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SCHOOL OF GRADUATE STUDIES

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

FLOOD MITIGATION ALTERNATIVES FOR SELECTED REACHS OF AWASH RIVER FROM LOGIYA TO DUBTI

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A thesis submitted to the school of graduate studies in partial fulfillment of the requirements for the degree of master of science in Hydraulic Engineering at Addis Ababa University Institute of Technology, Addis Ababa, Ethiopia.

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This to certify that the thesis prepared by Tefera Brhanu Shibeshi ,entitled ; Flood mitigation alternatives for selected reachs of awash river from logiya to dubti and submitted in partial fulfillment of the required for the degree of masters of science in Civil and Environmental Engineering (Hydraulic Engineering Major) compiles with the regulations of the university and meets the accepted standards with respect to originality & quality.

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Abstract

Nowadays, extraordinary floods are common in many parts of Ethiopia causing a lot of losses to human lives as well as damage to property. Problems related to flooding have been inseparable in past decades in awash basin. The objective of the study is to evaluate various alternatives and select best engineering measures on basis of the performance of hydraulic parameters. A river model, Hydrologic Engineering Center River Analysis System, combined with GIS analysis was used to simulate water levels for steady, gradually varied flow and mapping inundated flood extents. The modelling was performed for four different alternatives, considering various channel modifications with different dimensions and levee construction. Using the hydraulic parameters such as flood inundation area, flood level, flow velocity and stream power on the downstream and outside of the river bend were used as evaluation and decision criteria for each alternatives. The performance of the hydraulic parameters demonstrates significant flood risk & environmental impact in agricultural and residential areas adjacent the rivers in selected reach of Awash River for proposed measures. A multiple decision-making method was used to find optimal flood mitigation alternative based on 50 and 100 years peak flood discharges in a selected reach of Awash River in between logiya & dubti. The decision analysis method, technique for order of preference by similarity to ideal solution, was used to compare different flood hazard mitigation measures based on flood risk, and environmental impacts criteria. The output of the analysis confirmed that a levee construction at the right side of the river bank adjacent to the residential area, which was superior to the other three alternatives.

Key words

HEC-RAS; Decision analysis; flood mitigation; TOPSIS.

List of Abbreviations

UNDP=United Nation Development Program

UNEP= United Nations Environment Programme

UNOCHA= United Nation Office for the Coordination of Humanitarian Affaires

DPPC=Disaster Prevention and Preparedness Commission

HEC RAS=Hydraulic Engineering Center River Analysis System

HEC GeoRAS= Hydrologic Engineering Centre-Geospatial River Analysis System

Arc GIS= Arc Geographic information system software

MOA =Minister of Agriculture

WHO=World meteorological organization

1|2 D=one| two Dimensional

DHI=Danish Hydraulic Institute

USACE= United States Army Corps of Engineers

DEM=Digital Elevation Model

TIN= Triangulated Irregular Network

MCDA=Multicriteria decision analysis

TOPSIS=Technique of Order Preference by Similarity to Ideal Solution

WWDSE=Water works design & supervision enterprise

FAO= Food and Agriculture Organization of the United Nations

MoWIE=Minister of Water, Irrigation and Energy

EMA=Ethiopian Mapping Agency

NMA=National Metrological Agency

USGS=United States geological survey

ESRI= Environmental System Research Institute

XS=cross section

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1. INTRODUCTION

1.1 Back ground

According to Tate E, et al (1994), floods are natural phenomena that occur when streams, rivers, and lakes overflow their banks. In the context of natural disasters, floods are defined by the amount of damage they cause to people or property. If people did not inhabit flood-prone areas, the natural phenomena of a river exceeding its notional capacity and overflowing into the surrounding areas leads a natural disaster, property damage of any country.

In Ethiopia, flooding is a common problem from June to September in river basins in which about 80% of the rains received. Awash River one of the major basins that have flooding disasters attributed to over flow or burst their banks and inundates downstream plain lands. Hence, basin is largely located in rift valley with high concentration of dense ancient as well as present day population in prospective domain of riviran areas clearly suggests the frequent flooding damage in the area.

Frequency and magnitude of flood have increased rapidly in the last few decades in the lower awash basin with magnificent contribution of Logiya River & Tendho dam emerged from rugged Mountain. According to Hal crow (2008) the increases in flooding in this area are results of climate change as well as land-use change (particularly deforestation). As consequences those, flooding causes loss of human lives and properties, destruction of roads, irrigation canals and other infrastructures that result huge economical loss for the country. Another notable consequence of flooding is crop destruction and subsequent malnutrition. Large-scale irrigated agriculture of the country is concentrated along the Awash River. Almost all of the area (200,000 - 250,000 ha) delineated for irrigation development is subjected to flood during high flows of the river (Kefeyalew, 2003). Irrigation development in this river basin is quite advanced and located in the flood plains on either side of the Awash River. Therefore, high economic damage occurs during flooding along this river in downstream of the dam.

Although, the numbers of flood control and management activity were being carried out in the lower Awash River Basin, but not in advanced stage. So as to reduce flood inundations and damages, the comparation and evaluation of feasible flood mitigation structure will be take on scientific base using GIS-based hydraulic modeling concerns in this thesis.

1.2 Statement of the problem

Flood is probably the most devastating, widespread and frequent natural hazard of the world that producing many socioeconomic and environmental consequences within the affected floodplains. Studies have shown that more than one third of the world's land area is flood prone, and about196million people in more than 90 countries are exposed to catastrophic flooding (UNDP, 2004; Dilley et al., 2005).

According to UNEP (2002), the major environmental disasters in Africa are recurrent droughts and floods. This problem is more acute in highland areas like Ethiopia under strong environmental degradation due to population pressure mainly linked with the national topography of highland mountains and lowland plains with natural drainage systems formed by the principal river basins. Particularly the rift valley regions of the country are prone to floods due to its connection with the highland system bounding the rift valley in both directions (kefyalew.2003).

The rainy season in Ethiopia is predominantly concentrated in four months from June to September. Large scale flooding is rare and limited to the lowland areas where major rivers cross to neighboring countries. However, intense rainfall in the highlands causes flooding of settlements close to any stretch of river courses. Among the major river flood-prone areas are parts of Oromia and Afar regions lying along the upper, mid and downstream plains of the Awash River.

The lower part of the Awash Basin is known as one of the flood prone areas by the annual flooding in the lower awash floodplain. In 2006, extreme flooding affected 53,140 and displaced 7840 peoples respectively in the region (UNOCHA, 2006).

The lower awash basin and the flooding danger have been inseparable for the past decade. Huge destruction occurred in 1993 and 1996.According to the Ethiopian Disaster Prevention and Preparedness Commission (DPPC) estimation the number of people affected at different flooding events in lower awash are shown table below.

Table 1 Flood events at the Awash Basin and the estimated damage (DPPC report)

Zones/woredas	Flood period	Remarks
Afambo(lower Valley)	1993	144,400
Dupti(lower valley)* **	1994	154,900
Assaita (lower valley)	1995	145,700 peoples were affected

(Source: Compiled from DPPC, 1996)

Even though flooding is frequently observed in the lower Awash plain, the flood management strategy has not gone beyond strengthening rescue and relief arrangements and other post flood measures which are totally devoid of any pre-flood management and planning strategy to minimize loss of life, property and environment.

Information regarding the flooding characteristics and its effect are essential for flood management bodies for decision making in flood management strategies such as construction of flood protection structures (engineering structures), to optimize the feasible engineering mitigation alternatives.

With the advancement of new technology on flood modeling such as the Hydraulic Modelling (HEC-RAS) and GIS these days. It is possible to model for evaluation of the various flood mitigation alternatives their flood extent, depth, velocity magnitude etc. in the spatial dimensions.

The need to conduct this research was due to the inexperience of previous studies the above applications for selecting the appropriate flood resistance engineering structures in the study area. Therefore, knowing the amount of flood & its extent (using hydraulic model & GIS), and morphology of the river have a paramount importance to arrive at feasible engineering solution.

1.3 Research question

- How to determine the frequency of flood magnitude for various return periods using statistical distribution?
- How can we evaluate for various alternative flood mitigations measures using hydraulic Parameter?
- How can we select the best flood mitigation measure using Multi criteria decision analysis?

1.4 Objective of the study

1.4.1 General objective

The overall objective is to compare & evaluate the various alternative measures and select the most appropriate one using GIS-based hydraulic modeling.

1.4.2 Specific objective

To attain the general objectives, completions of the following activities are required:

- Investigating the various best extreme flood frequency analysis for estimation of peak flow and select the appropriate one.
- Generating flood inundation maps for the baseline and various alternative flood mitigation measure for a 50 & 100 years floods.
- Evaluating the various alternatives on basis of selected hydraulic parameters using Multi and selecting the best option.

1.5 Significance of the study

The significance of this study is to contribute an input for evaluation the flood mitigation alternatives using hydraulic modeling with ArcGIS of other researcher. On the top of this to suggest the appropriate flood mitigation alternatives in the study area.

1.6 Scope and limitation

The scope of the study is to determine the hydrologic & hydraulic consequences associated with proposed alternatives of flood risk and environmental impact.

The limitation of evaluation of structural measures performed in this study only relies on performance of hydraulic parameters rather than other factors, such as construction cost of alternatives and socio _economic impact assessment, which were not included in present study.

1.7 Thesis structure

This thesis consists of seven chapters and is mainly focused on different river training measures such as channel modification & levee construction to find optimal approach for meeting risk and environmental criteria among various alternatives. Chapter two is focused on the description of the study area, river systems, topography, climate and flooding history of the study area. Chapter three is devoted to literature survey this includes some background and discussion concept of flooding & flood types, flood modelling, HEC-RAS modelling, integration of HecRAS & ArcGIS and importance of geographic information system. Previous studies on the area also form part of this chapter. Chapter four describes the materials and methods used in this research. Description of the models used and the model setup adopted is the main part of this chapter. Data collection and preparation also be found here. Chapter five focuses on the presentation and discussion of the result and chapter seven contains the conclusion and recommendations.

2. STUDY AREA DESCRIPTION

2.1 Location of study Area

Ethiopia is endowed with large amount of water resource potential with 12 major river basins. Among this, Awash river basin is one of the major river basins. It is located in central east with an area of 110,000 km^2 (Hacrow, 1989) with 10.1% out of total major river basins and concentrated with quite advanced irrigation development in either sides of the basin. Basin is bounded in between latitudes 7°53'N and 12°N and longitudes of 37°57'E and 43°25'E in Ethiopia.

The basin has great geographical, topographical and climatologically diversity, from high rugged mountains to deep gorges, from lowest altitude at about 120meter below sea level to highest altitude of 3000 meter above sea level, from 2000mm high annual rain fall to 200mm of low annual rain fall. This great variation of the altitude exposes some part of the country for creation of high flood. This topographic variability significantly dangerous in lower awash besides, the Great Rift Valley divides the country in to two parts forming the eastern and western high lands (Hal crow, 2008).

The study area of the selected reach is located in downstream of the proposed Tendaho Dam and extends up to Dubti. It is located between latitude 11^0 44'0'' to 11^0 42'0'' North and longitudes 40^0 57'30'' to 41^0 1'50'' East.

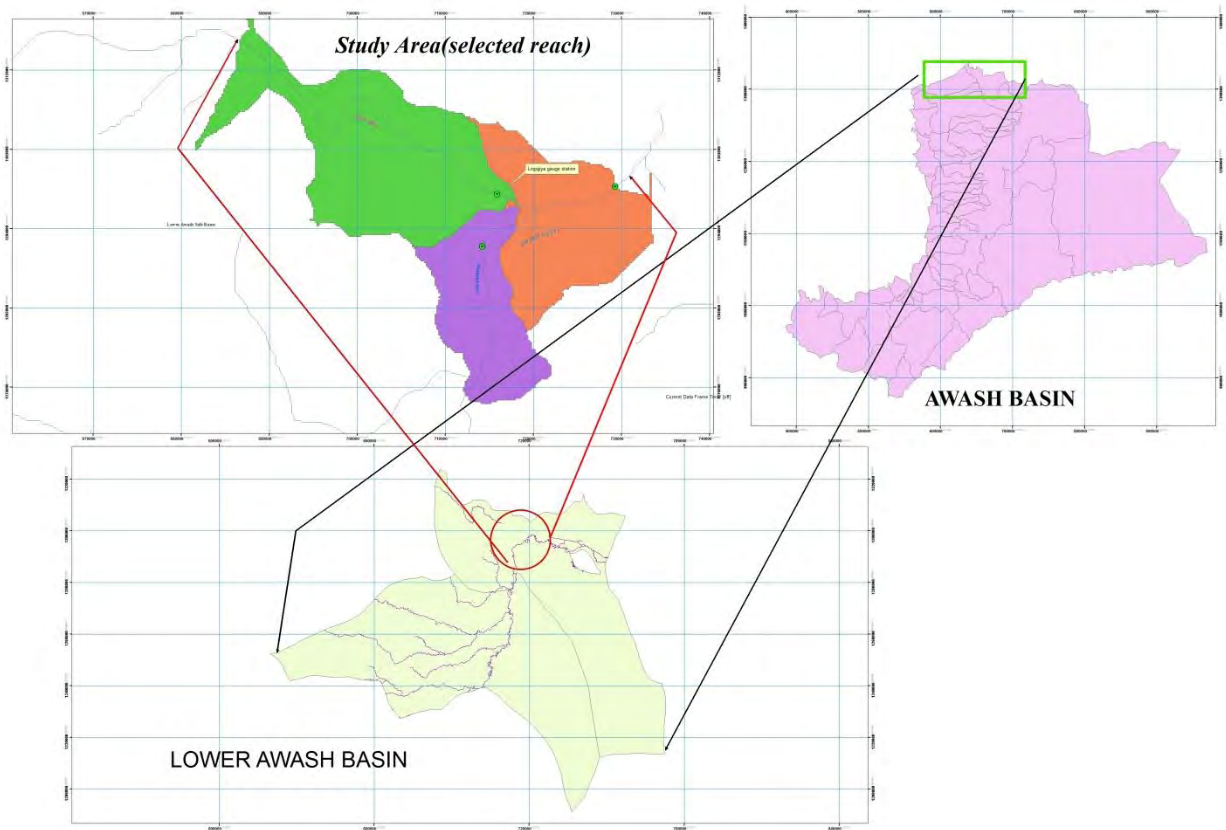

Figure 1 Location of Study Area in selected reach of Lower Awash river Basin

1.2. Topography

The Topography study area lies in the deltaic alluvial plains. The slope is very mild and varies within **0.05** to **0.1**m per km. The elevation of the command varies between 400m.a.s.l at Tendaho to 250m.a.s.l near Assayita. The area is prone to flooding by Awash River, the river has a tendency to change the course and significant changes in the course of the river have taken place in the past. The main stream of the river has split into parts at many places.

1.3. Climate

The climate of the study area is arid, with temperature varying from 17 to 43 degree Celsius, mean annual rainfall (200 mm), mean annual sunshine hours (9.8 hours), relative humidity varies from 31% in June to 96% in February and Wind speed from 187 Km/day(maximum March) to 95.4 km/day(minimum in September)(Awash synthesis report , 2013).

Temperature is highly associated with altitude of the area. It's variability has been increasing in the lower Awash much faster with bigger implications coupled with

declining and unreliable patterns in the rainfall. Average annual temperature ranges from 23^0c to 44^0c the hottest month are from February_September15.
According to several studies, the annual and monthly rainfalls are characterized by high variability. Spatially, annual rainfall varies from less than 200 mm at Logiya towards the east of the basin to 1700 mm at Ginager in the highlands northeast of Addis Ababa (Awash synthesis report, 2013).

1.4. Land uses|Land cover and soil

As per Awash Synthesis Report (2013) the land use|land cover map of Awash Basin was dominated by exposed rock (34.9%), cultivated land (27%), open shrub land (20.9%), Forest (7.4%) and grass land (6.6%). The current proportions of land use are expected to differ from the one shown in the table because of expansion of cultivated lands, urbanization and deforestation/land degradation as well as on-going catchment treatment activities. However, there is no up-to-date information on the changes of the different land uses.

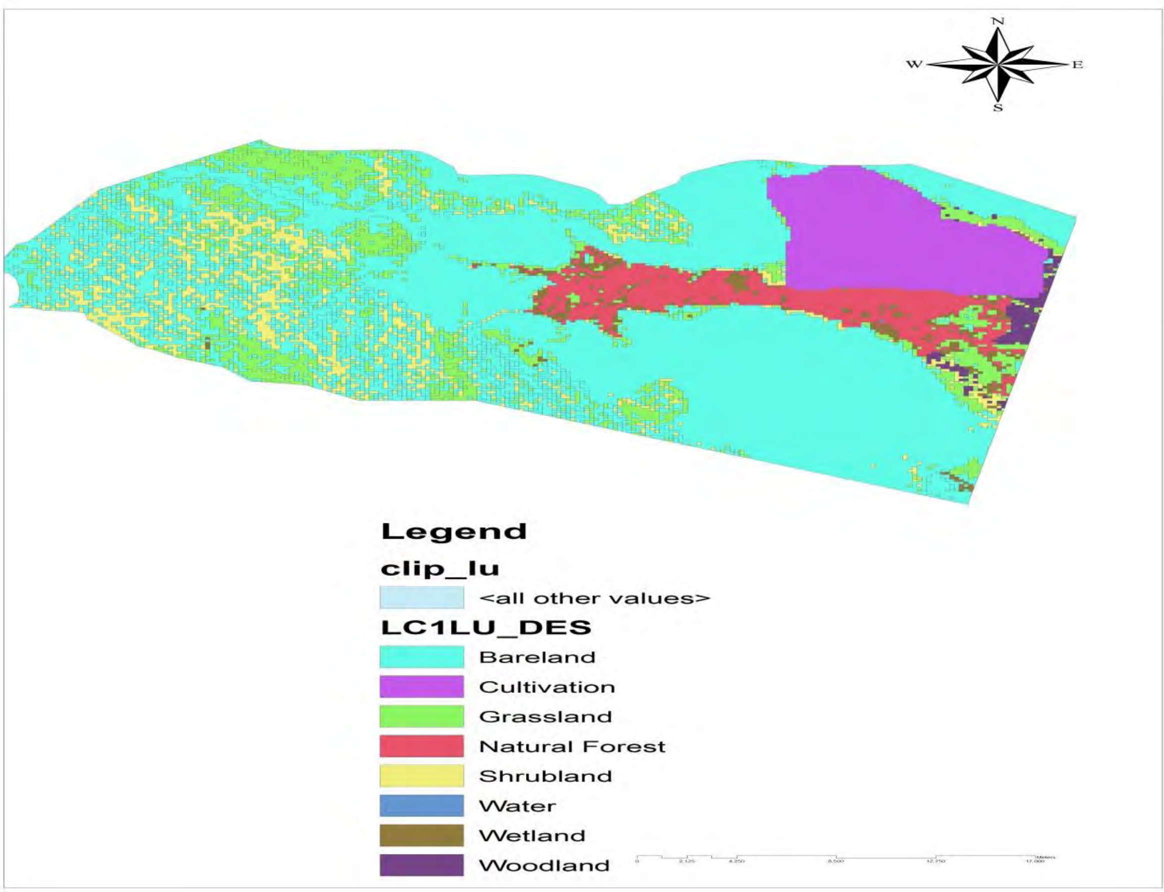

Figure 2 Landusemapofstudyarea

According to different studies, the Awash Basin exhibits 28 soil types dominated by Lithic Leptosols, Eutric Leptosols and Eutric Vertisols which cover about 70% of the total land area of the basin (Awash Synthesis Report, 2013).

Table 2 Land Use of Awash Basin Compiled by this Study

Land Use type	*Area(Km²)*	*%*
Cultivation Land	31263.4	26.980
Exposed rock	40412.3	34.88
Forest Land	8517.3	7.35
Grass Land	7623.9	6.58
Shrub Land	24246.8	20.92
Urban/Built Area	150.6	0.13
Water Body	3662.9	3.16
Total	115,877.2	

Source: MOA 2001

1.5. Flood characteristics

In recent years, flood hazards in Ethiopia have become more frequent and increasing severity. For instance, floods in 2006 have battered huge portions of eastern, southern and northern Ethiopia (ANRS, 2010). Floods that have also occurred in 2007 and 2008 have caused huge havoc on the livelihoods of many rural people. Recently repeated flash floods in the northern and eastern parts of Ethiopia have led to the loss of many lives and the destruction of household property and environmental resources (ANRS, 2010).

In Ethiopia, as per ANRS (2010) the issue of flood continues to be of growing concern to people residing in lowlands, near rivers, as well as towns located at the foot of hills and mountains. Flood disasters are occurring more frequently, and having an ever more dramatic impact on Ethiopia in terms of the costs on lives, livelihoods and environmental resources. Due to global climate change and local environmental pressures, the occurrence and frequency of flood hazards and the magnitude of destruction from floods are increasing through time.

Available evidences suggest that since the 1980s the Afar Region has been affected by floods. However, in recent times, the duration and intensity of floods have been increasing particularly in those low-lying areas and around settlements located near the lower Awash River. Many people associate flood and flash flood hazards in Afar with the torrential rainfall occurring in the central highland areas of the country. According to the World Meteorological Organization (WMO, 1994), flash floods can be categorized into three:

- Human-induced flash floods caused by the reduction of the stability of catchments or changes in its run-off, storage or hydraulic characteristics;
- Flash floods generated by sudden release of impounded water induced mainly by the failure of a dam or other manmade or natural barriers;
- Heavy rainfall generated flash floods.

Flash floods are aggravated over steep channel slopes with limited valley storage. They have the potential of causing land or mudslides. The flood hazards in Afar are the combined result of its topography, land cover, runoff from highland and intensive torrential rainfall condition (ANRS, 2010). Generally, the increase in the destructive nature of floods in the Afar Region can be partly attributed to climate variability and unsustainable practices from increased population (livestock and human) pressures on the environment.

The Awash River basin is mostly located in the arid lowlands of Afar Region in the northeastern part of Ethiopia. It frequently floods in August/September following heavy rains originating from Southern, Central and Northern Ethiopian highlands and escarpment areas (ANRS, 2010). A number of tributary certain areas which frequently, almost seasonally, get inundated are marshlands such as the area between the towns of Debel and Gewane in the vicinity of Lake Yardi and the lower plains around Dubti down to Lake Abe in the administrative Zone 1 of the Region(ANRS, 2010). As per ANRS (2010) over the years soil and water run-off in the escarpment areas has steadily increased as a result of deforestation, the most serious environmental degradation in the escarpment areas being caused by overpopulation in the highlands.

The vulnerability of the population living along the Awash River and in the marshlands has also been exacerbated due to seemingly inappropriate settlement patterns in these flood prone areas in recent years. During the dry season the riverside areas are the only places with grazing land and are essential for the survival of humans and livestock. In August and September 2010, high rainfall over the Awash catchment and the Amhara and Afar regions has led to flash flooding which has washed away bridges, residential places, destroyed crop fields, killed animals, destroyed schools (example Galifage School) and displaced people.

3. LITERATURE REVIEW

3.1 Flooding and its characteristics

The word flood originated from an old English word “flod”,as a word comparable to German’s “flute “and Duch’s “vloed “(Wikipedia,2007 b).Flooding is a broad term which means that an area is under water. The general public often refers to a flood as a high flow and water levels that may cause some damage to property and sometimes injuries and death. The United State Geological survey(2007) define a flood as relatively high water that overflows the natural or artificial banks of a stream or coastal area that submerges land not narrow below water .Flood is a natural phenomenon that occurs when the volume of water flowing in a system exceeds its total water holding capacity.

Depending upon the size of the basin and on the intensity and duration of the producing storms a flood may last from a few minutes, to hours, days, weeks, and sometimes even months. In general, a flood is a geophysical phenomenon that has a relatively quick onset and short duration. According Houston et al 2011, the causes of a flood vary and can by the origin be categorized.

- In *fluvial flooding* water level rises in rivers due to volume rich rain events in the sub-catchments upstream and the water level overtop the riverbanks and starting to spread on land.
- *In coastal flooding* sea level rises in over land, often due to a combination of storm surges, wave action and tide cycles.
- *Pluvial flooding* is associated with heavy rain events and occurs when precipitation flows on the surface and ends up in temporary ponds in depressions, due to insufficient conveyance capacity compared to rainfall intensity.

It is common to describe rain events by its frequency and consequently its return period.

ESCAP/UN (1997) outlined the important characteristics of floods which determines the magnitudes and the cost of disastrous effect .The list include the following

- *The peak depth of inundation*, which determines the extent and cost of damage to buildings and crops and the costs and feasibility of mitigation measures.
- *The areal extent of inundation*, which determine similar factors;

- *The duration of flooding*, which is an important factor in determining the effectiveness of flood warning and evacuation procedure;
- *The velocity of flood flow*, which determines the cost of flood damage and feasibility and design of levees and flood proofing structures;
- *The frequency of flooding* ,which express the statistical characteristics of flood events of given magnitude and determines the long-term average costs and benefits of flooding and flood mitigation ;and
- *The season ability of flooding*, which determines the cost of flood damage particularly where agricultural areas are inundated.

In recent history, floods are becoming more frequent & severe, and some organizations even counted it as the most damaging natural disaster in a region. The direct effects of floods includes; losses of lives, damage to property, disruption of transportations, communications, health and community services, crop and livestock damages, and interruption and loss in businesses. The indirect effects of floods are far more damaging. The negative impacts to economy in countries frequents by floods is usually severe especially if it is a poor or a developing.

The traumatic consequence of floods has called for the growing attention because of the need to prevent or control flood damage in our society. Mitigation flood damages can be drawn in two possible ways; structural measures and non-structural measures *(ESCAP/UN, 1991)*. Structural measures include the building of dikes, levees, dam and reservoirs and channel improvements designed to divert flood water away from people. These types of mitigation measure are proven effective but most often expensive.

Flood modeling which started from a very simple lumped model during its infancy days has now become very sophisticated with advance in remote sensing and geographic information system. During the early days of flood modeling, the usual problem that a modeler encountered was inadequacy of data. Detailed information regarding the hydrometer logical and topographical characteristics of a catchment is usually not known. However, with the introduction of remote sensing and geographic information, data scarcity problem was addressed. Nowadays, physically based-fully distributed models can be executed. Advancement in science which helps improved flood modeling was the development of discharge (rainfall) estimation algorithms which can be entered into a flood model .These algorithm help to extend the lead time in flood forecasting and is most useful for flash flooding rivers.

3.2 Flood frequency analysis

The first step for flood risk management is the estimation of the flood hazard for the region, this process can be done based on the study of triggering factors causing flood and/or investigation of spatial extent of historical events for given region, flood frequency and magnitude relationship estimation (*Geo-hazards, 2009*). To determine and quantify the flood frequency and flow variation within a given area the probabilistic approach tool is widely used *(Robson, 1999*). Gumbel extreme value distribution aims to build the relationship between the probabilities of the occurrence of a certain event, its return period and its magnitude (El-Naqa and Zeid, 1993). Different approaches of flood frequency statistical analysis for extreme events are given by Robson (1999). The first approach is based on estimation of peak flow and the event flow and the second technique was based on simulation techniques using parameter modeling in data poor regions *(Calver, 2009*). Pearson's statistics can be defined as significant tool for analysis of goodness of fit of the data and various observations for the same combination of explicative variables certain event, its return period and its magnitude (*El-Naqa and Zeid, 1993*). The allocation of best fitted probability functions can be studied using statistical reproduction for employment in peak flow (*Smyth, 2003*).

3.3 Hydraulic modeling of flood

Hydraulic modeling of natural rivers could be successfully analyzed with four equations: continuity, energy, momentum, and Manning. The Manning equation is considered to be empirical and is used to estimate friction loss while the energy equation is considered semi-empirical. (Dyhouse et al., 2003).

In hydraulic modeling, flow in the channel is simulated by solving the complete set of Saint Venant equations, first derived in 1871.This type of model is based on continuity equation (conservation of mass) and momentum equation (conservation of momentum).These equations are solved numerically by either explicit or implicit methods (Bedient et al., 1988).The explicit method solves the velocity and depth in a particular point in the river using the previously known data only. The implicit method solves the equations simultaneously at each time step and over all calculation points that cover the entire river. In order to derive the saint Venant equations, the following assumptions are often made (Chow et al., 1988).

1. The flow is one-dimensional; depth and velocity vary only in the longitudinal direction of the channel .This implies that the velocity is constant and the water surface is horizontal across any section perpendicular to the longitudinal axis.

2. Flow is assumed to vary gradually along the channel so that hydrostatic pressure prevails and vertical accelerations can be neglected.
3. The longitudinal axis of the channel is approximated as a straight line.
4. The bottom slope of the channel is small and the channel bed is fixed; that is, the effect of scour and deposition are negligible.
5. Resistance coefficients for steady uniform turbulent flow are applicable so that relationships such as manning's equation can be used to describe resistance effects.
6. The fluid is incompressible and of constant density thought the flow.

One-dimensional (1D) flow routing approaches such as Mike-11 (DHI, 1995) , FLDWAV (Fread et al., 1998) and HEC-RAS (USACE, 2002), based on the Saint Venant Equations or variations, still form the majority of traditional numerical hydraulic models used in practical river engineering. The widespread usage in practice might be explained not only by the fact that 1D models are (in comparison to higher dimensional hydraulic models) simpler to use and require a minimal amount of input data and computer power, but also because the basic concepts and programs have already been around for several decades (US Army Corps of Hydraulic Engineers, 2005; Pappenberger, 2005).

In this study, the hydraulic model used is HEC-RAS. HEC-RAS has been present in the public realm for more than 15 years and has been peer reviewed (HEC, 2010c). It is freely available for download from the HEC website and is supported by the US Army Corps of Engineers. It is also widely used by many government agencies and private firms.

3.3.1 HEC-RAS basic concepts and equations

HEC-RAS is an integrated software system, designed for interactive use in a multi-tasking environment and used to perform one-dimensional water surface calculations. HEC-RAS system is comprised of a graphical user interface, separate hydraulic analysis components, data storage and management capabilities, and graphing and reporting facilities (USACE, 2002 b).The most recent version of HEC-RAS supports steady and unsteady flow water surface profile calculations, sediment transport computations, and water temperature analysis (USACE, 2002 b). HEC-RAS is currently capable of performing one–dimensional water surface profile calculations for steady gradually varied flow in natural or constructed channels. It can handle a full network of channels or single river reach. Within steady flow it can be model subcritical, supercritical or mixed flow regime.

Computation engine of HEC-RAS is based on the solution of the one-dimensional energy equation. Energy losses are evaluated by friction (Manning's formula), contraction, and expansion. In cases where the water surface profile is rapidly varied, use of the momentum equitation is necessary. These cases include: mixed flow regime calculations, bridge hydraulic calculations and evaluation of profiles at river confluence. HEC-RAS is capable of calculating effects of bridges, culverts, dam's weirs, and other structures in the river and floodplain. The brief introduction of main concept and equations follows.

Steady and Unsteady Flow. Flow in an open channel is steady if the depth, discharge, and mean velocity of flow at a particular location does not change with time, or if it can be assumed constant during the time period under consideration. If the depth, discharge and velocity of flow at some point changes with time, the flow is unsteady. A time factor is taken into account explicitly in the case of unsteady flow analysis, while steady flow analysis neglect time factors altogether.

Uniform and Non *Uniform Flow*. We say that channel flow is uniform if the depth, the discharge and the mean velocity do not change in space. This implies that the energy grade line, water surface elevation and channel bottom are all parallel for uniform flow. This type of flow rarely occurs in reality. Non-uniform flow is sometimes designated as varied flow and can be further classified as gradually varied and rapidly varied. The flow is rapidly varied if the spatial changes to the flow occur rapidly and the pressure distribution is not hydrostatic, otherwise it is gradually varied. Based on these classifications the steady flow can be uniform or varied. The unsteady flow is usually varied, as the unsteady uniform flow is practically impossible, because it would require that the water surface fluctuates from time to time while remaining parallel to the channel bottom (Chow, 1959). The basic assumption of the gradually varied flow computation is that the streamlines are practically parallel and hydrostatic pressure distribution prevails over the channel section. The head loss at a section is the same as with a uniform flow that has the same hydraulic radius of the section. Accordingly, the uniform flow equation may be used to evaluate the energy slope of a gradually varied flow, while the corresponding coefficient of roughness developed primarily for uniform flow is applicable to the gradually varied flow (Chow, 1959). These assumptions are valid for most river flows including flood flows. The assumption of hydrostatic pressure distribution requires the stream to have a small slope of 1:10 or less. Most floodplain studies are performed on streams which meet this requirement (USACE, 2002 b).

Subcritical and Supercritical Flow. The effect of gravity upon the state of flow is defined by a ratio of inertial force to gravitational force as the dimensionless Froude Number.

$$F = \sqrt{gL} \ldots 3.1$$

Where,

F = Froude number (dimensionless)

V = mean channel flow velocity (m/s)

g = acceleration due to gravity (m/s^2) and

L = characteristic length (m).

In open channel flow, the characteristic length is often taken as the hydraulic depth D, which is defined as the cross sectional area channel normal to the direction of flow divided by the width of the free surface. The flow is classified as subcritical, critical or supercritical, depending on the Froude number. When the Froude number is less than 1, the effect of gravitational force is less than the inertial force and the state of flow is referred to as subcritical flow. When inertial and the gravitational forces are equal, the Froude number is equal to unity and the flow is said to be at the critical stage. When the inertial forces exceed the gravitational force, the Froude number is greater than 1, and the flow is referred to as supercritical flow. The flow regime is an important criterion for the calculation of water surface profiles. When the state of flow is subcritical, the state of flow is controlled by channel characteristics at the downstream end of the river reach. In the case of supercritical flow, the flow is governed by the upstream end of the river reach.

Continuity Equation. In the steady open channel flow analysis, the continuity equation states that flow remain constant between adjacent cross-sections.

$$Q = A_1V_1 = A_2V_2 \ldots 3.2$$

Where: Q = flow rate/discharge (m^3/s)

$V_{1,}V_2$ = mean flow velocity (m/s) and

$A_{1,}A_2$= cross-sectional flow area (m^2). ,

This equation allows tracing of changes in a cross-sectional area and velocity from location to location.

Energy Equation. Gradually varied water surface profiles are based on the principle of the conservation of energy, which states that the sum of the kinetic energy and potential energy at a particular cross section is equal to the sum of the potential and kinetic energy at any other cross section plus or minus energy loss or gains between the sections (Figure 3). Water surface is calculated from one cross section to the next by solving the energy equation written as:

$$Y_2 + Z_2 + \frac{\alpha_2 V_2^2}{2g} = Y_1 + Z_1 + \frac{\alpha_1 V_1^2}{2g} + he \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots 3.3$$

Where,

Y_1, Y2 = depth of water at cross sections (m)

Y_1,Y_2 = elevation of main channel inverts (m)

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

g = gravitational acceleration

α_2, α_1 = velocity weighting coefficients (dimensionless)   and ,

h_e= energy head loss (m).

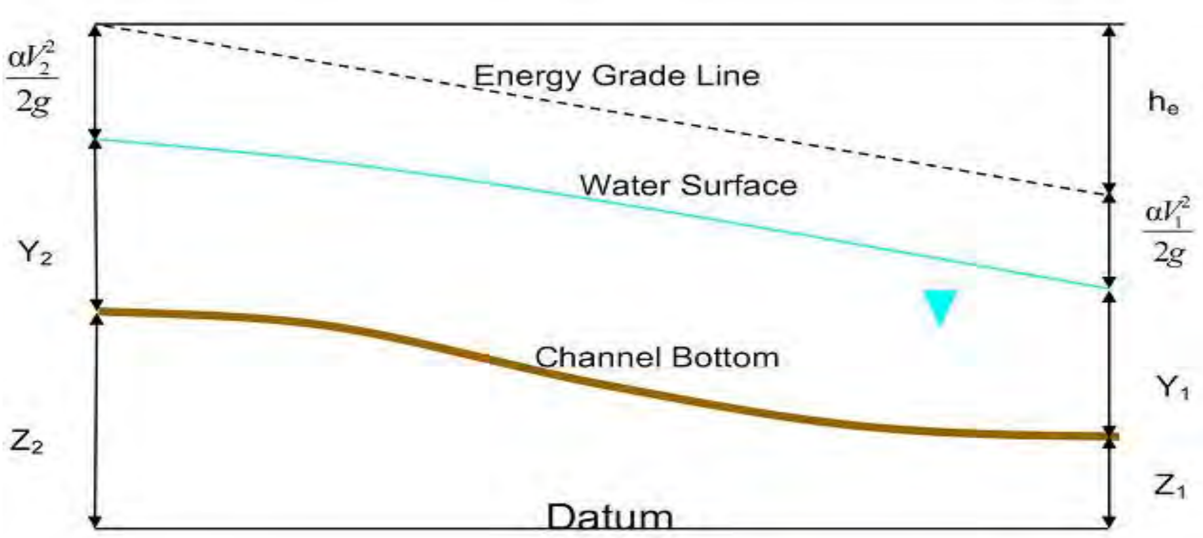

Figure 3 Representation of terms in the energy equation (after USACE, 2002 b)

Based on the energy equation, the energy head loss is the sum of friction losses and expansion, or contraction of coefficient.

$$he = L\bar{S}_f + c\left|\frac{\alpha_2 V_2}{2g} - \frac{\alpha_1 V_1}{2g}\right| \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots 3.4$$

Where,

L = reach length between the adjacent cross sections

S_f= friction slope between the two sections and

C = expansion or contraction loss coefficient (dimensionless).

The representative friction slope using the average conveyance equation and the distance weighted reach length are defined in Equation 3.5 and Equation 3.6, respectively.

$$\overline{Sf} = \left(\frac{Q_1 + Q_2}{K_1 + K_2}\right)^2 \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots 3.5$$

$$L = \frac{L_{lob}\bar{Q}_{lob} + L_{ch}\bar{Q}_{ch} + L_{roh}\bar{Q}_{roh}}{\bar{Q}_{lob} + \bar{Q}_{ch} + \quad \bar{Q}_{roh}} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots 3.6$$

Where K is conveyance, L_{lob}, L_{ch}, and L_{rob} are cross-section reach lengths for flow in the left over-bank, main channel, and right over-bank, respectively, and $\bar{Q}_{lob}$, $\bar{Q}_{ch}$, and $\bar{Q}_{rob}$ are arithmetic average of the flows between sections for the left over-bank, main channel, and right over-bank, respectively.

The magnitude of α depends upon the channel characteristics as shown in the table 3.

Table 3 Magnitude of α after (Debo and Reese, 2002)

Channel	Value of α		
	Min.	Avg.	Max.
Regular Channel	1.1	1.15	1.2
Natural Channel	1.15	1.3	1.5
Natural Channel-flooded overbanks	1.5	1.75	2

Manning's Loss Coefficient. The energy losses due to the roughness of the river bed are usually evaluated in terms of Manning's Equation:

$$Q = KS_f^{1/2} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots 3.7$$

$$K = \frac{1}{n} A R^{2/3} \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots 3.8.$$

Where:

K = conveyance of the section ($m^3 s^{-1}$)

n = Manning's roughness coefficient ($m^{-1/3} S$) and

R = hydraulic radius (m).

Selecting the appropriate Manning's n value is very important for accurate computation of water surface profiles. The value of Manning's n is highly variable and depends upon a number of factors including: surface roughness, channel irregularities, channel alignment, size and shape of channel, scour and deposition, vegetation, obstructions, stage and discharge, seasonal change, temperature, suspended materials and bed load (USACE,2002 b). The **n** value decreases with increases in stage and discharge. When the water depth is shallow, irregularities of the channel bottom are exposed and their effect may become pronounced. However, the n value may be large at high stages if the banks are rough and grassy (Chow, 1959).

If there is observed water surface data (high water marks, gagged data), Manning's n values should be calibrated. If there is no observed data (like in this study), then values of n obtained from another stream with similar conditions should be used. There are several references available listing the typical n values. Excerpts of the n value from Chow (1959) for natural streams are given in the Table 4.

Table 4 Manning's n values (Chow., 1959)

Type of Channel and Description	Minimum	Normal	Maximum
A. *Natural Streams*			
1. **Main Channels**			
a. Clean, straight, full, no rifts or deep pools	0.025	0.030	0.033
b. Same as above, but more stones and weeds	0.030	0.035	0.040
c. Clean, winding, some pools and shoals	0.033	0.040	0.045
d. Same as above, but some weeds and stones	0.035	0.045	0.050
e. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
f. Same as "d" but more stones	0.045	0.050	0.060
g. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
h. Very weedy reaches, deep pools, or floodways with heavy stands of timber and brush	0.070	0.100	0.150
2. **Flood Plains**			
a. Pasture no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated areas			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees, in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
2. Same as above, but heavy sprouts	0.050	0.060	0.080
3. Heavy stand of timber, few down trees, little undergrowth, flow below branches	0.080	0.100	0.120
4. Same as above, but with flow into branches	0.100	0.120	0.160
5. Dense willows, summer, straight	0.110	0.150	0.200
3. **Mountain Streams, no vegetation in channel, banks usually steep, with trees and brush on banks submerged**			
a. Bottom: gravels, cobbles, and few boulders	0.030	0.040	0.050
b. Bottom: cobbles with large boulders	0.040	0.050	0.070

Table 4 Manning's n values-continued

Type of Channel and Description	Minimum	Normal	Maximum
B. *Lined or Built-Up Channels*			
1. **Concrete**			
a. Trowel finish	0.011	0.013	
b. Float Finish	0.013	0.015	0.015
c. Finished, with gravel bottom	0.015	0.017	0.016
d. Unfinished	0.014	0.017	0.020
e. Gunite, good section	0.016	0.019	0.020
f. Gunite, wavy section	0.018	0.022	0.023
g. On good excavated rock	0.017	0.020	0.025
h. On irregular excavated rock	0.022	0.027	
2. **Concrete bottom float finished with sides of:**			
a. Dressed stone in mortar	0.015	0.017	0.020
b. Random stone in mortar	0.017	0.020	0.024
c. Cement rubble masonry, plastered	0.016	0.020	0.024
d. Cement rubble masonry	0.020	0.025	0.030
e. Dry rubble on riprap	0.020	0.030	0.035
3. **Gravel bottom with sides of:**			
a. Formed concrete	0.017	0.020	0.025
b. Random stone in mortar	0.020	0.023	0.026
c. Dry rubble or riprap	0.023	0.033	0.036
4. **Brick**			
a. Glazed	0.011	0.013	0.015
b. In cement mortar	0.012	0.015	0.018
5. **Metal**			
a. Smooth steel surfaces	0.011	0.012	0.014
b. Corrugated metal	0.021	0.025	0.030
6. **Asphalt**			
a. Smooth	0.013	0.013	
b. Rough	0.016	0.016	
7. **Vegetal lining**	0.030		0.500

Table 4 Manning’s n values-continued

Type of Channel and Description	Minimum	Normal	Maximum
C. *Excavated or Dredged Channels*			
1. **Earth, straight and uniform**			
a. Clean, recently completed	0.016	0.018	0.020
b. Clean, after weathering	0.018	0.022	0.025
c. Gravel, uniform section, clean	0.022	0.025	0.030
d. With short grass, few weeds	0.022	0.027	0.033
2. **Earth, winding and sluggish**			
a. No vegetation	0.023	0.025	0.030
b. Grass, some weeds	0.025	0.030	0.033
c. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
d. Earth bottom and rubble side	0.028	0.030	0.035
e. Stony bottom and weedy banks	0.025	0.035	0.040
f. Cobble bottom and clean sides	0.030	0.040	0.050
3. **Dragline-excavated or dredged**			
a. No vegetation	0.025	0.028	0.033
b. Light brush on banks	0.035	0.050	0.060
4. **Rock cuts**			
a. Smooth and uniform	0.025	0.035	0.040
b. Jagged and irregular	0.035	0.040	0.050
5. **Channels not maintained, weeds and brush**			
a. Clean bottom, brush on sides	0.040	0.050	0.080
b. Same as above, highest stage of flow	0.045	0.070	0.110
c. Dense weeds, high as flow depth	0.050	0.080	0.120
d. Dense brush, high stage	0.080	0.100	0.140

There are various methods and empirical formulae available for the estimation of the manning's n value. Cowen (1956) developed a procedure for the estimation of formula for **n** as a function of type and size of the bed materials and channel properties. In Cowen's procedure, the n value is determined by the following equation:

$n = (n_b + n_1 + n_2 + n_3 + n_4)m$ … .3.9

Where,

n_b= base value of n for straight uniform & smooth channel in natural materials

n_1 = value added to correct surface irregularities

n_2= value for variation in size and shape of channel

n_3 = value for obstructions

n_4 = value for vegetation and flow conditions and

m = correcting factor to take account of the meandering of channel.

Limerinos (1970) related the n related value as the function of bed materials and hydraulic radius:

$$n = \frac{0.0926R^{-0.16}}{1.16 + 2\log(\frac{R}{d_{84}})} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.10$$

Where,

R= hydraulic radius (feet) and

d_{84}= particle size diameter that is equal to or exceeds 84% of particle (feet).

Limerinos selected the reaches that had a minimum amount of roughness that were caused by factors other than the bed materials, and so these base n values should be increased to take account of other factors as shown in Cowen's method (USACE, 2002 b). This equation was developed for the data range of hydraulic radius 1.0 to 6.0 and d_{84} 1.5 to 250 mm.

Expansion and Contraction Coefficients. The following equation is used for the determination of contraction and expansion losses.

$$hCe = C\left|\frac{\alpha_2 V_2}{2g} - \frac{\alpha_1 V_1}{2g}\right| \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.11$$

Where:-C = the contraction or expansion coefficient.

The coefficient of expansion and the coefficient of contraction are introduced to take into account losses due to the expansion or contraction of flow caused by changes in the cross sections. The losses due to these fluctuations are significant, particularly at points where there is an abrupt change in the cross section – for example at bridges. Typical values of expansion and contraction coefficients for subcritical flow are given in Table 5.

Table 5 *Subcritical Flow Expansion and Contraction Coefficient (USACE, 2002 b)*

Type of Channel	Contraction	Expansion
No transition	0.00	0.00
Gradual transition	0.10	0.30
Typical bridge sections	0.30	0.50
Abrupt transition	0.60	0.80

Friction Loss Evaluation. Manning's equation is used for the calculation of energy slope as follows:

$$S_f = \left(\frac{Q}{k}\right)^2 \quad \ldots\ldots 3.12$$

There are also a few other alternative expressions for the representation of reach friction slope in HEC-RAS computer program.

Computation method

The method of computation of water surface profiles for gradually varied flow is based on the assumption that the slope of the energy grade line at a section is equal to the energy slope for a uniform flow with the velocity and hydraulic radius of the section (Chow, 1959). Some of the basic steps in the computation of water surface profiles in HEC-RAS are explained below.

Cross Section Subdivision for Conveyance Calculations. The determination of the conveyance coefficient in HEC-RAS involves subdivision of flow into units based on Manning's coefficient n (Figure 4). The conveyance for each subdivision is calculated by using Equation (3.8).The total conveyance for the cross section is obtained by adding the three subdivision conveyances (left, channel, and right).

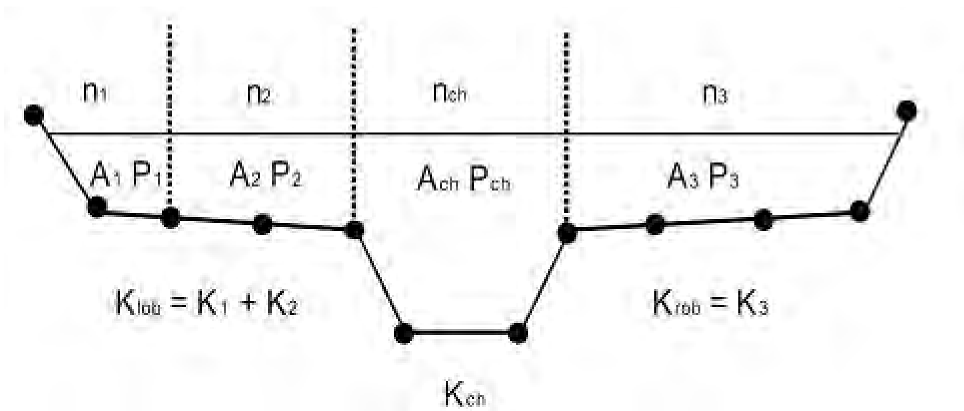

Figure 4 Conveyance Subdivision Method (after USACE, 2002 b)

Mean Kinetic Energy Head Calculation. Mean kinetic energy head for each cross section is obtained by computing the flow weighted kinetic energy heads for three subsections of the cross sections (main channel, right and left overbank). Figure 5 illustrates the mean kinetic energy calculation process for the cross section with the main channel and the right overbank.

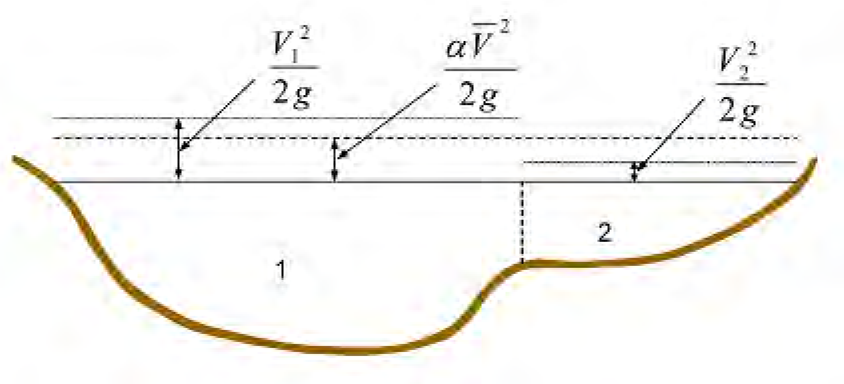

Figure 5 Example of Mean Energy calculations (after USACE, 2002 b)

$\boldsymbol{V_1}$ is mean velocity for the main channel and $\boldsymbol{V_2}$ is mean velocity for the right overbank. The calculation of the mean energy head requires the velocity weighing coefficient α .

The following equation, which is written in terms of conveyance and area, shows the calculation of α:

$$\alpha = (A_t)^2 \frac{\left[\frac{K_{lob}^3}{A_{lob}^2} + \frac{K_{ch}^3}{A_{ch}^2} + \frac{K_{rob}^3}{A_{rob}^2}\right]}{K_t^3} \qquad 3.13$$

A_t =total flow area of the cross section

A_{lob}, A_{ch}, A_{rob}=flow areas of left overbank, main channel, and right over bank, respectively

K_t =total conveyance of the cross section and

k_{lob}, k_{ch}, k_{rob}= =conveyance of left overbank, main channel and right overbank respectively.

Standard Step Method. The Standard Step Method can be used for both prismatic and non-prismatic channels, including the adjacent floodplain. This method can be applied to compute steady, gradually varied flow, and can also be used for both subcritical and supercritical flow.

The computation for this method is based on the energy Equation (3.3) by steps from station to station. Depending upon the conditions of flow (subcritical or supercritical), the computations must be made in different directions. For subcritical flow that is under downstream control, the computation starts from downstream and proceeds upstream. For supercritical flow that is under upstream control, the computation starts from upstream and proceeds downstream. The computation steps used in this procedure for the subcritical flow are as follows (USACE, 2002 b):

1. Assume the water surface elevation at the upstream cross section.
2. Based on the assumed water surface elevation, determine the corresponding total conveyance and velocity head.
3. With values from step 2, calculate the frictional slope S_f and solve Equation (3.4) for energy head loss (he).
4. With values from steps 2 and 3, solve Equation (3.3) for water surface elevation WS2.
5. The computed value of WS2 is compared with the assumed value in step 1, and steps 1 through 5 are repeated until the values agree with the predefined tolerance (.003 m).

3.3.2 HEC-Geo RAS extension

HEC-GeoRAS is an ArcView GIS extension, cooperatively developed by the HEC and the Environmental System Research Institute, Inc. (ESRI), specifically designed to process geospatial data for use with HEC-RAS (HEC, 2010).

HEC-GeoRAS prepares geospatial data for use with the Hydrologic Engineering Center River Analysis System (HEC-RAS) and presents HEC-RAS results in a geospatial context. The extension allows users with limited GIS experience to create an HEC-RAS import file containing river geometry and model parameters from an existing digital elevation model (DEM) and shape files that define the stream network, cross-section locations and other hydraulic characteristics of the river system. Prior to performing hydraulic computation in HEC-RAS, the geometric data must be imported and completed and flow data must be entered. After a hydraulic analysis is completed in

HEC-RAS, HEC-GeoRAS can develop and display themes of inundated area, water depth and water velocities from the results of that analysis.

One of the limitations of HEG-GeoRAS is that the Modelers are forced to use the stream geometry data extraction preprocessing in HEC-GeoRAS to ultimately use the post-processing visualization tools. Even when surveyed data is accurate and geo-referenced by the stream network, the geometry extraction from the terrain model is still required.

HEC-GeoRAS requires a DEM in the form of a TIN to construct the geometry of an HEC-RAS river model. The DEM must be a continuous surface that includes the bottom of the river channel and the floodplain to be modeled (HEC, 2002). The following themes can be used to develop HEC-RAS geometry using HEC-GeoRAS:

- A digital terrain model (TIN, required)
- A stream network (line shape file, required)
- Cross-section cut lines (line shape file, required)
- Bank lines (line shape file, optional)
- Flow paths (line shape file, optional)

HEC-GeoRAS performs no hydraulic calculations. The results from HEC-RAS are visualized through the following sequence of steps:

- GeoRAS creates a shape file of cross-section cut lines with water surface elevations for each exported profile as attributes.
- GeoRAS generates a TIN of water surface elevations for each profile, using the cross-section cut lines as break lines. The TIN is clipped by the bounding polygons for the profiles.
- GeoRAS converts the ground-surface TIN and the water-surface TIN to grids with the same cell size and origin and subtracts the ground TIN from the water TIN to get a depth grid.

- GeoRAS creates a shape file of inundated areas as the areas of the grids with positive depths.

3.3.3 Integration of GIS in HEC-RAS model

In recent years, efforts have been made to integrate hydraulic models and Geographic Information Systems (GIS) to facilitate the manipulation of the model output. The increasing availability of spatial data (terrain and rainfall), GIS software to manage spatial data, faster processors, and the availability of interfaces to connect simulation models with GIS, have increased use of GIS in watershed modeling (*Whiteaker et al, 2006)*.

HEC-GeoRAS is the geospatial tool used in this study, which serves as the interface between GIS and the simulation model HEC-RAS. HEC-GeoRAS allows engineers to concentrate on hydraulic model development and analysis rather than GIS mechanics. The user environment provides engineers an opportunity to view real-world systems of interest, which in turn assists them to rectify errors and make informed decisions in the model development *(Ackerman et al, 1999).Tate et al, (2002)* applied HEC-GeoRAS successfully to create a terrain model for floodplain mapping. A widely used approach is watershed modeling that divides the drainage basin into discrete units possessing similar rainfall-runoff and physical characteristics. This approach reduces model complexity and spatially distributed data requirements in basin-scale models *(Beighley et al, 2005).*

The two major digital formats in which surfaces are Digital Elevation Models (DEMs) and Triangular Irregular Networks (TINs). DEMs are commonly referred to as gridded data and contain terrain information in its cells. The resolution of the DEM refers to the cell size and each cell is assigned one elevation value. TINs generally are considered more precise than DEMs; however processing data in DEM format is faster than TIN format.

The TIN-based model has a vector-based data structure, but it can be converted into grid cells. In the TIN model, each point has defined x, y, and z coordinates. The coordinate z represents the height. These points represent the terrain surface in triangles connecting points and break lines (Lo and Yeung, 2005). Points can be used to represent valleys, high points, and any abrupt changes. Break lines can represent line features such as banks, roads, drainage divides etc. TINs generally are considered more precise than DEMs; however processing data in DEM format is faster than TIN format. Commonly TIN is developed through a photogrammetric method.

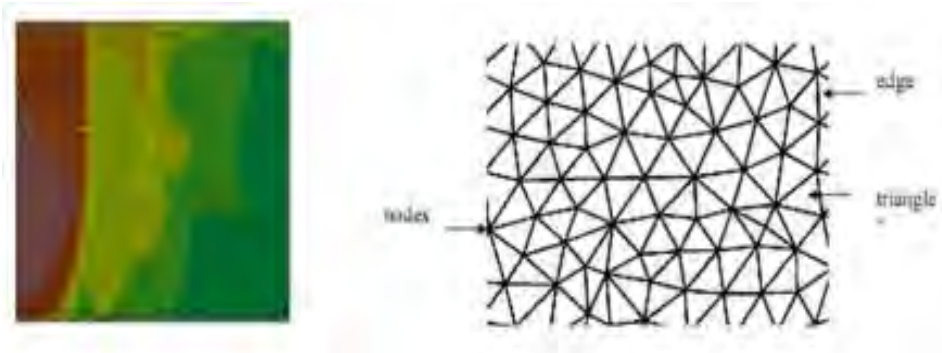

Figure 6 Triangular Mesh and TIN component

TIN data model is used to several advantages:

a. Linear geographic features such as streams and ridges are more accurately represented in TIN.
b. Fewer points are needed to represent the topography less hard disk space is needed.
c. Points can be concentrated in important areas where the topography is more variable or where more details are required.
d. Survey data and known elevation can be easily incorporated into a TIN.
e. Some functions cannot be performed with DEM data, but are easily done with TIN. Example of this includes flood plain delineation and computing time curve for hydrographs.

However, the limitation of this method is its inability to provide terrain information below the water surface of the hydrographic feature. This may be a problem for large rivers where significant low flow volumes exists *(Long, 1999).*

3.4 Concepts and practice of flood mitigation alternatives.

The hazard of flooding is receiving increasing attention in both the public and professional arenas. Flooding is a disastrous natural phenomenon, producing many socioeconomic and environmental consequences within the affected floodplain. Flood risks are rising in many parts of the world owing to continually increasing populations and rapidly escalating land developments *(Walker and Maidment, 2006).*Studies have shown that more than one third of the world's land area is flood prone, and about196million people in more than 90 countries are exposed to catastrophic flooding *(United Nation Development Programme (UNDP), 2004).*As a result, the frequency, extent and subsequent hazards associated with flooding are global concerns. Flood hazard mitigation plans could be implemented as either structural or non-structural measures, depending on the particular case *(Correia etal., 1999).*These measures involve managing the effects of flooding and preventing the negative consequences. Structural measures, including levees, high flow diversion, channel modification and dams, could be implemented to mitigate flood risk by reducing the volume of run-off, water level or extent of the area of flooding. However, non-structural methods, such as flood insurance, land use regulation and flood fore casting, serve as preventive measures for reducing flood hazards.

Structural measures are favoured in situations in which portions of adjacent residential areas are situated about the maximum flood level, or when it is important to protect land adjoining a river from inundations due to an existing flood risk. By implementing structural measures, flood hazards for these kinds of areas are reduced. However, structural flood control methods like channelization have some predictable negative effects. Studies have shown that sediment loads may increase below channelization works and other river training measures *(Hill,1976).*Moreover, changes may occur in discharge characteristics below channelization works; in particular, flood levels may be increased downstream*(Brookes,1987).*

Interaction between river flow and stream channel is dynamic, constantly responding to natural and human induced river and floodplain changes. Finding sustainable mitigation measures for flood hazards is made difficult by the complex and dynamic interrelationships of environmental, technical, economic and managerial factors in river systems. Therefore, accurate simulation of water surface profiles and reliable delineation of flood extents and depths within the flood plain *(Pappenberger et al, 2005*), as well as an understanding of the complex interaction between flood mitigation measures and river systems, combined with socio-economic impacts *(Yi et al.,2010)*,are necessary for flood mitigation planning and floodplain management.

Although some studies have been conducted to model flood hazards and floodplain mapping *(kefyalew etal.,2005;yidnekachew A.,2008)*,less effort has been done on the topic of modeling best river engineering and training measures.

Bana e, Costaet al. (2004) have used a multi criteria process to evaluate flood control options for the catchment of the Livramento Creek in the peninsula of Setúbal, in Portugal. They have demonstrated a quantitative evaluation model based on qualitative value judgments formulated by a group of experts to integrate socio-economic criteria into technical and environmental objectives.

Williams (2006) carried out a study on Santee River to evaluate the impact of the Santee River re diversion. The flooding pattern within the Santee flood plain has been altered by re diversion and operation of the hydroelectric plant on the re diversion canal.

Hydrologic Engineering Center River Analysis System (HEC-RAS) and the HEC-GeoRAS extension for ArcView were used to examine the flooding regime prior to and subsequent to re diversion operations. However, performance analysis of any flood mitigation measure should be considered before its implementation using simulation models and decision analysis methods. Moreover, economic analysis along with river hydraulic simulation play important roles in formulating measures for flood hazard mitigation (Simonovic and Carson, 2003).

Multicriteria decision analysis (MCDA) methods (Hwang and Lin, 1987; Ghanbarpour and Hipel, 2011) could be employed to integrate technical, environmental and socioeconomic objectives to reach an optimal decision.

In this research, the 20-km reach from logiya station to Dubti station of Awash River was selected. The main objective of this study is to compare different river training measures, such as diversion dike, dam, channel modifications and levee construction, to find the optimal approach form meeting technical and environmental criteria among the various alternatives. The core concern is to minimise the effects of the floods on the residential areas and environment (i.e. canal, agricultural districts etc.), through a technically sound measure.

An MCDA method, technique for order of preference by similarity to ideal solution (TOPSIS), is employed to find the best flood control measure, one which protects residential areas from flood hazards while minimizing the risk of flooding on surrounding effects. This decision analysis method is used to integrate hydraulic analysis with the results of the river model, HEC-RAS, combined with geographic information system (GIS) analysis.

3.5 Review of previous studies

There are a number of studies and project works that evaluate the water resources of Awash River Basin. Since most of the studies are done for academic purpose, they are targeting specific issues and particular model application. Thus they are referred for their respective area of address. But there are three latest efforts that tried to do the River Basin level water resource modeling, namely Awash River basin Simulation, Tendaho Dam & Irrigation Project (WWDSE, 2005), Awash River Basin Flood Control and Catchment Management study (HALCROW, 2007) and Coping with Water Scarcity – The Role of Agriculture Developing a Water Audit for Awash Basin, Ethiopia (MoWE and FAO, 2012). They are reviewed and their brief results are documented here as the background information of the current work.

Awash River basin Simulation, Tendaho Dam & Irrigation Project (WWDSE, 2005)

This study was aimed to study the impact of the Upper and Middle-Valleys irrigation developments and the decreasing Koka storage capacity on the inflow of the Awash River into the Tendaho dam reservoir. The second was to study the degree of expansion of irrigations in the Upper Valley using the existing Koka dam.

The study was used public domain HEC-5 Simulation of Flood Control and Conservation Systems software to simulate the different river basin model. The software version 8.0 developed by Hydrological Engineering Center of the US Army of Crops of Engineering and released on October 1998 is utilized to develop the monthly times step surface water balance of the Awash river basin. When it used this software, it took the previous work as background and 41 years of flow series (1963-2003). Four scenarios are also developed to see the monthly flow balance in the basin. The four scenarios defined were:

- Scenario I. The present (2005) withdrawal rate in the basin.
- Scenario II. The present withdrawal rate plus Tendaho project (48000/ 60000 ha)
- Scenario III. Scenario II + Wonji expansion by 5703 ha and Metehara expansion by 3200 ha and Kessem dam added with 20000 ha but MV1 Awaramelka irrigation not considered
- Scenario IV. Scenario III plus Wolenchit additional 9000 ha sugar cane expansion

Year 2008 was taken as starting year matching to the expected commissioning of the Kessem and Tendaho dams. In estimating irrigation frequency of success, the reservoir area capacity curves for the Koka, Kessem and Tendaho dams are varied due to increasing sedimentation through time.

Finally the study was concluded that under the existing storage condition in the Upper Valley, the addition of the Wolenchi 9000 ha irrigation was not attainable. Expansion of the Wonji irrigation scheme from 8203 ha to 13906 ha including Bofa 3462 and Dodota 2241ha was sustainable up to year 2018, without affecting the downstream irrigations including the expansion of Metehara with 3200 ha. And it was stated that Kessem dam and irrigation project has no significant influence on the Tendaho reservoir yield as simulated in the river basin model.

Awash River Basin Flood Control and Catchment Management study (Halcrow, 2007)

The second study reviewed in the course of the current river basin water balance modeling is known as Awash River basin Flood Control and catchment management study (HALCROW, 2007). Actually the main focused of the study was to develop mitigating strategies and catchment development plan for the flood risk of the basin. In the meantime the study developed basin water balance modeling using WEAP software. The water balance modeling was aimed

- To set up a water resources balance model to assess current resource availability conditions, but suitable for modification to represent potential future development conditions;
- To set up of a resource allocation model which could form the basis for optimising resource usage;
- To prepare a flood hydrology database for the Awash River providing flood flow frequency versus magnitude data at key node locations throughout the basin;
- To identify the need to improve facilities for the monitoring of water resources and flood flows within the catchment.

According to this study the Awash River Basin was divided into 7 physiographic units. They were named as the Upland basin, the upper valleys, the middle valleys, the lower valleys, the western highlands, the lower plains, and the eastern catchment. But they had no any hydrological boundary. To mitigate it, the study further divided the basin into 75 sub catchments. The study also stetted out three main scenarios as follows:

- 2007 scenario. This was assumed to be the pre-Kesem and pre-Tendaho Dam situations
- 2008 scenario. In this scenario, the 2007 scenario was assumed plus Kesem Dam being in operation together with the associated irrigation schemes.
- 2009 scenario. This assumes the 2008 scenario plus Tendaho Dam and its associated irrigation schemes

In line with the above scenarios the study also stated the following critical assumptions: Operation and management of Koka Reservoir is primarily to ensure that irrigation demands downstream met to the fullest extent possible and generation of electricity is to be considered as secondary to irrigation needs, good water management practices are assumed to be carried out at all irrigation schemes.

The water resources of the Awash River being able to meet the demand were examined by running the model over 43 years of observed data. These 43 years are assumed to include a representative sample of wet, dry and average years. And the results compared based on the failure frequency within 43 years. And the percentage and the month of highly frequent failure were identified for each scenario.

Coping with Water Scarcity: The Role of Agriculture Developing a Water Audit for Awash Basin, Ethiopia (MoWE and FAO, 2012)

The study has been the latest and the detail water resource potential and demand assessments with the new concepts of the water auditing. Water audit as a concept of water resource planning and management system need the knowledge of the current status of water resources supply and demand with time. It involves several issues like

- addressing the occurrence of surface and ground water, in space and time, and, in particular, assessing levels of sustainable use and the frequency of extreme events such as droughts and floods;
- providing a tentative assessment of the demand trends for different uses;
- identifying the main driving forces influencing demand and use (e.g. government policy, societal behaviour);
- assessing the functionality and effectiveness of institutions charged with developing and managing water resources; and,
- Understanding factors that affect access and entitlements to water for both domestic and productive uses.

These the study also used the WEAP software for the purpose of the Basin water audit modelling since the WEAP has a capability to analysis the supply (Hydrological system) and demand (water allocation system) as single modelling system. Again this study was the source of basic data and concept for the current study. The detail irrigated area survey data for the current study is accessed from this study.

4. MATERIALS AND METHODS

In this thesis, two keys software were used: ArcGIS for GIS applications and HEC-RAS4 (USACE, 2010) for the hydraulic part. The link between the two software was being performed by the HEC-GeoRAS. This later automates the stages of export - import between the two existing software (USACE, 2010f). The HEC-GeoRAS interface was created for ArcGIS in the form of a dropdown menu in the toolbar of ArcGIS. This application allows the design of the components of the topographical model and automated export to HEC-RAS and then imports the results of hydraulic simulation to ArcGIS. In addition to the above the following supportive software's are used: Global mapper 15, Google earth pro and USGS statically analysis excel spread sheet.

There are two main ways for specifying the inputs to build a model in HEC-RAS. One is to do a physical survey of the study site, and collect data manually regarding the river geometry. The other way is to use geospatial datasets like Digital Elevation Models and develop the geometric data in GIS.

The hydraulic geometry of the river was essential for accurate model simulations dependent on the DEM resolution. To generate the RAS import and export layers the procedure implemented justified in the three phases as shown in framework below such as Pre-processing of geometric data for HEC-RAS, using HEC-GeoRAS; Hydraulic analysis in HEC-RAS; and post processing of HEC-RAS and flood plain mapping using HEC –GeoRAS.

4.1 Research framework

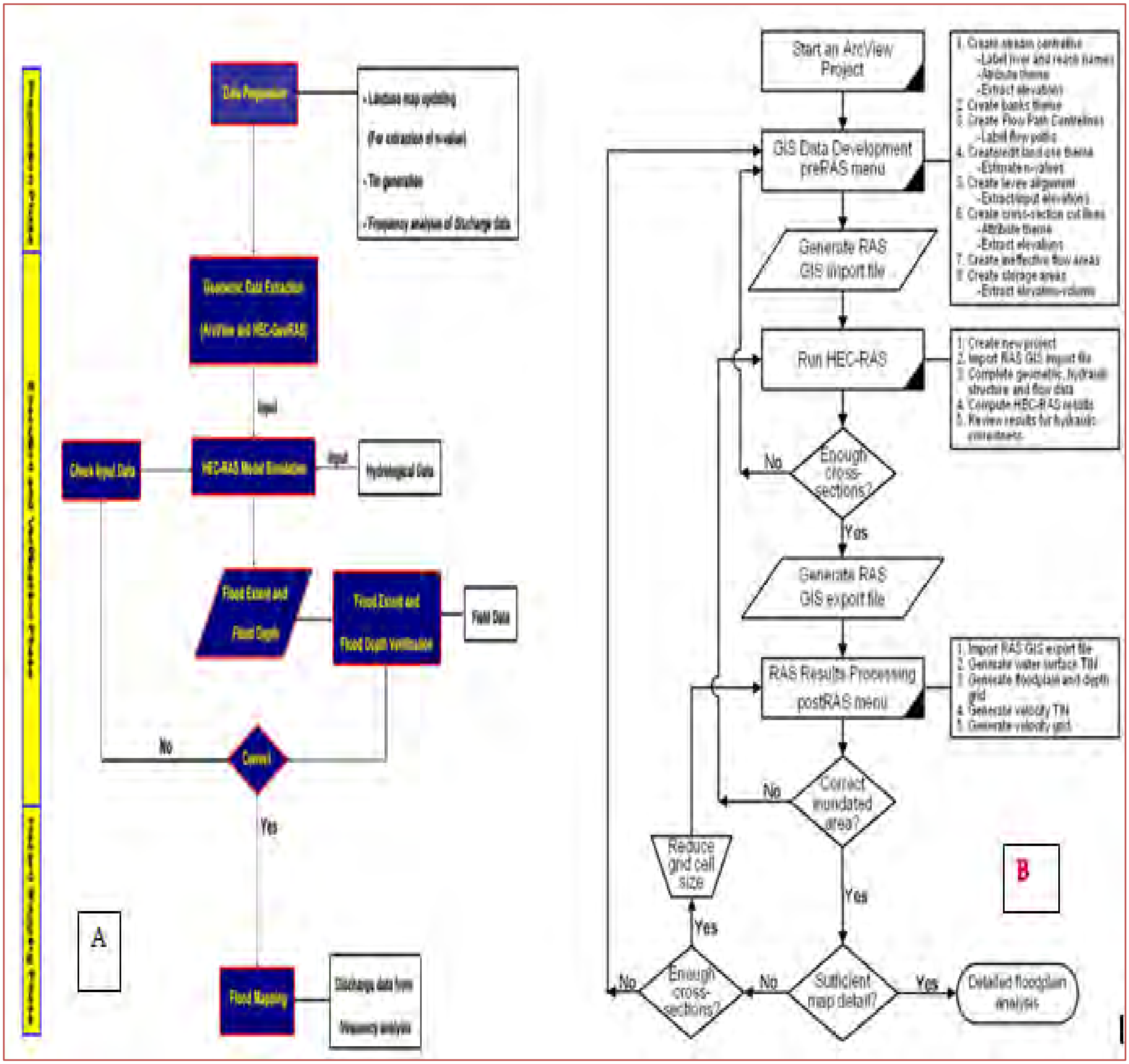

Figure 7 (A) one-dimensional floodplain analysis using HEC-RAS, GIS, and HEC-GeoRAS. (B) Process flow diagram for using HEC Geo RAS (after USACE, 2005)

4.2 Data collection and preparation

In this research two type of information, namely hydrological data and GIS-based spatial layers were collected.

Hydrological data: Daily river flow and rainfall data were obtained from MoWIE, WWDSE and NMA. Unlike the daily precipitation, the daily discharge has full data composition for the considered stations to represent the study area. Annual maximum river flow data recorded at logiya, Tendaho and dubti gauge Station were used for this study. The length of the selected data was 36 years from 1972-2007, based on consistency and availability during that time period. These data were used for flood frequency analysis using descriptive statistical analysis to determine flow magnitude for different return period as input data for the model. The quality of the data was analyzed to identify any kind of abnormalities such as outler test of stream flow.

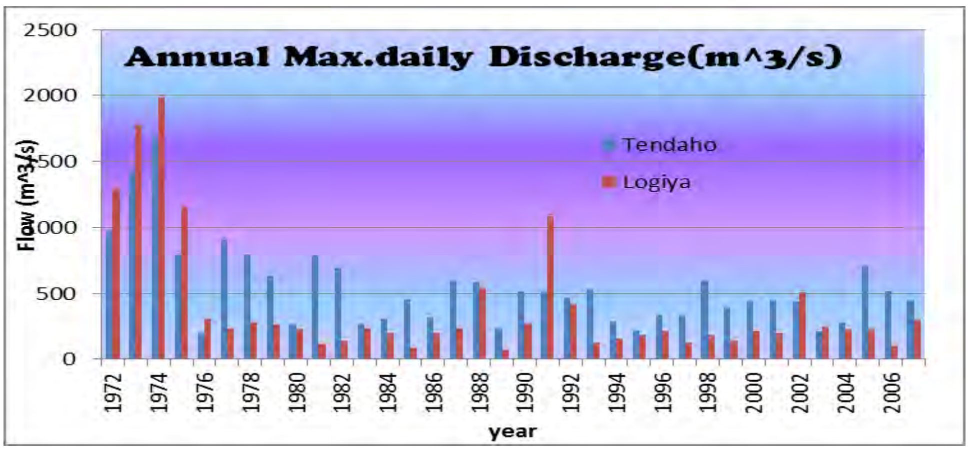

Figure 8 Yearly daily Maximum discharge of Logiya and Dubti station

GIS data: The shape file of the study area, land use map has been obtained in MoWIE. Moreover, DEM of 30m by 30m resolution were applied for TIN generation using 3D analysis capability of Arc GIs. The generated TIN used for extraction of cross sectional data of the river schematics and flood plain.

4.2.1 Frequency analysis

Flood frequency analyses were used to predict design floods for selected reach along a river. The technique involves using observed annual peak flow discharge data to calculate statistical information such as mean values, standard deviations, skewness, and recurrence intervals. Flood frequency distributions could take on many forms according to the equations used to carry out the statistical analyses. At present, there is no universally accepted frequency distribution model for frequency analysis of extreme floods, rather a whole group of models such as Gumbel (EV-1), Normal, Log normal, Pearson Type III, Log Pearson Type III etc have been suggested in the literature such as (Topaloglu,2002) and (Ojha et al,2008) for the prediction of extreme flood events.

If a prediction is to be based on a set of hydrologic data, then the distribution that best fits the set of data may be expected to give the best estimates usually an extrapolation of the probability of an event occurring. The probability distributions selected for this study are Normal, Log normal, Extreme value type 1(EV-I) and Log Pearson Type III distributions. Their essential properties are given below.

Table 6: Probability distribution parameters in relation to sample moments (Ojha et al, 2008; Chow et al,1988)

Distribution	Probability distribution function	Range	Equation of parameters in terms of sample moments
Log normal	$f(x) = \frac{1\sqrt{\pi}}{x\sigma} \exp[\frac{-1}{2}\frac{(lnx - \mu)^2}{\sigma}]$	x>0	$\mu_y = \overline{y}$ $\sigma_y = s_y$
Gumbel(EV-I)	$f(x) = \frac{1}{\beta}\exp[-\frac{x-u}{\beta} - \exp\left(-\frac{x-u}{\beta}\right)]$	(-$\infty < x < \infty$	$u = \overline{x} - 0.5772\beta$ $\beta = \frac{S_y\sqrt{6}}{\pi}$
Log Pearson Type III	$f(x) = \frac{(lnx-u)^{\gamma-1}\exp[\frac{(lnx-u)}{\beta}]}{\lvert\beta\rvert(\gamma)}$	$lnx \geq x$	$u = \overline{y} - S_\gamma\sqrt{\gamma}$ $\beta = \frac{\sqrt{\gamma}}{s_y}\gamma = (\frac{2}{c_s})^2$

The observed data at each gauging station were ranked and evaluated with three probability distribution functions namely: Normal, Lognormal, EV-I and Log Pearson Type III using Weibull plotting positions for all distribution in order to determine the best fit function.

Normal and Lognormal Probability distribution

The Normal distribution is the most familiar probability distribution; Its PDF is given by:

$$f(z) = \frac{1}{\sigma\sqrt{2\pi}} exp(\frac{-(x-\overline{x})^2}{2\sigma}) \qquad (1)$$

It is defined by two distribution parameters; the mean ($\overline{x}$), and standard deviation (z) evaluated by-:

$$\boldsymbol{\overline{x} = \frac{1}{N}\sum_{i=1}^{n} x_i} \qquad (2)$$

Where xi is the magnitude of the i^{th} event and N is the total number of events. The standard deviation (σ) which is a measure of the dispersion or spread of data set is given by:

$$\boldsymbol{\sigma = [\frac{\sum_{i=1}^{n}(x-\overline{x})}{N-1}]^{(\frac{1}{2})}} \qquad (3)$$

A particular event x can be related to the probability of exceedence P by the following equation:

$$\boldsymbol{x = \overline{x} + k\sigma} \qquad (4)$$

where k is the frequency factor.

The Log normal distribution assumes that the logarithms of the discharge are themselves normally distributed. The equation describing normal distribution is modified for use in the case of log normal distribution if the following substitution is made.

$$\boldsymbol{y_i = log x_i} \qquad (5)$$

With x replaced by y, the mean of the logarithms and standard deviation becomes was computed as the normal distribution.

The probability of exceedence is related to the occurrence of particular values if log values are used is:

$$\boldsymbol{log x = \overline{log x} + k} \qquad (6)$$

Log Pearson Type III distribution

The problem with most hydrologic data is that an equal spread does not occur above and below the mean. The lower side is limited to the range from mean to zero while there is theoretically no limitation on the upper range there by contributing to what is called a skewed distribution. The coefficient of skew is defined mathematically by:

$$\boldsymbol{\sigma}_{\boldsymbol{logx}} = \frac{\sum_{i=1}^{N}(x-\bar{x})^3}{(N-1)(N-2)\sigma^2{}_{logx}} \quad (7)$$

To determine the skew when log values are used, equation (9) becomes:

$$\boldsymbol{c}_{\boldsymbol{slogx}} = \frac{\sum_{i=1}^{n}(logx-\overline{logx})}{(n-1)(n-1)\sigma^2{}_{logx}} \quad (10)$$

It is to take account of the skew that may exist in data that the log Pearson type III distribution was developed to improve the fit. The distribution uses three parameters namely: mean standard deviation and skew coefficient which are obtained using the above equations. The frequency factor depends on the return period and coefficient of skewness (Cs =0), the frequency factor (k_T) is equal to the standard normal variable (z) and for **cs#0**, was approximated using the equation given as:

$$\mathbf{k_T = z + (z^2 + 1)k + \frac{1}{3}(z^2 + 1)k^2 - (z^2 - 1)k^3 + zk^4 + \frac{1}{3}k^5} \quad (11)$$

where $\mathbf{k} = \frac{\mathbf{C}}{\mathbf{6}}$ and The value of z for a given return period was calculated using same procedure as was with log normal case, and $\mathbf{y_T = \bar{y} + k_T s_y}$ and$\mathbf{X_T = 10^{y_T}}$

Gumbel (EV-I) distribution

The Extreme value Type 1 (Gumbel) distribution, one of the most commonly used distribution in flood frequency analysis. The distribution is based on theory of extremes and it is considered appropriate for this analysis as annual series data used for this study is composed of peak values (extreme values) for each year.

The fitting of EV-I distribution to the observed data was carried out using the following steps as given in.

i. The variates of the annual flood series were ranking descending order of magnitude

ii. Plotting position i.e. the probability of non exceedence corresponding to T-yr recurrence interval was assigned to each variate using weibull plotting position formula.

iii. The reduced variate for the distribution corresponding to the different plotting position was computed using. $y_T = -ln(ln(\frac{T}{T-1})$.

iv. The T-yr recurrence interval flood was estimated using the E V-I distribution given by .$\bar{X}_T = u + \beta y_T$

v. For the EV-I fit, the frequency factor K_T is evaluated as: $K_T = \frac{-\sqrt{6}}{\pi}(0.5772 + ln(ln(\frac{T}{T-1}))$

4.2.2 Distribution fitting for the data

In order to determine the best probability distribution function that describes the set of observed data at each gauging site, the four selected probability distribution models applied to the set of observed data at a particular station were subjected to statistical tests (measures of error in prediction). The tests chosen were Root mean square error (RMSE), Relative root mean square error (RRMSE), Maximum absolute error (MAE) and correlation coefficient (CC). The best fit was determined by means of a criterion depending on the differences between the observed and the theoretical density functions or distributions. In order to judge the overall goodness of fit of each distribution a ranking scheme was utilized by comparing the four categories of test criteria based on the relative magnitude of the statistical test results. A distribution with the lowest RMSE, lowest RRMSE, lowest MAE or highest CC was selected in Appendix A-3.

The tests chosen are:-

- Root mean square error (RMSE),

The root mean square error is the sum of the squares of the squares of the differences between the observed and predicted values and is given.

$$\text{RMSE} = \left(\frac{\sum(x_i - y_i)^2}{n-m}\right)^{1/2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 1$$

where x_i , i=1,--,---,n are the observed values and y_i , i=1,--,---,n are the values computed from the assumed probability distributions, the number of parameters estimated for the distribution is denoted by m.

- Relative root mean square error (RRMSE),

$$\text{RRMSE} = \left(\frac{\sum\left(\frac{x_i - y_l}{x_i}\right)^2}{n-m}\right)^{1/2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2$$

RRMSE calculates each error in proportion to the size of observation thereby reducing the influence of outliers. Which are common features of hydrological data [Tao et al, 2002] and thereby providing a better picture of the overall fit of distribution .

- Maximum absolute error (MAE)

This represents the largest absolute difference between the observed and computed or predicted values. It is given by:

$$\mathbf{MAE} = \mathbf{max}(\boldsymbol{X_i} - \boldsymbol{Y_i}) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 4$$

- Correlation coefficient (CC).

The correlation coefficient (CC) is defined mathematically as:

$$\mathrm{CC} = \frac{\sum[(X_i - \bar{X})(y_i - \bar{y})]}{\sum(x_i - \bar{x})^2 \sum(y_i - \bar{y})^2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3$$

Where $\bar{X}$and$\bar{y}$ represents the average value of the observations and predicted quantiles respective.

4.3 Pre-processing of geometric data

To create the geometric data file for HEC-RAS, HEC-GeoRAS requires a digital terrain model of the river system in the Triangulated Irregular Network (TIN) format. The DEM file was converted to contour lines at an interval of 2m in digital format shape file to the TIN format using the 3D Analyst toolbox in ArcGIS.

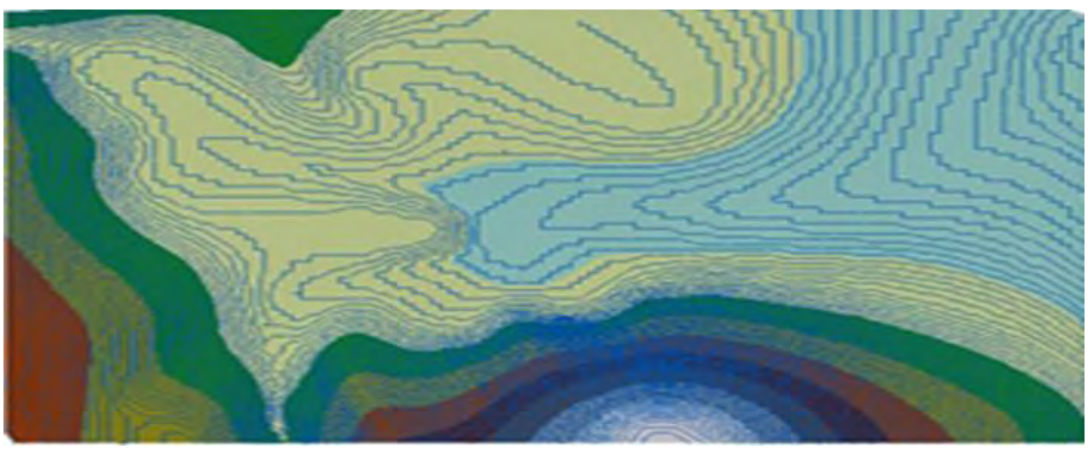

Figure 9 TIN of study Area

After conversion, the procedure for extracting the geometric data in HEC-GeoRAS was as follows: the procedure is useful if someone would want to conduct a similar study,

River Digitization

- Create empty GIS layers for the Stream Centerline, Bank Lines, Flow Path Centerlines and XS Cut Lines
- In the editor mode, use the Sketch tool to digitize the centerline of the river from upstream to the downstream direction. Even if, Aerial photographs and

topographical datasets were helpful guides to pinpoint the path of a stream, but usually its path was noticeable on the DEM due to its lower elevation.

- Complete the centerline attributes commands in the RAS Geometry Tab of HEC-GeoRAS to populate the missing fields of the new layer. A river may also have more than one reach.

- Similarly in the editor mode, digitize the River Banks, Flow Path Centerlines and XS Cut Lines using the Sketch tool.

- Complete their attributes commands in the RAS Geometry Tab of HEC-GeoRAS to populate the missing fields of the new layers. Figure13 shows a sample of the different layers in HEC-GeoRAS.

- Bank lines were used to distinguish the main channel from the overbank floodplain areas. The flow path lines were used to determine the downstream reach lengths between cross-sections in the main channel and over bank areas. Cross-section cut lines were 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 centerline and flow path lines were 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 n.

GeoRAS Data Export to RAS

- Assign Manning's n value to cross-sections. Land use data file along with Manning's value for those land use types were needed to assign Manning's value to cross-sections. This is not a compulsory step and can also be performed in HEC-RAS, but it is better to do it GeoRAS as it automatically picks the value from the data.

- Creating the GIS import file for HEC-RAS so that it could read the GIS data and create the river geometry. The Layer Setup was first checked to see if the correct layers were selected, then the Export GIS Data function was used to create an export file for use in HEC-RAS.

- This file was then imported in HEC-RAS for building a floodplain model.

- In HEC-RAS, the river geometry was represented by a sequence of cross-sections called river stations. The numbering of the river stations increases from downstream to the upstream side. The distance between adjacent cross-sections is termed the reach length. Each cross section is defined by a series of lateral and elevation coordinates, which are typically obtained from geospatial datasets. The numbering of the lateral coordinates begins at the left end of the cross-section, (looking downstream) and increases until reaching the right end. (Tate, 1999)

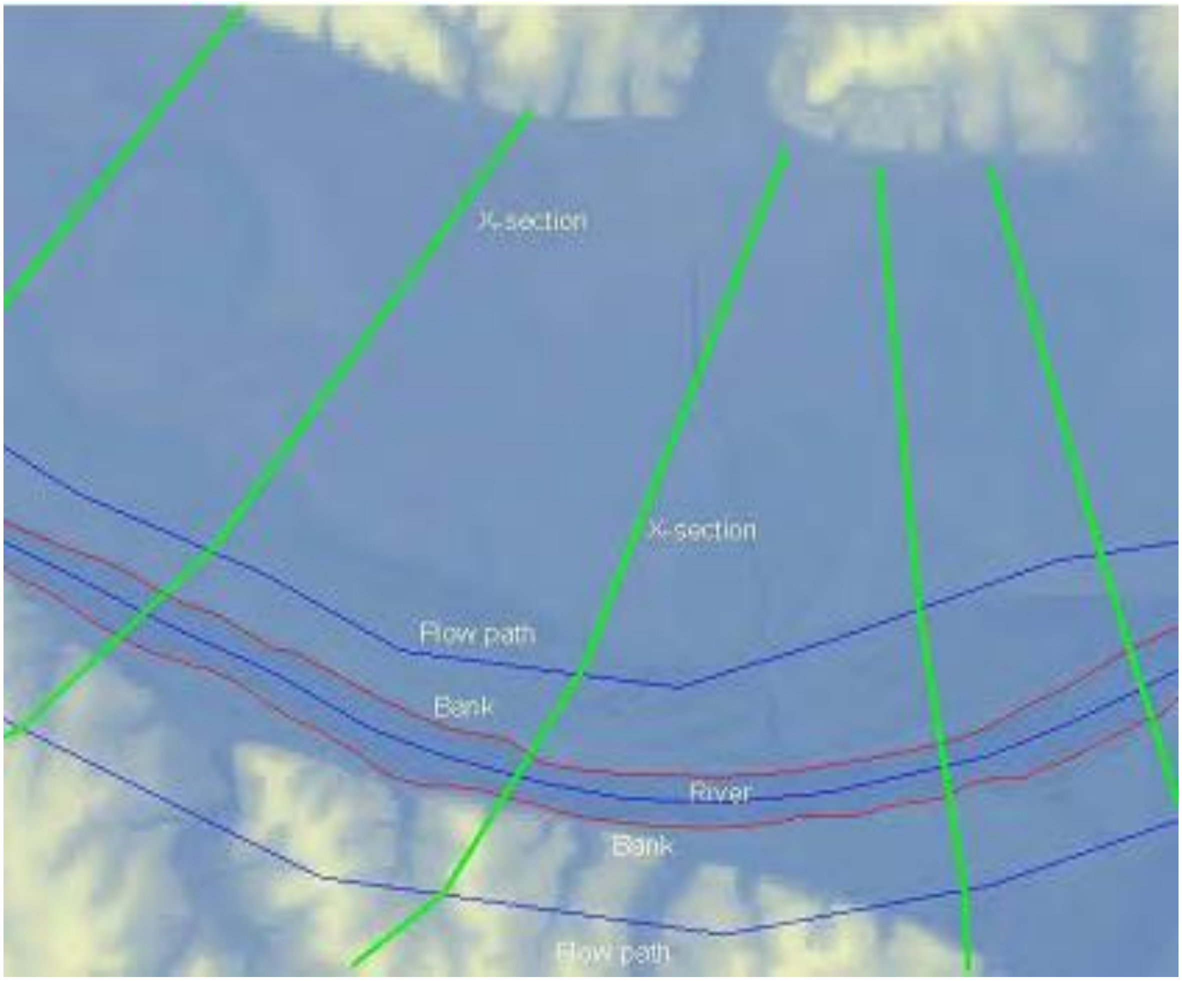

Figure 10 A snapshots of River, Banks, Flow Path Centerlines and Cross-section layers for study Area in HEC-GeoRAS.

4.4 Hydraulic modeling with HEC-RAS and GIS.

The objective of the hydraulic modeling process was to convert the flow values calculated into water surface elevations along the stream reach. Before importing the GIS data into HEC-RAS a new project was started and saved under a user-given name as shown below.

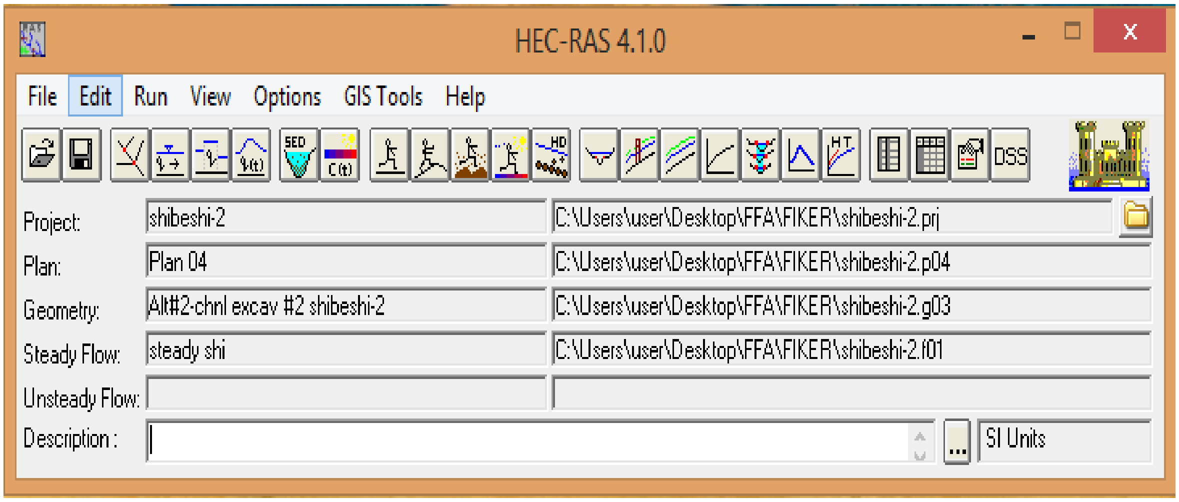

Figure 11 The main HEC-RAS window with the title and file name

Once the Geometric Data editor was opened the *Import Geometry Data/GIS Format* option was chosen from the File menu of the editor window. From the Import Options window, *SI (metric) units, "River and Reach Stream Lines" and "Cross Sections and IB Nodes"* tab were selected and save in geometric data window.

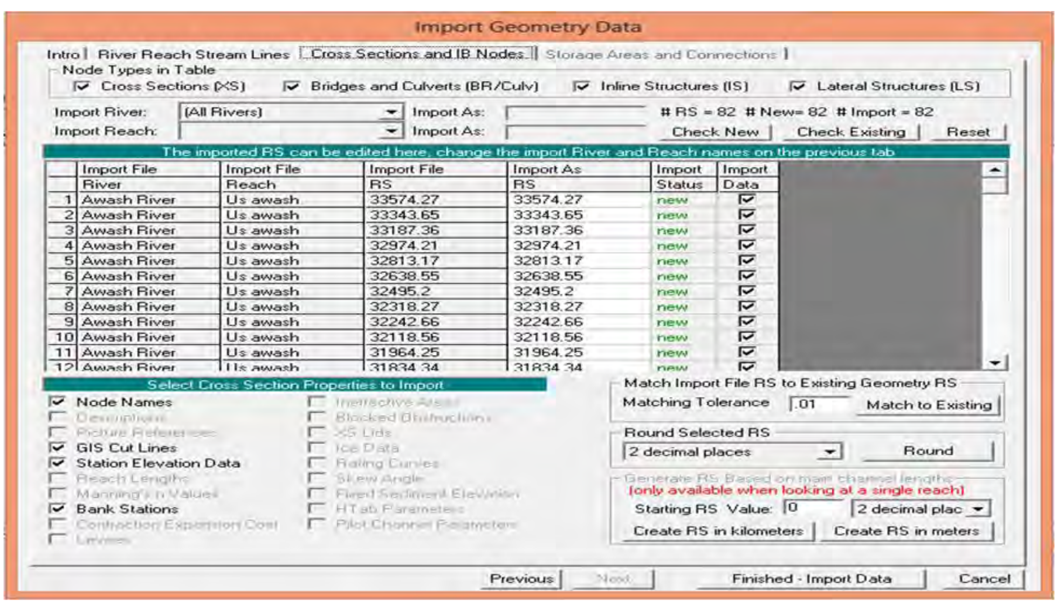

Figure 12 Import options window

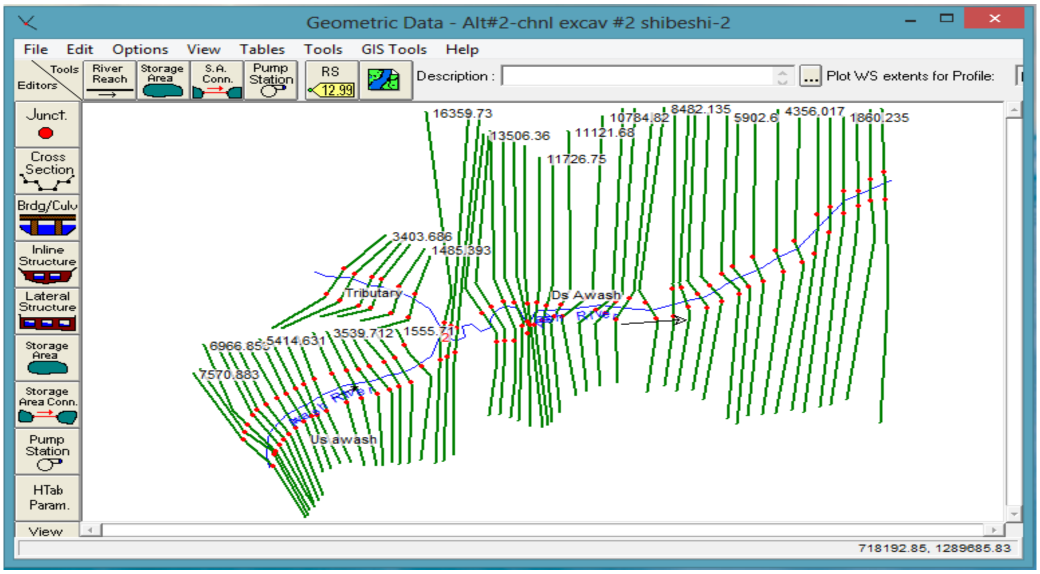

Figure 13 Geometric data window

After importing the Geometric data, firstly the quality of data checking was done. Banks station data might need to be modified using the graphical cross section editor. This was done by selecting **Tools| Graphical XS Edit** menu item on the Geometric Data editor. The current version of HEC-RAS also limits the number of cross-sectional station-elevation data to less than 500 points. Since the Cross-section cut lines created in ArcGIS run across the whole extent of the floodplain, most of the cross-sections have more than the maximum permitted number of station points. The **Cross Section Points Filter** function on the **Tools** menu of the **Geometric Data editor** was used to filter the number of station points in each cross-section to the required maximum. Then Interpolation of cross-section within the reach at 100m interval was made to get accurate result during water profile computation. The location of the banks along the cross-sections extracted from the TIN was not very accurate. Therefore, every cross-section was checked manually; the banks were relocated when necessary. On the selected reach Awash River, there were four structures along the river. The structures were manually added to the model through the HEC-RAS interface.

4.4.1 Flow data and Boundary conditions

HEC-RAS can solve for both steady flow and unsteady flow. Steady flow solutions are selected in analysis for flood plain modelling and compare different flood control alternatives. In order to perform a steady water surface calculation, steady flow data such as flow regime, boundary conditions and peak discharge were entered.

Boundary conditions were important inputs in hydraulic model for establish a starting water surface elevation and influence of external system to model domain through a connecting node. There are four different boundary condition types i.e., Known water surface, Critical depth, Normal depth and rating curve. The type selected depends largely upon the available data. Since no observed flow data (Known Water Surface) is available, it was very important to choose appropriate steady flow boundary conditions. Usually, if there is no observed data, the normal depth is used (*Merwade.v et al.., 2006*). Normal depth requires the energy slope or when this is not available the slope of the channel bottom. The slope of the channel bottom was obtained from DEM generated profile was 0.012, 0.021, & 0.0085 for logiya, upstream, &downstream awash percent. Since the flow was subcritical, the boundary condition was necessary at the downstream end of the river reach to proceed the computation upstream.

Discharge information was other boundary condition that required at each cross section in order to compute the water surface profile. Discharge data were entered from upstream to downstream for each reach. Discharges were estimated within a catchment based on flood frequency analyses for gauged station. The flow data for each reach were entered and water level simulations for 50 and 100 year return periods of floods computed.

Table 7 Flow and Boundary conditions

Logiya Station	**Flood Magnitude**		**Tendaho Station**	**Flood Magnitude**			
Events	50 year	100 year	Event	50 year		100 year	
Flow	2140	3200	Flow(m^3/s)	Q peak at station	Routed Flood	Qpeak at station	Routed flood
	Upstream station	Downstream station		1733	1216	1950	1564
				Upstream station		Downstream station	
Boundary condition	6608.541	966.6004	**Boundary condition**	6608.541		1555.71	

Once all data entered, it could be possible to calculate steady water surface profiles. This was done using HEC-RAS main window and by selecting *Steady Flow Analysis* from the *Run* menu. Then the compute button was pressed to start the simulation.

4.5 Calibration of HEC-RAS

Model calibration is the process of finding the optimum values of model parameters which will give the least difference between the observed and the simulated values. The main parameter that fine-tuned was the roughness coefficient of the surface because of the availability of the standards.

Calibration was commonly done manually by optimizing the parameter values, starting from the given initial values, until the simulated variable fits with the observations. Calibration was being assessed through visual interpretation and some other statistical means.

The performance of the model had been evaluated using efficiency criteria, determination coefficient (r^2) and Nash- Sutcliff Efficiency (Ens), to measure how well trend in the measure data were reproduced by simulated results over a specified period. Determination coefficient for n time step was calculated as

$$r^2 = \frac{[\sum_{i=1}^{n}(Qsi - \overline{Qs})(Qoi - \overline{Qo})]^2}{\sum_{i=1}^{n}(Qsi - \overline{Qs})^2 \sum_{i=1}^{n}(Qoi - \overline{Qo})^2} \dots\dots\dots\dots\dots\dots\dots\dots\dots..1$$

Where Qsi is simulated value, Qoi is measured value, Qom is the average observed value and Qsm is the average simulated value.

Nash- Sutcliff Efficiency (Ens) for n time step was calculated as:

$$E_{ns} = 1 - \frac{\sum_{i=1}^{n}(Qoi - Qsi)^2}{\sum_{i=1}^{n}(Qoi - \overline{Qo})^2} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots.2$$

Where Qoi is observed, Qsi is the simulated and Qom is the observed average values

The peak flow data was used for calibrating the model by varying roughness coefficient. Manning's n values were generalized to a single value for the channel and the over land area, as HEC-RAS has a limitation of 20 points where n values can be specified per cross section and in almost all the instances in this study the cross-sections had more than 20 points. Also, using a Left Bank, Channel, Right Bank simplification makes the calibration process easier and quicker to conduct.

4.6 Flood mitigation alternatives

The effects of various river training measures can be analysed based on the performance of the hydraulic parameters of flood risk and environmental impacts. For this study, four flood mitigation alternatives, including levee construction and different scale of channel modification along with dredging river channel of debris and sediment, are proposed in order to reduce flood hazards in a residential area and its surrounding flood plain, as described later.

Channel modification for flood control was based upon the idea that widening and deepening of the channel by sediment removal will provide a substantially larger channel capacity to allow a greater volume of water to flow downstream without causing an impact on the adjacent flood plain(DPHW,2003). In proposed alternatives, no channel straightening was considered in order to prevent any negative erosion effects downstream (Wyzga, 1993), as there is a natural bend in the river reach that was thought to be more effective in decreasing the energy and velocity of flow, and thereby prevent any further erosion. Removing the natural bend of a river usually leads to stream bank erosion and often requires costly protection measures. No water detention basins or flood diversion structures were considered in this study area because of limited land availability.

Alternative 1: A levee is an earthen embankment supported by Gabion-type revetments (riprap) and was constructed on the left side of the river bank adjacent to the residential area, with the capability of withstanding a 100-year flood. Levees were chosen for this reach because they seem to be an effective solution for reducing flood damage and were practical because of the availability of materials. Dimensions of the levee considered had an average elevation of 2.5 m and length of 7 km. Riprap revetment with an additional 1-m foundation form a steep slope in the riverside to prevent any limitation in the cross sectional area of the river channel.

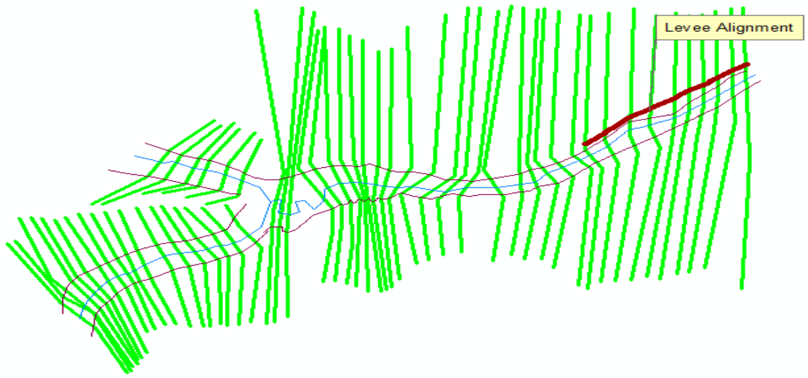

Figure 14 Levee alignment option for selected reach

Alternative 2: In this alternative, river bed elevation should be set to 362 m with a slope of 0.0065. River width was widened an average of 80m in the main channel using channel excavation and dredging.

Alternative 3: Channel modification was conducted in two stages to construct a compound river channel. First, river width was widened an average of 120m in the main channel without considerable deepening but more dredging. Second, the main channel was deepened by an elevation of 361 m with a slope of 0.0065 and 80 m width using machinery based channel excavation.

Alternative 4: Channel modification was conducted as the combination of alternatives 1 and 3. In this case, a levee as an earthen embankment supported by rock-type revetments (riprap) was constructed.

The selected flood mitigation alternatives could be positioned at appropriate locations along the selected reach of Awash River for HEC-RAS model runs to extract model results for comparison. Flood level, flow velocity and stream power at various locations along the river and flood plain were predicted based on proper flood plain inundation analysis in the 50- and 100-year flood associated with the four proposed alternatives, including current condition (baseline). Implementation of river training practices would likely alter flow characteristics along the river system including flood level, flow velocity and stream power. Therefore, changes in flow characteristics were adopted as decision criteria along the selected reach for selecting a reliable and environmentally sound flood control alternative.

Two specific control cross sections on the downstream and outside of the river bend were considered for comparison purposes. The downstream of the residential area (dubti) after the river bend was considered to control impacts of flood control measures on different hydraulics parameters in the downstream of the river system. The outside of the river bend adjacent to the residential area was also considered as the second control point, which is important in terms of higher risk of erosion and flood hazard.

4.7 Post-processing of hydraulic results and floodplain mapping

The result from HEC RAS analysis was further post-processing using HEC GEORAS to prepare flood inundation maps for different flow conditions. Detailed explanation of the data post-processing follows.

4.7.1 Data import from HEC-RAS

Formatting. The process starts with opening a desired Arc Map for post-processing. Since the HEC-GeoRAS cannot read the proprietary spatial data format (RAS Export. sdf) file created in the HEC-RAS, it was necessary to convert it into the XML file format, supported by HEC-GeoRAS. This was achieved by selecting the "***Import RAS SDF File***" option from the HEC-Geo RAS Toolbar.

Layer setup. Establishing the Layer Setup was a necessary step for processing the HEC-RAS results. In the Layer Setup window, the type of analysis and the input and output data were identified. *Figure 18, Shows* a typical Layer Setup window from the data post-processing. For the post-processing analyses, the rasterization cell size was set to 2 map units. Basically, a smaller number of map units results in a better representation of the resulting floodplain boundary during the floodplain delineation. Since at some places the floodplain lines were too coarse, an attempt is made to rasterization cell size to 2 Map Unit.

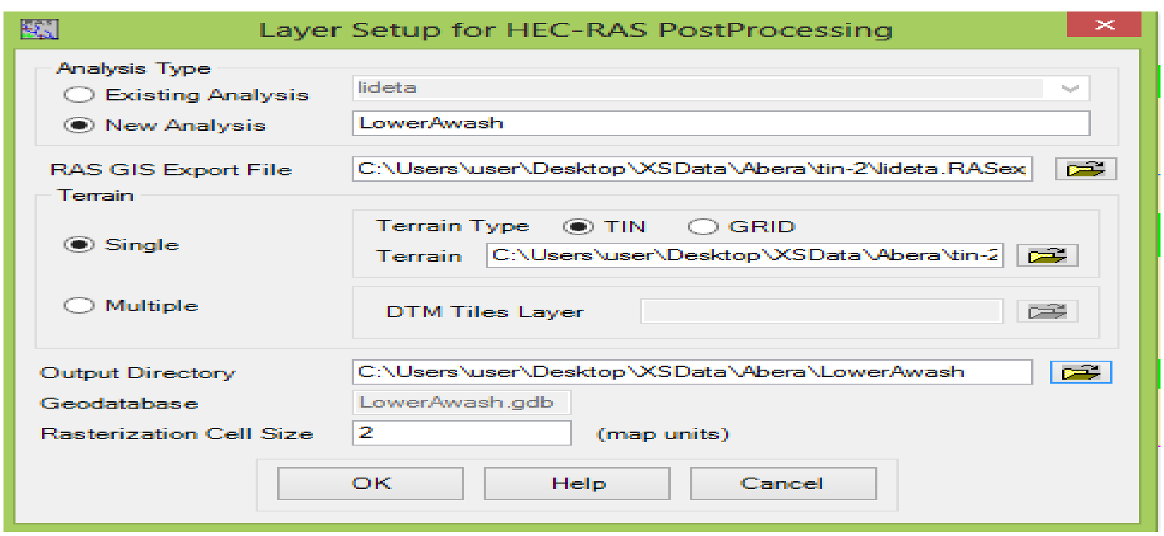

Figure 15 Layer setup window

Reading RAS GIS export file. After input data was entered in the layer setup, the HEC-RAS results had to be imported into the GIS in order to continue with the post-processing. The following computational steps were selected from the HEC-GeoRAS toolbar: ***"RAS Mapping" →"Read RAS GIS Export File".*** This selection introduces a new data frame with the following feature classes: River2D, XS Cut Lines, and Bounding Polygon.

4.7.2 Floodplain mapping

Floodplain mapping was performed using the water surface elevations on the XS cut lines, within the limits of the bounding polygon. Floodplain mapping was completed in two steps, which were explained in following paragraphs.

Water surface TIN. The first step was to create a water surface TIN from the cross section water surface elevations. The following computational steps were selected from the HEC-GeoRAS toolbar: **RAS Mapping |Inundation mapping |Water Surface Generation.** All four water surface profiles were selected from the window, as shown in below.

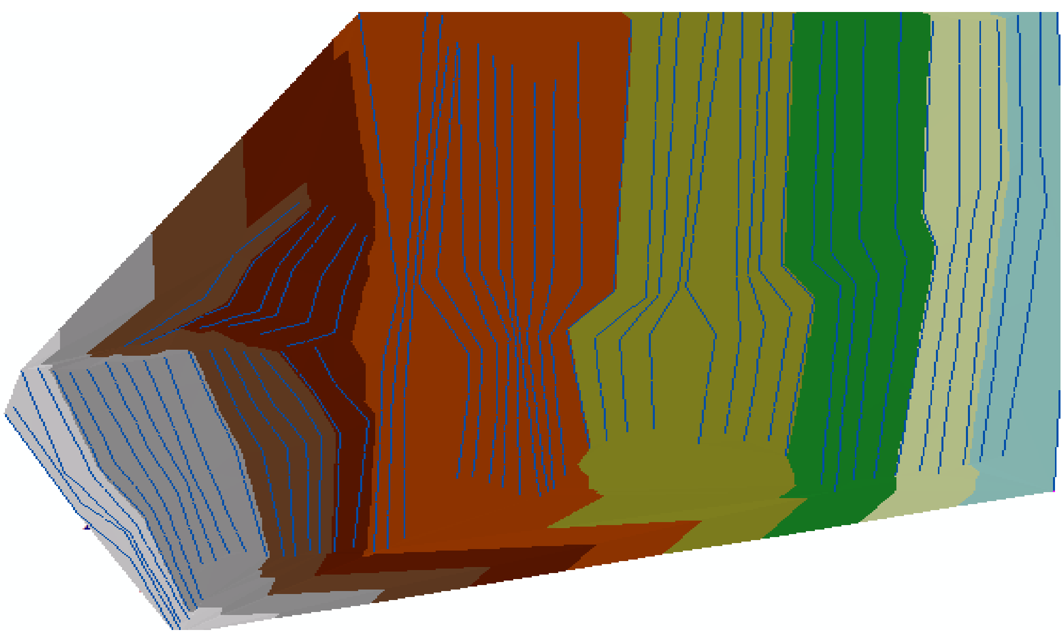

Figure 16 Water surface TIN (Awash River and logiya tributaries)

Floodplain delineation. The following computational procedures were used from the HEC-GeoRAS toolbar: **RAS Mapping |Inundation Mapping |Floodplain Delineation |GRID Intersection.** Again, all two water surface profiles are selected from the window.

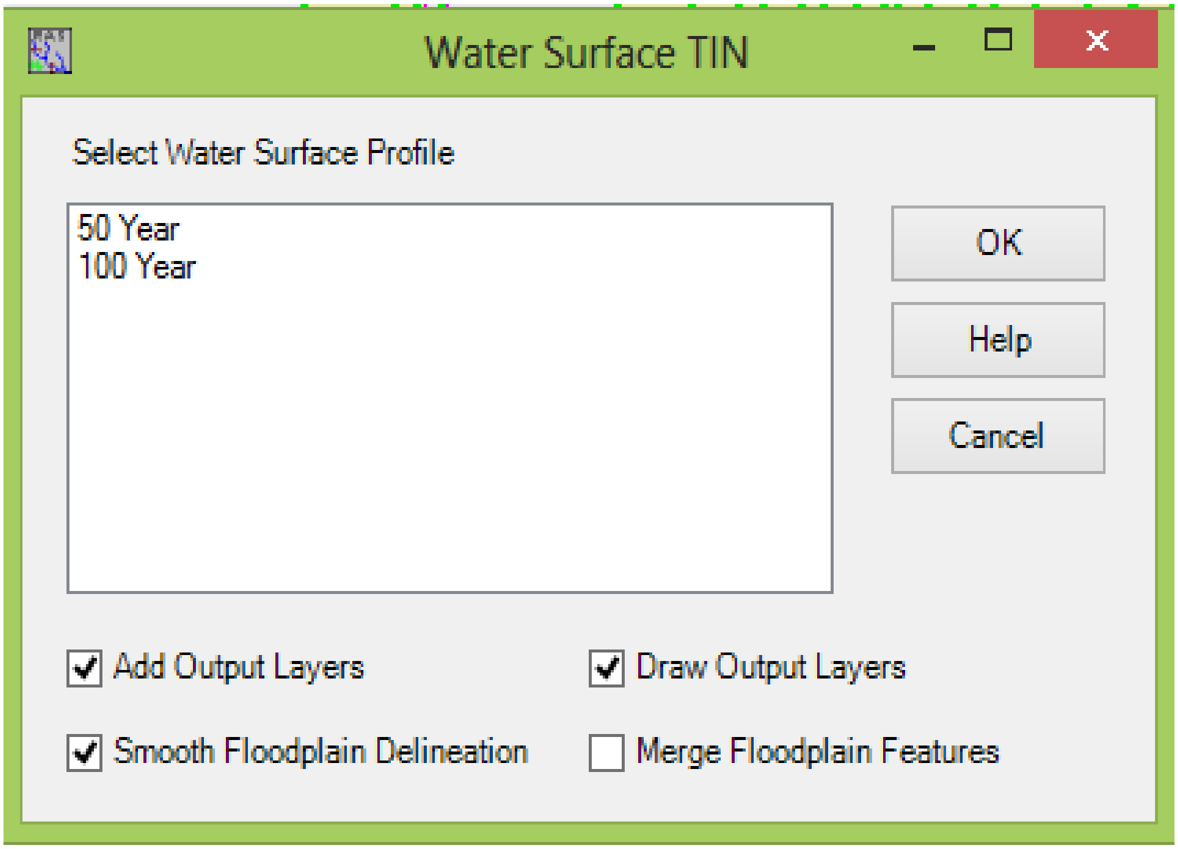

Figure 17 Selected water surface profile for the water surface TIN generation

The water surface TIN was converted into a grid based on the rasterization cell size. Then, it was compared with the TIN terrain model, which was also in grid format, allowing the elevation difference to be calculated within the bounding polygon. The areas with positive results (where water surface is higher than the terrain elevation) were included in the floodplain area (inundation depth grid), and the areas with negative results are considered as dry.

4.8 Decision analysis

An MCDA method, TOPSIS *(Hwang and Lin, 1987; Ghanbarpour and Hipel, 2011)*, was used to find an optimal flood mitigation alternative considering different criteria. In this method, the optimal alternatives could be found based on the shortest distance from the ideal solution and the farthest distance from the anti-ideal solution. As *Chen and Hwang (1992)* shown, a decision matrix is a two-dimensional (m*n) matrix for which A_1, $A_{2,..}$., Am are m possible alternative among which decision makers have to choose, and C_1, C_2, ... , Cn are n decision criteria which flood mitigation alternatives are compared based. X_{mn} indicates the performance rating of alternative A_m with respect to criterion C_n. The relative importance of each criterion is usually given by a set of weights that are normalised to sum to one. In the case of n criteria, a weight set is:

$$w = (w_1, w_2, \dots., w_n) \qquad when \sum_{j}^{n} w_j = 1 \tag{3}$$

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

$$r_{ij} = \frac{X_{ij}}{\sqrt{\sum_{i=1}^{m} x_{ij}^2}}, i = 1,2, \dots, m; j = 1,2, \dots, n \tag{4}$$

The weighted normalised value vij was calculated as:

$$V_{ij} = w_j x_{ij}, i = 1,2, \dots, m; j = 1,2, \dots.. n \tag{5}$$

Table 8 Weight of all decision-making criteria based on different scenario

Decision Criteria		Decision Making scenarios for flood risk mitigation		
Criteria	Sub-criteria	Risk-Oriented	Environmental-oriented	Balance
Flood risk	Inundation area(ha)	0.333	0.167	0.25
	Flood level(m)	0.333	0.167	0.25
Environmental Impact	Flow Velocity(m/s)	0.167	0.333	0.25
	StreamPower(N.m/s)	0.167	0.333	0.25

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

$$A^* = \{(\max_i V_{ij} | j \in J), (\min_i V_{ij} | j \in J') \mid i = 1,2, \dots, m\}$$

$$= \{V_1{}^*, V_2{}^*, \dots, V_J{}^*, \dots., V_n{}^*\} \dots \dots (6)$$

$$A^- = \{(\max_i V_{ij} | j \in J), (\min_i V_{ij} | j \in J') \; | i = 1,2, \dots, m\}$$

$$= \{V_1{}^-, V_2{}^-, \dots, V_J{}^-, \dots., V_n{}^-\} \dots \dots (7)$$

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

$$S_I{}^* = \sqrt{\sum_{j=1}^{n} (v_{ij} - v^*{}_j)^2} \quad , i = 1,2, \dots, m \dots \dots (8)$$

$$S_I{}^- = \sqrt{\sum_{j=1}^{n} (v_{ij} - v^-{}_j)^2} \quad , i = 1,2, \dots, m \dots \dots (9)$$

At the final stage, the relative closeness of A_i with respect to A^* defined as

$$C_i{}^* = \frac{S_i{}^-}{(S^-{}_i + S^*{}_i)} \; , 0 < C_i{}^* < 1, i = 1,2, \dots., m \dots \dots (10)$$

Two main decision criteria and their sub criteria including flood risk (inundation area and flood level) and environmental impact (flow velocity and stream power) were considered for comparison of flood mitigation measures (Table 8). For all decision criteria and sub criteria, a set of weights should be assigned to reflect the importance of that particular decision variable, regarding Eqn (3) and Eqn.(5). In this research, a scenario based decision analysis was conducted to show the importance of each criterion and to integrate them into the decision process. The sensitivity of final decision to the criterion weightings could be assessed by varying the weight of each criterion. Scenario analysis could indicate how the variation of the weights of the criteria would affect the final decision. This was essential to guarantee that the proposed alternative was optimal in terms of different considerations and point of views.

For this purpose, three decision-making scenarios were considered for comparison shown in Table 8. The first scenario was flood risk-oriented decision making in which higher weight were given to flood risk criteria (flood inundation area and flood level). Therefore, the safety of area adjoining to the river was the main concern in this scenario.

The second scenario was defined as environmental-oriented in which higher weight was given to environmental criteria (flow velocity and stream power). Therefore, higher priorities were given to environmental issues such as erosion and bank cutting in the river bend and downstream as a result of flood control measures implementation. The last scenario was defined as a balance decision-making option, which was considered as the balance between all decision criteria with equal weight and priorities. Table 8 shows details of criteria and sub criteria weighting for different scenarios considered in this research. It should be noted that the sum of all criteria weights equals one. Therefore, the original weight was divided by two if there are two sub criteria. For example, in risk-oriented scenario, flood level criteria (0.333) have two sub criteria at two control points on the downstream and outside of the river bend. Therefore, the weight of 0.167 was considered for each of the two control points.

5. RESULT AND DISCUSSION

5.1 Flood frequency analysis

It was shown that in the appendix A-3, Normal, Lognormal, Gumbel (EV-I) and Log Pearson Type III were used to estimate peak flow. For selection of the best fit model four types of goodness of fit tests were used. The tests chosen are Root mean square error (RMSE), Relative root mean square error (RRMSE), Maximum absolute error (MAE) and correlation coefficient (CC).

The overall goodness of fit of each distribution was judged based on the relative magnitude of the statistical test results of the lowest RMSE, lowest RRMSE, lowest MAE or highest CC value.

Table 9 Results of the of goodness of fit tests applied to the distribution models

Station	Distribution Model	RMSE	RRMSE	CC	MAE
Logiya Station	Gumbel	1881	84	33	6816
	Log pearson -III	2.5	15.2	25.7	9.4
	Log Normal	63.4	55.7	28.3	44.2
	Normal	28654.8	376.9	31.13	25153.4
Tendaho Station	Gumbel	316.26	0.002	0.008	1572.3
	Log pearson -III	1.01	-0.04	42321.86	4.13
	Log Normal	9.41	7.74	35.95	6.47
	Normal	15790.74	57.7	33.7	8717.2

A distribution with the lowest RMSE, lowest RRMSE, lowest MAE or highest CC was given a score of 4, 3, 2, and 1 from the best score to worst score for each test. The overall score of each distribution was obtained by summing the individual point scores obtained from all the tests at each of the stations and the distribution with the highest total score at each station was chosen as the best fit distribution model for the station. The best fit model for the discharge data at each station selected based on highest total score obtained at the station as shown in Table 9 is presented in Table 10.

Table 10 : Distribution model scoring scheme based on goodness of fit test results

Station	Distribution Model	RMSE	RRMSE	CC	MAE	Total score
Logiya Station	Gumbel	2	2	4	3	**11**
	Log pearson -III	4	4	1	4	**13**
	Log Normal	3	3	2	2	**10**
	Normal	1	1	3	1	**6**
Tendaho Station	Gumbel	2	3	1	2	**8**
	Log pearson -III	4	4	4	4	**16**
	Log Normal	3	2	3	3	**11**
	Normal	1	1	2	1	**5**

Therefore, Log -Pearson III was the best fit to estimate the flood peak flow design for the selected River reach, as shown table 10 above.

Therefore, peak flow showing a magnitude of 50-and100-year peak flows were estimated using Log Pearson-III. The results of the estimated magnitude presented in table 11 below, which were used as a base to run the model and compare different flood control alternatives.

Table 11 Flow changes locations and discharges for various floods return intervals

Hydraulic Model station	Description	Flood Return Interval (Qxx) Discharge(m3/s)	
		50Year	100 Year
3403.541	Logiya river stream gauge	2140	3200
6608.541	Routed from Tendaho dam spill way at 400m crest level with clear width of 31.5 m.	1216	1564
16359.73	Confluence of logiya river and spilled flow from dam- routed and sum up at the confluence point	3356	4764

The above routed flood magnitudes of 1216 and 1564 m^3/s from the crest the dam spillway (400m) was using the Area elevation capacity curve data prepared by waterworks design and supervision enterprise using Hechms version 4.This routed flood was using 1733 & 1950m^3/s flow discharge for 50 & 100 year respectively.

5.2 HEC-RAS calibration and validation

The calibration of the model was conducted to find the optimal values of Manning's roughness by comparing simulated and observed flood water depths at selected point. The selected gauge discharge was calibrated by adjusting the Manning's n values. The least difference between simulated and observed flood level were used as calibration criterion for selected discharge. Then the calibrated parameters were validated for four different peak events over the last 36 years to compare water surface elevations. The results of the calibration and validation process were close in values from the Table 12 below.

Although the percent of difference ranging from *-2 % to 6.83 %* (Table 12), it was the most achievable calibration run because of the lack of rating curve in the gauge station. This large discrepancy was most likely due to the changes in topography and hydraulic characteristics of the river system, especially as a result of development and man-made activities.

Table 12 HEC-RAS Model Calibration and Validation

		Water Elevation			Manning's n	
Event2004-06-05	Flow(m^3/s)	Observed(m)	Simulated(m)	%Difference	Main channel	overbanks
Dubti-gauge Station	184.5	5.84	5.78	1.04	0.031	0.043
Validation						
Event 1994-06-21	450.85	6.84	6.51	5.07		
Event 2000-04-27	256	6.88	6.41	6.83		
Event 2002-06-20	465.85	5.25	5.30	-0.95		
Event 2003-05-07	267	7.48	7.33	-2		

Based on trial and error model runs, a Manning's roughness coefficient of 0.031 and 0.043 produced the best fit against the observed data for the river main channel and flood plain, respectively. The flood water depths from the optimal Manning's n are in a relative agreement with those simulated ones (with 1.04% percent of difference). The validation results are also satisfactory with the difference in water surface elevations being small. The overall validation process $R^2$and Nash-Sutcliffe model efficiency co-efficient was **0.73** and **0.59** respectively, so the model was performing reasonably well, and could be used to assess the changes in flooding.

5.3 Flood inundation analysis

The result of flood frequency analysis showing the magnitudes of 50-and 100-year peak flows were used as a base to run for different alternatives after the model calibration done at base line condition. These magnitudes were used as base to run the model for different flood mitigation alternatives. In fact all flood mitigation alternatives were compared based on similar model setting.

The flood mitigation alternatives were positioned at appropriate locations along the selected reach of lower Awash River in HEC-RAS model runs. Delineation of flood extents and depths within the flood plain of lower Awash River was conducted in different return periods based on the integration of hydraulic simulation results and GIS analysis using the HEC-GeoRAS. Flood inundation extents, including different model outputs in all cross sections, were extracted after running the model for each alternative. Figures 20 and 21 show subsequent maps of predicted 50- and 100-year flood inundation extents and depths for all proposed flood mitigation alternatives. The flood boundary was delineated in blue, with gradations of blue corresponding to different flood depths. The rainbow colour in the background represents the TIN of the study area where cross sections generated. As can be seen in figure 22, flood-affected areas for the 100-year flood event were of larger extent than the 50-year flood (figure 21), as it had affected some parts of Dubti residential and agricultural areas.

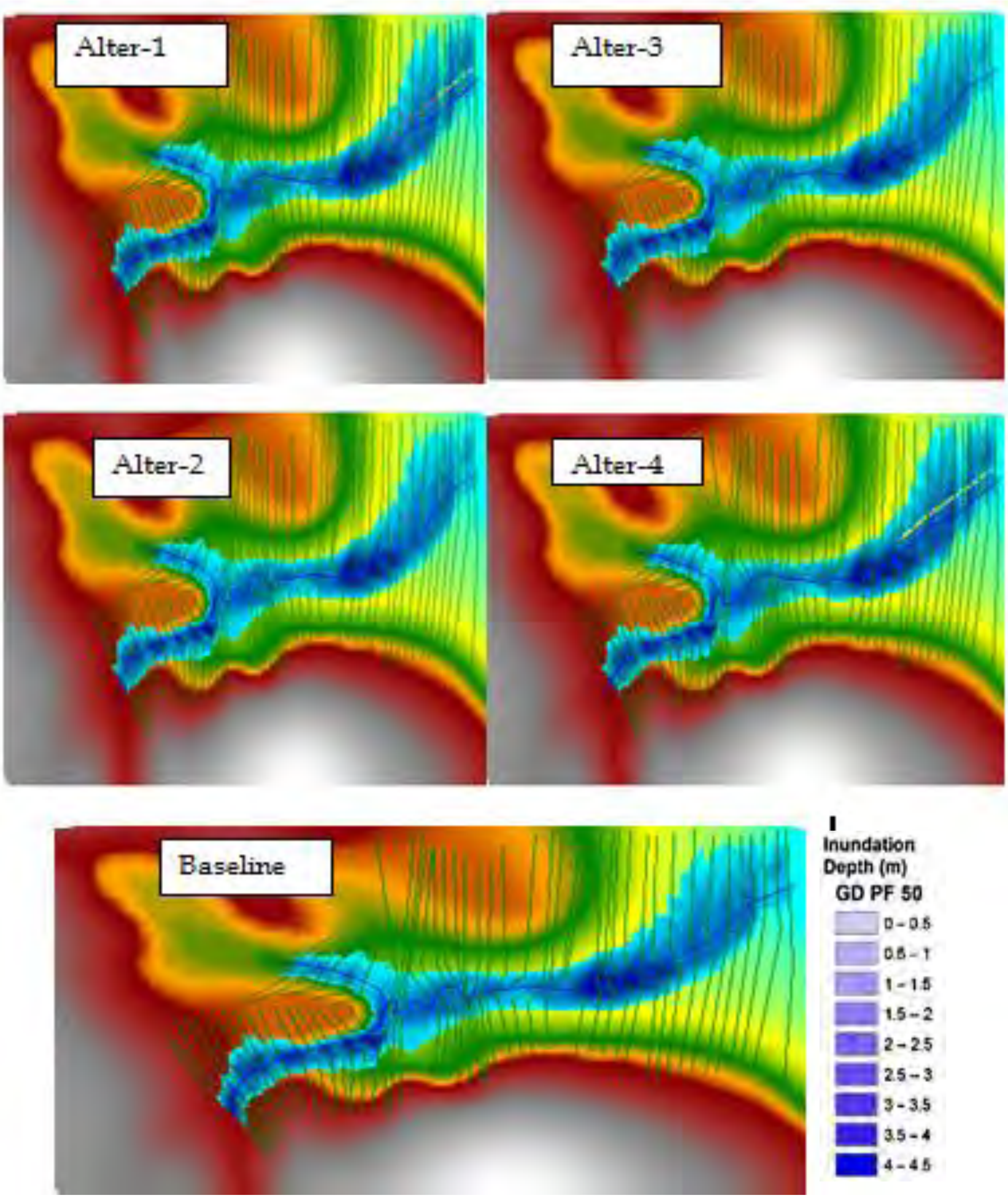

Figure 18 Flood inundation areas and depths for 50 –year flood mitigation alternatives

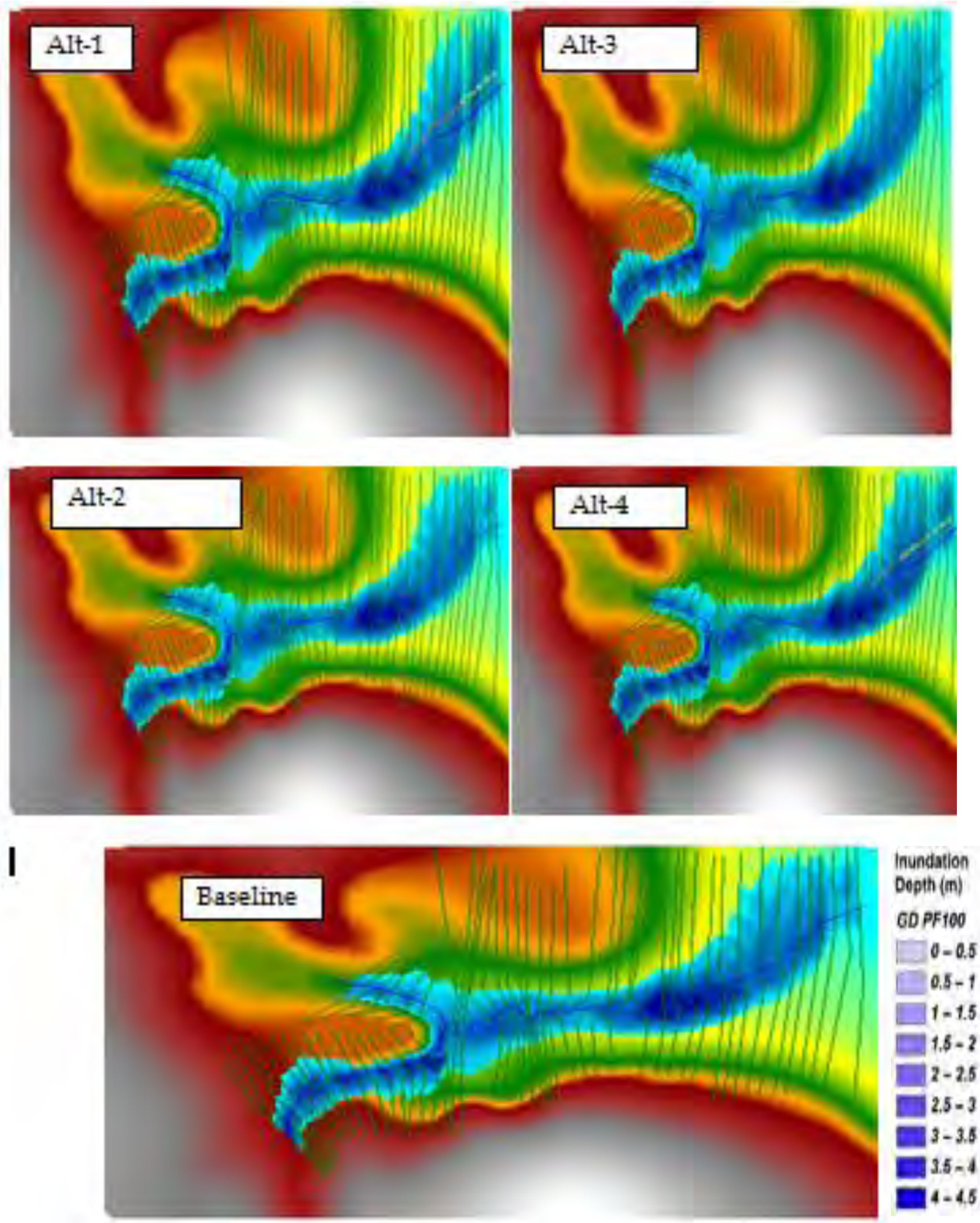

Figure 19 Flood inundation areas and depths for 100–year flood mitigation alternatives

5.4 Comparison of flood mitigation alternatives

All proposed flood mitigation alternatives had different flood extents. The evaluated results for all measures could be summarized as follows.

Flood inundation area, flood level, flow velocity and stream power at two control points along the Awash River system were extracted based on flood plain inundation analysis for the 50- and 100-year floods associated with the four proposed alternatives using the HEC-RAS simulation combined with GIS (Figures 21 and 22). Table 13 was determined based upon flood inundation layers in GIS depicted in Figures 21 and 22. Table13 shows a simple comparison of the inundation area (ha) and flood depth (m) for each alternative under the 50- and 100-year floods.

Without any flood mitigation measure (baseline), the total inundation areas were 39,960 and 44,550 ha, under the 50 and 100-year floods, respectively, including 15,660 and 18,090 ha with an inundation depth over 1.5. For the measures of **alternatives 3** and 1 the total inundation areas were 34,200 and 37,800 ha for a 50-year flood and 38,790 and 41,670 ha for a 100-year flood, respectively, which were less than those of the other alternatives including baseline. Alternative 3 had the smallest areas of inundation, showing significant diminution for areas of a depth over 1.5 m (Table 13). As a result, it could be found that alternatives 3 and 1 were superior to the other two measures in terms of flooded area protection.

Table 13 Comparisons of inundation Areas (ha) for proposed flood mitigation alternatives based on flood depth in 50-and100-year floods.

Return Period(year)	Flood Depth(m)	Flood mitigation alternatives				
		Alt#1	**Alt#2**	**Alt#3**	**Alt#4**	**Baseline**
50	D<0.5	8,550	8,600	7,560	6,120	9,720
	0.5<D<1.5	14,670	14,760	13,050	14,760	15,660
	1.5<D	14,580	14,400	13,590	14,580	15,480
	Total	**37,800**	**38,880**	**34,200**	**36,360**	**39,960**
100	D<0.5	9,990	10,440	8,640	5,040	10,440
	0.5<D<1.5	16,720	15,120	12,960	18,720	16,020
	1.5<D	13,860	18,090	17,190	17,290	18,090
	Total	**40,670**	**43,650**	**38,790**	**41,050**	**44,550**

Where: **Alt#1**.Levee construction **Alt#2**.Channel excavation and dredging

Alt#3.Compound channel (widening and Deeping)

Alt#4. Combination of alternative 1 and 2.

Table14 compares hydraulics parameters in baseline conditions with those reconstructed from flood mitigation alternatives under 50- and 100-year floods at two specific control sections. The effects on flood level reduction by the different alternatives were presented in Table 14. Alternatives 3 and 4 had better performance in terms of flood level reduction than the other alternatives on the outside and downstream of the river bend control points, respectively. However, baseline and levee construction had the highest flood levels in selected cross sections. There was no major improvement in flood level reduction using construction of a levee in comparison with the baseline. Although the high protection levee may not decrease river flood stage, it could decrease the risk of an overbank flooding disaster in residential areas.

Comparisons between all alternatives in terms of changes in flow velocity under 50- and 100-year floods were presented in Table 14 at two selected control points. There was a variation of flow velocity among the alternatives, depending on the local river geometry and hydraulic conditions of river flow. As shown in Table 14, alternative 1 causes a significant decrease in flow velocity on downstream cross section in comparison with the baseline. Generally, levee construction had the lowest negative effects on flow velocity on the downstream (Table 14) and was superior to other alternatives for erosion control and river bank protection. Alternative2 performs better than other options in terms of flow velocity on the outside of the river bend. However, alternatives 4 and 2 had the highest downstream flow velocity, which could accelerate downstream river bank erosion. Alternative 3 had highest flow velocity at the river bend control point, which could cause progressive river bank cut and erosion towards the land adjoining the river sides.

Table 14 shows a simple comparison of the stream power change on the downstream and the outside of the river bend cross sections for each alternative under 50- and 100-year floods. As shown in Table 14, there were critical and extreme stream power changes in alternatives 4 on the downstream. Generally, alternatives 2 and 1 show better results for stream power change on outside and downstream control points, respectively. Results show that changes in hydraulic parameters of stream flow occurred as a direct consequence of the channel modification practices.

Table 14 Comparisons of hydraulics parameters for proposed flood mitigation alternatives based on 50-and 100-year flood at some selected control cross sections.

Hydraulics Parameters	Return Periods(year)	Control Cross-sections	Flood mitigation alternatives				
			Alt#1	**Alt#2**	**Alt#3**	**Alt#4**	**Baseline**
Flood level(m)	50	1	363.57	363.3	362.57	362.73	363.51
		2	361.95	361.35	361.53	361.06	361.97
	100	1	364.59	364.45	363.84	363.92	364.51
		2	362.72	362.6	362.47	362.23	362.94
Flow velocity(m/s)	50	1	1.5	1.51	2.59	2.36	1.62
		2	3.41	5.28	3.91	5.72	3.67
	100	1	2.21	2.05	2.85	2.75	2.65
		2	4.2	4.95	4.83	5.87	4.16
Stream Power (N.m/s)	50	1	36.88	17.84	85.43	64.33	41.69
		2	249.54	707.78	281.3	1304.25	226.25
	100	1	85.83	25.88	66.98	60.65	82.44
		2	422.17	636.84	526.08	1163.27	367.47

*[1] on the outside of the river bend; [2] on the downstream of the river bend.

Trade-offs analysis between different decision variables to find the optimum flood control measure is a difficult task, as there is conflict between different variables in two control points. Figure 20 show some examples for comparison of flood control alternatives using trade-offs between pairs of decision variables. For example, alternative 1 was the most preferred option in terms of downstream flow velocity and stream power (Figure 20B).

However, alternative 2 was better than other alternatives in terms of outside river bend flow velocity (Figure 20A) and stream power (Figure 20D). As another example, alternative 3 had the highest flow velocity on the outside of the river bend. But it had the lowest flood level in the same control point, which was obviously in conflict (Figure 20A).

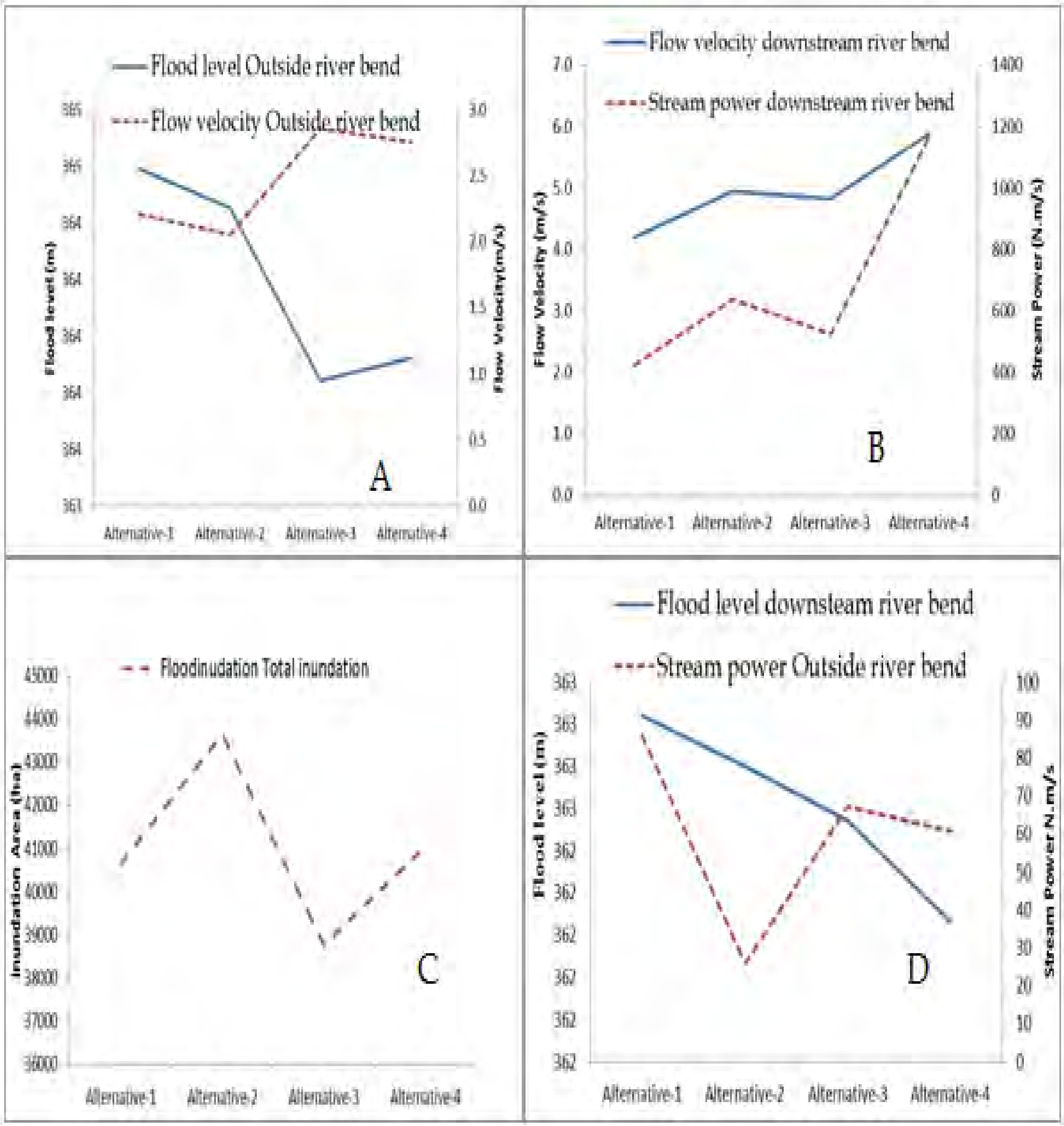

Figure 20 Comparison of flood mitigation alternatives using pair-wise trade-offs between decision criteria.

It could be concluded that an MCDA analysis was needed in this case, as there was conflicting criteria that make it difficult for an accurate trade-offs analysis. It should be noted that all variables used in Figure 20 were resulted from 100-year flood inundation analysis in previous sections. Application of TOPSIS as an MCDA method was discussed in the next section.

5.5 Scenario-based flood mitigation analysis

TOPSIS, an MCDA method, was used for prioritising the best flood mitigation alternatives. Results of a decision analysis based on three different scenarios for criteria weighting were shown in table 13. Only those criteria for 100-year flood in two selected control points were considered for decision analysis. A scenario analysis was conducted to show the importance of each criterion and how they are integrated in the decision process. In this study, I examined the effect of variations in criterion weightings of flood risk and environmental impact criteria for the final decision.

The first scenario was based on the fact that the most important criterion of flood risk including flood level and flood inundation area. Therefore, in this scenario, flood level and flood inundation area were assigned a weight twice as high as the other criteria in the decision matrix (Table 8).

Table 15 Priority rating and rank of proposed flood mitigation alternatives based on different decision-making scenarios

Flood Mitigation alternatives	Decision-making scenarios for flood risk mitigation		
	Risk-oriented	Environmental-oriented	Balance
Alt#1[Levee Construction]	0.702[1]*	0.339[2]	0.333[1]
Alt#2[Channel Modification]	0.184[4]	0.019[4]	0.034[4]
Alt#3[Two Stage Channel Modification]	0.678[2]	0.390[1]	0.304[2]
Alt#4[Combination of Alt#1and#3]	0.623[3]	0.315[3]	0.252[3]

*Numbers in bracket show the rank of each flood mitigation alternatives

For the first scenario with higher weight to flood risk criteria (inundation area and flood level), construction of the levee along selected reach of Awash River is the highest rated alternative (0.702). Alternative 2 was the least preferred practice (0.184) than others alternatives. Channel modifications alternatives 3 and 4 were the second and third rated alternatives (0.678 and 0.623) (Table 15).

In the second scenario, a higher weight was assigned to environmental impact criteria, which included flow velocity and stream power at the river bend and downstream control points. In this case, alternatives 3 (0.390) and 1 (0.339) were two more preferred options, respectively. However, channel modification based on alternative 3 performs slightly better than the alternative1 (Table 15).

Scenario three was considered as a balance between all decisions criteria including flood risk and environmental impacts. In this case, alternative 1 was the most rated option (0.333). In all scenarios, alternative 2 was the least preferred option among flood control measures (Table15).As shown above the different form of proposed channel modifications was not a preferred alternative for this river reach under study, with the exception in environmental oriented scenario.

Scenario analysis confirms the superiority of alternative 1 to the other proposed alternatives as well as the strong weakness of alternative 2. Although increasing the size or depth of the channel or decreasing its roughness can lead to a reduction in flood levels because of the additional channel capacity, channel modifications could also have negative effects, such as an increase in flow velocity and stream power (Table 14).

As Wyzga (1993) stated, an increase in flow velocity and stream power owing to river modification results in a progressive out washing of finer grains from bed material, which causes negative impacts near area adjoining the river and downstream. From this synthetic assessment, we can conclude that the levee construction plan is superior to the other three flood mitigation measures.

As a result, when taking risk, and environmental impact considerations into account, alternative 1 was beneficial in protecting residential areas (land adjoining the river) and contributes less to sedimentation and increasing downstream stream power and flow velocity. Different criteria-weighting scenarios enable us to take into consideration all of the evaluation criteria, such as least environmental impact as well as effectiveness in reducing the flood hazard in residential areas or land adjoining the river.

6. CONCLUSION AND RECOMMENDATIONS

6.1 Conclusion

This research emphasis on integrating flood plain inundation mapping and MCDA .As flood hazard control measures are complex, they required a wide-ranging evaluation framework. The appropriate flood mitigation alternatives in reducing the flood risk along the selected reach of Awash River were suggested, modelled and compared. The first step was the development of flood hazard maps, which show inundated areas and flood depths. The second step was to select the most appropriate alternative based on a multicriteria approach. The effects of the suggested alternatives were evaluated in terms of flood hazard reduction, and environmental impacts under 50- and 100-year flood inundation areas and depths.

The most appropriate measure to tackle flood problems along the selected reach of Awash River and Dubti round residential and agricultural area was to construct a levee as a structural flood mitigation measure. Other proposed alternatives, such as channel modification with different dimensions, failed to be optimal options in the study area. As the results of this study show, channel modification activities change the upstream and downstream stream power balance and bring about a greater possibility of channel degradation downstream because of an increase in flow velocity. Such negative impacts of channel modification support the superiority of levee construction.

The application of the hydraulic model, HEC-RAS, combined with GIS analysis and an MCDA method had allowed for the efficient analysis of proposed flood hazard mitigation alternatives. The combination of GIS analysis and hydraulic simulation facilitates the process of producing flood hazard maps and other hydraulics parameters like flow velocity, stream power and flood level at any cross section point along the river reach. The outputs had been transferred to an MCDA approach, which provides a basis for applying a scenario analysis.

Moreover, the multicriteria approach allows us to include different objectives in the decision-making process. Therefore, the evaluation of flood mitigation measures performed in this study pertains to flood risk reduction and environmental objective but not cover economic objective.

6.2 Recommendations

For resolving the flood hazard of the study area, the structural measures were proposed as the main strategy and the non-structural measures, such as flood warning system, retarding basin, land use planning and integrated watershed management, could be also adopted as supplements to strengthen flood hazard mitigation.

For futures studies in this research area, the following recommendations should be considered.

- Higher resolution DEM better 30m*30m resolution is suggested in the future for better visualization as well as improves the reliability.
- Economic oriented criteria will be suggested to evaluate the construction cost of the alternatives as additional criteria.

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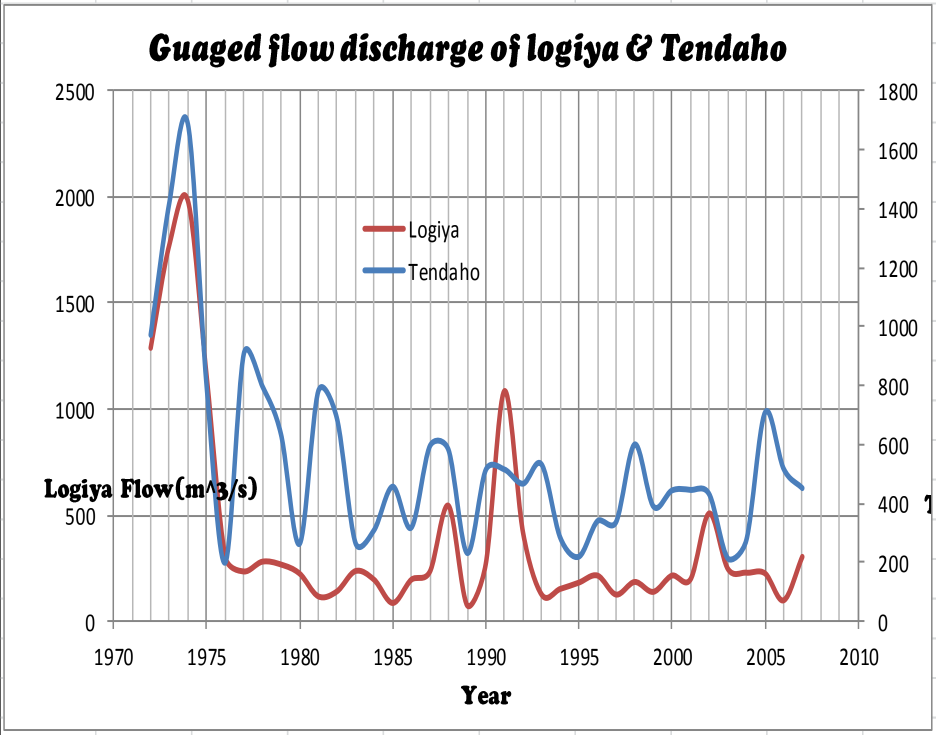
Appendix

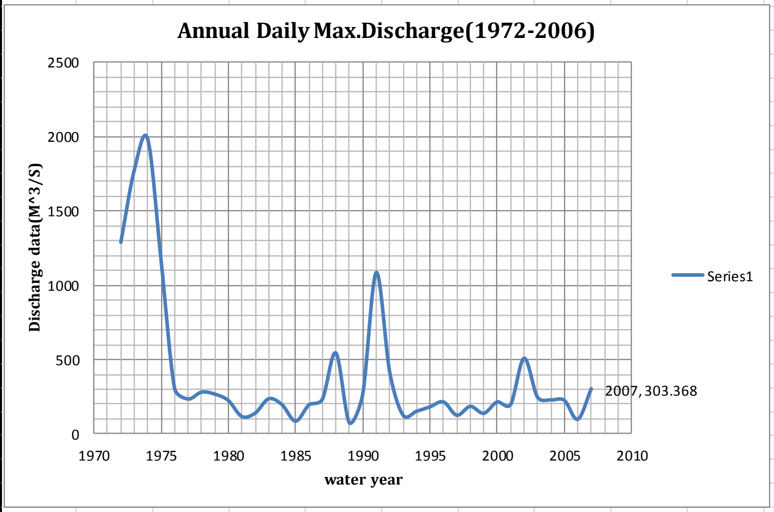
Appendix A: Statistical flood frequency Analysis

year	Tendaho	Logiya
1972	971.598	1288.585
1973	1419.939	1774.048
1974	1687.16	1981.716
1975	801.39	1160.606
1976	198.744	305.3697
1977	906.911	234.356
1978	797.491	280.5
1979	634.036	267.1186
1980	262.8862	224.1225
1981	779.994	117.0987
1982	689.283	140
1983	265.71	236.1545
1984	309.272	195.39
1985	456.725	85.69349
1986	317.8	195.364
1987	594.661	234.0655
1988	580.791	543.7485
1989	231.365	74.9125
1990	513.346	270.061
1991	514.985	1084.555
1992	466.856	417.282
1993	531.511	125.369
1994	283.2963	151.951
1995	218.948	181.756
1996	339.692	214.925
1997	336.988	125.369
1998	600.008	184.919
1999	389.771	138.266
2000	443.716	214.925
2001	445.542	197.912
2002	430.985	508.1458
2003	213.14	247.127
2004	276.275	228.65
2005	708.125	225.054
2006	517.1	98.343
2007	451.5	303.368



Appendix A-1: Annual daily maximum discharge at Logiya Station

		Logiya	Annual daily maximum discharge(m3/s)
4-Feb	1	1972	1288.585
4-Aug	2	1973	1774.048
21-Mar	3	1974	1981.716
15-Sep	4	1975	1160.606
3-Sep	5	1976	305.3696591
14-Aug	6	1977	234.356
27-Sep	7	1978	280.5
3-Sep	8	1979	267.1185969
16-Aug	9	1980	224.122495
28-Mar	10	1981	117.0987254
14-Oct	11	1982	140
25-Apr	12	1983	236.1545
12-May	13	1984	195.39
31-Aug	14	1985	85.69348782
4-Mar	15	1986	195.364
11-Apr	16	1987	234.0655
15-Aug	17	1988	543.7485
30-Mar	18	1989	74.9125
9-Feb	19	1990	270.061
13-Aug	20	1991	1084.5545
24-Aug	21	1992	417.282
13-Feb	22	1993	125.369
30-Jul	23	1994	151.951
11-Jul	24	1995	181.756
23-Aug	25	1996	214.925
30-Mar	26	1997	125.369
11-Aug	27	1998	184.919
8-Sep	28	1999	138.266
11-Aug	29	2000	214.925
1-Sep	30	2001	197.912
25-Dec	31	2002	508.1457772
7-Dec	32	2003	247.127
15-Jan	33	2004	228.65
1-Sep	34	2005	225.054
31-Mar	35	2006	98.343
11-Sep	36	2007	303.368

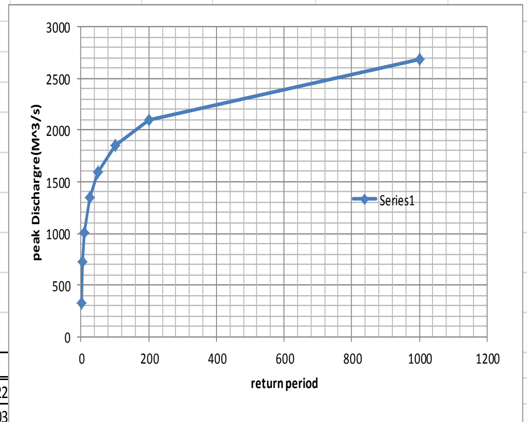


Appendix A-1-1: Gumbel Statistical Distribution

m	Qdaily max.(M^3/S)	(Q-avgQ)^2	(Q-avQ)^3	Reduced Variate,y=((Q-avgQ)+0.45σ)/(0.7797σ)	$T = \frac{1}{1-e^{-e^{-y}}}$	ExceedenceProbability, P=(1/T)*100	T/(T-1)	Yt=-ln(ln(T/(T-1))	K= (Yt-0.577)/(1.2825)	Simulated Qt
1	1981.716	2514422.445	3987102193	4.960911952	37	2.702702703	1.0277778	3.597249705	2.35497053	1488.543495
2	1774.048	1898953.035	2616804849	4.386798252	18.5	5.405405405	1.0571429	2.89011447	1.803598027	1232.750151
3	1288.585	796667.0111	711074740	3.044699521	12.33333333	8.108108108	1.0882353	2.470324827	1.476276668	1080.898863
4	1160.606	584587.2386	446965493	2.690892014	9.25	10.81081081	1.1212121	2.167963722	1.240517522	971.5252176
5	1084.5545	474075.6938	326416072	2.480641961	7.4	13.51351351	1.15625	1.929767083	1.054789149	885.3619045
6	543.7485	21822.83778	3223790.69	0.985543372	6.166666667	16.21621622	1.1935484	1.731997102	0.900582536	813.8222014
7	508.1457772	12571.52811	1409555.26	0.887116985	5.285714286	18.91891892	1.2333333	1.561979439	0.768015157	752.3213959
8	417.282	451.9471581	9607.96671	0.635917285	4.625	21.62162162	1.2758621	1.412137096	0.651179022	698.1186463
9	305.3696591	8218.019364	-744990.51	0.326527257	4.111111111	24.32432432	1.3214286	1.277571256	0.546254391	649.4418949
10	303.368	8584.939972	-795437.19	0.320993521	3.7	27.02702703	1.3703704	1.154925382	0.45062408	605.0769747
11	280.5	13345.55224	-1541717.6	0.257773227	3.363636364	29.72972973	1.4230769	1.041793371	0.362411985	564.1535218
12	270.061	15866.41314	-1998564.4	0.228913832	3.083333333	32.43243243	1.48	0.936386078	0.280223063	526.0243457
13	267.1185969	16616.33255	-2141917.6	0.220779338	2.846153846	35.13513514	1.5416667	0.837331498	0.202987523	490.1931481
14	247.127	22170.00427	-3301023.9	0.165511075	2.642857143	37.83783784	1.6086957	0.743548879	0.129862674	456.2689871
15	236.1545	25557.92167	-4085905.4	0.135176779	2.466666667	40.54054054	1.6818182	0.654165997	0.060168419	423.9363509
16	234.356	26136.20309	-4225360.3	0.130204692	2.3125	43.24324324	1.7619048	0.568462726	-0.006656744	392.9347471
17	234.0655	26230.21598	-4248178.9	0.129401583	2.176470588	45.94594595	1.85	0.485831205	-0.07108678	363.0442932
18	228.65	28013.70478	-4688736.4	0.114430028	2.055555556	48.64864865	1.9473684	0.405746748	-0.133530801	334.0751936
19	225.054	29230.38226	-4997487.8	0.104488618	1.947368421	51.35135135	2.0555556	0.327745806	-0.194350249	305.859768
20	224.122495	29549.76682	-5079618.4	0.101913403	1.85	54.05405405	2.1764706	0.251408559	-0.253872469	278.2461535
21	214.925	32796.46791	-5939373.1	0.076486241	1.761904762	56.75675676	2.3125	0.176344427	-0.312402006	251.0930655
22	214.925	32796.46791	-5939373.1	0.076486241	1.681818182	59.45945946	2.4666667	0.102179235	-0.370230616	224.2651527
23	197.912	39247.94896	-7775448.5	0.029452531	1.608695652	62.16216216	2.6428571	0.028542918	-0.427646848	197.6285506
24	195.39	40253.58109	-8076194.8	0.022480274	1.541666667	64.86486486	2.8461538	-0.044943303	-0.484946045	171.0462432
25	195.364	40264.01468	-8079335	0.022408395	1.48	67.56756757	3.0833333	-0.118681532	-0.542441741	144.3727765
26	184.919	44564.87819	-9407821.9	-0.006467588	1.423076923	70.27027027	3.3636364	-0.193115294	-0.600479762	117.4477133
27	181.756	45910.32635	-9837065.7	-0.015211938	1.37037037	72.97297297	3.7	-0.26875367	-0.659457053	90.08690292
28	151.951	59571.11734	-14539639	-0.097610087	1.321428571	75.67567568	4.1111111	-0.346205667	-0.719848473	62.07004813
29	140	65547.75151	-16781729	-0.130649519	1.275862069	78.37837838	4.625	-0.426232218	-0.782247344	33.12189463
30	138.266	66438.64586	-17125023	-0.135443292	1.233333333	81.08108108	5.2857143	-0.509829786	-0.847430633	2.881990941
31	125.369	73253.56127	-19826366	-0.171098012	1.193548387	83.78378378	6.1666667	-0.598374	-0.916470955	-29.14727236
32	125.369	73253.56127	-19826366	-0.171098012	1.15625	86.48648649	7.4	-0.693886907	-0.990944957	-63.69733372
33	117.0987254	77798.72373	-21699949	-0.193961804	1.121212121	89.18918919	9.25	-0.799587711	-1.073362738	-101.9326822
34	98.343	88613.35331	-26378419	-0.245813408	1.088235294	91.89189189	12.333333	-0.921200907	-1.168187842	-145.9240497
35	85.69348782	96304.37581	-29886085	-0.280783929	1.057142857	94.59459459	18.5	-1.070819877	-1.284849806	-200.045998
36	74.9125	103111.9218	-33110316	-0.310588776	1.027777778	97.2972973	37	-1.283962009	-1.451042502	-277.1462981
		SUM	**SUM**							
SUM	14256.82624	7532797.89	7800928860							
MEAN	396.0229511									
Stdv	463.9211106									

Range	1906.8035		n	36	K=
Variance	215222.7969				
Standard error	1.171450062			outlier	
Coff. of variation	1.171450062			XH=EXP(avgX+KNS)	
Coff. Skweness(Cs)	2.363575342			XY=EXP(avgX+KNS)	
Excess kurtosis	7.85376E-07				

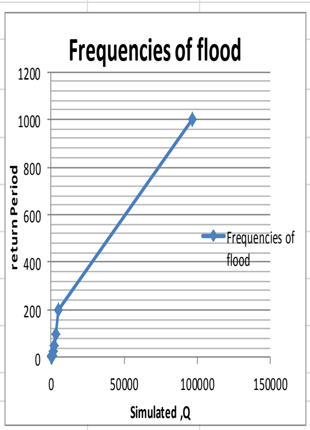
T	U=log(T/(T-1))	log U	Yt	k	Xt
2	0.301029996	-0.521390228	0.366512921	-0.164278424	319.810722
5	0.096910013	-1.013631348	1.499939987	0.71948537	729.807403
10	0.045757491	-1.339537801	2.250367327	1.3046139	1001.260881
25	0.017728767	-1.751321469	3.198534261	2.04392535	1344.24307
50	0.008773924	-2.056806117	3.901938658	2.592388817	1598.68685
100	0.004364805	-2.360035114	4.600149227	3.136802516	1851.251858
200	0.002176919	-2.662157679	5.295812143	3.679229741	2102.895299
1000	0.000434512	-3.361998451	6.907255071	4.935715455	2685.805546



Appendix A-1-2: Log Pearson Type III Statistical Distribution

Water YeaR	Annuaily daily max.Q	Rank(M)	Observed logX	[logX-Avrg(logX)]^2	[logX-Avrg(logX)]^3	[logX-Avrg(logX)]^4	Reduced Variate,y=((Q-avgQ)+0.45σ)/(0.7797σ)	T	Yt=-ln(ln(T/(T-1)))	K	Simulated logXtot.	Simulated Xtot
1972	1288.585	1	3.110113072	0.470864434	0.323105114	14.54976998	3.073898948	37	3.597249705	2.354971	3.254016945	1794.803656
1973	1774.048	2	3.248965366	0.680704068	0.561613475	9.234262207	3.579119318	18.5	2.89011447	1.803598	3.059664604	1147.267269
1974	1981.716	3	3.297041416	0.762345464	0.665622097	7.622135118	3.754046204	12.33333333	2.470324827	1.476277	2.944287663	879.6049457
1975	1160.606	4	3.064684811	0.410582815	0.263088068	16.43099476	2.908606163	9.25	2.167963722	1.240518	2.861185328	726.4158772
1976	305.369659	5	2.484825884	0.003709838	0.00022596	34.30919187	0.798763164	7.4	1.929767083	1.054789	2.795718254	624.7672476
1977	234.356	6	2.369876077	0.00292047	-0.000157826	34.35397452	0.380513078	6.166666667	1.731997102	0.900583	2.741362231	551.2672995
1978	280.5	7	2.447932866	0.00057674	1.38507E-05	34.48719801	0.664526243	5.285714286	1.561979439	0.768015	2.694633785	495.0325829
1979	267.118597	8	2.426704125	7.76557E-06	2.16401E-08	34.51959834	0.587284507	4.625	1.412137096	0.651179	2.653450417	450.2465744
1980	224.122495	9	2.350485449	0.005392258	-0.000395964	34.2138904	0.309959398	4.111111111	1.277571256	0.546254	2.616465715	413.4906705
1981	117.098725	10	2.068552168	0.126284482	-0.04487712	27.86907888	-0.715867628	3.7	1.154925382	0.450624	2.582757152	382.6107357
1982	140	11	2.146128036	0.077166957	-0.021436164	30.32966888	-0.433604315	3.363636364	1.041793371	0.362412	2.551663423	356.1749917
1983	236.1545	12	2.373196226	0.002572642	-0.000130488	34.37372148	0.39259359	3.083333333	0.936386078	0.280223	2.522692791	333.190639
1984	195.39	13	2.290902333	0.017693021	-0.002353439	33.5231272	0.09316353	2.846153846	0.837331498	0.202988	2.495468169	312.9451096
1985	85.6934878	14	1.932947819	0.241051176	-0.118348806	22.70432119	-1.209270198	2.642857143	0.743548879	0.129863	2.469692517	294.9120496
1986	195.364	15	2.290844539	0.017708399	-0.002356508	33.52227021	0.092953243	2.466666667	0.654165997	0.060168	2.44512611	278.6930316
1987	234.0655	16	2.369337406	0.002978981	-0.000162593	34.35065354	0.378553099	2.3125	0.568462726	-0.00666	2.421571023	263.9799993
1988	543.7485	17	2.735398072	0.097020179	0.030219906	29.31627852	1.710481431	2.176470588	0.485831205	-0.07109	2.398860191	250.5302612
1989	74.9125	18	1.874554291	0.301799878	-0.165797734	20.28045825	-1.4217377	2.055555556	0.405746748	-0.13353	2.376849406	238.149353
1990	270.061	19	2.431461871	5.69183E-05	4.29416E-07	34.51679843	0.604595784	1.947368421	0.327745806	-0.19435	2.355411264	226.6789874
1991	1084.5545	20	3.035251381	0.373729178	0.228473328	17.66749502	2.801511291	1.85	0.251408559	-0.25387	2.334430379	215.9883761
1992	417.282	21	2.620429651	0.038617046	0.007588721	32.37220821	1.292163619	1.761904762	0.176344427	-0.3124	2.313799404	205.9678348
1993	125.369	22	2.098190162	0.106098265	-0.0345591	28.86145433	-0.608028443	1.681818182	0.102179235	-0.37023	2.293415497	196.5239555
1994	151.951	23	2.181703563	0.058667566	-0.014210099	31.2973897	-0.304161148	1.608695652	0.028542918	-0.42765	2.273176948	187.5758609
1995	181.756	24	2.259488756	0.027036794	-0.004445625	33.00544858	-0.021136193	1.541666667	-0.044943303	-0.48495	2.252979653	179.0521965
1996	214.925	25	2.332286935	0.008396151	-0.000769344	34.04422746	0.243743282	1.48	-0.118681532	-0.54244	2.232713095	170.8886011
1997	125.369	26	2.098190162	0.106098265	-0.0345591	28.86145433	-0.608028443	1.423076923	-0.193115294	-0.60048	2.212255373	163.0254369
1998	184.919	27	2.266981536	0.02462888	-0.003865156	33.13827811	0.006126626	1.37037037	-0.26875367	-0.65946	2.191466569	155.4055659
1999	138.266	28	2.140715399	0.0802034	-0.022713767	30.17299995	-0.453298439	1.321428571	-0.346205667	-0.71985	2.170179302	147.9719175
2000	214.925	29	2.332286935	0.008396151	-0.000769344	34.04422746	0.243743282	1.275862069	-0.426232218	-0.78225	2.148184431	140.6644756
2001	197.912	30	2.296472128	0.01624231	-0.002070006	33.60404463	0.11342948	1.233333333	-0.509829786	-0.84743	2.125208086	133.4160524
2002	508.145777	31	2.705988321	0.079563978	0.022442681	30.20594117	1.603472718	1.193548387	-0.598374	-0.91647	2.100872182	126.1456219
2003	247.127	32	2.392920197	0.00096083	-2.97831E-05	34.46533891	0.464360154	1.15625	-0.693886907	-0.99094	2.07462097	118.7465416
2004	228.65	33	2.359171206	0.004192076	-0.000271421	34.28185495	0.341562919	1.121212121	-0.799587711	-1.07336	2.045569669	111.0630686
2005	225.054	34	2.352286736	0.005130959	-0.000367534	34.2286788	0.316513465	1.088235294	-0.921200907	-1.16819	2.012144931	102.8359422
2006	98.343	35	1.992743453	0.185911014	-0.080159995	25.08680318	-0.991701076	1.057142857	-1.070819877	-1.28485	1.971022957	93.54551209
2007	303.368	36	2.481969768	0.003370072	0.00019564	34.32846219	0.788371057	1.027777778	-1.283962009	-1.45104	1.91244198	81.74138277
		Sum	87.26102811	4.34867942	1.547782376	1050.173699						
		Mean	2.423917448									

Standard deviation	0.352488274
Range	0.628143303
Variance	0.124247983
Standard error	0.145420907
Coff. of variation	0.145420907
Coff. Skweness(Cs)	1.069132209
Excess kurtosis	1.360431343



Tr	k(1)	k(1.1)	slope	K(1.069132208831	Q (Cumec)
2	-0.164	-0.18	-0.16	-0.175061153	230.2557
5	0.758	0.745	-0.13	0.749012813	487.4563
10	1.34	1.341	0.01	1.340691322	787.9441
25	2.043	2.066	0.23	2.058900408	1411.418
50	2.542	2.585	0.43	2.57172685	2140.036
100	3.022	3.087	0.65	3.066935936	3199
200	3.489	3.575	0.86	3.5484537	4728.308
1000	7.153	7.479	1.63	7.2656855	96602.06

Appendix -A-1-3.Log Normal distribution

				Log Normal -Logiya Station				
Year	**Flow**	Rank	**Flow**	**logQ**	**P**	**w**	**z=k**	**Qt**
1972	1288.585	1	1981.716	3.297041	0.027027	2.687347	2.618056	6.345953
1973	1774.048	2	1774.048	3.248965	0.054054	2.415687	2.30173	5.579203
1974	1981.716	3	1288.585	3.110113	0.081081	2.241564	2.093274	5.073922
1975	1160.606	4	1160.606	3.064685	0.108108	2.109324	1.931332	4.68139
1976	305.3697	5	1084.555	3.035251	0.135135	2.00074	1.795646	4.352497
1977	234.356	6	543.7485	2.735398	0.162162	1.907437	1.676839	4.064519
1978	280.5	7	508.1458	2.705988	0.189189	1.824833	1.569746	3.804935
1979	267.1186	8	417.282	2.62043	0.216216	1.750129	1.471192	3.566047
1980	224.1225	9	305.3697	2.484826	0.243243	1.681483	1.379062	3.342733
1981	117.0987	10	303.368	2.48197	0.27027	1.617611	1.29187	3.131386
1982	140	11	280.5	2.447933	0.297297	1.557577	1.208513	2.929336
1983	236.1545	12	270.061	2.431462	0.324324	1.500674	1.128145	2.73453
1984	195.39	13	267.1186	2.426704	0.351351	1.446353	1.050089	2.545329
1985	85.69349	14	247.127	2.39292	0.378378	1.394174	0.973788	2.360382
1986	195.364	15	236.1545	2.373196	0.405405	1.343777	0.898767	2.178536
1987	234.0655	16	234.356	2.369876	0.432432	1.294858	0.824607	1.99878
1988	543.7485	17	234.0655	2.369337	0.459459	1.24716	0.750931	1.820194
1989	74.9125	18	228.65	2.359171	0.486486	1.200455	0.677382	1.641918
1990	270.061	19	225.054	2.352287	0.513514	1.200455	0.677382	1.641918
1991	1084.555	20	224.1225	2.350485	0.540541	1.24716	0.750931	1.820194
1992	417.282	21	214.925	2.332287	0.567568	1.294858	0.824607	1.99878
1993	125.369	22	214.925	2.332287	0.594595	1.343777	0.898767	2.178536
1994	151.951	23	197.912	2.296472	0.621622	1.394174	0.973788	2.360382
1995	181.756	24	195.39	2.290902	0.648649	1.446353	1.050089	2.545329
1996	214.925	25	195.364	2.290845	0.675676	1.500674	1.128145	2.73453
1997	125.369	26	184.919	2.266982	0.702703	1.557577	1.208513	2.929336
1998	184.919	27	181.756	2.259489	0.72973	1.617611	1.29187	3.131386
1999	138.266	28	151.951	2.181704	0.756757	1.681483	1.379062	3.342733
2000	214.925	29	140	2.146128	0.783784	1.750129	1.471192	3.566047
2001	197.912	30	138.266	2.140715	0.810811	1.824833	1.569746	3.804935
2002	508.1458	31	125.369	2.09819	0.837838	1.907437	1.676839	4.064519
2003	247.127	32	125.369	2.09819	0.864865	2.00074	1.795646	4.352497
2004	228.65	33	117.0987	2.068552	0.891892	2.109324	1.931332	4.68139
2005	225.054	34	98.343	1.992743	0.918919	2.241564	2.093274	5.073922
2006	98.343	35	85.69349	1.932948	0.945946	2.415687	2.30173	5.579203
2007	303.368	36	74.9125	1.874554	0.972973	2.687347	2.618056	6.345953

Appendix-A-1-4 Normal Distribution

				Log Normal -Logiya Station				
Year	Flow	Rank	Flow	logQ	P	w	z=k	Qt
1972	1288.585	1	1981.716	3.297041	0.027027	2.687347	2.618056	6.345953
1973	1774.048	2	1774.048	3.248965	0.054054	2.415687	2.30173	5.579203
1974	1981.716	3	1288.585	3.110113	0.081081	2.241564	2.093274	5.073922
1975	1160.606	4	1160.606	3.064685	0.108108	2.109324	1.931332	4.68139
1976	305.3697	5	1084.555	3.035251	0.135135	2.00074	1.795646	4.352497
1977	234.356	6	543.7485	2.735398	0.162162	1.907437	1.676839	4.064519
1978	280.5	7	508.1458	2.705988	0.189189	1.824833	1.569746	3.804935
1979	267.1186	8	417.282	2.62043	0.216216	1.750129	1.471192	3.566047
1980	224.1225	9	305.3697	2.484826	0.243243	1.681483	1.379062	3.342733
1981	117.0987	10	303.368	2.48197	0.27027	1.617611	1.29187	3.131386
1982	140	11	280.5	2.447933	0.297297	1.557577	1.208513	2.929336
1983	236.1545	12	270.061	2.431462	0.324324	1.500674	1.128145	2.73453
1984	195.39	13	267.1186	2.426704	0.351351	1.446353	1.050089	2.545329
1985	85.69349	14	247.127	2.39292	0.378378	1.394174	0.973788	2.360382
1986	195.364	15	236.1545	2.373196	0.405405	1.343777	0.898767	2.178536
1987	234.0655	16	234.356	2.369876	0.432432	1.294858	0.824607	1.99878
1988	543.7485	17	234.0655	2.369337	0.459459	1.24716	0.750931	1.820194
1989	74.9125	18	228.65	2.359171	0.486486	1.200455	0.677382	1.641918
1990	270.061	19	225.054	2.352287	0.513514	1.200455	0.677382	1.641918
1991	1084.555	20	224.1225	2.350485	0.540541	1.24716	0.750931	1.820194
1992	417.282	21	214.925	2.332287	0.567568	1.294858	0.824607	1.99878
1993	125.369	22	214.925	2.332287	0.594595	1.343777	0.898767	2.178536
1994	151.951	23	197.912	2.296472	0.621622	1.394174	0.973788	2.360382
1995	181.756	24	195.39	2.290902	0.648649	1.446353	1.050089	2.545329
1996	214.925	25	195.364	2.290845	0.675676	1.500674	1.128145	2.73453
1997	125.369	26	184.919	2.266982	0.702703	1.557577	1.208513	2.929336
1998	184.919	27	181.756	2.259489	0.72973	1.617611	1.29187	3.131386
1999	138.266	28	151.951	2.181704	0.756757	1.681483	1.379062	3.342733
2000	214.925	29	140	2.146128	0.783784	1.750129	1.471192	3.566047
2001	197.912	30	138.266	2.140715	0.810811	1.824833	1.569746	3.804935
2002	508.1458	31	125.369	2.09819	0.837838	1.907437	1.676839	4.064519
2003	247.127	32	125.369	2.09819	0.864865	2.00074	1.795646	4.352497
2004	228.65	33	117.0987	2.068552	0.891892	2.109324	1.931332	4.68139
2005	225.054	34	98.343	1.992743	0.918919	2.241564	2.093274	5.073922
2006	98.343	35	85.69349	1.932948	0.945946	2.415687	2.30173	5.579203
2007	303.368	36	74.9125	1.874554	0.972973	2.687347	2.618056	6.345953

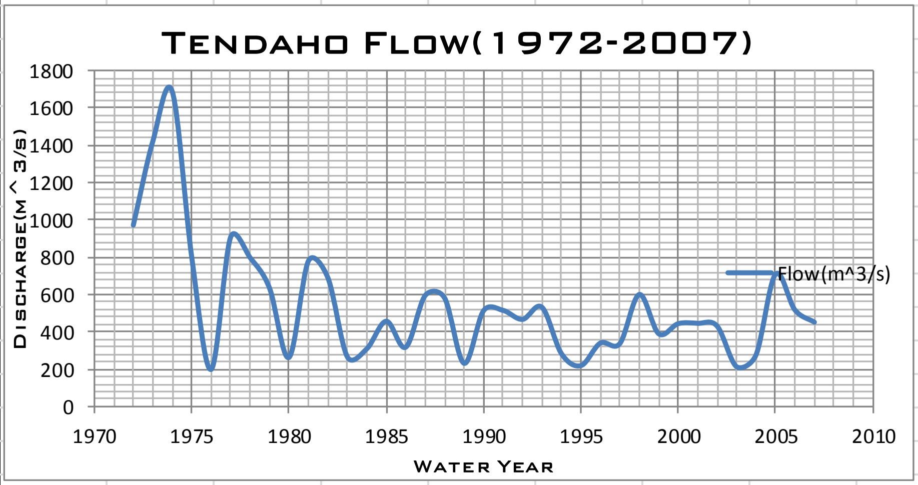
$$z = w - \frac{2.515517+0.80285w+0.010328w^2}{1+1.432788w+0.189269w^2+0.001308w^3}$$

$$w = \sqrt{\ln\left(\frac{1}{P^2}\right)},\ 0 < P \le 0.5$$

where P>0.5, 1-P is substituted for P.

Appendix A-2: Annual daily maximum discharge at Tendaho station

Water Year	Flow(m^3/s)
1972	971.598
1973	1419.939
1974	1687.16
1975	801.39
1976	198.744
1977	906.911
1978	797.491
1979	634.036
1980	262.8862
1981	779.994
1982	689.283
1983	265.71
1984	309.272
1985	456.725
1986	317.8
1987	594.661
1988	580.791
1989	231.365
1990	513.346
1991	514.985
1992	466.856
1993	531.511
1994	283.2963
1995	218.948
1996	339.692
1997	336.988
1998	600.008
1999	389.771
2000	443.716
2001	445.542
2002	430.985
2003	213.14
2004	276.275
2005	708.125
2006	517.1
2007	451.5



Appendix A-2-1: Gumbel Statistical Distribution

Year	Observed Flow(Q)	Rank(M)	Return Period, T=(N+1)/M	Exceedence Probability, P=1/T	(Q-avgQ)	(Q-avgQ)^2	(Q-avgQ)^3	Reduced Variate,y=((Q-avgQ)+0.45σ)/(0.7797σ)	Exceedence Probability, P=(1/T)*100	T/(T-1)	Yt=-ln(ln(T/(T-1)))	K= (Yt-0.577)/(1.2825)	Simulated Qt
1974	1687.16	1	37	0.027027	1143.061653	1306589.94	1493512859	5.123652802	2.702702703	1.02777778	3.597249705	2.35497053	1303.362983
1973	1419.939	2	18.5	0.054054	875.8406528	767096.849	671854605	4.060770373	5.405405405	1.05714286	2.89011447	1.803598027	1125.595305
1972	971.598	3	12.33333	0.081081	427.4996528	182755.953	78128106.5	2.277475754	8.108108108	1.08823529	2.470324827	1.476276668	1020.063822
1977	906.911	4	9.25	0.108108	362.8126528	131633.021	47758125.55	2.020180573	10.81081081	1.12121212	2.167963722	1.240517522	944.0528573
1975	801.39	5	7.4	0.135135	257.2916528	66198.9946	17032448.73	1.60046649	13.51351351	1.15625	1.929767083	1.054789149	884.172284
1978	797.491	6	6.166667	0.162162	253.3926528	64207.8365	16269794.02	1.584958058	16.21621622	1.19354839	1.731997102	0.900582536	834.4546224
1981	779.994	7	5.285714	0.189189	235.8956528	55646.759	13126828.54	1.515363027	18.91891892	1.23333333	1.561979439	0.768015157	791.7136534
2005	708.125	8	4.625	0.216216	164.0266528	26904.7428	4413094.909	1.229501148	21.62162162	1.27586207	1.412137096	0.651179022	754.0445852
1982	689.283	9	4.111111	0.243243	145.1846528	21078.5834	3060286.812	1.154556324	24.32432432	1.32142857	1.277571256	0.546254391	720.2158977
1979	634.036	10	3.7	0.27027	89.93765278	8088.78139	727486.0118	0.934809127	27.02702703	1.37037037	1.154925382	0.45062408	689.3837865
1998	600.008	11	3.363636	0.297297	55.90965278	3125.88927	174767.3839	0.799461369	29.72972973	1.42307692	1.041793371	0.362411985	660.943378
1987	594.661	12	3.083333	0.324324	50.56265278	2556.58186	129267.5607	0.778193458	32.43243243	1.48	0.936386078	0.280223063	634.4448968
1988	580.791	13	2.846154	0.351351	36.69265278	1346.35077	49401.18124	0.723024968	35.13513514	1.54166667	0.837331498	0.202987523	609.5434327
1993	531.511	14	2.642857	0.378378	-12.58734722	158.44131	-1994.355784	0.527011757	37.83783784	1.60869565	0.743548879	0.129862674	585.9672941
2006	517.1	15	2.466667	0.405405	-26.99834722	728.910753	-19679.3856	0.469691417	40.54054054	1.68181818	0.654165997	0.060168419	563.4972111
1991	514.985	16	2.3125	0.432432	-29.11334722	847.586986	-24676.09424	0.461278918	43.24324324	1.76190476	0.568462726	-0.006656744	541.9521504
1990	513.346	17	2.176471	0.459459	-30.75234722	945.70686	-29082.70572	0.454759728	45.94594595	1.85	0.485831205	-0.07108678	521.1793011
1992	466.856	18	2.055556	0.486486	-77.24234722	5966.3802	-460857.2114	0.269843857	48.64864865	1.94736842	0.405746748	-0.133530801	501.0467617
1985	456.725	19	1.947368	0.513514	-87.37334722	7634.1018	-667017.0277	0.229547391	51.35135135	2.05555556	0.327745806	-0.194350249	481.4380002
2007	451.5	20	1.85	0.540541	-92.59834722	8574.45391	-793980.2602	0.20876474	54.05405405	2.17647059	0.251408559	-0.253872469	462.2474769
2001	445.542	21	1.761905	0.567568	-98.55634722	9713.35358	-957312.6479	0.185066552	56.75675676	2.3125	0.176344427	-0.312402006	443.3770038
2000	443.716	22	1.681818	0.594595	-100.3823472	10076.6156	-1011514.329	0.177803563	59.45945946	2.46666667	0.102179235	-0.370230616	424.7325164
2002	430.985	23	1.608696	0.621622	-113.1133472	12794.6293	-1447243.349	0.127165491	62.16216216	2.64285714	0.028542918	-0.427646848	406.2209836
1999	389.771	24	1.541667	0.648649	-154.3273472	23816.9301	-3675603.641	-0.036764876	64.86486486	2.84615385	-0.044943303	-0.484946045	387.7471836
1996	339.692	25	1.48	0.675676	-204.4063472	41781.9548	-8540496.757	-0.235956141	67.56756757	3.08333333	-0.118681532	-0.542441741	369.2100311
1997	336.988	26	1.423077	0.702703	-207.1103472	42894.6959	-8883935.367	-0.246711412	70.27027027	3.36363636	-0.193115294	-0.600479762	350.4980275
1986	317.8	27	1.37037	0.72973	-226.2983472	51210.942	-11588951.52	-0.323032464	72.97297297	3.7	-0.26875367	-0.659457053	331.4831945
1984	309.272	28	1.321429	0.756757	-234.8263472	55143.4133	-12949126.33	-0.356952932	75.67567568	4.11111111	-0.346205667	-0.719848473	312.0124329
1994	283.2963	29	1.275862	0.783784	-260.8020472	68017.7078	-17739157.45	-0.460272339	78.37837838	4.625	-0.426232218	-0.782247344	291.8944503
2004	276.275	30	1.233333	0.810811	-267.8233472	71729.3453	-19210793.36	-0.488199846	81.08108108	5.28571429	-0.509829786	-0.847430633	270.8787453
1983	265.71	31	1.193548	0.837838	-278.3883472	77500.0719	-21575116.92	-0.530222564	83.78378378	6.16666667	-0.598374	-0.916470955	248.6194962
1980	262.8862	32	1.15625	0.864865	-281.2121472	79080.2717	-22238333.02	-0.541454344	86.48648649	7.4	-0.693886907	-0.990944957	224.6083781
1989	231.365	33	1.121212	0.891892	-312.7333472	97802.1465	-30585992.63	-0.666831203	89.18918919	9.25	-0.799587711	-1.073362738	198.0361109
1995	218.948	34	1.088235	0.918919	-325.1503472	105722.748	-34375788.32	-0.716220327	91.89189189	12.3333333	-0.921200907	-1.168187842	167.463606
2003	213.14	35	1.057143	0.945946	-330.9583472	109533.428	-36251002.16	-0.739321884	94.59459459	18.5	-1.070819877	-1.284849806	129.8506918
1976	198.744	36	1.027778	0.972973	-345.3543472	119269.625	-41190283.54	-0.796582562	97.2972973	37	-1.283962009	-1.451042502	76.26860441

Mean	544.0983
Standard Error	53.7349
Median	461.7905
Standard Deviation	322.4094
Sample Variance	103947.8
Kurtosis	4.379542
Skewness	1.870364
Range	1488.416
Minimum	198.744
Maximum	1687.16
Sum	19587.54

T	U=log(T/(T-1))	log U	Yt	k	Xt
2	0.301029996	-0.52139023	0.366512921	-0.164278424	491.1334389
5	0.096910013	-1.01363135	1.499939987	0.71948537	776.0671937
10	0.045757491	-1.3395378	2.250367327	1.3046139	964.7181323
25	0.017728767	-1.75132147	3.198534261	2.04392535	1203.079093
50	0.008773924	-2.05680612	3.901938658	2.592388817	1379.908871
100	0.004364805	-2.36003511	4.600149227	3.136802516	1555.432965
200	0.002176919	-2.66215768	5.295812143	3.679229741	1730.316601
1000	0.000434512	-3.36199845	6.907255071	4.935715455	2135.419406

Appendix A-2-2: Log Pearson Type III Statistical Distribution

Rank(m)	Observed X(M^3)	Returnperiod, Tr=(N+1)/m	Exceedence Probability (1/Tr*100%)	Observed logX	[logX-Avrg(logX)]^2	[logX-Avrg(logX)]^3	[logX-Avrg(logX)]^4	Reduced Variate,y=((Q-avgQ)+0.45σ)/(0.7797σ)	Yt=-ln(ln(T/(T-1)))	K= (Yt-0.577)/(1.2825)	Simulated logXtot.	Simulated Xtot
1	1687.16	37	2.702702703	3.22715627	0.277893733	0.146493424	0.077224927	3.46337175	3.597249705	2.35497053	3.20793124	1614.103
2	1419.939	18.5	5.405405405	3.152269688	0.20454787	0.092510801	0.041839831	3.053361158	2.89011447	1.803598027	3.08325813	1211.318
3	971.598	12.33333333	8.108108108	2.987486612	0.082648552	0.023760352	0.006830783	2.151159405	2.470324827	1.476276668	3.00924614	1021.518
4	906.911	9.25	10.81081081	2.95756467	0.066339559	0.017086727	0.004400937	1.987334152	2.167963722	1.240517522	2.95593767	903.5198
5	801.39	7.4	13.51351351	2.903843919	0.041552343	0.008470192	0.001726597	1.693208343	1.929767083	1.054789149	2.91394186	820.2417
6	797.491	6.166666667	16.21621622	2.901725791	0.040693295	0.008208887	0.001655944	1.681611405	1.731997102	0.900582536	2.87907356	756.9611
7	779.994	5.285714286	18.91891892	2.892091262	0.036899053	0.007087986	0.00136154	1.628861518	1.561979439	0.768015157	2.84909821	706.4773
8	708.125	4.625	21.62162162	2.850109927	0.02253299	0.003382426	0.000507736	1.399010038	1.412137096	0.651179022	2.82267991	664.783
9	689.283	4.111111111	24.32432432	2.838397567	0.019153887	0.002650851	0.000366871	1.334883844	1.277571256	0.546254391	2.79895496	629.4409
10	634.036	3.7	27.02702703	2.802113917	0.010427252	0.001064768	0.000108728	1.136227686	1.154925382	0.45062408	2.7773316	598.8687
11	600.008	3.363636364	29.72972973	2.778157041	0.006108523	0.000477424	3.73141E-05	1.005061692	1.041793371	0.362411985	2.7573856	571.9863
12	594.661	3.083333333	32.43243243	2.774269457	0.005515952	0.000409667	3.04257E-05	0.983776829	0.936386078	0.280223063	2.73880152	548.0265
13	580.791	2.846153846	35.13513514	2.764019878	0.004098545	0.000262388	1.67981E-05	0.927659487	0.837331498	0.202987523	2.72133747	526.4262
14	531.511	2.642857143	37.83783784	2.725512257	0.000650875	1.66053E-05	4.23639E-07	0.716826894	0.743548879	0.129862674	2.70480291	506.7607
15	517.1	2.466666667	40.54054054	2.713574538	0.000184268	2.50135E-06	3.39547E-08	0.651466838	0.654165997	0.060168419	2.68904406	488.7019
16	514.985	2.3125	43.24324324	2.71179458	0.000139112	1.64077E-06	1.93522E-08	0.64172141	0.568462726	-0.006656744	2.67393394	471.9912
17	513.346	2.176470588	45.94594595	2.710410182	0.000108372	1.12817E-06	1.17445E-08	0.634141715	0.485831205	-0.07108678	2.6593654	456.4208
18	466.856	2.055555556	48.64864865	2.669182945	0.000949691	-2.92667E-05	9.01913E-07	0.408418982	0.405746748	-0.133530801	2.64524592	441.8206
19	456.725	1.947368421	51.35135135	2.659654784	0.001627736	-6.56714E-05	2.64953E-06	0.356251472	0.327745806	-0.194350249	2.63149379	428.0493
20	451.5	1.85	54.05405405	2.654657755	0.002055919	-9.322E-05	4.2268E-06	0.328892296	0.251408559	-0.253872469	2.61803497	414.9875
21	445.542	1.761904762	56.75675676	2.64888865	0.00261237	-0.000133522	6.82448E-06	0.297305944	0.176344427	-0.312402006	2.60480062	402.5322
22	443.716	1.681818182	59.45945946	2.647105089	0.002797872	-0.000147993	7.82809E-06	0.287540793	0.102179235	-0.370230616	2.59172475	390.5933
23	430.985	1.608695652	62.16216216	2.634462155	0.004295209	-0.000281499	1.84488E-05	0.218319623	0.028542918	-0.427646848	2.57874213	379.0898
24	389.771	1.541666667	64.86486486	2.590809523	0.01192256	-0.00130183	0.000142147	-0.020682356	-0.044943303	-0.484946045	2.56578597	367.9476
25	339.692	1.48	67.56756757	2.531085319	0.028532169	-0.004819502	0.000814085	-0.347677598	-0.118681532	-0.542441741	2.55278538	357.0963
26	336.988	1.423076923	70.27027027	2.527614436	0.029716783	-0.005122744	0.000883087	-0.366680986	-0.193115294	-0.600479762	2.53966217	346.4672
27	317.8	1.37037037	72.97297297	2.502153893	0.039143082	-0.007744306	0.001532181	-0.506079687	-0.26875367	-0.659457053	2.52632657	335.9902
28	309.272	1.321428571	75.67567568	2.490340603	0.043957063	-0.009216011	0.001932223	-0.570758484	-0.346205667	-0.719848473	2.51267121	325.5901
29	283.2963	1.275862069	78.37837838	2.452240902	0.06138457	-0.015208586	0.003768065	-0.779357676	-0.426232218	-0.782247344	2.49856195	315.1824
30	276.275	1.233333333	81.08108108	2.441341588	0.066904174	-0.017305328	0.004476169	-0.839032378	-0.509829786	-0.847430633	2.48382308	304.6654
31	265.71	1.193548387	83.78378378	2.424407899	0.075951006	-0.020931497	0.005768555	-0.931745803	-0.598374	-0.916470955	2.46821209	293.9085
32	262.8862	1.15625	86.48648649	2.419767789	0.078530092	-0.022006661	0.006166975	-0.957150814	-0.693886907	-0.990944957	2.45137247	282.7304
33	231.365	1.121212121	89.18918919	2.364297661	0.11269606	-0.037832331	0.012700402	-1.260854615	-0.799587711	-1.073362738	2.43273664	270.8549
34	218.948	1.088235294	91.89189189	2.340340982	0.129354609	-0.046523552	0.016732615	-1.392019527	-0.921200907	-1.168187842	2.41129535	257.8074
35	213.14	1.057142857	94.59459459	2.328664961	0.137889711	-0.051203281	0.019013572	-1.455946763	-1.070819877	-1.284849806	2.38491643	242.6143
36	198.744	1.027777778	97.2972973	2.298294026	0.161367689	-0.064822365	0.026039531	-1.622230289	-1.283962009	-1.451042502	2.34733791	222.504
					1.38257054	0.1696478	53.1441	-14.20559113				

Mean	2.7	2.7							
Standard Error	0.040778		Tr	K(0.3)	K(0.4)	slope	K(0.365)	Q (cum/s)
Standard Deviation	0.234251		2	-0.050	-0.066	-0.16	-0.060	476
Sample Variance	0.054874		5	0.824	0.816	-0.08	0.827	783
Kurtosis	-0.11797		10	1.309	1.317	0.08	1.306	1014
Skewness	0.365005		25	1.849	1.880	0.31	1.838	1351
Range	0.928862		50	2.211	2.261	0.5	2.300	1733
Minimum	2.298294		100	2.544	2.615	0.71	2.519	1950
Maximum	3.227156		200	2.856	2.949	0.93	2.823	2298
Sum	96.31581		1000	5.352	5.621	2.69	5.258	8544

Appendix- A-2-3.Log Normal Distribution

		Log normal-Tendaho station				
Rank	Observed ,Q	Observed,logQ	P=m/(N+1)	w	z=K	Expected, Qt
1	1687.16	3.22715627	0.027027027	2.687347	2.618056	2.774102386
2	1419.939	3.152269688	0.054054054	2.415687	2.30173	2.762181396
3	971.598	2.987486612	0.081081081	2.241564	2.093274	2.754325582
4	906.911	2.95756467	0.108108108	2.109324	1.931332	2.748222719
5	801.39	2.903843919	0.135135135	2.00074	1.795646	2.743109272
6	797.491	2.901725791	0.162162162	1.907437	1.676839	2.738631946
7	779.994	2.892091262	0.189189189	1.824833	1.569746	2.734596097
8	708.125	2.850109927	0.216216216	1.750129	1.471192	2.730881991
9	689.283	2.838397567	0.243243243	1.681483	1.379062	2.72741004
10	634.036	2.802113917	0.27027027	1.617611	1.29187	2.724124127
11	600.008	2.778157041	0.297297297	1.557577	1.208513	2.72098277
12	594.661	2.774269457	0.324324324	1.500674	1.128145	2.717954036
13	580.791	2.764019878	0.351351351	1.446353	1.050089	2.71501245
14	531.511	2.725512257	0.378378378	1.394174	0.973788	2.712136995
15	517.1	2.713574538	0.405405405	1.343777	0.898767	2.709309767
16	514.985	2.71179458	0.432432432	1.294858	0.824607	2.706515022
17	513.346	2.710410182	0.459459459	1.24716	0.750931	2.703738473
18	466.856	2.669182945	0.486486486	1.200455	0.677382	2.700966729
19	456.725	2.659654784	0.513513514	1.200455	0.677382	2.700966729
20	451.5	2.654657755	0.540540541	1.24716	0.750931	2.703738473
21	445.542	2.64888865	0.567567568	1.294858	0.824607	2.706515022
22	443.716	2.647105089	0.594594595	1.343777	0.898767	2.709309767
23	430.985	2.634462155	0.621621622	1.394174	0.973788	2.712136995
24	389.771	2.590809523	0.648648649	1.446353	1.050089	2.71501245
25	339.692	2.531085319	0.675675676	1.500674	1.128145	2.717954036
26	336.988	2.527614436	0.702702703	1.557577	1.208513	2.72098277
27	317.8	2.502153893	0.72972973	1.617611	1.29187	2.724124127
28	309.272	2.490340603	0.756756757	1.681483	1.379062	2.72741004
29	283.2963	2.452240902	0.783783784	1.750129	1.471192	2.730881991
30	276.275	2.441341588	0.810810811	1.824833	1.569746	2.734596097
31	265.71	2.424407899	0.837837838	1.907437	1.676839	2.738631946
32	262.8862	2.419767789	0.864864865	2.00074	1.795646	2.743109272
33	231.365	2.364297661	0.891891892	2.109324	1.931332	2.748222719
34	218.948	2.340340982	0.918918919	2.241564	2.093274	2.754325582
35	213.14	2.328664961	0.945945946	2.415687	2.30173	2.762181396
36	198.744	2.298294026	0.972972973	2.687347	2.618056	2.774102386
	Mean	2.675439125				
	Standard Erro	0.037685689				
	Median	2.664418865				
	Standard Devi	0.226114133				
	Sample Variai	0.051127601				
	Kurtosis	-0.11797363				
	Skewness	0.365004706				

Appendix-A-2-4 .Normal Distribution

	Normal Distribution-Tendaho station				
Rank	Observed,Q	P=m/(N+1)	w	z=K	Expected, Qt
1	1687.16	0.027027027	2.687347	2.618056	684.7793482
2	1419.939	0.054054054	2.415687	2.30173	667.7815649
3	971.598	0.081081081	2.241564	2.093274	656.5801941
4	906.911	0.108108108	2.109324	1.931332	647.8783026
5	801.39	0.135135135	2.00074	1.795646	640.5871914
6	797.491	0.162162162	1.907437	1.676839	634.2031065
7	779.994	0.189189189	1.824833	1.569746	628.4485089
8	708.125	0.216216216	1.750129	1.471192	623.1526766
9	689.283	0.243243243	1.681483	1.379062	618.2021241
10	634.036	0.27027027	1.617611	1.29187	613.5168398
11	600.008	0.297297297	1.557577	1.208513	609.0376719
12	594.661	0.324324324	1.500674	1.128145	604.7190911
13	580.791	0.351351351	1.446353	1.050089	600.5247712
14	531.511	0.378378378	1.394174	0.973788	596.4247451
15	517.1	0.405405405	1.343777	0.898767	592.3934844
16	514.985	0.432432432	1.294858	0.824607	588.408542
17	513.346	0.459459459	1.24716	0.750931	584.4495433
18	466.856	0.486486486	1.200455	0.677382	580.4973953
19	456.725	0.513513514	1.200455	0.677382	580.4973953
20	451.5	0.540540541	1.24716	0.750931	584.4495433
21	445.542	0.567567568	1.294858	0.824607	588.408542
22	443.716	0.594594595	1.343777	0.898767	592.3934844
23	430.985	0.621621622	1.394174	0.973788	596.4247451
24	389.771	0.648648649	1.446353	1.050089	600.5247712
25	339.692	0.675675676	1.500674	1.128145	604.7190911
26	336.988	0.702702703	1.557577	1.208513	609.0376719
27	317.8	0.72972973	1.617611	1.29187	613.5168398
28	309.272	0.756756757	1.681483	1.379062	618.2021241
29	283.2963	0.783783784	1.750129	1.471192	623.1526766
30	276.275	0.810810811	1.824833	1.569746	628.4485089
31	265.71	0.837837838	1.907437	1.676839	634.2031065
32	262.8862	0.864864865	2.00074	1.795646	640.5871914
33	231.365	0.891891892	2.109324	1.931332	647.8783026
34	218.948	0.918918919	2.241564	2.093274	656.5801941
35	213.14	0.945945946	2.415687	2.30173	667.7815649
36	198.744	0.972972973	2.687347	2.618056	684.7793482

Mean	544.0983472
Standard Error	53.73490002
Median	461.7905
Standard Devia	322.4094001
Sample Varian	103947.8213
Kurtosis	4.379541813
Skewness	1.870364341

Appendix A-3: Distribution Fitting

a. Logiya

	Gumbel					
S/N	Obesrved,X	Expected,Y	Max.Absolute Erro	RMSE(rootmean square error)	RRMSE(relative rootmeansquare error)	Correlation Coefficient(CC)
1	1981.716	1981.82358	0.107576049	0.018183671	0.000167031	3.97692E-07
2	1774.048	1774.14836	0.100359336	0.017211484	0.000180412	5.26588E-07
3	1288.585	1288.66849	0.08348891	0.014533554	0.000227433	1.25519E-06
4	1160.606	1160.68504	0.079041485	0.013972693	0.000246085	1.71055E-06
5	1084.5545	1084.6309	0.076398604	0.013721594	0.000259192	2.10929E-06
6	543.7485	543.806105	0.057604942	0.010517175	0.000456779	4.5822E-05
7	508.145777	508.202145	0.056367705	0.01046722	0.000484711	7.95421E-05
8	417.282	417.33521	0.053210079	0.01005576	0.000578308	0.002212571
9	305.369659	305.41898	0.04932099	0.009491829	0.000774335	0.00012168
10	303.368	303.417251	0.04925143	0.009659	0.000779237	0.000116479
11	280.5	280.548457	0.048456739	0.009691348	0.000840565	7.49287E-05
12	270.061	270.109094	0.048093971	0.009817141	0.000872268	6.3024E-05
13	267.118597	267.166589	0.047991719	0.010006965	0.000881685	6.01797E-05
14	247.127	247.174297	0.047296987	0.010083751	0.000952105	4.51044E-05
15	236.1545	236.201416	0.046915679	0.010237841	0.000996273	3.91255E-05
16	234.356	234.402853	0.046853179	0.010476689	0.001003942	3.82598E-05
17	234.0655	234.112343	0.046843084	0.01074654	0.001005193	3.81226E-05
18	228.65	228.696655	0.046654889	0.010996663	0.00102915	3.56956E-05
19	225.054	225.10053	0.046529923	0.011285164	0.001045755	3.42098E-05
20	224.122495	224.168993	0.046497553	0.011624388	0.001050152	3.384E-05
21	214.925	214.971178	0.046177928	0.01192309	0.001095818	3.049E-05
22	214.925	214.971178	0.046177928	0.012341571	0.001095818	3.049E-05
23	197.912	197.957587	0.045586706	0.012643477	0.001192734	2.54782E-05
24	195.39	195.435499	0.045499064	0.013134448	0.001208703	2.48416E-05
25	195.364	195.409498	0.04549816	0.013718211	0.00120887	2.48352E-05
26	184.919	184.964135	0.045135184	0.014272998	0.001280241	2.24384E-05
27	181.756	181.801025	0.045025266	0.015008422	0.001303679	2.17808E-05
28	151.951	151.99499	0.043989506	0.015552639	0.001580103	1.67861E-05
29	140	140.043574	0.043574194	0.016469497	0.001729952	1.52555E-05
30	138.266	138.309514	0.043513936	0.01776449	0.001754247	1.5051E-05
31	125.369	125.412066	0.043065749	0.019259589	0.001960548	1.36507E-05
32	125.369	125.412066	0.043065749	0.021532875	0.001960548	1.36507E-05
33	117.098725	117.141504	0.042778347	0.02469809	0.002122408	1.28532E-05
34	98.343	98.3851266	0.042126563	0.029787979	0.002621274	1.12846E-05
35	85.6934878	85.7351748	0.041686977	0.041686977	0.003128791	1.03834E-05
36	74.9125	74.9538123	0.041312325	0	0.003764594	9.69786E-06
		SUM	1.888966836	0.512574824	0.042838937	0.003343551
MEAN	396.022951	396.075422				

	Log-Pearson-(III)					
S/N	Obesrved,X	Expected,Y	Max.Absolute Error(MAE)	RMSE(rootmeans quare error)	RRMSE(relative rootmeansquare error)	Correlation Coefficient(CC)
1	3.1101131	3.11017679	6.37139E-05	1.07696E-05	3.57213E-06	2.12367972
2	3.2489654	3.24903391	6.85392E-05	1.17544E-05	3.68622E-06	1.469016122
3	3.2970414	3.29711163	7.02099E-05	1.2222E-05	3.72375E-06	1.311695678
4	3.0646848	3.06474695	6.21352E-05	1.09841E-05	3.53282E-06	2.435477606
5	2.4848259	2.48486787	4.19845E-05	7.54063E-06	2.91858E-06	269.5442076
6	2.3698761	2.36991407	3.79898E-05	6.93596E-06	2.76425E-06	342.3987695
7	2.4479329	2.44797357	4.07024E-05	7.55824E-06	2.87052E-06	1733.822394
8	2.4267041	2.42674409	3.99646E-05	7.55261E-06	2.84225E-06	128769.0803
9	2.3504854	2.35052276	3.7316E-05	7.18146E-06	2.73683E-06	185.4446058
10	2.0685522	2.06857969	2.75184E-05	5.39681E-06	2.28379E-06	7.918354126
11	2.146128	2.14615825	3.02143E-05	6.04286E-06	2.41965E-06	12.95846414
12	2.3731962	2.37323433	3.81052E-05	7.77819E-06	2.7689E-06	388.6919012
13	2.2909023	2.29093758	3.52454E-05	7.34917E-06	2.64985E-06	56.51749719
14	1.9329478	1.93297063	2.2806E-05	4.86226E-06	2.02145E-06	4.14835251
15	2.2908445	2.29087978	3.52434E-05	7.69073E-06	2.64977E-06	56.46841622
16	2.3693374	2.36937538	3.79711E-05	8.49059E-06	2.76349E-06	335.6735944
17	2.7353981	2.73544876	5.06921E-05	1.16296E-05	3.21313E-06	10.30677594
18	1.8745543	1.87457507	2.07768E-05	4.89713E-06	1.89732E-06	3.313338813
19	2.4314619	2.431502	4.013E-05	9.73295E-06	2.84863E-06	17568.42174
20	3.0352514	3.03531249	6.11124E-05	1.52781E-05	3.50679E-06	2.675641373
21	2.6204297	2.62047635	4.66968E-05	1.20571E-05	3.08443E-06	25.89440024
22	2.0981902	2.09821871	2.85484E-05	7.62988E-06	2.33682E-06	9.42489731
23	2.1817036	2.18173501	3.14506E-05	8.72282E-06	2.47889E-06	17.0446009
24	2.2594888	2.25952291	3.41537E-05	9.85933E-06	2.60227E-06	36.98534788
25	2.3322869	2.33232362	3.66835E-05	1.10605E-05	2.71071E-06	119.0980579
26	2.0981902	2.09821871	2.85484E-05	9.02779E-06	2.33682E-06	9.42489731
27	2.2669815	2.26701595	3.44141E-05	1.14714E-05	2.61373E-06	40.60132813
28	2.1407154	2.14074543	3.00262E-05	1.06159E-05	2.41047E-06	12.46786602
29	2.3322869	2.33232362	3.66835E-05	1.38651E-05	2.71071E-06	119.0980579
30	2.2964721	2.29650757	3.54389E-05	1.44679E-05	2.65816E-06	61.56545928
31	2.7059883	2.70603799	4.96701E-05	2.22131E-05	3.18116E-06	12.56806511
32	2.3929202	2.39295899	3.87906E-05	1.93953E-05	2.7963E-06	1040.731169
33	2.3591712	2.35920882	3.76178E-05	2.17186E-05	2.74916E-06	238.5370133
34	2.3522867	2.35232411	3.73786E-05	2.64306E-05	2.73939E-06	194.888575
35	1.9927435	1.99276834	2.4884E-05	2.4884E-05	2.14133E-06	5.378730546
36	2.4819698	2.48201165	4.18852E-05	0	2.91491E-06	296.7192583
		SUM	0.001435241	0.000395067	0.000100135	151995.1519
MEAN	2.4239174	2.42395732				

Error Estimation method	Best fit criteria	Gumbel	Log-Pearson-III
RMSE	lowest	0.512574824	0.000395067
RRMSE	lowest	0.042838937	0.000100135
CC	highest	0.003343551	151995.1519
MAE	lowest	1.888966836	0.001435241

****Log-pearson-III- Is best fit

Logiya Station

			Lognormal							Normal distrn.			
rank	Obesrved,X	Expected,Y	MAE	RMSE	RRMSE	CC	rank	Obesrved,X	Expected,Y	MAE	RMSE	RRMSE	CC
1	3.297041416	6.345952678	3.048911262	0.5153601	0.15631	1	1	1981.716	1610.595	371.1214	62.73097	0.031655	1
2	3.248965366	5.579202888	2.330237522	0.6581138	0.279313	0.99241	2	1774.048	1463.844	310.204	82.95244	0.061643	0.99667
3	3.110113072	5.073922366	1.963809295	0.7504022	0.38923	0.98949	3	1288.585	1367.137	78.55176	85.30303	0.072254	0.996957
4	3.064684811	4.681390389	1.616705577	0.8138671	0.482485	0.98227	4	1160.606	1292.009	131.4028	89.68604	0.092269	0.997027
5	3.035251381	4.352497035	1.317245654	0.8600691	0.56043	0.97006	5	1084.555	1229.061	144.5064	94.7453	0.116199	0.995164
6	2.735398072	4.064518552	1.32912048	0.9073376	0.649142	0.96994	6	543.7485	1173.944	630.1953	150.047	0.3278	0.991621
7	2.705988321	3.804935497	1.098947176	0.9451424	0.724556	0.96686	7	508.1458	1124.261	616.1156	190.7355	0.552951	0.990949
8	2.620429651	3.5660468	0.945617148	0.9783318	0.792753	0.96314	8	417.282	1078.540	661.2578	230.8589	0.852427	0.990792
9	2.484825884	3.342733262	0.857907377	1.0098722	0.859198	0.96129	9	305.3697	1035.799	730.4294	273.9161	1.312759	0.990747
10	2.481969768	3.131385659	0.649415891	1.0369606	0.910513	0.95551	10	303.368	995.349	691.9806	310.3749	1.760098	0.990056
11	2.447932866	2.929335724	0.481402858	1.0618704	0.949844	0.9467	11	280.5	956.678	676.1776	344.2013	2.242222	0.987304
12	2.431461871	2.734529676	0.303067804	1.0855312	0.975287	0.9332	12	270.061	919.393	649.3321	375.4717	2.733016	0.980951
13	2.426704125	2.545328891	0.118624767	1.1091544	0.98548	0.91352	13	267.119	883.181	616.0628	404.4873	3.213918	0.969713
14	2.392920197	2.360381589	0.032538608	1.1341036	0.988379	0.89167	14	247.127	847.784	600.6567	432.9506	3.732115	0.954433
15	2.373196226	2.178536215	0.19466001	1.161569	1.006278	0.86699	15	236.155	812.980	576.8253	460.6696	4.265129	0.934939
16	2.369876077	1.998780201	0.371095876	1.193143	1.041292	0.8383	16	234.356	778.576	544.2197	487.4792	4.784387	0.911252
17	2.369337406	1.820194475	0.54914293	1.2306045	1.094464	0.8064	17	234.066	744.396	510.3301	513.6637	5.284579	0.884264
18	2.359171206	1.641917778	0.717253428	1.2755786	1.166124	0.77424	18	228.65	710.275	481.6247	539.8106	5.781059	0.855683
19	2.352286736	1.641917778	0.710368958	1.3238189	1.239368	0.74851	19	225.054	710.275	485.2207	567.7903	6.30397	0.832559
20	2.350485449	1.820194475	0.530290973	1.3709861	1.29577	0.73181	20	224.122	744.396	520.2731	599.5437	6.884315	0.818162
21	2.332286935	1.998780201	0.333506735	1.4185644	1.332691	0.7231	21	214.925	778.576	563.6507	636.0789	7.561453	0.810127
22	2.332286935	2.178536215	0.15375072	1.4689285	1.35031	0.71814	22	214.925	812.980	598.0548	677.5278	8.30514	0.805878
23	2.296472128	2.360381589	0.063909461	1.5244822	1.358028	0.71837	23	197.912	847.784	649.8717	725.8389	9.215858	0.804923
24	2.290902333	2.545328891	0.254426558	1.5884303	1.390089	0.71997	24	195.39	883.181	687.7914	781.1319	10.23202	0.805487
25	2.290844539	2.734529676	0.443685137	1.6644461	1.448485	0.72193	25	195.364	919.393	724.0291	844.5667	11.34944	0.806714
26	2.266981536	2.929335724	0.662354187	1.7582065	1.540878	0.72388	26	184.919	956.678	771.7586	918.7946	12.66921	0.807958
27	2.259488756	3.131385659	0.871896903	1.8759621	1.669506	0.7244	27	181.756	995.349	813.5926	1005.748	14.16131	0.808275
28	2.181703563	3.342733262	1.161029699	2.0316586	1.857655	0.71975	28	151.951	1035.799	883.8481	1111.584	16.21781	0.806213
29	2.146128036	3.5660468	1.419918764	2.237258	2.107723	0.70826	29	140	1078.540	938.5398	1240.15	18.75163	0.800621
30	2.140715399	3.804935497	1.664220098	2.5102089	2.425101	0.69023	30	138.266	1124.261	985.9954	1398.69	21.66291	0.790512
31	2.098190162	4.064518552	1.96632839	2.8869825	2.844209	0.66014	31	125.369	1173.944	1048.575	1602.343	25.40336	0.773712
32	2.098190162	4.352497035	2.254306873	3.4188902	3.381412	0.62075	32	125.369	1229.061	1103.692	1874.543	29.80514	0.7492
33	2.068552168	4.681390389	2.612838221	4.2261946	4.110677	0.56613	33	117.099	1292.009	1174.91	2268.337	35.59798	0.714253
34	1.992743453	5.073922366	3.081178914	5.6158627	5.204005	0.48595	34	98.343	1367.137	1268.794	2919.409	44.72087	0.665181
35	1.932947819	5.579202888	3.646255068	8.7390504	7.090375	0.38	35	85.693	1463.844	1378.15	4352.608	60.8032	0.599251
36	1.874554291	6.345952678	4.471398387	#DIV/0!	#DIV/0!	0.24403	36	74.913	1610.595	1535.682	#DIV/0!	#DIV/0!	0.510427
Mean	2.423917448	3.45286609					Mean	396.0229511	1056.87775				

	lognormal	Normal distrn.	Best fit
RMSE	63.4	28655	lowest
RRMSE	55.7	377	lowest
CC	28.3	31.1	highest
MAE	44.2	25153.4	lowest

b. Tendaho

Gumbel						
S/N	Obesrved,X	Expected,Y	Max.Absolute Error(MAE	RMSE(rootmeansquare erro	RRMSE(relative rootmeansquare erro	Correlation Coefficient(CC)
1	1687.16	1303.363	383.7970169	64.87353635	0.002166487	1.22945E-06
2	1419.939	1125.5953	294.3436946	50.47952718	0.002032286	2.13795E-06
3	971.598	1020.0638	48.46582224	8.43681673	-0.000503865	5.62438E-06
4	906.911	944.05286	37.14185732	6.565814794	-0.000426607	8.09275E-06
5	801.39	884.17228	82.78228399	14.86813712	-0.001110735	1.41325E-05
6	797.491	834.45462	36.96362243	6.74860327	-0.000514999	1.71075E-05
7	779.994	791.71365	11.71965344	2.176285008	-0.000172705	2.21813E-05
8	708.125	754.04459	45.91958515	8.677985901	-0.000771985	4.11787E-05
9	689.283	720.2159	30.93289774	5.953038946	-0.000554036	5.89042E-05
10	634.036	689.38379	55.34778648	10.85459397	-0.001119159	0.000143429
11	600.008	660.94338	60.93537798	12.1870756	-0.001354101	0.000441734
12	594.661	634.4449	39.78389684	8.120853937	-0.000929192	0.000797067
13	580.791	609.54343	28.75243268	5.995296652	-0.000717473	0.004240994
14	531.511	585.96729	54.45629406	11.6101209	-0.001552358	-0.001714564
15	517.1	563.49721	46.39721112	10.12470152	-0.001424219	0.001962475
16	514.985	541.95215	26.96715038	6.030038142	-0.000872749	0.000552658
17	513.346	521.1793	7.833301098	1.79708252	-0.000267707	0.000319639
18	466.856	501.04676	34.19076173	8.058839824	-0.001356226	0.000130292
19	456.725	481.438	24.71300018	5.993782944	-0.001060964	9.34046E-05
20	451.5	462.24748	10.74747686	2.686869214	-0.000495915	7.38515E-05
21	445.542	443.377	2.164996215	0.558999619	0.000107983	6.01346E-05
22	443.716	424.73252	18.98348356	5.073549391	0.001018642	5.18073E-05
23	430.985	406.22098	24.7640164	6.868302378	0.001473311	4.19503E-05
24	389.771	387.74718	2.023816378	0.584225465	0.000144231	2.93276E-05
25	339.692	369.21003	29.51803105	8.900021231	-0.002633226	2.09663E-05
26	336.988	350.49803	13.51002752	4.27224582	-0.001336351	1.89795E-05
27	317.8	331.48319	13.68319448	4.561064825	-0.001594666	1.61745E-05
28	309.272	312.01243	2.740432873	0.968889334	-0.000369205	1.44836E-05
29	283.2963	291.89445	8.598150327	3.249795357	-0.001445256	1.22453E-05
30	276.275	270.87875	5.396254681	2.203011749	0.001085121	1.11235E-05
31	265.71	248.6195	17.09050379	7.643105649	0.004288009	1.00043E-05
32	262.8862	224.60838	38.27782185	19.13891093	0.012133838	9.22914E-06
33	231.365	198.03611	33.32888915	19.24244312	0.016005921	7.78627E-06
34	218.948	167.46361	51.48439403	36.40496415	0.039190732	6.94696E-06
35	213.14	129.85069	83.28930816	83.28930816	0.13025759	6.25432E-06
36	198.744	76.268604	122.4753956	#DIV/0!	#DIV/0!	5.36507E-06
Mean	574.44571	572.38298				

Log Pearson-III						
S/N	Obesrved,X	Expected,Y	Max.Absolute Error(MAE	RMSE(rootmeansquare erro	RRMSE(relative rootmeansquare erro	Correlation Coefficient(CC)
1	3.207931	3.351654	0.143722894	0.024293603	-0.000426689	3.672783124
2	3.083258	3.222495	0.139236464	0.023878857	-0.000442734	6.412429174
3	3.009246	3.145819	0.136573102	0.023774326	-0.000458429	9.79195154
4	2.955938	3.090592	0.134654768	0.023803825	-0.000474521	14.20664557
5	2.913942	3.047085	0.133143526	0.023913283	-0.000491311	20.18756082
6	2.879074	3.010962	0.131888771	0.024079485	-0.000508994	28.57368224
7	2.849098	2.979908	0.130810091	0.024290824	-0.000527733	40.79505527
8	2.82268	2.952539	0.129859414	0.024541123	-0.000547687	59.46754
9	2.798955	2.927961	0.129005659	0.024827151	-0.00056902	89.79744253
10	2.777332	2.905559	0.128227531	0.025147488	-0.000591914	143.3601842
11	2.757386	2.884895	0.127509763	0.025501953	-0.000616573	250.2229476
12	2.738802	2.865643	0.126841005	0.025891312	-0.00064323	509.5137093
13	2.721337	2.84755	0.126212552	0.026317136	-0.000672158	1421.1641
14	2.704803	2.83042	0.125617546	0.026781751	-0.000703673	10634.83331
15	2.689044	2.814095	0.125050455	0.027288247	-0.000738154	24855.15685
16	2.673934	2.798441	0.12450671	0.027840547	-0.000776052	2119.2333
17	2.659365	2.783348	0.123982453	0.028443525	-0.000817913	748.5210073
18	2.645246	2.76872	0.123474356	0.029103185	-0.000864404	385.6436165
19	2.631494	2.754473	0.122979478	0.029826904	-0.000916347	237.2732851
20	2.618035	2.74053	0.122495155	0.030623789	-0.00097477	161.7899797
21	2.604801	2.72682	0.122018909	0.031505147	-0.001040975	117.9197742
22	2.591725	2.713273	0.121548367	0.032485168	-0.001116634	90.01951442
23	2.578742	2.699823	0.12108118	0.033581877	-0.001203938	71.07925887
24	2.565786	2.686401	0.120614946	0.034818536	-0.001305805	57.56711757
25	2.552785	2.672932	0.120147113	0.036225717	-0.001426215	47.54167983
26	2.539662	2.659337	0.119674866	0.037844516	-0.001570745	39.8612202
27	2.526327	2.645522	0.119194978	0.039731659	-0.00174745	33.81687428
28	2.512671	2.631375	0.118703582	0.041968054	-0.001968416	28.94826753
29	2.498562	2.616758	0.118195853	0.044673833	-0.002252645	24.94457314
30	2.483823	2.601489	0.117665467	0.048036726	-0.002631818	21.58835024
31	2.468212	2.585316	0.117103697	0.052370365	-0.003162983	18.72192746
32	2.451372	2.56787	0.116497713	0.058248857	-0.003960289	16.22596604
33	2.432737	2.548564	0.115827093	0.066872803	-0.005290206	14.00418618
34	2.411295	2.526351	0.115055516	0.081356536	-0.007952539	11.96931157
35	2.384916	2.499023	0.114106257	0.114106257	-0.015948324	10.02112222
36	2.347338	2.460092	0.112753973	#DIV/0!	#DIV/0!	7.973383661
Mean	2.695276	2.820551				42351.81991

GUMBEL		Best fit	
RMSE	316.2611223	lowest	1.01166
RRMSE	0.00186621	lowest	-0.0362
CC	0.007515781	highest	42321.9
MAE	1572.270741	lowest	4.13407

Tendaho Station

Lognormal

Rank	Obesrved,X	Expected,Y	MAE	RMSE	RRMSE	CC
1	3.227156	2.77410	0.45305	0.076580084	0.02373	1
2	3.152270	2.76218	0.39009	0.102530727	0.044953	0.99995
3	2.987487	2.75433	0.23316	0.111707242	0.058539	0.99957
4	2.957565	2.74822	0.20934	0.119322936	0.071051	0.99945
5	2.903844	2.74311	0.16073	0.124622066	0.080993	0.99933
6	2.901726	2.73863	0.16309	0.130134551	0.091255	0.99932
7	2.892091	2.73460	0.1575	0.135551844	0.101367	0.99933
8	2.850110	2.73088	0.11923	0.139779172	0.109273	0.9993
9	2.838398	2.72741	0.11099	0.143937783	0.116798	0.99928
10	2.802114	2.72412	0.07799	0.147474993	0.122256	0.99924
11	2.778157	2.72098	0.05717	0.150829653	0.126372	0.9992
12	2.774269	2.71795	0.05632	0.154368476	0.130516	0.99917
13	2.764020	2.71501	0.04901	0.158019366	0.134213	0.99916
14	2.725512	2.71214	0.01338	0.161595969	0.135259	0.99911
15	2.713575	2.70931	0.00426	0.165401366	0.135602	0.99907
16	2.711795	2.70652	0.00528	0.169490077	0.136037	0.99905
17	2.710410	2.70374	0.00667	0.173899886	0.136602	0.99904
18	2.669183	2.70097	0.03178	0.178822141	0.139409	0.999
19	2.659655	2.70097	0.04131	0.184279059	0.143176	0.99896
20	2.654658	2.70374	0.04908	0.190346401	0.147798	0.99893
21	2.648889	2.70652	0.05763	0.197151183	0.153415	0.99889
22	2.647105	2.70931	0.0622	0.204746927	0.159696	0.99887
23	2.634462	2.71214	0.07767	0.213565273	0.167873	0.99883
24	2.590810	2.71501	0.1242	0.225158834	0.181712	0.99876
25	2.531085	2.71795	0.18687	0.241825994	0.203973	0.99863
26	2.527614	2.72098	0.19337	0.260896385	0.228165	0.99852
27	2.502154	2.72412	0.22197	0.284788424	0.257735	0.99838
28	2.490341	2.72741	0.23707	0.313476861	0.291392	0.99825
29	2.452241	2.73088	0.27864	0.351279863	0.334339	0.99808
30	2.441342	2.73460	0.29325	0.397865346	0.383378	0.99791
31	2.424408	2.73863	0.31422	0.457934003	0.441341	0.99774
32	2.419768	2.74311	0.32334	0.536904896	0.508153	0.99757
33	2.364298	2.74822	0.38393	0.658398541	0.601906	0.99735
34	2.340341	2.75433	0.41398	0.857860468	0.726986	0.9971
35	2.328665	2.76218	0.43352	1.288326692	0.913152	0.99685
36	2.298294	2.77410	0.47581	#DIV/0!	#DIV/0!	0.99656
Mean	2.675439125	2.72912232				

Normal distrn.

Rank	Obesrved,X	Expected,Y	MAE	RMSE	RRMSE	CC
1	1687.16	684.779	1002.381	169.4333	0.100425	1
2	1419.939	667.782	752.1574	214.9219	0.19127	0.99732
3	971.598	656.580	315.0178	224.9407	0.24771	0.980625
4	906.911	647.878	259.0327	232.9728	0.298202	0.973037
5	801.39	640.587	160.8028	238.456	0.33424	0.965957
6	797.491	634.203	163.2879	244.2241	0.371623	0.962997
7	779.994	628.449	151.5455	249.9882	0.407701	0.961549
8	708.125	623.153	84.97232	254.9194	0.430379	0.958953
9	689.283	618.202	71.08088	259.9573	0.450225	0.957136
10	634.036	613.517	20.51916	264.9399	0.456572	0.954567
11	600.008	609.038	9.029672	270.1928	0.459581	0.951952
12	594.661	604.719	10.05809	275.772	0.463034	0.950125
13	580.791	600.525	19.73377	281.7333	0.470119	0.948618
14	531.511	596.425	64.91375	288.3975	0.496157	0.946299
15	517.1	592.393	75.29348	295.6411	0.527931	0.944225
16	514.985	588.409	73.42354	303.3866	0.559812	0.942666
17	513.346	584.450	71.10354	311.6952	0.591588	0.941527
18	466.856	580.497	113.6414	321.3546	0.648962	0.939585
19	456.725	580.497	123.7724	332.031	0.714689	0.937717
20	451.5	584.450	132.9495	343.8599	0.788305	0.935927
21	445.542	588.409	142.8665	357.0476	0.871098	0.934182
22	443.716	592.393	148.6775	371.7093	0.960651	0.932575
23	430.985	596.425	165.4397	388.4604	1.067115	0.930817
24	389.771	600.525	210.7538	408.8742	1.223205	0.92815
25	339.692	604.719	265.0271	434.4669	1.458444	0.92426
26	336.988	609.038	272.0497	463.7227	1.713734	0.920558
27	317.8	613.517	295.7168	498.6466	2.023905	0.916524
28	309.272	618.202	308.9301	540.0548	2.377067	0.912476
29	283.2963	623.153	339.8564	591.4602	2.830492	0.907849
30	276.275	628.449	352.1735	654.8285	3.350895	0.903238
31	265.71	634.203	368.4931	736.0148	3.971102	0.898507
32	262.8862	640.587	377.701	844.2818	4.689475	0.893886
33	231.365	647.878	416.5133	1004.113	5.728846	0.888393
34	218.948	656.580	437.6322	1268.119	7.142208	0.882628
35	213.14	667.782	454.6416	1850.122	9.275273	0.876752
36	198.744	684.779	486.0353	#DIV/0!	#DIV/0!	0.870298
Mean	544.0983472	620.6436167				

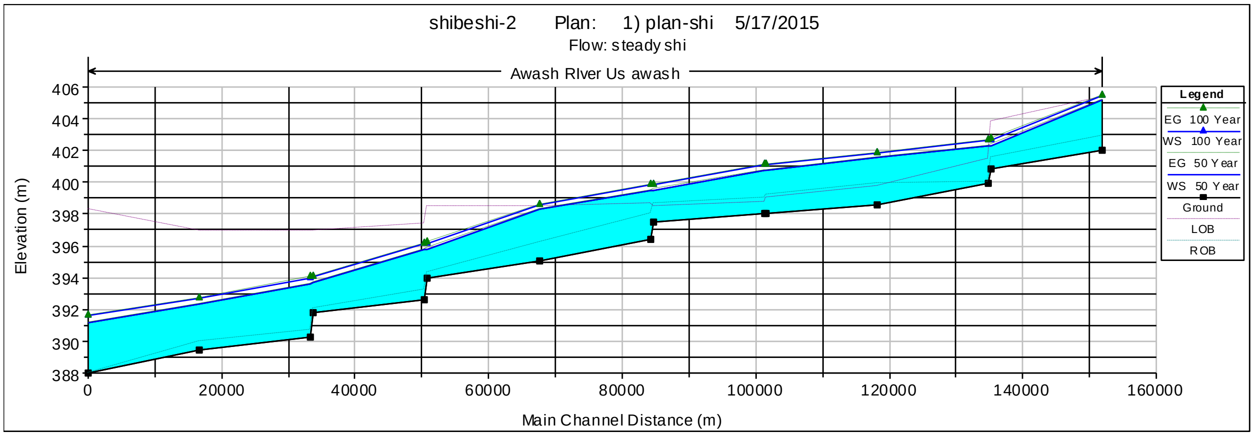
lognormal		Normal distn.	Best fit
RMSE	9.408873483	15790.7	lowest
RRMSE	7.738412491	57.692	lowest
CC	35.95376189	33.6719	highest
MAE	6.46712005	8717.22	lowest

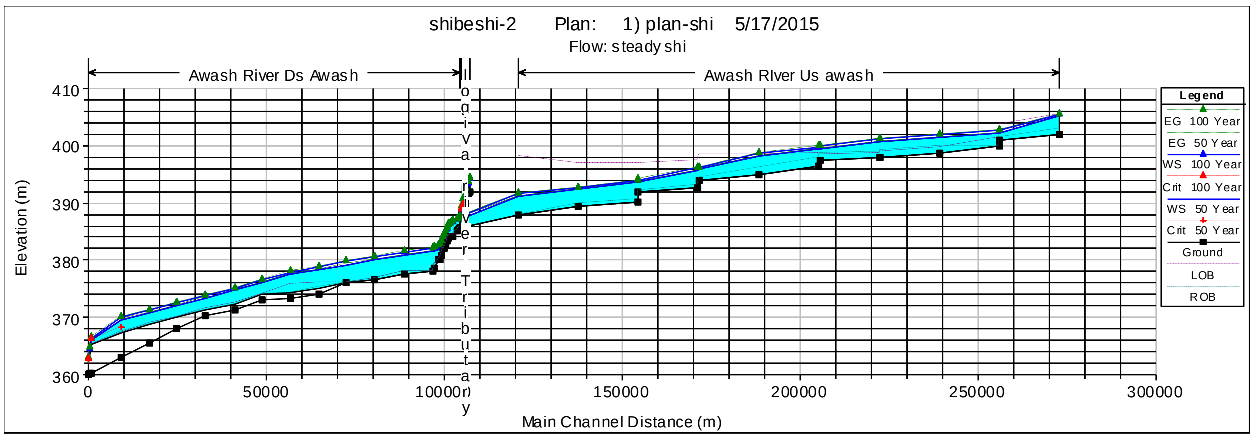
Appendix B: Hec_Ras Model Out put

a. Summary table

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)	
Ds Awash	16359.73	50 Year	3356.00	385.63	387.24		387.29	0.000665	1.12	3845.46	3312.81	0.29
Ds Awash	16359.73	100 Year	4764.00	385.63	387.56		387.62	0.000629	1.24	4960.66	3487.52	0.29
Ds Awash	15619.45	50 Year	3356.00	385.31	387.07		387.12	0.000596	1.13	4010.16	3368.41	0.28
Ds Awash	15619.45	100 Year	4764.00	385.31	387.41		387.46	0.000572	1.25	5169.66	3551.76	0.28
Ds Awash	15427.41	50 Year	3356.00	385.03	386.81		386.86	0.000719	1.26	3818.59	3381.78	0.31
Ds Awash	15427.41	100 Year	4764.00	385.03	387.18		387.23	0.000627	1.35	5098.00	3581.81	0.30
Ds Awash	13506.36	50 Year	3356.00	384.00	386.31		386.37	0.000375	1.10	3870.81	2821.99	0.24
Ds Awash	13506.36	100 Year	4764.00	384.00	386.71		386.77	0.000384	1.24	5039.92	3085.99	0.24
Ds Awash	13041.48	50 Year	3356.00	384.00	386.19		386.24	0.000383	1.08	3714.43	2686.29	0.24
Ds Awash	13041.48	100 Year	4764.00	384.00	386.58		386.64	0.000394	1.22	4804.17	2884.00	0.25
Ds Awash	12655.31	50 Year	3356.00	384.00	386.03		386.08	0.000399	1.06	3631.15	2449.83	0.24
Ds Awash	12655.31	100 Year	4764.00	384.00	386.41		386.48	0.000417	1.22	4602.11	2622.76	0.25
Ds Awash	12349.92	50 Year	3356.00	384.00	385.88		385.94	0.000588	1.23	3477.36	2319.67	0.29
Ds Awash	12349.92	100 Year	4764.00	384.00	386.25		386.33	0.000625	1.43	4399.17	2604.30	0.30
Ds Awash	12127.56	50 Year	3356.00	383.78	385.67		385.74	0.001014	1.56	3047.68	2184.47	0.37
Ds Awash	12127.56	100 Year	4764.00	383.78	386.02		386.12	0.001090	1.83	3848.17	2450.26	0.40
Ds Awash	12006.18	50 Year	3356.00	383.71	385.44		385.53	0.001242	1.55	2772.73	2157.67	0.40
Ds Awash	12006.18	100 Year	4764.00	383.71	385.79		385.90	0.001203	1.75	3551.40	2307.40	0.41
Ds Awash	11726.75	50 Year	3356.00	383.32	385.07		385.18	0.001279	1.67	2596.73	2038.17	0.41
Ds Awash	11726.75	100 Year	4764.00	383.32	385.43		385.56	0.001227	1.86	3353.26	2182.09	0.42
Ds Awash	11470.17	50 Year	3356.00	382.66	384.73		384.85	0.001260	1.87	2551.08	1915.90	0.42
Ds Awash	11470.17	100 Year	4764.00	382.66	385.09		385.24	0.001269	2.09	3269.36	2062.06	0.44
Ds Awash	11121.68	50 Year	3356.00	382.00	384.15		384.33	0.001580	2.21	2248.27	1853.78	0.48
Ds Awash	11121.68	100 Year	4764.00	382.00	384.51		384.72	0.001602	2.46	2951.29	2086.48	0.50
Ds Awash	5032.709	50 Year	3356.00	374.00	376.17		376.20	0.000281	0.93	5372.45	4314.04	0.20
Ds Awash	5032.709	100 Year	4764.00	374.00	376.54		376.58	0.000270	1.02	7012.31	4427.32	0.20
Ds Awash	4356.017	50 Year	3356.00	372.26	374.75		374.77	0.000156	0.74	7390.17	5268.42	0.15
Ds Awash	4356.017	100 Year	4764.00	372.26	375.12		375.14	0.000161	0.83	9423.71	5701.10	0.16
Ds Awash	3711.105	50 Year	3356.00	371.22	373.43		373.45	0.000204	0.77	6928.43	5303.44	0.17
Ds Awash	3711.105	100 Year	4764.00	371.22	373.79		373.81	0.000204	0.85	8907.41	5775.50	0.18
Ds Awash	2872.015	50 Year	3356.00	369.98	372.16		372.18	0.000148	0.67	7347.62	5586.92	0.15
Ds Awash	2872.015	100 Year	4764.00	369.98	372.52		372.54	0.000149	0.74	9406.66	5905.13	0.15
Ds Awash	2363.341	50 Year	3356.00	368.73	370.85		370.87	0.000238	0.82	6223.78	4718.19	0.18
Ds Awash	2363.341	100 Year	4764.00	368.73	371.26		371.28	0.000220	0.89	8229.85	5195.96	0.18
Ds Awash	1860.235	50 Year	3356.00	367.23	369.67	368.27	369.68	0.000142	0.71	7287.42	4659.40	0.15
Ds Awash	1860.235	100 Year	4764.00	367.23	370.07		370.09	0.000152	0.82	9233.46	5168.36	0.16
Ds Awash	1335.703	50 Year	3356.00	365.27	366.00	365.97	366.28	0.012399	2.92	1528.85	2508.92	1.12
Ds Awash	1335.703	100 Year	4764.00	365.27	366.19	366.17	366.51	0.011136	3.25	2042.38	2832.02	1.10
Ds Awash	856.3001	50 Year	3356.00	363.48	364.41		364.47	0.001710	1.27	2992.42	3171.76	0.43
Ds Awash	856.3001	100 Year	4764.00	363.48	364.64		364.73	0.001742	1.50	3756.55	3334.04	0.45
Ds Awash	342.0045	50 Year	3356.00	362.00	362.69	362.60	362.90	0.008012	2.34	1757.63	2740.65	0.90
Ds Awash	342.0045	100 Year	4764.00	362.00	362.85	362.75	363.12	0.008008	2.68	2197.83	2834.73	0.93

b. Water surface profile





Appendix C: Decision Analysis (TOPSIS)

	Scenarios	Flood Risk -Oriented				Enviromental -Oriented				Balance			
	Weight	0.334	0.334	0.167	0.167	0.167	0.167	0.334	0.334	0.25	0.25	0.25	0.25
		Inundation Area(ha)	Flood Level(m)	Flow velocity(m/s)	Stream power(Nm/s	Inundation Area(ha)	Flood Level(m)	Flow velocity(m/s)	Stream power(Nm/s	Inundation Area(ha)	Flood Level(m)	Flow velocity(m/s)	Stream power(Nm/s
	Alt#1	40670	363.7	1.69	65.8	40670	363.7	1.69	65.8	40670	363.7	1.69	65.8
	Alt#2	43650	363.56	1.5	19.84	43650	363.56	1.5	19.84	43650	363.56	1.5	19.84
	Alt#3	38790	362.95	2.46	51.35	38790	362.95	2.46	51.35	38790	362.95	2.46	51.35
	Alt#4	41050	363.03	2.34	46.49	41050	363.03	2.34	46.49	41050	363.03	2.34	46.49
Step(1a)		Calculate(∑X^2ij)^1/2 for each column											
		Inundation Area(ha)	Flood Level(m)	Flow velocity(m/s)	Stream power(Nm/s	Inundation Area(ha)	Flood Level(m)	Flow velocity(m/s)	Stream power(Nm/s	Inundation Area(ha)	Flood Level(m)	Flow velocity(m/s)	Stream power(Nm/s
	Alt#1	1654048900	132277.7	2.8561	4329.64	1654048900	132277.69	2.8561	4329.64	1654048900	132277.69	2.8561	4329.64
	Alt#2	1905322500	132175.9	2.25	393.6256	1905322500	132175.87	2.25	393.6256	1905322500	132175.874	2.25	393.6256
	Alt#3	1504664100	131732.7	6.0516	2636.8225	1504664100	131732.7	6.0516	2636.8225	1504664100	131732.703	6.0516	2636.8225
	Alt#4	1685102500	131790.8	5.4756	2161.3201	1685102500	131790.78	5.4756	2161.3201	1685102500	131790.781	5.4756	2161.3201
	∑Xij^2	6749138000	527977	16.6333	9521.4082	6749138000	527977.05	16.6333	9521.4082	6749138000	527977.047	16.6333	9521.4082
	(∑Xij^2)^1/2	82153.13749	726.62	4.07839429	97.5777034	82153.1375	726.6203	4.07839429	97.5777034	82153.1375	726.62029	4.07839429	97.5777034
Step 1(b)		Divide each column by (∑X^2ij)^1/2 to get rij											
		Inundation Area(ha)	Flood Level(m)	Flow velocity(m/s)	Stream power(Nm/s	Inundation Area(ha)	Flood Level(m)	Flow velocity(m/s)	Stream power(Nm/s	Inundation Area(ha)	Flood Level(m)	Flow velocity(m/s)	Stream power(Nm/s
	Alt#1	0.495051087	0.500537	0.414378767	0.674334379	0.495051087	0.0027495	0.591715976	0.674334379	0.495051087	0.50053653	0.414378767	0.674334379
	Alt#2	0.531324808	0.500344	0.367791805	0.203325138	0.531324808	0.0027485	0.525191695	0.203325138	0.531324808	0.50034386	0.367791805	0.203325138
	Alt#3	0.472166994	0.499504	0.603178561	0.52624727	0.472166994	0.0027438	0.86131438	0.52624727	0.472166994	0.49950436	0.603178561	0.52624727
	Alt#4	0.499676595	0.499614	0.573755217	0.47644081	0.499676595	0.0027445	0.819299044	0.47644081	0.499676595	0.49961445	0.573755217	0.47644081
Step 2(a)		Multiply each column by Wj to get Vij											
		Inundation Area(ha)	Flood Level(m)	Flow velocity(m/s)	Stream power(Nm/s	Inundation Area(ha)	Flood Level(m)	Flow velocity(m/s)	Stream power(Nm/s	Inundation Area(ha)	Flood Level(m)	Flow velocity(m/s)	Stream power(Nm/s
	Alt#1	0.165347063	0.167179	0.069201254	0.112613841	0.082673531	0.0004592	0.197633136	0.225227683	0.123762772	0.12513413	0.103594692	0.168583595
	Alt#2	0.177462486	0.167115	0.061421232	0.033955298	0.088731243	0.000459	0.175414026	0.067910596	0.132831202	0.12508596	0.091947951	0.050831284
	Alt#3	0.157703776	0.166834	0.10073082	0.087883294	0.078851888	0.0004582	0.287679003	0.175766588	0.118041749	0.12487609	0.15079464	0.131561817
	Alt#4	0.166891983	0.166871	0.095817121	0.079565615	0.083445991	0.0004583	0.273645881	0.15913123	0.124919149	0.12490361	0.143438804	0.119110202
Step 3(a)		Determine ideal solution A* .											
		Inundation Area(ha)	Flood Level(m)	Flow velocity(m/s)	Stream power(Nm/s	Inundation Area(ha)	Flood Level(m)	Flow velocity(m/s)	Stream power(Nm/s	Inundation Area(ha)	Flood Level(m)	Flow velocity(m/s)	Stream power(Nm/s
	Alt#1	0.165347063	0.167179	0.069201254	0.112613841	0.082673531	0.0004592	0.197633136	0.225227683	0.123762772	0.12513413	0.103594692	0.168583595
	Alt#2	0.177462486	0.167115	0.061421232	0.033955298	0.088731243	0.000459	0.175414026	0.067910596	0.132831202	0.12508596	0.091947951	0.050831284
	Alt#3	0.157703776	0.166834	0.10073082	0.087883294	0.078851888	0.0004582	0.287679003	0.175766588	0.118041749	0.12487609	0.15079464	0.131561817
	Alt#4	0.166891983	0.166871	0.095817121	0.079565615	0.083445991	0.0004583	0.273645881	0.15913123	0.124919149	0.12490361	0.143438804	0.119110202
	A*	0.157703776	0.16683	0.06142123	0.033955298	0.07885189	0.000458	0.17541403	0.067910596	0.11804175	0.1248761	0.09194795	0.05083128
		Find the negative ideal solution A'.											
		Inundation Area(ha)	Flood Level(m)	Flow velocity(m/s)	Stream power(Nm/s	Inundation Area(ha)	Flood Level(m)	Flow velocity(m/s)	Stream power(Nm/s	Inundation Area(ha)	Flood Level(m)	Flow velocity(m/s)	Stream power(Nm/s
	Alt#1	0.165347063	0.167179	0.069201254	0.112613841	0.082673531	0.0004592	0.197633136	0.225227683	0.123762772	0.12513413	0.103594692	0.168583595
	Alt#2	0.177462486	0.167115	0.061421232	0.033955298	0.088731243	0.000459	0.175414026	0.067910596	0.132831202	0.12508596	0.091947951	0.050831284
	Alt#3	0.157703776	0.166834	0.10073082	0.087883294	0.078851888	0.0004582	0.287679003	0.175766588	0.118041749	0.12487609	0.15079464	0.131561817
	Alt#4	0.166891983	0.166871	0.095817121	0.079565615	0.083445991	0.0004583	0.273645881	0.15913123	0.124919149	0.12490361	0.143438804	0.119110202
	A'	0.177462486	0.16718	0.10073082	0.112613841	0.08873124	0.000459	0.287679	0.225227683	0.1328312	0.1251341	0.15079464	0.16858359

step (4a) Determine the separation from ideal solution A* .S*I=[∑(Vj*-Vij)^2]^1/2 for each row

	Inundation Area(ha)	Flood Level(m)	Flow velocity(m/s)	Stream power(Nm/s	Inundation Area(ha)	Flood Level(m)	Flow velocity(m/s)	Stream power(Nm/s	Inundation Area(ha)	Flood Level(m)	Flow velocity(m/s)	Stream power(Nm/s
Alt#1	5.84198E-05	1.19E-07	6.05288E-05	0.006187166	1.4605E-05	8.966E-13	0.000493689	0.024748666	3.27301E-05	6.6587E-08	0.000135647	0.013865607
Alt#2	0.000390407	7.86E-08	0	0	9.76017E-05	5.931E-13	0	0	0.000218728	4.4048E-08	0	0
Alt#3	0	0	0.001545244	0.002908229	0	0	0.012603425	0.011632915	0	0	0.003462933	0.006517419
Alt#4	8.44231E-05	1.35E-09	0.001183077	0.002080301	2.11058E-05	1.02E-14	0.009649497	0.008321204	4.72986E-05	7.5761E-10	0.002651308	0.004662011

Determine the separation from ideal solution Si*

	∑(Vj*-Vij)^2	Si*=[∑(Vj*-Vij)^2]^1/			∑(Vj*-Vij)^	Si*=[∑(Vj*-Vij)^2]^1/2			∑(Vj*-Vij)^2	Si*=[∑(Vj*-Vij)^2]^1/2		
Alt#1	0.006306234	0.079412			0.025256959	0.1589244			0.01403405	0.1184654		
Alt#2	0.000390485	0.019761			9.76017E-05	0.0098794			0.000218772	0.01479094		
Alt#3	0.004453472	0.066734			0.02423634	0.1556802			0.009980352	0.09990171		
Alt#4	0.003347803	0.05786			0.017991807	0.1341335			0.007360618	0.08579404		

step 4(b) Find the separation from negative ideal solution A', Si'=[∑(Vj*-Vij)^2]^1/2 for each row

	Inundation Area(ha)	Flood Level(m)	Flow velocity(m/s)	Stream power(Nm/s	Inundation Area(ha)	Flood Level(m)	Flow velocity(m/s)	Stream power(Nm/s	Inundation Area(ha)	Flood Level(m)	Flow velocity(m/s)	Stream power(Nm/s
Alt#1	0.000146783	0	0.000994114	0	0.006057711	0	0.090045867	0	0.00906843	0	0.047199948	0
Alt#2	0	4.14E-09	0.001545244	0.006187166	0	1.767E-07	0.112264977	0.157317086	0	4.8168E-05	0.058846689	0.11775231
Alt#3	0.000390407	1.19E-07	0	0.0006116	0.009879355	9.469E-07	0	0.049461094	0.014789453	0.00025804	0	0.037021777
Alt#4	0.000111736	9.48E-08	2.41444E-05	0.001092185	0.005285252	8.459E-07	0.014033122	0.066096452	0.007912053	0.00023052	0.007355836	0.049473392

Determine separation from negative ideal solution Si'

	∑(Vj'-Vij)^2	Si'=[∑(Vj*-Vij)^2]^1/			∑(Vj'-Vij)^2	Si'=[∑(Vj*-Vij)^2]^1/2			∑(Vj'-Vij)^2	Si'=[∑(Vj*-Vij)^2]^1/2		
Alt#1	0.001140897	0.033777			0.096103578	0.3100058			0.056268379	0.23720957		
Alt#2	0.007732414	0.087934			0.26958224	0.5192131			0.176647167	0.42029414		
Alt#3	0.001002125	0.031656			0.059341396	0.2436009			0.052069275	0.22818693		
Alt#4	0.00122816	0.035045			0.085415672	0.2922596			0.064971801	0.25489567		

step 5 Calculate the relative closeness to the ideal solution Ci*=Si'/(Si*+Si')

	CI*=SI'/(SI*+SI')	Rank			CI*=SI'/(SI*+S	Rank			CI*=SI'/(SI*+S	Rank		
Alt#1	0.701586049	1	*BEST*		0.338908431	2			0.333072068	1		
Alt#2	0.183487881	4			0.018672266	4			0.033995517	4		
Alt#3	0.678258592	2			0.38990133	1			0.304496098	2		
Alt#4	0.622786761	3			0.314577159	2			0.251824581	3		