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DECHATU CATCHMENT (Dire Dawa Town) FLOOD STUDY

By

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DECHATU CATCHMENT

DIRE DAWA TOWN

FLOOD STUDY

By

Tesema Habte

A thesis submitted to the School of Graduate Studies of Addis Ababa University in partial fulfillment of the requirements for the degree of Master of Science in Hydraulics Engineering

Addis Ababa University

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DECLARATION AND COPYRIGHT

Tesema Habte, declares that this thesis is my own original work and that it has not been presented and will not be presented by me to any other University for similar or any other degree award.

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I would like to express my wholehearted gratitude to my advisors, Dr. Ing. Zelalem Hailu, for his priceless support in supervising my work and providing me important reference materials including HEC-HMS example also Dr. Solomon Seyoum and Dr. Mekuria for their all advice and comments.

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The Ethiopian Mapping Authority also gratefully acknowledged since they helped me in providing 1:50000 scale maps of the target area.

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Abstract

Nowadays, extraordinary floods are common to many parts of Ethiopia every year causing a lot of losses to human lives as well as damage to property. However, the researches and studies related to this problem are not as intensive as the damage.

Historically, Dire Dawa has been vulnerable to flash flooding from rainfall, in particular of the ungaged Dechatu catchment which passes through the centre of the city. The tributary rivers are originating from the southern highlands of Kersa, Lange and Dengego catchments.

Some organizations have tried to estimate the amount of flood and recommended the mitigation measures previously.

Traditional flood design methods are increasingly supplemented or replaced by risk-oriented methods which are based on comprehensive risk analysis. Besides meteorological, hydrological and hydraulic investigations such analyses require the estimation of flood impacts.

Hence, this thesis is to identify peak flood and delineate risk areas that can be affected by extraordinary floods and to recommend mitigation measures.

This thesis tries to consider more options and fills the gaps not covered by others adopting more than seven application softwares like GIS, ArcView, Global Mapper, HEC-HMS, HEC-GeoHMS, HEC-RAS and HEC-GeoRAS. Specially, delineation of the flood area and recommended mitigation measures selection method believed to be the gap not properly covered in the previous Dire Dawa flood studies which I have seen. The data usage for this thesis tried to make very intensive by considering different data options like gridded (DEM and contour), digitized (soil, land use and 1:50,000 scale map) and text (rainfall and discharge) type.

For precipitation modelling, ERA Intensity-frequency-duration curve is used for frequency storm and for the gage weights annual maximum daily rainfall for 24 hours and 6 hours duration storm are used since the concern is flood that uses more than one nearby metrological stations.

For geographic and terrain data, 57m resolution DEM for catchment, 4m contour interval for the town part and 0.5m contour interval for the Dechatu Stream cross section are used.

In this thesis, the flood magnitude estimated, flood area delineated and mitigation measures selection method recommended. The potential vulnerable areas for the 50 years return period flood are Kefira, Gende Gurage, Dechatu Kebele, Coca Cola factory areas, Ashewa Tera, Number one and Half-Cat.

Dire Dawa Administrative Council (DDAC), Organizations, engineers and planners who are involved in the study and management of the Dire-Dawa town flood are the direct beneficiaries. Others who are engaged in similar flood related problems believed to be benefitted from this thesis.

Now on ward, this will contribute to change the traditional trend of flood damage analysis which is found to be the outstanding problem by now.

Table of Contents

ABBREVIATIONS

DDAC	Dire Dawa Administrative Council
DD	Dire Dawa
DEM	Digital Elevation Model
E	East
EQN	Equation
ERA	Ethiopian Roads Authority
FUPI	Federal Urban Planning Institute
Geo	Geographical
GIS	Geographical Information Systems
GUI	Graphical User Interface
HEC	Hydrologic Engineering Centre
HMS	Hydrologic Modelling System
Km	Kilometre
M	Million
m	Meter
masl	meter above sea level
mm	millimetre
MoWR	Ministry of Water Resources
N	North
PET	Potential Evapo-Transpiration
RAS	River Analysis System
SCS	Soil Conservation Service
Sec	Second
U.S.	United States
UH	Unit Hydrograph
Yr	year

LIST OF TABLES

LIST OF FIGURES

1. INTRODUCTION

1.1. Description of the Project Area

1.1.1. Location

Dire Dawa town is the centre of the Dire Dawa Administrative Council (DDAC). It is located in the eastern part of Ethiopia within the eastern margin of Awash River Basin. About 69% of the total DDAC population (287,000) lives in the city. The railway from Djibouti to Addis passes through the city and its origin was as a railway town. The town lies 515 km east of Addis Ababa and journey time is about eight to ten hours by road or 45 minutes in air.

The Dire Dawa Administrative Council (DDAC) covers 1,333km^2. It is bounded by Oromia Regional State to the south and by Somali Regional State to the north, east and west.

The Dire Dawa basins include the catchments of the drainage lines that pass through Dire Dawa city. These areas composed of four watersheds namely; Lege Goro, Lege Butugi, Lege Dechatu and Lege Hare. In the south and east the watersheds include parts of Kersa and Haromya weredas of Oromiya Region. Geographically, the area stretches from 41^0 47'00" (800,182.5m) to 420 06'30" (835,233.8m) E longitude and 90 26'30" (1,043,792.6m) to 90 37'20"N (1,063,753.3m) latitude.

The total Dire-Dawa area can be divided in to three major areas. Namely;

i) The Harar Plateau

It is the south and south-eastern part of the city and characterized by a chain of mountains.

ii) The Escarpment zone

It is the foot of the mountain chain, covering 45% of the land area. Which is the dominant portion of the catchment, and is highly dissected and sloping area with very limited pockets of gentle slopes just at the end of the escarpments.

iii) The Afar Depression.

It is low lying flat land accounting for 40% of the land area.

The upper catchment contributing to the DD town flooding problem is composed of the first two units.

1.1.2. Climate

The climate is warm and dry with relatively low precipitation. Rainfall is low and erratic and exhibits spatial and temporal variability. The rainfall pattern is bio-modal with April as a peak for the small rains and August for the big rains.

From the seven rainy months only in the month of July and August, the rainfall exceeds half the potential evapo-transpiration, PET (5).

Mean annual temperature ranges from 20 - 35°C and as a result potential evapo-transpiration is relatively high, 3255 mm on average. Relative humidity at 36% and 40% at elevations of 1200 and 1800 masl respectively, is relatively low.

1.1.3.Rainfall

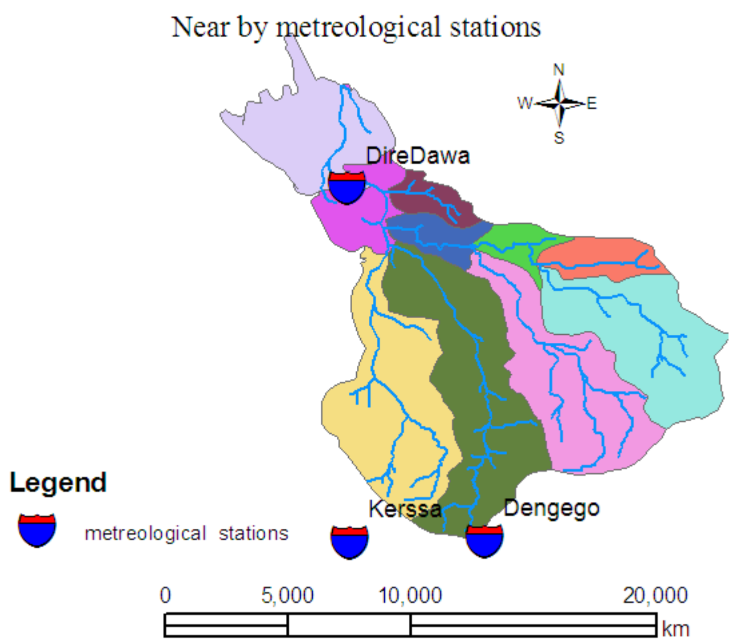
There are three important rainfall recording stations in the catchment area, Dire Dawa, Kersa, and Dengego. Dire Dawa is the centre of the flood problem at the downstream end of catchment. Dengego is located in the upper part of the Dechatu catchment near the watershed divide line and Kersa likewise in the Chirecha (Goro) catchment.

Rainfall data analysed for the Dire Dawa station covers the 52 years period from 1952 to 2003 (APPENDICES C). The Dengego rainfall record covers the period 1981 to 2005 with 3 years of missing data (1992-94). The available record is hence 22 years in length of time. The Kersa rainfall record is the shortest of the three stations with 10 years of data available from 1996 to 2005.

Rainfall intensity data are not available for any of the stations.

The highest maximum daily rainfall recorded at any of the above stations was at Kersa where an amount of 166 mm was measured on April 15, 2001. Very little or no rainfall was measured at the other two stations on the same date. 113 mm was recorded at Dire Dawa on December 7, 2003 during the “dry” season and again no or little rainfall is registered at the other two stations at the same date. 92.8mm was recorded at Dengego on April 23, 1981 but zero rainfall at Dire Dawa station while no information for Kersa at the same date. This clearly indicates that extreme rainfall events of this type will generally have limited aerial coverage and are relatively unrelated to the amount of annual rainfall. In addition, they do not necessarily occur during the main part of the rainy season which clearly complicates the issue of flood. Lower depth rainfall events at any particular point can have much larger areal coverage. Floods generated from such storms will also have longer durations.

Figure 1-1 Nearby metrological stations



1.2. Problem Statement and Background

Nowadays, extraordinary floods are common to many parts of Ethiopia every year causing a lot of losses to human lives as well as damage to property.

Dire Dawa is one of the towns facing such problem. The flood at Dire Dawa is as a result of flash flood from rainfall on the southern highlands.

Historically, Dire Dawa has been vulnerable to flash flooding, in particular of the Dechatu River which passes through the centre of the city.

From the past records, the last four major flood events occurred in April 1981, April 1994, in May, 2005 and in August 2006. The flood which occurred in May 2005 has caused loss of 35 human lives as well as an estimated amount of 10 million Birr damage to property.

On the night of 5/6 August 2006, the most severe and recent flood to date swept through Dire Dawa, resulting in over 256 fatalities, rendering nearly 10,000 homeless and significant damage to the flood defences, public infrastructure, housing and livelihoods. The high death toll was largely due to the fact that the flood peak occurred at night when people were asleep (6).

The four major flood causing rivers are namely Lege Hare, Dechatu, Lege Butugi and Goro/Chirecha, originate from the southern highland catchments. The city is bounded by Lege Hare in the eastern and by Goro Rivers in the west, Dechatu and Lege Butugi rivers pass through the middle of the city.

The elevation of the project area, defined as the four catchments and the city of Dire Dawa, varies from about 2,300m in the escarpment of the eastern edge of the Hararghe highlands to about 1,200m in the town. Slopes range from almost flat to over 50% in the mountains.

The catchments include the drainage lines that pass through Dire Dawa town. These catchments are located in the Awash River basin which is one of the 12 major basins in the country.

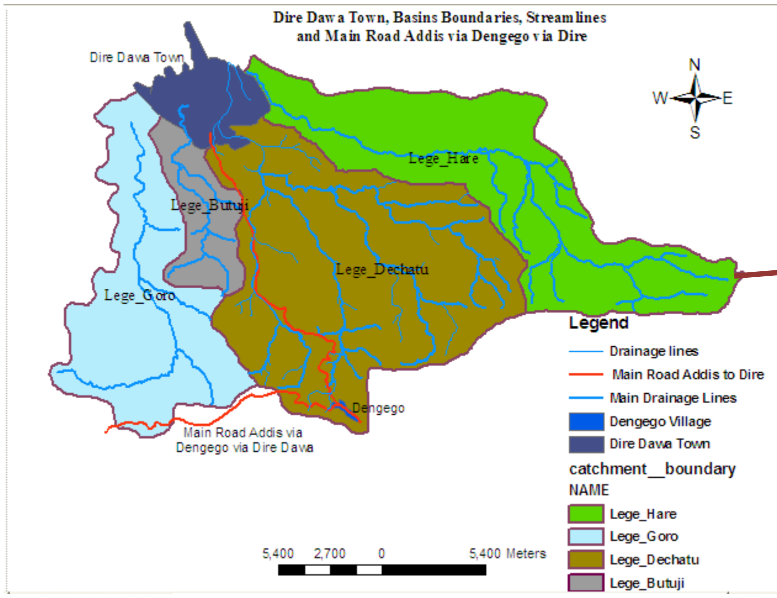
Dechatu River is the most known to cause such flood damage by crossing the Dire Dawa town. During high floods, it is considered as an enemy to the people residing or working in Dire Dawa since it causes hazards on both sides of the banks. The frequency and scale of flooding appears to be worsening due to this stream.

It is the major devastating river as it passes through the middle of the town. The tributary rivers are originating from the highlands of Kersa, Lange and Dengego catchments.

Some organizations have tried to estimate the amount of flood applying different methods and recommended the mitigation measures. Upon review of previously studied document, delineation of risk areas and flood damage analysis method are the gaps that covered by this thesis.

Through use of relevant application software it may possible to overcome the challenge of modelling ungauged flash floods.

Figure 1-2 Dire Dawa Town, Catchment Boundaries, Streamlines and Main Road Addis via Dire



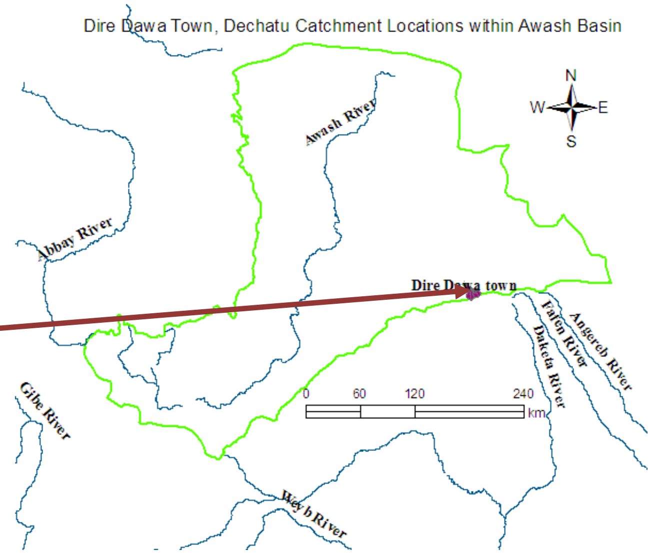


Figure 1-3 Location of Dire Dawa inside Awash

Figure 1-4 Dechatu Stream during dry season

Figure 1-5 Pictures showing small part of the night of 5/6 August 2006 damages

1.3. Objectives of the Study

1.3.1.General Objective

The general objective of the thesis is to contribute towards a study of;

- reducing risks and hazards caused by frequent and extraordinary floods.
- data intensive and multi application softwares flood analysis.

1.3.2.Specific Objective

- To estimate peak flood discharge as a result of precipitation,
- To delineate flood risk areas, and
- To recommend flood damage analysis approach to choose mitigation measures.

1.4. Outline Structure of the Dissertation

This thesis is divided into eight chapters. Chapter one provides introduction, objective, background and problem of the study area. Chapter two presents the literature review and previous study of the area. The general procedure and methodology of the research is included in chapter three. Chapter four, five and six deal with data analysis, the calibration process and the output of HEC-HMS respectively.

Chapter seven is about flood mapping while chapter eight is about recommendation mitigation measures analysis and conclusion.

2. LITERATURE REVIEW

2.1. Flood Magnitude Estimation

Many hydrologic methods are available to estimate peak flood. If possible, the method shall be calibrated to local conditions and tested for accuracy and reliability. The choice of these methods depends on data available and the practical existing situations. Among the many hydrologic methods, some are explained below.

2.1.1 Rational Method

A rational approach is to obtain the yield of a catchment by assuming a suitable runoff coefficient. It estimates the peak runoff at any location in catchment area as a function of the area, runoff coefficient, and rainfall intensity for duration equal to the time of concentration. It is best suited to urban storm drain systems and rural ditches. It shall be used with caution if the time of concentration exceeds 30 minutes. This method is used for catchment areas less than 50 hectares (0.5km2) and expressed as below.

$$Q = 0.00278CIA$$

Where:
- Q = maximum rate of runoff, m^3/sec
- C = runoff coefficient representing a ratio of runoff to rainfall
- I = average rainfall intensity for a duration equal to the time of concentration, for a selected return period, mm/h
- A = catchment area tributary to the design location, ha

2.1.2 SCS and other Unit Hydrograph Methods

The U.S. Soil Conservation Service has developed a synthetic unit hydrograph procedure that has been used widely for developing rural and urban hydrographs. The unit hydrograph used by the SCS method is based upon an analysis of a large number of natural unit hydrographs from a broad cross section of geographic locations and hydrologic regions. This method can be used for catchment areas greater than 50 hectares (0.5km2).

This technique requires the same basic data as the Rational Method: catchment area, a runoff factor, time of concentration and rainfall. The SCS approach, however, is more sophisticated in that it considers also the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm. With SCS method, the direct runoff can be calculated for any storm either real or fabricated, by subtracting infiltration and other losses from the rainfall to obtain the precipitation excess.

A relationship between accumulated rainfall and accumulated runoff was derived by SCS from experimental plots for numerous hydrologic and vegetative cover conditions. Data for land-treatment measures, such as contouring and terracing, from experimental catchment areas were included. The equation was developed mainly for small catchment areas for which daily rainfall and catchment area data are ordinarily available. It was developed from recorded storm data that included total amount of rainfall in a calendar day but not its distribution with respect to time. The SCS runoff equation is therefore a method of estimating direct runoff from 24-hours or 1-day storm rainfall.

The equation is: $$P_e = \frac{(P - I_a)^2}{P - I_a + S}$$

Where, P_e = accumulated precipitation excess at time t, mm

P = accumulated rainfall depth (potential maximum runoff) at time t, mm

I_a = the initial abstraction (initial loss) including surface storage, interception, and infiltration prior to runoff, mm

S = potential maximum retention, a measure of the ability of a watershed to abstract and retain storm precipitation.

Analysis of results from many small experimental watersheds, the SCS developed an empirical relationship of Ia and S:

$I_a = 0.2S$, Therefore, the cumulative excess at time *t* is:

$$P_e = \frac{(P-0.2S)^2}{P+0.8S}$$

The maximum retention, S, and watershed characteristics are related through an intermediate parameter, the curve number (CN) as:

$$S = \frac{25400}{CN} - 254 \qquad \text{(SI)}$$

2.1.3 Regional Regression Analysis

Peak flow can be calculated by using regression equations developed for specific geographic regions. In the equations the dependent variable would be the peak flow, and the independent variables may be area, slope, channel geometry, rainfall and other meteorological, physical or site specific data. This method shall be used for all routine designs at sites where applicable.

Regression Equations are a commonly accepted method for estimating peak flows at ungaged sites or sites with insufficient data. Also, they have been shown to be accurate, reliable, and easy to use as well as providing consistent findings. Regression equations are one of the preferred methods for estimating peak flows for larger catchment areas. A regional approach to estimating floods at ungauged sites can be adopted using this regression model to predict flood.

2.1.4 Analysis of Stream Gage Data

If a project is located near one of gauged station and the gauging record is of sufficient length, flood estimation may be made. The most important aspect of applicable station records is the series of annual peak discharges.

These methods shall preferably be used for all routine designs provided there is continuous or synthesized recorded discharge data. Example, at least 10 years of continuous or synthesized record for 10 years discharge estimates and 25 years for 100-year discharge estimates. With at least 25 years of continuous or synthesized stream gage data the log Pearson III is considered to be the most reliable method for estimating flood frequency relationships and shall be used for all designs (ERA Drainage Manual, 2001).

2.1.5 Suitable Computer Programs

Suitable Computer Programs such as HYDRAIN's HYDRO, HEC-1, HEC-HMS and others may be used to facilitate tedious hydrologic calculations.

2.2. Previous Studies of the Area

In the past, flood studies for Dire Dawa town have been done by different organizations before and after the most severe flood event occurred on the fifth day of August 2006.

Even though investigations were made in the past in the area related to this topic, due to considerable progress and invention of new approach including data type and application softwares flood studies still will continue in many aspects.

Some organizations have tried to estimate the amount of flood using different methods and recommended the mitigation measures.

Among them the followings can be mentioned:-

(i) Dire Dawa Administrative Council Integrated Resources Development Master Plan Study Project (January 2004) —

This study was conducted before the most severe flood event. It is intensive and consistent study participating Dire Dawa Administrative Council Waters, Mines & Energy Office as a client and Water Works Design & Supervision Enterprise as a consultant.

According to this study, Dechatu catchment flood estimation summary is given as follows.

Table 2-1: Summary of the Master Plan Study

Catchment	**Methods**	**Peak Flood (m^3/sec) at Various Return Periods**				
		50	**100**	**200**	**500**	**1000**
Dechatu	Rational	863	1082	1143	1330	1936
	Boldakov	792	865	913	985	1009
	SCS	96	105	141	152	156
	Creager	629	715	755	834	1032
Selected Peak		**761**	**887**	**937**	**1050**	**1326**

The design of flood protection works and spillways, the average values of the Rational, Boldakov and Creager methods are considered. Also the study states, as per the interview of the older people, a flood mark was fixed on the Dechatu river and its flood was estimated as 1170m^3/sec."

The general comment is that results from different methods shall be compared, not averaged. Usual practice is to use the discharge that best reflects local conditions.

(ii) Awash River Basin Flood Control and Watershed Management Study Project (February 2008) —

This study was conducted after the most severe flood event. The client and consultant are Ministry of Water Resources and Halcrow respectively.

This report is not consistent; the data are not commensurate/corresponding/matching and tailored to the methodology. The given data shows that average data for the whole catchment considered as one (lumped). However, the catchment is divided into three sub-catchments (distributed) to use HEC-HMS.

The following major points are not clear in the report:-

- How can we run three sub catchments (distributed) without having their specific data (lumped data)?

- The options used for each step to estimate the maximum flood discharge is not clear,
- Which Metrologic data used?

Even if these questions are not clear in the report, according to this study, Dechatu catchment flood estimation summary is given as follows.

Table 2-2: Summary of Awash River Basin Flood Control and Watershed Management Study

Return Period (years)	**Dechatu Catchment Estimated Peak Discharge** (m^3/Sec)
10	901
25	1166
50	1368
100	1582
200	1750
1000	2263

2.3. Studies outside the Area

The main intensive study indirectly related to this thesis outside the target area was done by EASTERN NILE TECHNICAL REGIONAL OFFICE, ADDIS ABABA as a client and SMEC INTERNATIONAL as a consultant. The main report is finalized in December 2006 with a title PROJECT PREPARATION, FLOOD PREPAREDNESS AND EARLY WARNING (FPEW). The project focuses on flood risk management and non-structural approaches to managing the impacts of floods: including flood plain management and flood mitigation planning; flood forecasting and warning; and emergency response and preparedness at regional, national, local and community levels. Also, the report describes the detail context of Institutional, Social, Availability of Flood Risk Related Data, Flood Forecasting, Warning, Emergency Response and Post-Flood Relief and Recover issues. For these issues, the report revealed that Ethiopia situation is far behind the two other Eastern Nile Region Countries (Sudan and Egypt). After comprehensive analysis of the situations, the report put recommendations to fill gaps and improve the situation. Since the report is big volume, it is difficult to summarize for this small pages thesis.

3. METHODOLOGY

The methodologies to attain the objectives are described below:

- Literature review,
- Data collection and analysis,
- Peak discharge estimation, and
- Flood vulnerable area delineation

3.1. Literature Review

The literature review as described above contains two parts. The first part is about the possible methods available to estimate peak discharge; while the second part contains the studies undertaken previously.

3.2. Data Collection and Analysis

Primary data from field visit and secondary data from different organizations collected. The following table describes the data type and its purpose.

Data Type	Purpose	Source
Awash basin 57m Resolution DEM	Generating catchment geo-spatial data using HEC-GeoHMS	Halcrow
Dire Dawa town 4m contour interval map	generating town part and Dechatu cross section geo-spatial data using GIS and HEC-GeoRAS	FUPI
Dechatu Stream 0.5m contour interval map	for cross checking Dechatu cross section geo-spatial data using GIS and HEC-GeoRAS	MoWR
Maximum daily Rain-fall for the stations Dire Dawa, Dengego and Kersa	to compute hydrograph using HEC-HMS	Metrology Agency
ERA Intensity-frequency-duration curve for the region	to compute hydrograph using HEC-HMS	ERA drainage manual
Soil type and land-use for Dechatu and Erer	For curve number computation	MoWR
Erer discharge	Calibration	MoWR
1:5000 scale top-map of the area	for cross checking geo-spatial data using Global Mapper	Ethiopian Mapping Authority

3.3. Estimation of Peak Flood Discharge

3.3.1 Selection of Method

Flash flood is a surface flow of short duration with a relatively high peak discharge. It is a typical example of unsteady non-uniform flow. This flash flood is common feature in the Dire Dawa Administrative region during the rainy season. It is formed as a result of intensive showers, sparse vegetation cover and steep slope of the area.

Many hydrologic methods are available to estimate peak flood as discussed in chapter two. However, some of them are applicable for this case due to the reasoning explained below.

Rational method is used for catchment areas less than 50 hectares ($0.5km^2$). Dechatu catchment area is much greater than $0.5km^2$ which is $166km^2$.

No peak flood discharge regression equation developed for the area. However, regression analysis performed in Master Plan for the Development of Surface Water Resources in the Awash Basin, volume 4, Annexes A, December 1989, page 31. The established relationship was between mean Annual flood, Mean Annual Rainfall and catchment area:

$$MAF = 0.35404A^{0.70727}(P-580)^{0.14101}$$

Where, MAF = mean annual flood in m^3/sec.

P = mean annual rainfall in mm

A_x = area of the required catchment in km^2

The development of regression analysis by itself can be a separate study and is not part of this thesis. In order for regression equations to be useful for Ethiopia, extensive study of drainage basins and hydrologic regions for the country is required.

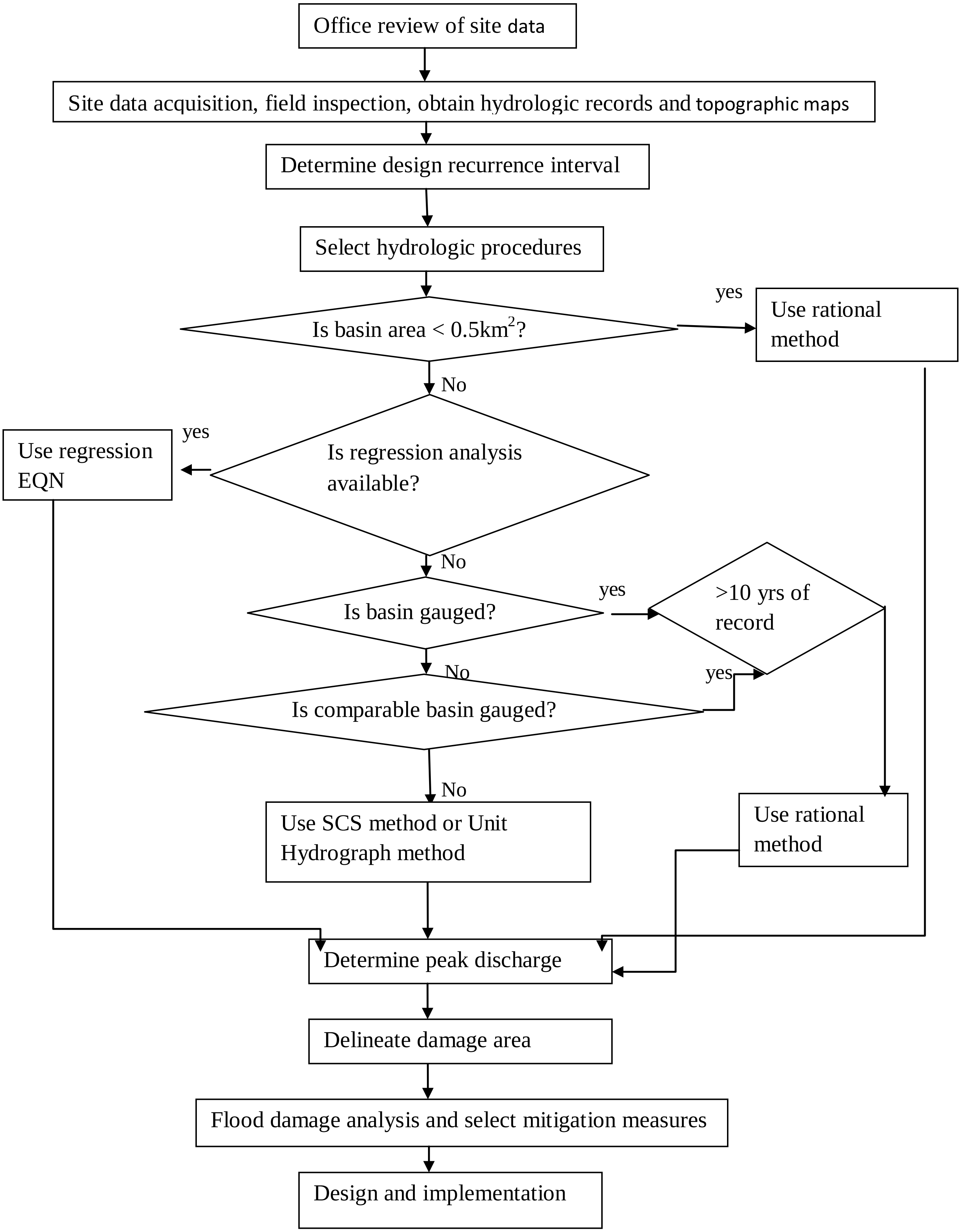
Also analysis of stream gage data is not possible since the catchment is not gauged. Hence, the preferred methodology for this study is suitable Computer Programs HEC-GeoHMS and HEC-HMS used to facilitate the SCS hydrologic calculations as well described in next chapter.

For this selected methodology, the following steps are followed:

i. Awash Basin 57m resolution DEM was cut for Dire Dawa using Grid Machine (ArcView extension)
ii. GeoHEC-HMS processed as it is a geospatial hydrology toolkit
iii. Using HEC-HMS modeled and simulated for hydrologic response

The ERA drainage Design hydrologic analysis procedure flowchart shows the steps for the hydrologic analysis and the designs that will use the hydrologic estimates. With modification for flood and the updated technology since the manual prepared, the procedure is presented below.

Figure 3-1 Hydrologic estimates procedure flowchart



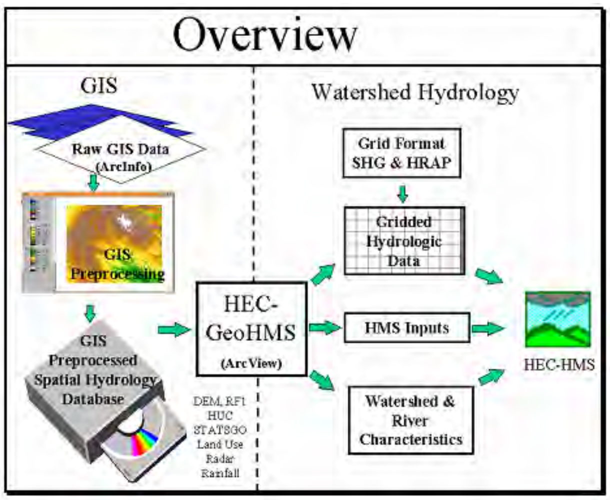
3.3.2 HEC-GeoHMS

HEC-GeoHMS has been developed as a geospatial hydrology tool kit for engineers and hydrologist. The program is an extension of ArcView and allows users to visualize spatial information, document watershed characteristics, perform spatial analysis, delineate sub-basins and streams, construct inputs to hydrologic models, and assist with report preparation. Eight data sets can be derived from DEM that collectively describe the drainage patterns of the watershed.

HEC-GeoHMS provides the connection for translating GIS spatial information into hydrologic models. The end result of the GIS processing is a spatial hydrology database that consists of the digital elevation model (DEM), soil types, land use information, rainfall, etc. HEC-GeoHMS operates on the DEM to derive sub-basin delineation and to prepare a number of hydrologic inputs. 57m resolution DEM is used for this case. HEC-HMS accepts the hydrologic inputs as a starting point for hydrologic modelling.

The relation between GIS, HEC-GeoHMS, and HEC-HMS is illustrated in Figure below.

Figure 3-2 Overview of GIS, HEC-GeoHMS and HEC-HMS



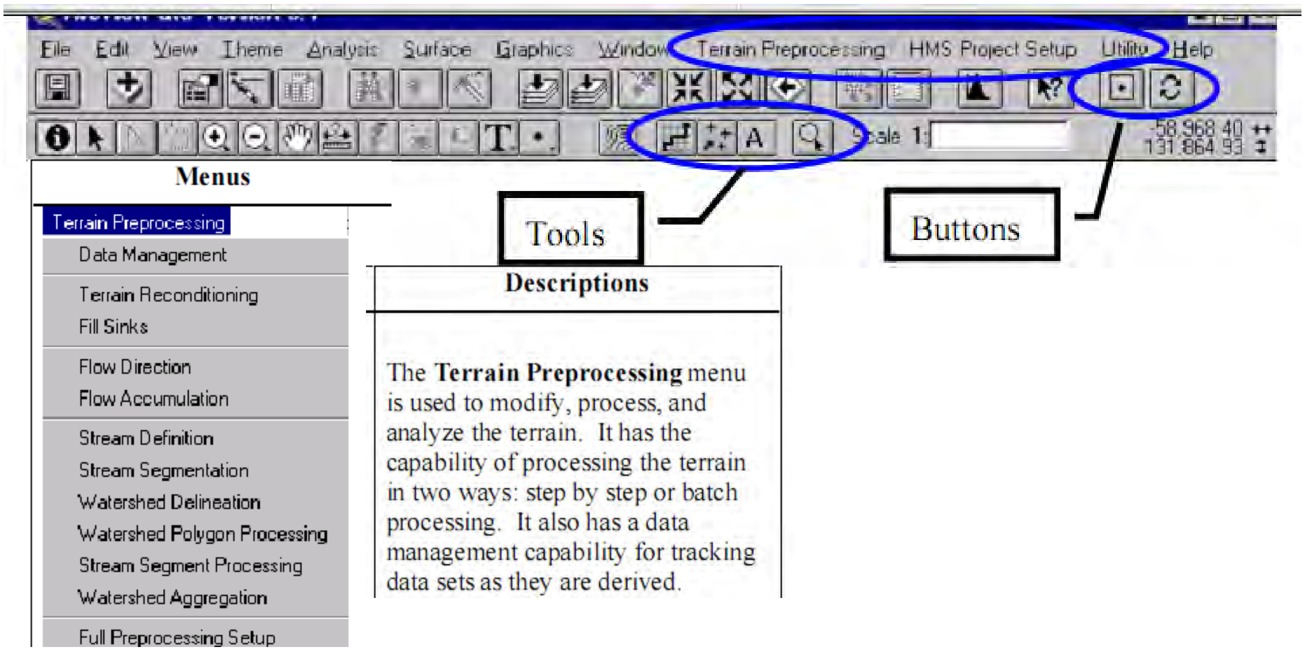
The following procedures describe the major steps in starting a project and taking it through the GeoHMS development of a hydrologic model using DEM. These are

i. Terrain Model Preprocessing
ii. Hydrologic Processing
    - Basin Processing
    - Stream and Watershed Characteristics
    - HMS Model Files
iii. Hydrologic Parameters and HEC-HMS

3.3.2.1 Terrain Model Pre-processing

The steps consist of computing the fill sinks, flow direction, flow accumulation, stream definition, stream segmentation, watershed delineation, watershed polygon processing, stream segmentation and watershed aggregation. These steps can be done step by step or in a batch manner. Watershed and stream delineation developed in this step is preliminary and they are used in later steps for sub-basin and stream delineation. Terrain pre-processing is performed in the Main View document of ArcView GUI.

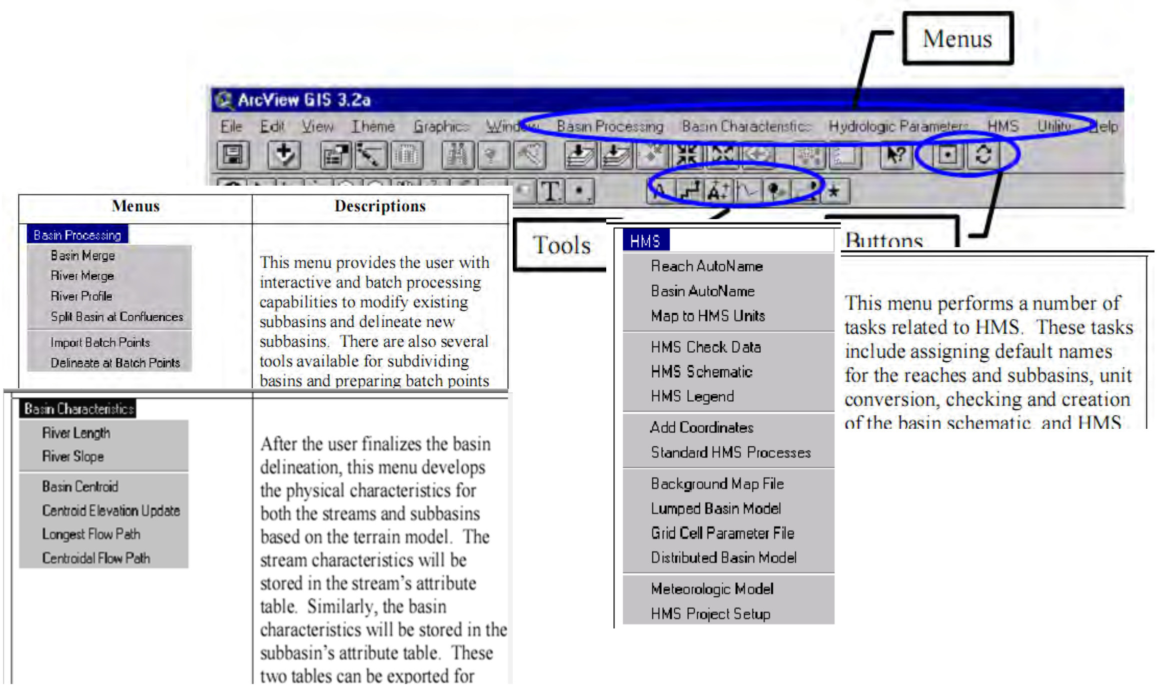
Figure 3-3 Overview of Terrain pre-processing GUI



3.3.2.2 Hydrologic Processing

The steps consist of computing basin processing, stream and watershed characteristics and HMS Model Files. It is performed in the ProjectView document of ArcView GUI. The menus, buttons, and tools in the ProjectView GUI are shown in Figure below.

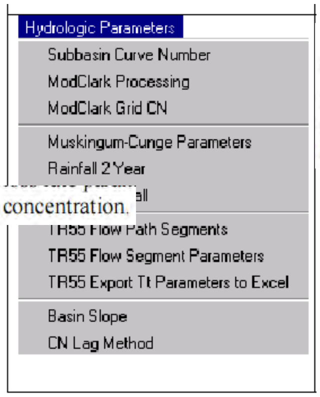
Figure 3-4 Overview of hydrologic processing GUI



3.3.2.3 Hydrologic Parameters and HEC-HMS

In addition to extracting stream and sub-basin physical characteristics, the user has the option to estimate initial values of various hydrologic parameters.

Figure 3-5 Hydrologic parameters computing window



This menu facilitates a process for the user to estimate a number of hydrologic parameters, such as loss rate parameters and time of concentration.

3.3.3 HEC-HMS

HEC-HMS 3.1.0 is basin model designed to simulate the precipitation-runoff processes of dendritic watershed systems. It is responsible for describing the physical properties of the watershed and the topology of the stream network.

It is designed to be applicable in a wide range of geographic areas for solving a broad range of problems. This includes large river basin water supply and flood hydrology to small urban or natural watershed runoff.

3.3.3.1 HMS Model Components

HEC-HMS model components are used to simulate the hydrologic response in a watershed. HMS model components include basin models, meteorologic models, control specifications, and input data. A simulation calculates the precipitation-runoff response in the basin model given input from the meteorologic model. The control specifications define the time period and time step of the simulation run.

Input data components, such as time-series data, paired data, and gridded data are often required as parameter or boundary conditions in basin and meteorologic models.

i. Basin Model Component

The basin model represents the physical watershed. The user develops a basin model by adding and connecting hydrologic elements. Hydrologic elements use mathematical models to describe physical processes in the watershed. Table 3 provides a list and description of used hydrologic elements. However, in this vase this step is performed using HEC-GeoHMS.

Table 3-1: Hydrologic Element Description.

Hydrologic Element	Description
Subbasin	The subbasin element is used to represent the physical watershed. Given precipitation, outflow from the subbasin element is calculated by subtracting precipitation losses, transforming excess precipitation to stream flow at the subbasin outlet, and adding baseflow.
Reach	The reach element is used to convey stream flow downstream in the basin model. Inflow into the reach element can come from one or many upstream hydrologic elements. Outflow from the reach is calculated by accounting for translation and attenuation of the inflow hydrograph.
Junction	The junction element is used to combine stream flow from hydrologic elements located upstream of the junction element. Inflow into the junction element can come from one or many upstream elements. Outflow is simply calculated by summing all inflows and assuming no storage at the junction.

In the case of the subbasin element, many mathematical models are available for determining precipitation losses, transforming excess precipitation to stream flow at the subbasin outlet, and adding baseflow. Table 4 lists the methods available for subbasin and river reach elements.

Table 3-2: Sub-basin and Reach Calculation Methods.

Hydrologic Element	Calculation Type	Method
Subbasin	Runoff-volume	Deficit and constant rate (DC), Exponential, Green and Ampt, Gridded DC, Gridded SCS CN, Gridded SMA, Initial and constant rate, SCS curve number (CN), Smith Parlange, and Soil moisture accounting (SMA)
	Direct-runoff	Clark’s UH, Kinematic wave, ModClark, SCS UH, Snyder’s UH, User-specified s-graph, and User-specified unit hydrograph (UH)
	Base flow	Bounded recession, Constant monthly, Linear reservoir, Nonlinear Boussinesq, and Recession
Reach	Routing	Kinematic wave, Lag, Modified Puls, Muskingum, and Muskingum-Cunge
	Loss/Gain	Constant and Percolation

ii. Meteorologic Model Component

The meteorologic model calculates the precipitation input required by a subbasin element. The meteorologic model can utilize both point and gridded precipitation and has the capability to model frozen and liquid precipitation along with evapotranspiration. A brief description of the methods available for calculating basin average precipitation or grid cell precipitation is included in Table 5.

Table 3-3: Description of Meteorologic Model Methods.

Precipitation Methods	Description
Frequency Storm	This method is used to develop a precipitation event where precipitation depths for various durations within the storm have a consistent exceedance probability.
Gage Weights	This method applies user specified weights to user defined precipitation gages.
Gridded Precipitation	This method allows the use of gridded precipitation products, such as RADAR.
Inverse Distance	This method calculates sub-basin average precipitation by applying an inverse distance squared weighting to user defined precipitation gages.
SCS Storm	This method applies a user specified SCS time distribution to a 24-hour total storm depth.
Specified Hyetograph	This method applies a user defined hyetograph to a specified sub-basin element.
Standard Project Storm	This method applies a time distribution to an index precipitation depth.

iii. Control Specifications Component

The control specifications set the time span of a simulation run. Information in the control specifications includes a starting date and time, ending date and time, and computation time step.

iv. Input Data Components

Time-series data, paired data, and gridded data are often required as parameter or boundary conditions in basin and meteorologic models. A complete list of input data is included in Table 3.4. Input data entered manually.

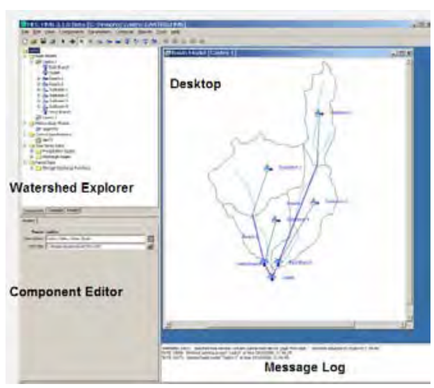
Table 3-4: Input data components.

Time-Series Data	Paired Data	Gridded Data
Precipitation gages	Storage-discharge functions	Precipitation grid sets
Discharge gages	Elevation-storage functions	Temperature grid sets
Stage gages	Elevation-area functions	Solar radiation grid sets
Temperature gages	Elevation-discharge functions	Crop coefficient grid sets
Solar radiation gages	Inflow-diversion functions	Storage capacity grids
Crop coefficient gages	Cross sections Unit hydrograph curves Percentage curves ATI-meltrate functions ATI-coldrate functions Groundmelt patterns Meltrate patterns	Percolation rate grids Storage coefficients grids Moisture deficit grids Impervious area grids SCS curve number grids Elevation grids Cold content grids Cold content ATI grids Meltrate ATI grids Liquid water content grids Snow water equivalent grids

3.3.3.2 User Interface

The user interface consists of a menu bar, tool bar, and four main panes. Starting from the upper left pane in Figure 3.5 below and moving counter-clockwise, these panes will be referred to as the Watershed Explorer, the Component Editor, the Message Log, and the Desktop.

Figure 3-6 HEC-HMS GUI



3.4. Delineation and Identification of Flood Risk Areas

The flood Inundation map shows the area extent to be delineated as buffer zone. Two models HEC_GeoRAS and HEC_RAS are used one after another (i.e first HEC_GeoRAS then HEC_RAS then back to HEC_GeoRAS) to accomplish the task. Dire Dawa town 1:5000 scale map having 4 meters for the town area and 0.5m for Dechatu stream cross section contour interval is converted to TIN format, 57m DEM resolution and all land use data used to proceed.

HEC-GeoRAS is a set of procedures, tools, and utilities for processing geographic information systems (GIS) data in ArcView GIS, using a graphical user interface (GUI). The interface allows preparation of geometric data for import into HEC-RAS and generation of GIS data from exported HEC-RAS simulation results. Automated GIS processing procedures in HEC-GeoRAS provides a valuable and expeditious method for repetitive hydraulic model development during floodplain analysis. HEC-GeoRAS Version 3.1.1 used to extract cross-sectional station-elevation data from a digital elevation model (DTM) represented by a triangulated irregular network (TIN). Downstream reach lengths and bank station locations were determined for each cross section. The automated procedures for extracting geometric data proved consistent and efficient for the development of floodplain models to evaluate wetland scenarios (http://cedb.asce.org/cgi/WWWdisplay.cgi). The geometric data was imported into HEC-RAS Version 3.1.1 using a data exchange format developed by HEC. The resultant water surface elevations exported from HEC-RAS simulations were processed by HEC-GeoRAS for floodplain delineation and water depth calculations. Analysis of cross-sectional velocities exported from HEC-RAS was also performed using HEC-GeoRAS.

GeoRAS allows the preparation of geometric data for import into HEC-RAS and processes simulation results exported from HEC-RAS with an existing 4m for the town area and 0.5m for Dechatu stream cross section contour interval changed to TIN format and 57m resolution digital terrain model (DTM) of the river system. The user creates a series of line themes pertinent to developing geometric data for HEC-RAS. The themes created are the Stream Centerline, Flow Path Centerlines (optional), Main Channel Banks (optional), and Cross Section Cut Lines referred to as the RAS Themes.

Additional RAS Themes may be created/used to extract additional geometric data for import in HEC-RAS. These themes include Land Use, Levee Alignment, Ineffective Flow Areas, and Storage Areas.

HEC-GeoRAS is an ArcView GIS extension that provides the user with a set of procedures, tools, and utilities for the preparation of GIS data for import into HEC-RAS, and generation of GIS data from RAS output.

Procedures followed:-

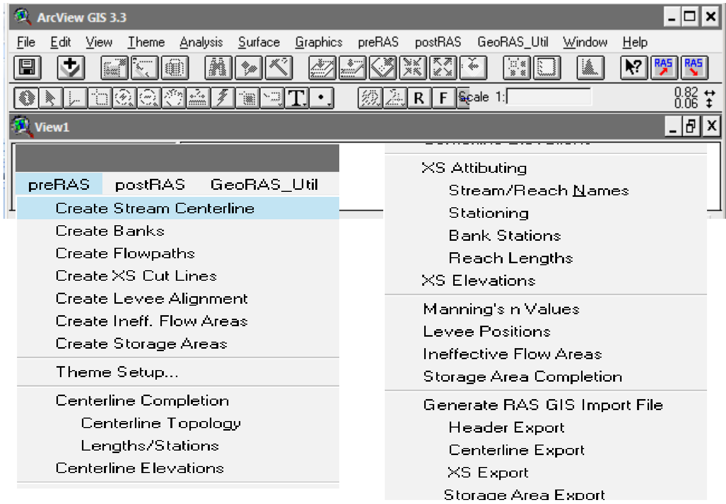
The 4m for the town area and 0.5m for Dechatu stream cross section contour interval map is changed to Terrain TIN (a triangulated irregular network) using ArcView GIS extension HEC-GeoRAS. Then the following main procedures are applied.

3.4.1 PreRAS (HEC-GeoRAS) Processing

The goal of this section is to develop the spatial data required to generate a HEC-RAS import file with a 3-D stream network and 3-D cross sections defined. The process is divided in three steps:

- Preparation of 3-D polyline themes defining stream centerline, cross-sections, stream banks, and flow path lines.
- Use of the HEC-GeoRAS **preRAS** menu functions to extract 3-D spatial data from the TIN to develop 3-D polylineZ themes of the previously defined stream centerline, cross-sections, stream banks, and flow path lines.
- Generation of the HEC-RAS Import File.

Figure 3-7 HEC-GeoRAS window:



3.4.1.1 Creating RAS Themes

The RAS Themes are the basis for the geometric data extracted in the GIS for hydraulic analysis in HEC-RAS. These Themes include: Stream Centreline, Banks, Flow Paths Centrelines, Cross-Sectional Cut Lines, Land Use, Levee Alignments, Ineffective Flow Areas, and Storage Areas.

3.4.1.2 Attributing RAS Theme

Once the RAS Themes have been created, the geometric data extraction process began. The Stream Centreline Theme completed and the cross-section attributes (geometric data for each cross section) calculated. Stream Centreline Theme created before completing the Cross-Section Cut Line Theme and the Cross-Sectional Cut Lines Theme completed.

3.4.1.3 Generating the RAS GIS Import File

To generate the RAS GIS Import File, the 3D stream Centreline and Cross Section Surface Line (3D) shape file created from the RAS Theme. Geometric data from the two 3D (stream Centreline and Cross Section Surface Line) shape files is written to the RAS GIS Export File. The geometric data includes: river, reach, and station identifiers; cross-section cut lines; cross-section surface lines; main channel bank stations; downstream reach lengths for the left overbank, main channel; and overbank.

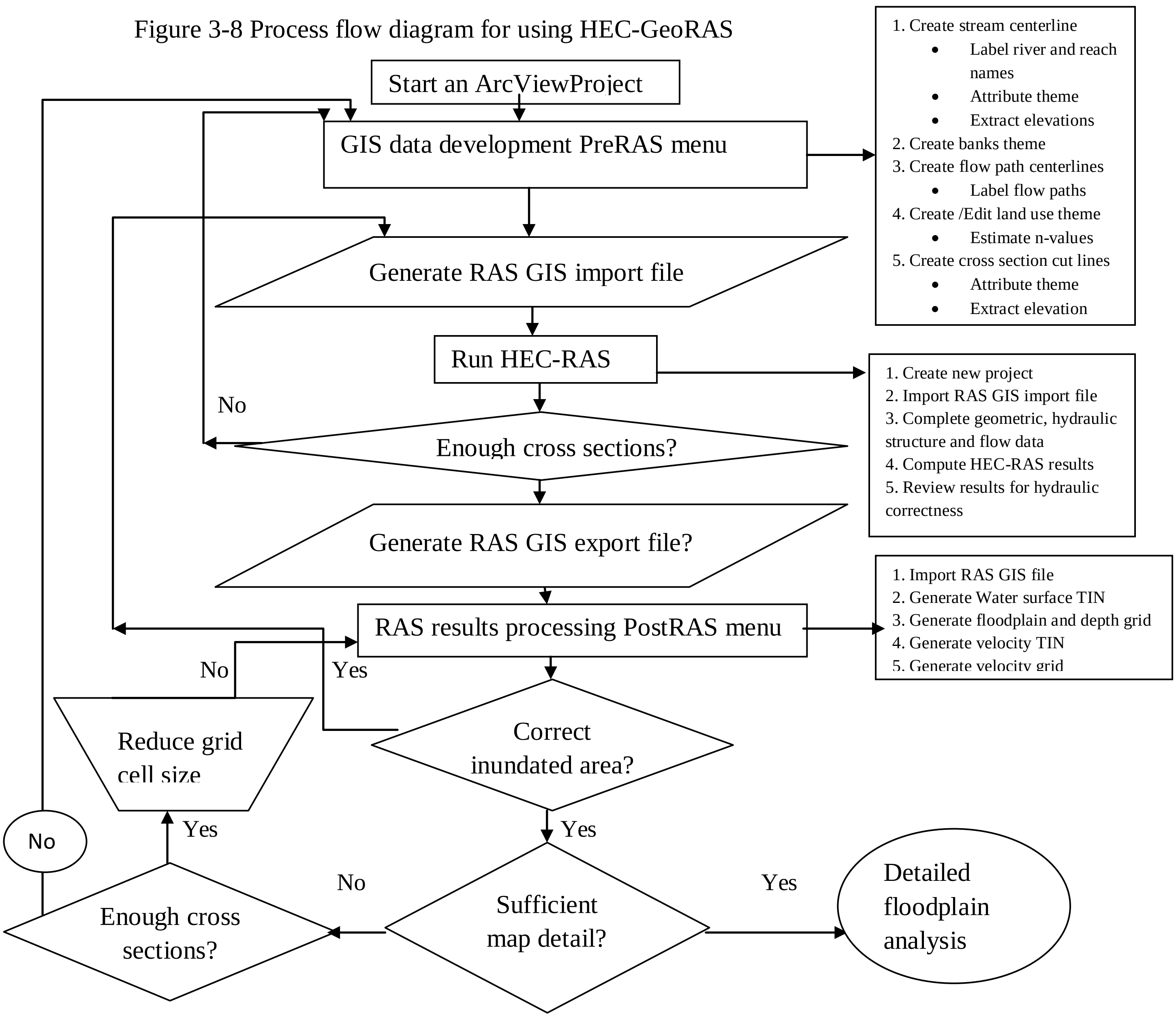
HEC-GeoRas is an ArcView GIS extension specifically designed to process geospatial data for use with the Hydrologic Engineering Canter's River Analysis System (HEC_RAS). This extension allows us to;

- Create an HEC-RAS import file containing geometric attribute data from an existing digital terrain model (DEM) and complementary data sets.
- Process results exported from HEC-RAS.

It creates an import file, referred as the RAS GIS Import File, containing river, reach and station identifier; cross-sectional cut lines; cross-sectional surface lines; cross-sectional bank stations; downstream reach lengths for the left overbank, main channel, and right over bank; and cross-sectional roughness coefficients.

HEC_GeoRAS also enables viewing of exported results from RAS. The import file is created from data extracted from data sets (ArcView shape files) and from a Digital Terrain Model (DTM) represented by a triangulated irregular network (TIN).
Prior to performing hydraulic computations in HEC-RAS, the geometric data must be imported and completed and flow data must be entered. Once the hydraulic computations are performed, exported water surface and velocity results from HEC-RAS may be imported back to the GIS using HEC-GeoRAS for spatial analysis. GIS data is transferred between HEC-RAS and ArcView using a specifically formatted GIS data exchange file.

Figure 3-8 Process flow diagram for using HEC-GeoRAS



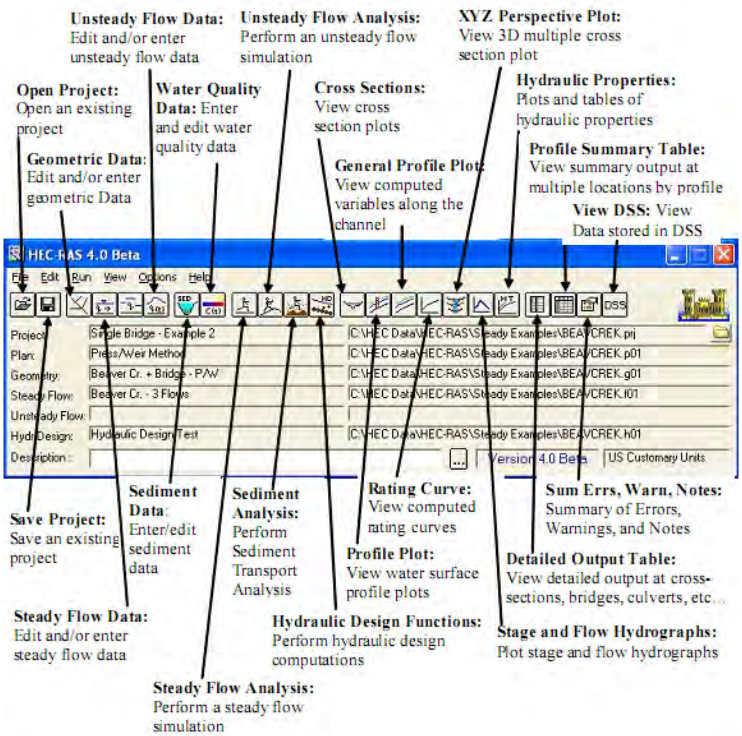
3.4.2 HEC-RAS

The HEC-RAS program, like the other above three softwares, it can be downloaded free of charge from the Hydrologic Engineering Center's. Hydrologic Engineering Center’s

River Analysis System (HEC-RAS) is the software predominately used in the field of hydraulic analysis for floodplain delineation.

HEC-RAS, combined with Hydrologic Engineering Center's Geographical River Analysis System (HEC-GeoRAS), offers engineers a powerful tool in the process of hydraulic modeling and analysis.

Figure 3-9 The HEC-RAS main window



For each HEC-RAS project, there are three required components--the Geometry data, Flow data, and Plan data. The Geometry data, for instance, consists of a description of the size, shape, and connectivity of stream cross-sections. Likewise, the Flow data contains discharge rates. Finally, Plan data contains information pertinent to the run specifications of the model, including a description of the flow regime. Each of these components is explored below individually.

3.4.2.1 Importing and Editing Geometric Data

The first of the components is the channel geometry. To analyze stream flow, HEC-RAS represents a stream channel and floodplain as a series of cross-sections along the channel. To create our geometric model, we need to import the geometry file that I just exported. This HEC-RAS geometry file contains physical parameters describing cross-sections.

3.4.2.2 Flow Data

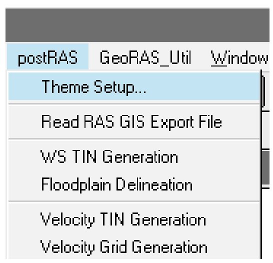
The flow data has been extracted from a HEC-HMS hydrologic model. Unsteady flow condition is adopted since it is rare to find the steady flow in the natural channel flow condition. This component of the HEC-RAS modelling system is capable of simulating one dimensional unsteady flow through a full network of open channels.

Figure 3-10 The HEC-RAS Geometric and Flow Data

3.4.3 PostRAS (HEC-GeoRAS)

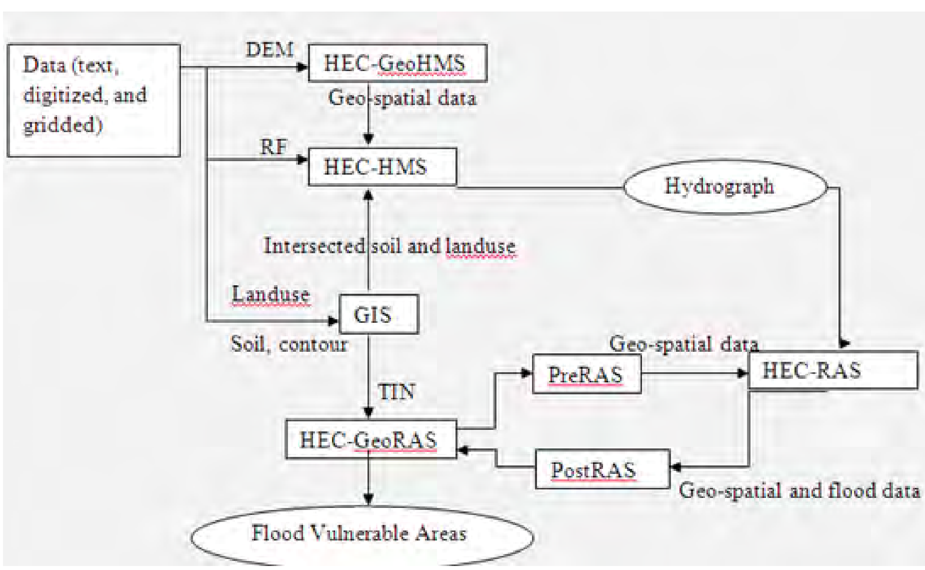
With the development of a GIS Import File from HEC-RAS, we can now begin the last portion of the exercise. Post-processing using GeoRAS incorporates the water surface profiles derived from the HEC-RAS model into the spatial environment of GIS. The water surface profile data is used to develop a water surface TIN, and the intersection of the water surface TIN with the terrain model TIN provides flood visualization. The results can be shown in 2-D or 3-D views.

Figure 3-11 PostRAS Window



3.5. Computer Programs usage Procedure Layout

After the models are selected, data collected in the field and offices. Then the data analysis started.



4. DATA ANALYSIS

4.1. Basin Model

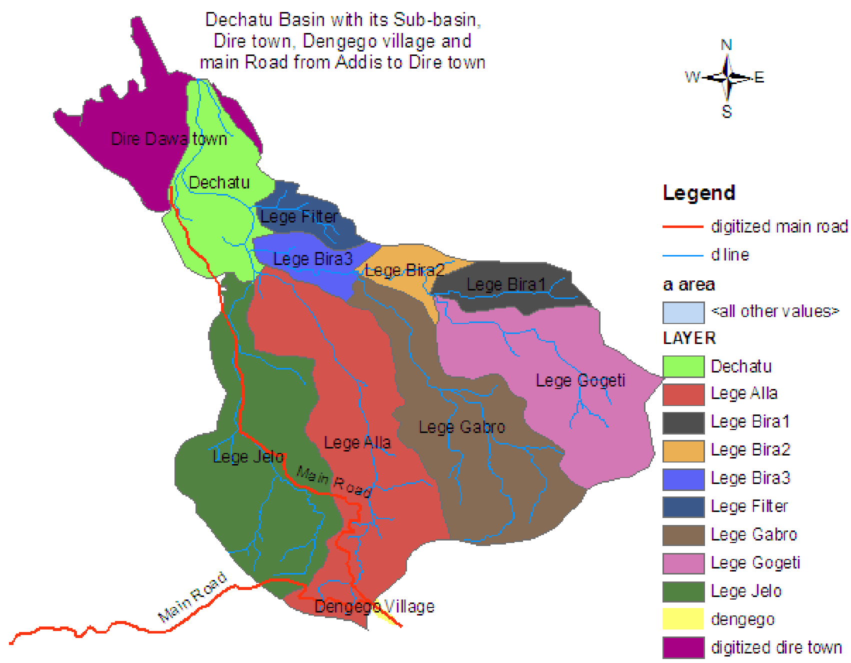
Basin model is one of the main components. Their principle purpose is to convert atmospheric conditions into stream flow at specific locations in the catchment. The basin model is responsible for describing the physical properties of the watershed and the topology of the stream network. This portion describes how watershed information is entered into the program using HEC-GeoHMS and HEC-HMS 3.1.0 basin model. It contains the modeling components that describe catchment data, infiltration, surface runoff and channel routing. Outflow is computed from meteorological data by subtracting losses of basin and transforming excess precipitation through the basin. Sub-basins are used to model the catchment.

Dechatu stream catchment is used for the analysis since it is the major one which causes devastating damage as it passes through the middle of the town.

4.1.1. Sub-basins

A sub-basin is an element that usually has no inflow and only one outflow. The sub-basins for the four basins/catchments are described in the following sections.

Figure 4-1: Dechatu Catchment, Sub- basins, Dire Dawa Town and Main Road Addis via Dire Dawa



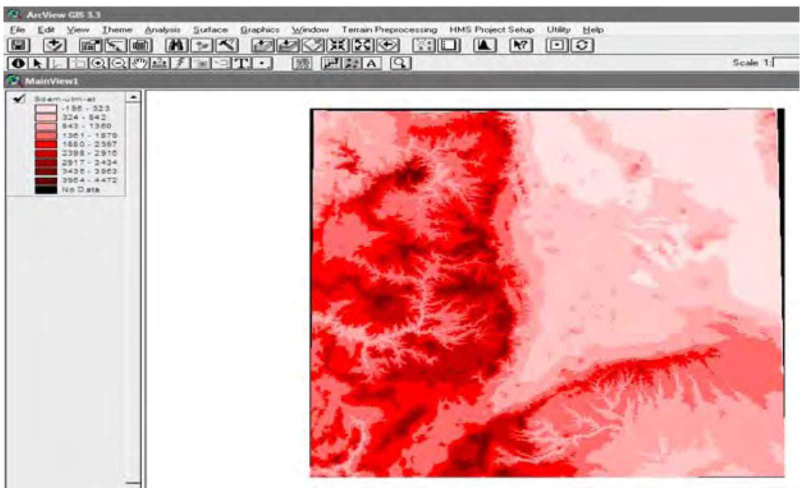
4.1.1.1 Physical Description of Sub-basin (HEC-GeoHMS)

Hydrologic elements are used to break the sub-basin into manageable pieces. They are connected together in a dendritic (branched extension or shape of a tree branch) network to form a representation of the stream system. The physical parameter analysis is done using HEC-GeoHMS.

Procedures adopted:

- As described above, 57m resolution Awash Basin DEM (found from Halcrow, (2008). Awash River Basin Flood Control and Watershed Management Study Project Study Project, Hydrology Report Phase III) is used to analyze digital terrain information.
- Grid Machine 3.2 toolbox by Johannes Weigel is used to cut the Dechatu Catchment from 57m resolution Awash Basin DEM.

Figure 4-2: 57m resolution Awash Basin DEM



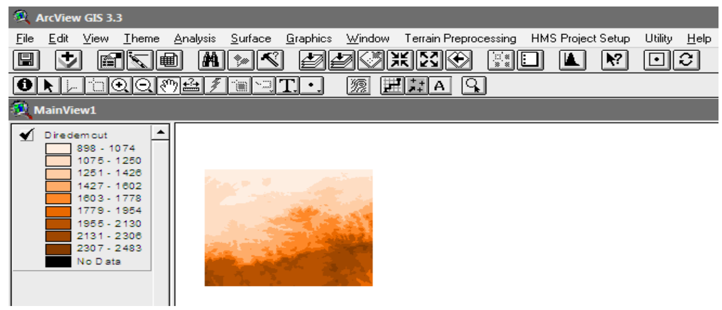
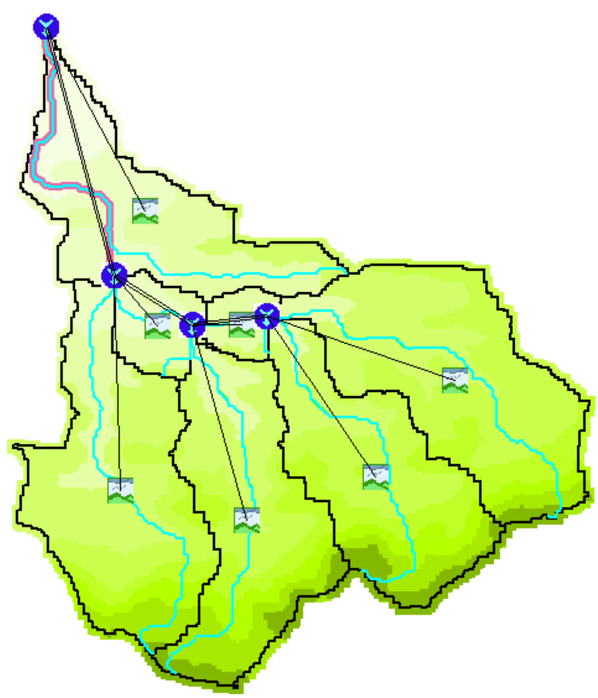


Figure 4-3: 57m resolution Dire Dawa Catchment DEM

Then using the methodology described above the following outputs are obtained.

Figure 4-4: HEC-GeoHMS Delinated Sub-basin



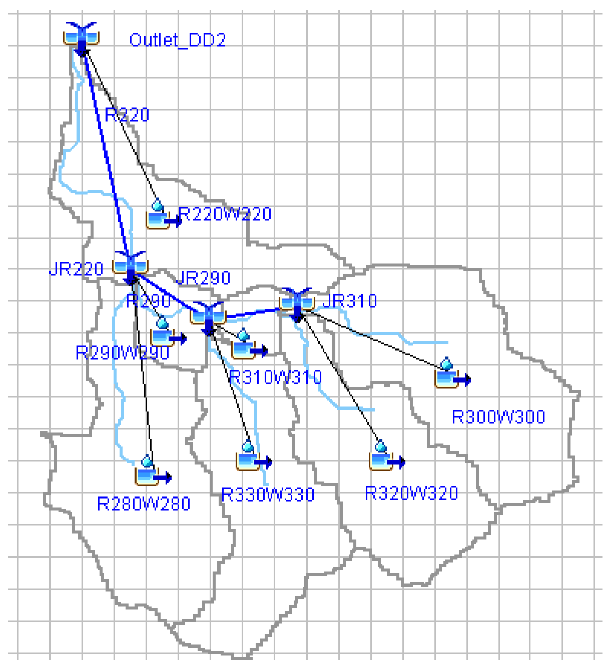


Figure 4-5: Dechatu Catchment Hec-HMS

Attributing Output tables:-

Table 4-1: Dechatu Catchment HEC-GeoHMS Output to be used for the HEC-HMS

SUB-BASIN NAME	UTM Centroid Location	
	EAST	NORTH
R220W220	817584.5	1059693.5
R320W320	824310.5	1051941.5
R290W290	817926.5	1056330.5
R310W310	820377.5	1056387.5
R300W300	826590.5	1054734.5
R280W280	816843.5	1051542.5
R330W330	820548.5	1050687.5

REACH NAME	RIVER LENGTH (m)	U/S ELV (m)	D/S ELV (m)	Slope
R220	9770.500	1320.000	1138.000	0.0186
R290	3607.200	1390.000	1320.000	0.0194
R310	2354.900	1441.000	1390.000	0.0217
R300	6234.100	1597.000	1441.000	0.0250
R320	5192.600	1608.000	1441.000	0.0322
R280	6969.400	1510.000	1320.000	0.0273
R330	6247.900	1604.000	1390.000	0.0343

SUB-BASIN NAME	AREA (km^2)	PERIMETER (m)	LONGEST LENGTH (m)	U/S ELV (m)	D/S ELV (m)
R220W220	27.035	38190.00	16612.000	1607.000	1138.000
R320W320	28.104	32490.00	12299.560	2265.000	1445.000
R290W290	6.404	14706.00	5739.814	1563.000	1320.000
R310W310	3.606	9576.00	3404.492	1588.000	1392.000
R300W300	35.762	35796.00	13471.271	2193.000	1444.000
R280W280	34.829	38646.00	14591.712	2177.000	1320.000
R330W330	29.914	37734.00	13117.119	2198.000	1392.000

4.1.1.2 Loss

While a sub-basin element conceptually represents infiltration, surface runoff, and subsurface processes interacting together, the actual infiltration calculations are performed by a loss method. Due to the available data type SCS Curve Number Loss method is adopted for loss computation. After selecting the method, three parameters are required as shown below to compute the loss.

These are:-

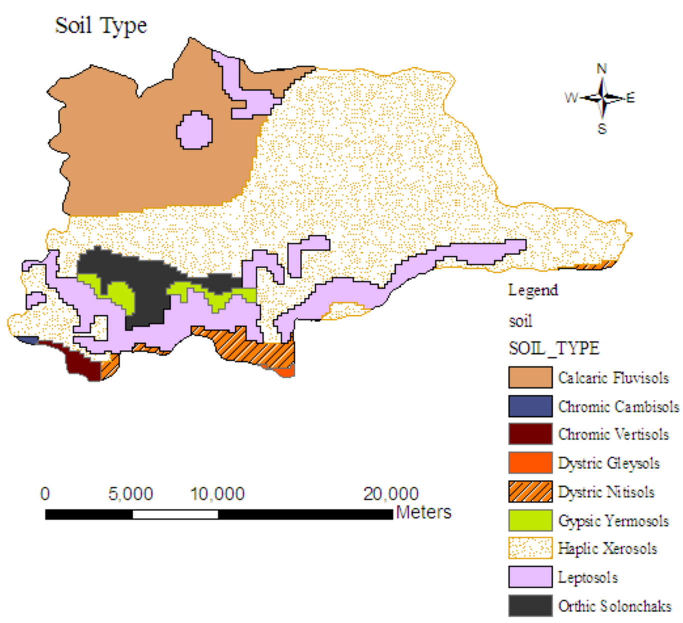
- Curve Number
- Initial Abstraction
- Impervious (%)

Curve Number

The curve number, commonly abbreviated as CN, is a composite curve number that represents the entire different soil group and land use combinations in the sub-basin. The composite curve number does not include impervious area that will be specified separately as the percentage of impervious area in the program.

CN values range from 100 (for water bodies) to approximately 30 for permeable soils with high infiltration rates. To compute the curve number, land use and soil types are required for the catchment.

Figure 4-6: Soil Data in the Required Range



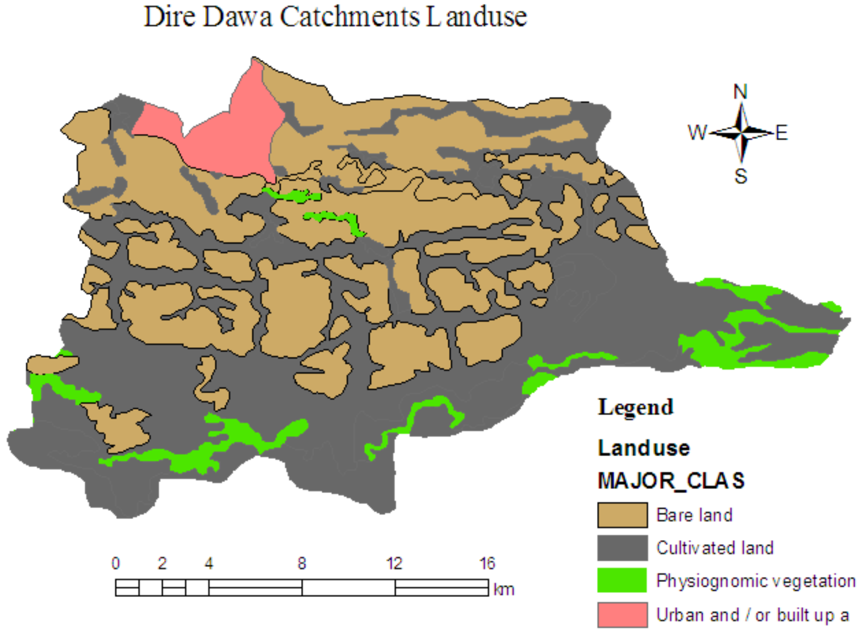


Figure 4-7: Landuse in the Required Range

For a watershed that consists of more than one soil types and land uses, a composite CN is calculated as:

$$CN_{composite} = \frac{\sum A_i CN_i}{\sum A_i}$$

in which, $CN_{composite}$ = the composite CN used for runoff volume computations;

i = an index of watersheds subdivisions of uniform land use and soil type;

CN_i = the CN for subdivision i; and

A_i = the drainage area of subdivision i.

Using GIS intersection of watershed map and soil, the following table is summarized.

Table 4-2: Dechatu sub-basins the corresponding soil-type, hydrologic soil group, landuse and area coverage

soil-type, hydrologic soil group and area coverage			
Sub-Basin Name	**Soil Type**	**Hydrologic Soil Group**	**Area (m^2)**
R220W220	Calcaric Fluvisols	B	16929218.740
	Haplic Xerosols	B	5076524.069
	Leptosols	D	3739455.782
R320W320	Haplic Xerosols	B	19645297.992
	Leptosols	D	7704527.228
R290W290	Calcaric Fluvisols	B	5719095.908
	Haplic Xerosols	B	684683.092
R310W310	Calcaric Fluvisols	B	57627.950
	Haplic Xerosols	B	3548762.050
R300W300	Haplic Xerosols	B	32486671.229
	Leptosols	D	2887306.420
R280W280	Leptosols	D	6967590.053
	Dystric Nitisols	B	1906952.364
	Haplic Xerosols	B	14970454.986
	Calcaric Fluvisols	B	3566888.928
	Orthic Solonchaks	B	4089640.080
	Gypsic Yermosols	B	3290284.051
R330W330	Dystric Nitisols	B	3220481.393
	Calcaric Fluvisols	B	82041.978
	Haplic Xerosols	B	18395061.109
	Leptosols	D	8116158.902
	Gypsic Yermosols	B	84515.974
	Dystric Gleysols	B	4901.995

landuse and area coverage		
Sub-Basin Name	**Land use**	**Area (m^2)**
R220W220	Bare land	13614529.814
	Cultivated land	2375623.222
	Physiognomic vegetation	1465426.089
	Urban and / or built up area	8289619.470
R320W320	Bare land	10937772.287
	Cultivated land	15457133.938
	Physiognomic vegetation	954918.964
R290W290	Bare land	3618165.897
	Cultivated land	2785613.103
R310W310	Bare land	1705976.569
	Cultivated land	1718837.683
	Physiognomic vegetation	181575.748
R300W300	Bare land	15996685.663
	Cultivated land	18197477.950
	Physiognomic vegetation	1179814.008
R280W280	Bare land	13465828.591
	Cultivated land	19759198.658
	Physiognomic vegetation	1566783.216
R330W330	Bare land	9314750.141
	Cultivated land	19515477.374
	Physiognomic vegetation	1072973.51286

Table 4-3: Composite curve number

NAME	Soil Type	Soil Group	Land Use	CN	Area (m^2)	Area $\sum A_i$ km^2	CN composite
R220W220	Calcaric Fluvisols	B	Bare land	79	7.322		
			Cultivated land	81	1.431		
			Physiognomic vegetation	58	0.989		
			Urban and / or built up	Impervious	7.188		
	Leptosols	D	Bare land	89	1.598		
			Cultivated land	91	0.849		
			Physiognomic vegetation	78	0.19		
			Urban and / or built up	Impervious	1.102	8.29	32.20%
	Haplic Xerosols	B	Bare land	79	4.695		
			Cultivated land	81	0.096		
			Physiognomic vegetation	58	0.286	17.456	79.129
R320W320	Haplic	B	Bare land	79	9.286		
			Cultivated land	81	10.077		
			Physiognomic vegetation	58	0.283		
	Leptosols Xerosols	D	Bare land	89	1.652		
			Cultivated land	91	5.38		
			Physiognomic vegetation	78	0.672	27.35	82.46
R290W290	Haplic Xerosols	B	Bare land	79	0.555		
			Cultivated land	81	0.129		
	Calcaric Fluvisols	B	Bare land	79	3.063		
			Cultivated land	81	2.656	6.404	79.87
R310W310	Haplic Xerosols	B	Bare land	79	1.686		
			Cultivated land	81	1.681		
			Physiognomic vegetation	58	0.182		
	Calcaric Fluvisols	B	Cultivated land	79	0.038		
			Bare land	81	0.02	3.606	78.886
R300W300	Haplic Xerosols	B	Bare land	79	15.997		
			Cultivated land	81	16.376		
			Physiognomic vegetation	58	0.114		
	Leptosols	D	Cultivated land	89	1.822		
			Physiognomic vegetation	91	1.066	35.374	80.735

NAME	Soil Type	Soil Group	Land Use	CN	Area (m^2)	Area $\sum A_i$ km^2	CN composite
R280W280	Calcaric Fluvisols	B	Bare land	79	1.873		
			Cultivated land	81	1.694		
	Haplic Xerosols	B	Bare land	79	10.025		
			Cultivated land	81	4.945		
	Leptosols	D	Bare land	89	1.558		
			Cultivated land	91	5.351		
			Physiognomic vegetation	78	1.31		
	Gypsic Yermosols	B	Bare land	79	0.177		
			Cultivated land	81	3.097		
			Physiognomic vegetation	58	0.017		
	Orthic Solonchaks	B	Bare land	79	1.084		
	Dystric Nitisols	B	Cultivated land	81	4.672		
			Physiognomic vegetation	58	0.241	36.043	81.827
R330W330	Haplic Xerosols	B	Bare land	79	6.88		
			Cultivated land	81	11.32		
			Physiognomic vegetation	58	0.195		
	Leptosols	D	Bare land	89	1.167		
			Cultivated land	91	4.82		
			Physiognomic vegetation	58	0.878		
	Dystric Nitisols	B	Cultivated land	79	3.307		
	Gypsic Yermosols	B	Bare land	79	0.008		
			Cultivated land	81	0.076	28.652	81.435

Initial Abstraction, I_a

From analysis of results from many small experimental watersheds, the SCS developed an empirical relationship of Ia and S:

$I_a = 0.2S$,

The maximum retention, S, and watershed characteristics are related through an intermediate parameter, the curve number (CN) as:

$$S = \frac{25400}{CN} - 254 \qquad \text{(SI)}$$

Table 4-4: Maximum retention (S) and Initial abstraction (Ia)

NAME	Soil Type	Soil Group	Land Use	Area $\sum A_i$ km^2	CN composite	Maximum Retention S	Initial Abstraction I_a
R220W220	Calcaric Fluvisols	B	Bare land				
			Cultivated land				
			Physiognomic vegetation				
			Urban and / or built up				
	Leptosols	D	Bare land				
			Cultivated land				
			Physiognomic vegetation				
			Urban and / or built up	**8.290**	**32.199**		
	Haplic Xerosols	B	Bare land				
			Cultivated land				
			Physiognomic vegetation	17.456	79.129	66.994	13.399
R320W320	Haplic Xerosols	B	Bare land				
			Cultivated land				
			Physiognomic vegetation				
	Leptosols	D	Bare land				
			Cultivated land				
			Physiognomic vegetation	27.350	82.460	54.029	10.806
R290W290	Haplic Xerosols	B	Bare land				
			Cultivated land				
	Calcaric Fluvisols	B	Bare land				
			Cultivated land	6.404	79.870	64.017	12.803
R310W310	Haplic Xerosols	B	Bare land				
			Cultivated land				
			Physiognomic vegetation				
	Calcaric Fluvisols	B	Cultivated land				
			Bare land	3.606	78.886	67.985	13.597
R300W300	Haplic Xerosols	B	Bare land				
			Cultivated land				
			Physiognomic vegetation				
	Leptosols	D	Cultivated land				
			Physiognomic vegetation	35.374	80.735	60.611	12.122

NAME	Soil Type	Soil Group	Land Use	Area $\sum A_i$ km^2	CN composite	Maximum Retention S	Initial Abstraction I_a
R280W280	Calcaric Fluvisols	B	Bare land				
			Cultivated land				
	Haplic Xerosols	B	Bare land				
			Cultivated land				
	Leptosols	D	Bare land				
			Cultivated land				
			Physiognomic vegetation				
	Gypsic Yermosols	B	Bare land				
			Cultivated land				
			Physiognomic vegetation				
	Orthic Solonchaks	B	Bare land				
	Dystric Nitisols	B	Cultivated land				
			Physiognomic vegetation	36.043	81.827	56.411	11.282
R330W330	Haplic Xerosols	B	Bare land				
			Cultivated land				
			Physiognomic vegetation				
	Leptosols	D	Bare land				
			Cultivated land				
			Physiognomic vegetation				
	Dystric Nitisols	B	Cultivated land				
	Gypsic Yermosols	B	Bare land				
			Cultivated land	28.652	81.435	57.904	11.581

After insertion of these three data, the Soil Conservation Service (SCS) Curve Number (CN) model estimates precipitation excess as a function of cumulative precipitation, soil cover, land use, and antecedent moisture, using the following equation.

The HEC-HMS program computes incremental precipitation during a storm by recalculating the infiltration volume at the end of each time interval. Infiltration during each time interval is the difference in volume at the end of two adjacent time intervals.

$$P_e = \frac{(P - I_a)^2}{P - I_a + S}$$

The precipitation excess, and hence the runoff will be zero until the accumulated rainfall exceeds the initial abstraction.

Therefore, the cumulative excess at time *t* is:

$$P_e = \frac{(P-0.2S)^2}{P+0.8S}$$

Incremental excess for a time interval is computed as the difference between the accumulated excess at the end of and beginning of the period.

4.1.1.3 Transforming

The Soil Conservation Service (SCS) proposed a parametric UH model; this model is included in the HEC_HMS program and adopted for the transformation process. The model is based upon averages of UH derived from gauged rainfall and runoff for a large number of small agricultural watersheds throughout the US.
The dimensionless UH, which is shown in Appendix A, expresses the UH discharge, U_t, as a ratio to the UH peak discharge, Up, for any time t, a fraction of Tp, the time to UH peak.

Research by the SCS suggests that the UH peak and time of UH peak are related by:

$$U_p = \frac{CA}{T_P}$$

in which A = watershed area; and C = conversion constant (2.08 in SI system). The time of peak (also known as the time of rise) is related to the duration of the unit of excess precipitation as:

$$T_p = \frac{\Delta t}{2} + t_{lag}$$

in which Δt = the excess precipitation duration (which is also the computational interval in the run); and t_{lag} = the basin lag, defined as the time difference between the center of mass of rainfall excess and the peak of the UH.

When the lag time is specified, the program solves the above equations to find the time of UH peak and the UH peak. With Up and Tp known, the UH can be found from the dimensionless form, which is built into the program, by multiplication.

For ungauged watersheds, the SCS suggests that the UH lag time may be related to time of concentration, tc, as:

$t_{lag} = 0.6\ t_c$

Time of concentration can be estimated as:

$t_c = t_{sheet} + t_{shallow} + t_{channel}$

Where, t_{sheet} = sum of travel time in sheet flow segments over the watershed land surface;
$t_{shallow}$ = sum of travel time in shallow flow segments, down streets, in gutters, or in shallow rills and rivulets; and
$t_{channel}$ = sum of travel time in channel segments.

i. Sheet Flow

Sheet flow is flow over the watershed plane surface, before water reaches a channel. Distances are short on the order of 10-100 meters (30-300 feet). For sheet flow of less than 100 meters, use Manning's kinematic solution (Overton and Meadows, 1976) to compute t_{sheet}:-

$$t_{sheet} = \frac{0.091(NL)^{0.8}}{(P_2)^{0.5} S^{0.4}}$$

Where: t_{sheet} = travel time, hr; N = an overland-flow roughness coefficient;
L = flow length, m; P_2 = 2-year, 24-hour rainfall depth, mm; and

S = slope of hydraulic grade line, which may be approximated by the land slope.

The topography of our catchement is mountainous. Hence, the sheet flow is negligible since rills and rivulets are the dominant ones in mountainous topography; which are considered in the next section (shallow flow).

ii. Shallow Flow

Sheet flow usually turns to shallow concentrated flow after 100 meters. The average velocity m/sec for shallow concentrated flow can be estimated as:

$$V = 16.1345\sqrt{S}, \quad \textit{for unpaved surface}$$

$$= 20.3282\sqrt{S}, \quad \textit{for paved surface}$$

Table 4-5: Time of concentration and basin lag

River / Basin	Sub-basin	U/S Eel (m)	D/S ELV (m)	Longest Length (m)	$t_{shallow}$				Basin Lag (t_{lag}) min
					Slope	V (m/s)	$t_{shallow}$ (sec.)	$t_{shallow}$ (min.)	
Dechatu	R220W220	1607.0	1138.0	16612.000	0.028	2.711	6127.608	102.127	61.276
	R320W320	2265.0	1445.0	12299.560	0.067	4.166	2952.378	49.206	29.524
	R290W290	1563.0	1320.0	5739.814	0.042	3.320	1728.972	28.816	17.290
	R310W310	1588.0	1392.0	3404.492	0.058	3.871	879.417	14.657	8.794
	R300W300	2193.0	1444.0	13471.271	0.056	3.804	3540.923	59.015	35.409
	R280W280	2177.0	1320.0	14591.712	0.059	3.910	3731.758	62.196	37.318
	R330W330	2198.0	1392.0	13117.119	0.061	3.999	3279.704	54.662	32.797

iii. Channel Flow

$t_{channel} = \frac{L}{V}$, Where, L = channel length For these channels, V = average velocity

Using Manning's equation: $$V = \frac{CR^{2/3}S^{1/2}}{n}$$

Where; R = the hydraulic radius (defined as the ratio of channel cross-section area to wetted perimeter);

S = slope of the energy grade line (often approximated as channel bed slope); and

C = conversion constant (1.00 for SI and 1.49 for foot-pound system); and

n = Manning's roughness coefficient, can be estimated from textbook tables.

Once velocity is estimated, channel travel time is computed as:

$t_{channel} = \frac{L}{V}$, Since there is no separate channel segments in which flow takes place, travel time in channel segments ($t_{channel}$) is not applicable for this case.

4.1.1.4 Base Flow

While a sub-basin element conceptually represents infiltration, surface runoff, and subsurface processes interacting together, the actual subsurface calculations are performed by a base flow method contained within the sub-basin. For the study area, there is no base flow and taken as zero.

4.1.2. Reach

While a reach element conceptually represents a segment of stream or river, the actual calculations are performed by a routing method contained within the reach. Flow routing is a procedure to determine the time and magnitude of flow (i.e., the flow hydrograph) at a point on a watercourse from known or assumed hydrographs at one or more points upstream. In broad sense, flow routing may be considered as an analysis to trace the flow through a hydrologic system, given the input.

Muskingum-Cunge routing model is selected due to its preference on the manual (shown below) when there is no gauged data and if the flood will go out of bank, into floodplain which is similar to Dechatu case.

This routing method is based on the combination of the conservation of mass and the diffusion representation of the conservation of momentum. It is sometimes referred to as a variable coefficient method because the routing parameters are recalculated every time step based on channel properties and the flow depth.

The following is given on the manual for selection criteria.

No observed hydrograph data available for calibration	Kinematic wave; Muskingum-Cunge
Significant backwater will influence discharge hydrograph	Modified Puls
Flood wave will go out of bank, into floodplain	Modified Puls, Muskingum-Cunge with 8-point cross section
Channel slope > 0.002 and $\frac{TS_o u_o}{d_o} \geq 171$	Any
Channel slopes from 0.002 to 0.0004 and $\frac{TS_o u_o}{d_o} \geq 171$	Muskingum-Cunge; modified Puls; Muskingum
Channel slope < 0.0004 and $TS_o\left(\frac{g}{d_o}\right)^{1/2} \geq 30$	Muskingum-Cunge
Channel slope < 0.0004 and $TS_o\left(\frac{g}{d_o}\right)^{1/2} < 30$	None

The length is the total length of the reach element. The slope is the average slope for the whole reach. These values are measured from the 57m resolution DEM using HEC-GeoHMS. The Manning's n, roughness coefficient, is the average value for the whole reach. This value is estimated from Ven Te Chow, David R. Maidment, Larry W.Mays. APPLIED HYDROLOGY.

Five options are provided for specifying the cross section shape: circle, eight point, rectangle, trapezoid, and triangle. The eight point shape is chosen since it fits the actual ground condition which requires cross section simplified with only eight station-elevation values. The cross section is configured to represent the main channel plus left and right overbank areas. With this, a representative cross section is described for the routing reach, which requires 8 pairs of x, y (distance, elevation) values. A separate Manning's n value is entered for each overbank.

The cross section created in the Paired Data Manager to use in the reach. Points labeled 3 and 6 represent the left and right banks of the channel at the representative cross section. Points 4 and 5 are within the channel. Points 1 and 2 represents the left overbank, and points 7 and 8 represent the right overbank.

Figure 4-8: Eight points channel geometry

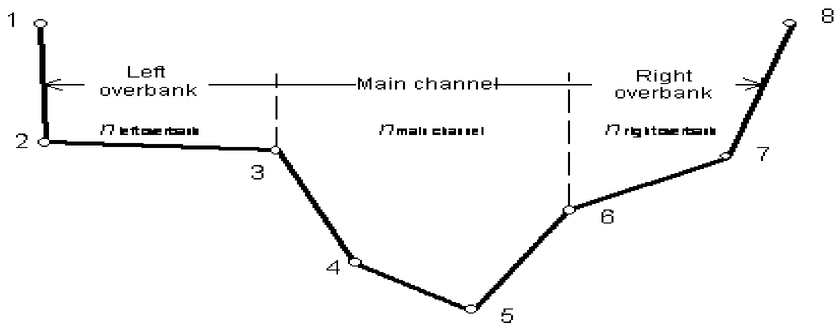
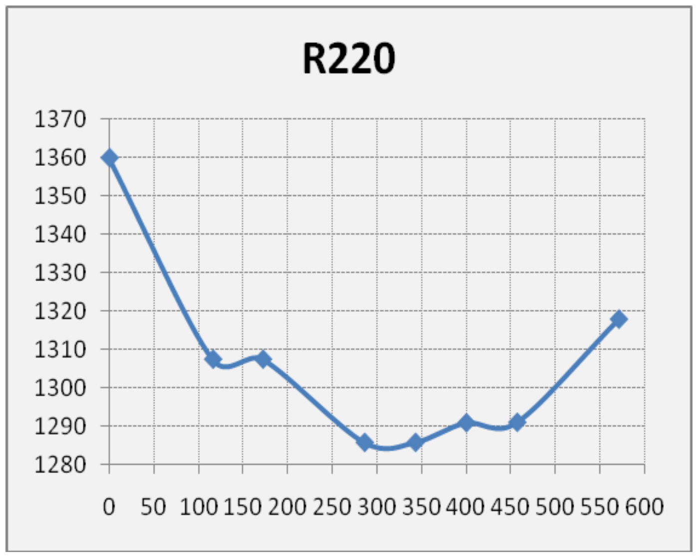
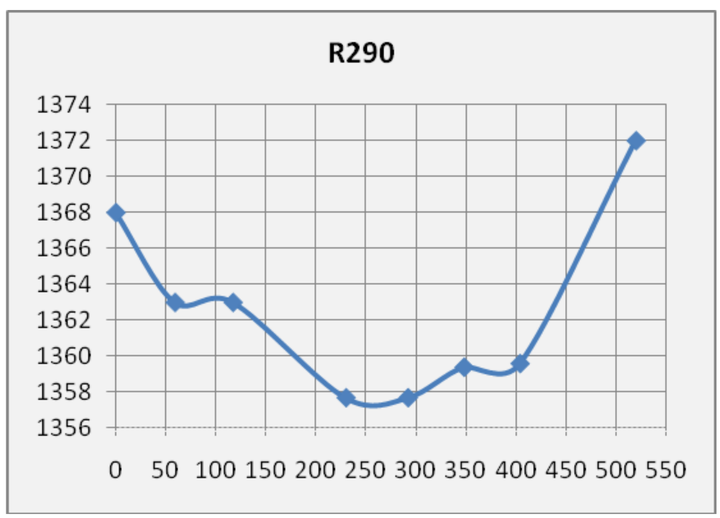


Table 4-6: Cross section data

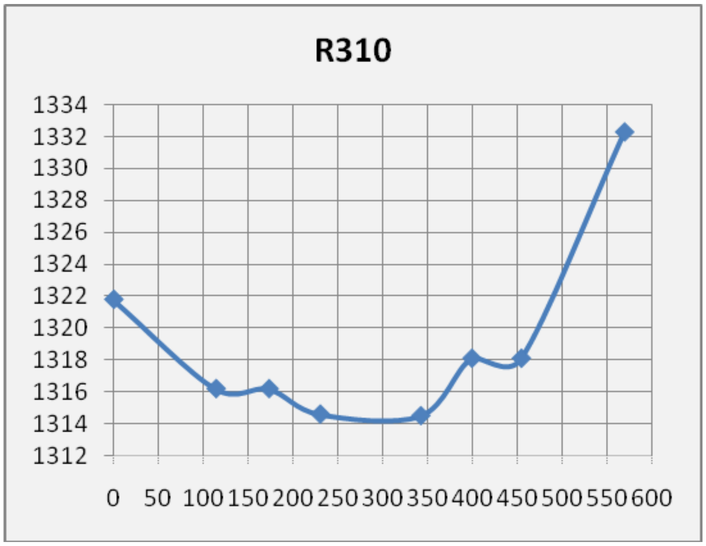
1. R220		
Points /station	**Distance (m)**	**Elevation (m)**
1	0	1359.8
2	116	1307.5
3	172	1307.5
4	286	1285.9
5	343	1285.9
6	400	1291
7	457	1291.1
8	571	1317.9



3. R290		
Points /station	**Distance (m)**	**Elevation (m)**
1	0	1368
2	59	1363
3	117	1363
4	230	1357.7
5	292	1357.7
6	348	1359.4
7	404	1359.6
8	520	1372



5. R310		
Points /station	**Distance (m)**	**Elevation (m)**
1	0	1527.2
2	639	1416.4
3	693	1417
4	755	1411
5	867	1412
6	925	1417
7	979	1417
8	1212	1440



4.1.3. Junction

A junction is an element with one or more inflows and only one outflow. All inflow is added together to produce the outflow by assuming zero storage at the junction. It is usually used to represent a river or stream confluence. The junction element does not have any special data or properties.

4.2. Meteorological Model

Meteorological model is one of the main components in a project. There are seven different precipitation methods. Some precipitation methods require parameter data for each sub-basin. Other methods use the same data for all sub-basins.

Two methods are adopted as an option for precipitation modelling since the concern is flood that uses more than one nearby metrological stations. These are:

i. Frequency storm
ii. Gage weights

EasyFit is used for the analysis allowing fitting probability distributions.

4.2.1. Frequency Analysis

Rainfall frequency analysis was used as a key input to the calculation of flood frequency and magnitude. Certain hydrologic procedures use rainfall and rainfall frequency as the basic input rather than flood frequency. It is commonly assumed that the 10-year rainfall will produce the 10-year flood. Depending on antecedent soil moisture conditions, and other hydrologic parameters, there may not be a direct relationship between rainfall and flood frequency (ERA Drainage Design Manual, 2002). The rainfall frequency analysis was carried out using the available 24 hour annual maximum rainfall data for the rain gauge stations situated in the nearby areas. It was necessary to combine data from different stations in order to arrive at an acceptable representative record.

i. Dire Dawa RF

Table 4-7: Dire Dawa goodness of fit summary

Goodness of Fit - Summary

#	Distribution	Kolmogorov Smirnov		Anderson Darling		Chi-Squared	
		Statistic	Rank	Statistic	Rank	Statistic	Rank
1	Exponential	0.38711	10	11.346	10	53.013	10
2	Exponential (2P)	0.24974	9	4.9485	9	19.521	9
3	Gamma	0.04983	4	0.1318	4	1.052	2
4	Gamma (3P)	0.04456	1	0.11527	2	1.1924	4
5	Gumbel Max	0.05276	5	0.16463	6	1.0583	3
6	Lognormal	0.04476	2	0.12306	3	1.2189	6
7	Lognormal (3P)	0.04706	3	0.11474	1	0.66363	1
8	Normal	0.07997	8	0.4937	7	3.0131	7
9	Weibull	0.06908	7	0.62722	8	1.2008	5
10	Weibull (3P)	0.05909	6	0.15371	5	3.3005	8

Descriptive Statistics

Statistic	Value
Sample Size	52
Range	90.1
Mean	54.475
Variance	319.36
Std. Deviation	17.871
Coef. of Variation	0.32805
Std. Error	2.4782
Skewness	0.81427
Kurtosis	1.0024

Using the Chi-Squared test ranking, the Dire Dawa rainfall data fits Lognormal 3 parameter distribution.

Table 4-8: Dire Dawa station rainfall T year's extreme rainfall magnitude (X_T)

Return Period - T (year)	X_T (Using log-normal distribution)
10	78.65
50	101.25
100	110.69
500	132.57
1000	142.09
2500	154.82
5000	164.6
7500	170.38
10000	174.51

ii. Dengego RF Analysis

Table 4-9: Dengego RF goodness of fit

#	Distribution	Kolmogorov Smirnov		Anderson Darling		Chi-Squared	
		Statistic	Rank	Statistic	Rank	Statistic	Rank
1	Exponential	0.38709	8	6.0266	8	28.176	8
2	Exponential (2P)	0.29236	7	4.7048	7	7.0419	6
3	Gamma	0.14234	4	0.61797	4	2.5419	5
4	Gamma (3P)	0.11487	2	0.35653	3	2.4714	4
5	Gumbel Max	0.17415	6	1.2633	6	2.424	2
6	Lognormal	0.15688	5	0.808	5	12.411	7
7	Lognormal (3P)	0.11817	3	0.33812	2	2.4118	1
8	Normal	0.11142	1	0.29988	1	2.4466	3

Again, using the Chi-Squared test ranking, Dengego rainfall data fits Lognormal 3 parameter distribution. Annual Maximum Daily Rainfall (mm) for selected return periods (X_T):

Table 4-10: Dengego station rainfall T year's extreme rainfall magnitude (X_T)

Return Period - T (year)	Using log-normal distribution, XT
10	89.01
50	114.67
100	125.4
500	150.26
1000	161.08
2500	175.56
5000	186.68
7500	193.25
10000	197.95

iii. Kersa RF Analysis

Table 4-11: Dengego RF goodness of fit

#	Distribution	Kolmogorov Smirnov		Anderson Darling		Chi-Squared	
		Statistic	Rank	Statistic	Rank	Statistic	Rank
1	Exponential	0.33908	10	1.4359	7	N/A	
2	Exponential (2P)	0.13912	4	3.4294	9	N/A	
3	Gamma	0.16728	6	0.31662	4	N/A	
4	Gamma (3P)	0.21572	8	2.6983	8	N/A	
5	Gumbel Max	0.16428	5	0.29706	3	N/A	
6	Lognormal	0.12619	2	0.1719	2	0.01294	2
7	Lognormal (3P)	0.11885	1	0.16692	1	8.8185E-8	1
8	Normal	0.21884	9	0.66418	5	N/A	
9	Weibull	0.1377	3	0.96245	6	0.1118	3
10	Weibull (3P)	0.19147	7	3.9609	10	N/A	

Similar to the above two rainfall stations, Kersa rainfall data fits Lognormal 3 parameter distribution. Annual Maximum Daily Rainfall (mm) for selected return periods (X_T):

Table 4-12: Kersa station rainfall T year's extreme rainfall magnitude (X_T)

Summary 24 hrs	
Return Period - T (year)	**X_T (Using log-normal distribution)**
10	115.77
50	170.13
100	194.88

Table 4-13: Summary of annual maximum daily rainfall (mm) for selected return periods

	Dire Dawa	**Dengego**	**Kersa**
Mean	54.48	61.22	68.82
Max	113.10	92.80	166
Min	23.00	22.80	28.5
St. Dev	17.87	17.23	39.03
T-2	51.73	58.47	61.13
T-5	68.11	77.05	92.97
T-10	78.65	89.01	115.77
T-25	91.70	103.82	146.28
T-50	101.25	114.67	170.13
T-100	110.69	125.40	194.88
T-500	132.57	150.26	Insufficient data
T-1000	142.09	161.08	Insufficient data
T-2500	154.82	175.56	Insufficient data
T-5000	164.60	186.68	Insufficient data
T-7500	170.38	193.25	Insufficient data

4.2.2. Using Different Mode of Storms

Though rainfalls are registered over 24 hours, the actual duration of the storm will normally be much less than 24 hours and will include all or nearly all of the recorded 24 hour rainfall amount. For this reason, three options are considered for meteorlogic modelling. These are:

i. Adopting the annual maximum daily rainfall as 24 hours duration storm

ii. Using ERA Intensity-frequency-duration curve

iii. Adopting the annual maximum daily rainfall as 6 hours duration storm

4.2.2.1 Using 24 hours Duration Storms

i. Dire Dawa RF

The 50 years return period daily maximum rainfall, 101.25mm, is changed to 24hrs incremental rainfall.

Using, p = M * square root (T), ► M = p / square root (T), ► M = 20.67mm

Table 4-14: 24hrs incremental rainfall for Dire Dawa station 50 years return period daily maximum rainfall

Time	Hourly Distributed Cumulative P P = M * sqrt (T_i)	Incremental Depth	Time Interval	Precipitation
hr	mm	mm	hr	mm
1	20.7	20.7	0--1	2.13
2	29.2	8.56	1--2	2.23
3	35.8	6.57	2--3	2.34
4	41.3	5.54	3--4	2.47
5	46.2	4.88	4--5	2.63
6	50.6	4.41	5--6	2.81
7	54.7	4.06	6--7	3.05
8	58.5	3.78	7--8	3.35
9	62	3.55	8--9	4.06
10	65.4	3.35	9--10	4.88
11	68.5	3.19	10--11	6.57
12	71.6	3.05	11--12	20.67
13	74.5	2.92	12--13	8.56
14	77.3	2.81	13--14	5.54
15	80	2.71	14--15	4.41
16	82.7	2.63	15--16	3.78
17	85.2	2.54	16--17	3.55
18	87.7	2.47	17--18	3.19
19	90.1	2.4	18--19	2.92
20	92.4	2.34	19--20	2.71
21	94.7	2.28	20--21	2.54
22	96.9	2.23	21--22	2.4
23	99.1	2.18	22--23	2.28
24	101.3	2.13	23--24	2.18

ii. Dengego RF Analysis

The 50 years return period daily maximum rainfall, 114.67mm, is changed to 24hrs incremental rainfall.

Table 4-15: 24hrs incremental rainfall for Dengego station 50 years return period daily maximum rainfall

Time	**Hourly Distributed Cumulative P P = M * sqrt (Ti)**	**Incremental Depth**	**Time Interval**	**Precipitation**
hr	mm	mm	hr	mm
1	23.4	23.4	0--1	2.41
2	33.1	9.7	1--2	2.52
3	40.5	7.44	2--3	2.65
4	46.8	6.27	3--4	2.8
5	52.3	5.53	4--5	2.97
6	57.3	5	5--6	3.19
7	61.9	4.59	6--7	3.45
8	66.2	4.28	7--8	3.8
9	70.2	4.02	8--9	4.59
10	74	3.8	9--10	5.53
11	77.6	3.61	10--11	7.44
12	81.1	3.45	11--12	23.41
13	84.4	3.31	12--13	9.7
14	87.6	3.19	13--14	6.27
15	90.7	3.07	14--15	5
16	93.6	2.97	15--16	4.28
17	96.5	2.88	16--17	4.02
18	99.3	2.8	17--18	3.61
19	102	2.72	18--19	3.31
20	104.7	2.65	19--20	3.07
21	107.3	2.59	20--21	2.88
22	109.8	2.52	21--22	2.72
23	112.3	2.47	22--23	2.59
24	114.7	2.41	23--24	2.47

iii. Kersa RF Analysis

The 50 years return period daily maximum rainfall, 170.13mm, is changed to 24hrs incremental rainfall.

Table 4-16: 24hrs incremental rainfall for Kersa station 50 years return period daily maximum rainfall

Time	**Hourly Distributed Cumulative P** **P = M * sqrt (T_i)**	**Incremental Depth**	**Time Interval**	**Precipitation**
hr	mm	mm	hr	mm
1	34.7	34.7	0--1	3.58
2	49.1	14.38	1--2	3.74
3	60.1	11.04	2--3	3.93
4	69.5	9.31	3--4	4.15
5	77.7	8.2	4--5	4.41
6	85.1	7.41	5--6	4.73
7	91.9	6.82	6--7	5.12
8	98.2	6.34	7--8	5.64
9	104.2	5.96	8--9	6.82
10	109.8	5.64	9--10	8.2
11	115.2	5.36	10--11	11.04
12	120.3	5.12	11--12	34.73
13	125.2	4.91	12--13	14.38
14	129.9	4.73	13--14	9.31
15	134.5	4.56	14--15	7.41
16	138.9	4.41	15--16	6.34
17	143.2	4.28	16--17	5.96
18	147.3	4.15	17--18	5.36
19	151.4	4.04	18--19	4.91
20	155.3	3.93	19--20	4.56
21	159.1	3.84	20--21	4.28
22	162.9	3.74	21--22	4.04
23	166.5	3.66	22--23	3.84
24	170.1	3.58	23--24	3.66

4.2.2.2 *Using ERA Intensity -duration-frequency Curve*

ERA has developed rainfall intensity-duration curves by dividing the country into different hydrological regions as shown below.

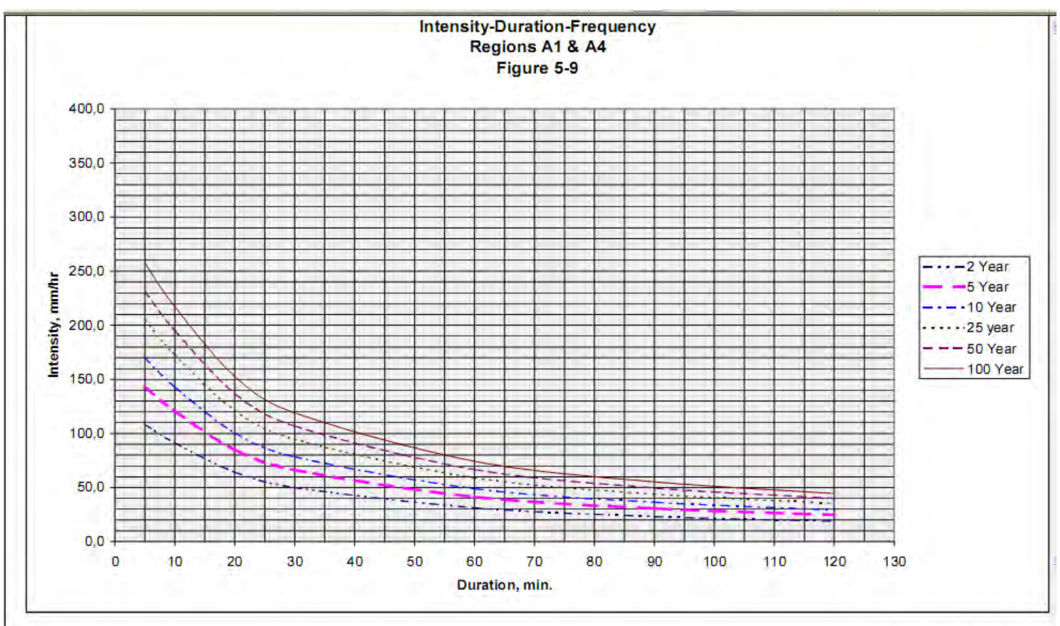
Table 4-17: ERA hydrological regions

Table 5-2 Meteorology Stations

Meteorological Region	Station	Years of Record	Meteorological Region	Station	Years of Record
A1	Axum	18	B	Bedele	19
	Mekele	35		Gore	45
	Maychew	24		Nekempte	27
A2	Gondar	40		Jima	45
	Debre Tabor	22		Arba Minch	11
	Bahir Dar	35		Sodo	28
	Debre Markos	44		Awasa	26
	Fitche	25	C	Kombolcha	46
	Addis Ababa	33		Woldiya	23
A3	Nazareth	40		Sirinka	17
	Kulumsa	31	D1	Gode	29*
	Robe/Bale	19		Kebri Dihar	38
A4	Metehara	28	D2	Kibre Mengist	24
	Dire Dawa	46		Negele	45
	Mieso	35		Moyale	18
* max 24 hour rainfall not given				Yabelo	34

Years of record through 1997

Figure 4-9: ERA drainage manual rainfall intensity-duration curves



Page 5-38 *Ethiopian Roads Authority*

The hyetograph produced from the IDF curves using the alternate block method as shown below.

Table 4-18: Hyetograph using alternate block method

Duration Min.	**Intensity** mm/hr	**Cumulative Depth** mm	**Incremental Depth** mm	**Time** Min.	**Precipitation** mm
10	195	32.5	32.5	0 - 10	2.17
20	135	45	12.5	20-Oct	3
30	107	53.5	8.5	20 -30	1.83
40	91	60.67	7.17	30 - 40	4.33
50	78	65	4.33	40 - 50	8.5
60	67	67	2	50 - 60	32.5
70	59	68.83	1.83	60 - 70	12.5
80	54	72	3.17	70 - 80	7.17
90	50	75	3	80 - 90	2
100	46	76.67	1.67	90 - 100	3.17
110	43	78.83	2.17	100 - 110	1.67
120	40	80	1.17	110 - 120	1.17

4.2.2.3 Adopting the Annual Maximum Daily Rainfall as 6 hours Duration Storm

i. Dire Dawa RF

The 50 years return period daily maximum rainfall, 101.25mm, is changed to 6hrs incremental rainfall.

Using, p = M * square root (T), ► M = p /square root (T), ► M =**41.34**mm

Table 4-19: 6hrs incremental rainfall for Dire Dawa station 50 years return period daily maximum rainfall

Time	**Hourly Distributed Cumulative P** **P = M * sqrt (T_i)**	**Incremental Depth**	**Time Interval**	**Precipitation**
hr	mm	mm	hr	
1	41.3	41.3	0--1	9.76
2	58.5	17.12	1--2	13.14
3	71.6	13.14	2--3	41.34
4	82.7	11.08	3--4	17.12
5	92.4	9.76	4--5	11.08
6	101.3	8.82	5--6	8.82

ii. Dengego RF Analysis

The 50 years return period daily maximum rainfall, 114.67mm, is changed to 6hrs incremental rainfall. Using, p = M * square root (T), ► M = p /square root (T)), ► M = 46.82mm

Table 4-20: 6hrs incremental rainfall for Dengego station 50 years return period daily maximum rainfall

Time	Hourly Distributed Cumulative P P = M * sqrt (T_i)	Incremental Depth	Time Interval	Precipitation
hr	mm	mm	hr	
1	46.8	46.8	0--1	11.05
2	66.2	19.39	1--2	14.88
3	81.1	14.88	2--3	46.82
4	93.6	12.54	3--4	19.39
5	104.7	11.05	4--5	12.54
6	114.7	9.99	5--6	9.99

iii. Kersa RF Analysis

The 50 years return period daily maximum rainfall, 114.67mm, is changed to 6hrs incremental rainfall. Using, p = M * square root (T), ► M = p /square root (T)), ► M = mm

Table 4-21: 24hrs incremental rainfall for Kersa station 50 years return period daily maximum rainfall

Time	Hourly Distributed Cumulative P P = M * sqrt (T_i)	Incremental Depth	Time Interval	Precipitation
hr	mm	mm	hr	
1	69.5	69.5	0--1	16.4
2	98.2	28.77	1--2	22.07
3	120.3	22.07	2--3	69.45
4	138.9	18.61	3--4	28.77
5	155.3	16.4	4--5	18.61
6	170.1	14.82	5--6	14.82

4.3. Runoff Data

No runoff data are available for Dire Dawa catchments. The Dechatu River has been gauged at U/s of the town for the years 2003 and 2004, but only water level data are available. No discharge measurements have been carried out. These data are of no use in the flood frequency analysis and calibration. Therefore, gaged catchment nearby and hydrologically similar to Dire Dawa catchment named Erer catchment is used for the calibration which is discussed on the next chapter.

5. CALIBRATION

Each model that is included in the program has parameters. The value of each parameter must be specified to use the model for estimating runoff or routing hydrographs. Earlier chapters identified the parameters and described how they could be estimated from various watershed and channel properties.

However, some of the models that are included have parameters that cannot be estimated by observation or measurement of channel or watershed characteristics.

If rainfall and stream flow observations are available, calibration is the answer. Calibration uses observed hydrometeorological data in a systematic search for parameters that yield the best fit of the computed results to the observed runoff. This search is often referred to as optimization.

Given initial estimates of the parameters, the models included in the program can be used with the observed boundary conditions (rainfall or upstream flow) to compute the output, either the watershed runoff hydrograph or a channel outflow hydrograph. At this point, the program compares the computed hydrograph to the observed hydrograph. The goal of this comparison is to judge how well the model "fits" the real hydrologic system.

If the fit is not satisfactory, the program systematically adjusts the parameters and reiterates.

When the fit is satisfactory, the program will report the optimal parameter values. The presumption is that these parameter values then can be used for runoff or routing computations that are the goal of the flood runoff analyses.

This chapter describes how observed stream flow can be used to automatically estimate parameters. A process called optimization is used that begins from initial parameter estimates and adjusts them so that the simulated results match the observed stream flow as closely as possible. Two different search algorithms are provided that move from the initial estimates to the final best estimates. A variety of objective functions are provided to measure the goodness of fit between the simulated and observed stream flow in different ways. While parameter estimation using optimization does not produce perfect results, it can be a valuable aid when calibrating models.

Erer hydrometeorological station is used for the calibration due to the hydrological similarity of the catchments assigned by the Ministry of Water Resources. Then the parameters are transferred to Dire Dawa catchment.

5.1. Watershed Physical Description

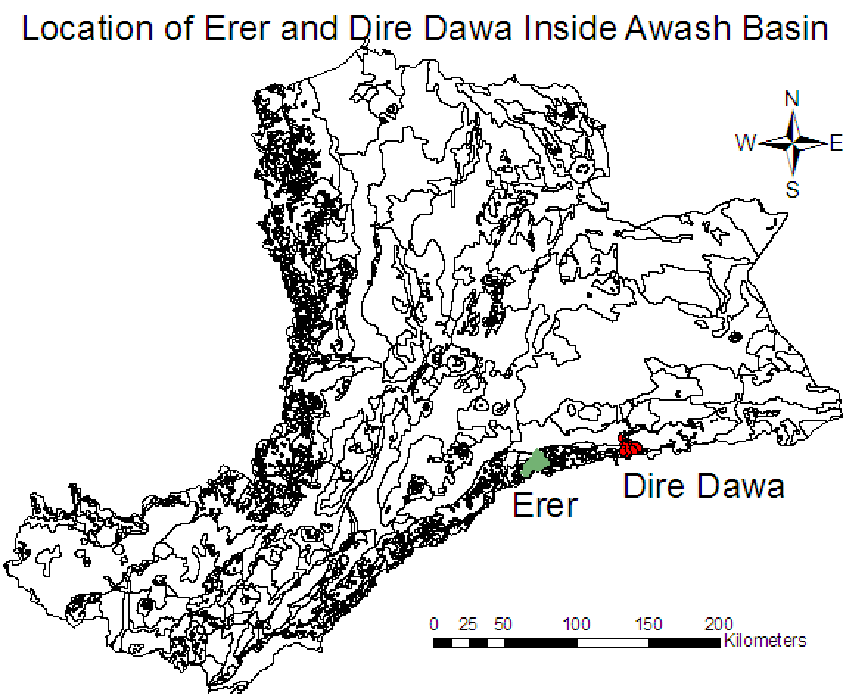
The Erer catchment is not only located in Awsh basin but also near to Dire Dawa.

Geographically the Erer hydrometeorological station is 9031'N and 41020'E.

The elevation of Erer Catchment Varies from about 2524m to 1183m amsl. Slopes range from almost 0.01 to 0.09 (i.e 1 to 9%)

The elevation Dechatu Catchment varies from about 2,300m in the escarpment of the eastern edge of the Hararghe highlands to about 1,200m in the town. Stream slopes range from 0.02 to 0.06 (i.e 2 to 6%).

Figure 5-1: Location of Erer and Dire Dawa within Awash basin



5.2. Sub-Basins Modelling for Erer Catchment

To consider the spatial analysis in the catchment, it is divided into four sub-basins as shown below.

Figure 5-2: Erer Catchment with its sub-basins

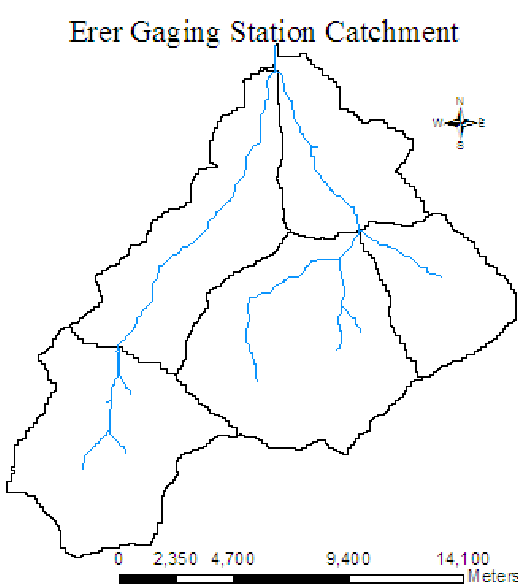
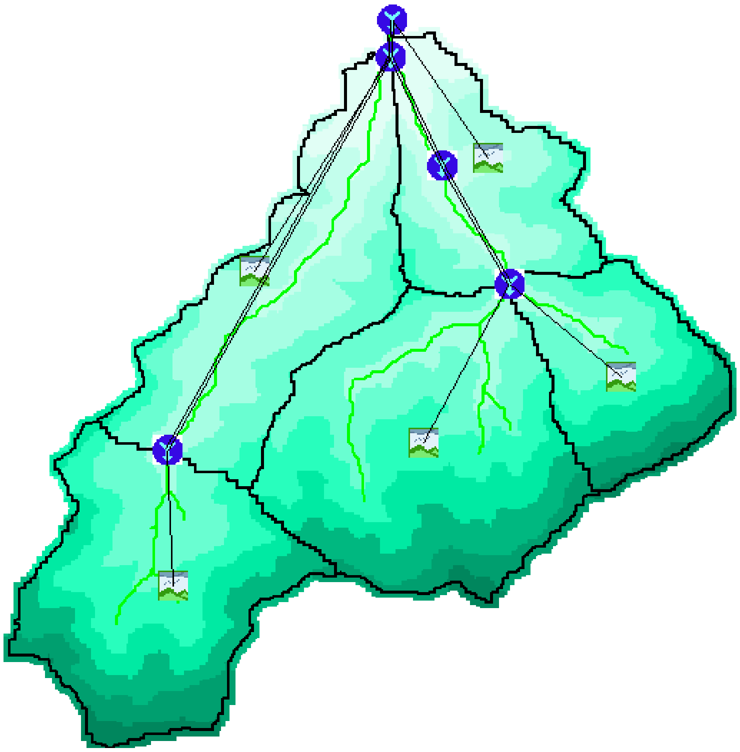


Figure 5-3: Erer Catchment with its sub-basins



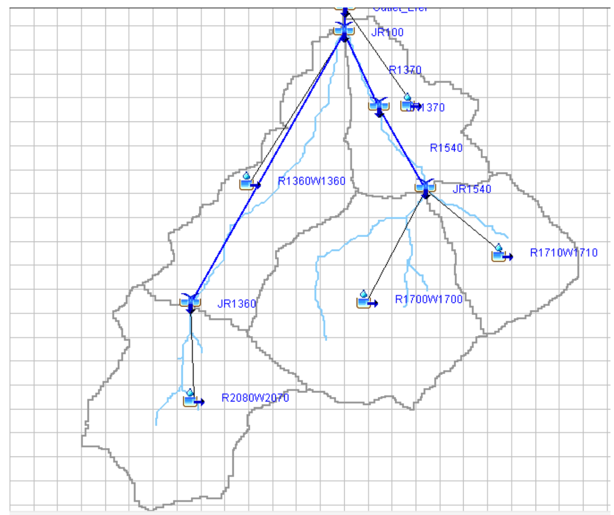


Figure 5-4 Erer Catchments Hec-HMS Modelling

5.2.1. Loss

Since it is for calibration purpose, the loss method should be similar to the target area method which is SCS Curve Number Loss method.

Table 5-1: Sub-Basin Catchment Areas of Dechatu Basin

Name of Sub_basin	Area (km^2)
R100W100	29.829
R1700W1700	60.776
R1360W1360	45.213
R1710W1710	25.927
R2080W2070	48.511

The three parameters to compute the loss required are shown below.

- Curve Number
- Initial Abstraction
- Impervious (%) (negligible for our case)

5.2.1.1 Curve Number

To compute the Curve Number (CN), soil and land use are required which are shown below.

Table 5-2: Erer Soil Data in the Required Sub_Catchment Range

Name	Soil_type	Hydrologic Soil Group	Area (km^2)
R100W100	Calcaric fluvisols	B	2.443
	Leptosols	D	15.001
	Haplic xerosols	B	12.385
R1700W1700	Chromic cambisols	B	25.508
	Leptosols	D	22.469
	Haplic xerosols	B	11.593
R1360W1360	Calcaric fluvisols	B	2.622
	Leptosols	D	15.089
	Haplic xerosols	B	20.059
	Chromic cambisols	B	7.443
R1710W1710	Leptosols	D	14.172
	Haplic xerosols	B	6.186
	Chromic cambisols	B	5.569
R2080W2070	Leptosols	D	25.626
	Haplic xerosols	B	5.638
	Chromic cambisols	B	17.036

Figure 5-5: Erer catchment Soil map

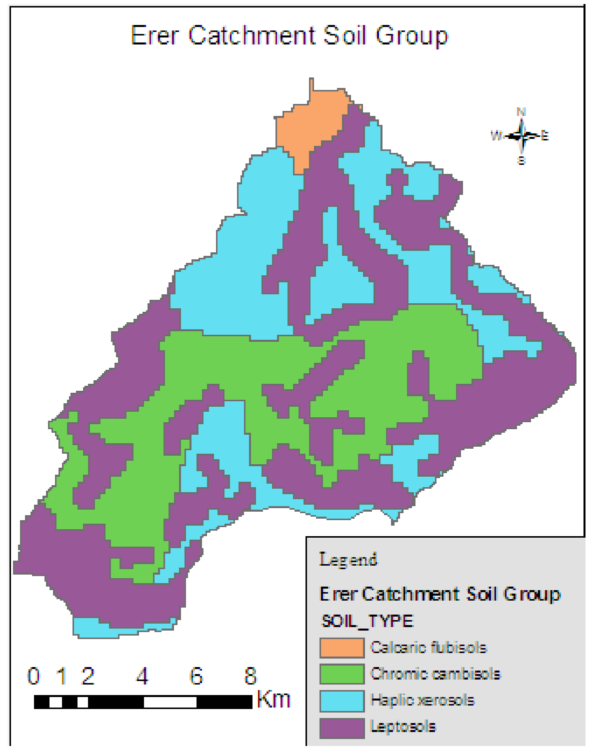


Table 5-3: Erer Catchment Landuse Data

Landuse	Area (Km^2)
Cropland with Grassland Savanna	116.863
Grassland with Cropland	0.544
Cropland with Grassland Savanna	14.697
Montane Dry Sparse/ Grassland	18.661
Cropland with Grassland Savanna	5.414
Grassland/ woodland Mosaic	43.771

Figure 5-6: Erer catchment landuse

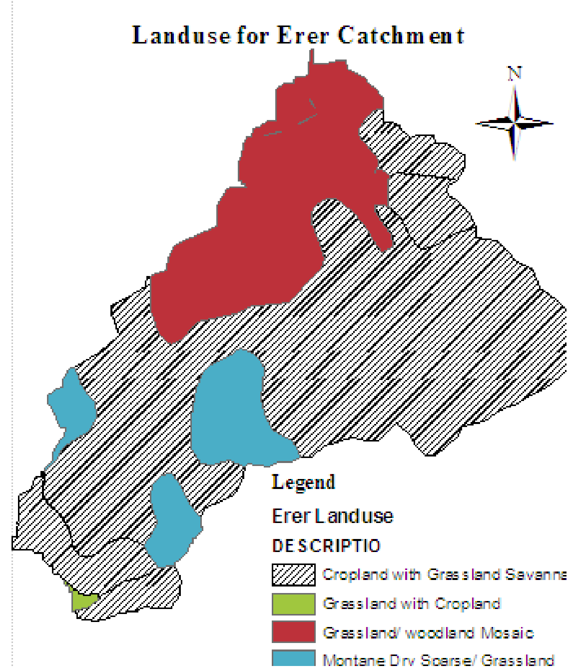


Table 5-4: Sub-Basins and the corresponding land use

Name	Description	Area (km^2)
R100W100	Cropland with Grassland Savanna	17.543
	Grassland/ woodland Mosaic	12.306
R1700W1700	Cropland with Grassland Savanna	41.699
	Grassland/ woodland Mosaic	0.212
	Montane Dry Sparse/ Grassland	8.417
R1360W1360	Cropland with Grassland Savanna	13.213
	Grassland/ woodland Mosaic	30.949
	Montane Dry Sparse/ Grassland	1.014
R1710W1710	Cropland with Grassland Savanna	25.788
R2080W2070	Cropland with Grassland Savanna	37.925
	Montane Dry Sparse/ Grassland	9.097
	Grassland with Cropland	0.540

Figure 5-7: Erer catchment soil type and land use intersection

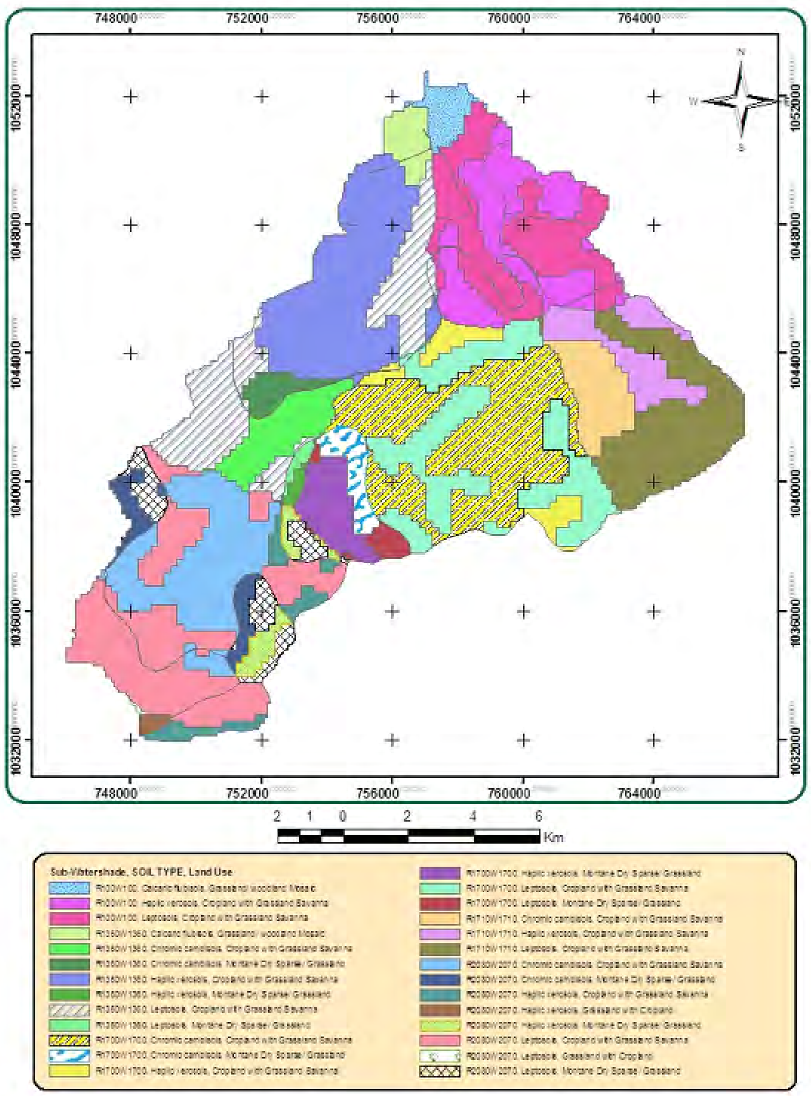


Table 5-5: Sub-basins Curve Number

Name	Soil_type	Landuse	Area (km^2)
R100W100	Leptosols	Cropland with Grassland Savanna	8.358
		Grassland/ woodland Mosaic	6.537
	Haplic xerosols	Cropland with Grassland Savanna	9.061
		Grassland/ woodland Mosaic	3.24
	Calcaric flubisols	Grassland/ woodland Mosaic	2.437
R1700W1700	Haplic xerosols	Cropland with Grassland Savanna	8.708
		Montane Dry Sparse/ Grassland	4.827
		Grassland/ woodland Mosaic	0.12
	Leptosols	Cropland with Grassland Savanna	17.518
		Montane Dry Sparse/ Grassland	1.491
		Grassland/ woodland Mosaic	0.081
	Chromic cambisols	Cropland with Grassland Savanna	24.449
		Montane Dry Sparse/ Grassland	2.938
R1360W1360	Leptosols	Cropland with Grassland Savanna	7.432
		Montane Dry Sparse/ Grassland	0.507
		Grassland/ woodland Mosaic	7.048
	Haplic xerosols	Cropland with Grassland Savanna	0.18
		Grassland/ woodland Mosaic	19.283
		Montane Dry Sparse/ Grassland	0.492
	Calcaric flubisols	Grassland/ woodland Mosaic	2.61
	Chromic cambisols	Cropland with Grassland Savanna	5.481
		Montane Dry Sparse/ Grassland	0.015
		Grassland/ woodland Mosaic	1.889
R1710W1710	Haplic xerosols	Cropland with Grassland Savanna	6.13
	Chromic cambisols	Cropland with Grassland Savanna	5.569
	Leptosols	Cropland with Grassland Savanna	14.089
R2080W2070	Leptosols	Cropland with Grassland Savanna	20.705
		Grassland with Cropland	0.076
	Haplic xerosols	Cropland with Grassland Savanna	2.843
		Grassland with Cropland	0.461
	Chromic cambisols	Cropland with Grassland Savanna	14.078

5.2.1.2 Maximum Retention, S

As discussed above,

$$S = \frac{25400}{CN} - 254 \qquad \text{(SI)}$$

And, $I_a = 0.2S$,

When the CN is specified, the program solves the above equations to find Ia.

Table 5-6: Weighted Curve Number

ame	Soil type	Soil Group	Land use	CN_i	Area (km^2)	Area $\sum A_i$ km^2	CN_c
R100W100	Leptosols	D	Cropland with Grassland Savanna	84	8.358		
			Grassland/ woodland Mosaic	80	6.537		
	Haplic xerosols	B	Cropland with Grassland Savanna	69	9.061		
			Grassland/ woodland Mosaic	61	3.24		
	Calcaric flubisols	B	Grassland/ woodland Mosaic	61	2.437	29.633	74.125
R1700W1700	Haplic xerosols	B	Cropland with Grassland Savanna	69	8.708		
			Montane Dry Sparse/ Grassland	58	4.827		
			Grassland/ woodland Mosaic	61	0.12		
	Leptosols	D	Cropland with Grassland Savanna	84	17.518		
			Montane Dry Sparse/ Grassland	78	1.491		
			Grassland/ woodland Mosaic	80	0.081		
	Chromic cambisols	B	Cropland with Grassland Savanna	69	24.449		
			Montane Dry Sparse/ Grassland	58	2.938	60.132	72.171

Name	Soil_type	Soil Group	Landuse	CN_i	Area (km^2)	Area $\sum A_i$ km^2	CN_c
R1360W1360	Leptosols	D	Cropland with Grassland Savanna	84	7.432		
			Montane Dry Sparse/ Grassland	78	0.507		
			Grassland/ woodland Mosaic	80	7.048		
	Haplic xerosols	B	Cropland with Grassland Savanna	69	0.18		
			Grassland/ woodland Mosaic	61	19.283		
			Montane Dry Sparse/ Grassland	58	0.492		
	Calcaric flubisols	B	Grassland/ woodland Mosaic	61	2.61		
	Chromic cambisols	B	Cropland with Grassland Savanna	69	5.481		
			Montane Dry Sparse/ Grassland	58	0.015		
			Grassland/ woodland Mosaic	61	1.889	44.937	68.95
R1710W1710	Haplic xerosols	B	Cropland with Grassland Savanna	69	6.13		
	Chromic cambisols	B	Cropland with Grassland Savanna	69	5.569		
	Leptosols	D	Cropland with Grassland Savanna	84	14.089	25.788	77.195
R2080W2070	Leptosols	D	Cropland with Grassland Savanna	84	26.502		
			Grassland with Cropland	84	0.097		
	Haplic xerosols	B	Cropland with Grassland Savanna	69	3.639		
			Grassland with Cropland	69	0.59		
	Chromic cambisols	B	Cropland with Grassland Savanna	69	18.02	48.847	77.168

5.2.1.3 Transformation

Table 5-7: Basin lags for the total catchment

River / Basin	U/S El (m)	D/S El (m)	Length (m)	$t_{shallow}$			Basin Lag (t_{lag}) min
				S	V (m/s)	$t_{shallow}$ (min.)	
Erer	2524.000	1183.000	25977.594	0.052	3.666	118.107	70.864

5.2.2. Base Flow

For the calibration area, Erer, 0.412 m3/sec the average of the months February and May from about 8 years recorded data taken as constant base flow. Meteorological Model

For the calibration, Erer station eight years (1982 to 1990) daily rainfall data is adopted for Gage weights precipitation modeling.

5.3. Runoff Data

Erer station eight years (1982 to 1990) daily recorded data are used for the calibration. These data were recorded before eighteen years. However, the hydrological data taken from the Ministry of Water Resources does not resemble the actual condition at site. As per the hydrologist assigned for this catchment, the actual values are highly greater than these values. Since the gauge station is not automatic, there is clearly personal error while recording.

5.4. Optimization

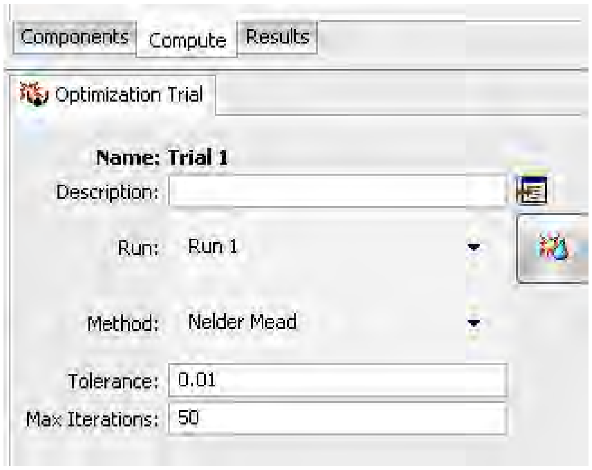
The model HEC-HMS describes how observed stream flow can be used to automatically estimate parameters that can only be estimated by comparing computed results to observed results such as observed stream flow. A process called optimization is used that begins from initial parameter estimates and adjusts them so that the simulated results match the observed stream flow as closely as possible. While parameter estimation using optimization does not produce perfect results, it can be a valuable aid when calibrating models. The iterative parameter estimation procedure used by the program is often called optimization.

A variety of objective functions are provided to measure the goodness of fit between the simulated and observed stream flow in different ways. The quantitative measure of the goodness-of-fit between the computed result from the model and the observed flow is called the objective function. An objective function measures the degree of variation between computed and observed hydrographs. It is equal to zero if the hydrographs are exactly identical. A minimum objective function is obtained when the parameter values best able to reproduce the observed hydrograph are found. Constraints are set to insure that unreasonable parameter values are not used.

Two different search algorithms are provided that move from the initial estimates to the final best estimates.

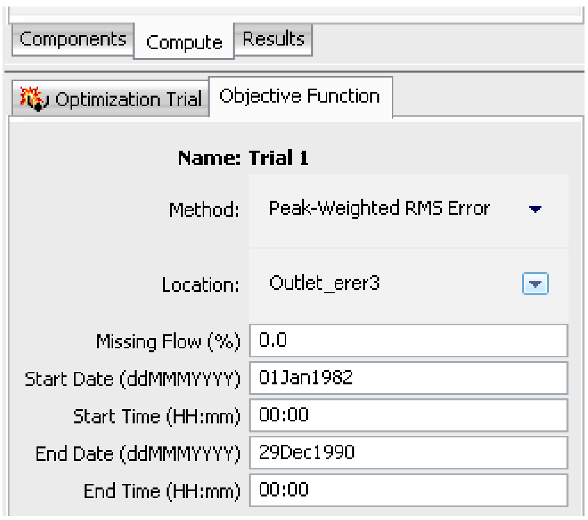
5.4.1. Simulation Run and Search Method

Two search methods are available for minimizing the objective function and finding optimal parameter values. The univariate gradient method evaluates and adjusts one parameter at a time while holding other parameters constant. The Nelder and Mead method uses a downhill simplex to evaluate all parameters simultaneously and determine which parameter to adjust. As can be seen below the second method is preferred for this simulation for easiness to simulate all parameters at once.



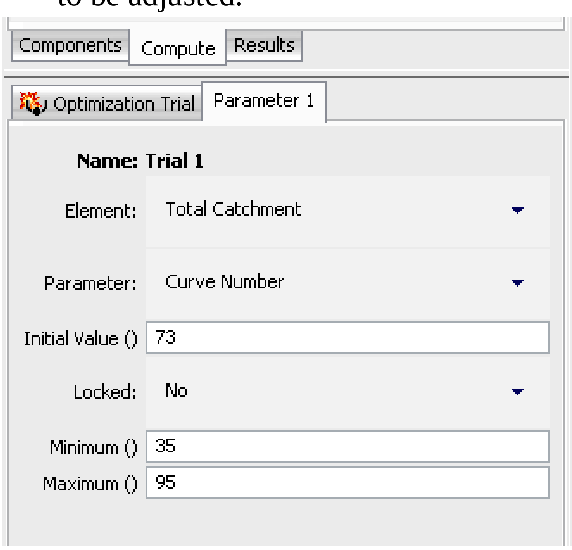
5.4.2. Objective Function

The objective function measures the goodness-of-fit between the computed outflow and observed stream flow at the selected element.



5.4.3. Parameters

The parameters that will be automatically estimated must be at the selected element location or upstream of it in the element network. Curve number is selected as a parameter to be adjusted.



5.5. Viewing Results for a Trial

5.5.1. Optimized Parameters

Optimized Parameter Results for Trial "Trial 1"

Project: HEC_HMS3 Optimization Trial: Trial 1

Start of Trial: 01Jan1982, 00:00 Basin Model: GeoHMS
End of Trial: 29Dec1990, 00:00 Meteorologic Model: Met 1
Compute Time: 01Sep2008, 22:03:10 Control Specifications: Control 1

Element	Parameter	Units	Initial Value	Optimized Value	Objective Function Sensitivity
Total Catchm...	Curve Number		73	82.416	-0.88

As we can see from the table above and the hydrograph below, the optimization result shows computed and observed stream flow best fits at CN value equal to 82.416. Even if the two hydrographs have some deviation we cannot conclude they are totally different. Hence, with the absence of gauged data for the study area we can use HEC_HMS for flood estimation for Dire Dawa chatchments by adjusting the CN values with a ratio $= \frac{82.416}{73} = 1.12898 \approx 1.12$

5.5.2. Summary of Result

Summary Results for Junction "Outlet_erer3"

Project : HEC_HMS3 Simulation Run : Run 1 Junction: Outlet_erer3

Start of Run : 01Jan1982, 00:00 Basin Model : GeoHMS
End of Run : 29Dec1990, 00:00 Meteorologic Model : Met 1
Compute Time : 20Sep2008, 15:10:08 Control Specifications : Control 1

Volume Units : MM 1000 M3

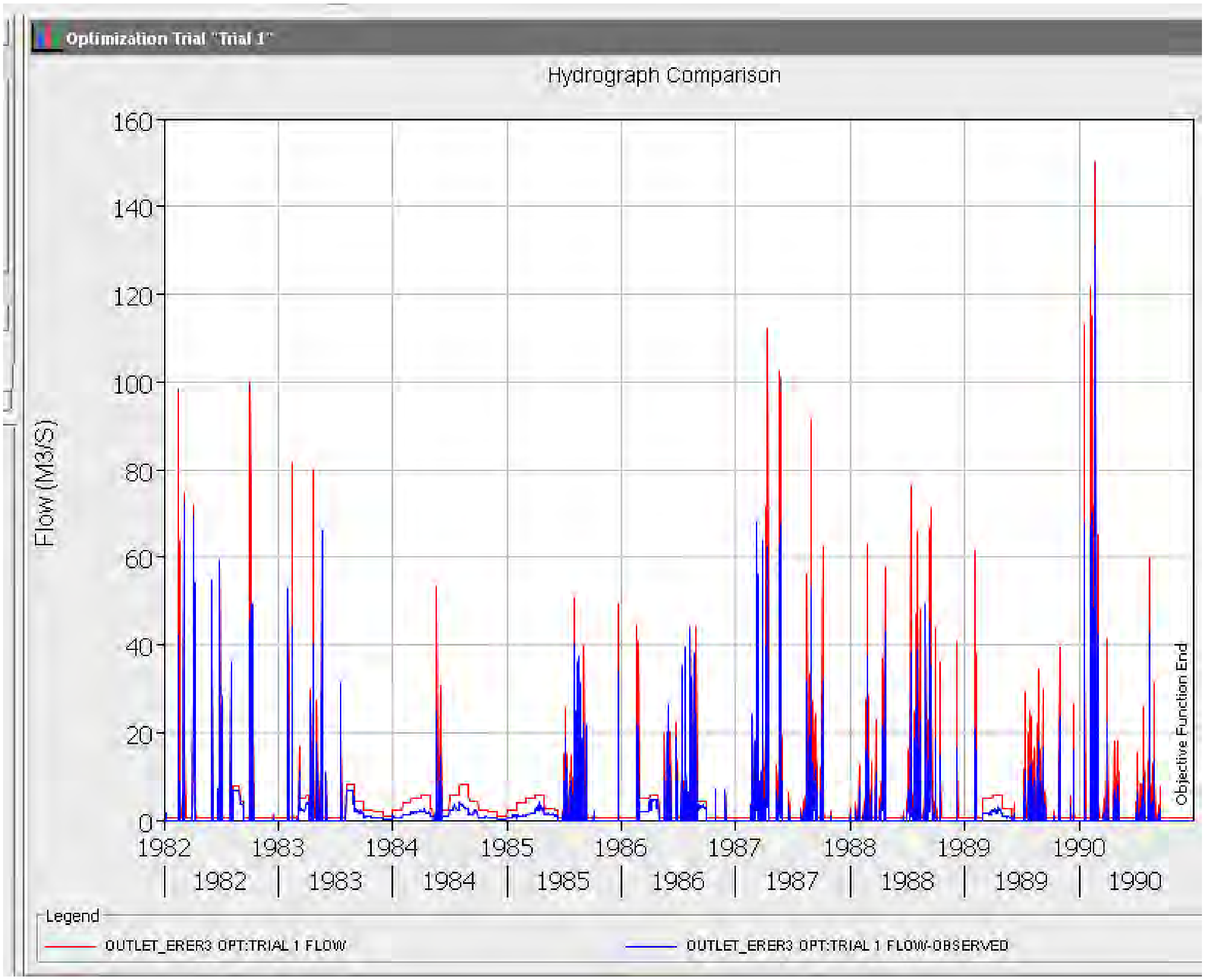
Computed Results

Peak Outflow : 150.21 (M3/S) Date/Time of Peak Outflow : 21Feb1990, 00:00
Total Outflow : 5962.83 (MM)

Observed Hydrograph at Gage Qerer

Peak Discharge : 131.00 (M3/S) Date/Time of Peak Discharge : 22Feb1990, 00:00
Avg Abs Residual : 2.36 (M3/S)
Total Residual : 2822.45 (MM) Total Obs Q : 2950.30 (MM)

Observed and Computed Hydrographs



6. OUT-PUT OF HEC-HMS FOR DECHATU CATCHMENT

As described above, though rainfalls are registered over 24 hours, the actual duration of the storm will normally be much less than 24 hours and will include all or nearly all of the recorded 24 hour rainfall amount. For this reason, three options are considered for meteorlogic modelling. The results of these computations are shown below:

6.1 Run_1:- Using the annual maximum daily rainfall as 24 hours duration storm Gage weight is used for metrological modelling. The results are summarized below. As we can see from the result, the maximum flood discharge at the outlet is 602.6m^3/sec.

i. Hydrologic Elements' Summary

Project: DDhms2 Simulation Run: Run 1

Start of Run: 05Aug2006, 01:00 Basin Model: DDProj23
End of Run: 06Aug2006, 00:00 Meteorologic Model: Met 1
Compute Time: 04Sep2008, 22:03:45 Control Specifications: Control 1

Volume Units: (•) MM ( ) 1000 M3

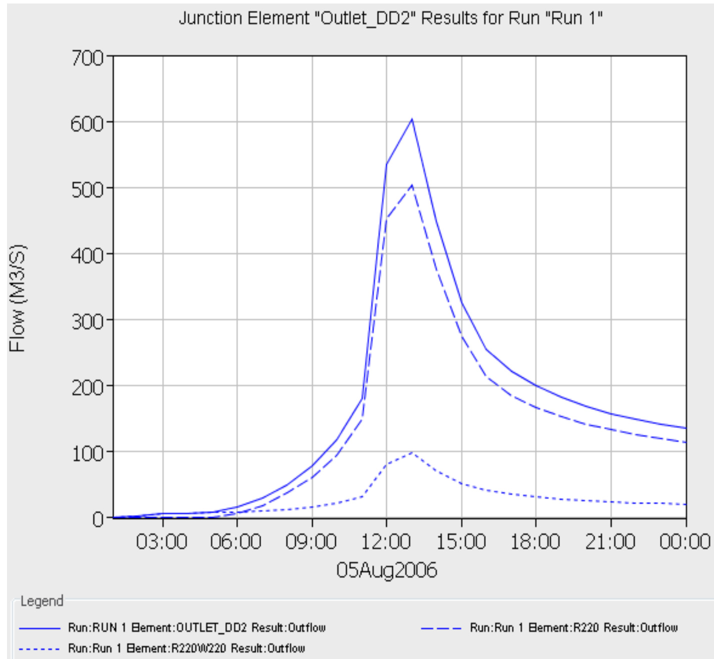
Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (MM)
JR220	138.619000	560.02	05Aug2006, 12:00	86.06
JR290	97.386000	341.60	05Aug2006, 12:00	71.41
JR310	63.866000	233.75	05Aug2006, 12:00	70.89
Outlet_DD2	165.654000	602.60	05Aug2006, 13:00	85.96
R220	138.619000	503.88	05Aug2006, 13:00	85.20
R220W220	27.035000	98.72	05Aug2006, 13:00	89.85
R280W280	34.829000	219.11	05Aug2006, 12:00	126.09
R290	97.386000	307.72	05Aug2006, 12:00	71.09
R290W290	6.404000	33.19	05Aug2006, 12:00	95.91
R300W300	35.762000	122.84	05Aug2006, 12:00	68.88
R310	63.866000	214.65	05Aug2006, 12:00	70.65
R310W310	3.606000	17.54	05Aug2006, 12:00	90.60
R320W320	28.104000	110.91	05Aug2006, 12:00	73.44
R330W330	29.914000	109.41	05Aug2006, 12:00	70.73

ii. Time Series Data at the Outlet

Project : DDhms2 Run : Run 1 Junction: Outlet_DD2

Start of Run : 05Aug2006, 01:00 Basin Model : DDProj23
End of Run : 06Aug2006, 00:00 Meteorologic Model : Met 1
Compute Time : 04Sep2008, 23:51:23 Control Specifications : Control 1

Date	Time	Inflow from... (M3/S)	Inflow from... (M3/S)	Outflow (M3/S)
05Aug2006	01:00	0.00	0.00	0.00
05Aug2006	02:00	0.00	2.71	2.71
05Aug2006	03:00	0.00	5.91	5.91
05Aug2006	04:00	0.02	7.28	7.30
05Aug2006	05:00	1.48	8.07	9.56
05Aug2006	06:00	7.36	8.81	16.17
05Aug2006	07:00	19.47	10.25	29.71
05Aug2006	08:00	37.16	12.72	49.88
05Aug2006	09:00	61.78	16.65	78.43
05Aug2006	10:00	95.45	22.38	117.83
05Aug2006	11:00	148.90	31.67	180.57
05Aug2006	12:00	453.47	80.51	533.97
05Aug2006	13:00	503.88	98.72	602.60
05Aug2006	14:00	376.94	71.94	448.88
05Aug2006	15:00	273.81	52.61	326.42
05Aug2006	16:00	214.72	41.41	256.12
05Aug2006	17:00	185.52	35.43	220.95
05Aug2006	18:00	168.09	31.77	199.87
05Aug2006	19:00	154.13	28.78	182.90
05Aug2006	20:00	142.64	26.52	169.16
05Aug2006	21:00	133.38	24.74	158.12
05Aug2006	22:00	125.79	23.29	149.07
05Aug2006	23:00	119.48	22.08	141.57
06Aug2006	00:00	114.07	21.05	135.12



iii. Outlet Summary Table

Project : DDhms2 Simulation Run : Run 1 Junction: Outlet_DD2

Start of Run : 05Aug2006, 01:00 Basin Model : DDProj23
End of Run : 06Aug2006, 00:00 Meteorologic Model : Met 1
Compute Time : 04Sep2008, 23:51:23 Control Specifications : Control 1

Volume Units : (•) MM ( ) 1000 M3

Computed Results

Peak Outflow : 602.60 (M3/S) Date/Time of Peak Outflow : 05Aug2006, 13:00
Total Outflow : 85.96 (MM)

6.2 Run_2:- using 50 years return period ERA Intensity-frequency-duration curve

This run is executed using 50 years return period ERA Intensity-frequency-duration curve as described in section 3.2.4. The frequency storm method is adopted since it deals with synthetic storm from statistical precipitation data. As we can see from the following summarized data, the maximum flood discharge at the outlet is 1189.10 m^3/sec.

i. Elements' Summary

Project: DDhms2 Simulation Run: Run 2

Start of Run:	05Aug2006, 01:00	Basin Model:	DDProj23
End of Run:	05Aug2006, 03:00	Meteorologic Model:	Met 2
Compute Time:	05Sep2008, 00:34:28	Control Specifications:	Control 2

Volume Units: (•) MM ( ) 1000 M3

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (MM)
JR220	138.619000	1023.05	05Aug2006, 02:50	14.73
JR290	97.386000	738.14	05Aug2006, 02:45	16.83
JR310	63.866000	492.88	05Aug2006, 02:40	18.27
Outlet_DD2	165.654000	1189.10	05Aug2006, 03:00	8.69
R220	138.619000	1011.38	05Aug2006, 03:00	7.91
R220W220	27.035000	177.72	05Aug2006, 03:00	12.69
R280W280	34.829000	254.58	05Aug2006, 02:50	16.27
R290	97.386000	733.94	05Aug2006, 02:50	13.54
R290W290	6.404000	69.26	05Aug2006, 02:25	24.52
R300W300	35.762000	254.24	05Aug2006, 02:45	16.07
R310	63.866000	492.55	05Aug2006, 02:45	15.79
R310W310	3.606000	50.00	05Aug2006, 02:10	24.42
R320W320	28.104000	245.72	05Aug2006, 02:40	21.06
R330W330	29.914000	231.50	05Aug2006, 02:40	18.13

ii. Time Series Data at the Outlet

Project : DDhms2 Run : Run 2 Junction: Outlet_DD2

Start of Run : 05Aug2006, 01:00 Basin Model : DDProj23
End of Run : 05Aug2006, 03:00 Meteorologic Model : Met 2
Compute Time : 05Sep2008, 00:34:28 Control Specifications : Control 2

Date	Time	Inflow from... (M3/S)	Inflow from... (M3/S)	Outflow (M3/S)
05Aug2006	01:00	0.00	0.00	0.00
05Aug2006	01:05	0.00	0.04	0.04
05Aug2006	01:10	0.00	0.18	0.18
05Aug2006	01:15	0.00	0.44	0.44
05Aug2006	01:20	0.00	0.86	0.86
05Aug2006	01:25	0.00	1.49	1.49
05Aug2006	01:30	0.00	2.40	2.40
05Aug2006	01:35	0.00	3.65	3.65
05Aug2006	01:40	0.00	5.30	5.30
05Aug2006	01:45	0.00	7.36	7.36
05Aug2006	01:50	0.00	9.79	9.80
05Aug2006	01:55	0.01	12.64	12.64
05Aug2006	02:00	0.01	16.08	16.10
05Aug2006	02:05	0.05	21.94	21.98
05Aug2006	02:10	0.26	30.29	30.55
05Aug2006	02:15	1.32	40.61	41.92
05Aug2006	02:20	7.66	52.75	60.41
05Aug2006	02:25	38.62	67.27	105.90
05Aug2006	02:30	118.75	84.74	203.49
05Aug2006	02:35	244.34	104.18	348.53
05Aug2006	02:40	389.78	123.94	513.72
05Aug2006	02:45	584.95	142.17	727.12
05Aug2006	02:50	809.33	157.20	966.54
05Aug2006	02:55	953.96	169.05	1123.01
05Aug2006	03:00	1011.38	177.72	1189.10

iii. Outlet Summary Table

Project : DDhms2 Simulation Run : Run 2 Junction: Outlet_DD2

Start of Run : 05Aug2006, 01:00 Basin Model : DDProj23
End of Run : 05Aug2006, 03:00 Meteorologic Model : Met 2
Compute Time : 05Sep2008, 00:34:28 Control Specifications : Control 2

Volume Units : (•) MM ( ) 1000 M3

Computed Results

Peak Outflow : 1189.10 (M3/S) Date/Time of Peak Outflow : 05Aug2006, 03:00
Total Outflow : 8.69 (MM)

6.3 Run_3:-Using the annual maximum daily rainfall as 6 hours duration storm

Gage weight is used for metrological modelling. The results are summarized below. As we can see from the result, the maximum flood discharge at the outlet is 1137.44m3/sec.

i. Elements' Summary

Project: DDhms2 Simulation Run: Run 3

Start of Run: 05Aug2006, 00:00 Basin Model: DDProj23
End of Run: 05Aug2006, 06:00 Meteorologic Model: Met 3
Compute Time: 05Sep2008, 00:49:10 Control Specifications: Control 3

Volume Units: (•) MM ( ) 1000 M3

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (MM)
JR220	138.619000	998.96	05Aug2006, 03:00	77.50
JR290	97.386000	611.47	05Aug2006, 03:00	64.97
JR310	63.866000	415.22	05Aug2006, 03:00	64.87
Outlet_DD2	165.654000	1137.44	05Aug2006, 04:00	74.07
R220	138.619000	950.21	05Aug2006, 04:00	73.78
R220W220	27.035000	187.23	05Aug2006, 04:00	75.56
R280W280	34.829000	393.09	05Aug2006, 03:00	114.72
R290	97.386000	553.77	05Aug2006, 04:00	63.45
R290W290	6.404000	60.03	05Aug2006, 03:00	88.80
R300W300	35.762000	215.76	05Aug2006, 03:00	62.50
R310	63.866000	385.62	05Aug2006, 03:00	64.01
R310W310	3.606000	31.52	05Aug2006, 03:00	83.73
R320W320	28.104000	199.45	05Aug2006, 03:00	67.89
R330W330	29.914000	194.33	05Aug2006, 03:00	64.76

ii. Outlet Summary Table

Project : DDhms2 Simulation Run : Run 3 Junction: Outlet_DD2

Start of Run : 05Aug2006, 00:00 Basin Model : DDProj23
End of Run : 05Aug2006, 06:00 Meteorologic Model : Met 3
Compute Time : 05Sep2008, 00:49:10 Control Specifications : Control 3

Volume Units : (•) MM ( ) 1000 M3

Computed Results

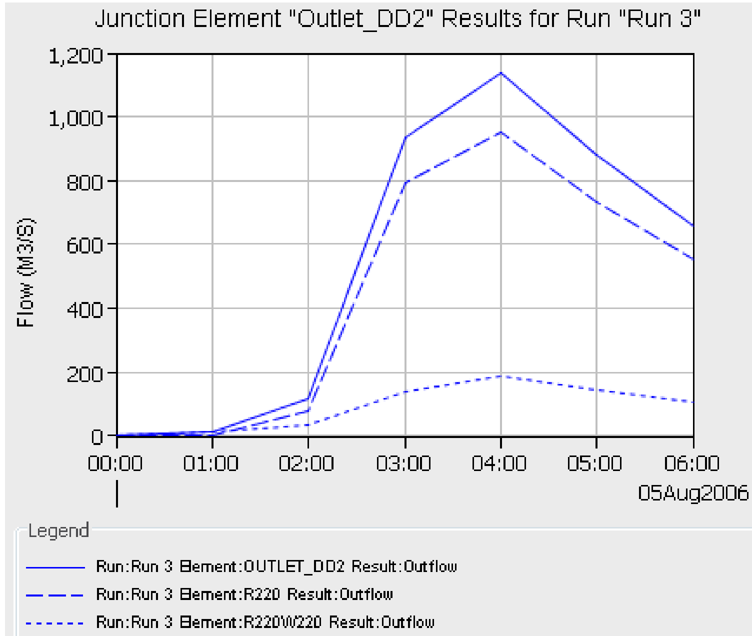
Peak Outflow : 1137.44 (M3/S) Date/Time of Peak Outflow : 05Aug2006, 04:00
Total Outflow : 74.07 (MM)

iii. Time Series Data at the Outlet

Project : DDhms2 Run : Run 3 Junction: Outlet_DD2

Start of Run : 05Aug2006, 00:00 Basin Model : DDProj23
End of Run : 05Aug2006, 06:00 Meteorologic Model : Met 3
Compute Time : 05Sep2008, 00:49:10 Control Specifications : Control 3

Date	Time	Inflow from... (M3/S)	Inflow from... (M3/S)	Outflow (M3/S)
05Aug2006	00:00	0.00	0.00	0.00
05Aug2006	01:00	1.75	11.67	13.43
05Aug2006	02:00	80.87	34.05	114.92
05Aug2006	03:00	796.30	137.69	933.99
05Aug2006	04:00	950.21	187.23	1137.44
05Aug2006	05:00	736.14	143.13	879.27
05Aug2006	06:00	551.66	107.26	658.92



6.4 Summary of the Three Run/Result

From the above results, Run_2 which is using 50 years return period ERA Intensity- and Run_3 which is using the annual maximum daily rainfall as 6 hours duration storm peak discharges for the 50 years return period result values are close to each other. Ignoring the two extreme values, Run_3 that is using the annual maximum daily rainfall as 6 hours duration storm peak discharges for the 50 years return period result value is chosen for the next step analysis which is flood delineation.

50yrs return period peak flood discharge estimated using Computer Programs HEC-GeoHMS and HEC-HMS found to be 1187 m^3/sec and 1137m^3/sec for ERA Intensity and annual maximum daily rainfall as 6 hours duration storm respectively.

The annual maximum daily rainfall as 6 hours duration storm peak discharges for the 50 years return period value 1137m^3/sec is chosen for the flood mapping.

6.5 Consideration of Other Return Periods

As explained above, the annual maximum daily rainfall as 6 hours duration storm peak discharges for the 50 years return period value is chosen for the flood analysis.

Here different return periods of the same mode of rainfall are considered for analysis.

6.5.1. 100 Years Return Period

i. Dire Dawa RF

The 100 years return period daily maximum rainfall, 110.69mm, is changed to 6hrs incremental RF.

Using, p = M * square root (T), ► M = p /square root (T), ► M = 45.19mm

Table 6-1: 6hrs incremental RF for Dire Dawa station 100 yrs return period daily maximum RF

Time	Hourly Distributed Cumulative P $P = M * sqrt(T_i)$	Incremental Depth	Time Interval	Precipitation
hr	mm	mm	hr	mm
1	45.2	45.2	0--1	10.67
2	63.9	18.72	1--2	14.36
3	78.3	14.36	2--3	45.19
4	90.4	12.11	3--4	18.72
5	101.0	10.67	4--5	12.11
6	110.7	9.64	5--6	9.64

ii. Dengego RF Analysis

The 100 years return period daily maximum RF, 125.40mm, is changed to 6hrs incremental RF. Using, p = M * $\sqrt{T}$, ► M = p / square root (T), ► M = 51.19mm

Table 6-2: 6hrs incremental RF for Dengego station daily maximum RF 100yrs RP

Time	Hourly Distributed Cumulative P $P = M * sqrt(T_i)$	Incremental Depth	Time Interval	Precipitation
hr	mm	mm	hr	mm
1	51.2	51.2	0--1	12.08
2	72.4	21.20	1--2	16.27
3	88.7	16.27	2--3	51.19
4	102.4	13.72	3--4	21.20
5	114.5	12.08	4--5	13.72
6	125.4	10.93	5--6	10.93

iii. Kersa RF Analysis

The 100 years return period daily maximum RF, 194.88mm, is changed to 6hrs incremental RF. Using, , p = M * $\sqrt{T}$, ► M = p / square root (T), ► M =79.56mm

Table 6-3: 6hrs incremental RF for Kersa station daily maximum RF 100yrs RP

Time	Hourly Distributed Cumulative P $P = M * sqrt(T_i)$	Incremental Depth	Time Interval	Precipitation
hr	mm	mm	hr	mm
1	79.6	79.6	0--1	18.78
2	112.5	32.95	1--2	25.29
3	137.8	25.29	2--3	79.56
4	159.1	21.32	3--4	32.95
5	177.9	18.78	4--5	21.32
6	194.9	16.98	5--6	16.98

6.5.2. 500 Years Return Period

Since the Kersa station data is not enough to do analysis for 500 years, only Dire Dawa and Dengego stations are used.

i. Dire Dawa RF

The 500 years return period daily maximum rainfall, 132.6 mm is changed to 6hrs incremental rainfall.

Using, p = M * $\sqrt{T}$, ► M = p / square root (T), ► M = 54.12mm.

Table 6-4: 6hrs incremental RF for Dire Dawa station 500 yrs return period daily maximum RF

Time	Hourly Distributed Cumulative P P = M * sqrt (T_i)	Incremental Depth	Time Interval	Precipitation
hr	mm	mm	hr	mm
1	54.1	54.1	0--1	12.78
2	76.5	22.42	1--2	17.20
3	93.7	17.20	2--3	54.12
4	108.2	14.50	3--4	22.42
5	121.0	12.78	4--5	14.50
6	132.6	11.55	5--6	11.55

ii. Dengego RF Analysis

The 100 years return period daily maximum Rf, 150.26 mm, is changed to 6hrs incremental RF. Using p = M * $\sqrt{T}$, ► M = p / square root (T), ► M = 61.34mm

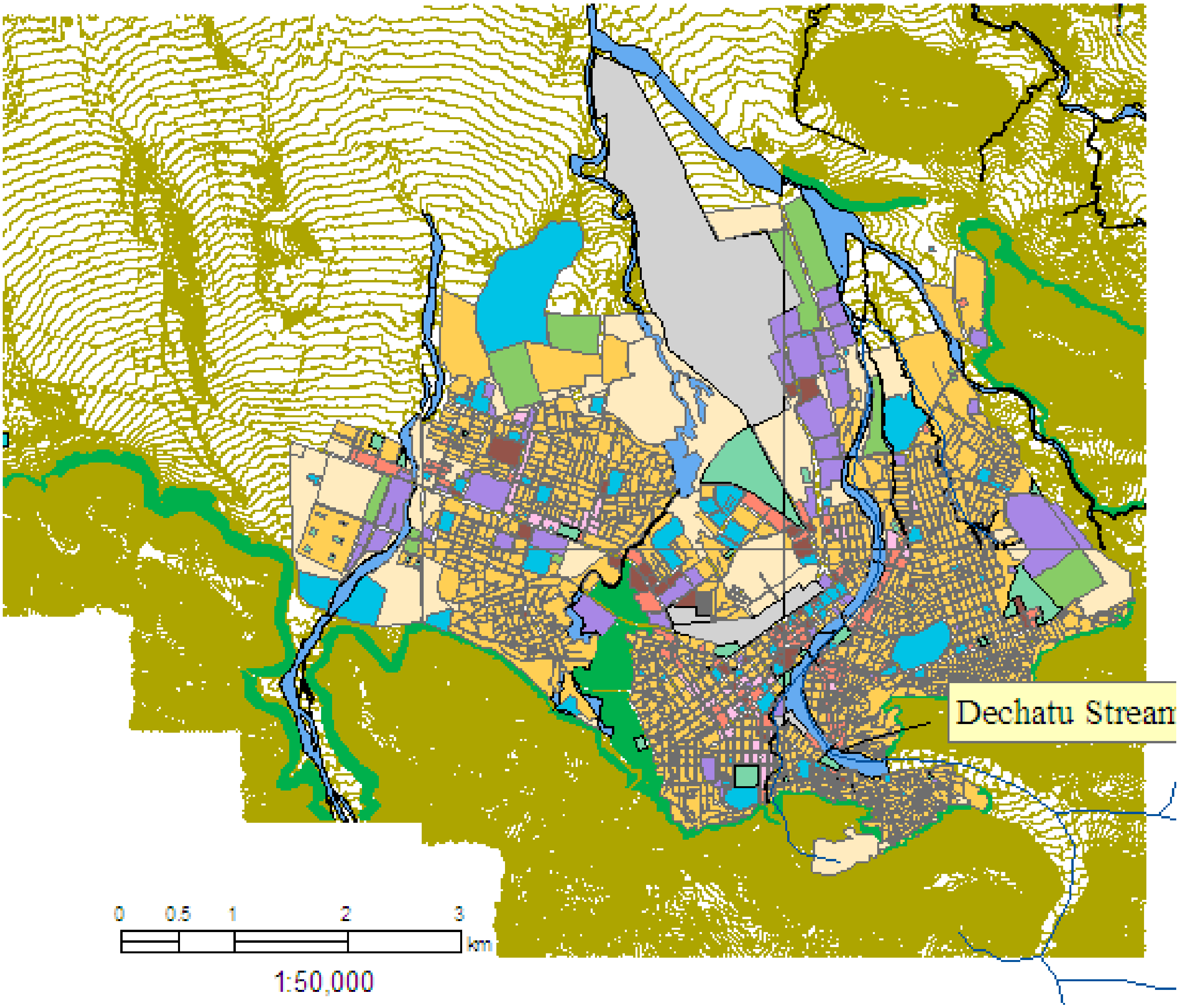
Table 6-5: 6hrs incremental RF for Dengego station daily maximum RF 500yrs RP

Time	Hourly Distributed Cumulative P P = M * sqrt (T_i)	Incremental Depth	Time Interval	Precipitation
hr	mm	mm	hr	mm
1	61.34	61.3	0--1	14.48
2	86.8	25.41	1--2	19.50
3	106.3	19.50	2--3	61.34
4	122.7	16.44	3--4	25.41
5	137.2	14.48	4--5	16.44
6	150.3	13.09	5--6	13.09

7 FLOOD INUNDATION MAP

The flood Inundation map shows the area extent that can be delineated as buffer zone. Two models HEC_GeoRAS and HEC_RAS are used one after another (i.e first HEC_GeoRAS then HEC_RAS then back to HEC_GeoRAS) to accomplish the task. Dire Dawa town 1:5000 scale map having 4 meters contour interval for the town area and 0.5m for Dechatu stream cross section is converted to TIN data format used to proceed with the HEC-GeoRAS.

Figure 7-1: Dire Dawa Town 4 meters contour interval 1:5000 scale map



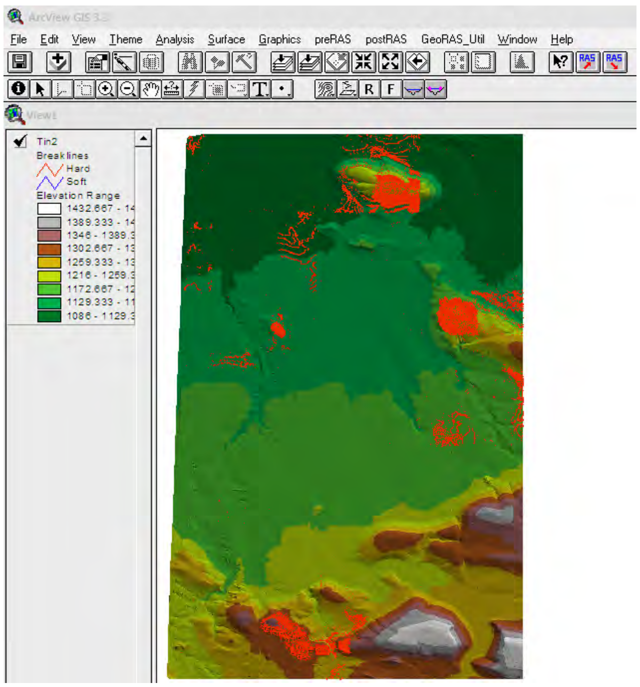
(Source FUPI)

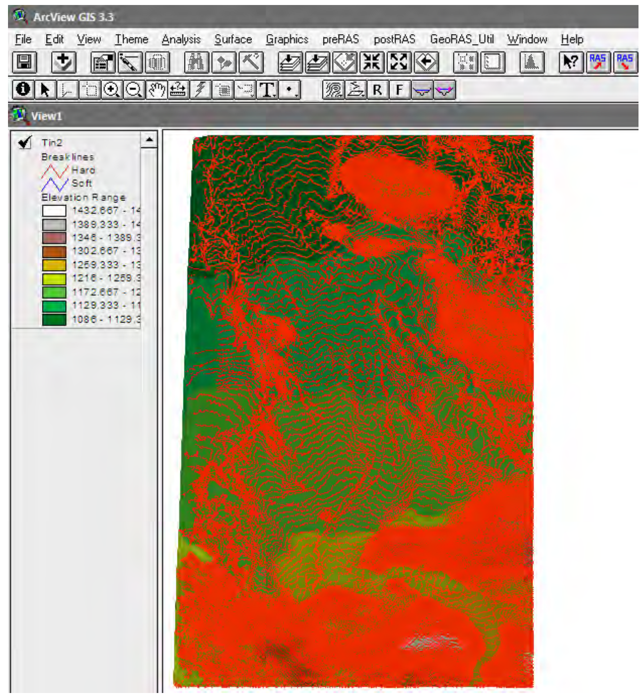
7.1 PreRAS (HEC-GeoRAS)

7.1.1 Procedures followed:-

The 4m contour interval map is changed to Terrain TIN (a triangulated irregular network) using ArcView GIS extension HEC-GeoRAS.

Figure 7-2: Dire Dawa Town catchment TIN format





Then the following main procedures are applied.

7.1.1.1 Creating RAS Themes

The RAS Themes are the basis for the geometric data extracted in the GIS for hydraulic analysis in HEC-RAS. These Themes include: Stream Centreline, Banks, Flow Paths Centrelines and Cross-Sectional Cut Lines.

7.1.1.2 Attributing RAS Theme

Once the RAS Themes have been created, the geometric data extraction process began. The Stream Centreline Theme completed and the cross-section attributes (geometric data for each cross section) calculated. Stream Centreline Theme created before completing the Cross-Section Cut Line Theme and the Cross-Sectional Cut Lines Theme completed.

7.1.1.3 Generating the RAS GIS Import File

To generate the RAS GIS Import File, the 3D stream Centreline and Cross Section Surface Line (3D) shape file created from the RAS Theme. Geometric data from the two 3D (stream Centreline and Cross Section Surface Line) shape files is written to the RAS GIS Export File. The geometric data includes: river, reach, and station identifiers; cross-section cut lines; cross-section surface lines; main channel bank stations; downstream reach lengths for the left overbank, main channel; and overbank.

Table 7-1: Reach stations and downstream length

Edit Downstream Reach Lengths

River: Dechatu

Reach: within town

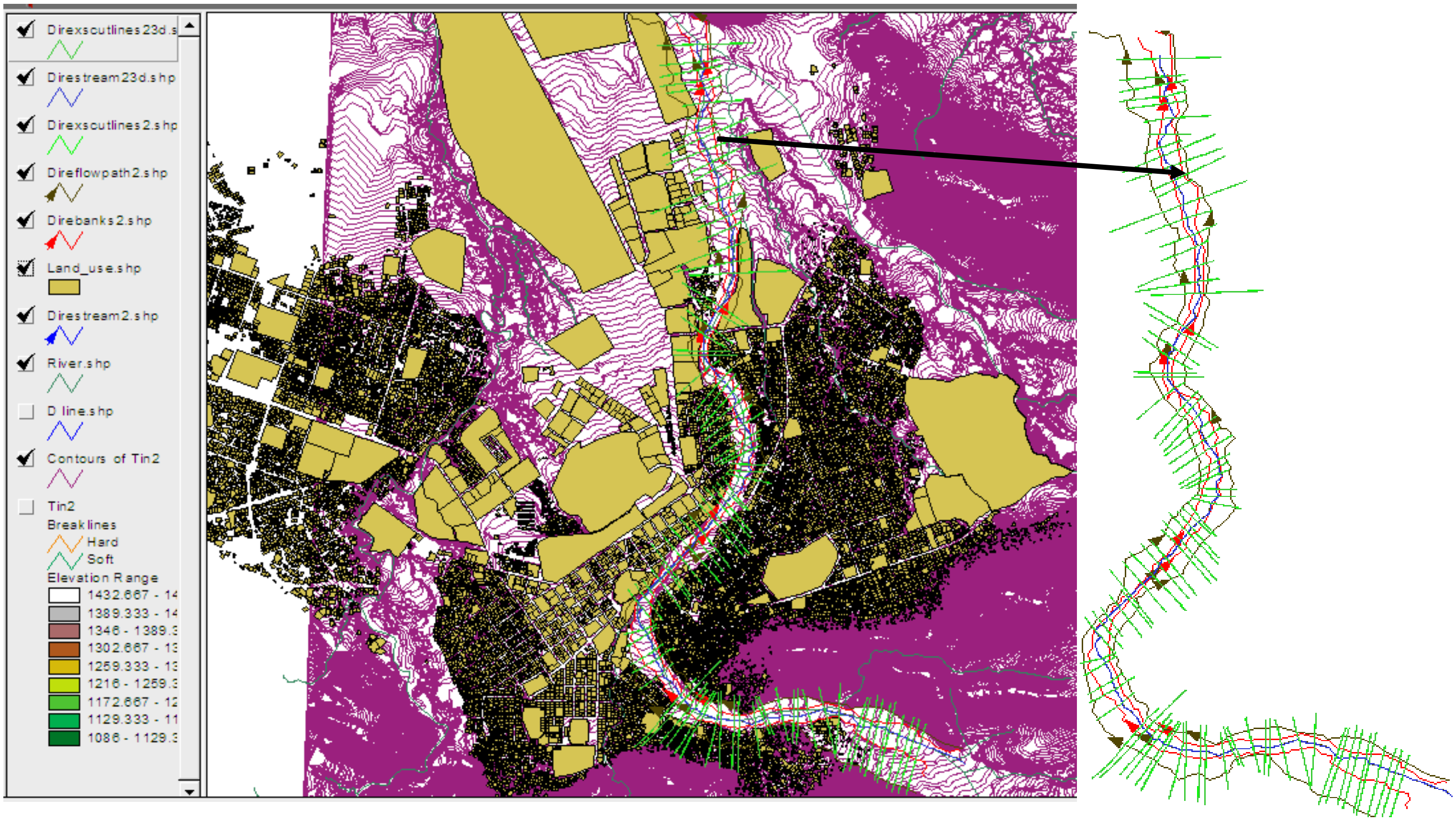
Selected Area Edit Options: Add Constant ... | Multiply Factor ... | Set Values

	River Station	LOB	Channel	ROB
1	91.000	57.38	54.16	52.63
2	90.000	41.06	41.78	44.4
3	89.000	46.45	64.8	71.28
4	88.000	76.99	78.39	83.46
5	87.000	23.59	33.73	31.74
6	86.000	75.63	61.6	57.33
7	85.000	41.5	42.91	43.01
8	84.000	70.36	65.24	52.77
9	83.000	40.67	31.28	25.28
10	82.000	100.92	172.25	186.81
11	81.000	23.52	109.52	125.5
12	80.000	140.49	119.82	123.47
13	79.000	188.12	78.91	51.23
14	78.000	31.34	27.58	22.9
15	77.000	64.15	81.5	111.17
16	76.000	98.88	110.72	123.37
17	75.000	24.03	26.76	31.16
18	74.000	131.14	67.08	42.76
19	73.000	155.91	120.66	118.82
20	72.000	45.87	33.63	33.89
21	71.000	102.82	94.07	102.6
22	70.000	75.81	26.35	14.34
23	69.000	159.21	96.01	48.68
24	68.000	27.6	26.09	25.62
25	67.000	61.61	55.28	53.06
26	66.000	42.99	38.24	32.72
27	65.000	65.04	64.46	55.27
28	64.000	40.13	39.35	45.21
29	63.000	254.45	208.83	205.5
30	61.000	149.82	104.1	98.63
31	60.000	60.35	64.05	66.75
32	58.000	128.46	126.77	133.56
33	57.000	84.3	36.94	21.73
34	56.000	272.07	241.05	16.32
35	55.000	66.15	63.28	84.28
36	54.000	59.5	64.89	62.21
37	53.000	107.9	98.03	103.82
38	52.000	98.45	85.09	90.44
39	51.000	27.04	23.14	31.37
40	50.000	157.72	110.73	144.32
41	48.000	88.55	89.49	95.14
42	47.000	92.81	83.38	76.85
43	46.000	62.4	68.17	60.29
44	45.000	109.32	113.6	129.06
45	44.000	49.06	38.58	41.22
46	43.000	67.96	71.27	77.45
47	42.000	49.86	40.31	35.94
48	41.000	39.54	59.29	67.96
49	40.000	56.66	23.36	18.26
50	39.000	40.95	143.7	228.2
51	38.000	30.07	59.82	96.71
52	37.000	51.37	57.68	47.69
53	36.000	41.37	124.12	146.83
54	35.000	37.63	48.54	57.11
55	34.000	41.64	45.32	43.79
56	33.000	109.27	98.11	176.55
57	32.000	70.01001	70.8501	106.28
58	31.000	74.19	84.32	77.88
59	29.000	74.41	95.24	100.76
60	28.000	76.83	66.69	61.72
61	27.000	53.85	51.13	45.3
62	26.000	141.31	127.02	106
63	25.000	146.58	84.97	34.33
64	24.000	36.92	31.53	32.84
65	23.000	194.86	115.91	53.22
66	22.000	80.64	89.09	67.69
67	21.000	109.99	110.61	119.51
68	20.000	55.48	50.25	44.46
69	19.000	210.26	275.18	313.27
70	18.000	64.59	76.31	86.21
71	17.000	95.09	124.87	129.34
72	16.000	121.18	143.1	152.11
73	15.000	117.9	85.05	94.21
74	14.000	151.33	128.28	116.35
75	13.000	94.8	95	109.57
76	12.000	108.5	131.31	151.21
77	11.000	59.56	52.46	58.06
78	10.000	112.99	97.49	91.6
79	9.000	79.52	88.97	97.03
80	8.000	72.67	57.99	51.98
81	7.000	125.74	110.89	50.58
82	6.000	90.93	121.85	120.38
83	5.000	161.19	115.28	134.67
84	4.000	63.73	53.9	62.49
85	3.000	96.25	90.31	97.88
86	2.000	78	49.44	42.01
87	1.000	984.44	162.21	518.3

OK | Cancel

Figure 7-3: Schematic Output of HEC-GeoRAS, pre RAS process

- Cross-Sections, Bank lines and stream lines data are included



7.2 HEC-RAS

7.2.1 Geometric Data:-

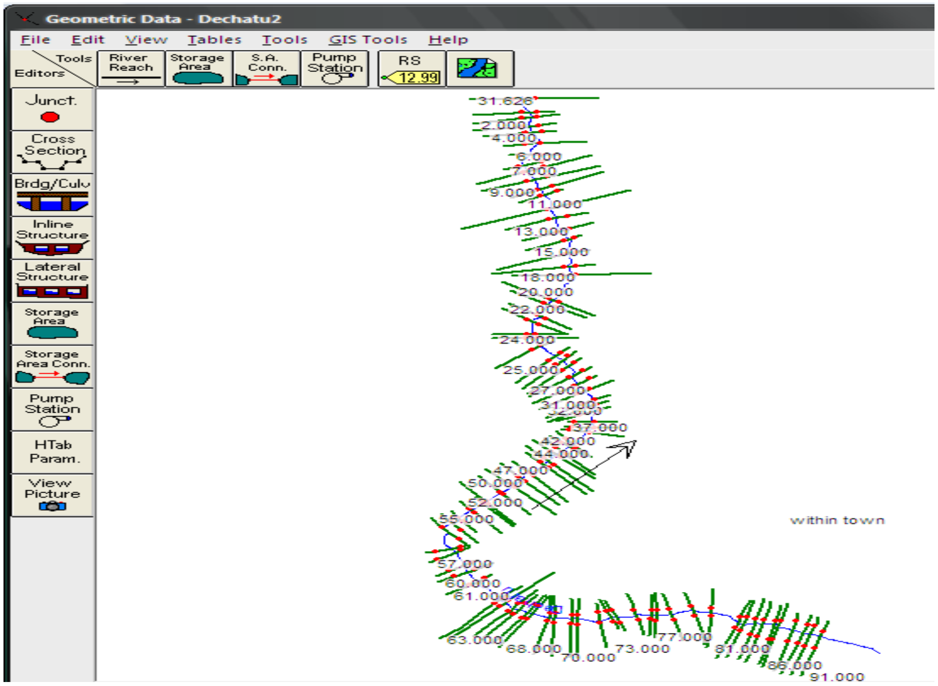
HEC-RAS has the ability to import three-dimensional (3D) river schematic cross section data created in a GIS /ArcView HEC-GeoRAS/ While the HEC-RAS utilizes two dimensional data

During the computations, the three-dimensional information is used in the program for display purpose. Surface profiles exported back to the GIS /ArcView HEC-GeoRAS/ system for development and display of a flood inundation map.

The following geometric data imported using HEC-RAS:-

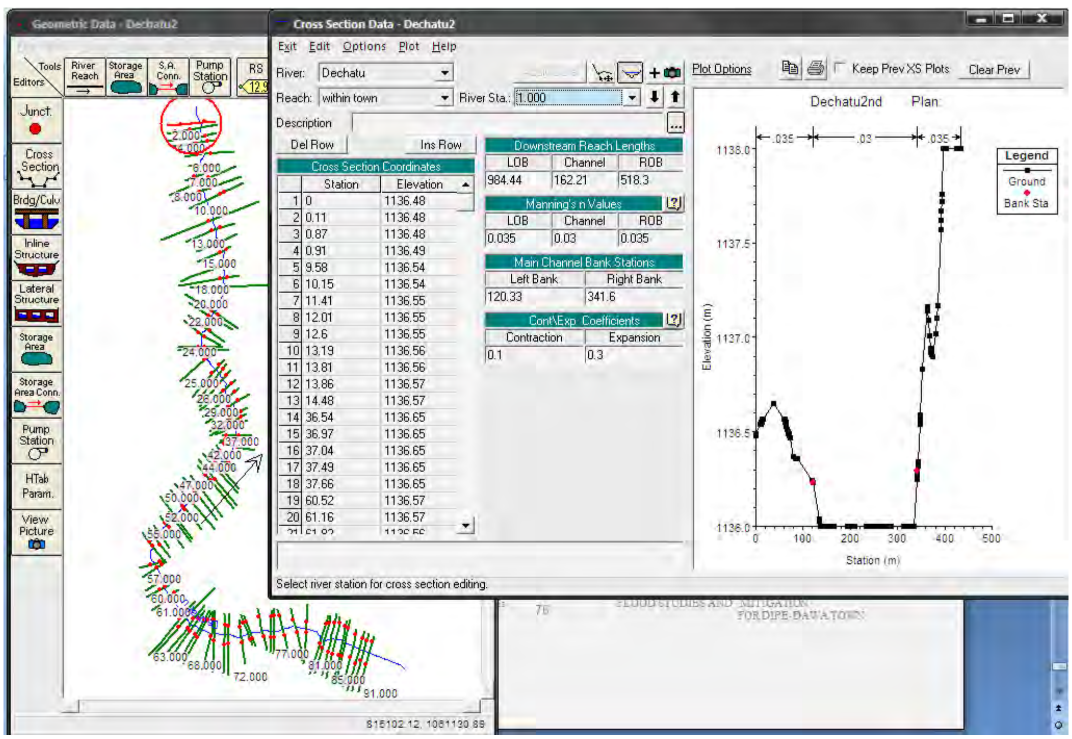
- River System Schematic
- Cross section Data
  - o Cross section surface line
  - o Cross section main channel bank stations
  - o River, Reach, and River Station identifiers
  - o Downstream reach lengths for the left overbank, main channel, and right over bank.
  - o Cross Section Cut Lines (X and Y coordinates of the plan-view line that represents the cross section).

Figure 7-4: Geometric Schematic data imported to HEC-RAS

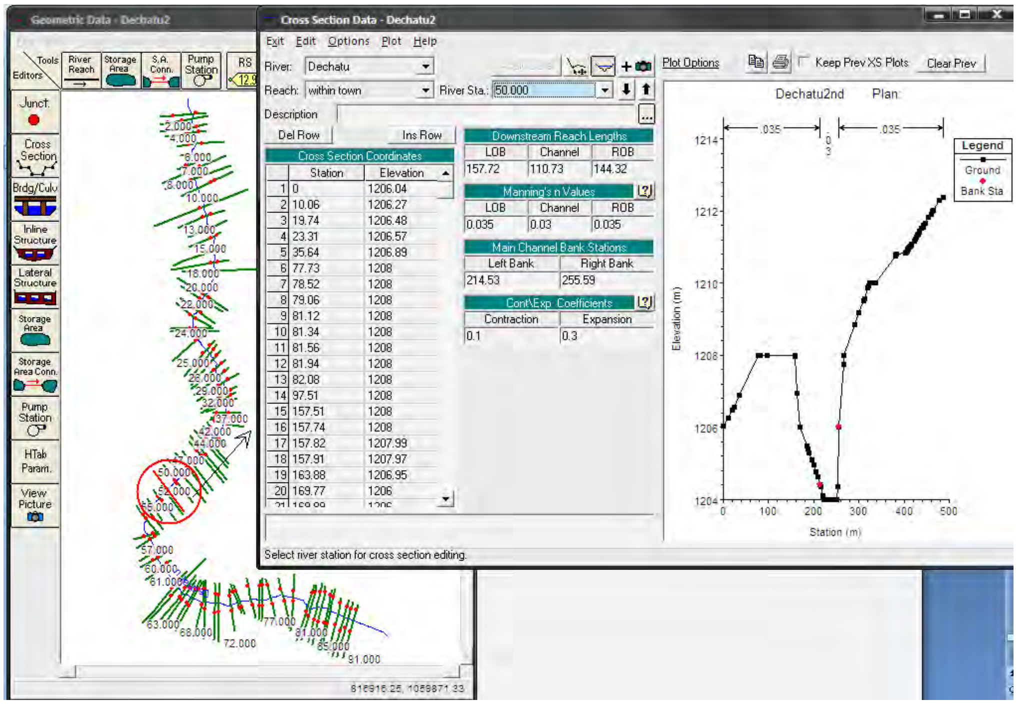


Cross-Section:- Cross sections are taken perpendicular to the stream and extended to achieve the maximum watershed elevation so as to identify areas where flood overtops.

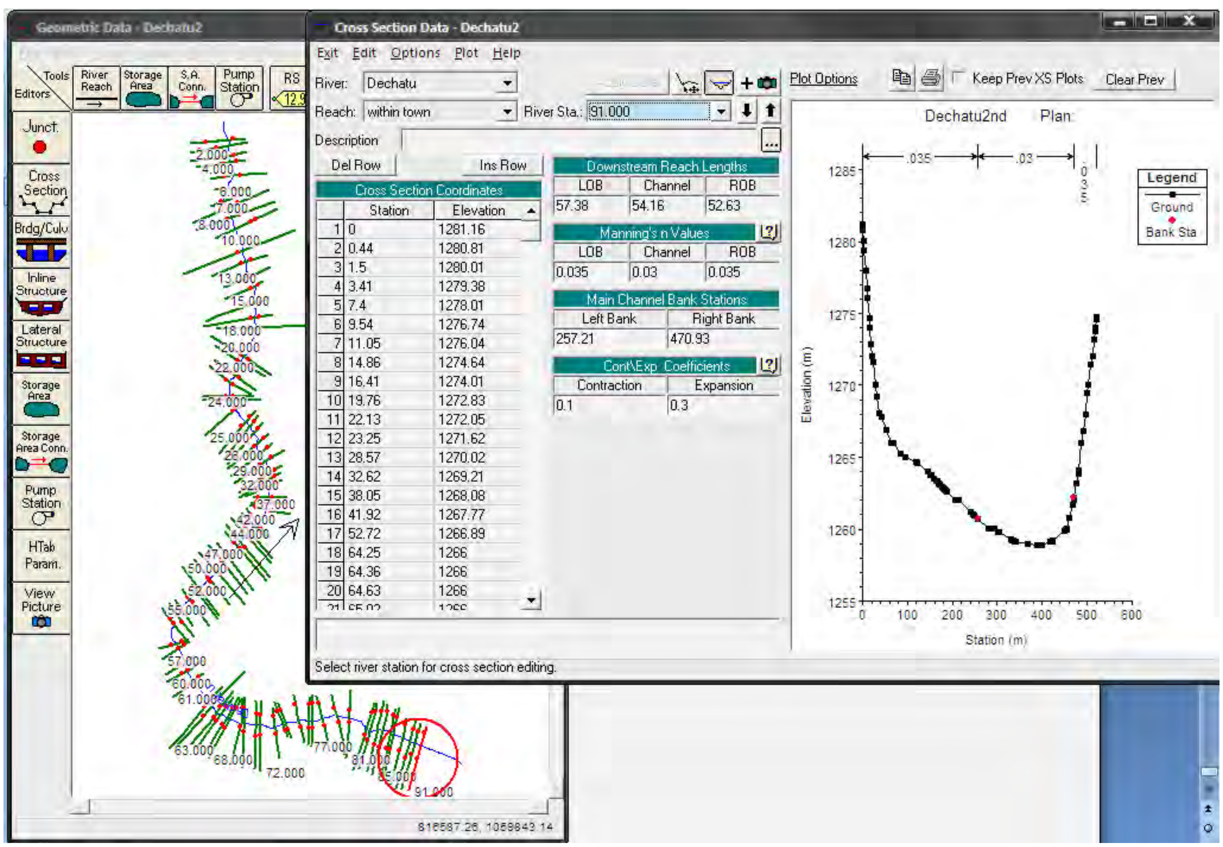
Figure 7-5: Sample Cross-Sections data imported to HEC-RAS
Station-1



Station 50



Station 91

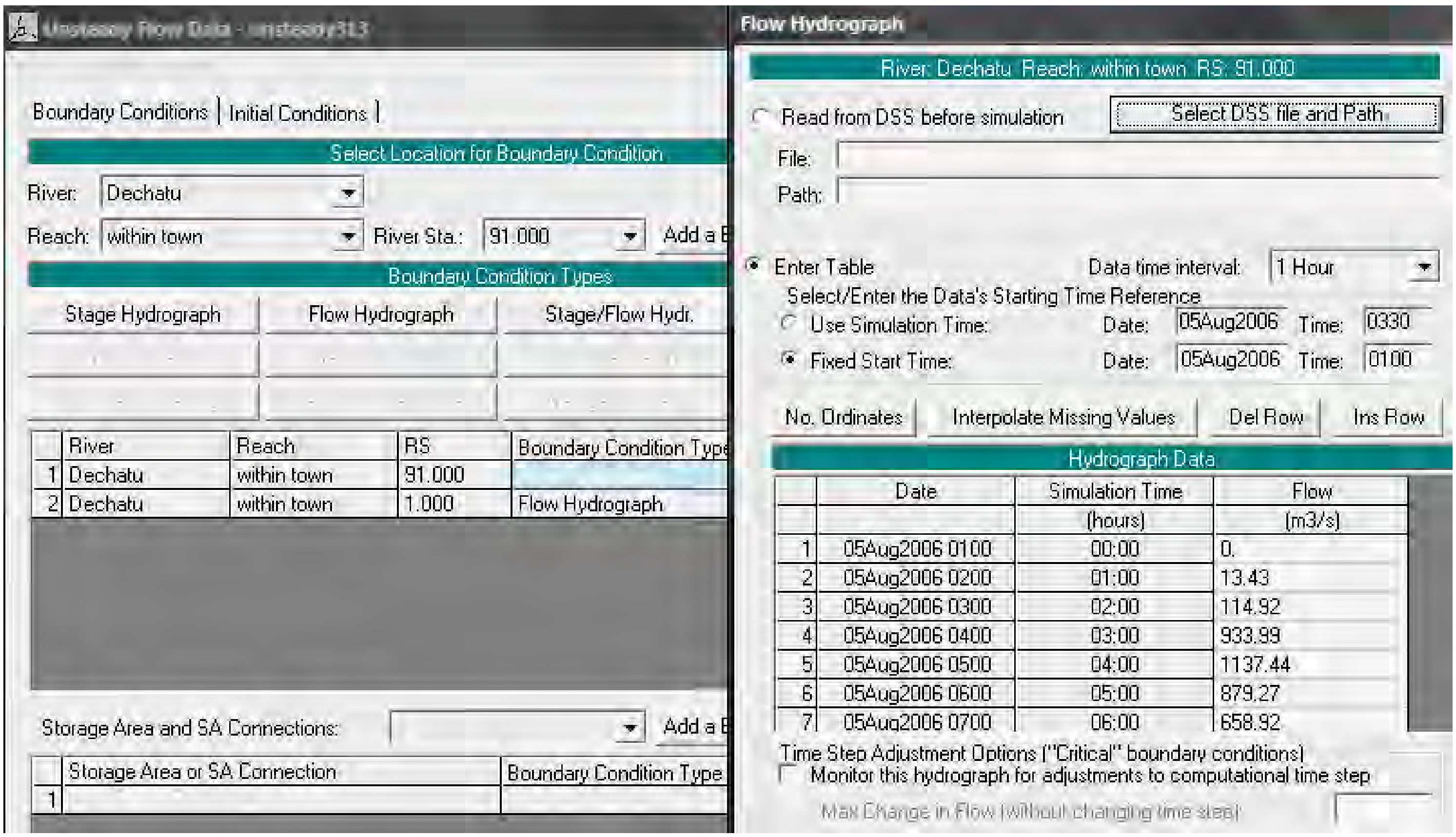


7.2.2 Unsteady Flow Analysis:-

Unsteady flow condition is adopted since it is rare to find the steady flow in the natural channel flow condition. This component of the HEC-RAS modelling system is capable of simulating one dimensional unsteady flow through a full network of open channels.

Flow Hydrograph for Dechatu catchment computed using HEC-GeoHMS and HEC-HMS for 6hrs storm duration which is used as boundary condition as shown on the following Figure.

Figure 7-6: Dechatu catchment flow hydrograph for 6hrs storm duration

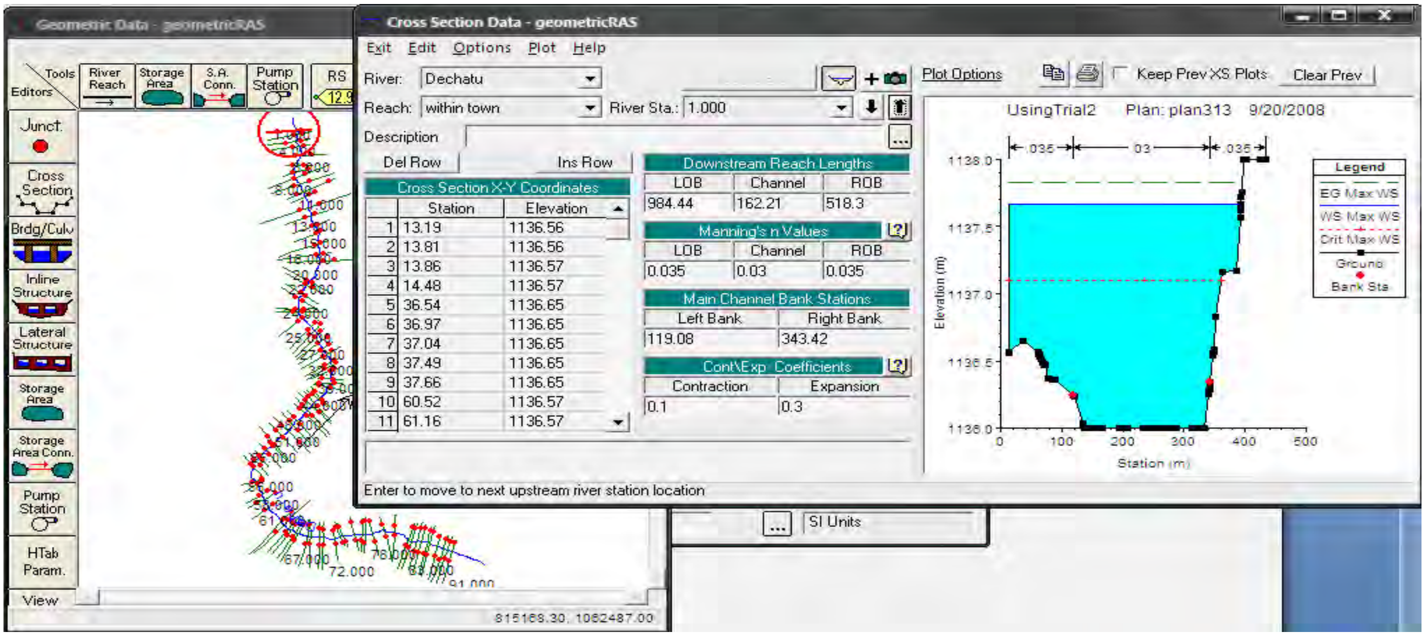


7.2.3 Results of HEC-RAS computation:-

7.2.3.1 Cross-Sections

Figure 7-7: Sample Cross-Sections

Station-1



Station-6

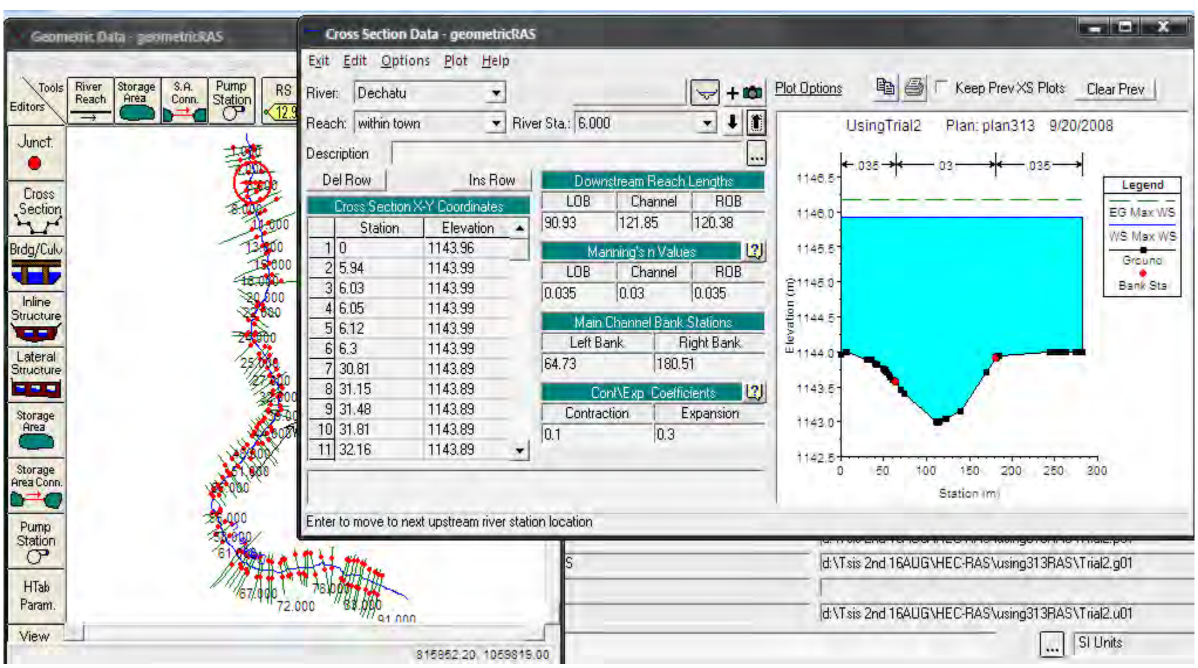


Table 7-2: Profile Summary HEC-RAS output

Profile Output Table - Standard Table 1

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HEC-RAS Plan: 2ndtrial River: Dechatu Reach: within town Profile: Max WS

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)	
within town	91.000	Max WS	629.35	1258.86	1259.71	1260.43	1263.29	0.140895	8.38	75.08	136.92	3.61
within town	90.000	Max WS	515.79	1257.97	1259.66		1259.81	0.001474	1.72	300.45	192.73	0.44
within town	89.000	Max WS	554.96	1257.08	1258.06	1258.52	1260.08	0.115561	6.30	88.04	212.19	3.12
within town	88.000	Max WS	608.43	1256.00	1257.54		1257.75	0.002297	2.00	304.23	216.71	0.54
within town	87.000	Max WS	528.10	1254.37	1255.21	1255.60	1256.82	0.096773	5.63	93.83	234.61	2.84
within town	86.000	Max WS	1252.16	1254.00	1257.80		1257.88	0.000260	1.25	1019.49	305.85	0.21
within town	85.000	Max WS	545.15	1252.80	1253.78	1254.14	1255.04	0.053123	4.96	109.90	211.79	2.20
within town	84.000	Max WS	671.96	1252.00	1253.32		1253.59	0.003780	2.29	294.01	249.41	0.67
within town	83.000	Max WS	523.78	1250.82	1251.93	1252.22	1252.93	0.037499	4.43	118.14	207.48	1.88
within town	82.000	Max WS	698.92	1250.00	1251.53		1251.93	0.005129	2.81	249.12	195.19	0.79
within town	81.000	Max WS	505.72	1247.20	1248.86	1249.29	1250.30	0.023501	5.31	95.20	89.61	1.64
within town	80.000	Max WS	1253.02	1244.79	1250.16		1250.31	0.000466	1.81	856.82	367.59	0.29
within town	79.000	Max WS	576.35	1243.16	1244.56	1244.75	1245.31	0.020510	3.83	150.55	209.49	1.44
within town	78.000	Max WS	675.30	1241.81	1243.18		1243.61	0.006073	2.92	231.56	194.14	0.85
within town	77.000	Max WS	603.77	1240.98	1242.50	1242.66	1243.23	0.015632	3.80	158.79	181.84	1.30
within town	76.000	Max WS	648.58	1239.78	1241.16		1241.63	0.006983	3.03	214.05	188.19	0.91
within town	75.000	Max WS	621.08	1237.45	1238.81	1238.98	1239.60	0.014793	3.94	157.77	164.56	1.28
within town	74.000	Max WS	732.95	1236.01	1238.22		1238.65	0.003944	2.91	252.12	153.61	0.72
within town	73.000	Max WS	540.48	1236.01	1237.45		1237.95	0.006464	3.12	173.50	138.13	0.89
within town	72.000	Max WS	691.53	1233.89	1235.13	1235.29	1235.97	0.012876	4.06	170.38	153.08	1.23
within town	71.000	Max WS	649.52	1233.04	1234.57	1234.55	1235.14	0.008096	3.34	194.30	165.05	0.98
within town	70.000	Max WS	598.13	1231.81	1232.98	1233.06	1233.59	0.011207	3.46	172.98	177.90	1.12
within town	69.000	Max WS	681.48	1230.96	1232.33	1232.41	1233.01	0.010345	3.63	187.53	168.84	1.10
within town	51.000	Max WS	689.12	1204.93	1207.22	1207.54	1208.55	0.010478	5.49	145.35	93.52	1.23
within town	50.000	Max WS	705.53	1204.00	1206.54	1206.63	1207.55	0.008687	4.44	158.78	92.24	1.08
within town	48.000	Max WS	588.34	1202.00	1204.43	1204.44	1205.14	0.007165	3.77	162.85	132.55	0.96
within town	47.000	Max WS	644.20	1201.72	1203.31	1203.35	1203.95	0.009576	3.56	181.17	158.87	1.06
within town	46.000	Max WS	605.64	1200.00	1202.03	1202.03	1202.67	0.008030	3.56	171.76	144.85	1.00
within town	45.000	Max WS	669.00	1198.00	1200.34	1200.52	1201.32	0.009042	4.40	157.80	122.09	1.10
within town	44.000	Max WS	576.56	1197.25	1199.24		1199.80	0.006007	3.51	181.31	129.64	0.89
within town	43.000	Max WS	700.29	1196.00	1197.50	1197.87	1198.79	0.017967	5.04	139.02	116.02	1.47
within town	42.000	Max WS	554.78	1195.68	1197.31		1197.66	0.004269	2.61	212.86	162.46	0.73
within town	41.000	Max WS	678.32	1194.00	1195.34	1195.86	1196.93	0.025264	5.60	123.27	125.22	1.71
within town	40.000	Max WS	1327.80	1193.99	1196.02	1196.08	1197.00	0.007292	4.44	312.15	183.27	1.02
within town	55.000	Max WS	664.83	1210.00	1212.83	1212.98	1213.65	0.006430	4.48	194.66	154.26	0.96
within town	54.000	Max WS	617.39	1210.00	1211.76	1211.69	1212.47	0.006579	3.74	165.94	106.02	0.93
within town	53.000	Max WS	613.70	1208.00	1210.58	1210.74	1211.71	0.007624	4.91	139.16	81.08	1.05
within town	52.000	Max WS	605.78	1207.30	1209.93		1210.73	0.006214	3.98	152.36	81.23	0.93

Profile Output Table - Standard Table 1

File Options Std. Tables Locations Help

HEC-RAS Plan: 2ndtrial River: Dechatu Reach: within town Profile: Max WS

Reload D

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)	
within town	39.000	Max WS	598.61	1193.15	1194.28	1194.72	1195.79	0.033355	5.52	114.07	157.29	1.90
within town	38.000	Max WS	602.35	1190.93	1193.14		1193.48	0.003226	2.68	265.64	277.33	0.66
within town	37.000	Max WS	550.57	1190.08	1191.12	1191.68	1193.08	0.050881	6.20	88.83	118.64	2.29
within town	36.000	Max WS	679.34	1188.37	1190.70		1191.10	0.003137	2.85	248.87	154.24	0.66
within town	35.000	Max WS	576.33	1186.24	1188.17	1188.64	1189.68	0.018068	6.21	131.76	173.41	1.55
within town	34.000	Max WS	668.78	1186.00	1188.13		1188.46	0.002799	2.65	278.19	196.76	0.62
within town	33.000	Max WS	611.28	1184.73	1186.14	1186.64	1187.82	0.032485	5.78	110.36	175.68	1.90
within town	32.000	Max WS	621.74	1183.04	1185.06		1185.31	0.002756	2.45	305.14	249.68	0.61
within town	31.000	Max WS	509.54	1182.00	1182.77	1183.25	1184.57	0.061736	6.28	88.63	150.24	2.47
within town	29.000	Max WS	590.15	1180.61	1182.65		1182.88	0.002119	2.30	303.10	236.40	0.54
within town	28.000	Max WS	582.60	1179.05	1180.13	1180.65	1182.06	0.043916	6.42	103.68	171.65	2.18
within town	27.000	Max WS	608.18	1177.98	1179.99		1180.24	0.002192	2.24	281.34	193.65	0.55
within town	26.000	Max WS	578.96	1176.84	1178.00	1178.53	1179.89	0.042557	6.19	98.08	188.15	2.14
within town	25.000	Max WS	1345.02	1174.02	1178.86		1179.19	0.001164	3.03	596.47	208.33	0.46
within town	24.000	Max WS	503.88	1173.73	1174.72	1175.46	1177.59	0.068492	7.54	68.08	93.98	2.68
within town	23.000	Max WS	695.68	1172.00	1174.86		1175.37	0.002815	3.24	252.40	220.73	0.65
within town	22.000	Max WS	491.84	1170.88	1171.96	1172.37	1173.35	0.038121	5.66	98.29	146.90	2.01
within town	21.000	Max WS	712.76	1168.20	1170.58		1170.90	0.003241	2.99	309.26	220.34	0.68
within town	20.000	Max WS	507.79	1167.40	1168.38	1168.60	1169.15	0.022799	4.49	149.68	283.23	1.56
within town	19.000	Max WS	684.37	1166.00	1167.87		1168.12	0.003336	2.85	360.38	363.75	0.68
within town	18.000	Max WS	617.02	1162.00	1164.24	1164.42	1164.85	0.005253	3.68	240.07	453.40	0.85
within town	17.000	Max WS	672.74	1161.45	1163.43		1163.73	0.003373	2.88	299.32	227.49	0.68
within town	16.000	Max WS	539.84	1159.35	1160.49	1160.76	1161.41	0.029361	5.25	139.39	230.81	1.79
within town	15.000	Max WS	624.79	1156.50	1158.57		1158.83	0.002910	2.76	305.29	240.71	0.64
within town	14.000	Max WS	580.05	1155.24	1156.30	1156.53	1157.11	0.026972	4.95	160.34	285.50	1.70
within town	13.000	Max WS	572.11	1153.30	1154.95		1155.15	0.003114	2.32	308.63	287.51	0.63
within town	12.000	Max WS	639.63	1152.00	1153.01	1153.30	1153.87	0.020259	4.15	168.95	324.43	1.46
within town	11.000	Max WS	516.03	1150.00	1152.55		1152.73	0.001520	2.31	294.32	168.07	0.47
within town	10.000	Max WS	633.17	1148.00	1148.84	1149.28	1150.49	0.080980	5.69	111.29	239.55	2.66
within town	9.000	Max WS	239.00	1147.02	1148.70		1148.73	0.000417	0.83	362.97	465.79	0.23
within town	8.000	Max WS	735.92	1145.50	1146.28	1146.65	1147.75	0.072195	6.24	149.09	350.69	2.61
within town	7.000	Max WS	305.30	1144.40	1146.68		1146.72	0.000341	1.00	344.88	220.42	0.22
within town	6.000	Max WS	1373.60	1142.99	1145.92		1146.18	0.001645	2.57	633.23	282.05	0.51
within town	5.000	Max WS	1305.46	1141.32	1151.75		1151.76	0.000005	0.35	4304.75	433.04	0.03
within town	4.000	Max WS	1145.63	1139.39	1151.83		1151.83	0.000003	0.29	4603.61	388.28	0.03
within town	3.000	Max WS	1029.99	1138.20	1164.47		1164.47	0.000000	0.11	11240.13	439.87	0.01
within town	2.000	Max WS	493.91	1137.12	1138.04	1138.47	1141.32	0.121862	8.33	75.90	370.79	3.42
within town	1.000	Max WS	906.63	1136.00	1137.66	1137.10	1137.83	0.001735	1.94	524.78	379.51	0.48

7.3 Flood Plain Delineation- HEC-GeoRAS -Post RAS

Flood plain delineation is performed using HEC-GeoRAS Post processing and the result is shown below.

Flood inundated areas namely Thaiwan Tera, Coca Cola Factory and most part of the Dechatu stream areas are liable to flood as shown below.

Figure 7-8: Delineated flood plain

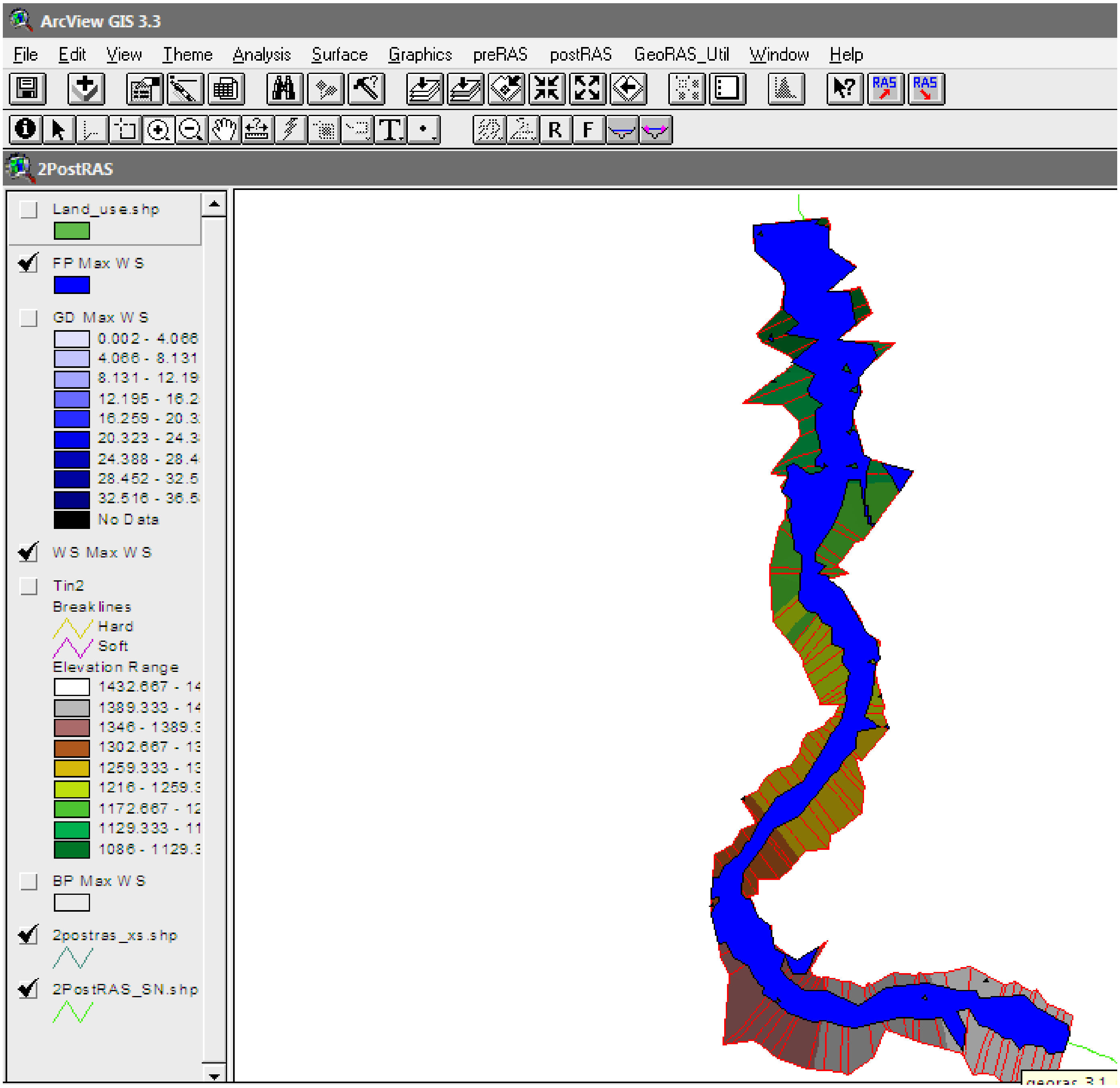
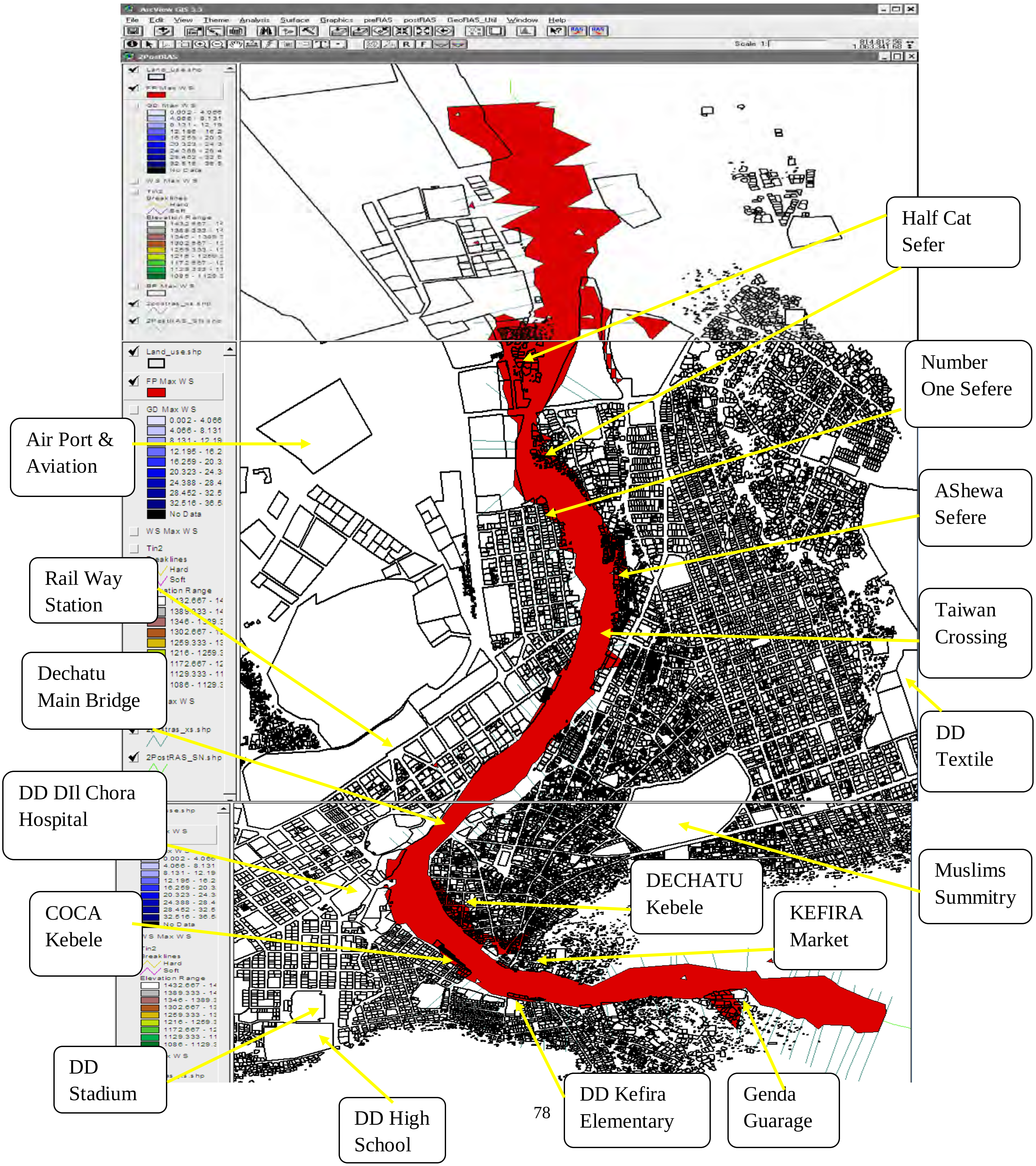


Figure 7-9: Flood Mapping Result



8 RECOMMENDETION AND CONCLUSION

8.1 Conclusion

The principal source of flooding in Dire Dawa town is a result of flash flood originating from rainfall on the southern highlands of Kersa, Lange and Dengego catchments. It is formed as a result of intensive showers, sparse vegetation cover and steep slope of the area.

The 50yrs return period peak flood discharge estimated using Computer Programs HEC-GeoHMS and HEC-HMS found to be 1187 m^3/sec and 1137m^3/sec for ERA Intensity and annual maximum daily rainfall as 6 hours duration storm respectively. These two figures/magnitudes are close to each other and double of annual maximum daily rainfall as 24 hours duration storm which is 602.6 m^3/sec.

As explained above, the annual maximum daily rainfall as 6 hours duration storm peak discharges for the 50 years return period value is chosen for the flood mapping.

Flood risk areas are delineated for 50yrs return period peak flood discharge using two models HEC_GeoRAS and HEC_RAS one after another (i.e first HEC_GeoRAS then HEC_RAS then back to HEC_GeoRAS) and Coca Cola Factory, Taiwan Tera, Ashewa Tera, left and right sides of Dechatu Stream are found to be the velnerable areas. The left and right sides distance range of Dechatu Stream which are affected by the 50yrs return period peak flood varies from place to place.

The use of HEC-GeoHMS and HEC-GeoRAS tool kits make the geospatial hydrology data more precise.

Finally, to select the type of mitigation measure it is recommended to undertake further flood damage analysis study instead of the traditional way of design. Also, there are programmes for this type of analysis like the hydrologic Engineering Center's Flood damage Reduction Analysis (HEC-FDA, or FDA

This contributes to change the trend of flood damage analysis which is an outstanding problem.

8.2 Recommendations

Based on the result obtained in this research, the following points are forwarded:

The use of HEC-GeoHMS and HEC-GeoRAS tool kits are powerful means for the geospatial hydrology data precision.

Traditional flood design methods are increasingly supplemented or replaced by risk-oriented methods which are based on comprehensive risk analysis. Besides meteorological, hydrological and hydraulic investigations such analyses require the estimation of flood impacts.

Among different mitigation measures such as dams, dyke, retaining wall, or other non structural measures, some of them are already recommended, designed and implemented; of course some are on the way to be implemented for Dechatu Stream. However, these analyses were traditional analysis especially the methods how the mitigation measures were selected or chosen.

This Thesis revealed that the basic problem lies on the method how these mitigation measures were selected.

Thus the intention of this thesis is set not to repeat the past and present trend like the one done in many of Dire Dawa catchment studies; but to give at least the major concept required to select the appropriate flood mitigation measures. For the detail analysis, it is recommend to be carried out further flood damage analysis study to select the appropriate types of mitigation measures which can satisfy risk-oriented methods which are based on comprehensive risk analysis.

But, here the major concept required to select the appropriate flood mitigation measures are recommended. After the flood is estimated and vulnerable areas are identified, a flood damage analysis follows. Also, there are programmes for this type of analysis like the hydrologic Engineering Center's Flood damage Reduction Analysis (HEC-FDA, or FDA) which may comprise the following major concepts.

HEC-FDA is inter-disciplinary program to formulate and evaluate flood damage reduction plans. The program performs economic (flood inundation damage analysis) and hydrologic engineering performance computations for plan evaluations.

During the course of a study, it is possible to formulate and evaluate several different plans. Then, selection of the mitigation measure can follow after the evaluation of these different plans.

A plan may represent the with- or without project condition. The first plan is always the without-project condition. The without-project condition is a hardwired plan. It is always listed first and cannot be deleted. The without-project condition is the plan which all subsequent plans are compared against.

The with-project condition plan consists of one or more flood damage reduction measures and action. Additional plans may contain levee, reservoirs, channels, non-structural measures, and other measures or combinations thereof. It includes all streams and damage reaches within the specified study limits which are Dechatu stream within Dire Dawa town in this case. The plan is evaluated over an analysis period (project life). It starts with the base year of implementation or operation. Static hydrologic engineering and economic conditions associated with a specified future analysis year are included to evaluate the equivalent economic and engineering performance of the plan over its project life.

Exceedance probability function should be defined for each plan for economic analysis.

Then, the results can be compared after the analyses of various plans are performed which enables the selection of the mitigation measure.

In addition, the following recommendations made by reference 12 are also accepted to be applicable for this case.

- Flood forecasting and emergency response centre in Dire Dawa that has linkage with various governmental, public, nongovernmental and private organizations.
- Early warning systems and reliable communication systems (mobile phone network, radio, television) to facilitate information sharing.
- Installation and operation of a real-time reporting network of rainfall and river gauges
- Proper watershed management for upper catchment.

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APPENDICES A: The Dimensionless Unit Hydrograph Used by SCS Method

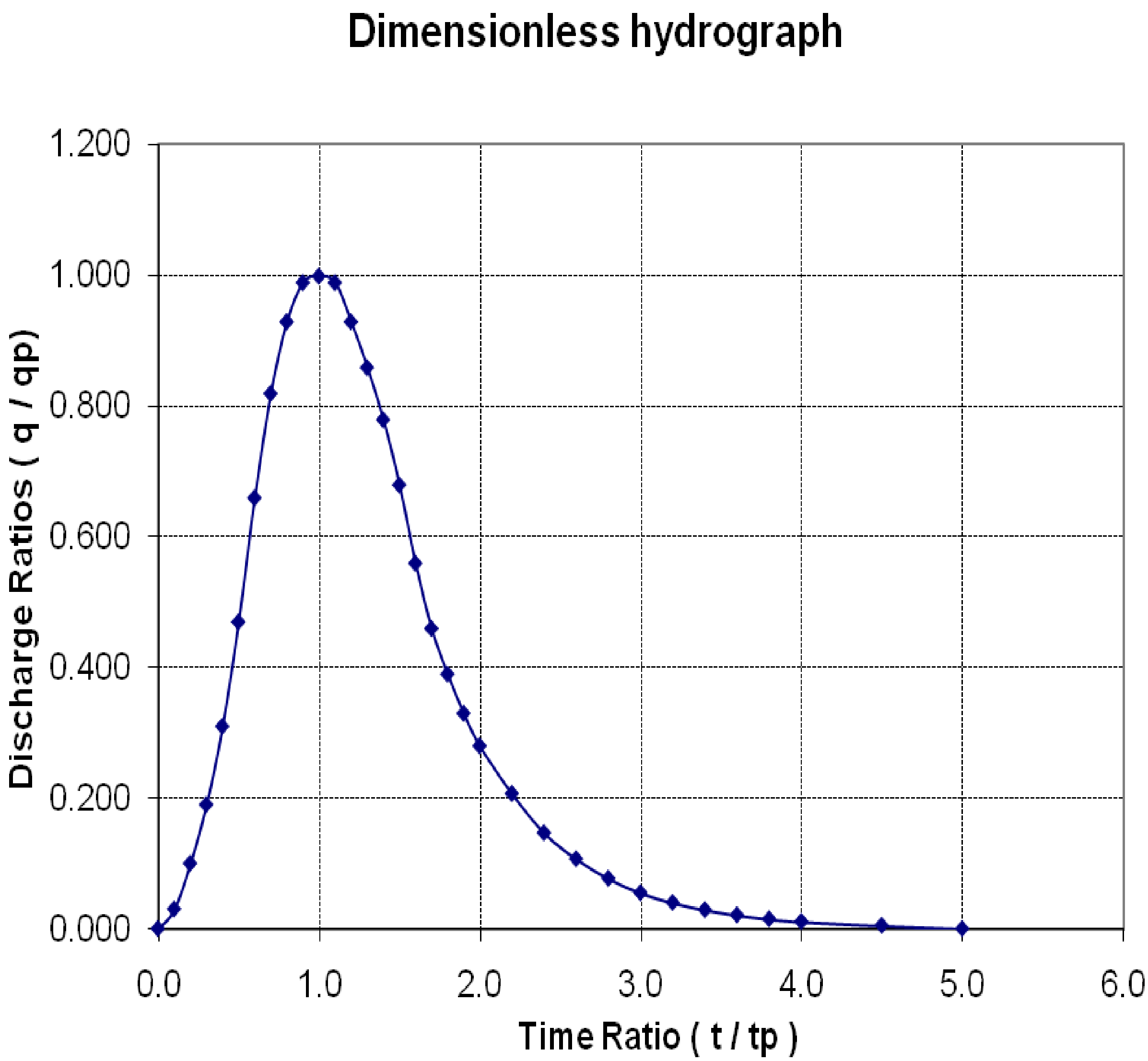

Table 0-1: Dimensionless Unit Hydrograph Values

Time Ratios (t / t_p)	**Discharge Ratios** (q / q_p)
0	0
0.1	0.03
0.2	0.1
0.3	0.19
0.4	0.31
0.5	0.47
0.6	0.66
0.7	0.82
0.8	0.93
0.9	0.99
1	1
1.1	0.99
1.2	0.93
1.3	0.86
1.4	0.78
1.5	0.68
1.6	0.56
1.7	0.46
1.8	0.39
1.9	0.33
2	0.28
2.2	0.207
2.4	0.147
2.6	0.107
2.8	0.077
3	0.055
3.2	0.04
3.4	0.029
3.6	0.021
3.8	0.015
4	0.011
4.5	0.005
5	0

APPENDICES B: Rainfall Data

Dire Dawa Monthly Rainfall

R.N	Year	Dec	Nov	Oct	Sep	Aug	Jul	Jun	May	Apr	Mar	Feb	Jan	Total	Max
1	1952	14.52	14.17	22.81	54	6.5	12.8	15.8	24.4	12.8	0.4	0	3.5	181.7	54
2	1953	0	1.5	22	28.2	7	30	20.8	31.6	20.8	0	2.3	52.1	216.3	52.1
3	1954	0	3.2	36	6.2	4	26.1	11.7	49.9	67	24.9	5.1	12	246.1	67
4	1955	14	0	10	26	12	6.4	17.2	26	30.2	4	0	0	145.8	30.2
5	1956	70	0	13.1	45	0	5	31.2	44.2	7.8	11.3	14.5	3	245.1	70
6	1957	12.2	60	0	42	16.8	12.8	53	28	0	10.3	2	0.2	237.3	60
7	1958	20.9	69.2	18.4	17.5	0	23	35	22	8.6	2.5	2	0	219.1	69.2
8	1959	0	0	14	0	0	0	43	47	27	2	18	0	151	47
9	1960	0.5	2	63	0	26	4	65	30.5	22.6	10.93	8.22	8.57	241.32	65
10	1961	0	0	19.9	36	17.5	0	49	43.7	18.5	0	45	0	229.6	49
11	1962	0	0	0	15.8	0	3.5	42	26.2	10.5	32	26	0.2	156.2	42
12	1963	0	0.2	0	33.5	43	17.5	24.5	50	43	0	16.8	7.7	236.2	50
13	1964	0.5	0	30	33	22	4	55.2	51	35	0	0	0	230.7	55.2
14	1965	0	0	2.1	57	0	0	12	52.7	15.5	12.2	8.1	0	159.6	57
15	1966	0	28	4	26.4	2.9	13	10.2	18.2	27.4	2.4	0	0	132.5	28
16	1967	1	0	16.2	22.1	34.2	11.9	42.8	41.3	29	7.4	48	0	253.9	48
17	1968	0	28	21.9	39.6	0.2	7	29	20.4	25	3.5	0	13.5	188.1	39.6
18	1969	62.5	43.5	41.9	30	1.7	6.2	41.3	29	4.1	6.5	0.2	0	266.9	62.5
19	1970	35.5	22.6	38.1	24.3	0	5.5	23	32.9	22.6	0	0	0	204.5	38.1
20	1971	0	0	29.3	3.3	16	2.5	44.6	27	17.1	0	27.4	0	167.2	44.6
21	1972	5	12	0	40.8	14.8	35.2	13	34	6.2	8.1	0	0	169.1	40.8

R.N	Year	Dec	Nov	Oct	Sep	Aug	Jul	Jun	May	Apr	Mar	Feb	Jan	Total	Max
22	1973	0	0	0	13.2	20.8	19	23	18.8	17.3	9.5	0	3.1	124.7	23
23	1974	8.9	4	0.5	0	29.7	29.7	30	29.3	35.1	12.7	0	0	179.9	35.1
24	1975	25.2	8	38.6	54.9	50.8	39.8	19.3	38.2	20	0.9	0	0	295.7	54.9
25	1976	0	5.8	13.5	24.9	9.9	16.2	27.5	30.4	21.5	1.2	22	9.3	182.2	30.4
26	1977	18	0.4	11.3	44	26.2	8.2	34.9	80.9	15.2	47	0	0	286.1	80.9
27	1978	0	76.5	5.3	3.2	3.9	4.9	24	47.3	8	27	0	0	200.1	76.5
28	1979	0	0	17.4	1.8	11	8.5	39.5	18.5	8.8	0	0	87	192.5	87
29	1980	79.2	0	11.6	15.9	18	9	13.7	19	23.7	30.8	11	0	231.9	79.2
30	1981	0	0.9	37.5	34	9.7	0	21.7	33.3	14.5	4.6	0	0	156.2	37.5
31	1982	44.1	37.5	22.9	32.1	32.6	0.1	9	43	27.9	17	20.9	4.5	291.6	44.1
32	1983	1.2	37.8	72.7	68.3	18.5	11.1	23.5	36	23.6	17.7	0	0	310.4	72.7
33	1984	0	0	8.2	23.6	27.2	11.2	30.5	30	12.3	0	7.2	0	150.2	30.5
34	1985	50	0	19.8	33	8	8.1	15.5	21.4	34.5	1.7	1.3	0	193.3	50
35	1986	0	23.2	46.7	51.1	23.6	17	22.3	11.3	19.2	0.2	0	0	214.6	51.1
36	1987	0	3.6	38.3	84.5	50	1.5	1.3	26.7	4.6	7	0	0	217.5	84.5
37	1988	19	43.3	5.4	86.4	0	5.8	47	27	24.2	14	0	1.4	273.5	86.4
38	1989	0	13.1	21.1	44	3.9	8.2	21.2	25.3	9.6	21.6	0	9.7	177.7	44
39	1990	45.4	57.4	23.4	31	14	6.1	22.8	24.5	16.5	3.6	0	0	244.7	57.4
40	1991	15.9	40	38.2	24.2	16.9	3.8	7.9	26	15.7	9.4	0	34	232	40
41	1992	6.2	30.5	12.3	18.5	22.7	6.3	30.2	34	31.8	23	40.8	17.9	274.2	40.8
42	1993	68.9	11.1	18	29.8	9.8	9.4	25.7	24.6	13.5	41	0	0	251.8	68.9
43	1994	0	0	21.9	61.3	38.2	3.7	37.3	58.4	28.8	1.1	43.6	0	294.3	61.3

R.N	Year	Dec	Nov	Oct	Sep	Aug	Jul	Jun	May	Apr	Mar	Feb	Jan	Total	Max
44	1995	0	32.8	29.6	40	7	25.8	18.3	43.7	8.7	0	3.5	8.57	217.97	43.7
45	1996	16.7	1	41.9	35.1	45.9	23.2	68.3	33.6	47.1	1	18.6	0	332.4	68.3
46	1997	51.6	0	33.7	50	13.8	30.2	21.3	33.1	4.5	49.3	7.3	0	294.8	51.6
47	1998	41.7	18.3	27.4	22.7	41.9	4.6	21.1	59.2	36.3	31	5	0	309.2	59.2
48	1999	0	1.3	41	12.3	6.6	9.9	33.1	32.5	18.4	17.5	2.4	1.7	176.7	41
49	2000	0	0	11.3	7.2	1.9	17.4	22.3	20.4	28.6	8.2	14.7	37.3	169.3	37.3
50	2001	0	0	41.1	13.7	26.4	16.6	27.8	55	18.7	11.9	4.7	6	221.9	55
51	2002	13.9	0	58	28.8	25.8	5	16.8	26.2	20.8	18	0	11.5	224.8	58
52	2003	12.3	5.9	14.7	36.8	1.4	45.9	32.4	21.4	24	2.1	0.8	113.1	310.8	113.1
Max		79.2	76.5	72.7	86.4	50.8	45.9	68.3	80.9	67	49.3	48	113.1	332.4	113.1
Mean		14.516	14.169	22.808	30.904	16.167	12.165	28.240	33.827	20.848	10.93	8.220	8.574	221.37	54.47
Min		0	0	0	0	0	0	1.3	11.3	0	0	0	0	124.7	23
St. Dev		21.860	20.330	17.018	20.000	14.344	10.857	14.366	13.274	12.194	12.654	12.966	21.263	52.36	17.87

Dire Dawa Annual Daily Maximum Rainfall

R.N	Year	Max	R.N	Year	Max
1	1952	54	36	1987	84.5
2	1953	52.1	37	1988	86.4
3	1954	67	38	1989	44
4	1955	30.2	39	1990	57.4
5	1956	70	40	1991	40
6	1957	60	41	1992	40.8
7	1958	69.2	42	1993	68.9
8	1959	47	43	1994	61.3
9	1960	65	44	1995	43.7
10	1961	49	45	1996	68.3
11	1962	42	46	1997	51.6
12	1963	50	47	1998	59.2
13	1964	55.2	48	1999	41
14	1965	57	49	2000	37.3
15	1966	28	50	2001	55
16	1967	48	51	2002	58
17	1968	39.6	52	2003	113.1
18	1969	62.5			
19	1970	38.1			
20	1971	44.6			
21	1972	40.8			
22	1973	23			
23	1974	35.1			
24	1975	54.9			
25	1976	30.4			
26	1977	80.9			
27	1978	76.5			
28	1979	87			
29	1980	79.2			
30	1981	37.5			
31	1982	44.1			
32	1983	72.7			
33	1984	30.5			
34	1985	50			
35	1986	51.1			