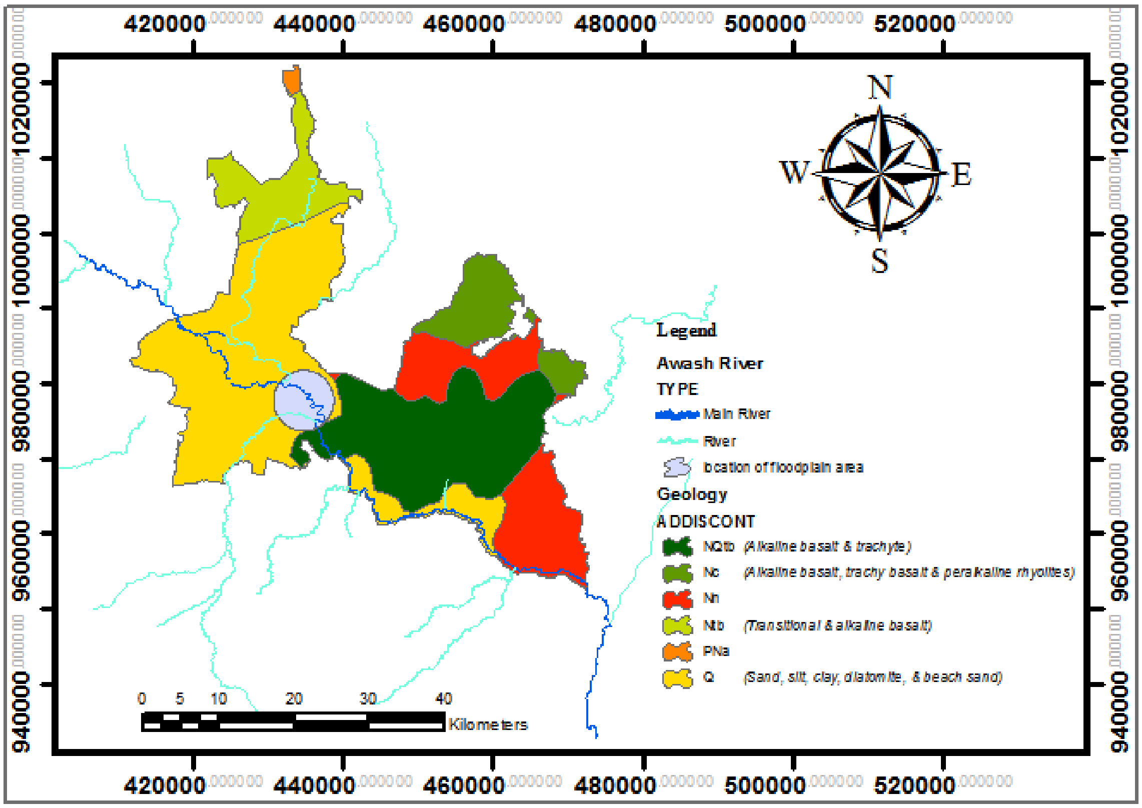


Figure 2.2: - Geology of the study area

2.3 Climate

According to the Wereda Agricultural and Rural Development Office report (2014) the temperature of the study area is between 15 ˚C and 25 ˚C and the average rain fall varies from 700 mm to 800 mm. The most important hydrological stream flow gauging stations in the study area is name as Awash Bello station was found in catchment. Stream flow data were analyzed for this research 51 years flow data was record daily from 1960 to

2010 was taken. The area receives the highest rainfall in the months of June to August and small rains from February to April.

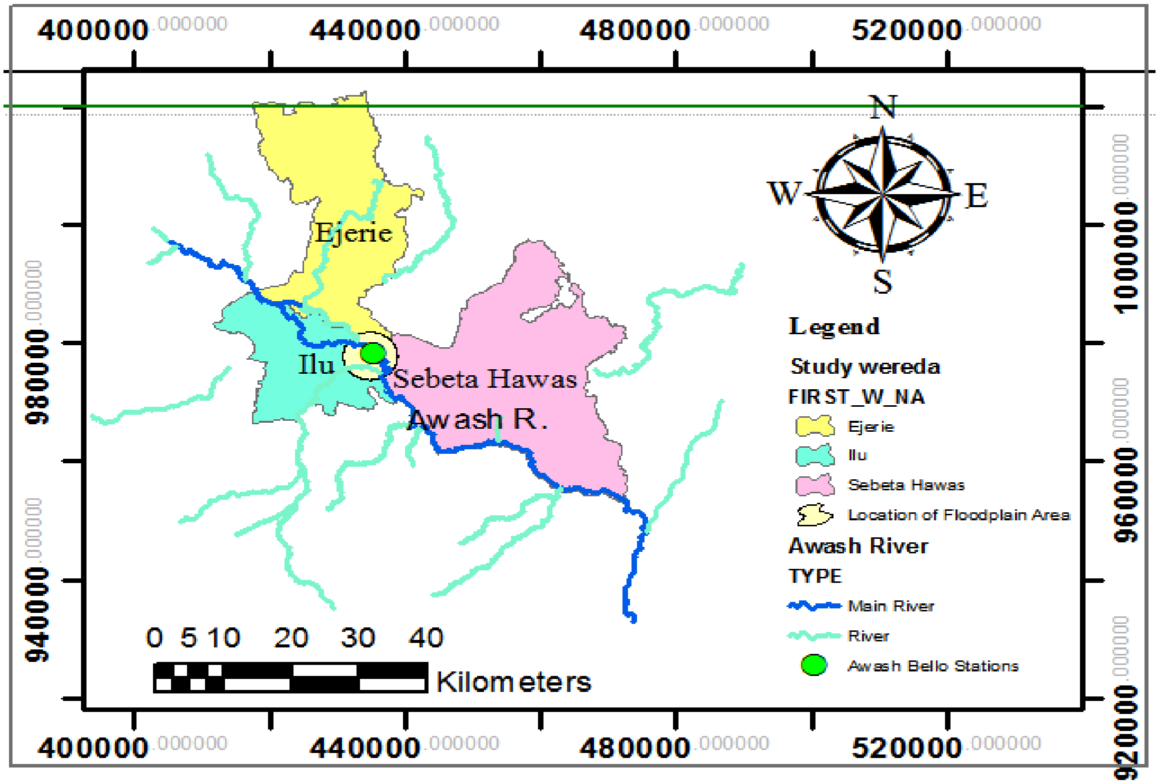


Figure 2.3:-Awash Bello Stream Flow Gauging Station of the study area

2.4 Land use/Land cover

The catchment is densely populated and intensively cultivated by smallholders with crop and livestock production was the main farming system. According to the Wereda Agricultural and Rural Development Office report (2014) the main crops are grown in the study area are Teff, Chick Peas, Lentils and Grass Pea. However, crop production is so intensive that there is little room left for pasture to sustain the large number of livestock. Therefore, traditional farming systems that exist in the study area, a number of small-scale irrigation schemes have been developed, along the river. As shown in Fig.2.4 the land cover of the study area is can be classified as Cropland with Shrubland.

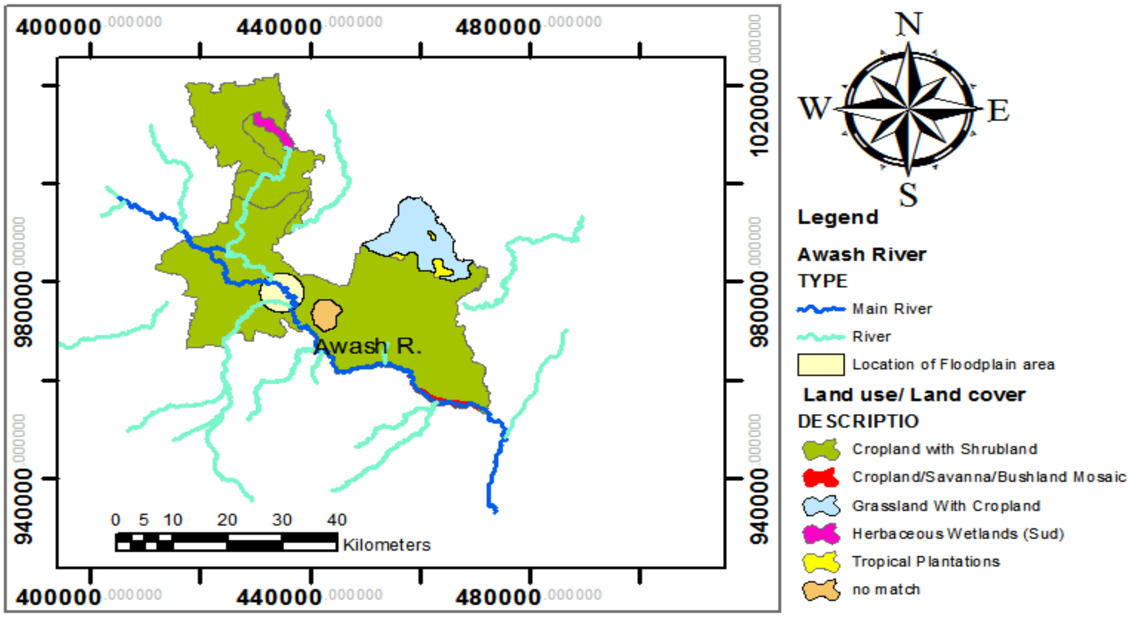


Figure 2.4: - Land Cover of the study area

2.5 Soil type

As shown in Fig. 2.5 the soil type of the study area can be classified as the most part of the floodplain is Pellic vertisoils and along the Awash river flow the lift and right bank of the river is Chromic cambisoils.

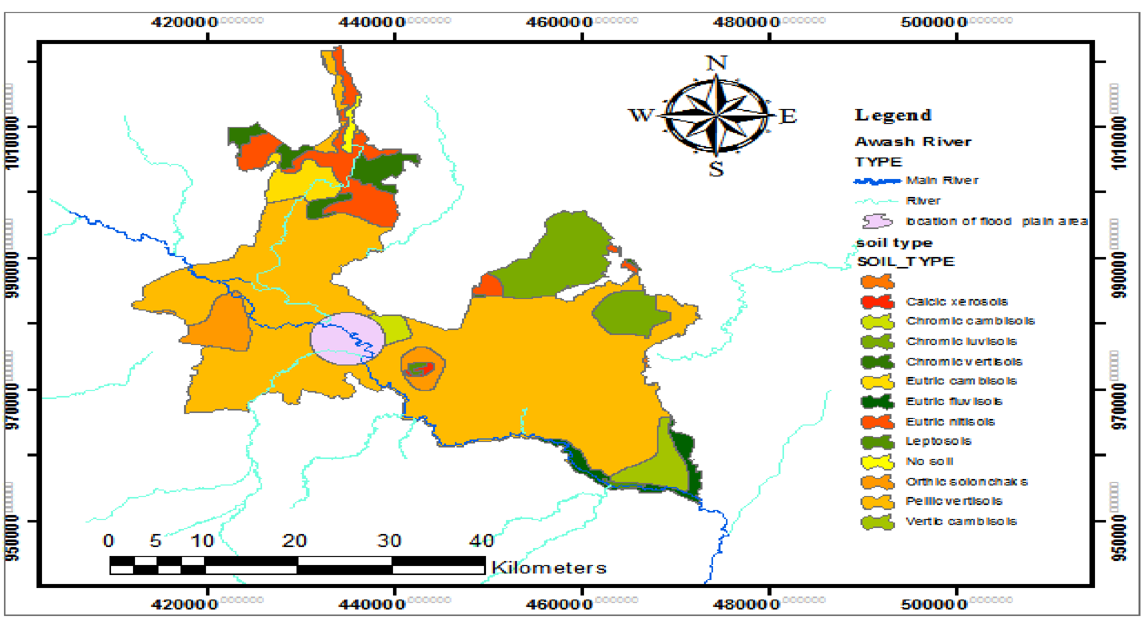


Figure 2.5: - Soil type of the Study Area

2.6 Population

The population of the study area is increased from time to time According to the Wereda Agricultural and Rural Development Office report (2014) the total estimated population of the study area was 36,200 men and 34,996 women, total population 71,196.

3. LITERATURE REVIEW

3.1 Flood

Flooding is a natural process that can happen at any time in a wide variety of locations. Flooding from the sea and from rivers is probably best known but prolonged, intense and localized rainfall can also cause sewer flooding, overland flow and groundwater flooding. Flooding has significant impacts on human activities; it can threaten people‟s lives, their property and the environment. Assets at risk can include housing, transport and public service infrastructure, and commercial, industrial and agricultural enterprises. The health, social, economic and environmental impacts of flooding can be significant and have a wide community impact. The frequency, pattern and severity of flooding are expected to increase as a result of climate change. Development can also exacerbate the problems of flooding by accelerating and increasing surface water run-off, altering watercourses and removing floodplain storage**.** (Gormley et al, 2009)

According to Raghunath (2000) a flood is an unusual high stage of a river due to runoff from rainfall and or melting of snow in quantities too great to be confined in the normal water surface elevations of the river or stream, as the result of unusual meteorological combination. The maximum flood that any structure can safely pass is called the „design flood‟ and is selected after consideration of economic and hydrologic factors. The design flood is related to the project feature; for example, the spillway design flood may be much higher than the flood control reservoir design flood or the design flood adopted for the temporary coffer dams. A design flood may be arrived by considering the cost of constructing the structure to provide flood control and the flood control benefits arising directly by prevention of damage to structures downstream, disruption communication, loss of life and property, damage to crops and underutilization of land and indirectly, the money saved under insurance and workmen‟s compensation laws, higher yields from intensive cultivation of protected lands and elimination of losses arising from interruption of business, reduction in diseases resulting from inundation of flood waters.

3.2 Flood disaster in Ethiopia

The past, there have been floods which have taken both human lives and destroyed properties. As a result of prolonged and intensive rainfall, the soil in most areas, particularly in the western, central highlands and northwestern parts of the country became saturated causing an overflow of rivers and flash floods in many areas According to Joint Government and Humanitarian Partners (2006) Dire Dawa, SNNPR, Amhara, Oromiya, Gambella, Tigray, Somali and Affar Regions. The flood situation resulted in considerable human death, displacement and suffering as well as loss of property and crop damage. The current problem is the worst that has been observed in recent years. The most affected areas are Dire Dawa, South Omo Zone of SNNPR, and parts of Amhara, Oromiya, Gambella, Somali and Tigray regions.

Dire Dawa: The heavy rains in the highlands of East Hararghe Zone of Oromiya Region caused flash flood from the overflow of Dechatu dry season stream that hit Dire Dawa town in the middle of the night on the sixth of August, while residents were asleep. According to the Diredawa Administration, the flood affected over 9,000 people and killed 256 others. The death toll was largely due to the fact that the flooding took place at a time when people were in deep asleep. Currently, over 6,000 people have been temporarily sheltered in six sites, including schools and other compounds. It has also washed away houses and properties of many people living along the riverbank. Roads, bridges and other public properties were damaged and washed away. The estimate of overall loss or damage of properties of individuals and infrastructure is recently disclosed by the Administration to be Birr 27 million.

SNNPR: The extraordinary overflow of Omo River in August severely affected about 8,000 people in Dasenech and Gnangatom woredas of South Omo Zone. It has also killed 364 people and swept away some 3,200 cattle and destroyed other properties, including 760 traditional grain stores. This area is one of the worst affected as compared to others in the country. Efforts have been underway to rescue and save the lives of those stranded and also find the bodies of those drowned by the flood. Furthermore, flashflood from Bilate River was reported to have affected 5370 households in Humbo Woreda of Wolayita Zone out of which 2,515 households were severely affected and require immediate emergency assistance. Moreover, landslides due to heavy rainfall were reported from Guraghe and Kefa zones although their impact was not as serious as that of the floods.

Amhara Region: Overflow of Rib and Gomara rivers and Lake Tana in Libo Kemekem and Fogera woredas of South Gonder, Bahirdar Zuria and Bahidar town of West Gojjam and flash floods in Dewchefa and Ansokiya woredas of Oromiya and North Shewa zones, respectively, displaced people from their residential places and forced them to stay under temporary shelter. Currently, there are a total of 13,362 people reported to be under temporary shelter in the region.

West Shewa Zone of Oromiya Region: Flooding of the upper basin of Awash River affected 14 peasant associations (PAs) in Illu, Sebeta Hawas and Ejere woredas of West Shewa Zone. The flood was reported to have affected a total of 14,790 people out of which 2052 people were displaced and forced to live in temporary shelters. On the other hand, heavy rainfall in the central highlands in the coming weeks is considered as a major threat around the major dams in the region (Koka, Gilgel Gibe and Melka Wakena). The dams are already full and contain excessive water. Some of them have already started to overflow.

Gambella Region: Reports of overflow of Baro River have been received from the Region. Areas affected by the flood are Gambella Zuria, Jikawo, Itang and Gillo woredas. So far, the impact of the flood on human beings is not yet serious. However, it has affected a large area of crop fields. Flooding in Gambella is normal but its occurrence at this time of the year is somewhat early. All rivers in the Region are full. High rainfall in the coming weeks in the western highlands could cause severe flooding. Generally, the situation in the Region is worrisome calling for close monitoring and follow-up.

Somali Region: According to recent information from the Region, as a result of overflow of Wabishebelle River, 3,000 and 4,500 households have been affected from Mustahil and Kelafo woredas, respectively, and a total of 650 hectares of farmland damaged, which remains yet to be verified.

Tigray Region: Overflow of Tekeze River in Kafta Humera woreda of Western Tigray

Zone has displaced 450 households, destroyed houses and damaged crops fields.

3.3 Tests on hydrologic data

The basic assumptions in statistical flood frequency analysis are the independence and stationarity of the data series. In addition, the assumption that the data come from the same distribution (homogeneity) and outlier is made. The following tests, which are also discussed by Bobee and Ashkar (1991), are commonly used to test for stationarity, homogeneity and independence of data. Other (similar) tests are found in Kite (1977).

3.3.1 Test of Independence and Stationarity

Given a sample of size N, the Wald-Wolfowitz (1943) (W-W) test is used to test for the independence of a dataset and to test for the existence of trends in it. For a data set x_1, x_2 ...x_n the statistic R is calculated.

$$R = \sum_{i=1}^{N-1} x_i x_{i+1} + x_1 x_n \qquad (3.3.1.1)$$

When the elements of the sample are independent, R follows a normal distribution with mean and variance given.

$$\bar{R} = \frac{S_1^2 - S_2}{N-1} \qquad (3.3.1.2)$$

Where $S_r = N m_r'$ and $m_r' = \frac{1}{n}\sum_{i=1}^{n} X_i^r$ (3.3.1.3)

mr'' is the rth moment of the sample about the origin.

The statistic U is approximately normally distributed with mean zero and variance unity and is used to test the hypothesis of independence at significance level, by comparing the statistic u with the standard normal variate $U_{\alpha/2}$ corresponding to a probability of exceedance $\alpha/2$.

$$u = (R - \bar{R})/(Var(R))^{1/2} \qquad (3.3.1.4)$$

3.3.2 Tests of Homogeneity and Stationarity

In this test two samples of size p and q with P is less than or equal to q are compared. The combined data set of size N = p + q is ranked in increasing order. The Mann-Whitney (1947) (M-W) test considers the quantities V and W in

$$V = R - \frac{(P(P+1))}{2} \qquad (3.3.2.1)$$

$$W = pq - V \qquad (3.3.2.2)$$

R is the sum of the ranks of the elements of the first sample (size p) in the combined series (size N), and V and W are calculated from R, p, and q. V represents the number of times an item in sample 1 follows an item in sample 2 in the ranking. Similarly, W can be computed for sample 2 following sample 1. The M-W statistic U is defined by the smaller of V and W. When N > 20 and p, q > 3, and under the null hypothesis that the two samples came from the same population, U is approximately normally distributed with mean and variance var (U),

$$\bar{U} = \frac{pq}{2} \qquad (3.3.2.3)$$

$$Var(U) = \left(\frac{pq}{N(N-1)}\right)\left(\frac{N^3 - N}{12} - \sum T \qquad (3.3.2.4)$$

$$T = \left(\frac{J^3 - J}{12}\right) \qquad (3.3.2.5)$$

Where T and J is the number of observations tied at a given rank. T is summed over all groups of tied observations in both samples of size p and q. The statistic u is used to test the hypothesis of homogeneity at significance level α by comparing it with the standard normal variate for that significance level.

$$u = \left(\frac{U - \bar{U}}{(Var(U))^{1/2}}\right) \qquad (3.3.2.6)$$

3.3.3 Testing of Outliers

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

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

Yh = y + Knsy (3.3.3.1)

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

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

A similar equation can be used to detect low outliers:

Yl = y-Knsy (3.3.3.2)

Where: Yl is the low outlier threshold in log units.

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

Table 3.1:- Outlier Test of Kn values

Sample size n	K_n	Sample size n	K_n	Sample size n	K_n	Sample size n	K_n
10	2.036	24	2.467	38	2.661	60	2.837
11	2.088	25	2.486	39	2.671	65	2.866
12	2.134	26	2.502	40	2.682	70	2.893
13	2.175	27	2.519	41	2.692	75	2.917
14	2.213	28	2.534	42	2.700	80	2.940
15	2.247	29	2.549	43	2.710	85	2.961
16	2.279	30	2.563	44	2.719	90	2.981
17	2.309	31	2.577	45	2.727	95	3.000
18	2.335	32	2.591	46	2.736	100	3.017
19	2.361	33	2.604	47	2.744	110	3.049
20	2.385	34	2.616	48	2.753	120	3.078
21	2.408	35	2.628	49	2.760	130	3.104
22	2.429	36	2.639	50	2.768	140	3.129
23	2.448	37	2.650	55	2.804		

Source: U.S. Water Resources Council, 1981. This table contains one-sided 10-percent significance level K_n values for the normal distribution.

3.4 Flood frequency analysis

According to Chow (1998) Hydrologic systems are sometimes impacted by extreme events, such as severe storms, floods, and droughts. The magnitude of an extreme event is inversely related to its frequency of occurrence, very severe events occurring less frequently than more moderate events.

The objective of frequency analysis of hydrologic data is to relate the magnitude of extreme events to their frequency of occurrence through the use of probability distributions.

The hydrologic data analyzed are assumed to be independent and identically distributed, and the hydrologic system producing them (e.g., a storm rainfall system) is considered to be stochastic, space-independent, and time-independent in the classification. The hydrologic data employed should be carefully selected so that the assumptions of independence and identical distribution are satisfied. In practice, this is often achieved by selecting the annual maximum of the variable being analyzed (e.g., the annual maximum discharge, which is the largest instantaneous peak flow occurring at any time during the year) with the expectation that successive observations of this variable from year to year will be independent.

The results of flood flow frequency analysis can be used for many engineering purposes: such as for the design of dams, bridges, culverts, and flood control structures; to determine the economic value of flood control projects; and to delineate flood plains and determine the effect of encroachments on the flood.

According to Ojha, Berndtsson and Bhunya (2008) in hydrologic analysis, the annual peak discharge is considered to be a random variable. Probability and statistical methods are employed for analysis of random variables. Some elementary probability distributions are presented, which are used for frequency analysis in hydrology. In order to have meaningful estimates from flood frequency analysis, the following assumptions are implicit.

The data to be analyzed describe random events.

- The data is homogeneous,
- The population parameters can be estimated from the sample data and
- It is of good quality.

If the data available for analysis do not satisfy any of the listed assumptions, then the estimates are not considered reliable. Moreover, the data should be

- Relevant,
- Adequate, and

- Accurate. For flood frequency analysis, either annual flood series or partial duration series may be used.

In general, an array of annual peak flood series may be considered as a sample of random and independent events. The non-randomness of the peak series will increase the degree of uncertainty in the derived frequency relationship.

Various tests are available to check the randomness of the peak flow data. The annual maximum flood series can generally be regarded as consisting of random events as the mean interval of each observed flood peak is 1 year. However, in case data is used for partial duration series analysis, then independence among the data is doubtful. The peaks are selected in such a way that they constitute a random sample.

The term „relevant" means that the data must deal with the problem. For example, if the problem is of the duration of flooding, then the data series should represent the duration of flows in excess of some critical value. If the problem is of interior drainage of an area, then the data series must consist of the volume of water above a particular threshold.

The term „adequate" primarily refers to the length of data. The length of data primarily depends upon variability of data; and hence, there is no guideline for the length of data to be used for frequency analysis.

The term „accurate" primarily refers to the homogeneity of data and accuracy of the discharge figures. The data used for analysis should not have any effect of man-made changes. Changes in the stage discharge relationship may render stage records non-homogeneous and unsuitable for frequency analysis. It is therefore preferable to work with discharges; and if stage frequencies are required, then most recent rating curve is used.

Watershed history and flood records should be carefully examined to ensure that no major watershed changes have occurred during the period of record. Only those records, which represent relatively constant watershed conditions, should be used for frequency analysis.

According to Shaw (1994) the measured instantaneous flood peak discharges abstracted from calibrated levels on autographic charts or automatic digital recorders constitute one of the most valuable data sets for the hydrologist. The longer a record continues, homogeneous and with no missing peaks, the more is its value enhanced. Even so, it is very rare to have a satisfactory record long enough to match the expected life of many engineering works required to be designed. As many peak flows as possible are needed in assessing flood frequencies. The hydrologist defines two data series of peak flows: the annual maximum series and the partial duration series.

Annual Maximum Series takes the single maximum peak discharge in each year of record so that the number of data values equals the record length in years. For statistical purposes, it is necessary to ensure that the selected annual peaks are independent of one another.

This is sometimes difficult, e.g. when an annual maximum flow in January may be related to an annual maximum flow in the previous December. For this reason, it is sometimes advisable to use the Water Year rather than the calendar year; the definition of the Water Year depends on the seasonal climatic and flow regimes.

Partial Duration Series takes all the peaks over a selected level of discharge, a threshold. Hence the series is often called the „Peaks over Threshold" series (POT). There are generally more data values for analysis in this series than in the Annual Series, but there is more chance of the peaks being related and the assumption of true independence is less valid.

According to Raghunath (2006) when stream flow peaks are arranged in the descending order of magnitude they constitute a statistical array whose distribution can be expressed in terms of frequency of occurrence. There are two methods of compiling flood peak data the annual floods and the partial duration series.

In the annual floods, only the highest flood in each year is used thus ignoring the next highest in any year, which sometimes may exceed many of the annual maximum. But all the larger floods are used in this analysis, advantage while in annual flood series some big floods are omitted because they were not the highest floods in any year considered. Usually the basic stage is assumed sufficiently low so that as many peaks (4 or 5) as possible each year are above this stage.

In the partial duration series, all floods above a selected minimum are taken for analysis, regardless of the time-interval, so that in some years there may be a number of floods above the basic stage, while in some other years there may not any such flood at all. The disadvantage of the partial duration series is that the data do not furnish a proper frequency (true distribution) series and so a reasonable statistical analysis cannot be made.

The two series give very nearly the same recurrence interval for the larger floods, but the partial series indicates higher floods for shorter recurrence intervals. For information about floods of fairly frequent occurrence, as is required during the construction period of a large dam (say, 4-5 years), the partial series are the best, while for the spillway design flood the annual series are preferable, since the flood should not be exceeded in the dam"s life time, say 100 years Annual.

3.5. Parameter estimation techniques

A number of methods can be used for parameter estimation. These include the method of moments (MOM), the maximum likelihood method (MLM), the probability weighted moments method (PWM), the least squares method (LS), maximum entropy (ENT), mixed moments (MIX), the generalized method of moments (GMM), and incomplete means method (ICM).

Three of the more commonly used methods are considered here, namely, the method of moments (MOM), the maximum likelihood method (MLM) and the probability weighted moments method (PWM).

The maximum likelihood method (MLM) is considered the most efficient method since it provides the smallest sampling variance of the estimated parameters, and hence of the estimated quantiles, compared to other methods. However, for some particular cases, such as the Pearson type III distribution, the optimality of the ML method is only asymptotic and small sample estimates may lead to estimates of inferior quality (Bobeé and Ashkar, 1991). Also, the ML method has the disadvantage of frequently giving biased estimates, but these biases can be corrected. Furthermore, it may not be possible to get ML estimates with small samples, especially if the number of parameters is large. The ML method requires higher computational efforts, but with the increased use of high-speed personal computers, this is no longer a significant problem.

The PWM method (Greenwood et al., 1979; Hosking, 1986a) gives parameter estimates comparable to the ML estimates, yet in some cases the estimation procedures are much less complicated and the computations are simpler. Parameter estimates from small samples using PWM are sometimes more accurate than the ML estimates (Landwehr et al., 1979c). Also, in some cases, such as the symmetric lambda and Weibull distributions, explicit expressions for the parameters can be obtained by using PWM, which is not the case with the ML or MOM methods. Kebaili-Bergaoui (1994) showed that the ML and ME methods of parameter estimation for Weibull, P (3), Galton, and Gumbel distributions are a particular case of generalized method of moments.

The method of moments (MOM) is a natural and relatively easy parameter estimation method. However, MOM estimates are usually inferior in quality and generally are not as efficient as the ML estimates, especially for distributions with large number of parameters (three or more), because higher order moments are more likely to be highly biased in relatively small samples.

3.5.1 Method of maximum likelihood (MLM)

Estimation by the ML method involves the choice of parameter estimates that produce a maximum probability of occurrence of the observations. For a distribution with a probability density function (pdf) given by *f(x)* and parameters$\alpha 1, \alpha 2, \ldots, \alpha k$, the likelihood function is defined as the joint pdf of the observations conditional on given values of the parameters$\alpha 1, \alpha 2, \ldots, \alpha k$

$$L(\alpha_1, \alpha_2, \ldots, \alpha_k) = \prod_{i=1}^{n} f(x_i; \alpha_1, \alpha_2, \ldots, \alpha_k) \quad (3.5.1.1)$$

The value of $\alpha 1, \alpha 2, \ldots, \alpha k$, that maximize the likelihood function are computed by partial differentiation with respect to $\alpha 1, \alpha 2, \ldots, \alpha k$, and setting these partial derivatives equal to zero.

The resulting sets of equations are then solved simultaneously to obtain the values of $\alpha 1$, $\alpha 2 \ldots \alpha k$.

$$\frac{\partial L(\alpha_1, \alpha_2, \ldots, \alpha_k)}{\partial \alpha_i} = 0; \quad i = 1, 2, \ldots, k \quad (3.5.1.2)$$

In many cases it is easier to maximize the natural logarithm of the likelihood function by using

$$\frac{\partial \ln L(\alpha_1, \alpha_2, \ldots, \alpha_k)}{\partial \alpha_i} = 0; \quad i = 1, 2, \ldots, k \quad (3.5.1.3)$$

3.5.2 Method of probability weighted moments (PWM)

Parameter estimates are obtained in this method, as in the case of MOM, by equating moments of the distributions with the corresponding sample moments. For a distribution with *k* parameters $\Phi 1$, $\Phi 2 \ldots \Phi k$, which are to be estimated, the first *k* sample moments are set equal to the corresponding population moments. The resulting equations are then solved simultaneously for the unknown parameters$\Phi 1$, $\Phi 2 \ldots \Phi k$.

3.5.3 Method of moments (MOM)

Estimates of the parameters of a probability distribution function are obtained in the MOM by equating the moments of the sample with the moments of the probability distribution function. For a distribution with k parameters $\alpha 1, \alpha 2, \ldots, \alpha k$, which are to be estimated, the first k sample moments are set equal to the corresponding population moments that are given in terms of unknown parameters. These k equations are then solved simultaneously for the unknown parameters$\alpha 1$, $\alpha 2 \ldots$ αk i.e.

3.6 Probability distributions of hydrologic variables

According to Chow (1998), there are a number of distributions in hydrology used to analyze the probability of occurrence of a stream flow.

3.6.1 Normal distribution

The normal distribution arises from the central limit theorem, which states that if a sequence of random variables Xi are independently and identically distributed with mean μ and variance o2then the distribution of the sum of nμ such random variables, $Y = \Sigma_{i=1}^{n} X_i$ tends towards the normal distribution with mean n and variance no2 as n becomes large. The important point is that this is true no matter what the probability distribution function of X is. So, for example, the probability distribution of the sample mean $\bar{x} = 1/n \Sigma_{i=1}^{n} x_i$ can be approximated as normal with mean μ and variance $(1/n)^2 n\sigma^2 = \sigma^2/n$ no matter what the distribution of x is. Hydrologic variables, such as annual precipitation, calculated as the sum of the effects of many independent events tend to follow the normal distribution. The main limitations of the normal distribution for describing hydrologic variables are that it varies over a continuous range$[-\infty, \infty]$, while most hydrologic variables are nonnegative, and that it is symmetric about the mean, while hydrologic data tend to be skewed.

3.6.2 Lognormal distribution

If the random variable Y = log X is normally distributed, then X is said to be lognormal distributed. Chow (1954) reasoned that this distribution is applicable to hydrologic variables formed as the products of other variables since if X = X1, X2, X3 . . . Xn then

$$Y = \log X = \Sigma_{i=1}^{n} \log X_i = \Sigma_{i=1}^{n} Y_i, \qquad (3.6.2.1)$$

This tends to the normal distribution for large n provided that the X1- are independent and identically distributed. The lognormal distribution has been found to describe the distribution of hydraulic conductivity in a porous medium (Freeze, 1975), the distribution of raindrop sizes in a storm, and other hydrologic variables. The lognormal distribution has the advantages over the normal distribution that it is bounded (X > 0) and that the log transformation tends to reduce the positive skewness commonly found in hydrologic data, because taking logarithms reduces large numbers proportionately more than it does small numbers. Some limitations of the lognormal distribution are that it has only two parameters and that it requires the logarithms of the data to be symmetric about their mean.

3.6.3 Pearson type III distribution

The Pearson Type III distribution, also called the three-parameter gamma distribution, introduces a third parameter, the lower bound ϵ, so that by the method of moments, three sample moments (the mean, the standard deviation, and the coefficient of skewness) can be transformed into the three parameters λ, β, and ϵ of the probability distribution. This is a very flexible distribution, assuming a number of different shapes as λ, β, and ϵ vary (Bobee and Robitaille, 1977). The Pearson system of distributions includes seven types; they are all solutions for f(x) in an equation of the form

$$\frac{d[f(x)]}{dx} = \frac{f(x)(x-d)}{C_0 + C_1x + C_2x^2} \qquad \boxed{(3.6.3.1)}$$

Where d is the mode of the distribution (the value of x for which (x) is a maximum) and Co, C_1, and C_2 are coefficients to be determined. When $C_2 = 0$, the solution is a Pearson Type III distribution, having a probability density function. For $C_1 = C_2 = 0$, a normal distribution is the solution. Thus, the normal distribution is a special case of the Pearson Type III distribution, describing a non-skewed variable. The Pearson Type III distribution was first applied in hydrology by Foster (1924) to describe the probability distribution of annual maximum flood peaks. When the data are very positively skewed, a log transformation is used to reduce the skewness.

3.6.4 Log-Pearson type III distribution

If log X follows a Pearson Type III distribution, then X is said to follow a log-Pearson Type III distribution. This distribution is the standard distribution for frequency analysis of annual maximum floods in the United States (Benson, 1968). As a special case, when log X is symmetric about its mean, the log-Pearson Type III distribution reduces to the lognormal distribution.

The location of the bound ϵ in the log-Pearson Type III distribution depends on the skewness of the data. If the data are positively skewed, then log X $\geq \epsilon$ and ϵ is a lower bound, while if the data are negatively skewed, log X $\leq \epsilon$ and ϵ is an upper bound. The log transformation reduces the skewness of the transformed data and may produce transformed data which are negatively skewed from original data which are positively skewed. In that case, the application of the log-Pearson Type III distribution would impose an artificial upper bound on the data. Depending on the values of the parameters, the log-Pearson Type III distribution can assume many different shapes, (Bobee, 1975).

3.6.5 Exponential distribution

Some sequences of hydrologic events, such as the occurrence of precipitation, may be considered Poisson processes, in which events occur instantaneously and independently on a time horizon, or along a line. The time between such events, or inter arrival time, is described by the exponential distribution whose parameter λ is the mean rate of occurrence of the events. The exponential distribution is used to describe the inter arrival times of random shocks to hydrologic systems, such as slugs of polluted runoff entering streams as rainfall washes the pollutants off the land surface. The advantage of the exponential distribution is that it is easy to estimate λ from observed data and the exponential distribution lends itself well to theoretical studies, such as a probability model for the linear reservoir (λ = l/k, where k is the storage constant in the linear reservoir). Its disadvantage is that it requires the occurrence of each event to be completely independent of its neighbors, which may not be a valid assumption for the process under study for example, the arrival of a front may generate many showers of rain and this has led investigators to study various forms of compound Poisson processes, in which λ is considered a random variable instead of a constant (Kavvas and Delleur, 1981; Waymire and Gupta, 1981).

3.6.6 Gamma distribution

The time taken for a number **β** of events to occur in a Poisson process is described by the gamma distribution, which is the distribution of a sum of **β** independent and identical exponentially distributed random variables. The gamma distribution has a smoothly varying form like the typical probability density function and is useful for describing skewed hydrologic variables without the need for log transformation. It has been applied to describe the distribution of depth of precipitation in storms, for example. The gamma distribution involves the gamma function $\Gamma(\beta)$, which is given by:

$$\Gamma(\beta)=(\beta-1)! \; =(\beta-1)(\beta-2)...3\cdot2\cdot1 \qquad (3.6.6.1)$$

For positive integer, and in general by

$$\Gamma(\beta) = \int_0^\infty u^{\beta-1}e^{-u}\,du \qquad (3.6.6.2)$$

(Abramowitz and Stegun, 1965) the two-parameter gamma distribution (parameters **β** and **λ**) has a lower bound at zero, which is a disadvantage for application to hydrologic variables that have a lower bound larger than zero.

3.6.7 Extreme value distribution

Extreme values are selected maximum or minimum values of sets of data. For example, the annual maximum discharge at a given location is the largest recorded discharge value during a year, and the annual maximum discharge values for each year of historical record make up a set of extreme values that can be analyzed statistically.

Distributions of the extreme values selected from sets of samples of any probability distribution have been shown by Fisher and Tippett (1928) to converge to one of three forms of extreme value distributions, called Types I, II, and III, respectively, when the number of selected extreme values is large. The properties of the three limiting forms were further developed by Gumbel (1941) for the Extreme Value Type I (EVI) distribution, Frechet (1927) for the Extreme Value Type II (EVII), and Weibull (1939) for the Extreme Value Type III (EVIII).

The three limiting forms were shown by Jenkinson (1955) to be special cases of a single distribution called the General Extreme Value (GEV) distribution.

The probability distribution function for the GEV

$$F(x) = \exp\left[-\left(1 - k\,\frac{x-u}{\alpha}\right)^{1/k}\right] \qquad \boxed{(3.6.7.1)}$$

Where: - k, u, and α are parameters to be determined.

The three limiting cases are (1) for k = 0, the Extreme Value Type I distribution, for which the probability density function (2) for k < 0, the Extreme Value Type II distribution and (3) for k > 0, the Extreme Value Type III distribution. In all three cases, **α** is assumed to be positive.

For the EVI distribution x is unbounded, while for EVII, x is bounded from below and for the EVIII distribution, x is similarly bounded from above.

The EVI and EVII distributions are also known as the Gumbel and Frechet distributions, respectively. If a variable x is described by the EVIII distribution, then -x is said to have a Weibull distribution.

3.7 Selection of models

According to World Meteorological Organization (1994).The choice of models is not restricted to the models described below. It is often difficult to determine the relative advantages and disadvantages of models proposed for operational use. The selection of a model for a specific hydrological situation has implications in water-resources planning, development, and management.

Some of the factors and criteria involved in the selection of a model include the following:

- The purposes and benefits of the model-output, e.g., continuous hydrograph of discharges, forecast of floods, water quality, and water-resource management;
- The climatic and physiographic characteristics of the basin;
- The lengths of the records of the various types of data;
- The quality of the data both in time and space;
- The availability and size of computers for both development and operation of the model;
- The possible need for transferring model parameters from smaller catchments to larger catchments; and
- The ability of the model to be updated on the basis of current hydro-meteorological conditions.

A WMO international project on the inter comparison of conceptual models used in operational hydrological forecasting, which was completed in 1974, produced useful information and guidance on the selection and application of conceptual models in various hydrological conditions.

There are different flood modeling tools which have their own distinct model structure and solution procedures. Mostly widely used 1D flood modeling tools are: HEC-RAS, FLDWAV, HEC-FDA,ISIS, FLUCOMP, GFT, DUFLOW and MIKE11.Although today‟s researches prefer1D and 2D interaction flood model, the use of one dimensional model has also great significance in the research areas. In this research 1D hydraulic model with HEC-GEORAS and GIS is used because the model has the capacity to simulate flood plain areas with great accuracy.

3.8 Hydraulic models

River flood routing (flood propagation in rivers) can be described by one dimensional (1D) mathematical model Dyhous et al (2003). This solution is suitable for the modeling of inundation of open floodplains as well but in case of sophisticated morphological conditions application of quasi 2D or 2D models might be necessary. At present, one of the ways to study and understand the flood behavior is by generating the flood extent or flood risk map Hassan et al (2009).A hydraulic modeling especially computer model is required to carry out the flood simulation to produce flood level at various locations along the river and flood plain. However, to analyses a river system requires tremendous amount of data such as rainfall distribution, river properties and most important the flood plain topography. Andrysia et al (2000), states that GIS software is able to handle the processing of such problem as an input to the hydraulic model. The output of the hydraulic simulation can be transferred to GIS software to generate flood layer for various scenarios. Further analysis such as flood damage assessment can be carried out for planning and design purpose. Dhondia et al (2002) proposed that the combination of GIS software and hydraulic software will be able to speed up the process of producing flood risk map which is suitable for a decision support system.

Thus, HEC-RAS is the most popular commonly used in flood inundation mapping and among the hydraulic models. Bikram (2010) uses the HEC-RAS model to analyses the risk of flood on 1217 hectare of land of Lothar Khola, Nepal and found 229.41, 239.37, 246.75, 249.53 and 252.14 hectare of land is inundated by 2,10,50,100 and 200 year return periods of flood respectively.

3.9 Geographic information system

Consuguera et al (1993) defines Geographic Information System as a tool that can assist floodplain managers in identifying flood prone areas in their community. With GIS, geographical information is stored in a database that can be queried and graphically displayed for analysis. By overlaying or intersecting different geographical layers, flood prone areas can be identified and targeted for mitigation

The flood hazard maps coupled with the GIS technology and other information such as the extent of the flood, the number of people involved could help to establish flood relief operations for the particular river basin.

The maps could include probabilities of depths of flood inundation for risk assessment in a flood plain. With the availability of flood hazards maps, flood insurance premiums can be determined more reasonably based on the degree of flood vulnerability. Bikram (2010) uses the GIS tool to analyses the risk of flood on 1,217 hectare of land of Lothar Khola, Nepal and found 229.41, 239.37, 246.75, 249.53 and 252.14 hectare of land is inundated by 2,10,50,100 and 200 year return periods of flood respectively.

3.10 Floodplain mapping

Encroachment on floodplains, such as by artificial fill material, reduces the flood-carrying capacity, increases the flood heights of streams, and increases flood hazards in areas beyond the encroachment. One aspect of floodplain management involves balancing the economic gain from floodplain development against the resulting increase in flood hazard. For purposes of FEMA studies, the 100 year flood area is divided into a floodway and a floodway fringe. The floodway is the channel of a stream plus any adjacent flood

The floodway fringe is the area between the designated floodway limit and the limit of the selected flood. The floodway limit is defined so that encroachment limited to the floodway fringe will not significantly increase flood elevation. The 100 year flood is commonly used and a one-foot allowable increase is standard in the United States.

Plain areas that must be kept free of encroachment in order for the 100 year flood to be carried without substantial increases in flood heights. FEMA's minimum standards allow an increase in flood height of 1.0 foot, provided that hazardous velocities are not produced. The floodway fringe is the portion of the floodplain that could be completely obstructed without increasing the water surface elevation of the 100 year flood by more than 1.0 foot at any point.

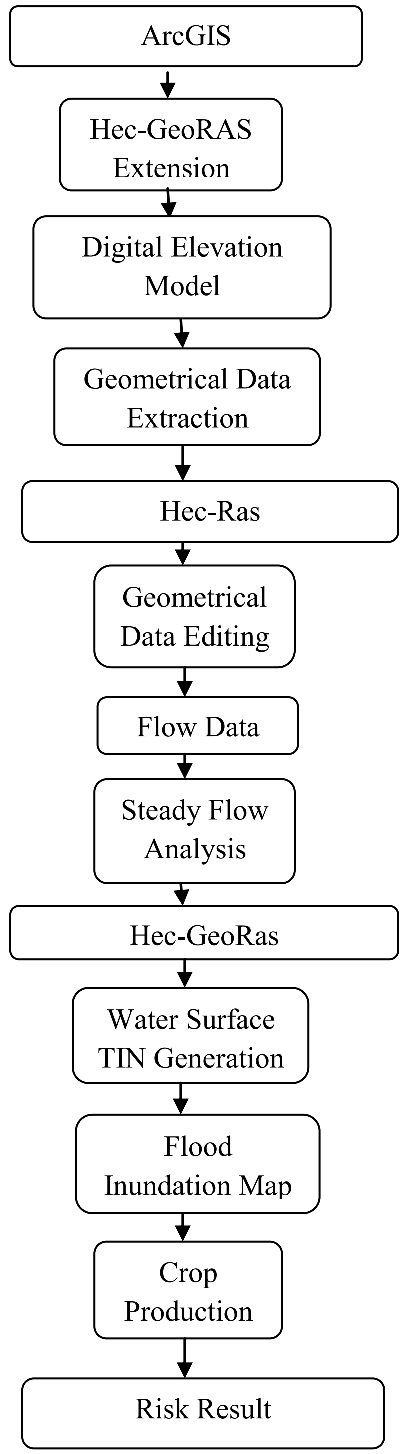
Two types of floodplain inundation maps, flood-prone area and flood hazard maps have been used.

Flood prone area maps show areas likely to be flooded by virtue of their proximity to a river, stream, bay, ocean, or other watercourse as determined from readily available information.

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

4 METHODOLOGY

4.1 Conceptual framework



4.2 Data collection

Data which are important in this study consists of flow data, cross-sectional data, manning‟s roughness coefficient of the site, production data, topographic and land use/land cover data. The flow data was collected from Ministry of Water, Irrigation and Energy at hydrology department whereas the cross-sectional data were collected from detailed survey of the Awash River on site and also by extract cross-section from Awash DEM 30m resolution. The socio-economic and Estate map that had been collected from the Illu, Sebeta Hawas and Ejerie wereda and Ministry of Water, Irrigation and Energy GIS department. The manning‟s roughness coefficient value we found from relevant literature Chow (1959). The land use/land cover, soil type and geology of the studied area were taken from GIS department of Ministry of Water, Irrigation and Energy in addition to field visit.

4.2.1. Digital elevation model (DEM)

Due to the rapid development of digitalized technologies and other related sciences, many digitalized products have been produced, among which DEM (Digital Elevation Model), is the one. This product has been widely used in hydrology, surveying and mapping, agricultural planning, soil mapping and so on. For this research the digital elevation model of the Awash River near to study area is derived from SRTM having 30m resolution was used to gives better and accurate result.

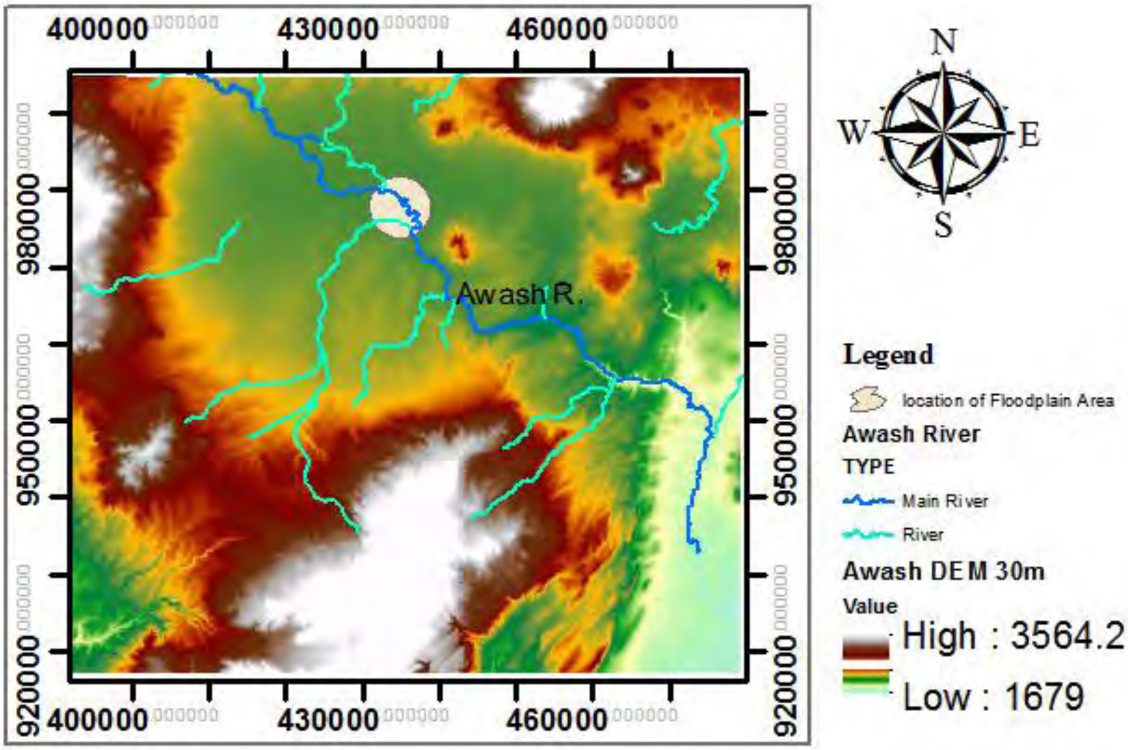


Figure 4.1: - Digital Elevation Model 30 m resolution for Awash River at study area

4.2.2. Cross-sectional data

Cross section data represent the geometric boundary of the stream. Cross sections are required at representative locations throughout the stream and at locations where changes occur in discharge, slope, and shape roughness; at locations where levees begin and end; and at hydraulic structure (bridge, culvert and weirs).The required information for a cross section consists of the river name, reach name; river station identifiers, a description x & y coordinates (stations and elevation points), downstream reach length, manning"s roughness coefficients, main channel bank stations, and contraction and expansion coefficients.

This paper were carried out detailed survey of the cross-section of the Awash River on site and by extracting the cross section with DEM 30m resolution 44 cross section point had been taken to gives better and accurate result of the site**.** The cross-section of the river was not uniform but mostly looks like irregular shape as a result its depth and width also varies from cross-section to cross-section along the downstream.

The downstream reach lengths were located at 54m to 693m interval along the river station of the Cross sections. It has minimum depth of water averagely 0.50cm - 1m depth at wide cross-sections during dry season, since the survey was carried out in two trips January and February of 2015. As morphology of the river was also important factor to the development of the flood, to investigate the cross-sectional and longitudinal profile of Awash River. Since this paper conducts analysis only Awash River having 14.5km length found on the study area so that a lot of analysis was based on the river characteristics.

4.2.3. Flow data

The flow data was used to identify the magnitude of the flood and areas inundated because of the flood. In general were required to apply different scenarios of flooding event in the model. The annual maximum flow of the study area was collected from the Ministry of Water, Irrigation and Energy. It has full data composition for the considered stations to represent the study area. Since the topic was focused to determining the flood hazard and risk assessment, by taking annual maximum values it describes more for this purpose as a result it gives better estimate.

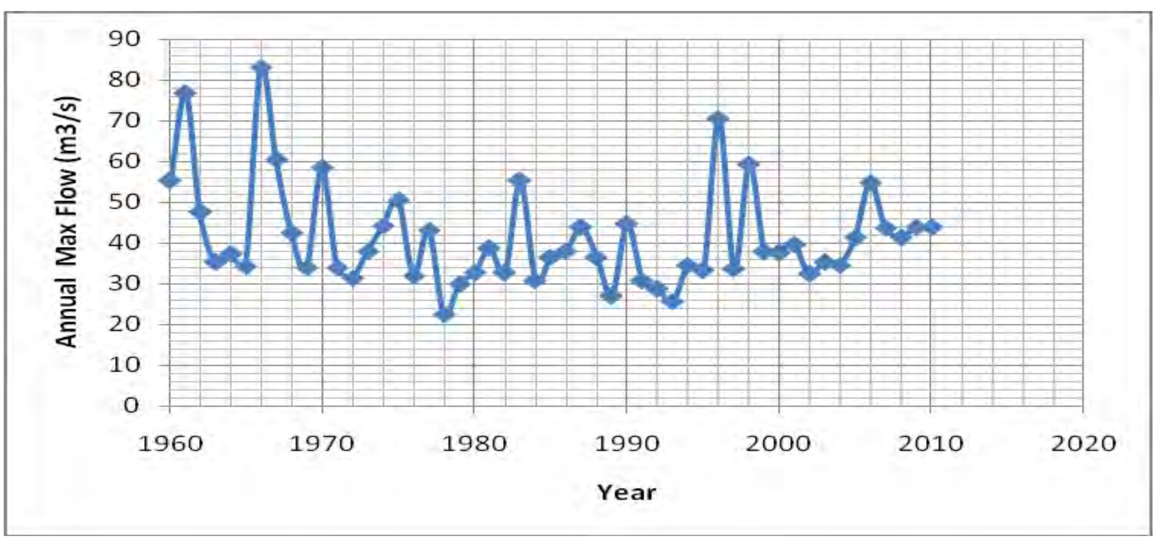


Figure 4.2: - Annual maximum flow data of Awash River at Awash Bello station.

4.2.4. Production data

This data was important to quantify the amount of risk of the flood on the crop production. This means, the inundated area has its own productivity. Therefore the amount of crop that has been produced from the inundated area was quantified in order to reach a better estimate of risk. According to Wereda Agricultural and rural development office (2014) from production department the main crop production, percent of each crop coverage and average crop yield per hectare as shown in the Table 4.1.Therefore, for this thesis the crop loss of inundated area for different return period was analysis using:-

Crop Loss = Average Crop Grown per hectare * % of Crop Coverage * Flood Inundated Area of different return period (4.1.4.1)

Table 4.1: - Crop Production in the study area

Crop	Teff	Chick Peas	Lentils	Grass Pea
(%) of Crop Coverage	50	20	20	10
Average crop yield per hectare (Quintal)	24	25	15	25

4.3 Tests on hydrologic data

The basic assumptions in statistical flood frequency analysis are the independence and stationarity of the data series and that the data come from the same distribution homogeneity and outlier, it is better to check the flow data at different significance levels to reach an accurate estimate.

4.3.1 Test of Independence and Stationarity

Wald-Wolfowitz (1943) (W-W) test was used to test for the independence of a dataset and to test for the existence of trends in it by given a sample of size N. For a dataset x1, x2....xn the statistic R was calculated.

$$R = \sum_{i=1}^{N-1} x_i x_{i+1} + x_1 x_n \qquad (4.3.1.1)$$

When the elements of the sample are independent, R follows a normal distribution with mean and variance given.

$$\bar{R} = \frac{S_1^2 - S_2}{N - 1} \qquad \text{Where } S_r = N m_r' \text{ and } m_r' = \frac{1}{n}\sum_{i=1}^{n} X_i^r$$ (4.3.1.2)

mr'' is the rth moment of the sample about the origin. (4.3.1.3)

The statistic U is approximately normally distributed with mean zero and variance unity.

Therefore in this thesis is used to test the hypothesis of independence at 5 % significance level, by comparing the statistic u with the standard normal variate $u_{\alpha/2}$ corresponding to a probability of exceedence $\alpha/2$.

$$u = (R - \bar{R})/(Var(R))^{1/2}$$ (4.3.1.4)

4.3.2 Tests of Homogeneity and Stationarity

The Mann-Whitney (1947) (M-W) test considers the quantities V and W by test two samples of size p and q with p is less than or equal to q are compared. The combined data set of size N = p + q was ranked in increasing order.

$$V = R - \frac{(P(P+1))}{2}$$
$$W = pq - V$$ (4.3.2.1)

R is the sum of the ranks of the elements of the first sample (size p) in the combined series (size N), and V and W are calculated from R, p, and q. V represents the number of times an item in sample 1 follows an item in sample 2 in the ranking. Similarly, W can be computed for sample 2 following sample 1. The M-W statistic U was defined by the smaller of V and W. When N > 20 and p, q > 3, and under the null hypothesis that the two samples came from the same population, U is approximately normally distributed with mean and variance var (U),

$$\bar{U} = \frac{pq}{2}$$ (4.3.2.2)

$$Var(U) = \left(\frac{pq}{N(N-1)}\right)\left(\frac{N^3 - N}{12} - \sum T\right) \quad (4.3.2.3)$$

$$T = \left(\frac{J^3 - J}{12}\right) \quad (4.3.2.4)$$

Where T and J is the number of observations tied at a given rank. T is summed over all groups of tied observations in both samples of size p and q.

Therefore in this thesis the statistic u was used to test the hypothesis of homogeneity at 5 % significance level α by comparing it with the standard normal variate for that significance level.

$$u = \left(\frac{U - \bar{U}}{(Var(U))^{1/2}}\right) \quad (4.3.2.5)$$

4.3.3 Testing of Outliers

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

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

$$\mathbf{Y_h = y + K_n S_y} \quad (4.3.3.1)$$

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

The Kn values are used in one-sided tests that detect outliers at the 10% of significance level in normally distributed data.

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

A similar equation can be used to detect low outliers:

$$\mathbf{Y_L = y - K_n S_y} \tag{4.3.3.2}$$

Where: - Yl is the low outlier threshold in log units.

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

4.4 Extreme value distribution

Measured stream flow can be analyzed by statistical methods. This method produces a probabilistic statement about the future occurrence of a stream flow event of specific magnitude. Assumption of this method assumes there exists reliable representative sample of the universe or population of stream flow data (no watershed or climate changes). It also assumes the events are random and independent of each other. To obtain both a consistent and accurate estimate requires development, acceptance, and widespread application of a uniform, consistent and accurate technique for determining flood-flow frequencies is important.

Extreme values are selected maximum or minimum values of sets of data. For example, the annual maximum discharge at a given location is the largest recorded discharge value during a year, and the annual maximum discharge values for each year of historical record make up a set of extreme values that can be analyzed statistically.

Distributions of the extreme values selected from sets of samples of any probability distribution have been shown by Fisher and Tippett (1928) to converge to one of three forms of extreme value distributions, called Types I, II, and III, respectively, when the number of selected extreme values is large. The properties of the three limiting forms were further developed by Gumbel (1941) for the Extreme Value Type I (EVI) distribution, Frechet (1927) for the Extreme Value Type II (EVII), and Weibull (1939) for the Extreme Value Type III (EVIII).
The Gumbel Distribution is the most widely used probability distribution function for extreme values in hydrologic and meteorological studies for prediction of flood peaks and maximum rainfalls (Subramanya, 1994).The extreme value distribution (EV I) of the flood data is recommended as the basic distribution for defining the annual flood series.

The Extreme Value Type I (EVI) (Gumble‟s distribution) probability distribution function is given by:-

$$F(x) = Exp[-\exp\left(-\frac{x-\mu}{\alpha}\right)] \qquad (4.4.2)$$

The parameters μ and α are determined by the method of moments as the following formula:

$$\alpha = \frac{\sqrt{6}}{\pi} S \qquad (4.4.3)$$

$$\mu = \bar{x} - 0.5572\alpha \qquad (4.4.4)$$

The magnitude XT (quantile estimate) of a hydrologic event is therefore represented as

$$x_T = \bar{x} + K_T s \qquad (4.4.5)$$

Where KT is the frequency factor and is given by

$$K_T = -\frac{\sqrt{6}}{\pi}\left\{0.5772 + \ln\left[\ln\left(\frac{T}{T-1}\right)\right]\right\} \quad (4.4.6)$$

4.5. HEC-GeoRAS

According to the USACE (2010) HEC-GeoRAS is a set of ArcGIS tools specifically design to process geospatial data for use with the (HEC-RAS). In ArcGIS using a graphical user interface (GUI). The extension allows users with limited GIS experience to create an HEC-RAS import file containing geometric attribute data from an existing digital terrain model (DTM) and complementary data sets. The interface allows the preparation of geometric data for import into HEC-RAS and processes simulation results exported from HEC-RAS. To import file is create from data extracted from data sets (ArcGIS) and digital terrain model (DTM), of the river system represent TIN or GRID format.

The user creates a series of line themes pertinent to developing geometric data for HEC-RAS. The themes created are the Stream Centerline, Flow Path Centerlines (optional), Main Channel Banks (optional), and Cross Section Cut Lines referred to as the RAS Themes. Additional RAS Themes may be used to extract additional geometric data for imports in HEC-RAS these themes include Land Use, Levee Alignment, Ineffective Flow Areas, and Storage Areas. Water surface profile data and velocity data exported from HEC-RAS simulations may be processed by HEC-GeoRAS for GIS analysis for floodplain mapping, flood damage computations, ecosystem restoration, and flood warning response and preparedness.

4.6. HEC-GeoRAS Tools

As shown in Fig.4.3 the HEC-GeoRAS toolbar has four menus such as RAS Geometry, RAS Mapping, ApUtilities, and Help and seven tools (Assign River name, Reach name, Assign from Station to Station, Assign Line Type, Construct XS Cutline, Plot Cross Section, and Assign Levee Elevation). The RAS Geometry menu contains functions for pre-processing of GIS data for input to HECRAS. The RAS Mapping menu contains functions for post-processing of HEC-RAS results to produce Water surface TIN and floodplain mapping using raster. The ApUtilites menu contains functions mainly for data management. The Help menu is self-explanatory.

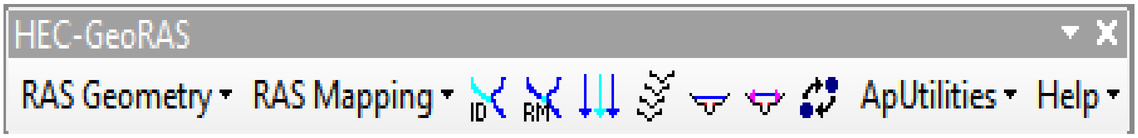


Figure 4.3:- Hec-GeoRAS Extention

4.7. Geometric data Pre-RAS processing

First of all the Awash main river and the study area should be digitized or extract from DEM 30m and overlay to the wereda map shape file to get the appropriate feature class, so that some correcting measurements like overlapping with the wereda map of the study should be done.

The geometry file for HEC-RAS contains information on cross-sections of the river, hydraulic structures, river banks and other physical attributes of river channels. The pre-processing using HEC-GeoRAS involves creating these attributes in GIS, and then exporting them to the HEC-RAS the geometry file. In HEC-GeoRAS, each attribute is stored in a separate feature class called as RAS Layer.

The HEC-GeoRAS creates a geodatabase in the same folder where the map document is saved, gives the name of the map document to the geodatabase (Illu floodplain.mdb), and stores all the feature classes or RAS layers in this geodatabase. After creating RAS layers, these are added to the map document with a pre-assigned symbology. Since these layers are empty, the task is to populate these layers depending on the project needs, and then create a HEC-RAS geometry file. Because of the topographic feature of the study area as a flat terrain and the whole as cultivation, the only performing activities of the geometric data layer here is creating and digitizing the main river, the bank stations, flow paths and cross sections across the cultivated area.

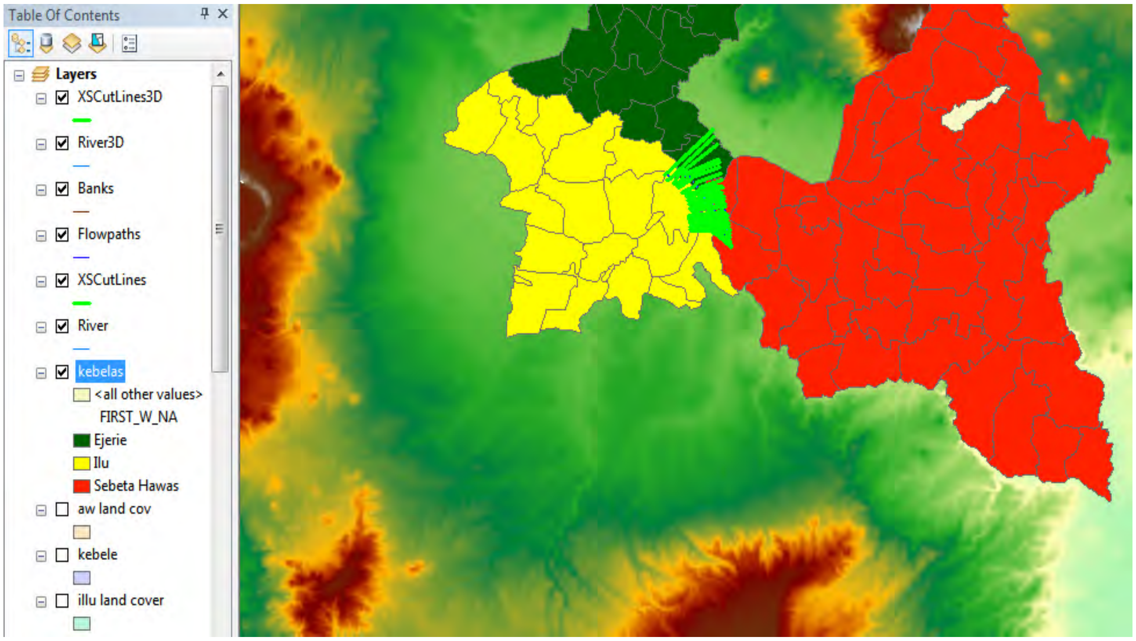


Figure 4.4: - Ras Layer with DEM 30m resolution of Awash River at study area.

4.8 HEC-RAS

HEC-RAS is a hydraulic model developed by the Hydrologic Engineering Center (HEC) of the U.S. Army Corps of Engineers. Hydrologic Engineering Center‟s River Analysis System (HEC-RAS) is the software predominately used in the field of hydraulic analysis for floodplain delineation. The HEC-RAS program, like the other software, it can be downloaded free of charge from the Hydrologic Engineering Center's.

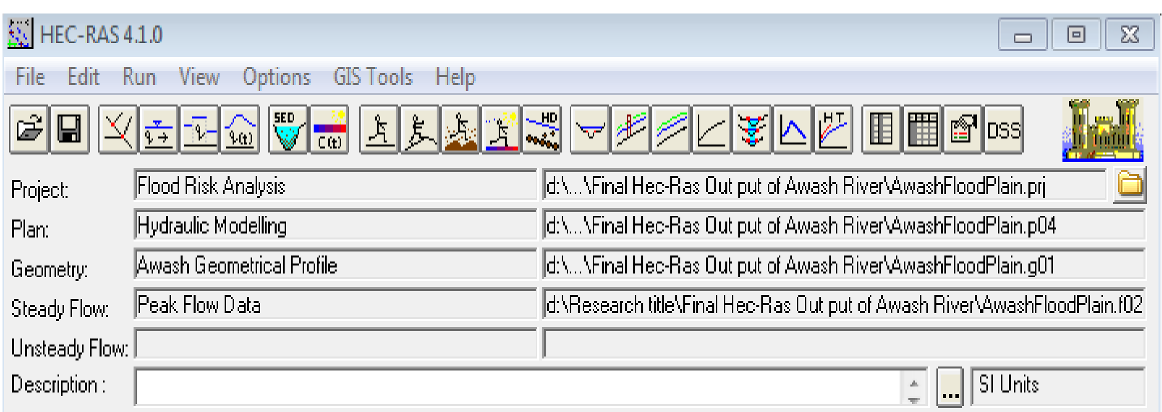


Figure 4.5: - HEC-RAS window of Awash River at study area

The model is used for determination of water surface profiles for different flow scenarios, is intended for steady flow water surface profile computations and unsteady flow simulation; perform sediment transport simulation and perform water quality simulation. The system is capable of modeling subcritical, supercritical, and mixed-flow regimes for streams consisting of a full network of channels, a dendrite system, or a single river reach.

For each HEC-RAS project, there are three required components:

- Geometry data,
- Flow data, and
- Plan data.

The Geometry data, for instance, consists of a description of the size, shape, and connectivity of stream cross-sections. Likewise, the Flow data contains discharge rates. Finally, Plan data contains information pertinent to the run specifications of the model, including a description of the flow regime.

4.8.1 Importing and editing geometric data

4.8.1.1 Geometrical profile

The Geometric data has been imported from GIS to the HEC-RAS but the geometric data processed may not appropriate with the actual data, so that it is necessary to edit them in HEC-RAS software. Therefore before any proceed, it is good to perform a quality check on the data to make sure no erroneous information is imported from GIS. Therefore cross-section data editor should be displayed with the downstream reach lengths (left over bank, right over bank and the channel), the manning‟s n value for the channel, main channel bank station (left and right bank) and the contraction and expansion coefficient of the channel should be edit and entered new data.

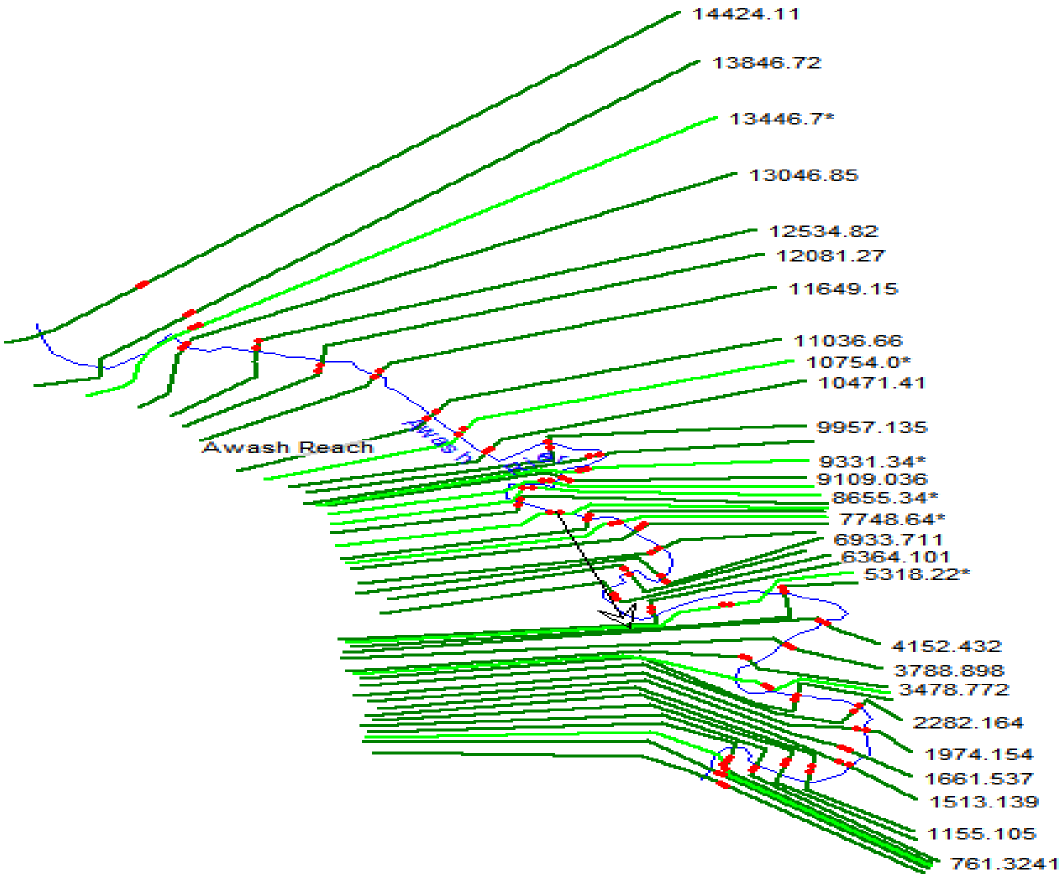


Figure 4.6:- Geometrical profile in Hec-Ras imported from Hec-GeoRas

4.8.1.2 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 general effects of roughness on flow in a channel are similar to those for overland flow.

Cowan method is potentially more accurate method is required for a number of hydraulic computations. It is a necessary input in floodplain delineation, also, a number of methods for estimating the timing of runoff use n as an input and used in the design of stable channel systems.

The "n" values were assigned based on the characteristics of upper Awash River channel and floodplain area. Each characteristic was given n-value based on table values for similar conditions (Cowan, 1956) and engineering judgment using field survey photos of the study area.

Figure 4.7:-Upper Awash River channel and floodplain profile of the study area.

This method is basically a table look-up solution, with the basic n value and each of the corrections obtained from a table.

The method involves the selection of a base value of n and then correcting the base value based on the following five factors:

1. The degree of regularity of the surfaces of the channel cross section.
2. The character of variations in the size and shape of cross sections.
3. The presence and characteristics of obstructions in the channel.
4. The affect of vegetation on flow conditions.
5. The degree of channel meandering.

Cowan (1956). The manning‟s roughness coefficient value and field survey photos of upper Awash River channel as shown in Annex E #3.The general procedure for estimating n involves first, the selection of a basic value of n for a channel and floodplain materials involved, then, through critical consideration of the factors listed above, the selection of a modifying value associated with each factor. The modifying values are added to the basic value to obtain n for the channel under consideration

Step 1: Selection of basic value (n_1).

The selection of a basic n value of the study area is assumed that for the character of a bottom and sides of the channel and floodplain are of different materials involved.

Step 2: Correction for channel irregularity (n_2).

The modify value n of the study area is assumed that the degree of roughness or irregularity of the channel surfaces of sides and bottom is good dredged channels; slightly eroded or scoured side slopes of canals or drainage channels.

Step 3: Correction for cross-section variation (n_3).

The modifying value n of the study area is assumed that the character of variations of size and shape of cross sections is large and small sections alternating frequently or shape changes causing frequent shifting of main flow from side to side these changes causing the greatest turbulence.

Step 4: Correction for obstructions (n_4).

The selection of modifying value n for obstructions is based on the presence and characteristics of obstructions such as debris deposits, stumps, exposed roots, boulders, fallen and lodged logs. But in these step care should be taken that conditions considered in other steps are not re-evaluated or double-counted for both of channel and floodplain the obstruction is negligible.

Step 5: Correction for vegetation (n_5)

The retard effect of vegetation is probably due primarily to the turbulence induced as the water flows around and between the limbs, stems and foliage, and secondarily to reduction in cross section. As depth and velocity increase, the force of the flowing water tends to bend the vegetation. Furthermore, the amount and character of foliage; that is, the growing season condition versus dormant season condition is important. Therefore, modify value n of the study area is assumed that the effect of vegetation for channel is low and floodplain is high

Step 6: Correction for channel meandering (n_6).

The modify value n of the Awash River is assumed that the degree of meander is appreciable channel or can be measured using the ratio of the total meander length of the channel in the reach (lm) to the straight length of the reach (ls).

7th step Computation of n for the reach.

The actual value of the reach roughness coefficient equals the sum of the values of the basic value n_1 and the modifying values n_2 to n_6. The calculations can be performed on the computation sheet see from the table 4.2.

Table 4.2:-Summary computation sheet for manning's roughness coefficient

Variable	Description Alternatives	Recommended Value	Actual Value of Channel	Actual Value of Floodplain
Basic $\mathbf{n_1}$	Earth	0.02	n_1= 0.022	n_1= 0.02
	Rock	0.025		
	Fine gravel	0.024		
	Coarse gravel	0.028		
Irregularity $\mathbf{n_2}$	Smooth	0.000	n_2= 0.005	n_2= 0.005
	Minor	0.005		
	Moderate	0.010		
	Severe	0.020		
Cross section $\mathbf{n_3}$	Gradual	0.000	n_3= 0.0125	n_3= 0.000
	Occasional	0.005		
	Alternating	0.010 - 0.015		
Obstructions $\mathbf{n_4}$	Negligible	0.000	n_4= 0.000	n_4= 0.000
	Minor	0.010 - 0.015		
	Appreciable	0.020 - 0.030		
	Severe	0.040 - 0.060		
Vegetation $\mathbf{n_5}$	Low	0.005 - 010	n_5= 0.0075	n_5= 0.0375
	Medium	0.010-0.020		
	High	0.025 - 050		
	Very high	0.050 - 0.100		
Meandering $\mathbf{n_6}$	Minor	0.000	n_6= 0.00705	n_6= 0.000
	Appreciable	0.15***ns**		
	Severe	0.30***ns**		
Total Reach **n**			n= 0.054	n= 0.0625

Therefore, the total reach "n" values were obtained for main channel of upper Awash River and floodplain. Finally, were performed the computation sheet calculation of a manning roughness coefficient value of 0.054 for the main channel and 0.0625 for the lift and right bank of the floodplain.

4.8.1.3 Expansion and Contraction coefficient

According USACE, (2010) the contraction and expansion coefficient for natural channel which has gradual change in cross-section is estimated to be 0.1 and 0.3 respectively. By understanding this concept, since Awash River is a natural channel, it has the same value with stated above. Even if Awash River is not straight but it meander nature with irregular shape in cross-section and when coming to its longitudinal profile, it is gradually varies. Therefore, to perform flow calculation in the channel, 0.1 was used for contraction coefficient where as 0.3 was used for expansion coefficient.

4.8.2. Entering and editing flow data

Flows are typically defined at the most upstream location of river. After analysis of the flow data recorded for the site found in the Ministry of Water, Irrigation and Energy, those should be used as an input for the software. Each flow that needs to be simulated is called a profile in HEC-RAS. For carrying out the analysis here, the peak flood having 2, 5, 50, 100 and 500 year return period of flow was used.

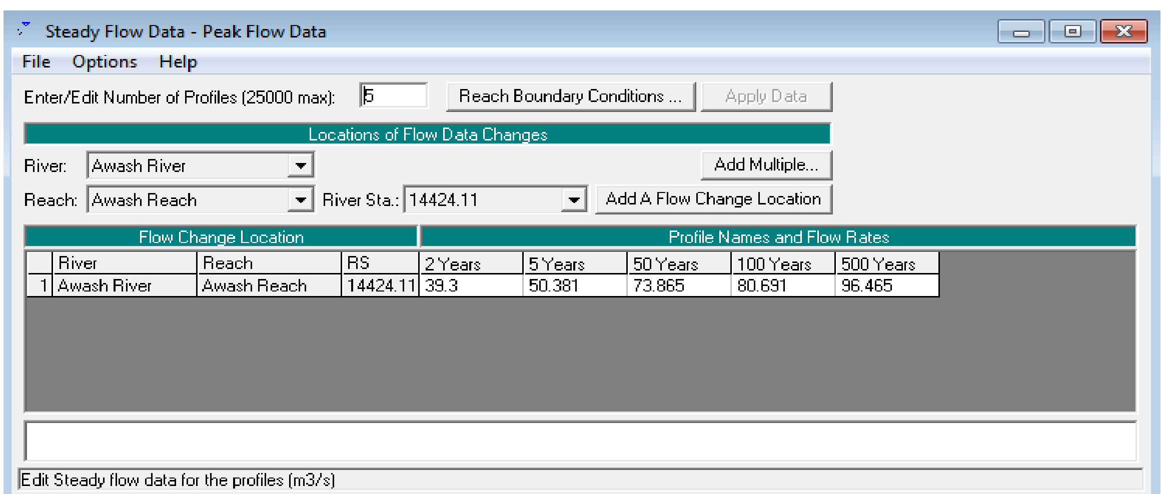


Figure 4.8: - Steady Flow Analysis interface.

4.8.3. 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 namely:

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

So that for this thesis normal depth had been selected. A normal depth should be calculating for upstream and downstream end profile of the study area the slope of the channel bottom was 0.00135 and 0.0007

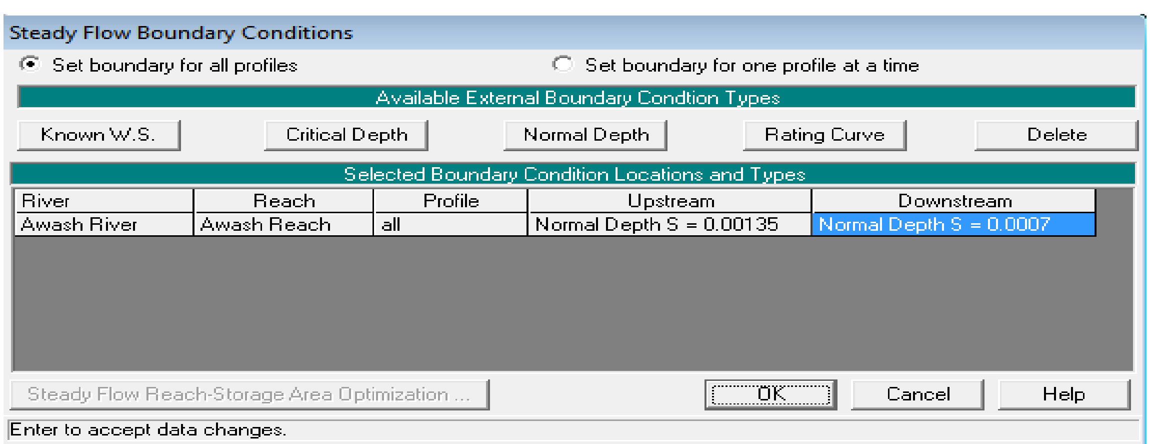


Figure 4.9: - Boundary Condition in Steady Flow.

4.8.4 Steady flow simulation

Simulate the model, with the type of flow to be calculated should be distinguished. So that for this paper, the steady flow analysis was selected. Not only the type of flow but also the regime of flow should be identified. Therefore in this case the mixed flow regime was selected because it starts flow calculation from upstream and downstream ends of the river system.

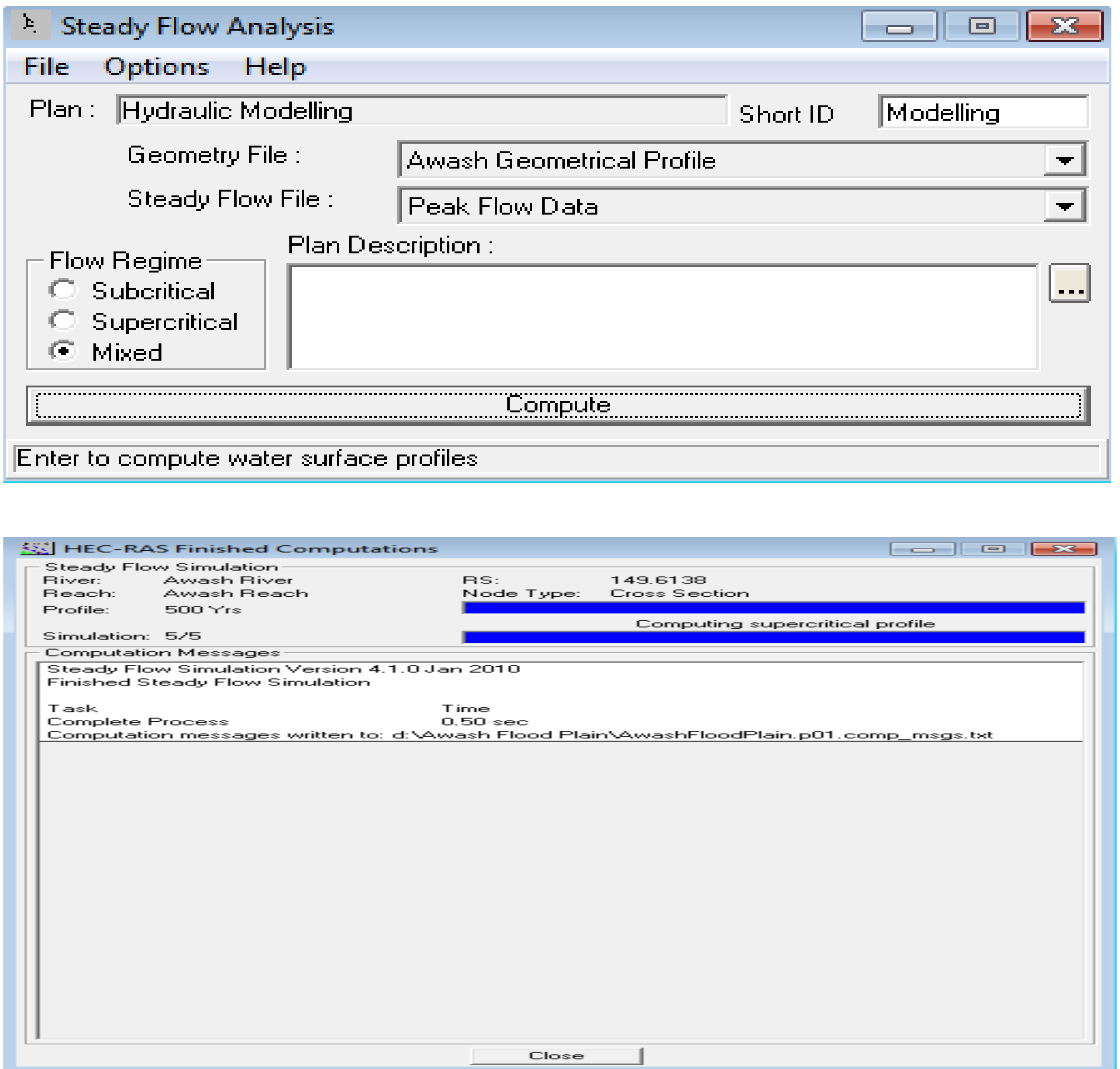


Figure 4.10: - Steady flow simulation in HEC-RAS

4.9 Post-RAS processing of geometric data

After successful simulation, exported the HEC-RAS results to ArcGIS to post-processing them, as a result flood inundation mapping should be produced for the different return period to get hazard extent and finally the risk has been estimated.

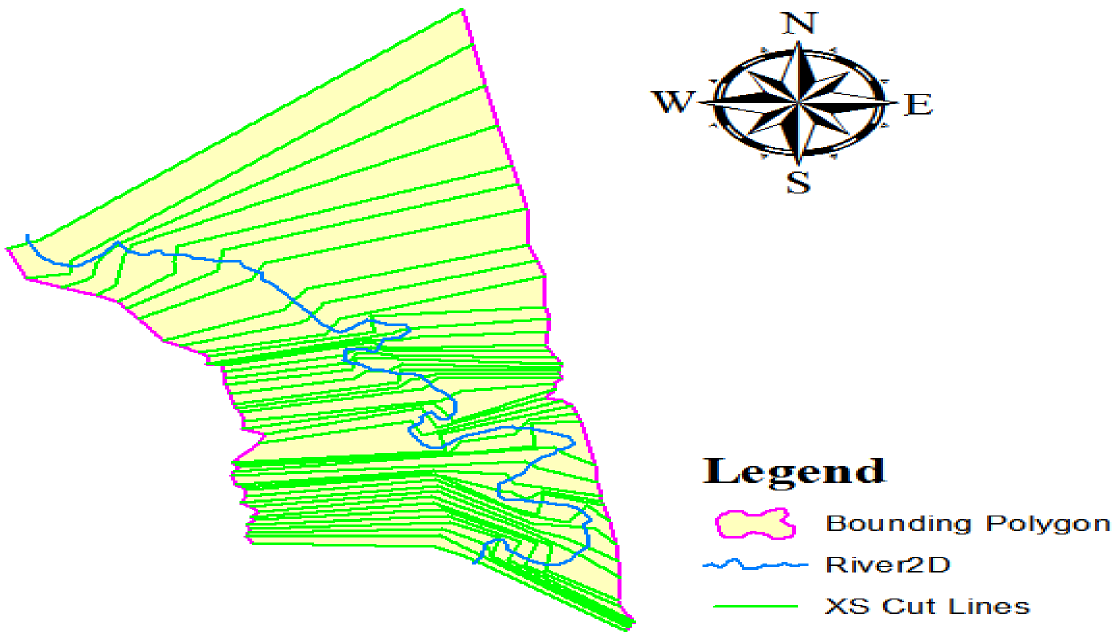


Figure 4.11:- Bounding Polygon in HEC-GEORAS imported from the HEC-RAS

4.10 Flood risk assessment

First of all, it is important to identify the low-lying areas in danger of flooding. This requires a digital elevation model (DEM) with a sufficiently fine grid resolution to resolve the river channel geometry (e. g., across channel width and depth), which predicts the inundation areas based on the transient hydrological variables. In practice, setting up such models is not easy; they require extensive data including stream channel geometry. Although inundation areas are expected to vary along the river course and reach, and localization of rainfall event, for a first approximation the areas nearest to the stream channel can be identified from SRTM DEM. At the continental and even on the regional scale, SRTM DEM is of a very fine resolution.

The flood risk assessment developed by Gilard (1996) who demonstrates its own methodology describes, the flood risk is divided into the hazard component and its probability. The results of these two analyses are combined for the flood risk assessment. The vulnerability assessment is facilitated by the use of the binary model, based on the presence or absence of a flood of particular intensity in a particular land use type. This risk assessment process is automated by the use of customized graphical user interface in the Arc-GIS.

Also according to Chang et al (2003) the assessment and quantification of flood risk will be based on the standard formulation as a product of hazard (natural event) and its probability of occurrence.

The flood risk maps are prepared by overlaying the flood extent grids with the map. Because almost all portion of the risk area is flat covered by different crop, then if the flood expands, it distributed to the two banks.

Therefore, the risk is now estimated as the amount of crop production from the land in terms of quintal. Hence the cost of the crop is fluctuating through time; it is difficult to estimate the value of crop in terms of money; So that it is better to quantify the risk in terms of crop production.

4.10.1. Flood hazard analysis

Hazard level may be defined by the parameters like flood depth, magnitude of flood and accidence probability of the flood event. For the quantification of the flood hazard and potential of damage, magnitude of the flood is a determining parameter. According to Gillard (1996), the hazard aspect of the flood risk is related to the hydraulic and the hydrological parameters. For this weighted spatial coexistence model facilitates the analysis by ranking the hazard level in terms of magnitude of the flood. That is a flood having high magnitude, its hazard level becomes also high where as a flood having small magnitude have small hazard, Also Cheng et al. (2003) proposed that flood hazard identification and assessment are based on information on the intensity and the frequency of flood events. Usually the spatial distribution of both flood flow depth and flow velocity has to be considered.

In this study, the flood flow depth was selected as indicator of flood identification and assessment. Dynamics of inundation flow on lowland area can be modeled as a shallow flow where the velocity distribution is integrated over the vertical direction. Therefore the definition of flood hazard is based on two-dimensional hydraulic model (flood inundation model) for the assumed flood events with the various return periods flood hazard map should be produced with GIS.

5. RESULTS AND DISCUSSIONS

5.1 Geometrical Ras layer

5.1.1. Stream centerline

The river and reach network is represented by the stream centerline layer. The network is created on a reach basis, starting from the upstream end and working downstream following the Awash River channel. Each reach is comprised of a river name and reach name. The Stream Centerline layer is used for assigning river station to cross section and to display as a schematic in the HEC-RAS geometric editor.

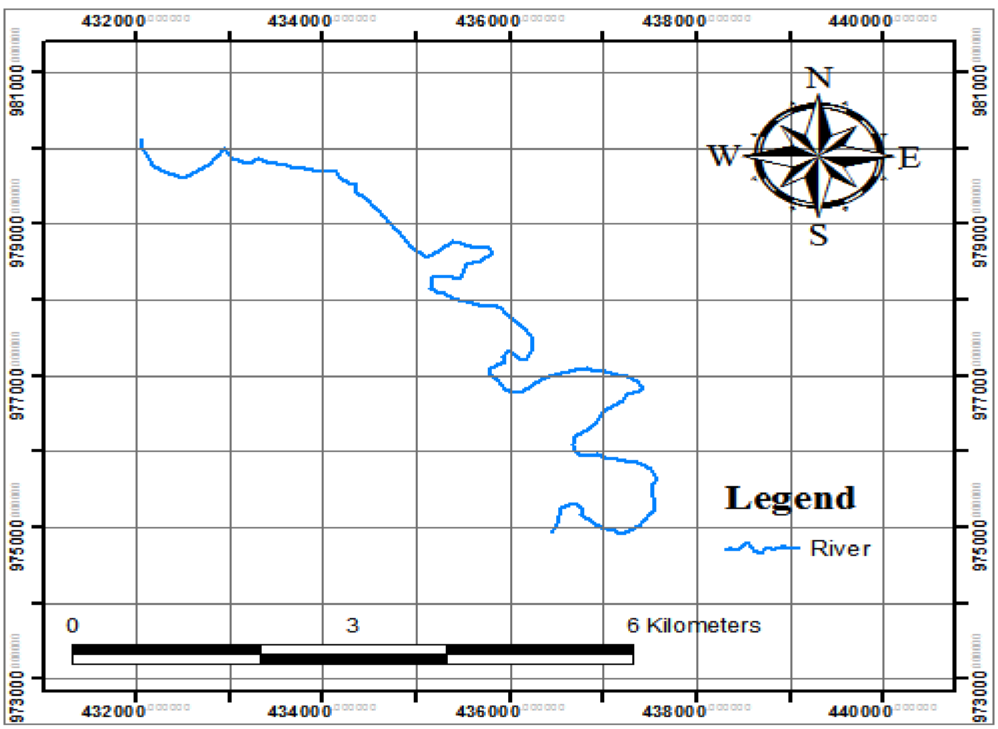


Figure 5.1: - River centerline of Awash River

5.1.2. Bank lines

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

In general, the bank lines will serve three purposes,

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

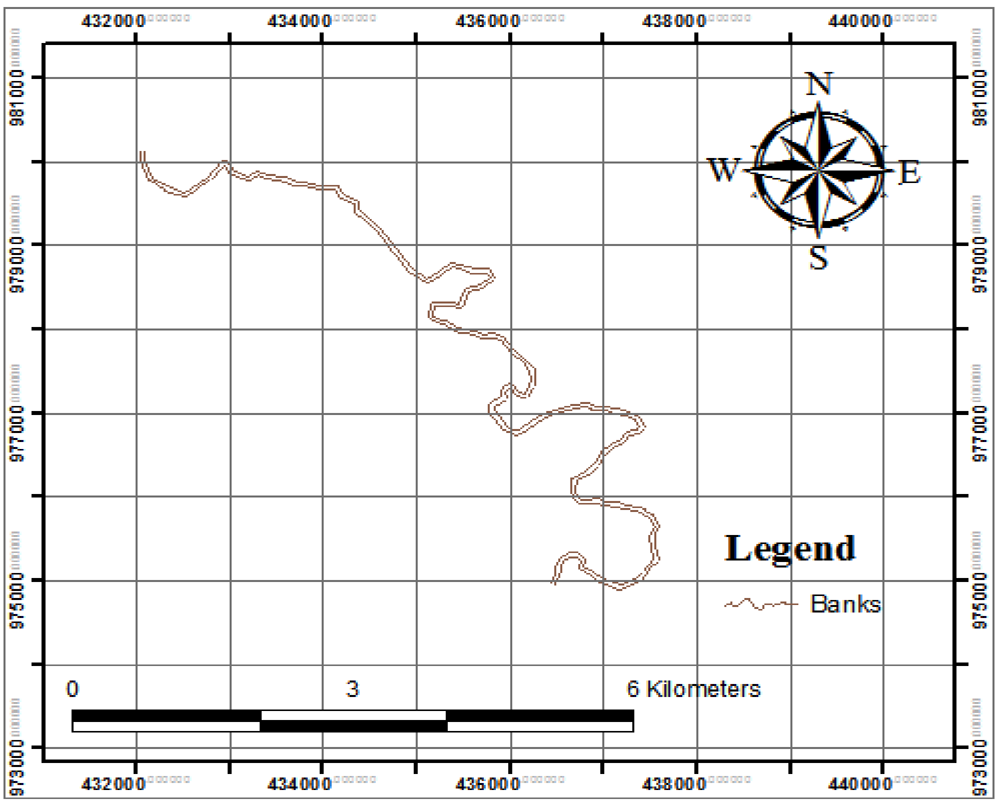


Figure 5.2: - Schematization of Bank lines of Awash River

5.1.3. Flow paths center lines

The flow path centerlines layer is used to identify the hydraulics flow path in the, left overbank, main channel and right over bank by identifying the center of mass of flow in each region. Further creating the flow path centerlines layer will assist you in properly laying out the cross sections cut lines. If the stream centerline layer already exists you have the option to copy stream center line for the flow path in the main channel. Flow path are created in the direction of flow from upstream to downstream. The flow path lines are used to determine the downstream reach lengths between cross sections in the main channel and over bank areas.

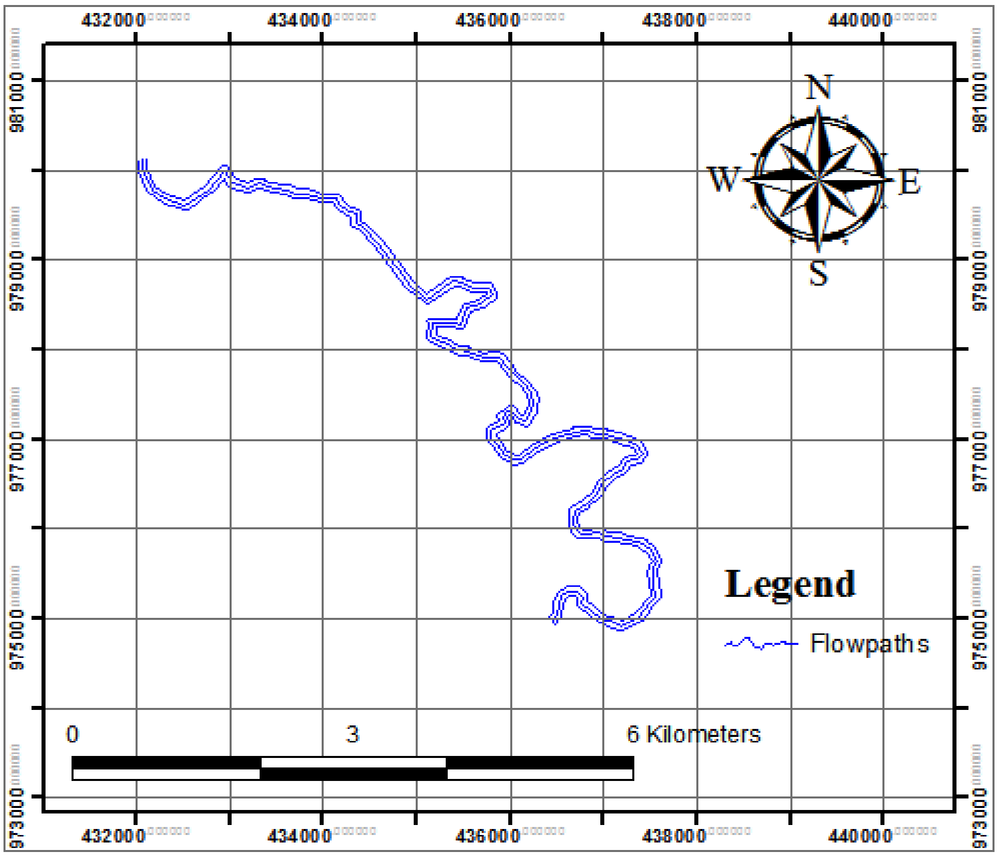


Figure 5.3: - Schematization of Flow path of Awash River

5.1.4. Cross-section cut lines

Cross-sections are one of the key inputs to HEC-RAS. Cross-section cut lines are used to extract the elevation data from the terrain to create a ground profile across channel flow. The intersection of cut lines with other RAS layers such as stream centerline and flow path lines are used to compute HEC-RAS attributes such as bank stations (locations that separate main channel from the floodplain), downstream reach lengths (distance between cross-sections) and Manning‟s coefficient, n. Therefore, creating adequate number of cross-sections to produce a good representation of channel bed and floodplain is critical.

Certain guidelines must be followed in creating cross-section cut lines:

- They are digitized perpendicular to the direction of flow;
- Must span over the entire flood extent to be modeled; and
- Always digitized from left to right (looking downstream).

Even though it is not required, but it is a good practice to maintain a consistent spacing between cross-sections.

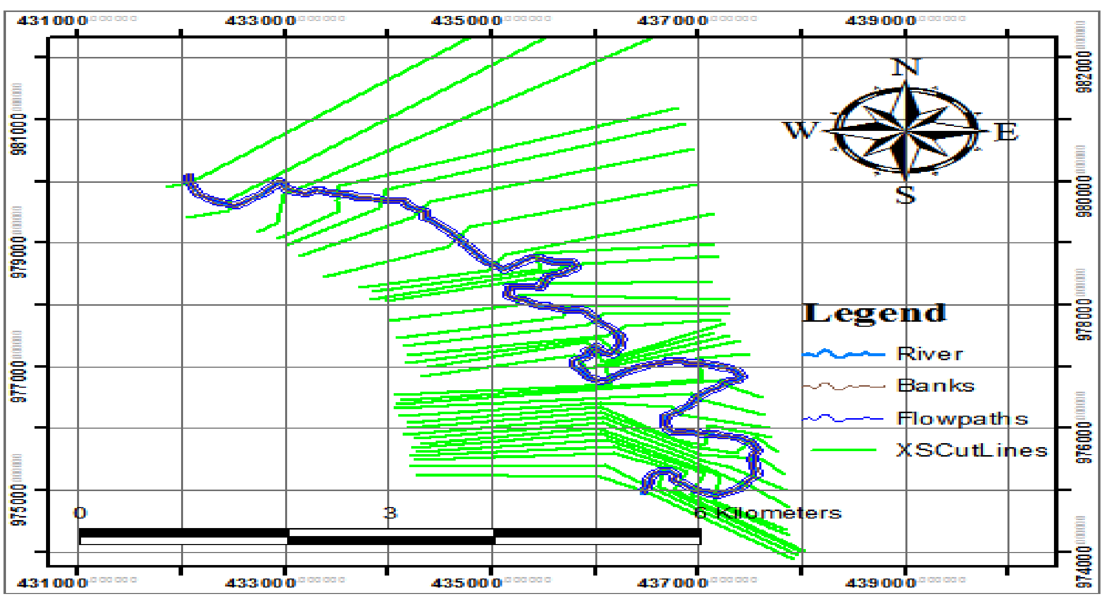


Figure 5.4: - Cross-sectional cut line of Awash River

5.2. Tests on hydrologic data

The basic assumptions in statistical flood frequency analysis are the independence and stationarity of the data series and that the data come from the same distribution.

5.2.1 Outlier test

Because the observed annual maximum flow, 83.12 m^3/s, was found to be less than high outlier threshold value, 84.69 m^3/s. and the lowest observed discharge, 22.39 m^3/s, was greater than the lower outlier threshold value, 18.70 m^3/s. Therefore, there are no high and low outliers at 10% significance level in the annual maximum flow data from 1960 to 2010 of Awash Bello stream flow gauging station.

5.2.2 Stationary and Independence (W-W) Test

The test value u = 1.22 is less than the critical value at 5% significance level, $u_{0.025}$ =1.96. Therefore we can accept the hypothesis of independence and stationarity of Awash River flow data at 5% significance level.

5.2.3 Homogeneity and Stationary (M-W) Test

The test value u = 0.0049 is less than the critical value at 5% significance level, $u_{0.025}$ = 1.96. Therefore we can accept the hypothesis of homogeneity and stationary of Awash River flow data at 5% significance level.

Therefore, the flow data found at Awash Bello station is independent, homogenous and stationary at 5% significance level.

5.3. Quantile estimate

After the parameters of a distribution were estimated with annual maximum flow data using method of moments, quantile estimates (Xtr) which correspond to different return periods were computed by Extreme Value Type I (Gumbel) method.

Table 5.1:- Quantile estimates for different return periods of the Awash River at the station.

Return Period (Year)	2	5	50	100	500
Mean (Xm)	41.36	41.36	41.36	41.36	41.36
Standard deviation (Sx)	12.54	12.54	12.54	12.54	12.54
Frequency Factor (Ktr)	-0.16	0.72	1.31	2.59	3.14
Quantile Estimate (Xtr) (m3/s)	39.30	50.38	73.87	80.69	96.47

5.4 Flow duration curve

Flow-duration curve, gives the percentage of time during which any selected discharge may be equaled or exceeded. The shape of the flow duration curve of upper Awash River gives a good indication of a catchment‟s characteristic response to its average rainfall history.

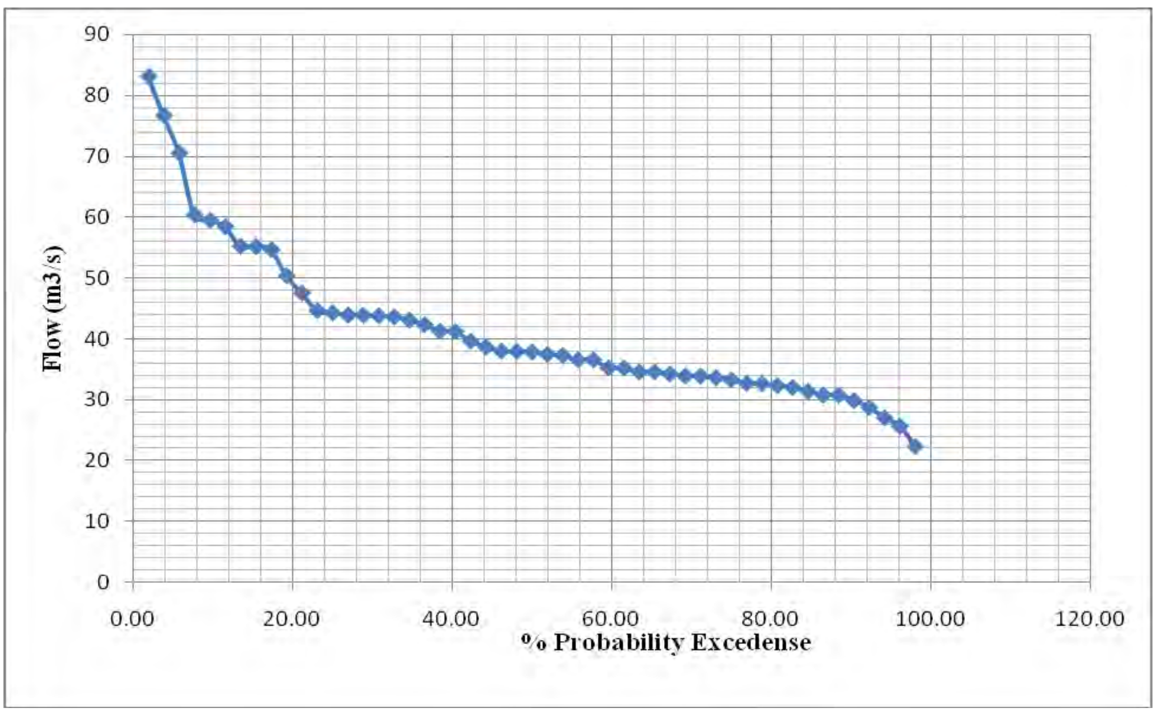


Figure 5.5: Flow duration curve of Awash River at the station.

Flow duration curves of upper Awash River that has a very flat slope indicate little variation in flow regime, the resultant of the damping effects of large storages and steeply sloped curve results from a very variable discharge, usually from small catchments with little storage where the stream flow reflects directly the rainfall pattern.

5.5. HEC-RAS outputs

Representative graphical outputs of HEC-RAS model are shown in Figs 5.6, 5.7 and 5.8. The complete graphical and tabular outputs are indicated in Annex A.

5.5.1. Cross-sectional view

The sample cross-sections at stations 14424.11 and 8181.39* are shown in Figs. 5.6, and 5.7, respectively. The 2 and 5 year return period floods 39.30 m^3/s and 50.38 m^3/s respectively have not brought significant change because of depth of the cross-section. However, the 50, 100 and 500 year return period floods have higher magnitudes of 73.865, 80.691 and 96.465 m^3/s, respectively; these discharges completely fill the whole cross-section and inundate the adjacent floodplains.

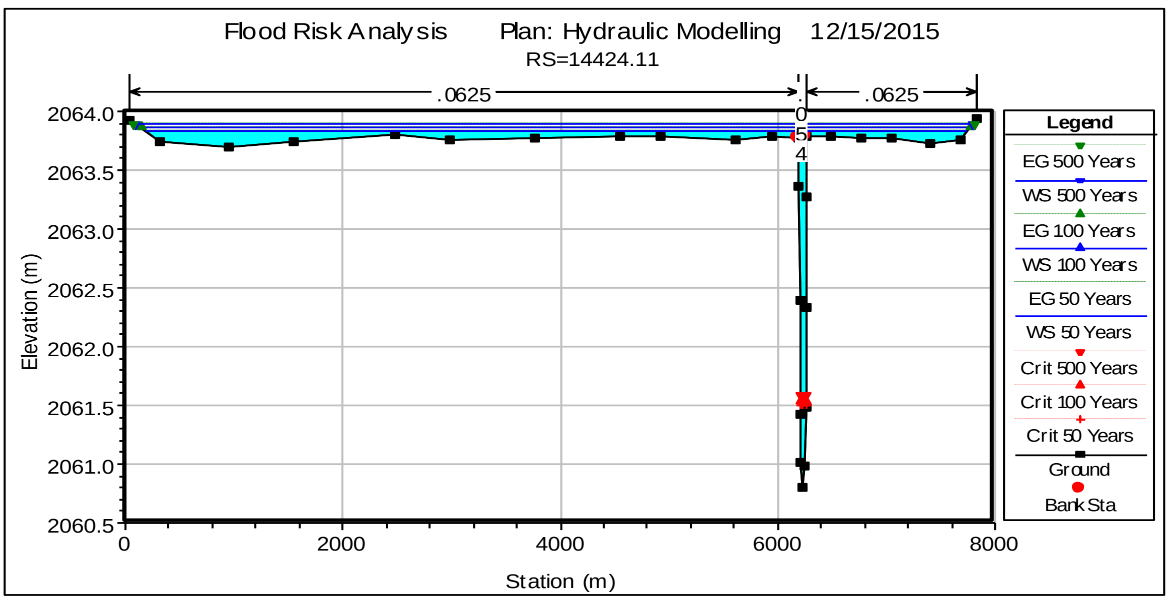


Figure 5.6: - Cross-sections view at river station 14424.11 of Awash River

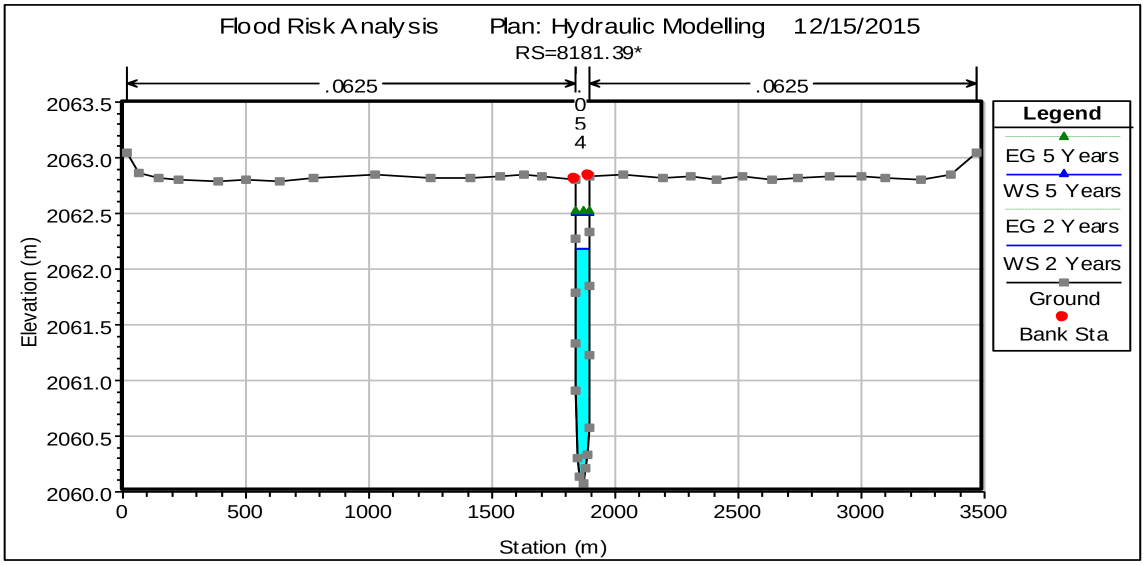


Figure 5.7: - Cross-sections view at river station 8181.39* of Awash River.

5.5.2. Water surface profile

The water surface profiles of the different return periods, i.e. 500, 100, 50, 5 and 2 years, are shown in Fig. 5.8. The water surface profile for each return period is shown in Annex A#2.

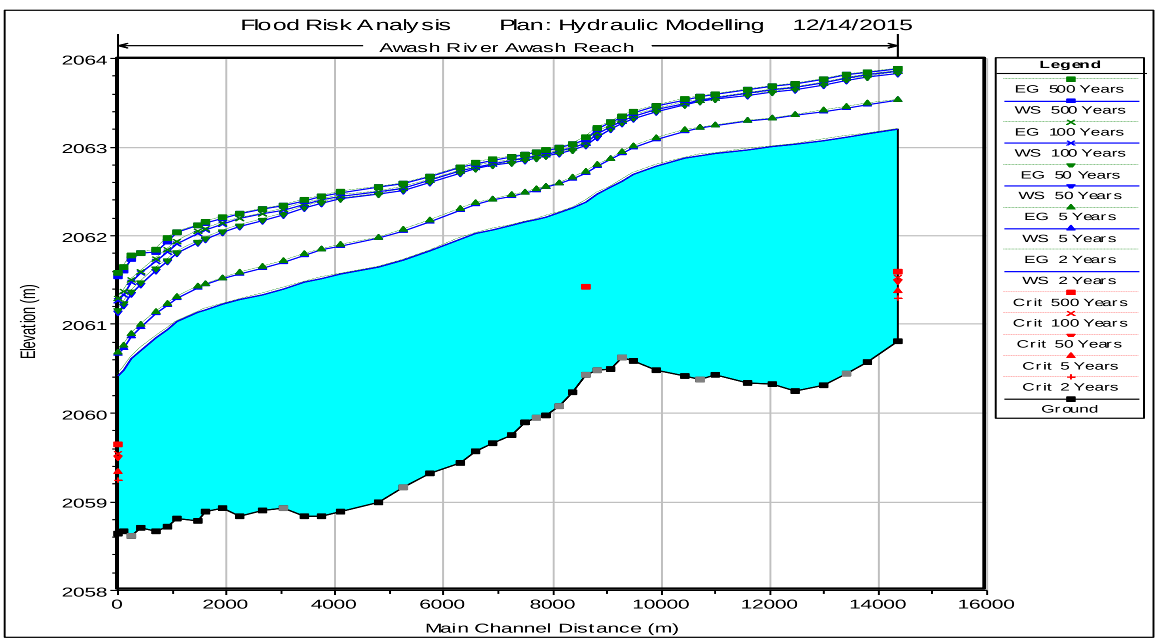


Figure 5.8: - Water surface profile for all return periods of Awash Reaches.

5.6 Flood inundation mapping

5.6.1 Water surface TIN generation

One water surface TIN was created for each selected water surface profile. The TIN was created based on the water surface elevation at each cross section and the bounding polygon data specified in the RAS GIS export file. The water surface TIN was generated without considering the terrain surface. The water surface TIN created was named as concatenation of "t" and the water surface profile name (e.g. t 500 years.) and was saved into the output directory specified in the layer setup. After a water surface TIN has been created with water surface elevation for the selected profile, now it is added to the map.

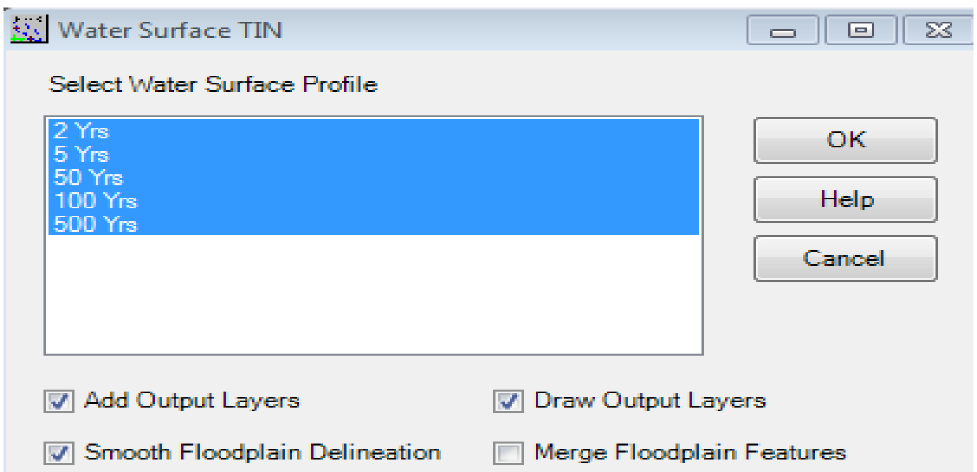


Figure 5.9: - Water surface profile selection in HEC-GEORAS

As shown in Fig. 5.10 the Arc-GIS triangulation method creates the surface TIN using cross-sectional cut lines and connecting the outer points of the bounding polygon, so that the TIN were included area outside the possible inundation.

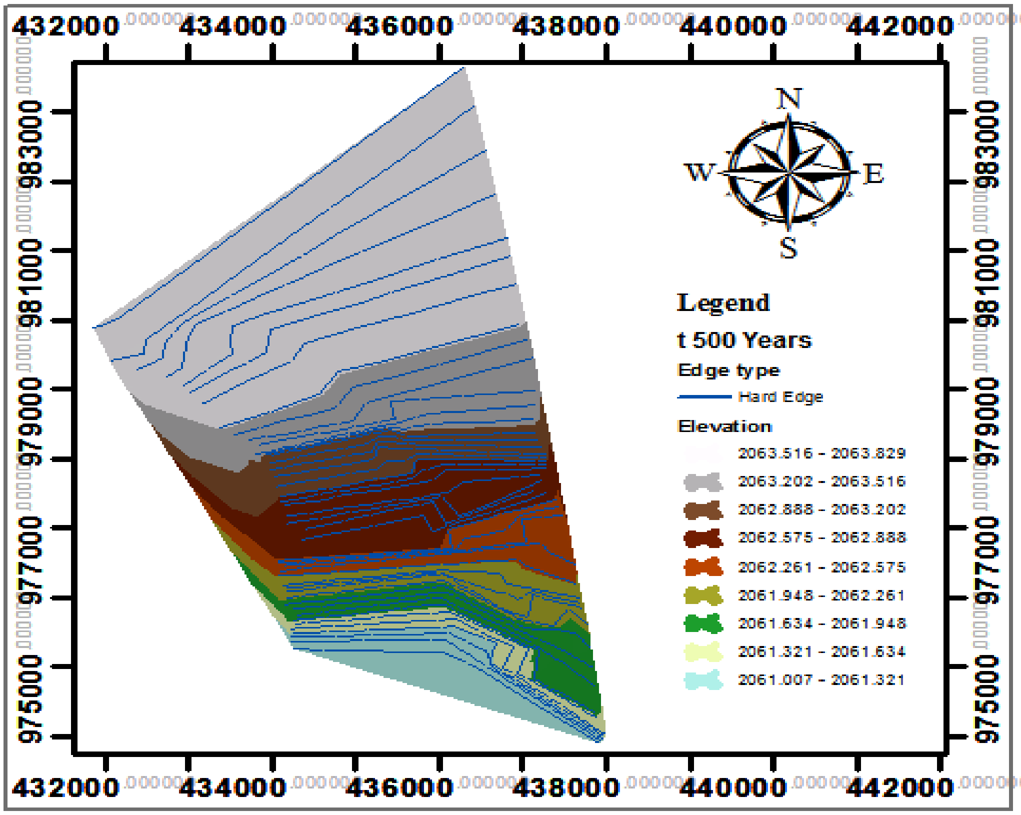


Figure 5.10: - Water surface TIN generated using cross-sectional cut line

5.6.2 Floodplain delineation using raster

Floodplain delineation used the Water surface TIN and terrain model to calculate the floodplain boundary and inundation depth. At this stage the water surface TIN was first converted to a GRID, and then DTM GRID was subtracted from the water surface grid. The area with positive results (meaning water surface is higher than the terrain) is flood area, and the area with negative results is dry. All the cells in water surface grid that result in positive values after subtraction were converted to a polygon, which is the final flood inundation polygon.

After the inundation map was created, check on quality of the inundation polygon was made. The ability to judge the quality of terrain and flood inundation polygon comes with the knowledge of the study area and experience.

The flood inundation map for 500 years return period discharge is shown in Fig. 5.11.

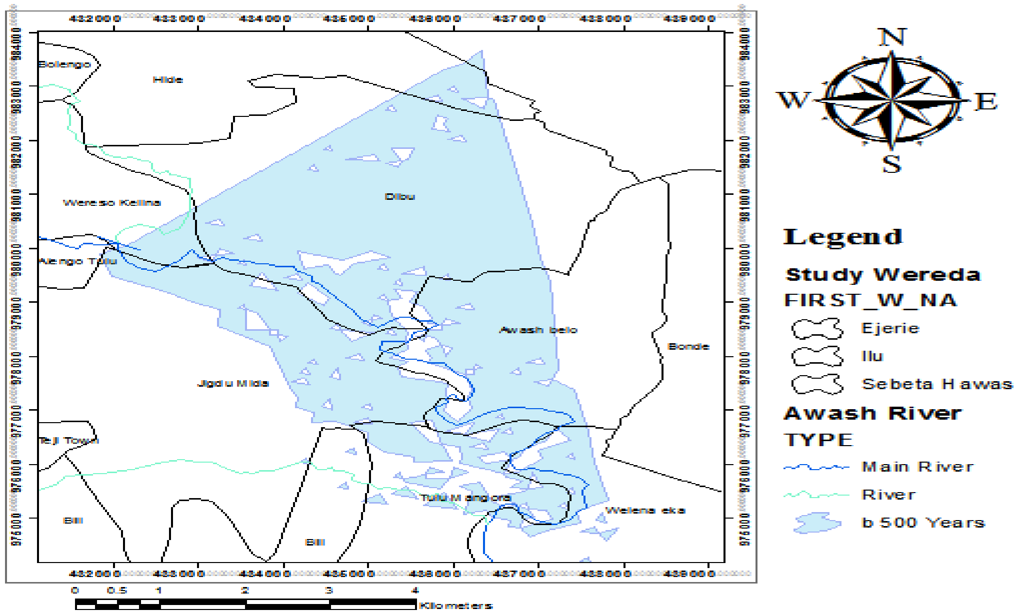


Figure 5.11: - Flood inundation Map for 500 years return period.

In the attribute table of the polygon of the flood extent, the inundation area was displayed and used for analysis of the flooding effect on crop yield.

The extent of inundation areas for the floods of different return periods floods are indicated in Table 5.2. Flood inundation map corresponding to each return period is presented in Annex C**.**

Table 5.2:- Flood magnitude and corresponding inundated area.

Return period (years)	2	5	50	100	500
Flow (m^3/s)	39.30	50.38	73.86	80.69	96.46
Inundated area (ha)	1,959.50	2,107.38	2,299.16	2,318.84	2,354.06

Floods that correspond to longer return periods have high flows which can submerge and damage a large amount of cropland. Also this flood can bring in water logging in the area which consequently can result in yield reduction.

5.7. Flood risk result

The estimated crop loss due to flooding for different return periods is indicated in Table 5.3.

Table 5.3:- Estimated crop loss due to flooding

				Return Period				
				2 Yrs	5 Yrs	50 Yrs	100 Yrs	500 Yrs
				Flood Inundated Area				
No.	**Crop**	**Average**	**% of Coverage**	1,959.50	2,107.38	2,299.16	2,318.84	2,354.06
1	Teff	24	50	23513.95	25288.51	27589.88	27826.05	28248.67
2	Chick Peas	25	20	9797.48	10536.88	11495.78	11594.19	11770.28
3	Lentils	15	20	5878.49	6322.13	6897.47	6956.51	7062.17
4	Grass Pea	25	10	4898.74	5268.44	5747.89	5797.09	5885.14
Total Crop Loss				44,088.66	47,415.96	51,731.03	52,173.84	52,966.25

Crop Loss = Average Crop Grown per hectare *% of Crop Coverage *Flood Inundated Area for different return period

Crop Loss = 24*0.5*1,959.50

Crop Loss = 23,513.95 quintal of Teff Loss for 2 years return period

Flood events with long return periods have small probability of occurrence but their magnitude and adverse effect on crop are very high. Flood with short return period occurs frequently and may result in repeated damage to crop. Therefore, this flood wire lost the crop due to on wet season from June to November.

This research mainly focused on the risky areas especially near Awash River that include Jigdu Mida, Tulu Manger, Awash Bello, Welena Eka and Dibu from three Wereda. The location and topography of these areas make them highly vulnerable to flooding.

6. CONCLUSION AND RECOMMENDATION

6.1 Conclusions

This research focused on checking the quality of the hydrological data of Awash Bello station, hydrological and hydraulic computations of flood profiles of the Upper Awash river, flood frequency analysis, delineation of flood hazard areas and effect of flood risk on crop production at different return periods.

It was found out that the recorded hydrological data of Awash Bello gauging station is of good quality, homogenous and stationary. Flood affected areas were delineated for 500 years and 2 years return periods with peak flood discharges of 96.46 m^3/s and 39.30 m^3/s, respectively. One-dimensional numerical model HEC-RAS and Arc-GIS for spatial data processing and HEC-GEORAS for interfacing between HEC-RAS and Arc-GIS were used. The flood hazard map delineation indicated Jigdu Mida and Tulu Mangero adjacent to Illu Wereda, Awash Bello and Welena Eka adjacent to Sebeta Hawas Wereda and Dibu from Ejerie Wereda to be the most affected areas.
It was also estimated that a total land area of 2,354.06 hectares could be inundated by the 500 years return period flood which may result in a loss of 52,966.25 quintals of crop. The area affected by the 2-years return period flood was 1,959.50 hectares with an estimated loss of 44,088.66 quintals of crop.

A large tract of land is inundated by flood magnitudes of different return periods that may result in a huge damage to crops and other infrastructures. But also has positive impact and benefit from flood such as accumulation of fertile soil comes from upstream of awash river as result, the farmers they use for agricultural activity to increase the production Appropriate flood management measures should therefore be identified and practiced.

6.2 Recommendations

In order to minimize the amount of flood damage, it is recommended that areas inundated by the 100 year return period flood should not be used for agricultural activities, infrastructure development, settlement and other investment projects on the wet season because has high probability occurrence in the study area.

The following specific recommendations are also forwarded.

- Because of limitation of data and finance, the study focused only on the risk of flood on crop yield. It is recommended to carry out additional researches concerning the risk of flooding on other properties.

- The responsible bodies of the Woreda as well as the Region should incorporate the flood hazard and flood risk assessment studies in their development strategies

- Development of flood protection structures and soil conservation practices should be carried out in the upstream and downstream part of the site to reduce the magnitude of flood, sediment transport and siltation

- Adopt appropriate land use planning practices in flood prone area to reduce the adverse effects.

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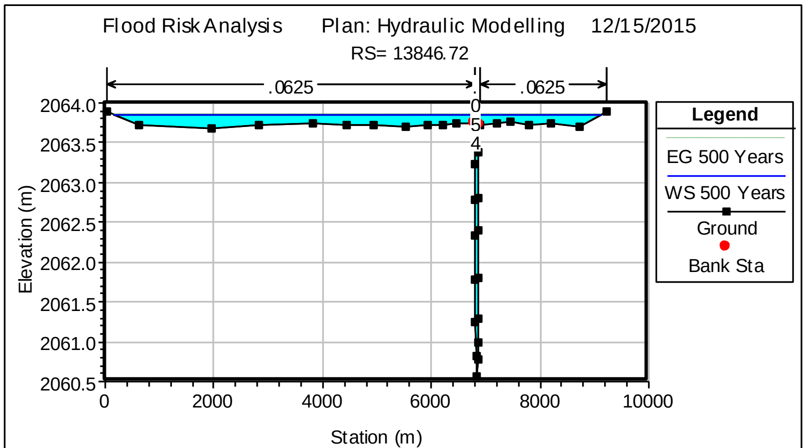
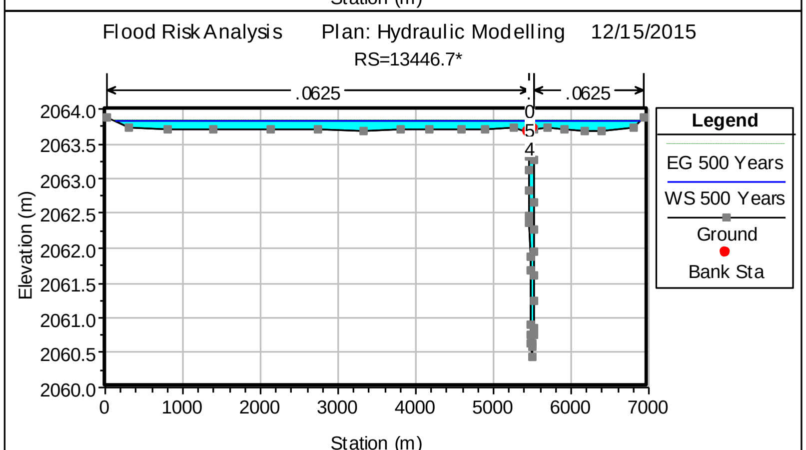
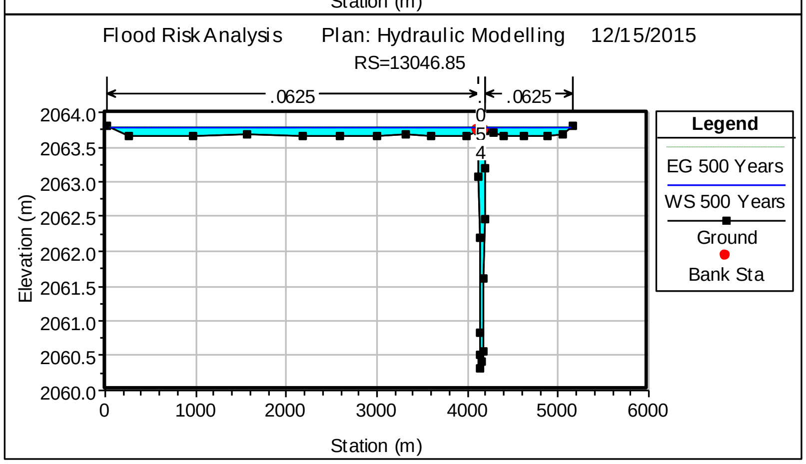
ANNEXES

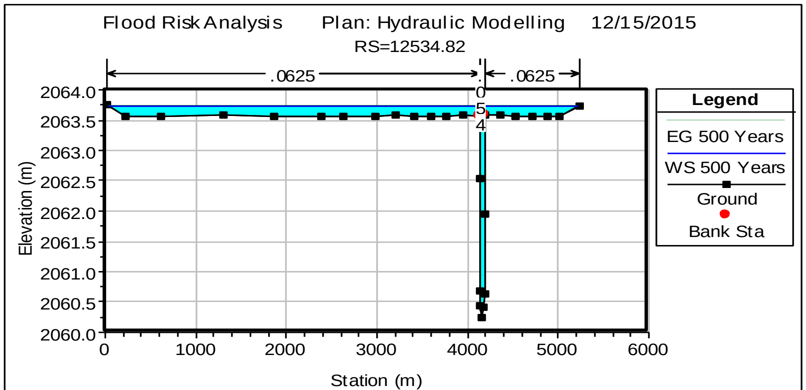
Annex A: HEC-RAS outputs

1. **Sample cross-sectional profile view of Awash River at study area after Simulate the model.**





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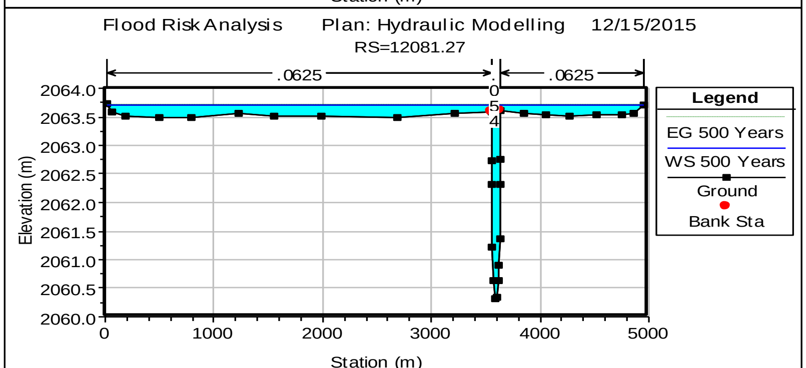
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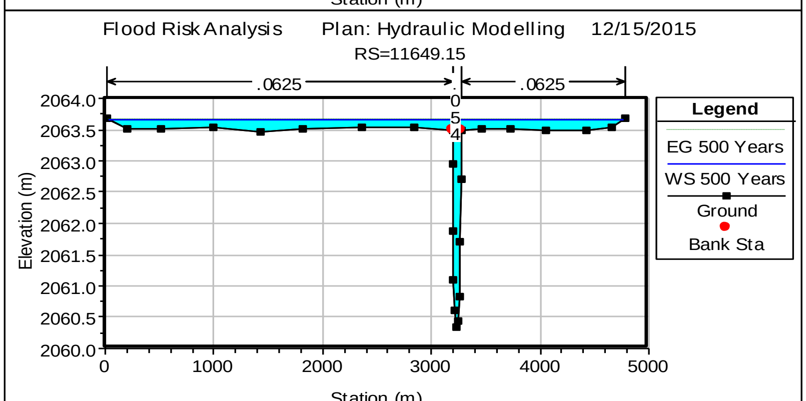
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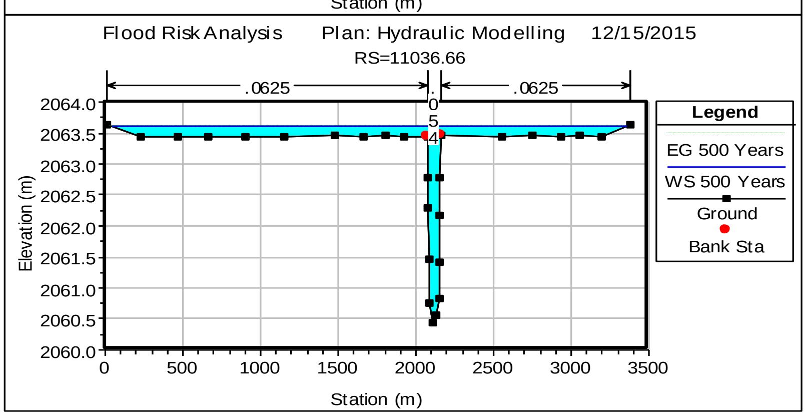
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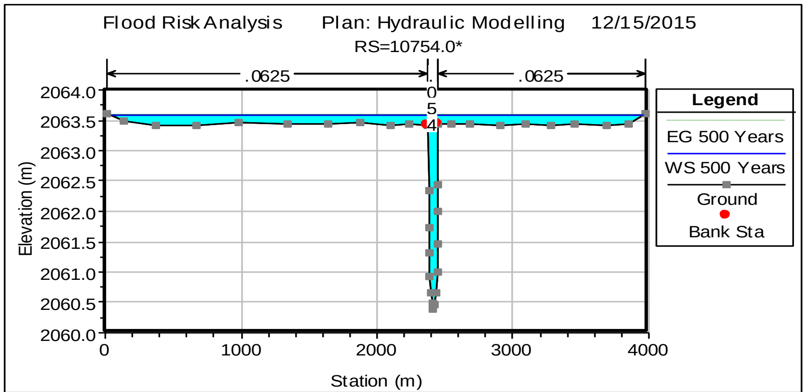
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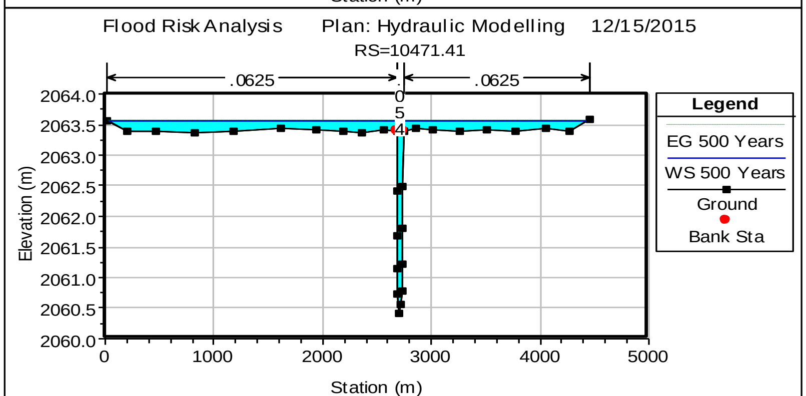
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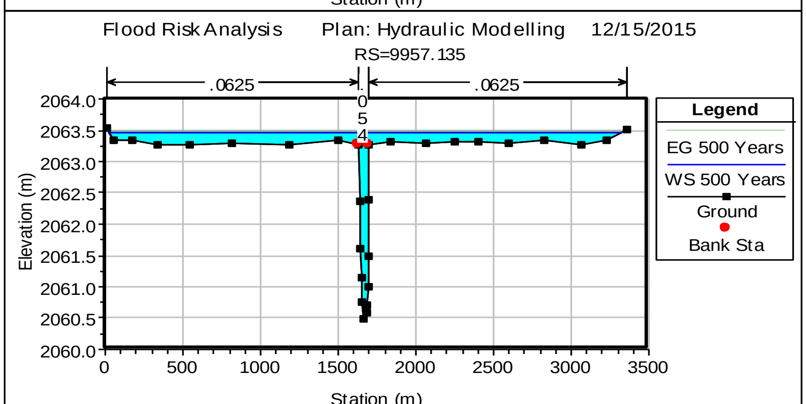
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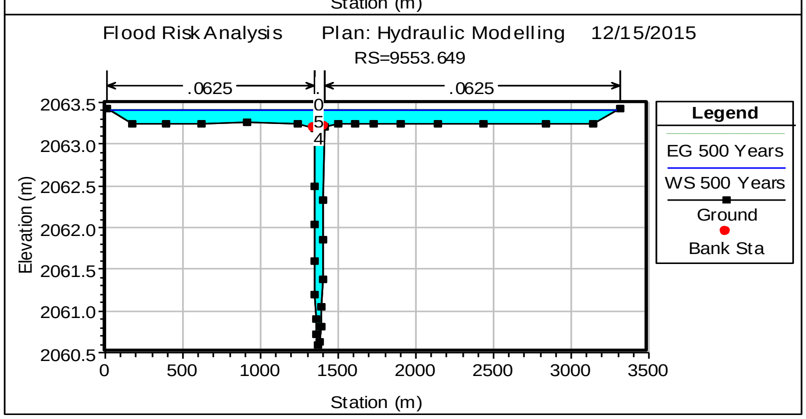
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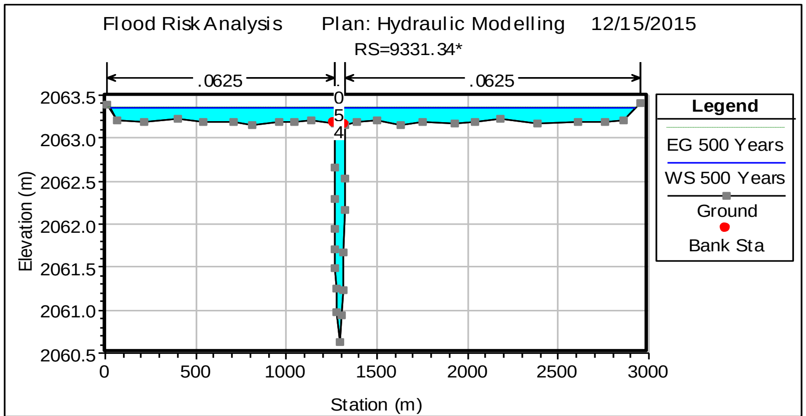
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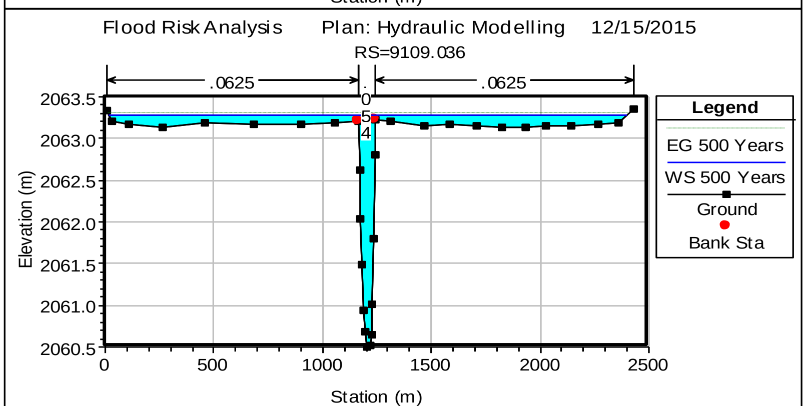
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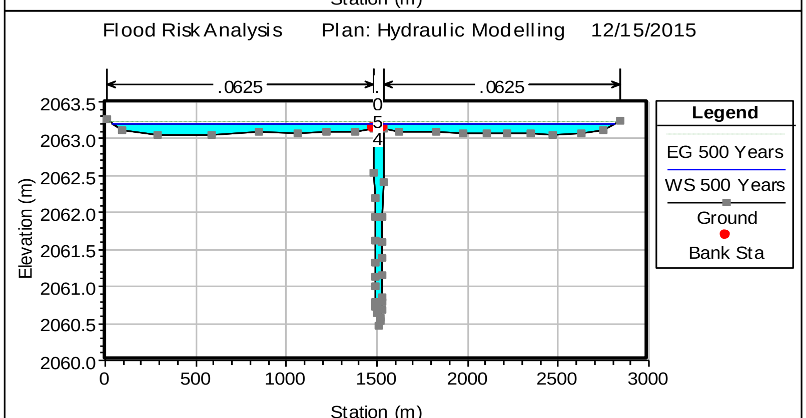
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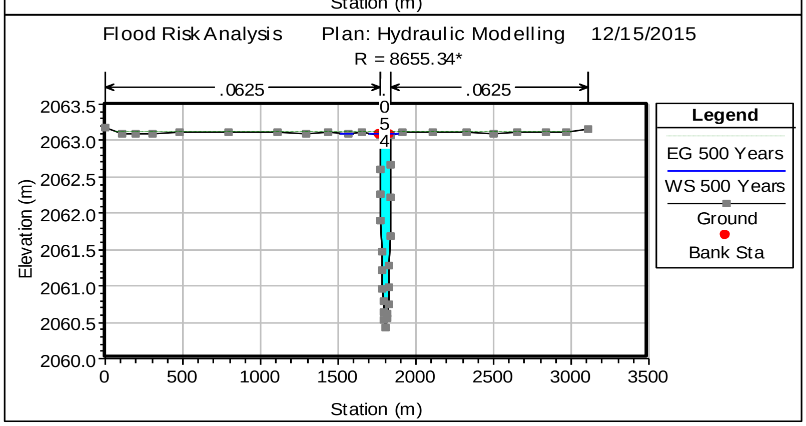
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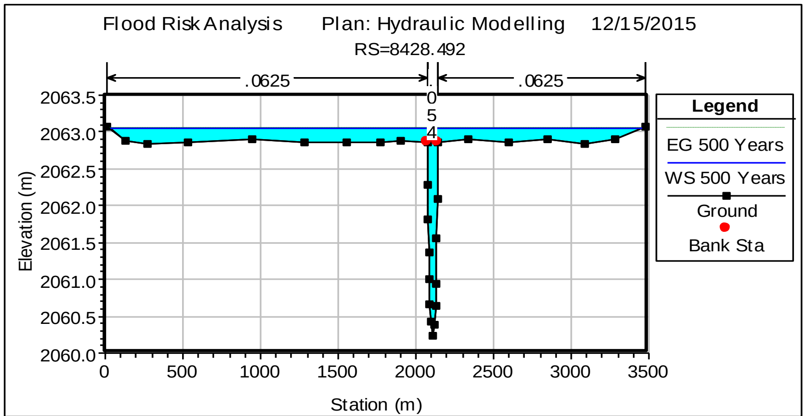
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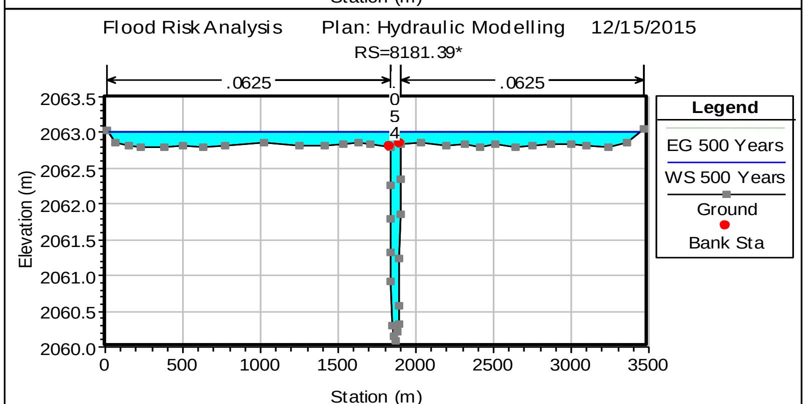
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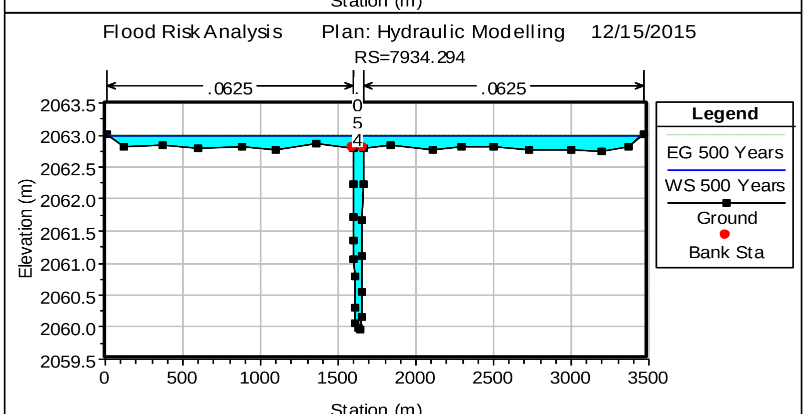
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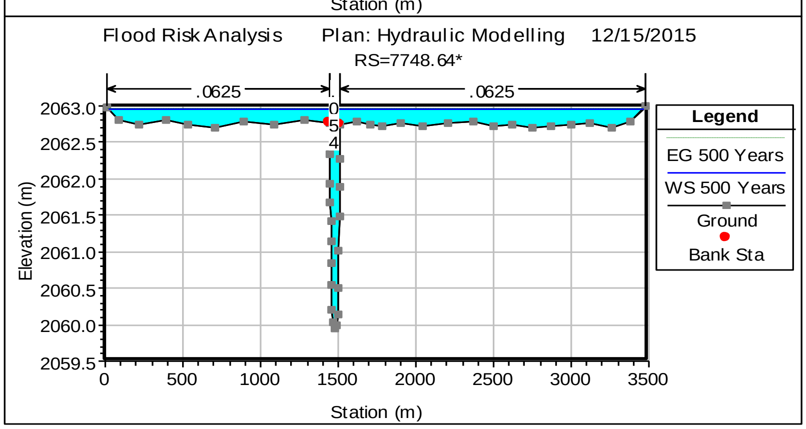
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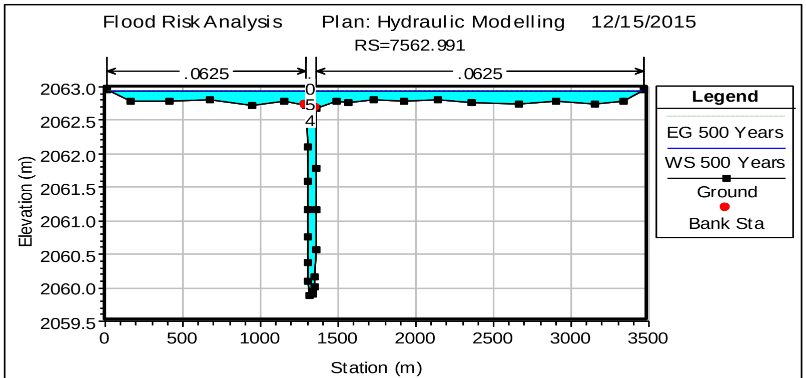
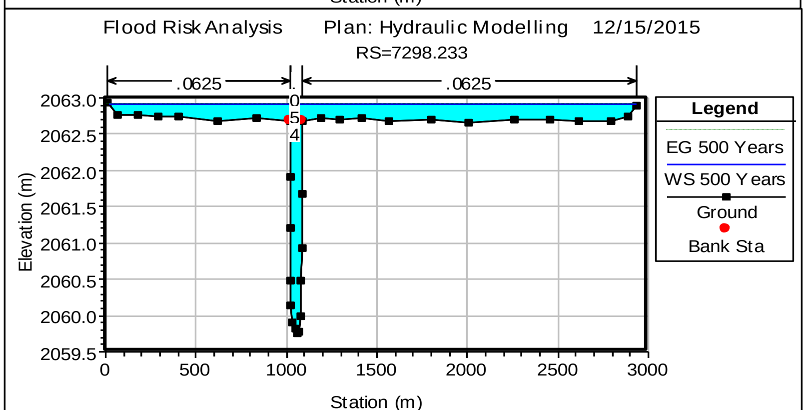
Ground
Bank Sta
Elevation (m)
Station (m)


Flood Risk Analysis Plan: Hydraulic Modelling 12/15/2015
RS=7562.991
.0625
.0625
Elevation (m)
Station (m)
2063.0
2062.5
2062.0
2061.5
2061.0
2060.5
2060.0
2059.5
0
500
1000
1500
2000
2500
3000
3500
Legend
EG 500 Years
WS 500 Years
Ground
Bank Sta


Flood Risk Analysis Plan: Hydraulic Modelling 12/15/2015
RS=7298.233
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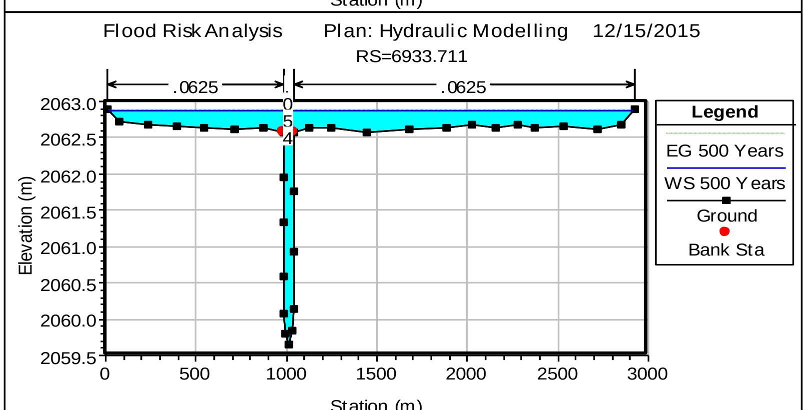
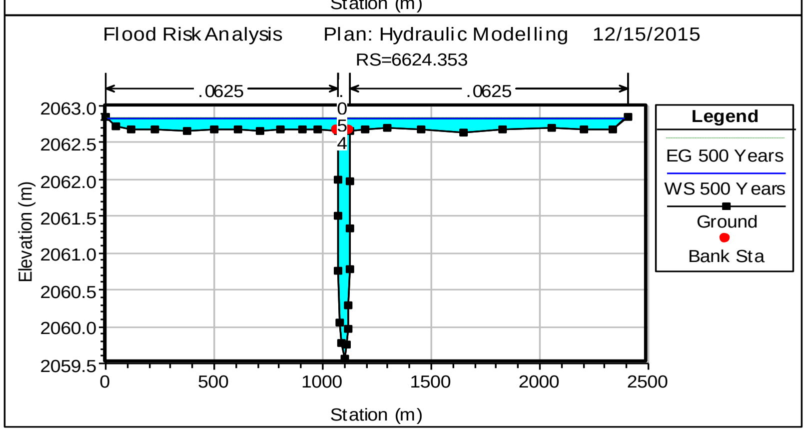
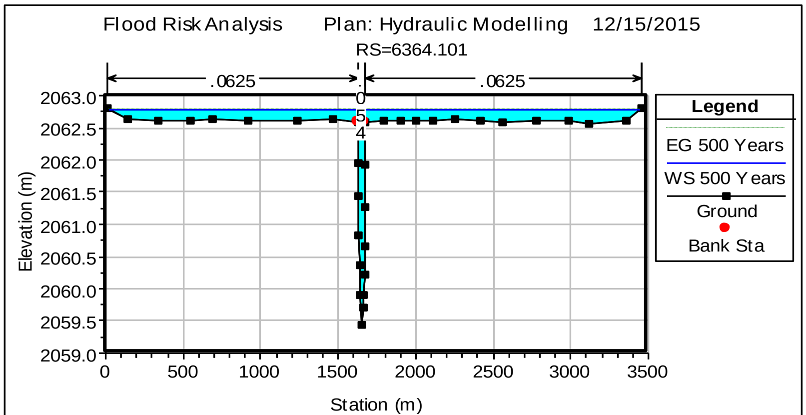
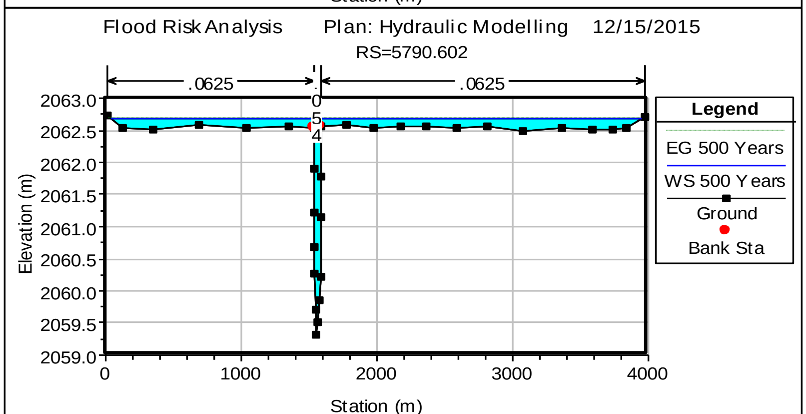
.0625
Elevation (m)
Station (m)
2063.0
2062.5
2062.0
2061.5
2061.0
2060.5
2060.0
2059.5
0
500
1000
1500
2000
2500
3000
Legend
EG 500 Years
WS 500 Years
Ground
Bank Sta


Flood Risk Analysis Plan: Hydraulic Modelling 12/15/2015
RS=6933.711
.0625
.0625
Elevation (m)
Station (m)
2063.0
2062.5
2062.0
2061.5
2061.0
2060.5
2060.0
2059.5
0
500
1000
1500
2000
2500
3000
Legend
EG 500 Years
WS 500 Years
Ground
Bank Sta


Flood Risk Analysis Plan: Hydraulic Modelling 12/15/2015
RS=6624.353
.0625
.0625
Elevation (m)
Station (m)
2063.0
2062.5
2062.0
2061.5
2061.0
2060.5
2060.0
2059.5
0
500
1000
1500
2000
2500
Legend
EG 500 Years
WS 500 Years
Ground
Bank Sta


Flood Risk Analysis Plan: Hydraulic Modelling 12/15/2015
RS=6364.101
.0625
.0625
0
5
4
Elevation (m)
Station (m)
Legend
EG 500 Years
WS 500 Years
Ground
Bank Sta


Flood Risk Analysis Plan: Hydraulic Modelling 12/15/2015
RS=5790.602
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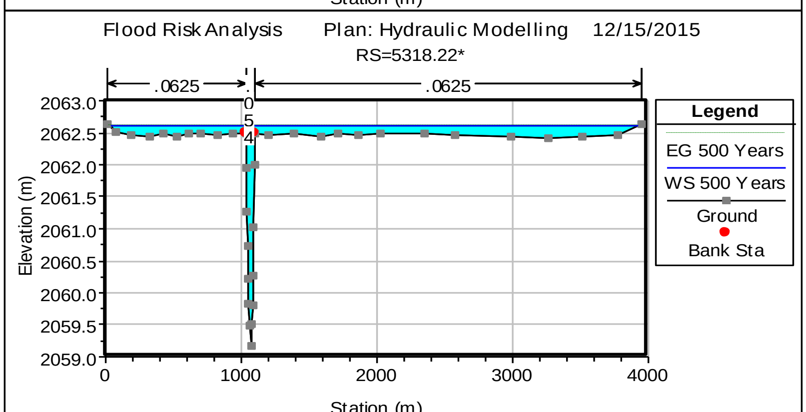
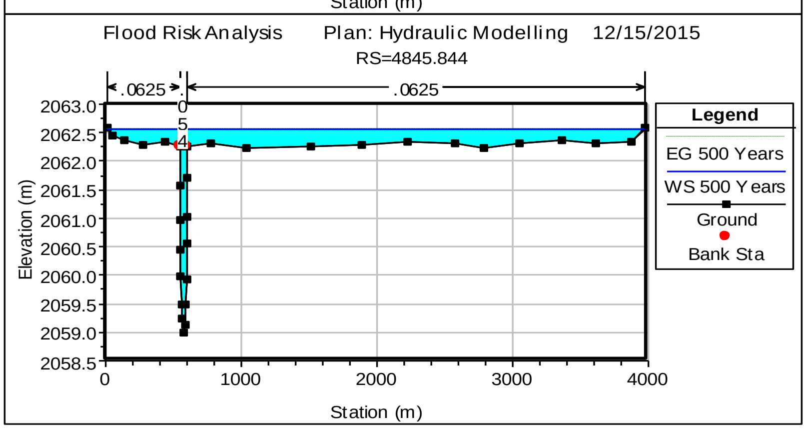
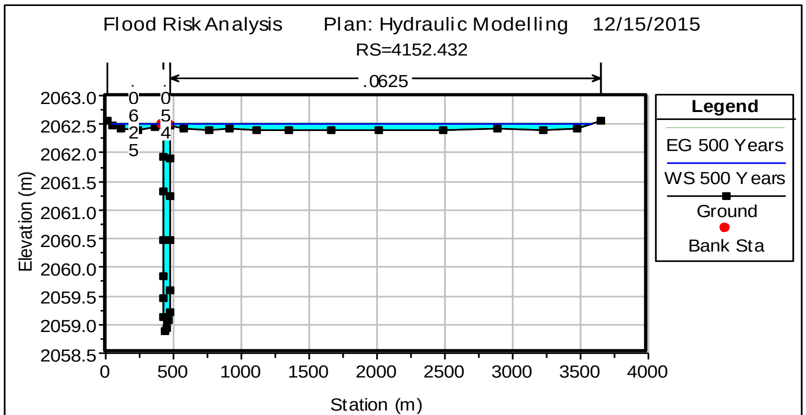
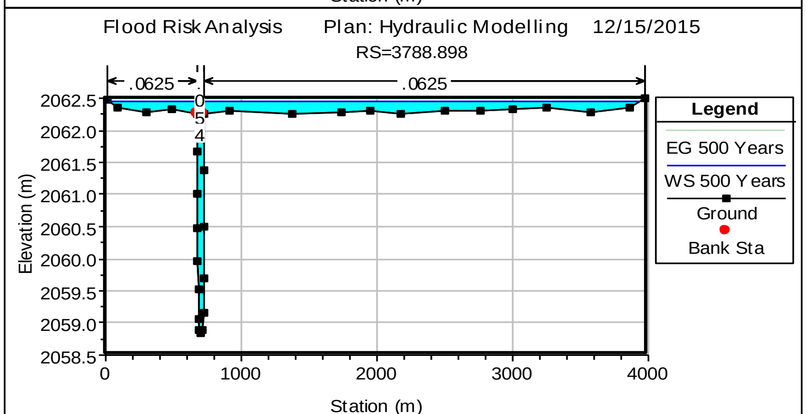
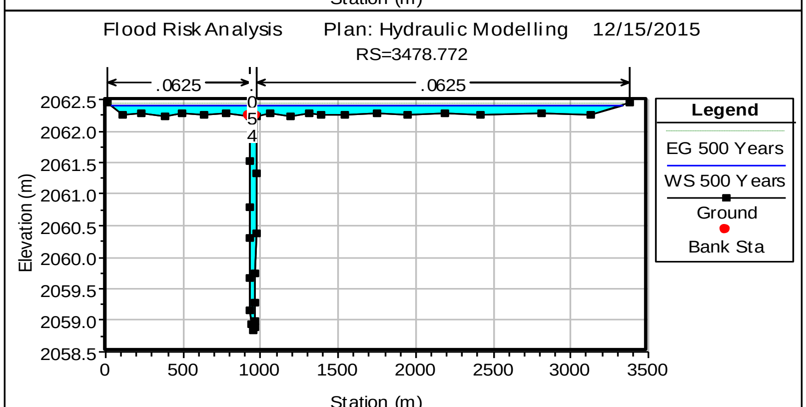
.0625
0
5
4
Elevation (m)
Station (m)
Legend
EG 500 Years
WS 500 Years
Ground
Bank Sta

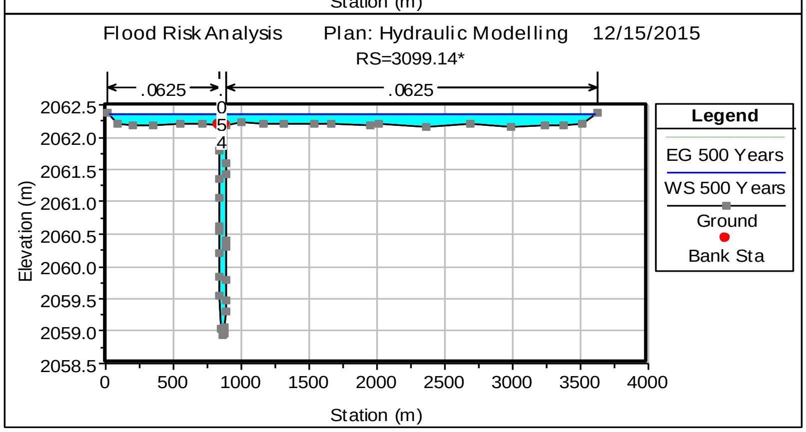

Flood Risk Analysis Plan: Hydraulic Modelling 12/15/2015
RS=5318.22*
.0625
.0625
0
5
4
Elevation (m)
Station (m)
Legend
EG 500 Years
WS 500 Years
Ground
Bank Sta

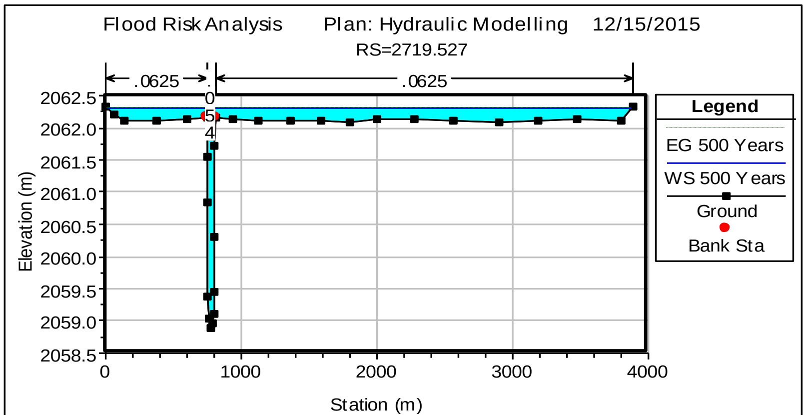

Flood Risk Analysis Plan: Hydraulic Modelling 12/15/2015
RS=4845.844
.0625
.0625
0
5
4
Elevation (m)
Station (m)
Legend
EG 500 Years
WS 500 Years
Ground
Bank Sta

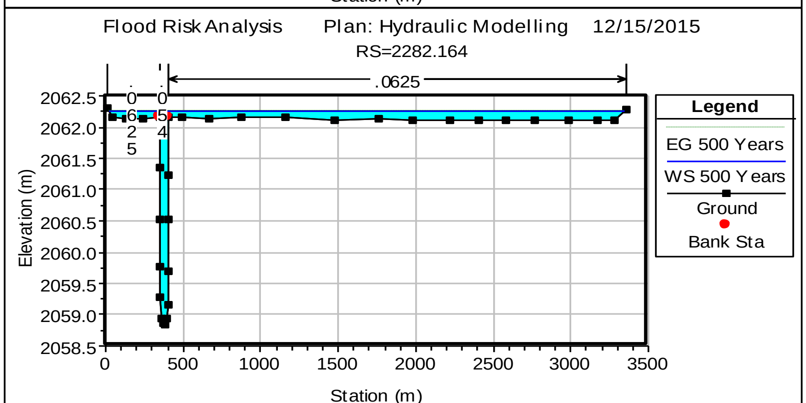

Flood Risk Analysis Plan: Hydraulic Modelling 12/15/2015
RS=4152.432
.0625
Legend
EG 500 Years
WS 500 Years
Ground
Bank Sta
Elevation (m)
Station (m)
2063.0 2062.5 2062.0 2061.5 2061.0 2060.5 2060.0 2059.5 2059.0 2058.5
0 500 1000 1500 2000 2500 3000 3500 4000

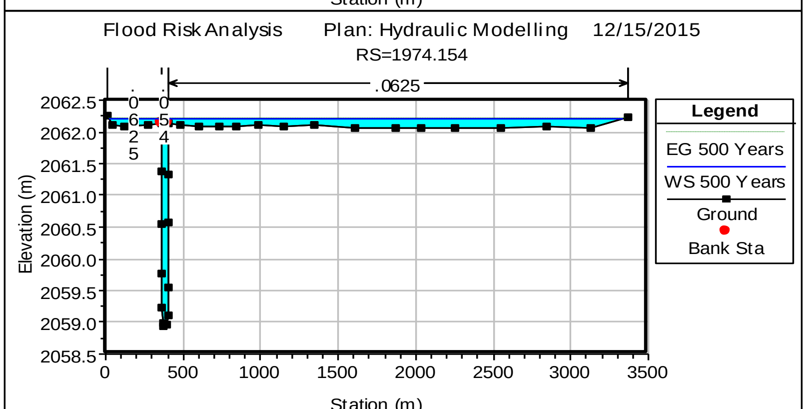

Flood Risk Analysis Plan: Hydraulic Modelling 12/15/2015
RS=3788.898
.0625
.0625
Legend
EG 500 Years
WS 500 Years
Ground
Bank Sta
Elevation (m)
Station (m)
2062.5 2062.0 2061.5 2061.0 2060.5 2060.0 2059.5 2059.0 2058.5
0 1000 2000 3000 4000

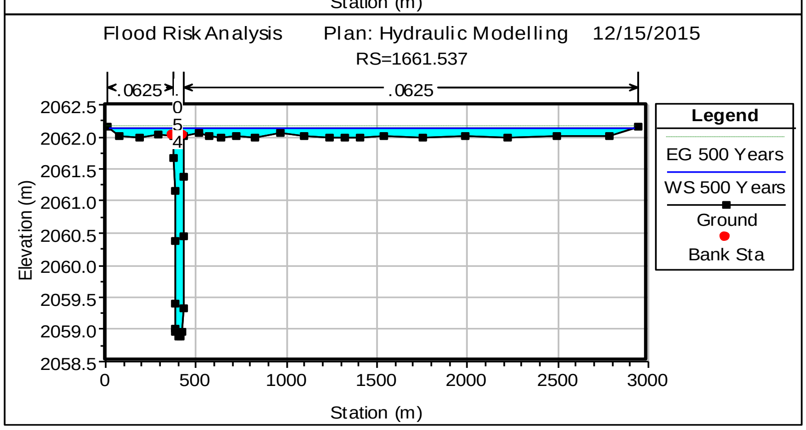

Flood Risk Analysis Plan: Hydraulic Modelling 12/15/2015
RS=3478.772
.0625
.0625
Legend
EG 500 Years
WS 500 Years
Ground
Bank Sta
Elevation (m)
Station (m)
2062.5 2062.0 2061.5 2061.0 2060.5 2060.0 2059.5 2059.0 2058.5
0 500 1000 1500 2000 2500 3000 3500

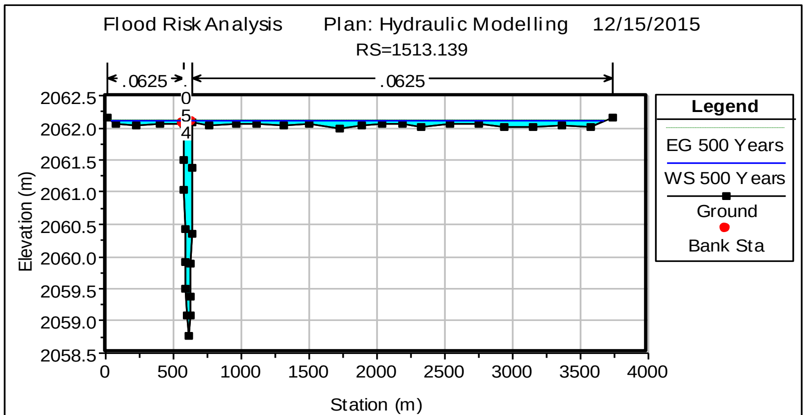

Flood Risk Analysis Plan: Hydraulic Modelling 12/15/2015
RS=3099.14*
.0625
.0625
Legend
EG 500 Years
WS 500 Years
Ground
Bank Sta
Elevation (m)
Station (m)
2062.5 2062.0 2061.5 2061.0 2060.5 2060.0 2059.5 2059.0 2058.5
0 500 1000 1500 2000 2500 3000 3500 4000

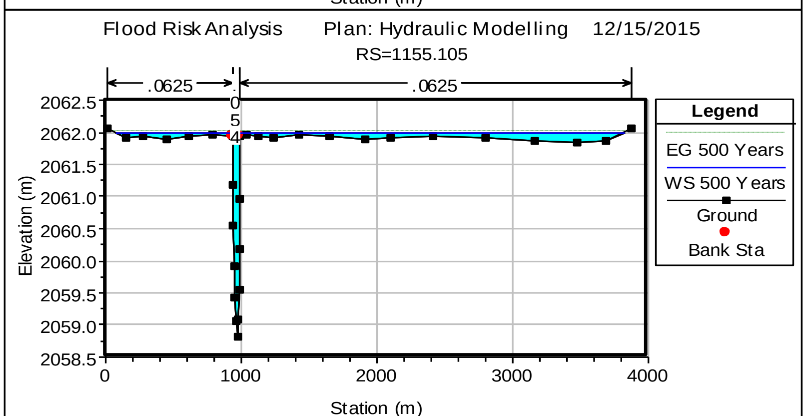

Flood Risk Analysis Plan: Hydraulic Modelling 12/15/2015
RS=2719.527
.0625
.0625
054
Elevation (m)
2062.5
2062.0
2061.5
2061.0
2060.5
2060.0
2059.5
2059.0
2058.5
0
1000
2000
3000
4000
Station (m)
Legend
EG 500 Years
WS 500 Years
Ground
Bank Sta

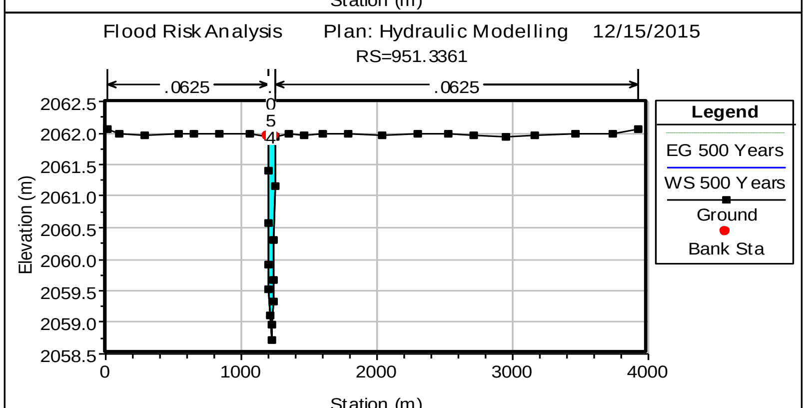

Flood Risk Analysis Plan: Hydraulic Modelling 12/15/2015
RS=2282.164
.0625
.0625
.054
Elevation (m)
2062.5
2062.0
2061.5
2061.0
2060.5
2060.0
2059.5
2059.0
2058.5
0
500
1000
1500
2000
2500
3000
3500
Station (m)
Legend
EG 500 Years
WS 500 Years
Ground
Bank Sta

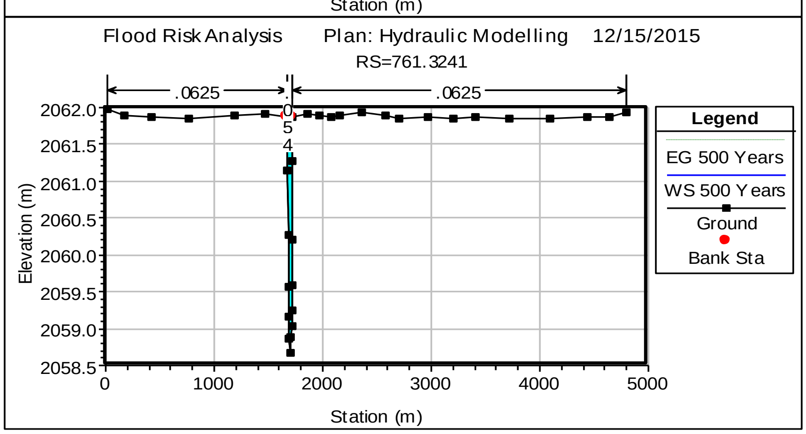

Flood Risk Analysis Plan: Hydraulic Modelling 12/15/2015
RS=1974.154
.0625
.0625
.054
Elevation (m)
2062.5
2062.0
2061.5
2061.0
2060.5
2060.0
2059.5
2059.0
2058.5
0
500
1000
1500
2000
2500
3000
3500
Station (m)
Legend
EG 500 Years
WS 500 Years
Ground
Bank Sta

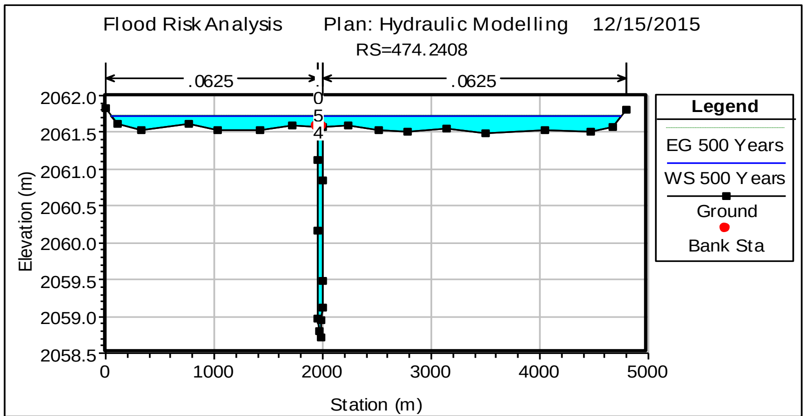

Flood Risk Analysis Plan: Hydraulic Modelling 12/15/2015
RS=1661.537
.0625
.0625
.054
Elevation (m)
2062.5
2062.0
2061.5
2061.0
2060.5
2060.0
2059.5
2059.0
2058.5
0
500
1000
1500
2000
2500
3000
Station (m)
Legend
EG 500 Years
WS 500 Years
Ground
Bank Sta

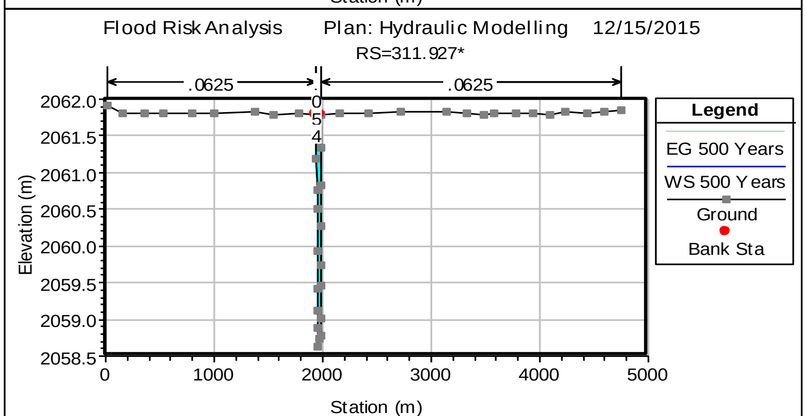

Flood Risk Analysis
Plan: Hydraulic Modelling
12/15/2015
RS=1513.139
.0625
.0625
Elevation (m)
2062.5
2062.0
2061.5
2061.0
2060.5
2060.0
2059.5
2059.0
2058.5
0
500
1000
1500
2000
2500
3000
3500
4000
Station (m)
Legend
EG 500 Years
WS 500 Years
Ground
Bank Sta

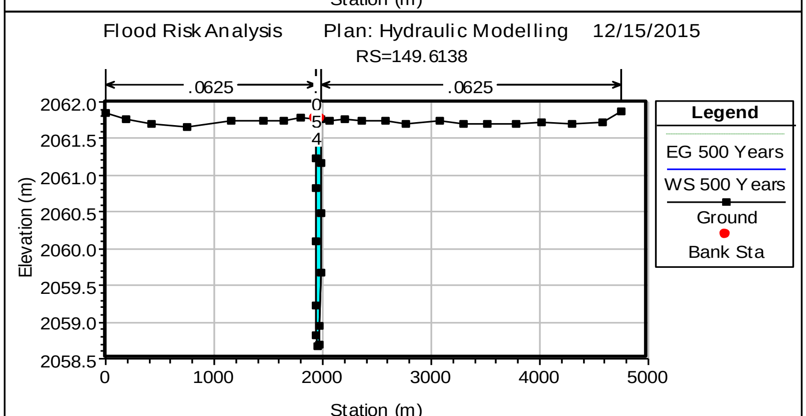

Flood Risk Analysis
Plan: Hydraulic Modelling
12/15/2015
RS=1155.105
.0625
.0625
Elevation (m)
2062.5
2062.0
2061.5
2061.0
2060.5
2060.0
2059.5
2059.0
2058.5
0
1000
2000
3000
4000
Station (m)
Legend
EG 500 Years
WS 500 Years
Ground
Bank Sta

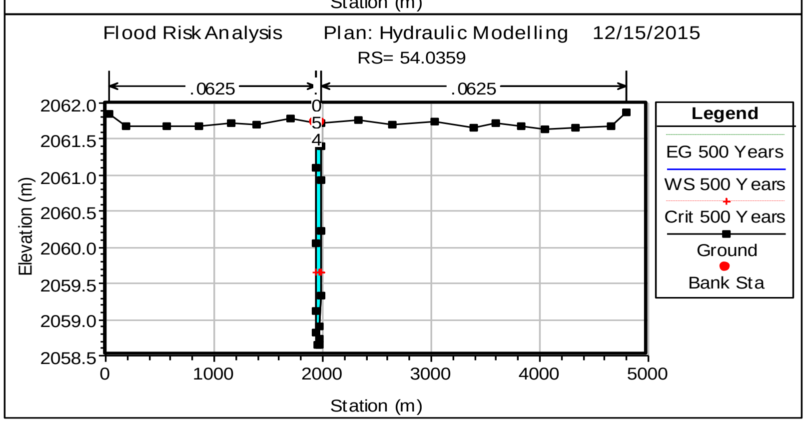

Flood Risk Analysis
Plan: Hydraulic Modelling
12/15/2015
RS=951.3361
.0625
.0625
Elevation (m)
2062.5
2062.0
2061.5
2061.0
2060.5
2060.0
2059.5
2059.0
2058.5
0
1000
2000
3000
4000
Station (m)
Legend
EG 500 Years
WS 500 Years
Ground
Bank Sta


Flood Risk Analysis
Plan: Hydraulic Modelling
12/15/2015
RS=761.3241
.0625
.0625
Elevation (m)
2062.0
2061.5
2061.0
2060.5
2060.0
2059.5
2059.0
2058.5
0
1000
2000
3000
4000
5000
Station (m)
Legend
EG 500 Years
WS 500 Years
Ground
Bank Sta

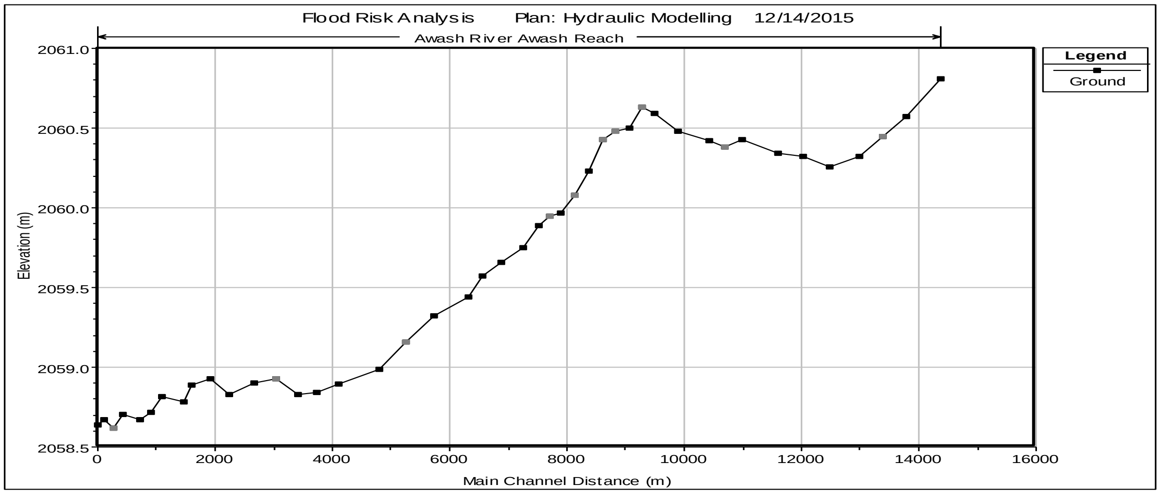

Flood Risk Analysis Plan: Hydraulic Modelling 12/15/2015
RS=474.2408
.0625
.0625
Elevation (m)
Station (m)
Legend
EG 500 Years
WS 500 Years
Ground
Bank Sta


Flood Risk Analysis Plan: Hydraulic Modelling 12/15/2015
RS=311.927*
.0625
.0625
Elevation (m)
Station (m)
Legend
EG 500 Years
WS 500 Years
Ground
Bank Sta

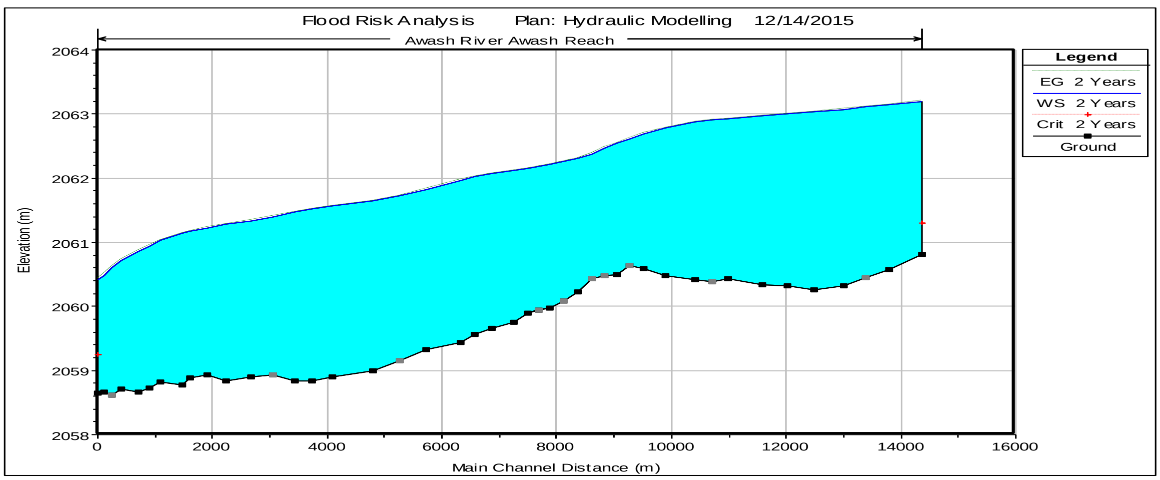

Flood Risk Analysis Plan: Hydraulic Modelling 12/15/2015
RS=149.6138
.0625
.0625
Elevation (m)
Station (m)
Legend
EG 500 Years
WS 500 Years
Ground
Bank Sta


Flood Risk Analysis Plan: Hydraulic Modelling 12/15/2015
RS= 54.0359
.0625
.0625
Elevation (m)
Station (m)
Legend
EG 500 Years
WS 500 Years
Crit 500 Years
Ground
Bank Sta

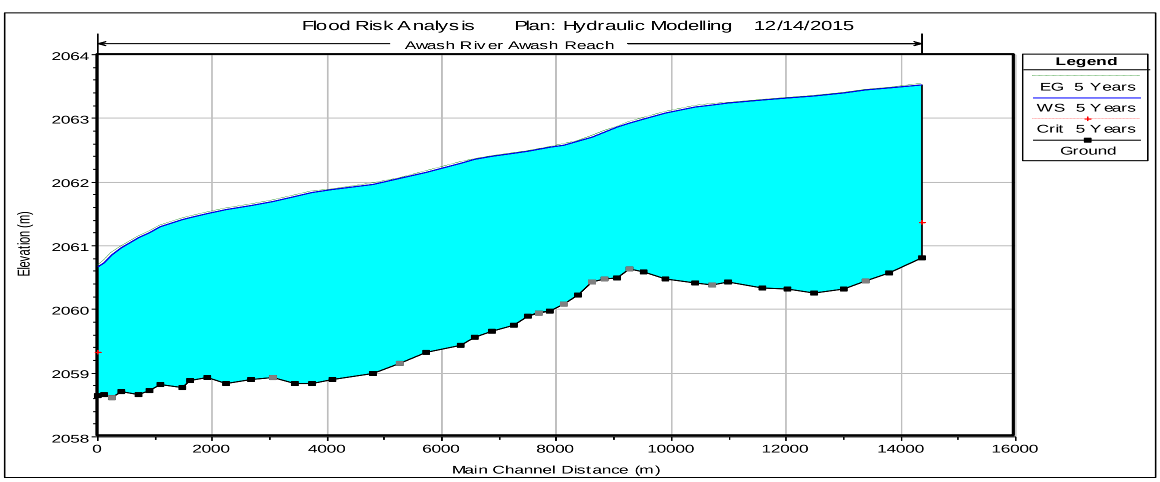
2. Water surface profile of Awash River at study area for different return periods obtained from HEC-RAS model.



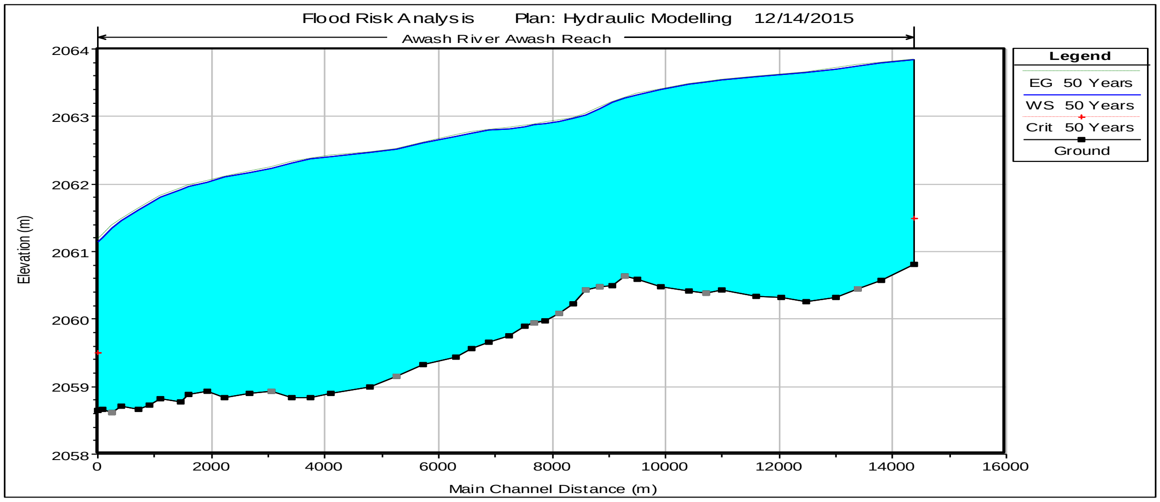
Ground Profile of the Reach



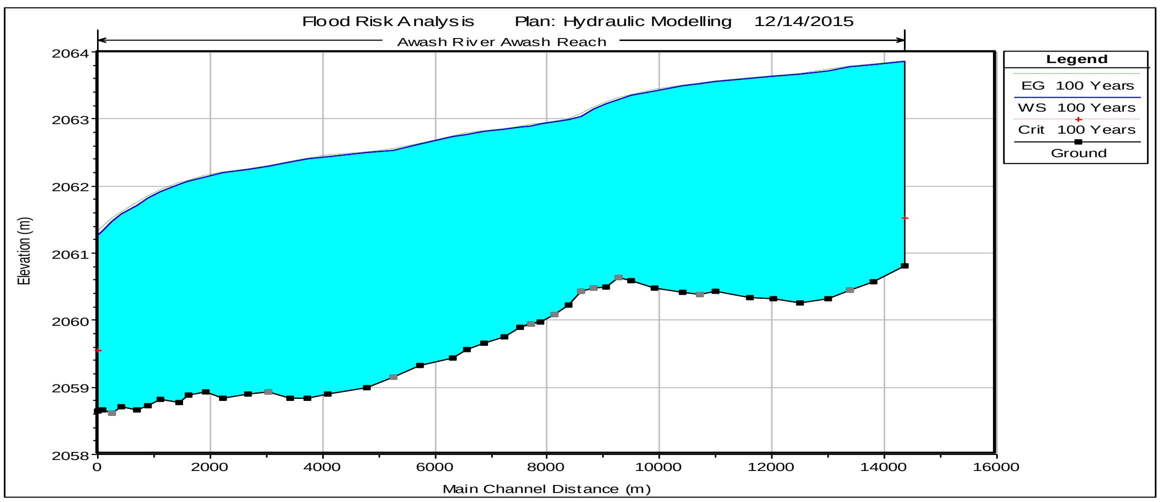
Water surface profile of 2-year return period



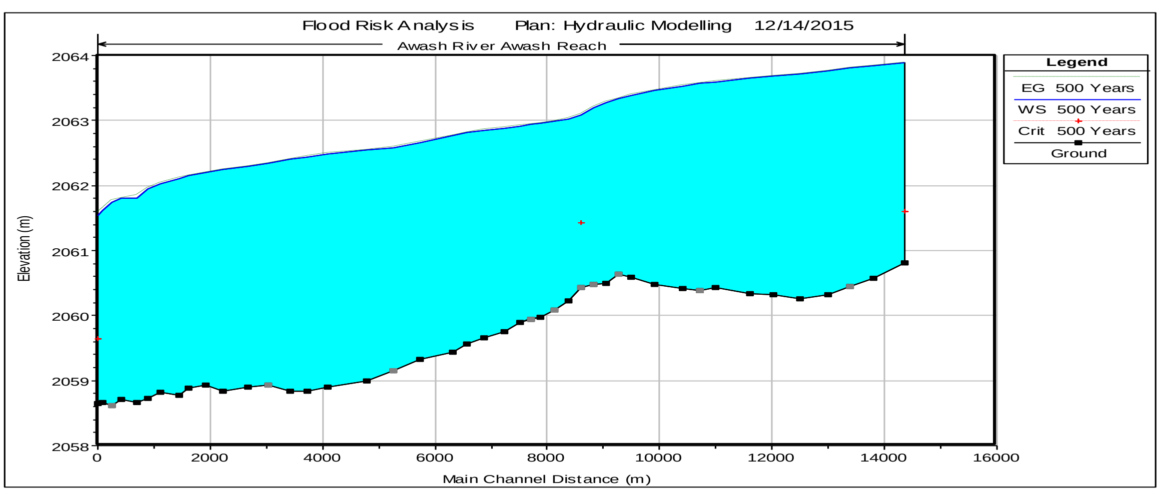
Water surface profile of 5-year return period



Water surface profile of 50-year return period

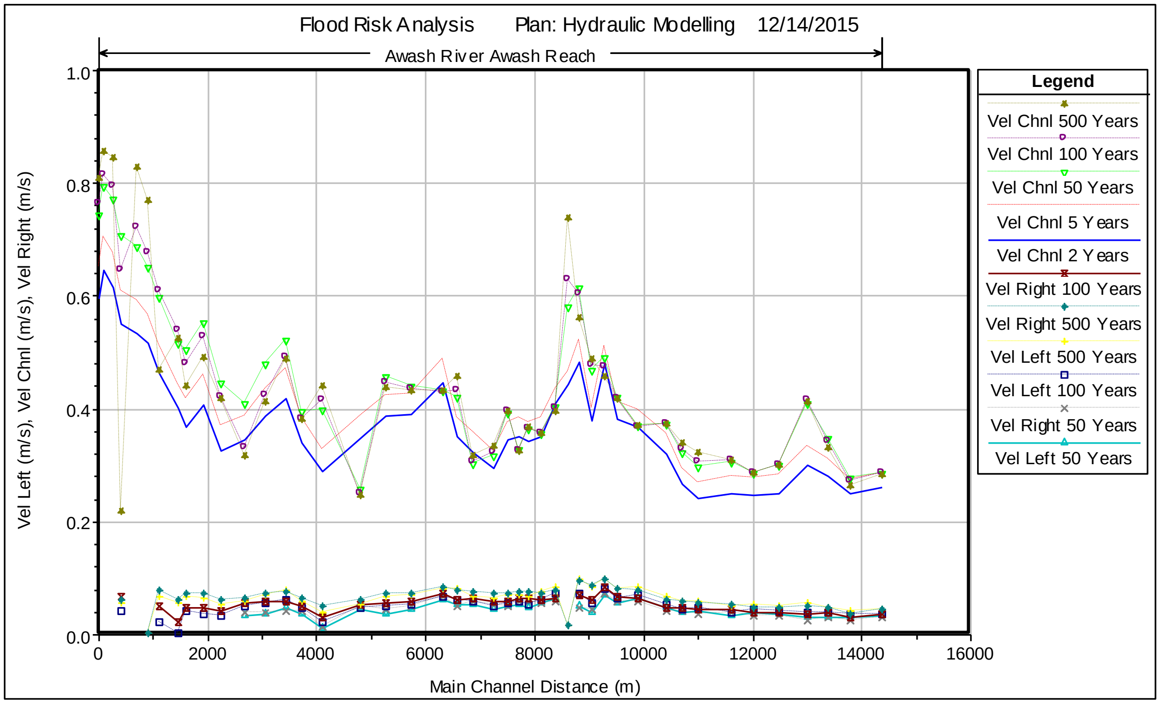


Water surface profile of 100-year return period

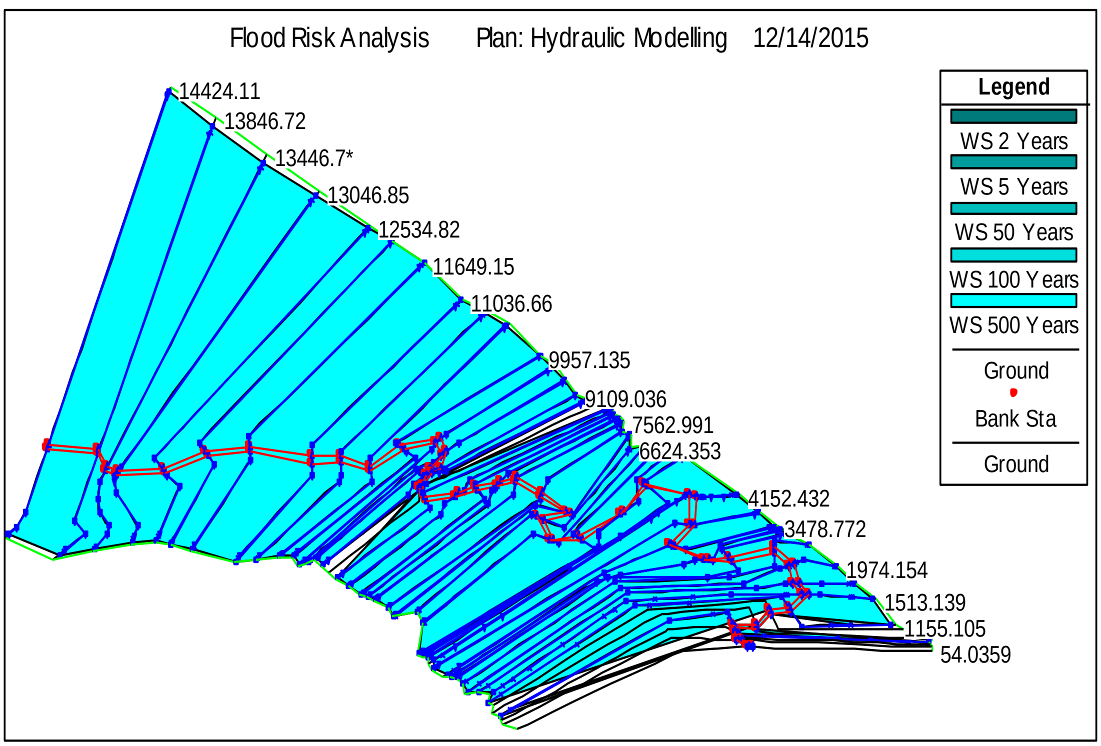


Water surface profile of 500-year return period

3. General Profile Plot-Velocities at study area



4.X-Y-Z Persepective Plot at study area



5.Profile output table of the whole cross-sections in the Awash River at study area.

HEC-RAS Plan: Modelling River: Awash River Reach: Awash Reach

Reach	River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl
Awash Reach	14424.11	2 Years	39.30	2060.81	2063.12	2061.30	2063.13	0.000086	0.27	145.55	73.08	0.06
Awash Reach	14424.11	5 Years	50.38	2060.81	2063.44	2061.37	2063.45	0.000088	0.30	169.14	74.32	0.06
Awash Reach	14424.11	50 Years	73.87	2060.81	2063.83	2061.49	2063.84	0.000079	0.31	706.85	7568.96	0.06
Awash Reach	14424.11	100 Years	80.69	2060.81	2063.86	2061.52	2063.86	0.000077	0.31	877.55	7621.92	0.06
Awash Reach	14424.11	500 Years	96.46	2060.81	2063.90	2061.60	2063.90	0.000073	0.30	1191.47	7718.36	0.06
Awash Reach	13846.72	2 Years	39.30	2060.57	2063.08		2063.08	0.000078	0.26	151.83	75.95	0.06
Awash Reach	13846.72	5 Years	50.38	2060.57	2063.40		2063.40	0.000081	0.29	176.29	77.52	0.06
Awash Reach	13846.72	50 Years	73.87	2060.57	2063.79		2063.79	0.000075	0.30	721.59	8548.66	0.06
Awash Reach	13846.72	100 Years	80.69	2060.57	2063.81		2063.82	0.000071	0.30	933.46	8707.10	0.06
Awash Reach	13846.72	500 Years	96.46	2060.57	2063.86		2063.86	0.000065	0.29	1323.30	8991.32	0.06
Awash Reach	13446.7*	2 Years	39.30	2060.45	2063.04		2063.05	0.000096	0.29	134.98	65.74	0.06
Awash Reach	13446.7*	5 Years	50.38	2060.45	2063.36		2063.36	0.000101	0.32	156.04	67.14	0.07
Awash Reach	13446.7*	50 Years	73.87	2060.45	2063.75		2063.76	0.000110	0.37	483.82	6540.87	0.07
Awash Reach	13446.7*	100 Years	80.69	2060.45	2063.78		2063.78	0.000106	0.36	656.18	6619.49	0.07
Awash Reach	13446.7*	500 Years	96.46	2060.45	2063.83		2063.83	0.000098	0.35	973.83	6762.00	0.07
Awash Reach	13046.85	2 Years	39.30	2060.32	2063.00		2063.01	0.000102	0.31	126.33	58.36	0.07
Awash Reach	13046.85	5 Years	50.38	2060.32	2063.32		2063.32	0.000112	0.35	145.01	60.49	0.07
Awash Reach	13046.85	50 Years	73.87	2060.32	2063.70		2063.71	0.000140	0.42	326.45	4708.81	0.08
Awash Reach	13046.85	100 Years	80.69	2060.32	2063.73		2063.73	0.000143	0.43	452.48	4913.67	0.08
Awash Reach	13046.85	500 Years	96.46	2060.32	2063.78		2063.78	0.000141	0.43	701.27	5075.27	0.08
Awash Reach	12534.82	2 Years	39.30	2060.26	2062.96		2062.96	0.000065	0.26	151.74	65.76	0.05
Awash Reach	12534.82	5 Years	50.38	2060.26	2063.27		2063.28	0.000072	0.29	172.26	66.40	0.06
Awash Reach	12534.82	50 Years	73.87	2060.26	2063.65		2063.66	0.000072	0.32	608.76	4998.61	0.06
Awash Reach	12534.82	100 Years	80.69	2060.26	2063.68		2063.68	0.000072	0.32	739.75	5059.78	0.06
Awash Reach	12534.82	500 Years	96.46	2060.26	2063.73		2063.73	0.000071	0.32	996.47	5177.58	0.06
Awash Reach	12081.27	2 Years	39.30	2060.33	2062.93		2062.93	0.000067	0.25	154.47	69.77	0.05
Awash Reach	12081.27	5 Years	50.38	2060.33	2063.24		2063.24	0.000073	0.29	176.01	70.17	0.06
Awash Reach	12081.27	50 Years	73.87	2060.33	2063.62		2063.62	0.000070	0.31	631.16	4859.10	0.06
Awash Reach	12081.27	100 Years	80.69	2060.33	2063.65		2063.65	0.000070	0.31	759.49	4884.14	0.06
Awash Reach	12081.27	500 Years	96.46	2060.33	2063.70		2063.70	0.000069	0.31	1007.85	4932.25	0.06

HEC-RAS Plan: Modelling River: Awash River Reach: Awash Reach

Reach	River Sta	Profile	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl
Awash Reach	11649.15	2 Years	39.30	2060.34	2062.90		2062.90	0.000069	0.26	152.78	69.50	0.06
Awash Reach	11649.15	5 Years	50.38	2060.34	2063.21		2063.21	0.000075	0.29	174.21	71.04	0.06
Awash Reach	11649.15	50 Years	73.87	2060.34	2063.59		2063.59	0.000080	0.32	540.27	4579.53	0.06
Awash Reach	11649.15	100 Years	80.69	2060.34	2063.61		2063.62	0.000080	0.33	662.30	4633.71	0.06
Awash Reach	11649.15	500 Years	96.46	2060.34	2063.66		2063.67	0.000081	0.33	898.69	4736.91	0.06
Awash Reach	11036.66	2 Years	39.30	2060.43	2062.86		2062.86	0.000071	0.25	157.59	77.38	0.06
Awash Reach	11036.66	5 Years	50.38	2060.43	2063.16		2063.16	0.000075	0.28	181.18	78.81	0.06
Awash Reach	11036.66	50 Years	73.87	2060.43	2063.54		2063.54	0.000079	0.31	486.63	3167.11	0.06
Awash Reach	11036.66	100 Years	80.69	2060.43	2063.56		2063.57	0.000083	0.32	567.41	3219.11	0.06
Awash Reach	11036.66	500 Years	96.46	2060.43	2063.61		2063.62	0.000091	0.34	721.58	3316.08	0.07
Awash Reach	10754.0*	2 Years	39.30	2060.38	2062.84		2062.84	0.000090	0.28	142.78	72.49	0.06
Awash Reach	10754.0*	5 Years	50.38	2060.38	2063.14		2063.14	0.000097	0.31	164.98	75.34	0.07
Awash Reach	10754.0*	50 Years	73.87	2060.38	2063.51		2063.52	0.000100	0.34	478.87	3806.57	0.07
Awash Reach	10754.0*	100 Years	80.69	2060.38	2063.54		2063.54	0.000103	0.34	572.80	3843.30	0.07
Awash Reach	10754.0*	500 Years	96.46	2060.38	2063.58		2063.59	0.000108	0.36	749.38	3911.42	0.07
Awash Reach	10471.41	2 Years	39.30	2060.42	2062.80		2062.81	0.000132	0.33	118.43	59.68	0.08
Awash Reach	10471.41	5 Years	50.38	2060.42	2063.10		2063.11	0.000138	0.37	136.32	60.34	0.08
Awash Reach	10471.41	50 Years	73.87	2060.42	2063.48		2063.49	0.000131	0.40	484.43	4242.57	0.08
Awash Reach	10471.41	100 Years	80.69	2060.42	2063.50		2063.51	0.000130	0.40	589.87	4293.03	0.08
Awash Reach	10471.41	500 Years	96.46	2060.42	2063.55		2063.55	0.000129	0.40	787.49	4386.06	0.08
Awash Reach	9957.135	2 Years	39.30	2060.48	2062.72		2062.72	0.000207	0.38	103.01	59.52	0.09
Awash Reach	9957.135	5 Years	50.38	2060.48	2063.01		2063.02	0.000222	0.42	121.25	64.88	0.10
Awash Reach	9957.135	50 Years	73.87	2060.48	2063.40		2063.41	0.000176	0.40	459.19	3242.28	0.09
Awash Reach	9957.135	100 Years	80.69	2060.48	2063.43		2063.43	0.000172	0.40	543.46	3265.95	0.09
Awash Reach	9957.135	500 Years	96.46	2060.48	2063.48		2063.48	0.000168	0.40	700.53	3309.61	0.09
Awash Reach	9553.649	2 Years	39.30	2060.59	2062.63		2062.63	0.000246	0.40	99.02	61.63	0.10
Awash Reach	9553.649	5 Years	50.38	2060.59	2062.92		2062.93	0.000240	0.43	117.23	63.56	0.10
Awash Reach	9553.649	50 Years	73.87	2060.59	2063.32		2063.33	0.000212	0.45	362.61	3101.78	0.10
Awash Reach	9553.649	100 Years	80.69	2060.59	2063.35		2063.36	0.000207	0.45	451.09	3156.39	0.10
Awash Reach	9553.649	500 Years	96.46	2060.59	2063.40		2063.41	0.000201	0.45	614.25	3254.70	0.10

HEC-RAS Plan: Modelling River: Awash River Reach: Awash Reach

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Awash Reach	9331.34*	2 Years	39.30	2060.63	2062.55		2062.56	0.000439	0.50	78.34	52.79	0.13
Awash Reach	9331.34*	5 Years	50.38	2060.63	2062.84		2062.86	0.000408	0.54	94.00	54.16	0.13
Awash Reach	9331.34*	50 Years	73.87	2060.63	2063.26		2063.27	0.000334	0.55	311.78	2835.86	0.12
Awash Reach	9331.34*	100 Years	80.69	2060.63	2063.29		2063.30	0.000307	0.53	405.48	2861.68	0.12
Awash Reach	9331.34*	500 Years	96.46	2060.63	2063.35		2063.36	0.000272	0.51	569.01	2906.20	0.11
Awash Reach	9109.036	2 Years	39.30	2060.50	2062.47		2062.48	0.000280	0.40	98.65	67.37	0.11
Awash Reach	9109.036	5 Years	50.38	2060.50	2062.77		2062.78	0.000257	0.42	119.29	69.99	0.10
Awash Reach	9109.036	50 Years	73.87	2060.50	2063.19		2063.21	0.000267	0.48	209.65	2189.07	0.11
Awash Reach	9109.036	100 Years	80.69	2060.50	2063.23		2063.24	0.000274	0.50	285.73	2346.53	0.11
Awash Reach	9109.036	500 Years	96.46	2060.50	2063.28		2063.30	0.000279	0.51	422.95	2386.47	0.11
Awash Reach	8882.18*	2 Years	39.30	2060.48	2062.39		2062.41	0.000393	0.51	77.72	47.28	0.13
Awash Reach	8882.18*	5 Years	50.38	2060.48	2062.70		2062.71	0.000376	0.55	92.20	48.16	0.13
Awash Reach	8882.18*	50 Years	73.87	2060.48	2063.11		2063.13	0.000409	0.64	185.50	2528.24	0.13
Awash Reach	8882.18*	100 Years	80.69	2060.48	2063.15		2063.16	0.000401	0.64	276.86	2699.38	0.13
Awash Reach	8882.18*	500 Years	96.46	2060.48	2063.21		2063.23	0.000348	0.60	459.52	2779.79	0.13
Awash Reach	8655.34*	2 Years	39.30	2060.43	2062.31		2062.32	0.000390	0.47	84.47	58.53	0.12
Awash Reach	8655.34*	5 Years	50.38	2060.43	2062.62		2062.63	0.000342	0.49	102.91	59.80	0.12
Awash Reach	8655.34*	50 Years	73.87	2060.43	2063.02		2063.04	0.000373	0.58	127.39	60.97	0.13
Awash Reach	8655.34*	100 Years	80.69	2060.43	2063.05		2063.07	0.000428	0.63	128.97	61.04	0.14
Awash Reach	8655.34*	500 Years	96.46	2060.43	2063.10		2063.13	0.000567	0.73	134.23	290.56	0.16
Awash Reach	8428.492	2 Years	39.30	2060.23	2062.23		2062.24	0.000288	0.43	91.70	57.18	0.11
Awash Reach	8428.492	5 Years	50.38	2060.23	2062.55		2062.56	0.000270	0.46	110.22	59.26	0.11
Awash Reach	8428.492	50 Years	73.87	2060.23	2062.97		2062.98	0.000194	0.43	449.86	3286.35	0.09
Awash Reach	8428.492	100 Years	80.69	2060.23	2063.00		2063.01	0.000187	0.43	537.54	3333.32	0.09
Awash Reach	8428.492	500 Years	96.46	2060.23	2063.05		2063.05	0.000177	0.43	705.93	3421.73	0.09
Awash Reach	8181.39*	2 Years	39.30	2060.08	2062.18		2062.19	0.000177	0.37	107.23	58.40	0.09
Awash Reach	8181.39*	5 Years	50.38	2060.08	2062.50		2062.51	0.000174	0.40	126.09	59.27	0.09
Awash Reach	8181.39*	50 Years	73.87	2060.08	2062.93		2062.94	0.000129	0.39	516.63	3363.97	0.08
Awash Reach	8181.39*	100 Years	80.69	2060.08	2062.96		2062.97	0.000126	0.38	609.07	3387.17	0.08
Awash Reach	8181.39*	500 Years	96.46	2060.08	2063.01		2063.02	0.000123	0.38	782.06	3430.19	0.08

HEC-RAS Plan: Modelling River: Awash River Reach: Awash Reach

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Awash Reach	7934.294	2 Years	39.30	2059.97	2062.14		2062.14	0.000162	0.36	109.91	58.15	0.08
Awash Reach	7934.294	5 Years	50.38	2059.97	2062.46		2062.47	0.000162	0.39	128.78	59.37	0.08
Awash Reach	7934.294	50 Years	73.87	2059.97	2062.90		2062.91	0.000130	0.39	482.82	3335.91	0.08
Awash Reach	7934.294	100 Years	80.69	2059.97	2062.93		2062.94	0.000128	0.39	576.58	3366.47	0.08
Awash Reach	7934.294	500 Years	96.46	2059.97	2062.98		2062.99	0.000126	0.39	750.88	3422.55	0.08
Awash Reach	7748.64*	2 Years	39.30	2059.95	2062.11		2062.11	0.000175	0.37	107.13	57.89	0.09
Awash Reach	7748.64*	5 Years	50.38	2059.95	2062.43		2062.44	0.000174	0.40	125.93	59.30	0.09
Awash Reach	7748.64*	50 Years	73.87	2059.95	2062.88		2062.89	0.000115	0.36	572.78	3372.55	0.07
Awash Reach	7748.64*	100 Years	80.69	2059.95	2062.91		2062.91	0.000111	0.36	669.52	3398.42	0.07
Awash Reach	7748.64*	500 Years	96.46	2059.95	2062.96		2062.97	0.000108	0.36	846.74	3445.31	0.07
Awash Reach	7562.991	2 Years	39.30	2059.89	2062.08		2062.08	0.000164	0.36	109.65	58.35	0.08
Awash Reach	7562.991	5 Years	50.38	2059.89	2062.40		2062.40	0.000164	0.39	128.63	59.84	0.09
Awash Reach	7562.991	50 Years	73.87	2059.89	2062.85		2062.86	0.000146	0.41	410.61	3286.26	0.08
Awash Reach	7562.991	100 Years	80.69	2059.89	2062.88		2062.89	0.000144	0.41	507.67	3334.90	0.08
Awash Reach	7562.991	500 Years	96.46	2059.89	2062.94		2062.94	0.000143	0.42	685.67	3422.30	0.08
Awash Reach	7298.233	2 Years	39.30	2059.75	2062.04		2062.05	0.000111	0.31	127.90	63.94	0.07
Awash Reach	7298.233	5 Years	50.38	2059.75	2062.36		2062.37	0.000114	0.34	148.66	65.55	0.07
Awash Reach	7298.233	50 Years	73.87	2059.75	2062.83		2062.83	0.000093	0.34	512.64	2868.86	0.07
Awash Reach	7298.233	100 Years	80.69	2059.75	2062.85		2062.86	0.000094	0.35	595.73	2887.25	0.07
Awash Reach	7298.233	500 Years	96.46	2059.75	2062.91		2062.91	0.000099	0.36	745.68	2918.36	0.07
Awash Reach	6933.711	2 Years	39.30	2059.66	2062.00		2062.00	0.000128	0.34	117.05	56.56	0.07
Awash Reach	6933.711	5 Years	50.38	2059.66	2062.32		2062.32	0.000133	0.37	135.22	57.37	0.08
Awash Reach	6933.711	50 Years	73.87	2059.66	2062.79		2062.80	0.000088	0.34	589.51	2837.80	0.06
Awash Reach	6933.711	100 Years	80.69	2059.66	2062.82		2062.83	0.000088	0.34	672.61	2860.07	0.06
Awash Reach	6933.711	500 Years	96.46	2059.66	2062.87		2062.88	0.000092	0.35	817.02	2898.37	0.07
Awash Reach	6624.353	2 Years	39.30	2059.57	2061.95		2061.96	0.000164	0.36	107.85	55.80	0.08
Awash Reach	6624.353	5 Years	50.38	2059.57	2062.27		2062.28	0.000165	0.40	125.68	56.41	0.09
Awash Reach	6624.353	50 Years	73.87	2059.57	2062.75		2062.76	0.000157	0.44	326.52	2325.52	0.09
Awash Reach	6624.353	100 Years	80.69	2059.57	2062.78		2062.79	0.000162	0.45	392.92	2346.26	0.09
Awash Reach	6624.353	500 Years	96.46	2059.57	2062.83		2062.84	0.000179	0.48	505.49	2381.02	0.09

HEC-RAS Plan: Modelling River: Awash River Reach: Awash Reach

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Awash Reach	6364.101	2 Years	39.30	2059.44	2061.89		2061.90	0.000281	0.47	84.45	45.36	0.11
Awash Reach	6364.101	5 Years	50.38	2059.44	2062.21		2062.22	0.000280	0.51	98.93	45.97	0.11
Awash Reach	6364.101	50 Years	73.87	2059.44	2062.71		2062.72	0.000199	0.49	431.09	3323.85	0.10
Awash Reach	6364.101	100 Years	80.69	2059.44	2062.74		2062.75	0.000186	0.47	532.61	3363.62	0.09
Awash Reach	6364.101	500 Years	96.46	2059.44	2062.78		2062.79	0.000180	0.47	692.76	3425.40	0.09
Awash Reach	5790.602	2 Years	39.30	2059.32	2061.75		2061.76	0.000219	0.41	96.36	52.56	0.10
Awash Reach	5790.602	5 Years	50.38	2059.32	2062.07		2062.08	0.000218	0.44	113.30	53.96	0.10
Awash Reach	5790.602	50 Years	73.87	2059.32	2062.59		2062.60	0.000201	0.48	318.59	3782.82	0.10
Awash Reach	5790.602	100 Years	80.69	2059.32	2062.63		2062.64	0.000181	0.46	472.91	3841.02	0.09
Awash Reach	5790.602	500 Years	96.46	2059.32	2062.68		2062.69	0.000171	0.46	673.77	3915.48	0.09
Awash Reach	5318.22*	2 Years	39.30	2059.16	2061.65		2061.66	0.000215	0.40	97.32	53.21	0.10
Awash Reach	5318.22*	5 Years	50.38	2059.16	2061.97		2061.98	0.000213	0.44	114.49	54.52	0.10
Awash Reach	5318.22*	50 Years	73.87	2059.16	2062.49		2062.50	0.000210	0.50	251.23	3650.66	0.10
Awash Reach	5318.22*	100 Years	80.69	2059.16	2062.54		2062.55	0.000180	0.47	451.55	3800.91	0.09
Awash Reach	5318.22*	500 Years	96.46	2059.16	2062.60		2062.61	0.000165	0.45	674.22	3891.53	0.09
Awash Reach	4845.844	2 Years	39.30	2058.99	2061.57		2061.57	0.000149	0.36	108.59	52.74	0.08
Awash Reach	4845.844	5 Years	50.38	2058.99	2061.88		2061.89	0.000155	0.40	125.42	53.40	0.08
Awash Reach	4845.844	50 Years	73.87	2058.99	2062.43		2062.44	0.000090	0.35	659.66	3858.37	0.07
Awash Reach	4845.844	100 Years	80.69	2058.99	2062.50		2062.50	0.000064	0.30	920.53	3910.14	0.06
Awash Reach	4845.844	500 Years	96.46	2058.99	2062.56		2062.57	0.000059	0.29	1160.42	3952.59	0.05
Awash Reach	4152.432	2 Years	39.30	2058.90	2061.49		2061.49	0.000088	0.30	131.24	56.28	0.06
Awash Reach	4152.432	5 Years	50.38	2058.90	2061.80		2061.81	0.000097	0.34	148.82	56.69	0.07
Awash Reach	4152.432	50 Years	73.87	2058.90	2062.36		2062.37	0.000112	0.41	180.93	57.74	0.07
Awash Reach	4152.432	100 Years	80.69	2058.90	2062.44		2062.45	0.000118	0.42	294.39	3252.42	0.08
Awash Reach	4152.432	500 Years	96.46	2058.90	2062.50		2062.51	0.000129	0.45	508.00	3543.42	0.08
Awash Reach	3788.898	2 Years	39.30	2058.84	2061.45		2061.46	0.000124	0.35	111.65	49.11	0.07
Awash Reach	3788.898	5 Years	50.38	2058.84	2061.76		2061.76	0.000138	0.40	126.90	50.32	0.08
Awash Reach	3788.898	50 Years	73.87	2058.84	2062.31		2062.32	0.000155	0.47	226.13	2680.13	0.09
Awash Reach	3788.898	100 Years	80.69	2058.84	2062.40		2062.40	0.000122	0.42	517.57	3829.66	0.08
Awash Reach	3788.898	500 Years	96.46	2058.84	2062.46		2062.46	0.000112	0.41	762.93	3910.25	0.07

HEC-RAS Plan: Modelling River: Awash River Reach: Awash Reach

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Awash Reach	3478.772	2 Years	39.30	2058.83	2061.40		2061.41	0.000190	0.43	91.15	39.97	0.09
Awash Reach	3478.772	5 Years	50.38	2058.83	2061.70		2061.71	0.000211	0.49	103.25	40.59	0.10
Awash Reach	3478.772	50 Years	73.87	2058.83	2062.25		2062.26	0.000247	0.59	127.39	262.78	0.11
Awash Reach	3478.772	100 Years	80.69	2058.83	2062.34		2062.35	0.000206	0.55	364.69	3171.77	0.10
Awash Reach	3478.772	500 Years	96.46	2058.83	2062.41		2062.42	0.000183	0.52	591.57	3301.43	0.09
Awash Reach	3099.14*	2 Years	39.30	2058.93	2061.33		2061.34	0.000181	0.40	98.09	47.08	0.09
Awash Reach	3099.14*	5 Years	50.38	2058.93	2061.62		2061.63	0.000196	0.45	112.04	47.95	0.09
Awash Reach	3099.14*	50 Years	73.87	2058.93	2062.16		2062.17	0.000222	0.53	138.18	49.88	0.10
Awash Reach	3099.14*	100 Years	80.69	2058.93	2062.27		2062.28	0.000180	0.49	402.32	3495.41	0.09
Awash Reach	3099.14*	500 Years	96.46	2058.93	2062.35		2062.36	0.000146	0.45	687.29	3583.22	0.08
Awash Reach	2719.527	2 Years	39.30	2058.90	2061.27		2061.28	0.000147	0.36	109.76	53.53	0.08
Awash Reach	2719.527	5 Years	50.38	2058.90	2061.56		2061.57	0.000160	0.40	125.35	54.48	0.08
Awash Reach	2719.527	50 Years	73.87	2058.90	2062.08		2062.10	0.000184	0.48	154.82	90.76	0.09
Awash Reach	2719.527	100 Years	80.69	2058.90	2062.21		2062.22	0.000134	0.42	506.80	3785.35	0.08
Awash Reach	2719.527	500 Years	96.46	2058.90	2062.31		2062.31	0.000097	0.36	864.79	3862.85	0.07
Awash Reach	2282.164	2 Years	39.30	2058.83	2061.21		2061.22	0.000122	0.34	116.84	53.61	0.07
Awash Reach	2282.164	5 Years	50.38	2058.83	2061.49		2061.50	0.000135	0.38	132.06	53.96	0.08
Awash Reach	2282.164	50 Years	73.87	2058.83	2062.01		2062.02	0.000157	0.46	160.01	54.60	0.09
Awash Reach	2282.164	100 Years	80.69	2058.83	2062.14		2062.16	0.000159	0.48	224.60	2496.39	0.09
Awash Reach	2282.164	500 Years	96.46	2058.83	2062.25		2062.26	0.000136	0.45	571.96	3321.07	0.08
Awash Reach	1974.154	2 Years	39.30	2058.93	2061.16		2061.17	0.000209	0.42	93.28	45.17	0.09
Awash Reach	1974.154	5 Years	50.38	2058.93	2061.44		2061.45	0.000230	0.48	105.84	45.47	0.10
Awash Reach	1974.154	50 Years	73.87	2058.93	2061.94		2061.96	0.000264	0.57	128.91	45.99	0.11
Awash Reach	1974.154	100 Years	80.69	2058.93	2062.07		2062.09	0.000271	0.60	145.33	1300.21	0.11
Awash Reach	1974.154	500 Years	96.46	2058.93	2062.20		2062.21	0.000214	0.54	507.06	3312.70	0.10
Awash Reach	1661.537	2 Years	39.30	2058.89	2061.10		2061.11	0.000166	0.38	103.41	49.25	0.08
Awash Reach	1661.537	5 Years	50.38	2058.89	2061.37		2061.38	0.000184	0.43	116.78	49.49	0.09
Awash Reach	1661.537	50 Years	73.87	2058.89	2061.87		2061.88	0.000215	0.52	141.40	49.98	0.10
Awash Reach	1661.537	100 Years	80.69	2058.89	2062.00		2062.01	0.000222	0.55	153.76	702.82	0.10
Awash Reach	1661.537	500 Years	96.46	2058.89	2062.14		2062.15	0.000174	0.50	517.71	2905.17	0.09

HEC-RAS Plan: Modelling River: Awash River Reach: Awash Reach												
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Awash Reach	1513.139	2 Years	39.30	2058.78	2061.07		2061.08	0.000244	0.42	94.43	54.23	0.10
Awash Reach	1513.139	5 Years	50.38	2058.78	2061.34		2061.35	0.000256	0.46	109.19	55.63	0.11
Awash Reach	1513.139	50 Years	73.87	2058.78	2061.83		2061.85	0.000276	0.54	137.25	58.61	0.11
Awash Reach	1513.139	100 Years	80.69	2058.78	2061.96		2061.98	0.000280	0.56	144.90	59.40	0.11
Awash Reach	1513.139	500 Years	96.46	2058.78	2062.10		2062.12	0.000277	0.57	370.34	3636.53	0.11
Awash Reach	1155.105	2 Years	39.30	2058.82	2060.97		2060.98	0.000346	0.48	81.65	48.91	0.12
Awash Reach	1155.105	5 Years	50.38	2058.82	2061.23		2061.24	0.000357	0.53	94.67	49.76	0.12
Awash Reach	1155.105	50 Years	73.87	2058.82	2061.71		2061.73	0.000374	0.62	119.06	51.20	0.13
Awash Reach	1155.105	100 Years	80.69	2058.82	2061.84		2061.86	0.000377	0.64	125.81	124.94	0.13
Awash Reach	1155.105	500 Years	96.46	2058.82	2062.00		2062.02	0.000303	0.60	440.45	3753.16	0.12
Awash Reach	951.3361	2 Years	39.30	2058.72	2060.88		2060.90	0.000464	0.54	73.14	46.41	0.14
Awash Reach	951.3361	5 Years	50.38	2058.72	2061.14		2061.16	0.000474	0.59	85.45	47.84	0.14
Awash Reach	951.3361	50 Years	73.87	2058.72	2061.62		2061.65	0.000489	0.68	109.04	50.58	0.15
Awash Reach	951.3361	100 Years	80.69	2058.72	2061.75		2061.77	0.000491	0.70	115.51	51.28	0.15
Awash Reach	951.3361	500 Years	96.46	2058.72	2061.90		2061.93	0.000577	0.78	123.32	52.12	0.16
Awash Reach	761.3241	2 Years	39.30	2058.67	2060.79		2060.81	0.000464	0.55	70.89	42.76	0.14
Awash Reach	761.3241	5 Years	50.38	2058.67	2061.05		2061.07	0.000485	0.61	82.03	43.75	0.14
Awash Reach	761.3241	50 Years	73.87	2058.67	2061.52		2061.55	0.000516	0.72	103.17	45.51	0.15
Awash Reach	761.3241	100 Years	80.69	2058.67	2061.65		2061.68	0.000522	0.74	108.92	46.01	0.15
Awash Reach	761.3241	500 Years	96.46	2058.67	2061.78		2061.82	0.000633	0.84	114.96	46.52	0.17
Awash Reach	474.2408	2 Years	39.30	2058.71	2060.65		2060.67	0.000513	0.57	68.88	42.72	0.14
Awash Reach	474.2408	5 Years	50.38	2058.71	2060.90		2060.92	0.000533	0.63	79.72	43.54	0.15
Awash Reach	474.2408	50 Years	73.87	2058.71	2061.37		2061.40	0.000559	0.74	100.31	44.91	0.16
Awash Reach	474.2408	100 Years	80.69	2058.71	2061.49		2061.52	0.000564	0.76	106.24	154.76	0.16
Awash Reach	474.2408	500 Years	96.46	2058.71	2061.73		2061.73	0.000136	0.40	953.35	4703.28	0.08
Awash Reach	311.927*	2 Years	39.30	2058.62	2060.55		2060.57	0.000696	0.64	61.45	40.56	0.17
Awash Reach	311.927*	5 Years	50.38	2058.62	2060.80		2060.82	0.000722	0.70	71.67	42.15	0.17
Awash Reach	311.927*	50 Years	73.87	2058.62	2061.26		2061.29	0.000743	0.81	91.71	44.76	0.18
Awash Reach	311.927*	100 Years	80.69	2058.62	2061.38		2061.42	0.000740	0.83	97.29	45.27	0.18
Awash Reach	311.927*	500 Years	96.46	2058.62	2061.65		2061.69	0.000751	0.88	109.46	46.97	0.18
Awash Reach	149.6138	2 Years	39.30	2058.67	2060.43		2060.45	0.000813	0.67	58.56	40.37	0.18
Awash Reach	149.6138	5 Years	50.38	2058.67	2060.67		2060.70	0.000837	0.73	68.57	42.13	0.18
Awash Reach	149.6138	50 Years	73.87	2058.67	2061.12		2061.16	0.000878	0.83	88.75	46.86	0.19
Awash Reach	149.6138	100 Years	80.69	2058.67	2061.25		2061.29	0.000878	0.85	94.67	48.21	0.19
Awash Reach	149.6138	500 Years	96.46	2058.67	2061.51		2061.56	0.000848	0.90	107.68	49.48	0.19
Awash Reach	54.0359	2 Years	39.30	2058.64	2060.36	2059.24	2060.38	0.000700	0.62	63.51	44.32	0.17
Awash Reach	54.0359	5 Years	50.38	2058.64	2060.60	2059.33	2060.62	0.000700	0.68	74.35	45.14	0.17
Awash Reach	54.0359	50 Years	73.87	2058.64	2061.05	2059.50	2061.08	0.000701	0.78	95.22	47.06	0.17
Awash Reach	54.0359	100 Years	80.69	2058.64	2061.18	2059.54	2061.21	0.000701	0.80	101.13	47.87	0.18
Awash Reach	54.0359	500 Years	96.46	2058.64	2061.44	2059.64	2061.48	0.000701	0.84	114.17	49.49	0.18

Annex B: Tables of Flood Risk Results

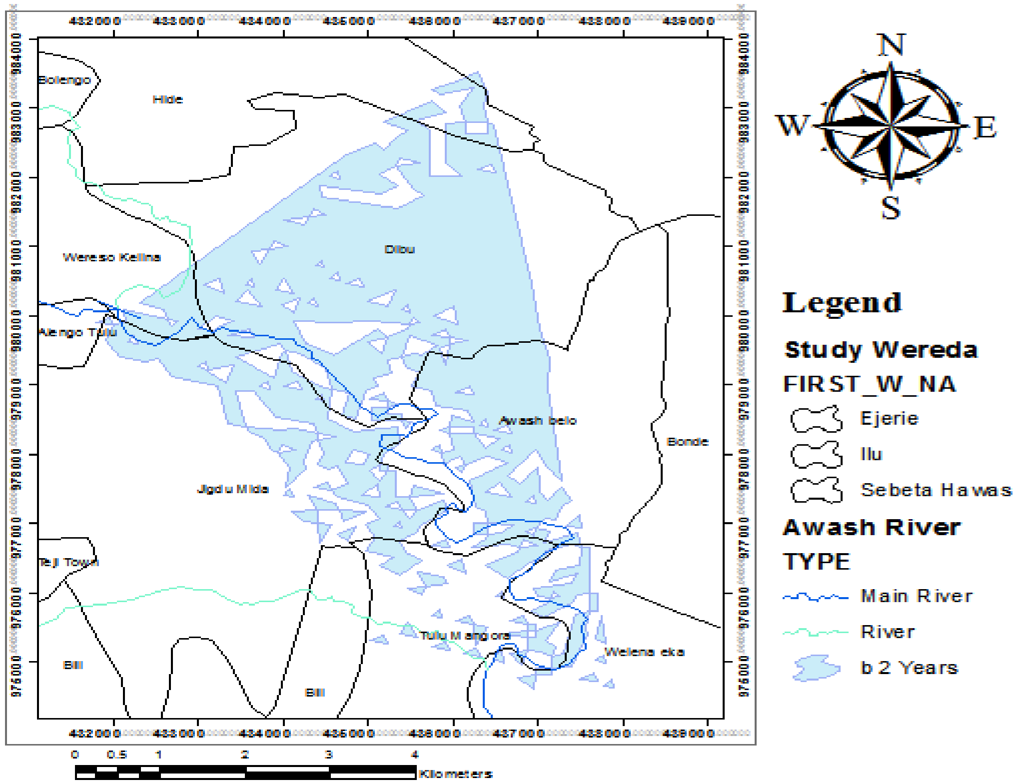
Table: - Flood magnitude and corresponding inundated area.

Return period (years)	2	5	50	100	500
Flow (m^3/s)	39.30	50.38	73.86	80.69	96.46
Inundated area (ha)	1,959.50	2,107.38	2,299.16	2,318.84	2,354.06

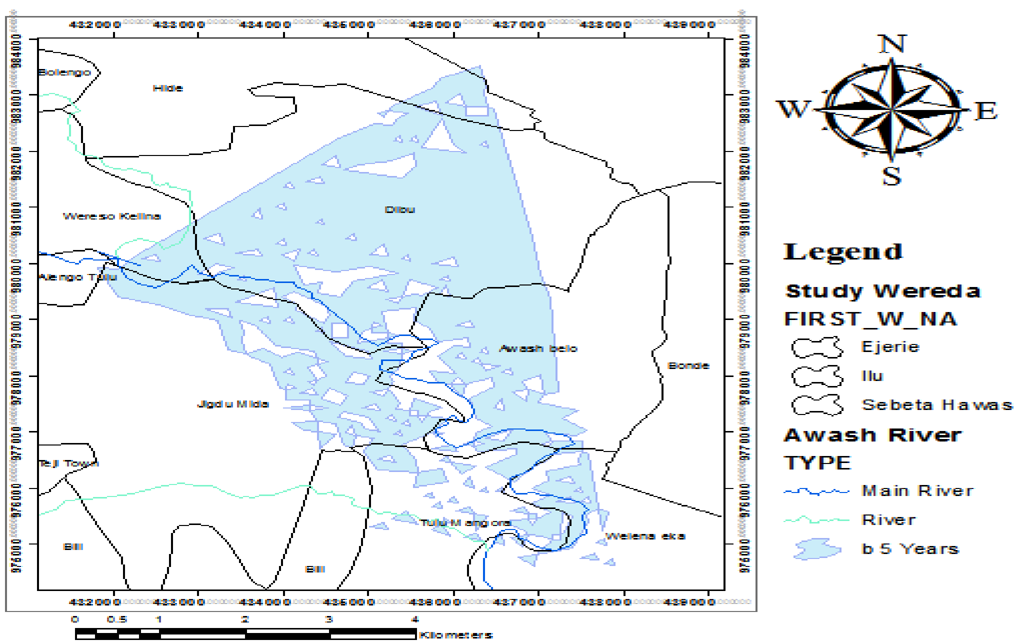
Table: - Estimated crop loss due to flooding

				Return Period				
				2 Yrs	**5 Yrs**	**50 Yrs**	**100 Yrs**	**500 Yrs**
				Flood Inundated Area				
No.	**Crop**	**Average**	**% of Coverage**	1,959.50	2,107.38	2,299.16	2,318.84	2,354.06
1	Teff	24	50	23513.95	25288.51	27589.88	27826.05	28248.67
2	Chick Peas	25	20	9797.48	10536.88	11495.78	11594.19	11770.28
3	Lentils	15	20	5878.49	6322.13	6897.47	6956.51	7062.17
4	Grass Pea	25	10	4898.74	5268.44	5747.89	5797.09	5885.14
Total Crop Loss				44,088.66	47,415.96	51,731.03	52,173.84	52,966.25

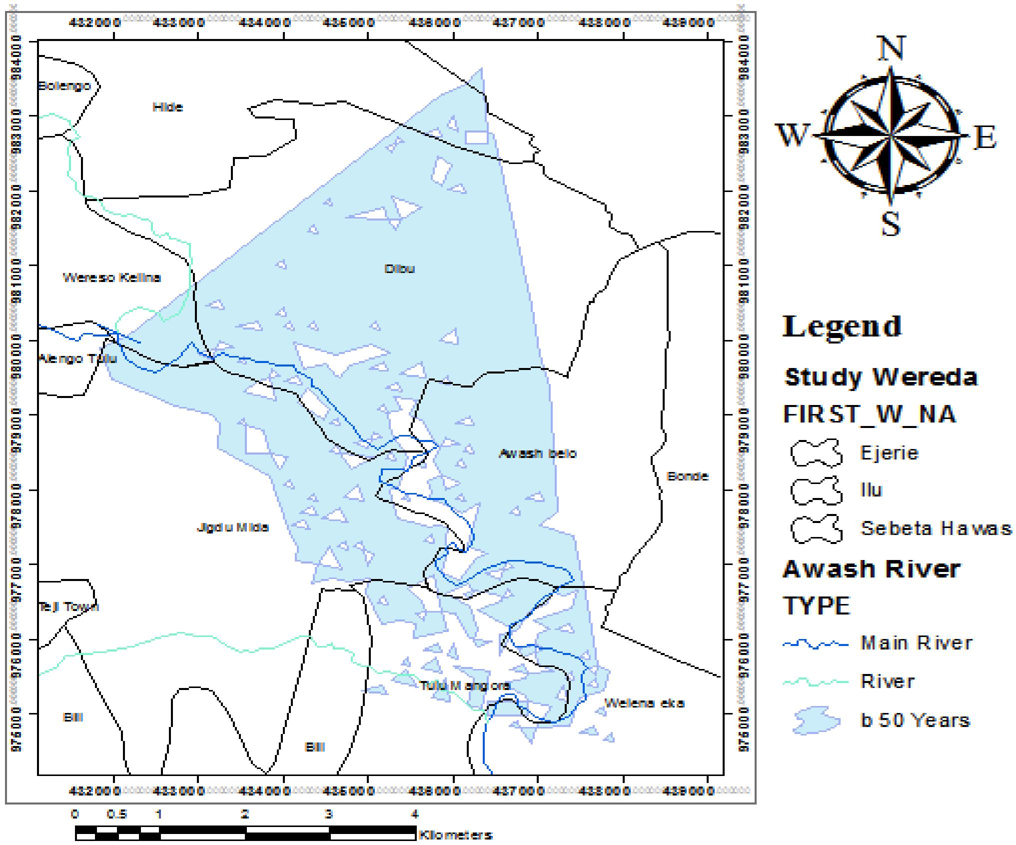
Annex C: Flood hhazard map



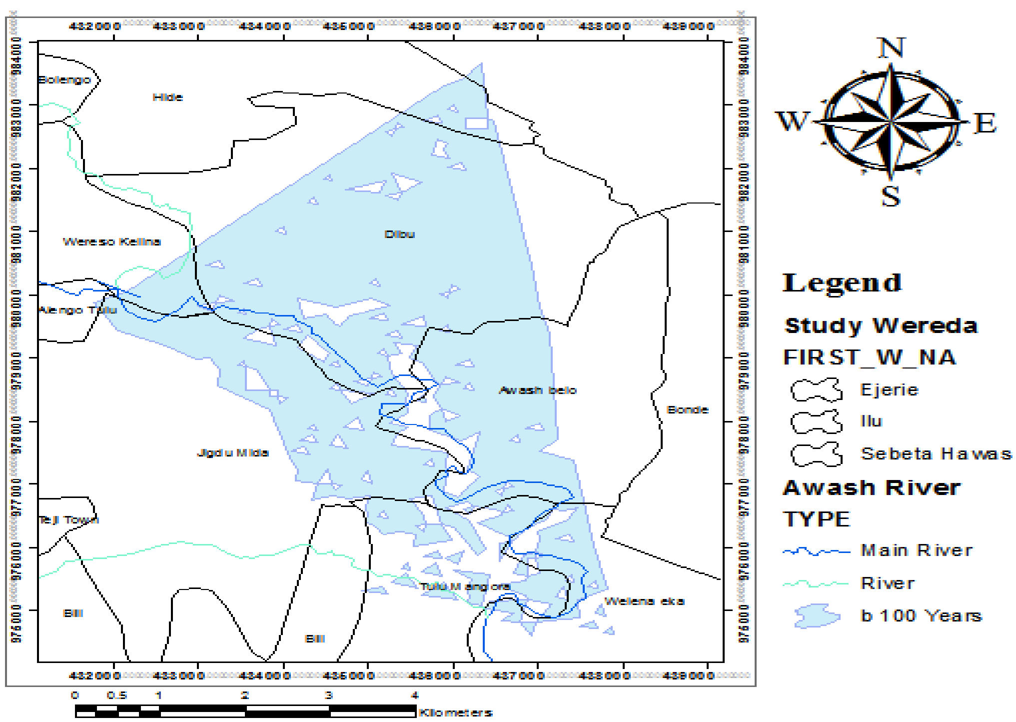
Flood Inundation mapping of 2 year return period



Flood Inundation mapping of 5 year return period

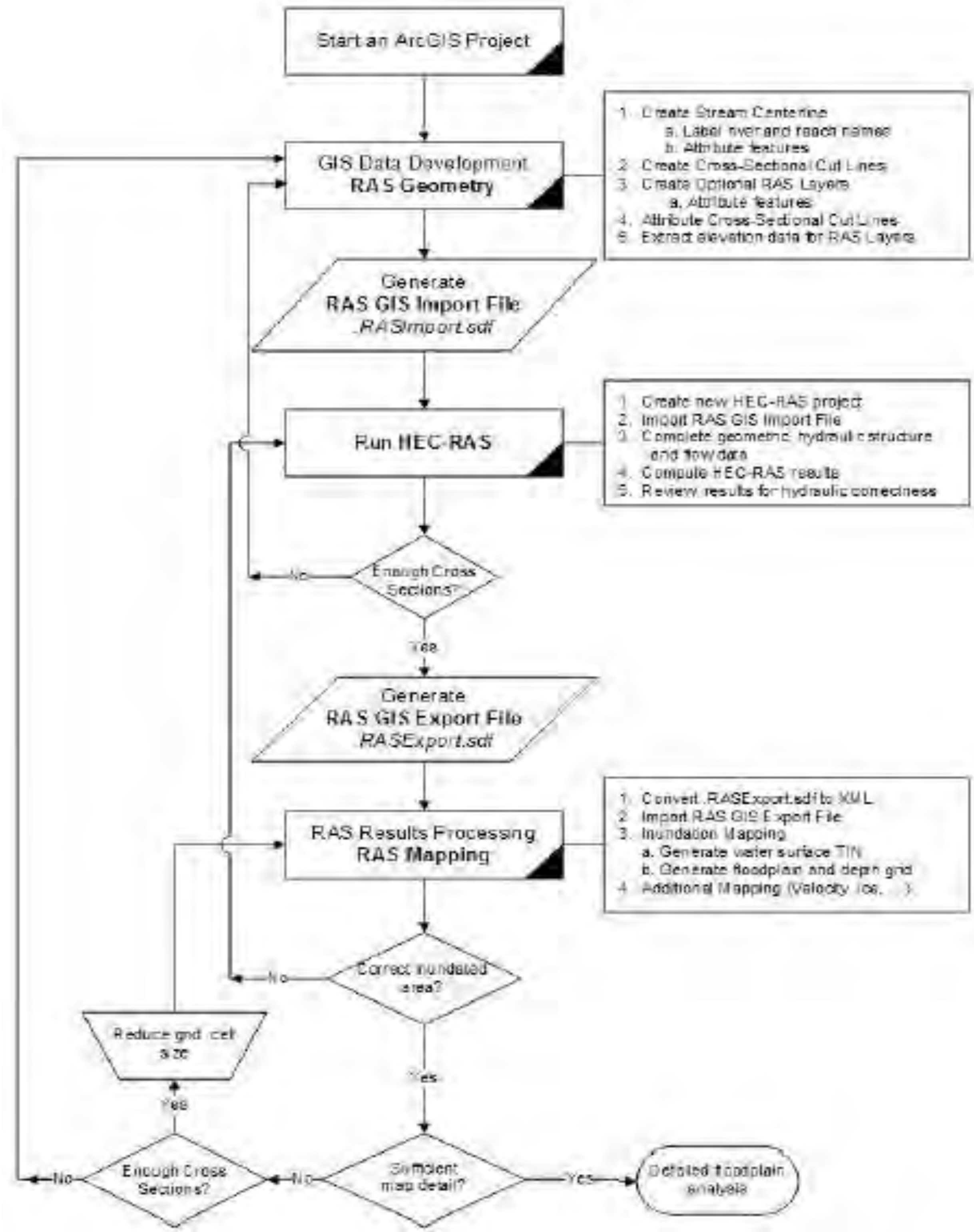


Flood Inundation mapping of 50 year return period



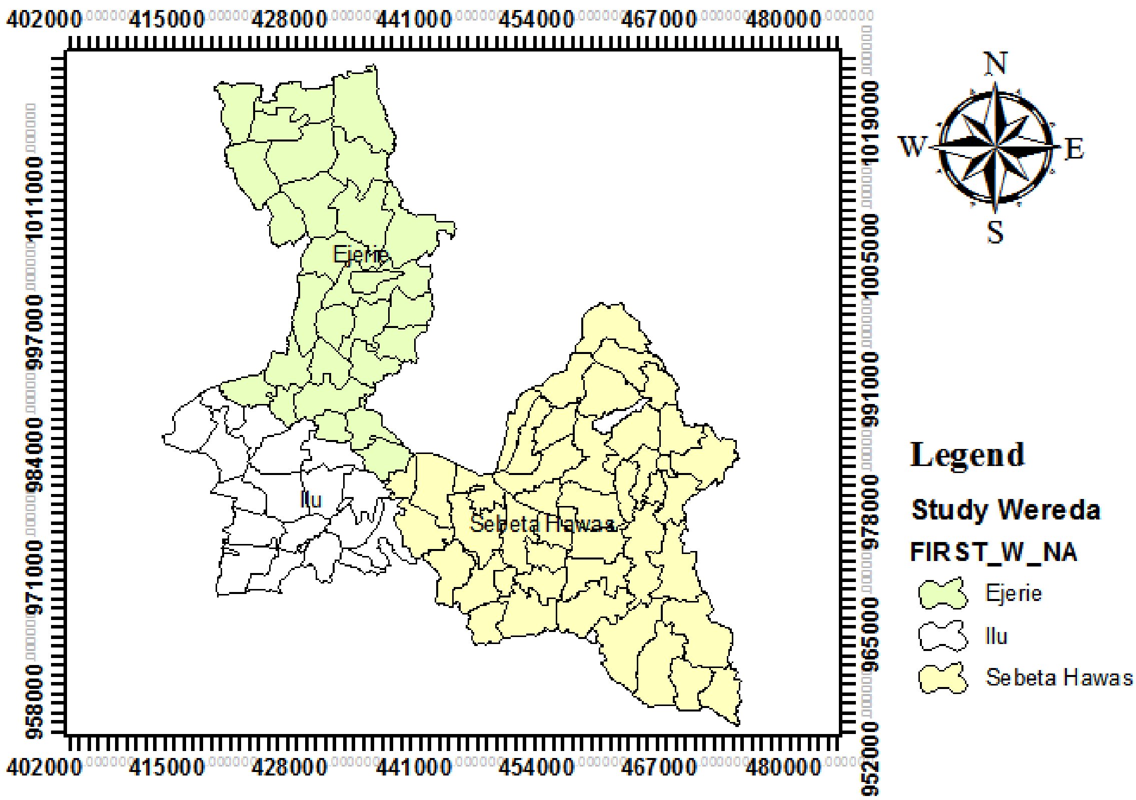
Flood Inundation mapping of 100 year return period

Annex D: General Overview of Hec-GeoRas



Annex E: List of data’s

1. Boundary Map of wereda (Source: Ministry of Water, Irrigation and Energy, GIS Department)



2 Maximum monthly stream flow data of Awash River at study area (Source: ministry of water, Irrigation and Energy, Hydrology Department).

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual Max Flow
1960	0.48	0.54	0.52	0.46	2.65	12.55	35.26	54.76	55.21	15.71	0.84	0.44	55.21
1961	0.35	0.42	0.25	0.79	0.6	4.21	50.38	59.88	49.53	76.67	2.54	1.09	76.67
1962	0.56	0.28	0.44	0.41	2.48	12.47	42.65	47.57	36.83	39.6	1.78	1.06	47.57
1963	0.95	0.44	0.42	0.44	4.36	20.73	34.93	35.26	24.13	2.54	1.02	1.02	35.26
1964	0.77	0.42	0.58	0.37	0.6	3.77	14.97	36.26	37.29	23.62	1.2	1.5	37.29
1965	0.73	0.41	0.39	0.54	0.73	10.94	22.15	34.27	30.81	11	5.99	0.98	34.27
1966	0.77	1.33	6.12	5.27	0.86	18.1	43.88	83.12	39.75	35.68	3.94	1.03	83.12
1967	0.33	0.28	0.73	0.56	1.54	10.71	20.73	23.37	48.69	60.37	1.9	1.09	60.37
1968	0.44	1.04	0.39	1.2	1.23	14.79	23.62	39.4	42.35	24.9	1.09	0.52	42.35
1969	0.86	3.36	18.1	3.84	1.09	14.6	32.67	31.73	33.94	5.19	0.77	0.54	33.94
1970	0.62	0.56	1.4	0.41	0.95	9.86	33.3	58.45	24.9	12.88	2.48	0.44	58.45
1971	0.35	0.21	0.97	0.33	2.33	15.34	30.81	33.94	33.94	25.96	3.36	0.67	33.94
1972	0.5	0.44	0.54	1.62	4.44	7.83	31.42	0.48	0.48	0.48	0.48	0.48	31.42
1973	0.48	0.48	0.48	0.48	0.48	0.48	26.08	13.91	37.99	15.15	1.2	0.64	37.99
1974	0.44	0.28	1.14	0.37	0.6	2.59	20.73	27.33	44.27	23.37	0.54	0.42	44.27
1975	0.37	0.41	0.26	0.41	1.09	14.79	33.3	50.38	32.67	12.72	0.67	0.56	50.38
1976	0.48	0.25	0.86	0.82	1.4	6.31	32.04	18.31	16.88	11	4.28	0.73	32.04
1977	0.52	0.75	0.5	1.2	0.41	1.62	20.73	43.11	26.78	22.15	17.69	8.19	43.11
1978	0.95	0.62	0.64	1.54	1.95	5.92	16.48	21.67	21.67	22.39	5.1	1.54	22.39
1979	4.21	0.5	0.39	0.73	2.09	3.11	16.88	26.78	29.92	22.39	3.24	0.98	29.92
1980	0.42	0.52	0.62	1.66	1.59	4.61	21.42	32.73	28.35	17.97	2.23	0.8	32.73
1981	0.67	0.46	0.6	1.1	1.09	6.12	25.96	38.69	26.78	13.55	1.23	0.62	38.69
1982	0.91	0.41	0.58	0.54	1.7	7.27	18.1	30.51	32.67	28.75	3.05	2.04	32.67
1983	0.73	0.69	1.29	1.26	2.28	6.62	22.15	55.21	31.73	18.95	9.06	2.04	55.21
1984	0.59	0.47	0.87	0.2	2.59	8.56	27.89	30.81	21.43	4.93	0.64	0.69	30.81
1985	0.44	0.26	0.44	0.91	3.77	12.39	30.51	32.04	36.6	16.68	7.83	0.79	36.6
1986	0.44	0.31	2.32	1.26	4.14	13.96	27.71	37.96	30.49	13.55	4.51	0.78	37.96
1987	0.44	0.35	4.21	1.62	4.52	15.52	24.9	43.88	24.38	10.42	1.2	0.77	43.88
1988	0.48	0.6	0.71	0.33	0.91	4.68	22.15	36.6	24.13	17.48	2.23	0.52	36.6
1989	0.44	1.23	0.95	6.62	2.7	24.13	16.68	23.62	27.05	17.18	1.54	0.48	27.05
1990	0.56	1.12	1.62	1.14	2.81	14.6	26.78	44.66	44.27	16.88	0.84	0.44	44.66

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual Max Flow
1991	0.44	0.41	0.56	0.35	0.86	2.87	16.09	30.81	18.1	5.01	0.52	0.52	30.81
1992	0.69	1.02	0.56	1.33	2.04	11	24.13	28.75	25.43	10.71	1.66	0.56	28.75
1993	0.3	1.09	0.46	3.24	6.52	23.12	25.16	25.69	23.31	7.69	6.83	0.73	25.69
1994	0.33	0.23	0.52	0.33	0.79	5.45	34.6	22.63	21.2	4.68	0.75	0.5	34.6
1995	0.41	0.41	0.56	1.33	1.2	3.63	14.97	33.3	25.16	6.31	0.84	0.69	33.3
1996	0.47	0.26	4.31	1.53	11.3	70.51	35.15	54.37	28.72	17.65	1.31	0.78	70.51
1997	0.81	0.41	0.41	1.35	1.64	14.59	33.65	28.72	20.24	17.15	6.07	3.34	33.65
1998	2.61	0.75	0.92	0.26	1.11	21.07	31.48	59.46	34.77	30.08	4	0.58	59.46
1999	0.43	0.26	0.45	0.35	1.57	8.84	30.42	37.87	27.72	23.06	3.18	0.51	37.87
2000	0.35	0.23	0.19	0.81	1.84	4.74	25.48	37.47	29.39	30.08	6.6	1.72	37.47
2001	0.41	0.37	1.8	2.01	4.74	39.66	32.69	28.18	29.05	12.59	2.33	0.49	39.66
2002	0.56	0.3	0.58	0.68	0.47	10.05	32.37	25.66	22.51	4.25	0.47	0.63	32.37
2003	0.41	0.58	0.75	0.89	0.33	2.42	25.66	35.25	30.53	18.23	0.6	0.49	35.25
2004	0.75	0.35	0.45	1.08	0.6	3.97	18.68	34.6	29.94	23.27	1.05	0.84	34.6
2005	0.81	0.39	0.45	0.93	1.97	15.02	36.67	41.3	30.28	17.1	1.51	1	41.3
2006	0.83	0.4	0.45	0.86	2.65	26.07	54.66	48.01	30.62	10.93	1.98	1.16	54.66
2007	0.86	0.42	0.45	0.79	3.33	19.89	32.14	43.64	34.77	21.92	0.83	0.38	43.64
2008	0.28	0.22	0.16	0.36	4.37	20.09	41.24	37.51	38.63	8.67	19.5	0.9	41.24
2009	0.47	0.36	0.42	6.06	0.86	2.15	25.39	43.94	33.44	20.89	1.38	1.21	43.94
2010	0.67	0.39	0.44	3.42	2.1	11.02	28.76	43.79	34.1	21.41	1.1	0.79	43.79

3. Cross section and channel profile Upper Awash River

4. Manning's roughness coefficient value (Cowan, 1956)

Table:-Recommended design values of manning roughness coefficients

	Manning n Range[b]
I. Unlined open channels[c]	
A. Earth, uniform section	
1. Clean, recently completed	0.016–0.018
2. Clean, after weathering	0.018–0.020
3. With short grass, few weeds	0.022–0.027
4. In graveled soil, uniform section, clean	0.022–0.025
B. Earth, fairly uniform section	
1. No vegetation	0.022–0.025
2. Grass, some weeds	0.025–0.030
3. Dense weeds or aquatic plants in deep channels	0.030–0.035
4. Sides, clean gravel bottom	0.025–0.030
5. Sides, clean, cobble bottom	0.030–0.040
C. Dragline excavated or dredged	
1. No vegetation	0.028–0.033
2. Light brush on banks	0.035–0.050
D. Rock	
1. Based on design section	0.035
2. Based on actual mean section	
a. Smooth and uniform	0.035–0.040
b. Jagged and irregular	0.040–0.045
E. Channels not maintained, weeds and brush uncut	
1. Dense weeds, high as flow depth	0.08–0.12
2. Clean bottom, brush on sides	0.05–0.08
3. Clean bottom, brush on sides, highest stage of flow	0.07–0.11
4. Dense brush, high-stage	0.10–0.14
II. Roadside channels and swales with maintained vegetation[d,e] (values shown are for velocities of 2 and 6 ft/sec):	
A. Depth of flow up to 0.7 ft	
1. Bermuda grass, Kentucky bluegrass, buffalo grass	
a. Mowed to 2 in.	0.07–0.045
b. Length 4 to 6 in.	0.09–0.05
2. Good stand, any grass	
a. Length about 12 in.	0.18–0.09
b. Length about 24 in.	0.30–0.15
3. Fair stand, any grass	
a. Length about 12 in.	0.14–0.08
b. Length about 24 in.	0.25–0.13

	Manning n Range[b]
B. Depth of flow 0.7–1.5 ft	
1. Bermuda grass, Kentucky bluegrass, buffalo grass	
a. Mowed to 2 in.	0.05–0.035
b. Length 4 to 6 in.	0.06–0.04
2. Good stand, any grass	
a. Length about 12 in.	0.12–0.07
b. Length about 24 in.	0.20–0.10
3. Fair stand, any grass	
a. Length about 12 in.	0.10–0.06
b. Length about 24 in.	0.17–0.09
III. Natural stream channels[f]	
A. Minor streams[g] (surface width at flood stage less than 100 ft)	
1. Fairly regular section	
a. Some grass and weeds, little or no brush	0.030–0.035
b. Dense growth of weeds, depth of flow materially greater than weed height	0.035–0.05
c. Some weeds, light brush on banks	0.04–0.05
d. Some weeds, heavy brush on banks	0.05–0.07
e. Some weeds, dense willows on banks	0.06–0.08
f. For trees within channel, with branches submerged at high stage, increase all above values by	0.01–0.10
2. Irregular sections, with pools, slight channel meander; increase value in la-e by	0.01–0.02
3. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage	
a. Bottom of gravel, cobbles, and few boulders	0.04–0.05
b. Bottom of cobbles, with large boulders	0.05–0.07
B. Floodplains (adjacent to natural streams)	
1. Pasture, no brush	
a. Short grass	0.030–0.035
b. High grass	0.035–0.05
2. Cultivated areas	
a. No crop	0.03–0.04
b. Mature row crops	0.035–0.045
c. Mature field crops	0.04–0.05
3. Heavy weeds, scattered brush	0.05–0.07
4. Light brush and trees[h]	
a. Winter	0.05–0.06
b. Summer	0.06–0.08

	Manning n Range[b]
5. Medium to dense brush[h]	
a. Winter	0.07–0.11
b. Summer	0.10–0.16
6. Dense willows, summer, not bent over by current	0.15–0.20
7. Cleared land with tree stumps, 100-150 per acre	
a. No sprouts	0.04–0.05
b. With heavy growth of sprouts	0.06–0.08
8. Heavy stand of timber, a few down trees, little undergrowth	
a. Flood depth below branches	0.10–0.12
b. Flood depth reaches branches	0.12–0.16
C. Major streams (surface width at flood stage more than 100 ft): Roughness coefficient is usually less than for minor streams of similar description on account of less effective resistance offered by irregular banks or vegetation on banks. Values of n may be somewhat reduced. Follow recommendation of note g if possible. The value of n for larger streams of most regular sections, with no boulders or brush, may be in the range shown.	0.028–0.033

Table 1:- Roughness Coefficient Modifiers (n_1)

Character of Channel	Basic *n*
Channels in earth	0.02
Channels cut into rock	0.025
Channels in fine gravel	0.024
Channels in coarse gravel	0.028

Source: Cowan, 1956.

Table 2:- Roughness Coefficient Modifiers (n_2)

Degree of Irregularity	Surface Comparable to	Modifying Value
Smooth	The best attainable for the materials involved	0.000
Minor	Good dredged channels; slightly eroded or scoured side slopes of canals or drainage channels	0.005
Moderate	Fair to poor dredged channels; moderately sloughed or eroded side slopes of canals or drainage channels	0.010
Severe	Badly sloughed banks of natural streams; badly eroded or sloughed sides of canals or drainage channels; unshaped, jagged, and irregular surfaces of channels excavated in rock	0.020

Source: Cowan, 1956.

Table 3:- Roughness Coefficient Modifiers (n_3)

Character of Variations of Size and Shape of Cross Sections	Modifying Value
Change in size or shape occurring gradually	0.000
Large and small sections alternating occasionally or shape changes causing occasional shifting of main flow from side to side	0.005
Large and small sections alternating frequently or shape changes causing frequent shifting of main flow from side to side	0.010–0.015

Source: Cowan, 1956.

Table 4:- Roughness Coefficient Modifiers (n_4)

Relative Effect of Obstructions	Modifying Value
Negligible	0.000
Minor	0.010–0.015
Appreciable	0.020–0.030
Severe	0.040–0.060

Source: Cowan, 1956.

Table 5:- Roughness Coefficient Modifiers (n_5)

Vegetation and Flow Conditions Comparable to:	Degree of Effect on n	Range in Modifying Value
Dense growths of flexible turf grasses or weeds, of which Bermuda grass and bluegrass are examples, where the average depth of flow is two or more times the height of the vegetation. Supple seedling tree switches, such as willow, cottonwood, or salt cedar, where the average depth of flow is three or more times the height of the vegetation.	Low	0.005–0.010
Turf grasses where the average depth of flow is one to two times the height of the vegetation. Stemmy grasses, weeds, or tree seedlings with moderate cover, where the average depth of flow is two to three times the height of the vegetation. Bushy growths, moderately dense, similar to willows one to two years old, dormant season, along side slopes with no significant vegetation along bottom, where the hydraulic radius is greater than 2 ft.	Medium	0.010–0.020
Turf grasses where the average depth of flow is about equal to the height of vegetation. Willow or cottonwood trees 8 to 10 years old intergrown with some weeds and brush, dormant season, where the hydraulic radius is 2 to 4 ft. Bushy willows about 1 year old interwoven with some weeds in full foliage along side slopes, no significant vegetation along channel bottom where hydraulic radius is 2 to 4 ft.	High	0.025–0.050
Turf grasses where the average depth of flow is less than one-half the height of the vegetation. Bushy willows about 1 year old intergrown with weeds along side slopes, dense growth of cattails along channel bottom, all vegetation in full foliage, any value of hydraulic radius up to 10 or 12 ft. Trees intergrown with weeds and brush, all vegetation in full foliage, any value of hydraulic radius up to 10 to 12 ft.	Very high	0.050–0.100

Source: Cowan, 1956.

Table 6:- Roughness Coefficient Modifiers (n_6)

Ratio of Meander Length to Straight Length	Degree of Meander	Modifying Value[a]
1.0–1.2	Minor	0.000
1.2–1.5	Appreciable	$0.15\ n_s$
1.5 and greater	Severe	$0.30\ n_s$

[a] $n_s = n_1 + n_2 + n_3 + n_4 + n_5$

Source: Cowan, 1956.