



ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

**THE PERFORMANCE ASSESSMENT AND REMEDIAL MEASURES OF BRIDGE
STRUCTURE**

(Case study on Bilate River Bridge)

A Thesis Submitted to Addis Ababa Institute of Technology, School of Graduate Studies,
Addis Ababa University

In Partial Fulfillment of the Requirement for the Degree of Masters of Science in Civil and
Environmental Engineering
Major in Hydraulic Engineering

By: -Mandaferow Aba

Advisor. Dr.Yilma Seleshi

**Addis Ababa
Ethiopia
Jun, 2019**

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Abstract

The bridge is an infrastructure which built across the river also used to pass safely from one side of the road to another to cross the river; on the road shall be kept strong to serve intended purposes. The bridge structure requires detail, assessment before and after construction both due to possible failures. Many countries have come upon different mitigation measure to overcome failure by providing bridge assessment. Ethiopian Road Authority follows up bridge structure failure by recording all necessary data about bridge and Road situation.

The Bilate River Bridge is the one among those encountered local abutment failure.

The abutment failure of the bridge was seen at different times, which is observed in the field assessment. The analysis was conducted on the abutment scour failure for a different return period of time. The existing bridge opening size was the 26m length, 5m wide. Both sides of riverbank material were taken and analysis is done in a laboratory result express that particle diameter of 1.5mm and 34.5mm for D_{50} and D_{95} respectively.

The maximum design flood estimation resulted in $284.2\text{m}^3/\text{s}$, and $327.3\text{m}^3/\text{s}$ at the Bilate River for 50-year and 100-year return period respectively. In the design flood estimation rainfall-runoff model analyzed by Initial and constant loss method and direct runoff by the SCS unit hydrograph method also base flow by recession method.

The Bridge instability evaluated by HEC RAS 5.0.1; has been prepared by taking inputs like; river cross section, design flood, and riverbank materials.

The local abutment scouring observed in the vicinity of the structure of the bridge incorporate with overtopping which endures Bridge to fail.

The model results show that dominate cause of abutment scours of the bridge was an inadequate opening size to pass design flood at the bridge; because of constriction at the bridge within the river has resulted in an increase the flow velocity so that, the abutment has been scoured. Countermeasures were reviewed based on the hydraulic analysis result calculated. Based on hydraulic analysis results; the recommended river training work and increase the opening size of a bridge across the river.

Keywords: Design Discharge, HEC-HMS, Hydraulic analysis, HEC-RAS, scour, Bilate, Bridge.

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List of Abbreviations

NMA	National Metrological Agency
ERA	Ethiopian Roads Authority
HEC-RAS	Hydraulic Engineering Center River Analysis System
HEC-HMS	Hydraulic Engineering Center Hydrologic Modeling System
amsl	above Mean See Level
GIS	Geographical Information System
OGL	Original Ground Level
SCS	Soil Conservation Service
CN	Curve Number
DEM	Digital Elevation Model
UH	Unit Hydrograph
DDM	Drainage design manual
CR	Constant Rate
DPA	Discharge per Area
RP	Ratio to peak
RC	Recession Constant
NSE	Nash and Sutcliffe Error
RVE	Relative Volumetric Error
PEP	Percent error in peak
DMC	Double mass curve

1. INTRODUCTION

1.1 General background

In order to protect drainage structures from any major problems of damage, identification of deteriorations are very essential. To reduce any trouble and difficulty of travel due to drainage structures, correct measures through proper maintenance and repair must be used. Proper drainage is essential for a highway to function properly. When the provided structures fail to accommodate the discharge the road is said to have the drainage problem.

Bridges are defined as structures that transport vehicular traffic over waterways or other obstructions, or part of a stream crossing system that includes the approach roadway over the floodplain, relief openings, and the bridge structure. Generally, bridge design comprises two important aspects; the Hydraulic and the Structural design. In the past, different studies conducted in the US and UK and the results showed that many bridge failures are due to hydraulic effects (USDOT, 2013). Unfortunately, engineers give greater attention to the Structural design than the Hydraulic design (USDOT, 2013). Undermining the bridge opening capacity, sediment and debris accumulation and scouring effects are the major problems in bridge hydraulic design and these factors play a great role behind the failures of bridge structure all around the world.

In road design it has been given more attention to the drainage systems for the highway designer, the primary focus is with the water that moves on the earth's surface and in particular that part which in our country (Mehari & Bogale, 2015), the attempt to alleviate the failures on the drainage structures is very little, even though the problem is so much larger. Many times side ditches, culverts, and bridges are found to be clogged, collapsed and washed away by the flood. Consequently, the quality of the roads is much deteriorated and their lifetime is shortened. To address these problems investigations are necessary. Special attention shall be given to the failures in bridge structures since any malfunction on these structures creates a wide-ranging problem. The development of infrastructures is vital for a country. Even if most infrastructures, roads are the basic ones. Out of the reasons, the failure of cross drainage structures comes first (Mehari & Bogale, 2015). In our country, there is a good work of expansion of roads, but many of them are not functioning well to the desired lifetime and quality. Thus giving attention for investigation of failures with the cross drainage structures is the far most important duty of the academicians as well as professionals. In my case, the failure of the Bilate River Bridge will be considered.

1.2 Significance of the Study

The study mainly focuses on the Bridge structural failure due to a different condition which is one of the problems in our counties Ethiopia and construction of road drainage structures require skilled manpower and require follow-up from responsible organization to reduce the impact on the Bridge structure. If the Bridge structures fail, high investment is required to maintain them in order to avoid traffic interruption. Therefore, the study is useful to the region for future Bridge structures, construction to avoid problems related to design as well as the performances of the existing Bridge structures with intended function. The study is estimated to propose appropriate solutions to the Bridge structure failure systems whose execution will contribute to the sustainability of the case study of the Bilate River Bridge.

The study is advantageous for academicians and researchers who conduct similar researches on other Bridge failure structures to address similar problems, scouring prevention mechanisms and aggradations/degradations of the stream channel. It may also support policymakers in their efforts to address similar problems

1.3 Statement of the problem

Currently, Ethiopia is investing billions of Birr in the construction of roads and highway structures in order to boost the economic activity of the country. Even though a huge amount of money is invested in the sector, due to gaps in detail study and design problems the country loses a huge amount of money each year.

The Bilate River Bridge under risk to give service, since local scouring at the abutment in both sides. The bridge is located in the steep area due also, skewed at 45 degrees due to this high flow velocity around abutment this facilitates erosion of soil near the bridge on the upstream side.

1.4 Scope and Limitation of the Study

The thesis is limited to the performance assessment of existing Bridge structures and the evaluation mainly done for only Alaba Kulito to Denboya road. The research does not include structural design of all types of Bridge structures except proposing and the size of the required Bridge structures. However, hydrologic analysis and hydraulic analysis considered for Bridge structures through this research work that is susceptible to failure are included.

1.5 Research question of the Study

The important questions that are addressed and considered are:

- What is the significant effect of scouring on Alaba to Denboya road?
- What are the major causes of Bridge structure failures on the Bilate River Bridge?
- What is the efficiency of the Bridge?

1.6 Objective of the study

1.6.1 General objective

The general objective of this study is the performance assessment of the bridge from the hydrologic and hydraulic analysis.

1.6.2 Specific objective

- To estimate scour depth of the Bridge.
- To estimate the overtopping height of the Bilate River Bridge
- To evaluate the adequacy of the Bridge opening

2. LITERATURE REVIEW

2.1 Introduction

The most common cause of bridge failures is from floods scouring bed material from around Bridge foundations. Scour is the engineering term for the erosion caused by water from the soil surrounding a bridge foundation piers and abutments (FHWA, 2001).

Designers and inspectors need to carefully study site-specific subsurface information in evaluating scour potential at bridges, giving particular attention to foundations on rock. Massive rock formations with few discontinuities are highly resistant to scour during the lifetime of a typical bridge (HEC 18, 1993). The hydraulic analysis is a critical component for the design of bridges that cross watercourses. Hydraulic analysis is not required for overpass bridges or similar structures as they do not convey water (USDOT, 2013) Because of the importance of hydraulics in the performance and safety of most bridges, hydraulic analysis should be undertaken by either the Hydraulics and Marine Studies Unit (HMS) of Engineering and Technology Branch or a suitably prequalified and experienced consultant.

2.2 Bridge failures

Bridge's resistance to severe damage or a complete loss, as a result of Hydraulic action, including sedimentation and scour needs to be minimized as much as possible. The end result of the reduction in Hydraulic performance of a Bridge. The most common reason for Highway Bridge failure is due to adverse hydraulic action. It is therefore crucial that sufficient consideration is paid to the prevention of such failures when designing Bridges over rivers or floodplains (HEC 18, 1993).

The main causes of failure of drainage structures are due to the following factors:

- Basin Characteristics: Size, shape, land use, geology, soil type, surface infiltration, and storage.
- Stream channel Characteristics: geometry and configuration, natural and artificial controls, channel modifications, aggradations, degradation, and debris.
- Flood plain characteristics.
- Meteorological characteristics: precipitation amount and type, storm distribution, storm direction and time of precipitation

Generally, the failures of drainage structures can be categorized into the following groups based on the above factors;

- Hydrological failure,
- Hydraulic failure,

- Failure due to aggradations or degradation, and
- Failure due to the orientation or location of the structure.

2.2.1 Hydrological failure of Bridges

Hydrological failures mainly associated with the determination of the peak discharge that the Bridge is capable to pass without having significant problems. Fewer input data, error in data analysis and selection of inappropriate modeling methods will lead to overestimation or underestimation of peak discharges which will result in a reduction or other design of Bridge opening size (USDOT, 2013). Over design, the results increase in the total cost of the structure, whereas the reduction in opening size will result in total failure of the Bridge during peak flow seasons of the year.

2.2.2 The Hydraulic failures of Bridges

Generally, evidence suggests that there are three principal factors involved in Bridge failures related to hydraulics which occurs either singly or in combination which include an inadequately sized opening, scour, and accumulation of debris and sediment (Les Hamill, 2004). In Ethiopia, it is very challenging to put in number Bridge failures due to hydraulic actions, but it is very clear that Bridge failures due to difficulties related to hydraulic design exist through the country. The hydraulic failure of the bridge area can be classified as follows: These are:-

A. Inadequate opening size

Bridge opening size has a major effect on the hydraulic performance of the Bridge structure. Whenever the opening size under the Bridge span gets smaller then it acts as a barrier to the flow and causes the inundation of a large residential or commercial area on the upstream side, then the Bridge can hardly be considered to be adequate, even if it survives the flood event (HEC 18, 1993). Inadequate size of Bridge opening may be a result of an error during the design phase or river morphology change due to the construction of hydraulic structures on the main channel. The construction of the hydraulically ineffective Bridge could cause flooding upstream, or it expands the problems which already exist. Consequences of hydraulically inefficient Bridge could be extremely damaging and expensive if a large number of properties or factories were flooded, so it is important to make sure that a hydraulically effective structure is designed and the backwater level calculated accurately.

B. Bridge Scour

Overall score includes scouring from sources that often cannot be adequately quantified through analytical studies. This class includes scouring caused by bending migration,

fluctuating downstream water surface elevations that control backwater elevations through the bridge, and channel morphology characteristics, such as the scour hole at the junction of two streams. The possible scour in the vicinity of the bridge due to the design flood. General scour is often measured from field inspections, aerial mapping, and the projected worst case for general scour at a bridge. For more serious conditions one or more multidimensional numerical model(s) or a physical model of the bridge may be needed to best evaluate scour (Dakota S, 2011).

I. Contraction Scour

Contraction scour occurs when a channel's cross-section is reduced by natural or artificial features. Possible constrictions include the construction of long causeways to reduce bridge lengths (and costs), the placement of large (relative to the channel cross-section) piers in the channel, abutment encroachment, and the presence of bluffs. The reduction of cross-sectional area results in an increase in flow velocity due to the conservation of flow similarly, this may cause the situation of more sediment leaving than entering the area and thus an overall lowering of the bed in the contracted area. This process is known as contraction scour.

II. Live bed contraction scour equation

The live bed scour equation assumes that the upstream flow velocities are greater than the sedimentation critical velocity.

III. Clear-water contraction scour

The Clear-water scour equation assumes that the upstream flow velocities are less than the sedimentation critical velocity.

IV. Local Scour

Local scour occurs at and within the bridge opening itself and is influenced by a variety of parameters relating to the bridge piers and abutments.

V. Scour at piers

Piers cause the flow to pass around each pier and also move vertically down the pier face toward the channel bottom. Pier scour is affected by the number and the shape of the piers, the angle of attack of the flow on the pier, the type of pier foundation, and the debris and ice carried by the stream. Turbulence around piers results in vortex systems and excessive scour, especially during flood events. As flow approaches the pier, it decelerates, theoretically coming to rest on the face of the pier, causing a stagnation pressure. Stagnation pressures are highest near the water surface, where velocities are higher than toward the channel bottom (HEC 18, 1993).

The change in stagnation pressure creates a downward pressure gradient along the face of the pier, forcing the direction of flow along the pier downward. This velocity down the pier, if great enough, begins to move particles from the bottom of the pier, causing scour at this location.

VI. Scour at Abutments

Contracting flow may break sharply into the bridge opening at the abutments, causing flow concentration and resulting in abutment scour. Key factors in abutment scour include the abutment shape vertically or spill through the abutment location in relation to the channel banks, and the incorporation of upstream spur dikes or guide walls.

VII. Lateral Scour

This type of erosion is caused by the horizontal movement of the channel, without necessarily creating a deeper channel. Meandering streams shift laterally in the floodplain, with the meander loops moving slowly in the downstream direction, potentially causing difficulties at bridges. Bridges cross wide floodplains often permit substantial floodplain flow through the bridge opening, as well as flow from the channel. If the velocities in the channel cause erosion of the backline, the main channel may move laterally and displace into a portion of the floodplain under the bridge superstructure; Because, piers located in the floodplain may not be as substantial or as deeply based as piers in the channel, such a situation may result in the loss of one or more piers in the overbank area and potentially the loss of the bridge itself.

C. Debris accumulation and Sediment Aggradation

I. Debris accumulation

Accumulation of trash and debris against Bridge components can significantly affect the hydraulic performance of Bridges. Difficulties are normally associated with small single span Bridges which tend to be more easily blocked than large multi-span structures. This may result in a large increase in water level upstream and associated flooding. Debris may partially restrict the flow leading to trivial scour around piers or abutments threatening the safety of the structure (ERA, 2013). Bridge piers, particularly those in the center of the main channel can collect large quantities of debris, which significantly reduces the Hydraulic performance. This should be taken into consideration at the design stage (Les Hamill, 2004). Debris which is caught against or between piers can result in improved hydraulic forces by increasing the effective pier width. The extent of these forces is not easily predicted and usually be most severe when the river is in flood (AASHTO, 2014).



Figure 2.1: Bridge in Arba Minch completely blocked by Debris accumulation

Source: (ERA, 2013)

II. Sediment Aggradation

Sediment aggradation at the Bridge, the cross-section has also resulted in a serious problem in the overall Hydraulic performance of a Bridge. The highest consequence of channel bed aggradation and fill is to reduce bridge clearance. However, aggradation may continue to the extent where new hazards like meanders cutoff and channel pattern change will also generate. In addition, aggradation may lead to bank erosion as bar formation change flow paths, and decreased channel capacity will increase flooding with the possibility for damage to the bridge and its approaches (Newlin, 2007).



Figure 2.2: Sediment Aggradation on Segen Bridge (Right) and Baso Bridge (Source: (Beza N, 2010))

2.3 Bridge and culvert analysis

The following methods were performed both hydrological and hydraulic modeling approaches (Eng J, et al., 2016). The SCS method and rational method have been used. This is because it is applicable for areas which do not have sufficient rainfall and streamflow used for bridge peak discharge estimation and rational method estimating runoff as a function of runoff coefficient, frequency factor, rainfall intensity and area which used for culverts culvert peak discharge estimation. Hydraulic analyses of River Bridge and culverts have been computed using HEC RAS 4.1.0 and HY8 respectively; from the rate of flood runoff (discharge) and the volume of runoff that will pass through the bridge. The results, according to (Eng J, et al., 2016). Peak discharge computation; from the computed drainage characteristics above the peak flood of the three catchment areas is computed. And the summarized and rainfall intensity and peak discharge, from 5 up to 100-years return period of the three drainage areas are the results as shown in the Table below.

Agulae River Bridge: the bridge is not adequate for 50-years return period steady flow; there is overtopped by 0.24 m to the left and right farming lands due to the size of the bridge have a small size in comparison to the incoming flow. Scouring condition of the bridge: In order to perform Agulae river bridge scouring analysis; first computing the mean size fraction of the bed material (D50) of the river bed channel and left and right over banks using the sieve method and water temperature was adjusted itself by default when the SI unit adjust to metric. Hydraulic performance of the existing culverts: Outlet velocity and Froude number of the two structures are very high at both Kehen culvert and Betehaweriat culvert but the structures have enough capacity to the incoming flow up to 100 years return period. According to (Eng J, et al., 2016). This research does not look into the dynamics of the morphology of the channels, future studies should focus to consider these as well as there are forming a lot of gullies due to a road crossing and side channel construction in many places; so researchers have to do a lot of work in this topic. Finally, Construction of road and road drainages must have in consideration of hydrologic and hydraulic designs to minimize the effect of erosion and socioeconomic impact of these road drainages. According to (Hailu N, 2016), the hydrologic modeling (HEC-HMS) is performed primarily to generate peak discharge (Rainfall-Runoff) estimation and flood hydrographs with certain statistical return periods with single design storm events by the same statistical return periods which are derived from IDF curves of the study area. HEC RAS model for hydraulic analyses which estimate water surface profile as well as sediment modeling approach both scour, and aggradations in the bridge location. Based

on this the models shown below were chosen for each of the four components of the runoff process.

According to (Mehari & Bogale, 2015) the general workflow is presented as follows:

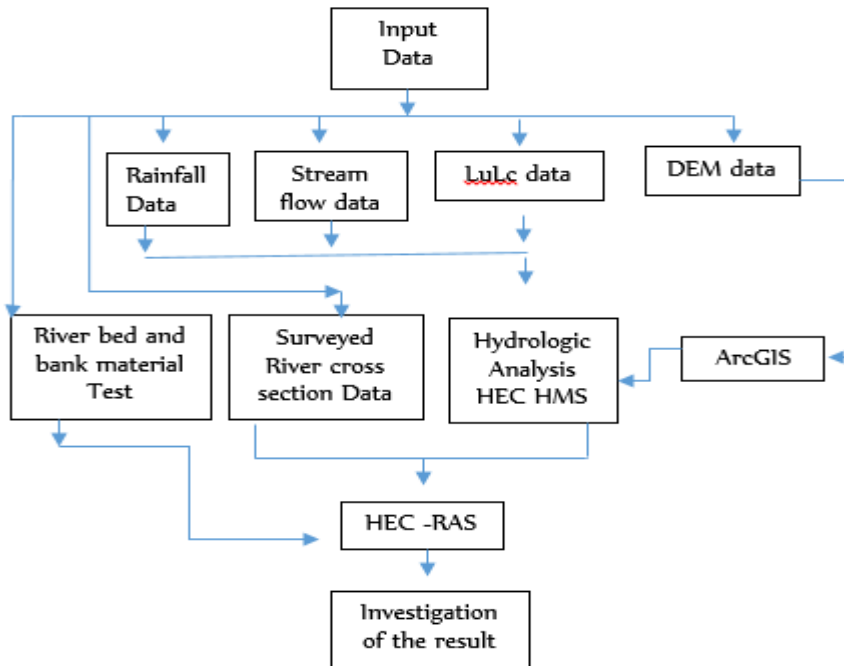


Figure 2.3: Conceptual Frame Work (source: - Mehari B, et al., 2015)

The results, according to (Mehari & Bogale, 2015) are as follow in sile river bridge analysis. The peak discharge on the River Sile at the bridge outlet is estimated to be 208.4m³/s after transferring parameter from kulfo watershed to sile watershed; because sile watershed was engaged catchment then after calibration and validation of HEC-HMS model at a different period. The evaluation of the bridge opening capacity: The Hydraulic model results (HEC-RAS) show that the bridge is not enough to pass the discharge for the 100yr design discharge. There was a 3m clear height of the bridge and additional 3m scour totaled into 6m height for passage of water, though the discharge still can't be accommodated with this opening. The river bank is still being scoured heavily every wet season. The Bridge should be located in another position where the river reach is straight. Since the area is flat, besides the main bridge some additional culverts should be provided to accommodate the surplus discharge from the main channel (Mehari & Bogale, 2015).

Table: 2.1 List of selected models.

Methods	Selected model
Loss method	NRCS curve number (CN) method
Transform method	NRCS dimensionless UH method
Routing method	Muskingum method
Base flow method	Not considered

The results and discussion tell us the hydrologic analysis result show that at 50-year design discharge fail. The water surface profile reveals this fact.

2.4 Hydraulic model systems

The computer-based hydraulic analysis is required for the design of bridges. However, decisions must be made regarding the type of model computational methods, model's extent, and amount of topographic data that needs to be collected (ERA, 2013). An assessment of flow resistance caused by channel and floodplain conditions needs to be made and the impacts of flow due to different seasonal conditions also need to be evaluated. An understanding of flow type, historic flow conditions, and flooding at the site also provides valuable insight into the model approaches that need to be employed. Once the preliminary data have been collected and an understanding of the flow complexity of the bridge site is obtained, a decision must be made regarding the type of hydraulic model that should be used in the hydraulic crossing. Some situations call for a one-dimensional gradually-varied steady-state flow model while others require the use of unsteady flow models, or two-dimensional steady or unsteady flow models to more fully understand the flow conditions at the hydraulic crossing (Dakota S, 2011). Some situations call for a more sophisticated modeling approach because of other factors. These can include the need for a more complete understanding of the flow conditions because of bridge scour or bank stabilization.

2.4.1 HEC-RAS model

Evidence suggests that there are three principal factors involved in Bridge failures related to hydraulics which occurs either singly or in combination which include an inadequately sized opening, scour, and accumulation of debris and sediment (les Hamill, 2004).

The three major laws of hydraulic calculations. These are: -

- ✚ Conservation of mass (continuity principle);
- ✚ Conservation of energy;

✚ Conservation of momentum.

Depending on what information is available and what is needed every design calculation basically involves the application of one or more of these laws.

The primary purpose of HEC RAS model was to analysis the opening sizes, scour, and sediment aggradations of the highway cross drainage structures from the rate of peak flood and the volume of runoff that will pass through the bridge (HEC RAS, 2016).

Computes based one-dimensional steady flow, unsteady flow, sediment transport/mobile bed computations and water temperature modeling. For steady flow water surface profiles, the system can handle a full network of channels, a branching system or a single river reach.

- The steady flow component is capable of modeling subcritical, supercritical and mixed flow regimes, water surface profiles (Dakota S, 2011). Also: The basic computational procedure is based on the solution of the one-dimensional energy equation. Energy losses are evaluated by friction (Manning's equation) and contraction/expansion (coefficient multiplied by the change in velocity head).
- The momentum equation may be used in situations where the water surface profile is rapidly varied. These situations include mixed flow regime calculations (e.g., Hydraulic jumps), hydraulics of bridges and evaluating profiles at river confluences (e.g., Stream junctions).
- The effects of various obstructions (e.g., Bridges, culverts, weirs, structures) in the floodplain may be considered in the computations. The steady flow system is designed for application in floodplain management and flood insurance studies to evaluate floodway encroachments. Also, capabilities are available for assessing the change in water surface profiles due to channel improvements and levees.
- Special features of the steady flow component include multiple plan analyses, multiple profile computations, multiple bridges and/or culvert opening analyses and split-flow optimization

2.4.2 FHWA Hydraulic Toolbox

The FHWA Hydraulic Toolbox Program is a standalone suite of calculators that performs routine hydrologic and hydraulic computations. The program allows a user to perform and save hydraulic calculations in one project file, analyze multiple scenarios and create plots and reports of these analyses (Dakota S, 2011).

Five calculators are available:

- Channel Analysis (HEC 15, 1988)

- Weir Analysis (HEC 22)
- Rational Basin Analysis (HDS 3)
- Detention Basin Analysis (HEC 22)
- Curb and Gutter Analysis (HEC 22)

2.4.3 Watershed Modeling System (WMS) channel calculator

The Channel Calculator is a feature of the WMS, The Channel Calculator determines the shear stress for the given discharge and uses the allowable shear stress for the lining selected to determine the safety factor (Dakota S, 2011).

Another open channel modeling software is ISIS. ISIS is a full hydrodynamic simulator for modeling flows and levels in open channels. ISIS is able to model complex looped and branched networks, and is designed to provide a comprehensive range of methods for simulating floodplain flows. ISIS incorporates both unsteady and steady flow solvers, with options that include simple backwaters, flow routing, and full unsteady simulation. The simulation engine provides a direct, steady-state solver and adaptive time-stepping methods to optimize run-time and enhance model stability (ERA, 2013).

2.5 Hydrological modeling system

Hydrology is the study of the properties, distribution, and effects of water on the earth's surface, and in the soils, underlying rocks, and atmosphere. For the purpose of this manual, hydrology will deal with estimating flood magnitudes as the result of precipitation (VIEUX, 2004). In the design of highway drainage structures, floods are usually considered in terms of peak runoff or discharge in cubic meters per second and hydrographs as discharge per time (ERA, 2013).

Errors in the estimate design discharge will result in a Bridge structure that is either undersized and causes more drainage problems or oversized and costs more than necessary. On the other hand, it is important to realize that any hydrologic analysis is the only estimation. Although some hydraulics analysis is necessary for all highway drainage structure design, the extent of such studies should be commensurate with the hazards associated with the hydraulic structures and with other concerns, including economic, engineering, social, and environmental factors. Because hydrology is not an exact science, different hydrologic flow estimation methods developed for determining flood runoff may produce different results for a particular situation. Therefore, the engineer should exercise sound engineering judgment to select the proper flow estimation method or methods in estimation design flows. While performing the hydrological and hydraulic analysis for the design of highway drainage systems, the hydraulic engineer

should recognize and evaluate potential environmental problems that would impact the specific design of a drainage structure early in the design process (ERA, 2013).

2.5.1 HEC HMS model

The Hydrologic Modeling System (HEC-HMS) is designed to simulate the Rainfall-runoff processes of watershed systems. It is applicable to large river basin water supply and flood hydrology and small urban or natural watershed runoff. Hydrographs produced by the program are used directly or in conjunction with other software for studies of water availability, urban drainage, flow forecasting, future urbanization impact, reservoir spillway design, flood damage reduction, floodplain regulation, and systems operation.

2.5.2 HEC-HMS Model Description

HEC-HMS requires daily precipitation, daily evapotranspiration, the flow of the basin (for simulation run, calibration, and validation), DEM and river of the basin to get the simulated runoff as output.

HEC-HMS Model setup consists of four main model components: basin model, meteorological model, control specifications, and input data (time series, paired data, and gridded data). The Basin model contains the hydrologic element and their connectivity that represents the movement of water through the drainage system (HEC, 2006). The meteorological component is also the first computational element by means of which precipitation input is spatially and temporally distributed over the river basin. The meteorological model component calculates the precipitation input required by a sub-basins element. Control specification components set the time span of a simulation run. Input data components, such as time series data, paired data, and gridded data are often required as a parameter for boundary conditions in the basin and meteorological model.

2.5.3 Flood Frequency analysis

Program PeakFQ provides estimates of instantaneous annual-maximum peak flows for a range of recurrence intervals, including 1.5, 2, 2.33, 5, 10, 25, 50, 100, 200, and 500 years (annual exceedance probabilities of 0.6667, 0.50, 0.4292, 0.20, 0.10, 0.04, 0.02, 0.01, 0.005, and 0.002, respectively). The Pearson Type III frequency distribution is fit to the logarithms of instantaneous annual peak flows following Bulletin 17B guidelines of the Interagency Advisory Committee on Water Data. The parameters of the Pearson Type III frequency curve are estimated by the logarithmic sample moments (e.g., Mean, standard deviation, coefficient of skewness) with adjustments for low outliers, high outliers, and historic peaks and generalized skew (Dakota S, 2011).

2.5.4 Watershed Modeling System

The Watershed Modeling System (WMS) is a comprehensive graphical modeling environment for all phases of watershed hydrology and hydraulics. WMS includes powerful tools to automate modeling processes such as automated basin delineation, geometric parameter calculations, GIS overlay computations (e.g., CN, rainfall depth, roughness coefficients), cross-section extraction from terrain data and many more. With the release of WMS 8, the software now supports hydrologic modeling with HEC-1 (HEC-HMS), TR-20, TR-55, Rational Method.

2.5.5 Rational Method

The Rational Method provides estimates of peak runoff rates for small urban and rural watersheds of less than 50 hectares (0.5 square km) and in which natural or man-made storage is small. It is best suited to the design of urban storm drain systems, small side ditches and median ditches, and driveway pipes. It shall be used with caution if the time of concentration exceeds 30 minutes. Rainfall is a necessary input for this method of flow estimation.

2.5.6 SCS method

The Natural Resources Conservation Service (formerly Soil Conservation Service) developed the runoff curve number method as a means of estimating the amount of rainfall appearing as runoff. Technical Release 20 employs the Runoff Curve Number Method and a dimensionless unit hydrograph to provide an estimation of peak discharges and runoff hydrographs from complex watersheds. The procedure allows the designer to estimate the effect of urbanization, channel storage, flood control storage, and multiple tributaries.

2.6 Model selection

The hydrological practice would be improved if the models were objectively chosen on the basis of making the best use of the information available and following some systematic procedure of selection and verification. Generally speaking, items that should be considered in the selection process include:

- a) The nature of the physical processes involved,
- b) The use to be made of the model,
- c) The quality of the data available and
- d) The decisions that rest on the outcome of the model's use.

2.6.1 Criteria of selection

Thus far the problems to be considered in choosing a suitable model, in general, have been discussed. In most situations, however, absolute, objective methods of choosing the best model for a particular problem have not yet been developed, so this choice remains a part of the art of hydrological modeling. Suggested four criteria that can be used to choose between alternative models:

1. Accuracy of prediction,
2. The simplicity of the model,
3. Consistency of parameter estimates and
4. The sensitivity of results to changes in parameter values

Through hydrologic model selection, different conditions need to be taken into consideration. Depending on the situation on the ground, which is being modeled and the available data on hand, an appropriate mathematical model for each model component needs to be chosen.

In this Research, the hydrologic modeling (HEC-HMS) is executed mainly to generate flood hydrographs with certain statistical return periods resulting from single design storm events with the same statistical return periods which are derived from IDF curves of the study area also, Bilate River Bridge was founded in the gauged catchment.

Table 2.2: list of selected models

Methods	Selected model
Loss method	Initial and constant
Transform method	SCS unit hydrograph
Routing method	Not considered
Base flow method	Recession

3. RESEARCH METHODOLOGY

3.1 STUDY AREA

The Bilate Watershed was located in the Rift valley basin, It rises on the southwestern slopes of Mount Gurage near 6°2'N and 38°7'E, flowing south along the western side of the Great Rift Valley, to empty into Lake Abaya at 6°37'54"N and 37°59'6"E.

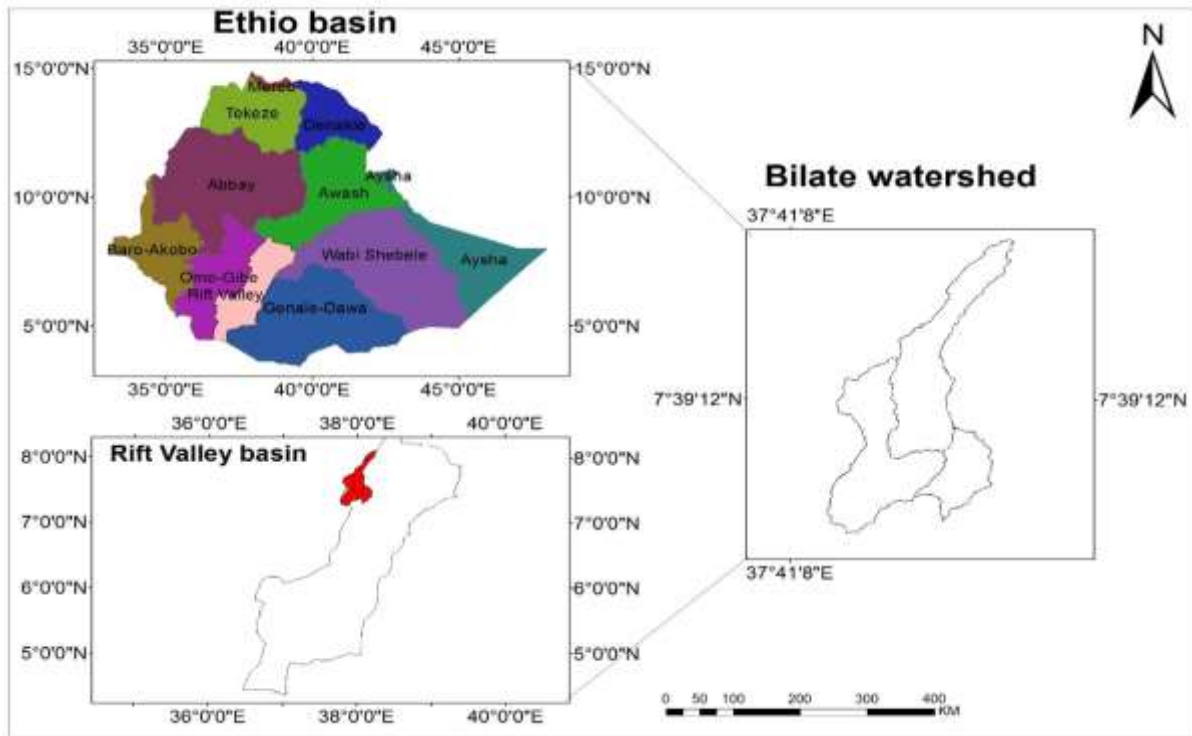


Figure 3.1: Location of Bilate River watershed

3.1 Location of the bridge

Bilate River Bridge is constructed in South Nation, Nationality People Region that joins Demboya to Alaba Kulito main road from West to East direction. It is constructed on the Bilate River as a cross drainage structure on the road. The River is part of the Rift valley basin, which finally joins Lake Abaya. The Bridge is located at 7°19'11.72"N latitude and 38°4'33.34"E longitude as shown in Figure 3.2 below.

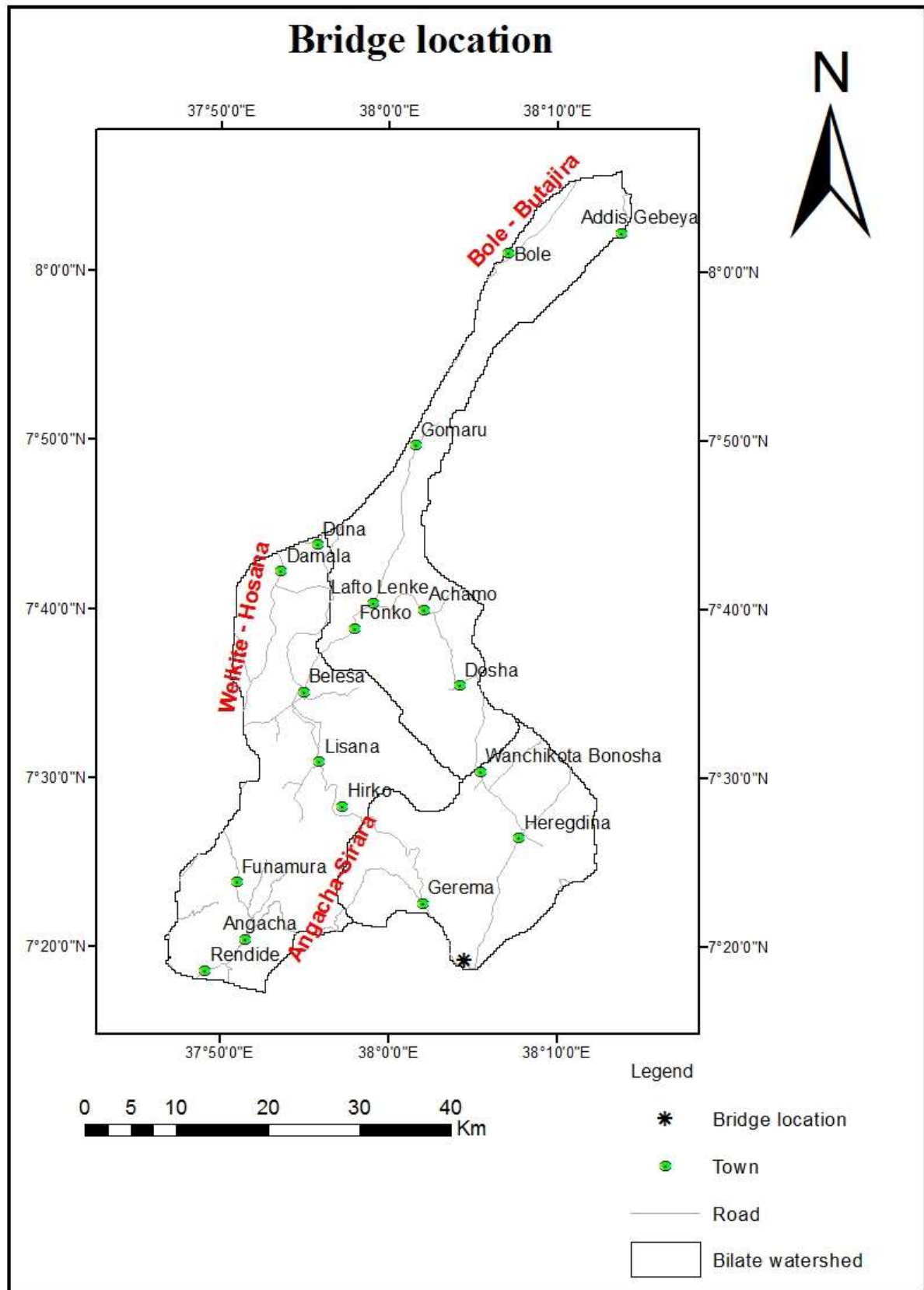


Figure 3.2: Bridge location



Figure 3.3: The Bilate River Bridge

3.2 Dataset used

For this research, National Meteorological Agency data were used. Observed Precipitation data, Temperature data, and streamflow data as well as river cross-section data collected during a field visit in the study area at the Bilate River Bridge. The Ministry of Water Irrigation and Electricity, National Meteorological Agency and Ethiopian mapping agency were visited to collect secondary data.

3.2.1 Rainfall

Bilate watershed was the moderate tropical semi-arid climatic condition. Lower altitudes are known for high temperatures and bimodal rainfall distribution, with the first rainy season from March-May and the next rainfall season that covers the monsoon season (June–October). Mean annual total rainfall varies from 600mm-1,680 mm in the basin and the average annual rainfall variability is linearly correlated to the altitude in the watershed. The observed daily time series rainfall data collected at six gauging stations from the national meteorological agency of Ethiopia, which shown below that used for HEC-HMS model simulation to determine Peak discharge in the Bridge site.

Table 3.1: Rainfall Station, coordinates, elevation, length of record period, and source of data.

S.No	Station	Coordinate		Elevation (amsl)	Length of Record Period	Source of Data
		Long	Lat			
1	Alaba Kulito	38.1128	7.3316	1802	1989-2016	NMA
2	Hosanna	37.8475	7.5393	2236	1953-2016	NMA
3	Fonko	37.9651	7.6421	2271	1986-2016	NMA
4	Wulbarge	38.1203	7.7363	2229	1987-2016	NMA
5	Angecha	37.8502	7.3392	2265	1982-2016	NMA
6	Butajira	38.3667	8.15	3283	1953-2016	NMA

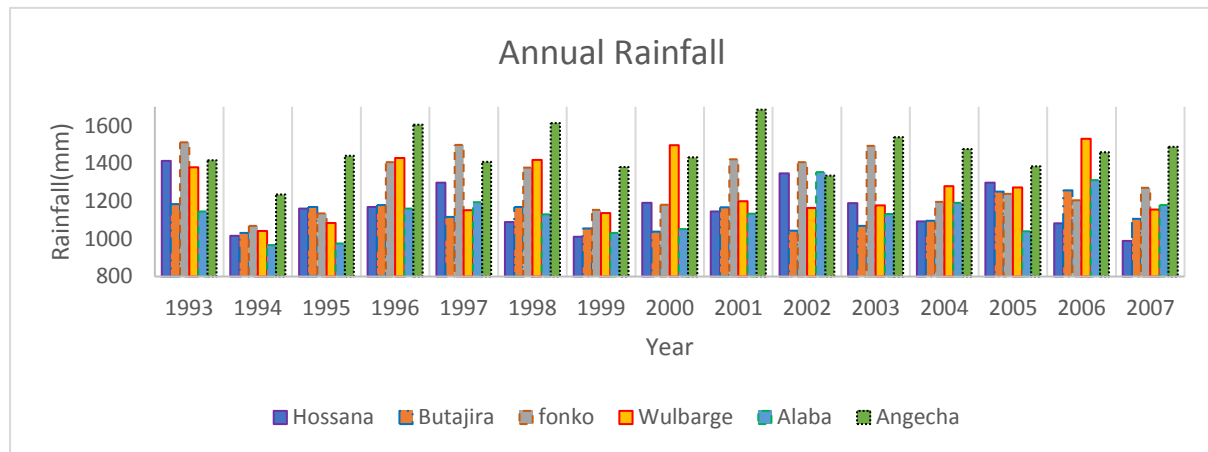


Figure 3.4: Annual rainfall data

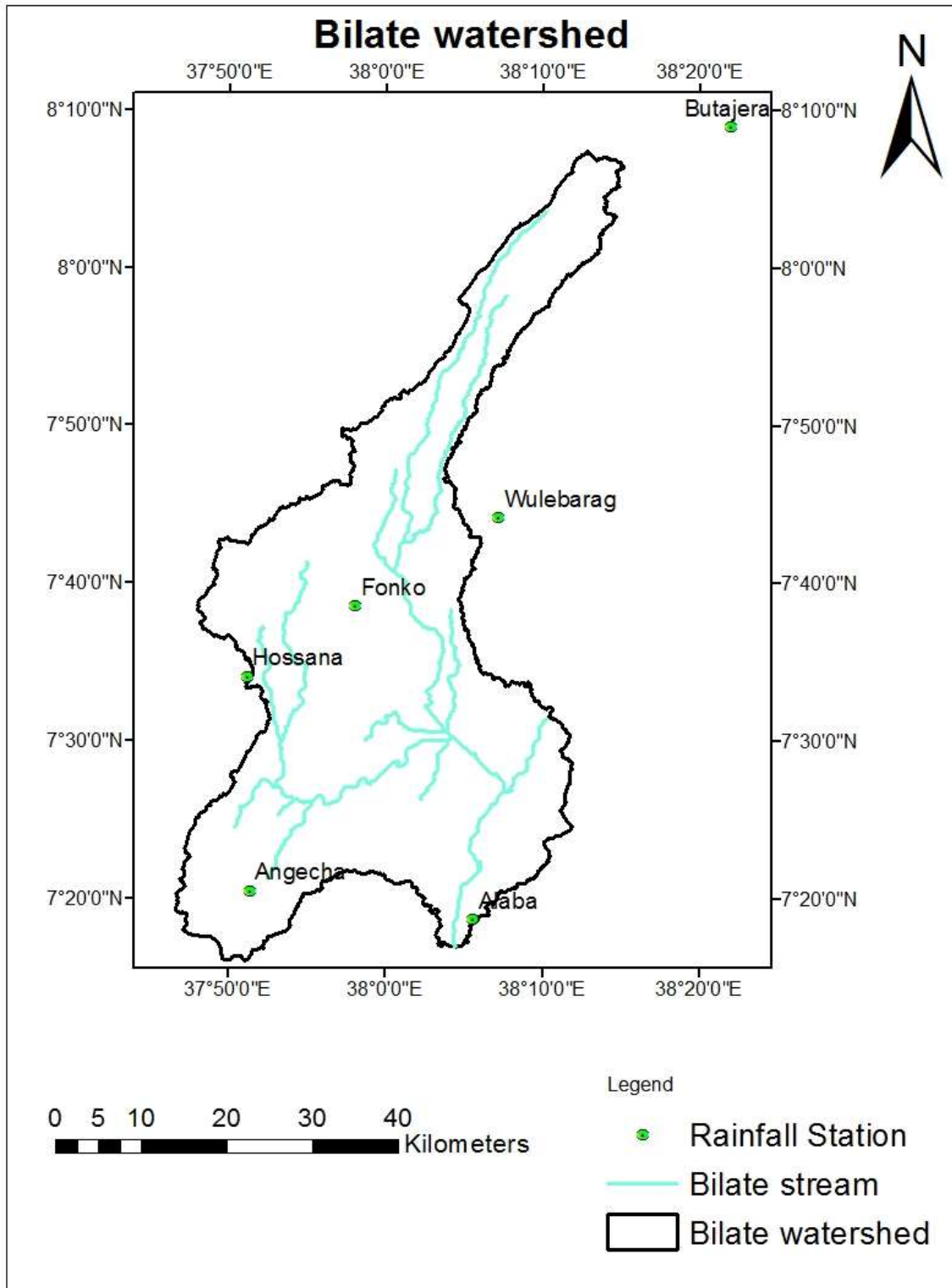


Figure 3.5: Bilate watershed Rainfall Station

3.2.2 Topography

The topography of the watershed was varied between 1614amsl up to 3357amsl and about 60% of the Bilate watershed area is located at an elevation greater than 1,800amsl Bilate watershed

covers 1968.0km² area catchments extending from near Gurage to lower regions of the Rift valley basin drainage system and tributary for Lake Abaya.

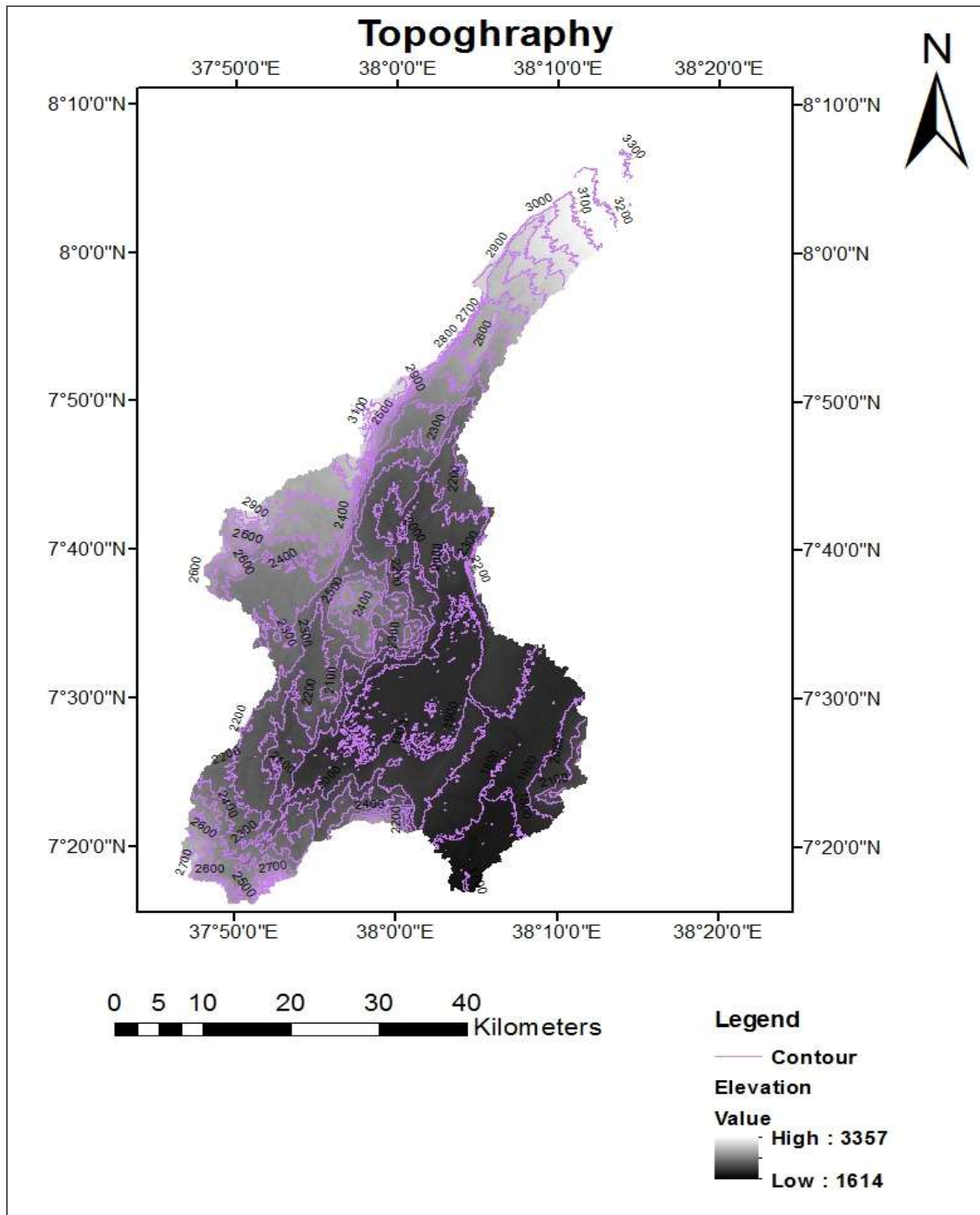


Figure 3.6: Topography of Bilate watershed

3.2.3 Socioeconomic activities

In the upper and lower areas of the Bilate watershed are characteristically suitable for agricultural production; however, frequent weather anomalies substantially affected the sector and endangered the socioeconomic of the region. A large fraction of the watershed cultivable

area is occupied by household farming. On the other hand, medium-scale estate and private-owned farming units dominantly produce cash crops like cotton, tobacco, and maize.

The lower flat plains, adjacent to the primary river course, utilize irrigation water for crop production and for this reason antagonism for irrigation water is eminent (Negash.Wagesho, Goel, N.K., & Jain, 2012).

3.2.4 Land use/land cover

The investigations of watersheds for land use dynamic are capable means of characterizing constantly changing catchment behavior on the basis of multiple considerations. The rural cultivated area and bare soil are increased by 23.1% again erodible soil, unsuitable for crop production raised to 17% in 2000 compared to baseline in 1973's. Percentage change in cultivated land and rural settlement area increased up to 64% of Bilate sub-catchment (Wagesho, Jain, & Goel, 2013). The main reasons in an increase of the above problems are; Deforestation, cattle overgrazing, and timbering substantially reduced the vegetation cover in the watershed. However, on aggregate, the rangelands increased by 26.7%, whereas the pasture land units decreased by 43.8% (Wagesho, Jain, & Goel, 2013). The decrease in pasture land might be the result of the growing demand for available land for crop cultivation in most parts of the watershed. Land units that lost its fertile topsoil formation due to excessive erosion and weathering activities are commonly located as small patches in the middle and lower Bilate sub-watershed (Wagesho, Jain, & Goel, 2013). In the watershed, greater than 30% was intensively cultivated area this also one of the most dangerous activities because of it increases rainfall-runoff in the Bilate River bridge which contributes overtopping of the structure in a short period of time.

Table 3.2: land use/ land cover, 2006

S.No	Land use/land cover	Area in Km ²	Percentage of Total area
1	Dense mixed high forest	211.16	12.27
2	Moderately cultivated	544.58	31.64
3	Open grassland	847.65	49.2
4	Perennial crop cultivation	67.94	3.95
5	Perennial marsh	46.27	2.69
6	Water body	3.72	0.22

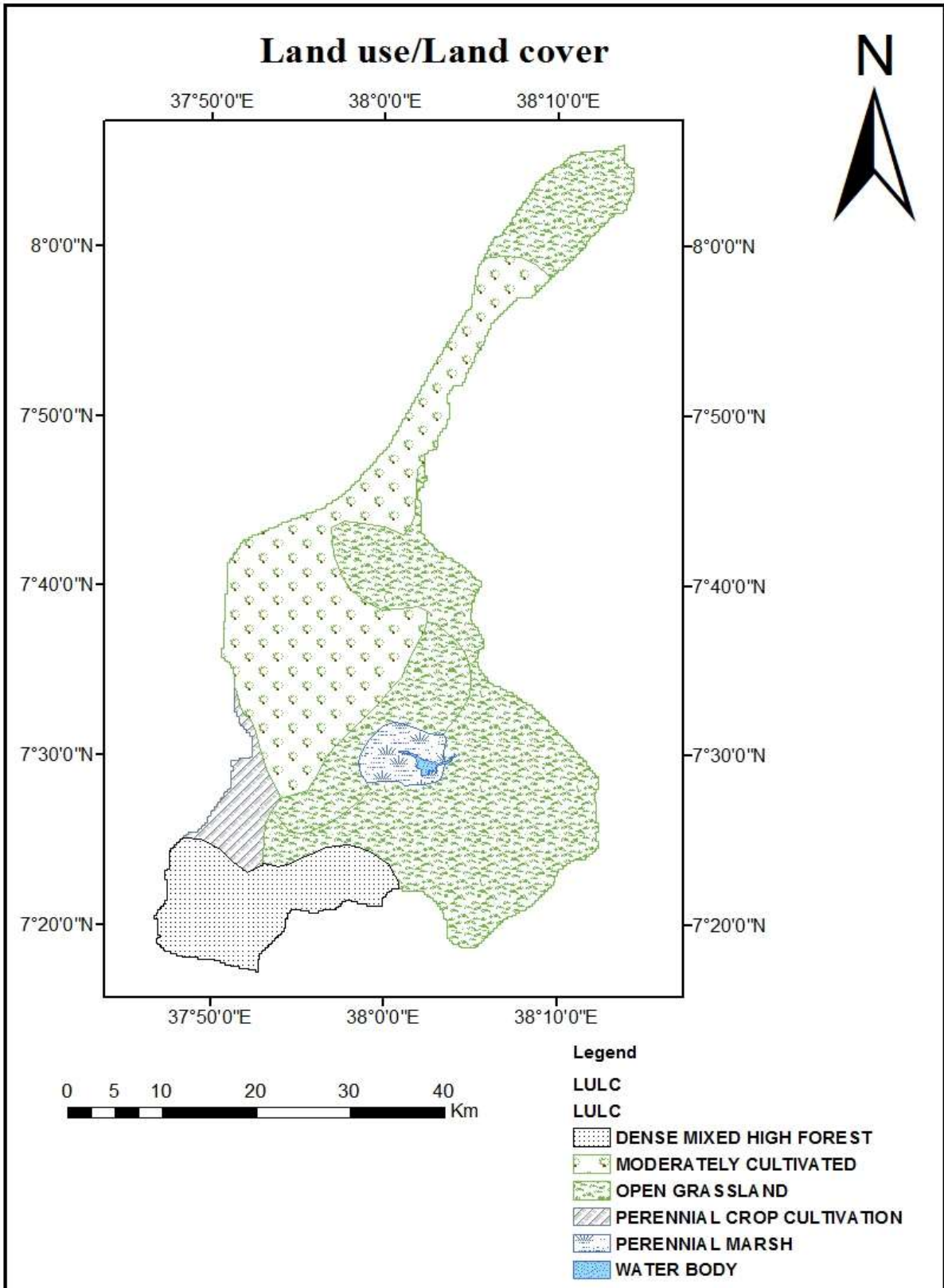


Figure 3.7: Land use/land Cover of Bilate watershed

3.2.5 Soil type

Soil data: soil map of Bilate watershed was placed on the map below. The major soil, which exists in the study area are Chromic Luvisols, Chromic Vertisols, Dystric Cambisols, Eutric Nitisols, Lithosols, Luvic Phaeozems, Pellic Vertisols, and Vitric Andosols. Soil data are one of the required data for estimation of erosion, infiltration, depression storage, groundwater estimation and sediment capacity estimation of the catchment area which contributes both aggradations and degradation which is effected on the bridge site from upper catchment to the river channel which carries suspended load and bed load to the River at the bridge location and data obtained from minister of water resources, irrigation and electric.

Table 3.3: soil type, 2000

S.No	Soil type	Area in km ²	Percentage of the total area
1	Pellic Vertisols	122.44	7.11
2	Eutric Nitisols	201.74	11.72
3	Lithosols	136.39	7.92
4	Chromic Luvisols	667.53	38.78
5	Vitric Andosols	484.83	28.17
6	Chromic Vertisols	43.14	2.51
7	Luvic Phaeozems	54.84	3.19
8	Dystric Cambisols	10.41	0.6

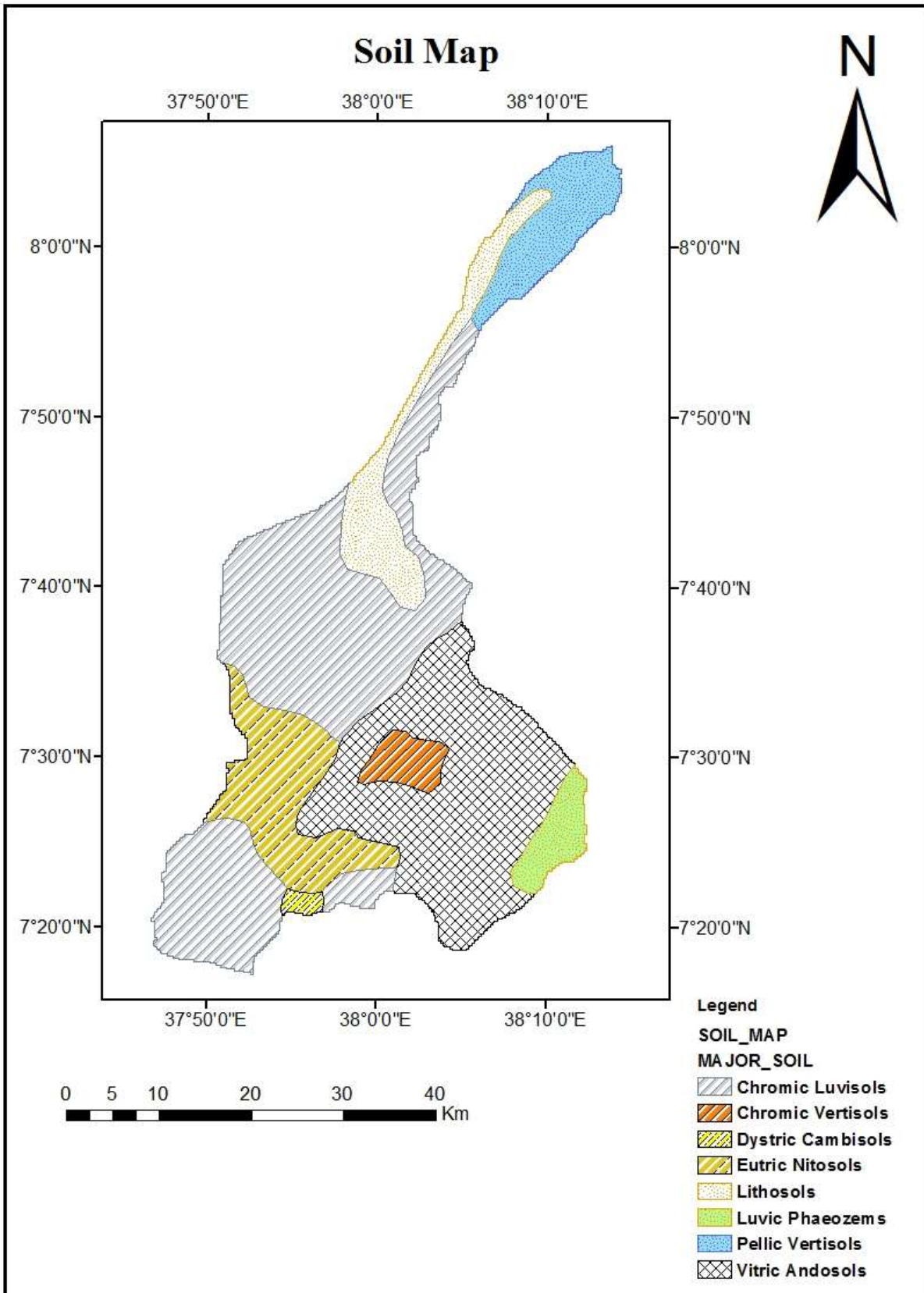


Figure 3.8: Soil map of Bilate watershed

3.2.6 Streamflow data

Bilate River Bridge was gauged near Alaba Kulito at the bridge outlet. The daily streamflow at Alaba Kulito of Bilate catchment is characterized by increasing trends in the analysis period. Observed time series daily streamflow data obtained from the MoWIE.

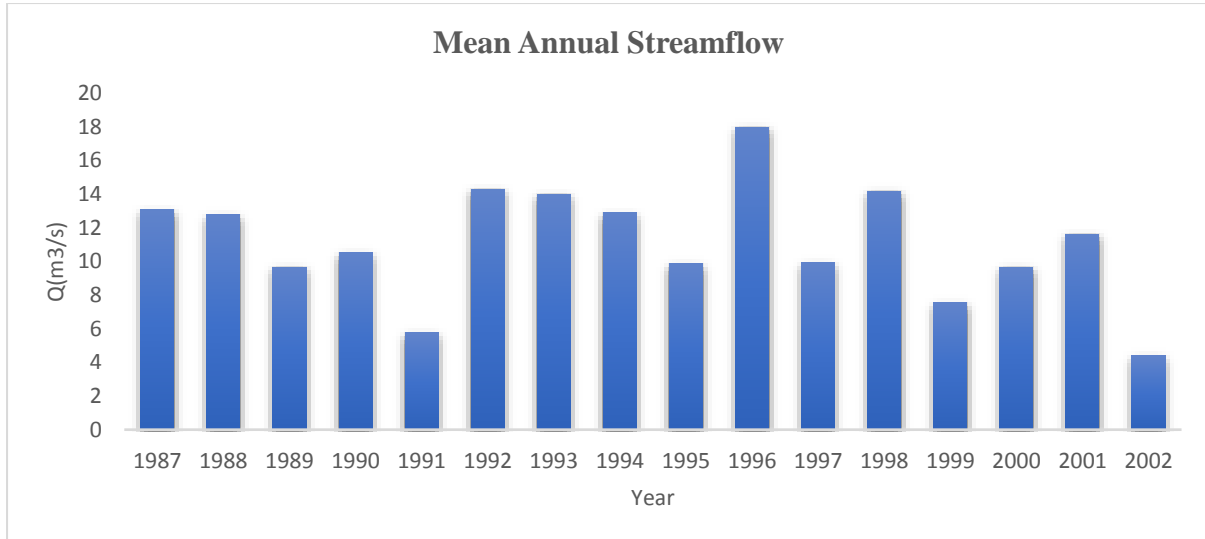


Figure 3.9: streamflow data

3.3 Data analysis

3.3.1 Estimating Missing Precipitation Data

In the analysis of hydrological data, the stations required to have daily records for the required period of simulation. It may so happen that a particular rain-gauge may not operative, shifting of gauge station, blockage and reading error for a part of a month or year (since it may break or for some other reason) hence it will become necessary to supplement the missing records. In this research, I use (arithmetic mean) to fill the missed records for the stations with less than 10% missed records while for the stations having greater than 10% of missed records, neighboring stations will be used.

3.3.2 Consistency Test

The data consistency of rainfall can be checked by using a double mass curve. Rainfall data must be checked for continuity and consistency before they are analyzed for any purpose. Double Mass Curve compares the accumulated annual rainfall at a given station with the concurrent accumulated values of average rainfall for a group of the surrounding stations. The rainfall values in station x beyond the period of change are corrected using the relation

$$P_{cx} = P_x \frac{Sc}{\delta a} \dots \dots \dots 3.1$$

Where: - P_{cx}=corrected value of rainfall at station x at any time t

PX =original recorded the value of rainfall in station x at any time t

Sc =corrected slope of the double mass curve

δa =original slope of the curve

3.3.3 Test for outliers

An outlier is an observation that deviates significantly from the bulk of the data, which may be due to errors in data collection, or recording, or due to natural causes. The presence of outliers in the data causes difficulties when fitting a distribution to the data.

Low & high outliers are both possible and have different effects on the analysis.

The Grubbs & Beck (1972) test (G-B) May be used to detect outliers. In this test, the quantities XH& XL are calculated by using equations shown below:-

$$XH = \exp(XM + KNS) \dots \dots \dots 3.2$$

$$XL = \exp(XM - KNS) \dots \dots \dots 3.3$$

Where XH and S are the mean and standard deviation of the natural logarithms of the sample respectively. The G-B statistic tabulated for various sample sizes and significance at (5%-10%) significance level, the following approximation is used, where N is the sample size.

$$KN = -3.62201 + 6.2844N^{\frac{1}{4}} - 2.49835N^{\frac{1}{2}} + 0.491436N^{\frac{3}{4}} - 0.037911N \dots \dots \dots 3.4$$

3.3.4 Average Areal Catchment Rainfall

Point estimates of rainfall record need to be changed to areal averages. There are different methods which are used to extend point estimate to areal average, among those methods Theissen Polygon method which assigns weights to the rain gages according to the proportions of the total watershed area that are geographically closest to each of the rain gages is the best option for this study.

3.3.4 The sampling techniques for Sieves analysis

The test procedure is adapted from University of Illinois laboratory procedure, including the sieve analysis with the following procedures; 1) Write down the weight of each sieve as well as the bottom pan to be used in the analysis, 2) Record the weight of the given dry soil sample, 3) Make sure that all the sieves are clean, and assemble them in the ascending order of sieve numbers. Place the pan below the last sieve. Carefully pour the soil sample into the top sieve and place the cap over it, 4) Place the sieve stack in the mechanical shaker and shake for 10 minutes, 5) Remove the stack from the shaker and carefully weighs and record the weight of each sieve with its retained soil. In addition, remember to weigh and record the weight of the bottom pan with its retained fine soil.

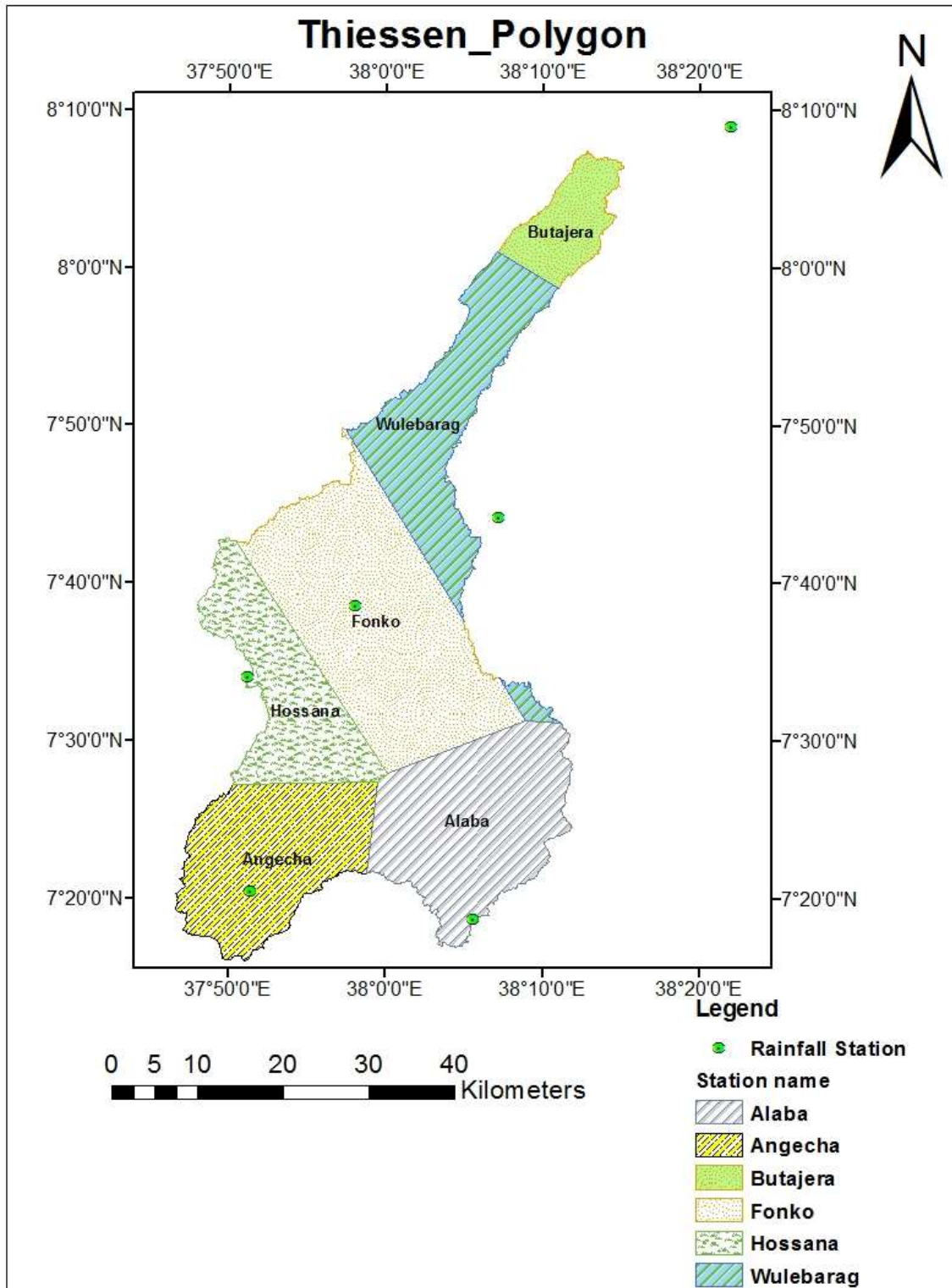


Figure 3.10: Thiessen polygon

Table 3.4: Contribution area of the Rainfall stations

Number of stations	Station Name	Contribution Area(Km ²)
1	Hossana	264
2	Butajira	106
3	fonko	581
4	Wulbarga	326
5	Alaba Kulito	365
6	Angecha	327
Total Area		1968

3.4 Rainfall-Runoff Model

For rainfall-runoff modeling, the conceptual semi-distributed model HEC-HMS (USACE-HEC, 2008); chosen, which comprises typical concepts used for flood simulations and allows sufficient fast computations with larger data sets. The reason for the selection of this model is that; it is not data demanding, it is also easy to use, run and produce simulations of discharge. HEC-HMS simulates precipitation-runoff and routing processes, both natural and controlled (USACE, 2000). It did not require long periods of simulation and it can acquire and upgraded freely.

3.4.1 HEC-HMS Input Data Pre-Processing

The extraction of hydrologic information from a DEM is accomplished through GIS applications. This study combined GIS with HEC-HMS by HEC-GeoHMS, and analyzed the model suitability for the study area.

The methodology in this section divided into four major sections: 1) Obtaining the geographic locations of the study area; 2) DEM processing, delineating streams and watershed characteristics, terrain processing, and basin processing; 3) Importing the processed data to HEC-HMS and 4) Merging the observed historical data with the processed DEM for model simulations.

A geographic location Bilate watershed is extracted from the whole Rift valley, River basin DEM (30m by 30m). The outlet is in Alaba Kulito. HEC-GeoHMS use DEM of the study area to generate HEC-HMS input file, stream network, sub-basin boundaries, and connectivity of various hydrologic elements in an ArcView GIS environment via a series of steps called terrain pre-processing and basin processing. Importing the processed data to HEC-HMS and finally merging historical data with processed DEM for model simulations.

In order to increase for better performance of modeling, the study area is considered as one Subbasin with the outlets in near Alaba Kulito, The Bilate watershed classification is primarily adapted from the strategic River Basin plan for the Rift valley Basin. According to this report

Rift valley, River Basin is classified into two sub-basins like; upper and lower Subbasin to facilitate water resources planning in the Basin. In this study, the upper stream Subbasin covers the study area. The sub-basin classification is mainly based on evaluating the geographical locations of each sub-basin in the study area and their contributing tributaries based on their geographical locations. The other is depending on their flood magnitude and their potential risk on the area and the area downstream of the sub-basins.

3.4.2 Sensitivity Analysis

Model sensitivity analysis is the process of analyzing the influence of each input or parameter by changing the value of one or more of them at a time and it is an important method to identify sensitive parameters for a model to avoid overparameterization. It is essential to know which parameter of the model has a high influence on the model output or in hydrography and potentially useful in all phases of the modeling process such as model development, model calibration, and model validation (McCuen, 2003). Model parameters are ranked based on their contribution to the overall error in model predictions. The most sensitive parameter corresponds to a greater change in output response.

Initially, the model is run with the base data. The model will be used again and the output values will be analyzed to determine variation with respect to the initial estimates of the parameters. Greater percentage change in the simulated volumes represents greater variable sensitivity. Each parameter will be changed individually while keeping all other parameters constant. The final set of the parameters of the calibrated model was deemed as a baseline/nominal parameter set. The hydrographs resulting from the scenarios of adjusted model parameters were then compared with the baseline model hydrography.

3.4.3 Model Calibration and Validation

Model calibration is a systematic search for optimal parameter value which attains the match of simulated and observed flow hydrograph. The procedure of calibration illustrated in figure 4.2 below. Model validation is a process of testing the model's ability to reproduce the pattern of hydrograph which is observed outside the calibration period with reasonable accuracy.

In this study infiltration loss was simulated using initial and constant loss model. This model has a quasi-continuous method of calculating rainfall losses. The method uses a single soil layer to account for continuous changes in the moisture content and the soil has assumed to have fixed water holding capacity (Halwatura and Najim, 2013). Initial loss (mm), infiltration rate (mm/hr) and percentage of impervious area are the three parameters to be defined in the initial and the constant loss model.

The SCS unit hydrograph method used to model the transformation of rainfall excess in to direct surface runoff. This method was originally developed to compute the peak flow as a unit of precipitation. Analyzing the direct runoff they convert the effective rainfall over a watershed into a flood hydrograph at the outlet of the watershed. These models account for the surface roughness and geometry of the watershed. The Soil Conservation Service (SCS) suggested a parametric UH model. The SCS suggests that the UH peak and time of UH peak are related by:

$$Q_p = C \frac{A}{T_p} \dots \dots \dots 3.5$$

Where: - A = watershed area; and

C = conversion constant (2.08 in SI and 484 in foot-pound system).

Q_p= peak discharge

The time of peak (also known as the time of the rise) is related to the duration of the unit of excess precipitation as:

$$T_p = \frac{\Delta t}{2} + t_{lag} \dots \dots \dots 3.6$$

Where 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.

The Recession base flow method is designed to approximate the typical behavior observed in sub-basins when the channel flow recedes exponentially after an event. Two methods are available in the model for specifying the initial condition: initial discharge and initial discharge per area (Scharffenberg, W. A. and Fleming, M. J., 2006). Here initial discharge per area was selected after several trials over initial discharge. The other parameter is recession constant (K), which describes the rate at which base flow recedes between storm events. Its initial value was determined by the equation derived from equation 4.7. There are two different methods for determining how to reset the base flow during a storm event: ratio to peak and threshold flow (Scharffenberg, W. A. and Fleming, M. J., 2006). In this study ratio to the peak was selected after many trials over threshold flow. The relationship of the base flow (Q_t) at any time t, initial base flow at time zero (Q₀) and recession constant (K) defined by:

$$Q_t = Q_0 K^t \dots \dots \dots (3.7)$$

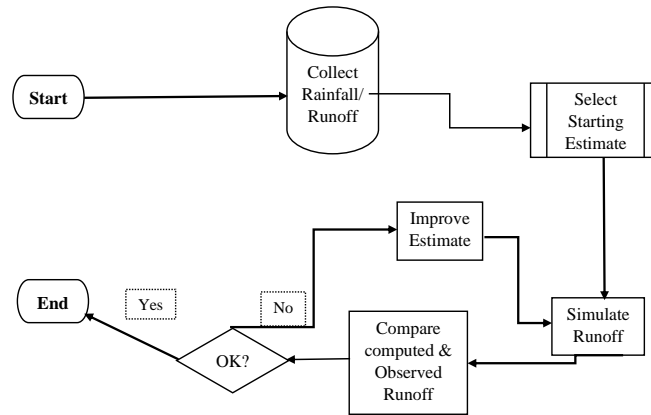


Figure 3.11: Schematic of HEC-HMS calibration procedure

Source: (HEC Application Manual, 2016)

Initial Parameter Estimation

Each method in HEC-HMS has parameters and the values of these parameters should be entered as input to the model to obtain the simulated runoff hydrographs. Some of the parameters may be estimated by observation and measurements of stream and basin characteristics. In this study, rainfall loss calculation is done by the initial and constant method. This method has three parameters: initial loss which is the amount of water required to fill the soil layer to the maximum storage, maximum storage is the amount of water the soil layer can hold to the maximum extent and constant rate defines the percolation rate when the soil layer is saturated. The range of values for the above parameters is presented in table 3.5 below. The initial value for the three parameters is determined by trial and error from the value range given in table 3.5, which yields the best fit between the observed and simulated runoff. The percent of the impervious area was generated from land use, land cover of the study area using ArcGIS. Urban settlement and rock outcrop were considered as impervious.

Table 3.5: calibration parameter constraints (HEC, 2006)

Method	Model	Parameter	Unit	Minimum	Maximum
Loss	Initial and Constant	Initial constant	mm	0	500
		Constant Rate	mm/hr	0.1	300
Transform	SCS Unit Hydrograph	Initial Abstraction	mm	0	500
		Lag time	min	0	30000
		Curve number	-	1	100
Base flow	Recession	Initial Discharge	m ³ /s/km ²	0	100000
		Recession Constant	-	0.00001	-
		Ratio to peak	-	0	1

SCS unit hydrography has two parameters, lag time and area. Lag time is calculated using equation 3.5. Parameters of recession base flow are mainly determined from the observing flow

of each gauging station used for the sub-basins. In this study, the initial condition is specified by the initial discharge per area. Initial base flow is the average flow of January of time series data. Initial discharge is computed by dividing the initial base flow by sub-basin area. The recession constant (k) is calculated by the following equation which is derived from equation 3.7. Base flow during a storm event is rested by ratio to peak method by trial and error during computing, simulation run, the initial values were determined by dividing the initial base flow by peak time flow average observed flow of August.

Simple canopy is a representation of plant canopy and it has three parameters: initial storage, maximum storage, and crop coefficient. In this study, the initial value for this parameter is determined by trial and error which yields the best fit between observed and computed runoff.

3.4.4 Model Performance Evaluations

Model datasets are divided into calibration and validation data sets. Optimum values of parameters are identified in the calibration dataset by minimizing the difference between observed and predicted discharges. Model performance could be tested for the validation dataset by visually inspecting and comparing the calculated and observed hydrograph for different flow regimes. The model performance will be evaluated for both calibration and validation datasets by using different statistical measures such as Nash and Sutcliffe simulation efficiency (NSE), Relative Volumetric Error (RVE) and percent error in peak flow (PEPF). NSE is used to evaluate the capability of the model to reproduce the pattern of observed hydrography. The RVE function is used to quantify the volumetric error of the simulated stream flow. The PEPF measure the agreement between the magnitude of observed and simulated peaks. The reason for selecting the above objective functions is that they are used and were successful in measuring the model performance.

Nash-Sutcliffe Efficiency (NSE): Measures the fraction of the variance of the observed flows, explained by the model in terms of the relative magnitude of the residual variance to the variance of the flows. The value of NSE can range from $-\infty$ to 1. The optimal value is 1.0, and values in between 0.6 and 1 are generally taken as acceptable performance. Values larger than 0.6 indicate 'minimally acceptable' performance. The model efficiency is estimated as follows:

$$NSE = 1 - \frac{\sum_{i=1}^n (Q_{obs,i} - Q_{sim,i})^2}{\sum_{i=1}^n (Q_{obs,i} - Q_{obs,mean})^2} \dots\dots\dots 3.8$$

Relative Volumetric Error (RVE): The value ranges in between $-\infty$ and ∞ . The model best performs if the RVE is close to zero and the model performance is very good for RVE between -5% to 5%, while RVE between -10% to -5% and 5% to 10% suggest satisfactory performance.

$$RVE = \frac{\sum_{i=1}^n (Q_{sim,i} - Q_{obs,i})}{\sum_{i=1}^n (Q_{obs,i})} * 100\% \dots\dots\dots 3.9$$

Percent error in volume (PEV). The PEV function only considers the computed volume and does not account for the magnitude or timing of the peak flow.

$$PEV = 100 * \left| \frac{V_o - V_s}{V_o} \right| \dots\dots\dots 3.10$$

Where: - V_o (V_s) is the volume of the observed (simulated) runoff

Percent Error in Peak Flow (PEPF). The PEPF measure only considers the magnitude of computed peak flow and does not account for total volume or timing of the peak.

$$PEPF = 100 * \left| \frac{Q_{obs(peak)} - Q_{sim(peak)}}{Q_{obs(peak)}} \right| \dots\dots\dots 3.11$$

Where $Q_{obs(peak)}$ is the peak observed discharge, $Q_{sim(peak)}$ is the peak simulated discharge.

3.5 HEC-RAS model

The U.S. Army Corps of Engineers’ River Analysis System (HEC-RAS) is software that allows you to perform one- dimensional steady and unsteady flow river hydraulic calculations.

HEC-RAS system will eventually contain three one-dimensional hydraulic analysis components. For this research; used steady flow water surface profile computations;

A basic element is that all three components will use a common geometric data representation and common geometric and hydraulic computation routines.

In addition to the three hydraulic analysis components, the system encompasses several hydraulic design features that can be appealed once the basic water surface profiles are computed (HEC RAS, 2016).

A) Input data for model HEC RAS

The data needed to perform these computations are divided into the following categories:

I. Geometric data

Survey data collected from the field at the bridge site by using a survey instrument and field observation Those are; ground surface profile, flow direction, reach length, the elevation of the areas, Manning’s n value, pier shape, abutment condition, banks both the right and left, and bridge alignment. It can be useful to analysis water surface profile in the Bridge location also which is an input parameter for HEC-RAS model simulation after analysis we can check the structure overtopped or not. Detail bridge cross-section and structural design of the bridge obtained from the Ethiopian Road Authority (ERA). Basic elements of geometric data are the following: -

A. River system schematic

- B.** Cross section Geometry
- C.** Reach length; and
- D.** Energy loss coefficients

Several types of loss coefficients are utilized by the program to evaluate energy loss;

- 1) Manning’s (n) values or equivalent roughness “K” values for friction loss,
- 2) Contraction and expansion coefficients to evaluate transition losses and
- 3) Bridge and culvert loss coefficients to evaluate losses related to weir shape, pier configuration, pressure, flow, and entrance, and exit conditions.

Manning's (n) selection of an appropriate value for Manning is very significant to the accuracy of the computed water surface profile (HEC RAS, 2016). The value of manning’s n is highly variable and depends on a number of factors including; surface roughness; vegetation; channel irregularities; channel alignment; scour and deposition; obstructions; size and shape of the channel; stage and discharge; seasonal changes; temperature; and suspended material and bed load.

$$Q = \frac{AR^{\frac{2}{3}}S^{\frac{1}{3}}}{n} \dots \dots \dots 3.12$$

Where: - Q= is the volumetric flow rate passing through the channel reach in (m³/s).

A= is the cross-sectional area of flow normal to the flow direction in (m²).

S= is the bottom slope of the channel in m/m (dimensionless).

n= is a dimensionless empirical constant called the Manning roughness coefficient.

R= is the hydraulic radius = A/P.

P = is the wetted perimeter of the cross-sectional area of flow in m.

3.5.1 Steady flow data

Steady flow data are required in order to perform a steady water surface profile calculation steady flow data consists of flow regime, boundary conditions and peak discharge information which is estimated by above-mentioned methods. This element of the model is intended to analyze water surface profiles for steady gradually varied flow (GVF). It is also capable of modeling subcritical, supercritical, and mixed flow regime water surface profiles (HEC RAS, 2016). The basic computational procedure is based on the solution of the one-dimensional energy equation.

I. Energy equation

According to (HEC RAS, 2016), water surface profiles are calculated from one cross-section to the next by solving the energy equation with an iterative technique called the standard step method. The energy equation is written as follows:

$$Z_2 + Y_2 + \alpha_2 \frac{V_2^2}{2g} = Z_1 + Y_1 + \alpha_1 \frac{V_1^2}{2g} + h_e \dots \dots \dots 3.13$$

Where: Z1, Z2= elevation of main channel inverts

Y1, Y2= depth of water at cross sections

V1, V2= average velocities

α_1, α_2 = velocity weighting coefficient

h_e = energy head loss

This can be further simplified using the figure given below; Equation (3.14)

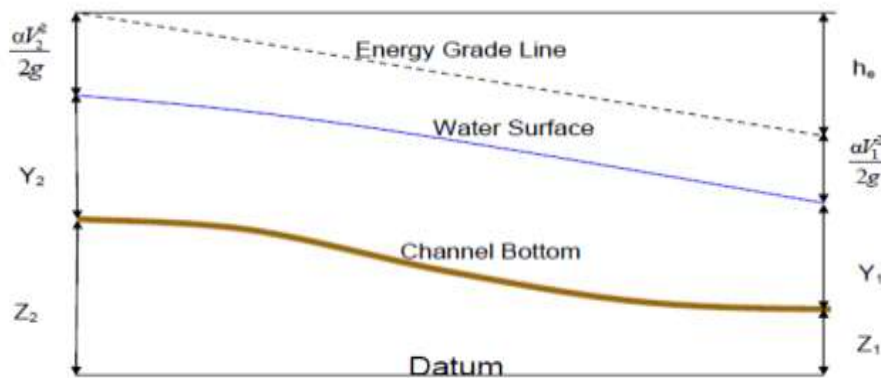


Figure 3.12: The terms in the energy equation. (Source: - HEC RAS Manual, 2016).

The energy head loss (h_e) can be estimated by adding friction head loss (h_f) and contraction/expansion head loss (h_c). Friction head loss is a product of energy grade line slope (S_f) and length of the channel (L) between each cross section.

The energy head loss can be expressed using Equation 3.14;

$$h_e = L S_f + C \left[\frac{\alpha_2 V_2^2}{2g} - \frac{\alpha_1 V_1^2}{2g} \right] \dots \dots \dots 3.14$$

Stream cross sections may not always take perpendicular to the flow direction.

Due to this left and right side of the channel will have a different length. For this reason, HEC-RAS considers weighted channel length during the computation process and expressed in Equation (3.15) below.

$$L = \frac{L_{lob} Q_{lob} + L_{ch} Q_{ch} + L_{rob} Q_{rob}}{Q_{lob} + Q_{ch} + Q_{rob}} \dots \dots \dots 3.15$$

Where: L= discharge-weighted reach length...Equation (3.15)

L_{lob}, L_{ch}, L_{rob} , are left overbank, main channel and right overbank length respectively.

Q_{lob}, Q_{ch}, Q_{rob} , are discharged at the left overbank, main channel and the right overbank section of the channel respectively. The energy grade line and the velocity head can also be determined from manning equation applying successive iterations:

$$Q = K S f^{\frac{1}{2}} \dots \dots \dots 3.16$$

$$K = \frac{1}{n} A R^{\frac{2}{3}} \dots \dots \dots 3.17$$

Where: - Q=discharge in the channel, K=conveyance constant, and S_f=friction slope

3.5.2 The performance assessment of the bridge structures.

The hydraulic performance of Bridge assessment can be classified as follows;

These are: - inadequately sized opening, scour, accumulation of debris and sediment.

A) Opening size

Bridge opening size has a major effect on the hydraulic performance of the Bridge structure. Whenever the opening size under the Bridge span gets smaller then it acts as a barrier to the flow and causes the inundation at the upstream side, then the Bridge considered to be inadequate, even if it does not survive in the high flood event (HEC 18, 1993); so it is important to make sure that a hydraulically effective structure is designed and the backwater level calculated accurately.

B) Scour

Generally, scour includes scouring from sources that often can't be adequately quantified through analytical studies. This class includes scour caused by contraction scour, live bed contraction scours, clear-water contraction scours, local scour, scour at the piers, scour at abutments, and lateral scour. For more serious conditions one or more multidimensional numerical model(s) or a physical model of the bridge may be needed to best evaluate scour (Dakota S, 2011). Some basic scour examination was discussed below according to this study.

I. Local scour at abutments

Local scour occurs at abutments when the abutment and embankment obstruct the flow. The obstruction of the flow forms a horizontal vortex starting at the upstream end of the abutment and running along the toe of the abutment and forms a vertical wake vortex at the downstream end of the abutment.

I - Froelich's (1989) Live-bed Abutment Scour Equation

(The ratio of the length of the abutment (normal to flow) to flow depth <= 25)

$$\frac{Y_s}{Y_a} = 2.27 K_1 K_2 (L')^{0.43} y^{0.57} Fr^{1.0.61} + 1 \dots \dots \dots 3.18$$

Where

K₁ = Coefficient for abutment shape

K₂ = Coefficient for angle of embankment to flow

$$K2 = (\theta / 90)^{0.13}$$

$\theta < 90$ if embankment points downstream

$\theta > 90$ if embankment points upstream

L' = Length of active flow obstructed by the embankment

A_e = Flow area of the approach cross section obstructed by the embankment

Fr = Froude Number of approach flow upstream of the abutment

$$= V_e / (g y a)^{1/2}$$

$$V_e = Q_e / A_e$$

Q_e = Flow obstructed by the abutment and approach embankment

Y_a = Average depth of flow on the floodplain (A_e / L)

L = Length of embankment projected normal to the flow

Y_s = Abutment Coefficients Scour Depth

Table 3.6: Abutment Coefficients

Description	K1
Vertical-wall abutment	1
Vertical-wall abutment with wing walls	0.82
Spill through abutments	0.55

$K2$ = Coefficient for angle of embankment to flow

$$K2 = (\theta / 90)^{0.13}$$

$\theta < 90$ if embankment points downstream

$\theta > 90$ if embankment points upstream

II - HIRE (Richardson 1990) Live-bed Abutment Scour Equation

(Ratio of the length of the abutment (normal to flow) to flow depth > 25)

$$\frac{Y_s}{Y_a} = 4 \left[\frac{K1}{0.055} \right] K2 Fr^{10.33} \dots \dots \dots 3.19$$

Where

$K1$ = Coefficient for abutment shape

$K2$ = Coefficient for the angle of an embankment to flow as calculated for Froelich's equation

Fr = Froude Number based upon the velocity and depth adjacent to and upstream of the abutment, $Y1$ = Depth of flow at the abutment on the overbank or in the main channel, and

Y_s = Scour Depth

4. RESULT AND DISCUSSION

4.1 Sensitivity analysis of HEC-HMS input parameters

Sensitivity analysis has been carried out for 10 parameters (CR, IL, DPA, RP, RC, TL, IS, MS, CC, and % Imp). It was done by changing the one parameter value at a time and keeping the other parameters constant and estimating the value of objective functions for selected percentage value ranges of parameters. The objective functions used to evaluate sensitive parameters are Nash and Sutcliffe efficiency (NSE), Relative volumetric error (RVE) and Percent Error in Peak (PEP). For each initial parameter, five different parameter percentage change values used to assess the sensitivity.

Parameters identified sensitive as evaluated by NSE are CR, IL, DPA, RP, RC, TL, IS, MS, CC, and % Imp. From the aforementioned parameters % Imp, TL, MS, and CC are the most sensitive parameters which affect the pattern of simulated hydrography. The other parameters CR, IL, DPA, RP, IS, and RC is only slightly sensitive parameters. Model response to the volume of simulated flow is highly sensitive for TL, IS, MS, CC, and % Imp. RC, but for IL, DPA, RC, and RP low sensitive parameter. In terms of PEP, the model response is most sensitive RC, TL, CC and % imp. As indicated in graph 4.1 below the change in slope for both parameters is very high. The next significant parameters which affect model simulated peak flow are CC and % Imp. On the course of carrying out sensitivity analysis five parameters CR, and IL are identified as non-sensitive parameters. % Imp and CC are the two most sensitive parameters which affect the three objective functions (NSE, RVE, and PEP).

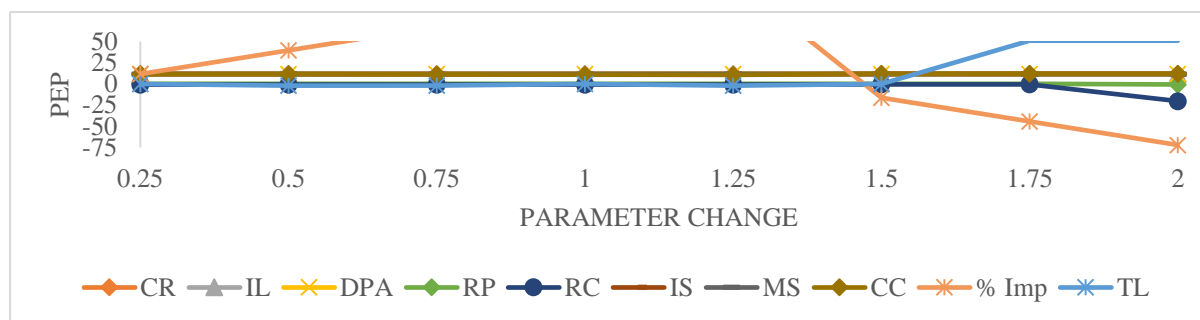


Figure 4.1: model sensitivity analysis evaluated by PEP

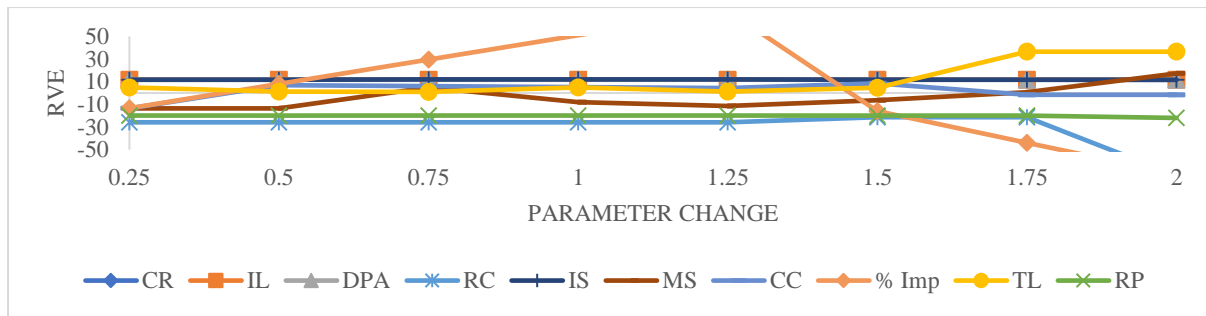


Figure 4.2: model sensitivity analysis evaluated by RVE

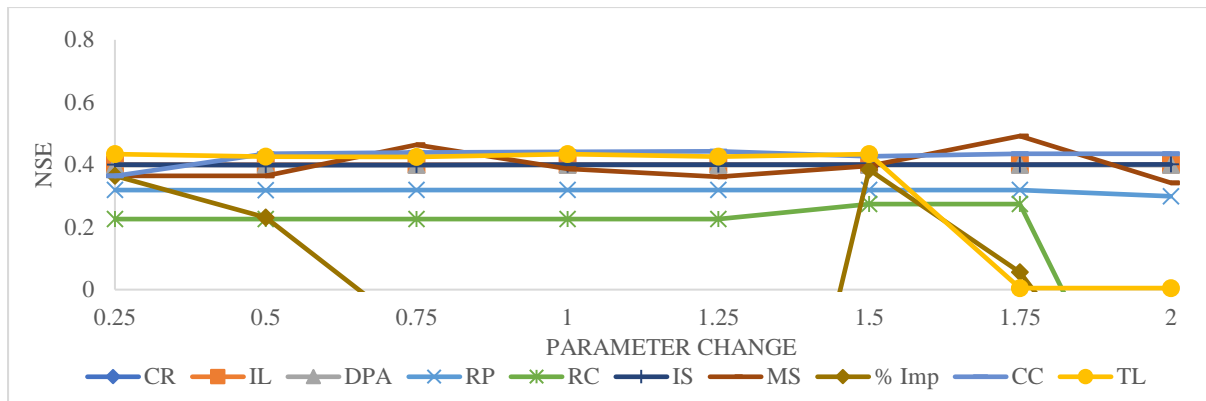


Figure 4.3: model sensitivity analysis evaluated by NSE

4.2 HEC HMS calibration and validation Result

Historical observed streamflow data of Bilate watershed at near Alaba Kulito were HEC HMS model has been applied for five events. Calibrated for three events and validated for two events. Calibration was done by trial and error procedure using different parameter values. In the process of calibration qualitative (visualization) and quantitative (objective functions) analysis was used. Using both qualitative and quantitative approach observed and simulated streamflow hydrographs were compared and adjustments of calibrated parameters were done to improve the match between the hydrographs. Prior to calibration two years of streamflow data were used to initialize the model.

Table 4.1: Calibrated and Validated parameters for Bilate watershed.

Period	Calibration									
	CR	IL	DPA	RP	RC	TL	IS	MS	CC	%Imp
01Jun1999-01Dec1999	5	2	0.001	0.65	0.94	1230	5	232	0.21	10.98
01Jan2000-31Dec2000	5	2	0.001	0.6	0.87	1000	5	220	0.23	11.7
01Jan2001-31Dec2001	5	2	0.001	0.63	0.87	1000	5	200	0.21	12
Average	5	2	0.001	0.63	0.89	1076	5	217	0.21	11.56
Validation										
01Mar2005-31Dec2005	5	2	0.001	0.63	0.97	1000	5	175	0.07	13
01Jan2007-31Dec2007	5	2	0.001	0.61	0.97	1350	5	165	0.02	12.78
Average	5	2	0.001	0.61	0.97	1075	5	170	0.045	13

It is important to evaluate the generally accepted level of model performance using objective functions like NSE, RVE, and PEP.

Table 4.2: Objective function

Period	Objective Function		
	NSE	RVE	PEP
01Jun1999-01Dec1999	0.556	0.21	0
01Jan2000-31Dec2000	0.708	-5.76	0
01Jan2001-31Dec2001	0.710	-6.13	0
01Mar2005-31Dec2005	0.519	-3.39	0
01Jan2007-31Dec2007	0.685	-5.03	0

Over the calibration, period shows very good performance in simulation of trends NSE, volumetric fit RVE, and peak flow PEP. The NSE value of observed and modeled daily streamflow of calibration and validation is >0.5 respectively. According to model evaluation criteria, the model simulated observed flow trend in a very good manner, both in calibration and validation period. The relative volumetric error (RVE) value ranges from -6.13 to 0.21 during calibration and verification period. These values indicate that the model simulated the stream flow volumetric fits like a very good range of performance during calibration and validation period. The percent error in peak flow (PEP) for validation and calibration period ranges from -0.2 to 0. This value shows as the model captures peak flow in a fairly good manner.

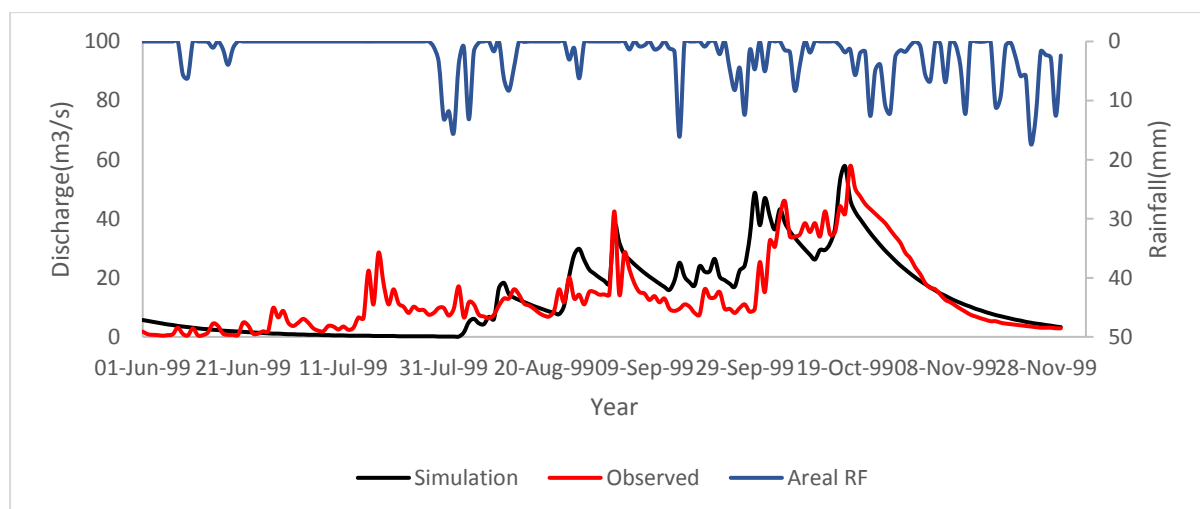


Figure 4.4: Calibration event from 01Jun1999-01Dec1999

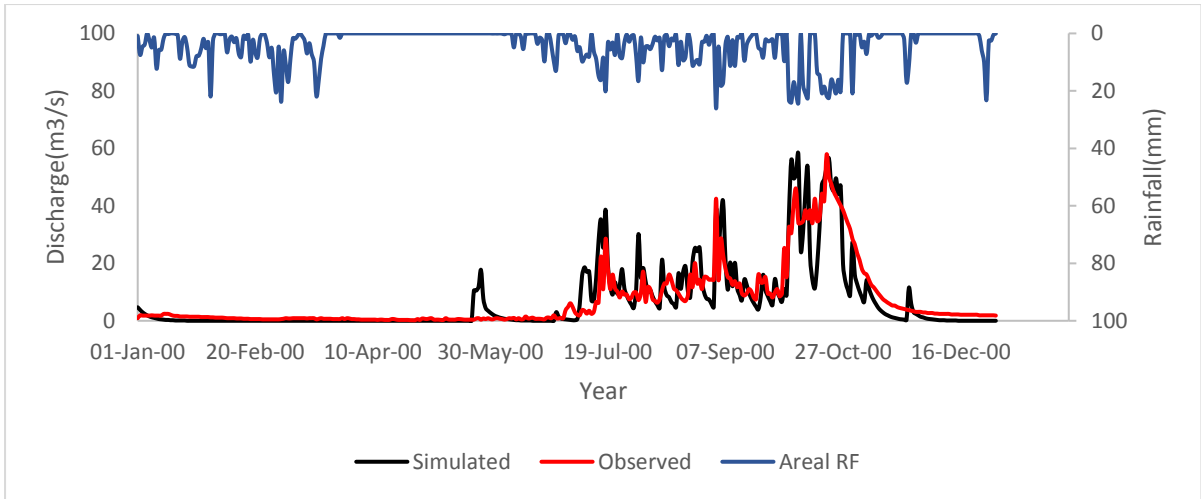


Figure 4.5: Calibration event from 01 Jan 2000-31 Dec 2000

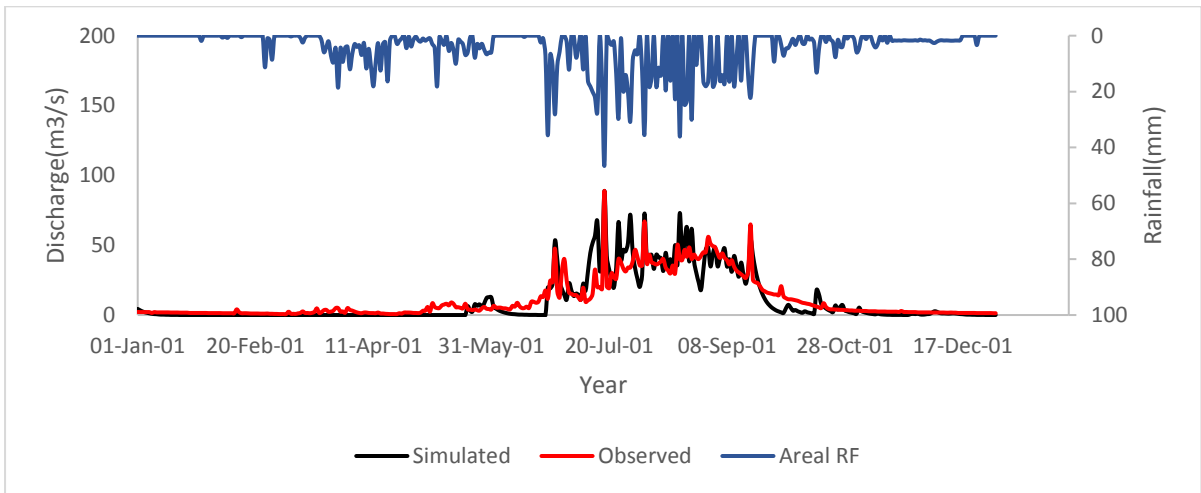


Figure 4.6: Calibration event from 01 Jan 2001-31 Dec 2001

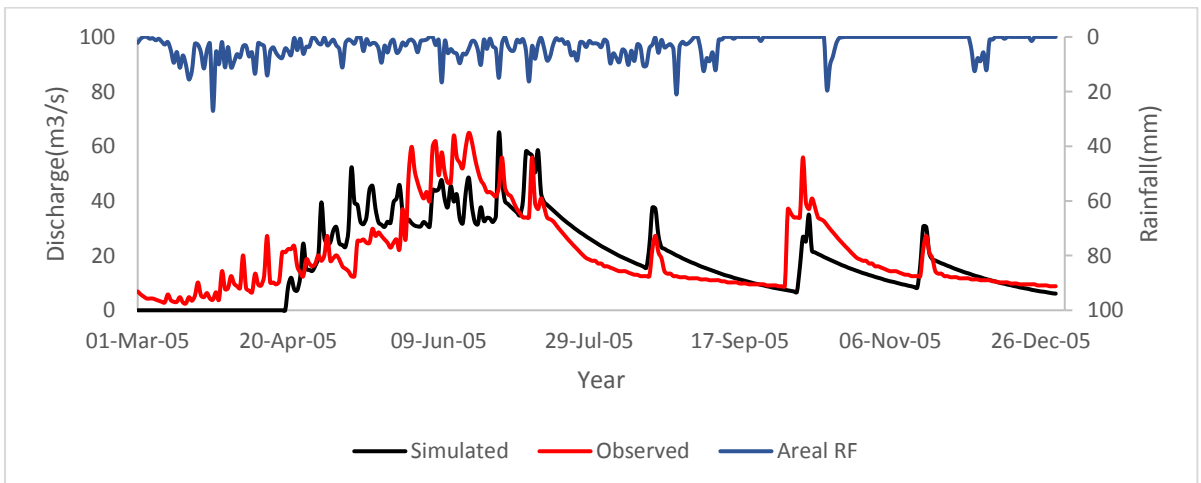


Figure 4.7: Validation event from 01 Mar 2005-31 Dec 2005

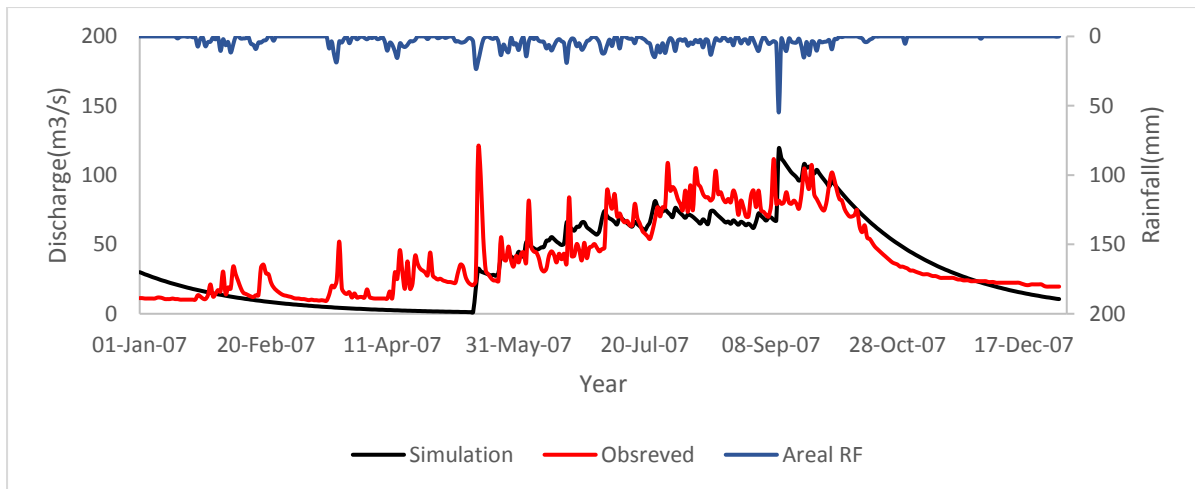


Figure 4.8: Validation event from 01 Jan 2007-31 Dec 2007

4.3 Design discharge estimation at Bilate River Bridge

After calibrating and validating the model, the parameters are selected and used to estimate the design discharge. According to the (ERA, 2013) rainfall region classification the region in which Bilate river watersheds located in Region B2. For each region, the respective SCS 24hr rainfall is allocated, which is shown below in table 4.3.

Table 4.3 24 hr Rainfall Depth (mm) vs Frequency (yr)

24 Hr Rainfall Depth (mm) vs Frequency (yr)								
Return Period year	2	5	10	25	50	100	200	500
RR-A1	50.3	66.02	76.28	89.13	98.63	108.06	117.48	130
RR-A2	51.92	65.52	74.45	85.7	94.07	102.45	110.91	122.27
RR-A3	47.54	59.61	67.66	77.92	85.62	93.34	101.13	111.58
RR-A4	50.39	63.83	72.28	82.55	89.97	97.2	104.32	113.63
RR-B1	58.87	71.26	79.29	89.35	96.84	104.37	112.02	122.41
RR-B2	55.26	69.95	79.68	92.03	101.29	110.61	120.07	132.87
RR-C	56.52	71.04	80.54	92.52	101.48	110.5	119.66	132.06
RR-D	56.23	76.84	90.37	107.46	120.23	133.05	146	163.44

Source: - (ERA, 2013)

Using these validated parameters as well as the SCS 24-hour storm, which is adapted from the ERA Drainage Design Manual, as inputs, the event-based modeling with HEC-HMS has resulted in the following discharge which shown below in the table.

Table 4.4: Design discharge estimation

Return period(year)	50	100
Discharge(m ³ /s)	284.2	327.3

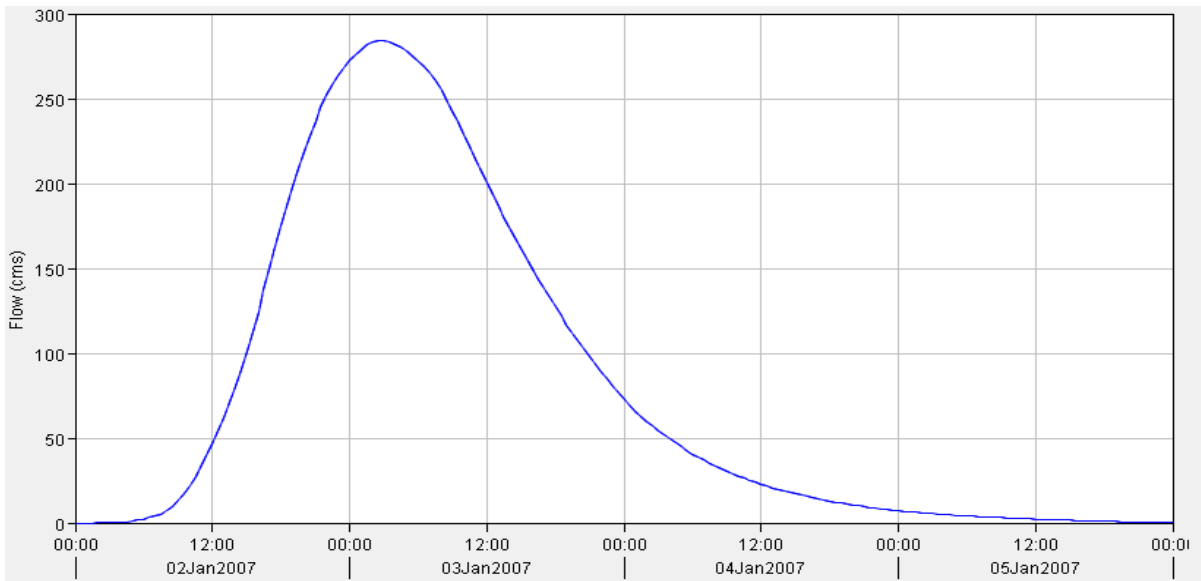


Figure 4.9: The hydrograph for Bilate River Bridge for a return period of 50yr.

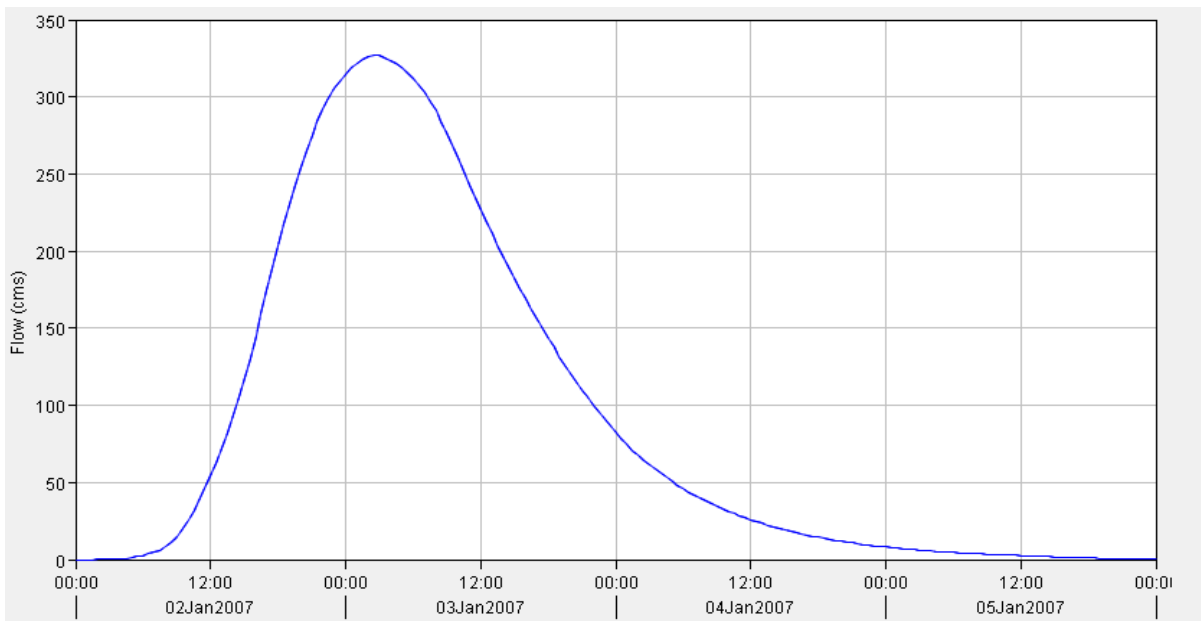


Figure 4.10: The hydrograph for Bilate River Bridge for a return period of 100yr.

The peak discharge on the River Bilate at the bridge outlet is estimated for the different return period in the above table. It is difficult to find the hydrologic analysis for the Bilate river bridge because the bridge was built along a year ago this shows that there is no hydrologic analysis performed to estimate the design discharge from ERA and consults office. As a result, it is not possible to compare the design discharge estimated to the original design on which the bridge opening capacity was determined.

4.4 Evaluating bridge stability

4.4.1 Calculating the bridge opening capacity

The figure shows that the bridge cannot accommodate the design discharge of $284.2\text{m}^3/\text{s}$, and $327.3\text{m}^3/\text{s}$ respectively. The Hydraulic model results (HEC RAS) show that for 50yr, and 100yr the bridge is not enough to pass the discharge value. Figure 4.11 reveals this fact. The result from the model shows that the water surface profile will acquire a height of 1726.93m, and 1727.03m respectively, which means that 0.93m, and 1.03m clear height of the bridge for 50yr and 100yr, also main channel bed change throughout the year, though the discharge still cannot be served within this opening.

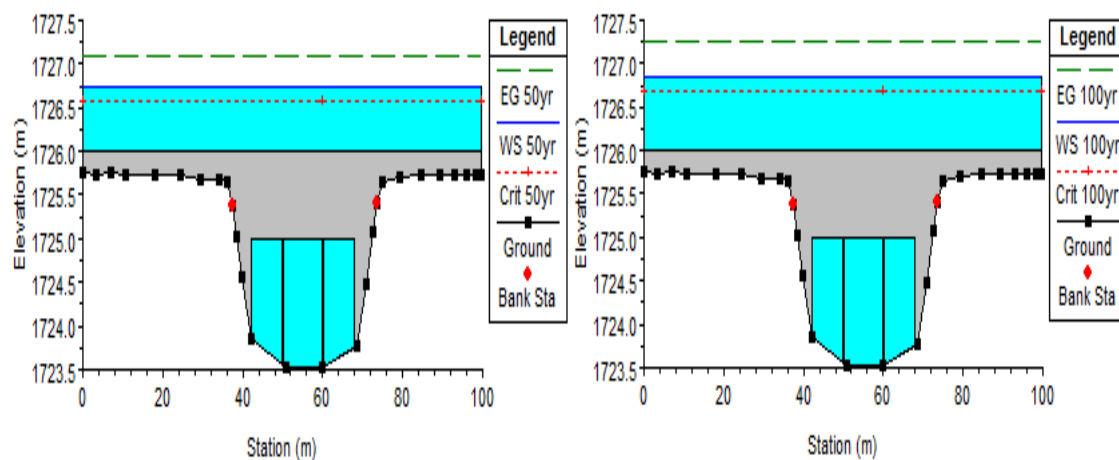


Figure 4.11: Modelled Water Surface profile for 50yr and 100yr return period of the Bilate River Bridge respectively.

4.4.2 The scouring effect on the bridge

The hydraulic structure provided, the bridge, has induced some sort of instability. The opening of the bridge was both understated and constricted. Therefore, the flood is forced to increase velocity when it passes through the bridge. High velocity has an erosive power and the bed material is eroded below the original bed level. The overall scour in the bridge was observed in this research work by using all necessary steps were seen to abutment scour was high. Abutment scour was the main problem of the Bilate River, but pier scour depth does not observe due to near the pier the bed exacting was a rock bed which shown in figure 4.12.

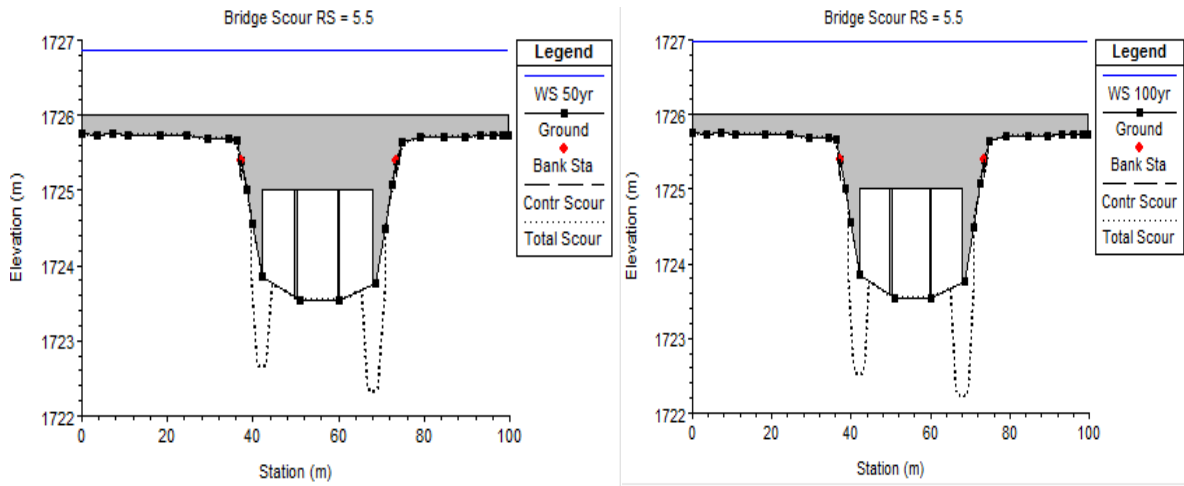


Figure 4.12: Abutment scouring of Bilate River Bridge

Table 4.5: Scour results at the Bilate River Bridge

Return period	Abutment Scour	
	Left(m)	Right(m)
50yr	1.23	1.42
100yr	1.37	1.53

The bed elevation change along the Bilate River was changing its condition which shown in the table below and the bridge was degraded from time to time. The depth each year, along with the river bed level, which shown below in figure 4.13.

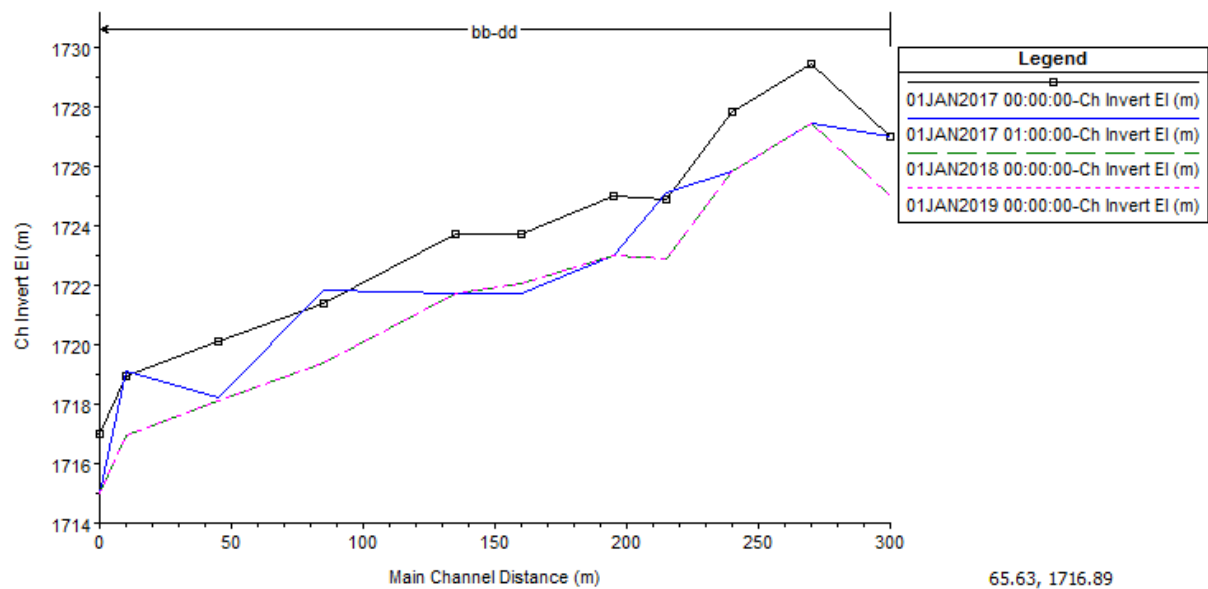


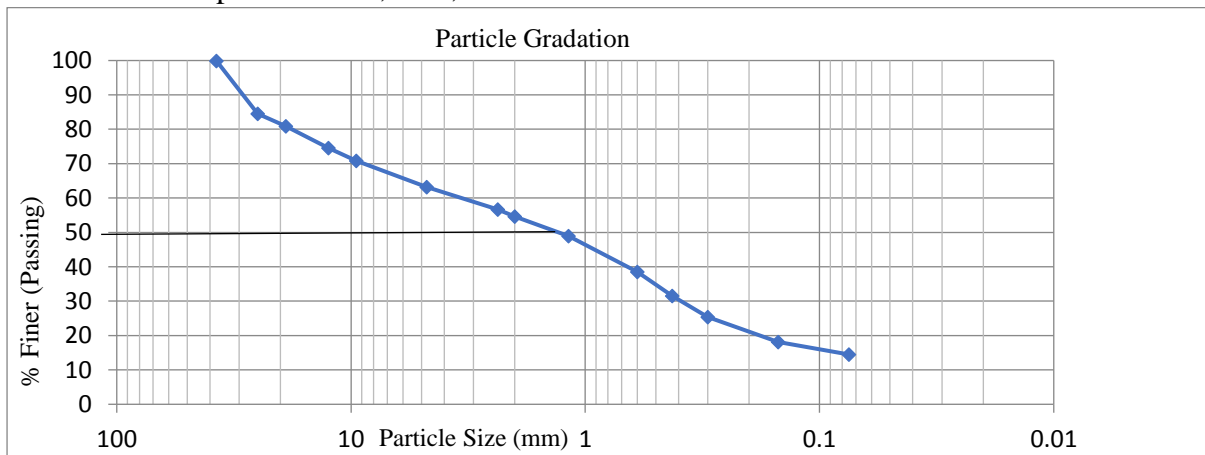
Figure 4.13: main channel bed change along the Bilate River Bridge

Table 4.6: Sieve Analysis Result

Diam (mm)	Mass of Empty Sieve (gm)	Mass of +Soil (gm)	Mass of Sieve Retained (gm)	Soil Retained (gm)	% Retained	Cumm. % Return.	% Finer (Passing)
37.5	440.5	442.5	2	2	0.13	0.13	99.87
25	453.5	688.5	235	235	15.34	15.47	84.53
19	444	500.5	56.5	56.5	3.69	19.16	80.84
12.5	447	543.5	96.5	96.5	6.30	25.46	74.54
9.5	584.5	641.5	57	57	3.72	29.18	70.82
4.75	431.5	548.5	117	117	7.64	36.81	63.19
2.36	387.5	487	99.5	99.5	6.49	43.31	56.69
2	556	587.5	31.5	31.5	2.06	45.37	54.63
1.18	354.5	441	86.5	86.5	5.65	51.01	48.99
0.6	323.5	484.5	161	161	10.51	61.52	38.48
0.425	294	400	106	106	6.92	68.44	31.56
0.3	302	396.5	94.5	94.5	6.17	74.61	25.39
0.15	282.5	394.5	112	112	7.31	81.92	18.08
0.075	256	311	55	55	3.59	85.51	14.49
Pan	254	476	222	222	14.49	100.00	0.00
Total Weight				1532			

4.4.3 Sieve analysis

Sieve analysis result: By taking the sample mass of 1533gm the sediment under the bridge is graded. The test result is shown in the table below, Table 4.6. The error in the test is 0.5 gm. The value of 50 percent finer, D50, is 1.5mm



$d_{50} = 1.5\text{mm}$

$d_{95} = 34.5\text{mm}$

Figure 4.14: sieve analysis

4.5 Causes of abutment scour

Bilate River Bridge was one of the important structures, but encounter abutment scouring effect around the right and left sides of the embankment. The major cause of failure discussed below those are:-

- I. **Constriction scours:** The abutment is susceptible by a geological failure of the main channel bank. The scour may be caused by a combination of local as well as the constriction scour process and by main channel shifting.



Figure 4.15: Constriction scour effect on the bridge

- II. **Man-made activity:** People around the right river bank extract basalt rock as construction material and used as income for living around the Bilate river bridge. The most area of the river banks was excavated which cover large parts of failure also river morphology at the upper parts are changed through time to time and changes its direction of movement along the flow direction by creating pond then forms meandering like movement.



Figure 4.16: Right River banks

III. Sideways channel movement: Lateral channel migration was generate small stream and created during peak rainfall event at a time and cause erosion of embankment near abutment. The sideways also highly depend on the slope of the side slope of the embankment was too much causes failure of the abutment.



Figure 4.17: Lateral channel migration

IV. Length of the abutment skewed to the flow: Abutment skew in the Flow direction was the significant effect on bridge failure based on a skew angle in flow direction so Bilate River Bridge skew 45 degrees along the flow direction.



Figure 4.18: Flow direction view from downstream

V. Land use land cover change: Land use land cover change has a direct impact on the number of local abutments scour on river banks. For the Bilate River catchment area there is any clear information about the land use changes. There is deforestation of the catchment forest coverage for the purpose of agricultural land which increases the amount of

impervious area on the river and also from the data's taken from the ministry of agriculture the land use coverage more than 50% of the catchment is cultivated land which adversely affects the amount of erosion from the bare land also increase peak discharge.

4.6 Mitigation measures

Sustainable Mitigation measures were essential to extend the life span of the bridge and reduce the negative impact of the failure in the economic, social and political area. The following mitigation measures were mentioned for the existing and future bridge construction of the bridges are horizontal to failure due to enhanced scour launch, protect bridge structure caused by both natural and human activities.

- I. River Training work:** River training works are highly mentioned to control and guide the flow within well-defined banks. The bank material of the river is highly erodible, erosion protection measures like planting a tree on the banks with natural vegetation and ripraps are necessary. These erosion defense measures are highly mandatory, particularly for those reaches of the river affected by human activity measures.
- II. Relocating & increasing the opening size of the bridge:** The selected bridge location should be on a straight reach with stable natural banks and a sufficiently prominent channel river. Factors to consider in the determination of the Span length and the clear height of the Bridge. Span length: - With regard to the opening size of the bridge, sufficient span length should be selected to pass the design discharge also possible clear opening height of the bridge. The new proposed bridge will be 32m wide across the river in order to throw away the problem.
- III. Concern land use changes:** Valuing future land use, land cover changes in the catchment area of the river providing erosion protection mechanism in the catchment area of the Bilate River to reduce the amount of the local souring abutment on the bridge crossing location.

5. CONCLUSION AND RECOMMENDATION

5.1 Conclusion

The failure of the Bilate River Bridge was analyzed in two basic groups these are; hydrological analysis and hydraulic analysis. The hydraulic performance in the bridge reveals that abutment scouring was one of the series problems for the coming 50yr and 100yr peak flood in the vicinity of the structure.

The hydrologic evaluation tells bridge doesn't accommodate 284.2m³/s and 327.3m³/s design flood for the 50-year and 100-year return period. In the design flood estimation rainfall-runoff modeled by Initial and constant loss method and direct runoff by the SCS unit hydrograph method as well as Base flow by the recession. The problem of abutment scouring by HEC RAS has been prepared by taking inputs for analysis like; river cross section, design flood, and riverbank materials.

The abutment scours failure occurs in the return period of time at 50yr, and 100yr. The water surface profiles become increasing up 1726.93m, and 1727.03m respectively, which is above road deck elevation also model implies that the backwater effect in the bridge.

The failure of the bridge was an inadequate opening size to pass design flood so that, the abutment has been scoured because of constriction at the bridge. Mitigation measure was studied based on the hydraulic analysis result evaluated. Based on hydraulic analysis results; the recommendation river training work and new bridge proposed with increasing width to eliminate inadequate opening size.

5.2 Recommendation

The following recommendations were drawn from this thesis work:-

- Upstream of the bridge was changed in land use, land cover must require response organization to control this scenario and training necessary for society by the governmental and non-governmental organization to minimize the effect. Unless we lose both economic, social and political importance of the bridge.
- To estimate an accurate peak design flood in the study area, there is poor data recorded for both rainfalls this must require follow-up for the responsible organization.
- Study area open for further research which is not included in this thesis work.
- All designers must consider future scenario those are urbanization and reservoir effect on the Bilate river bridge.
- Further study of hydrogeological properties of the groundwater effect on the bridge.
- The geotechnical investigation needs around the right and left sides of the embankment to evaluate foundation.

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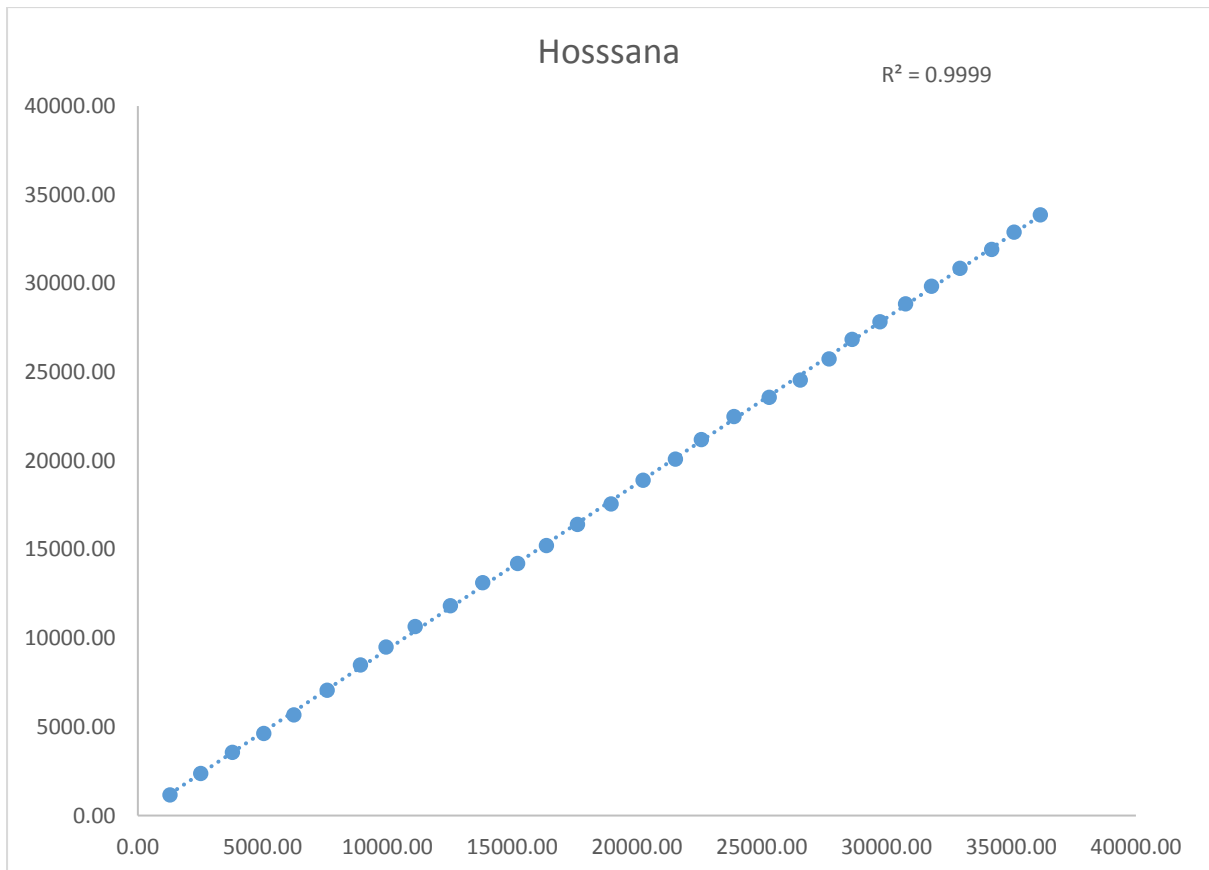
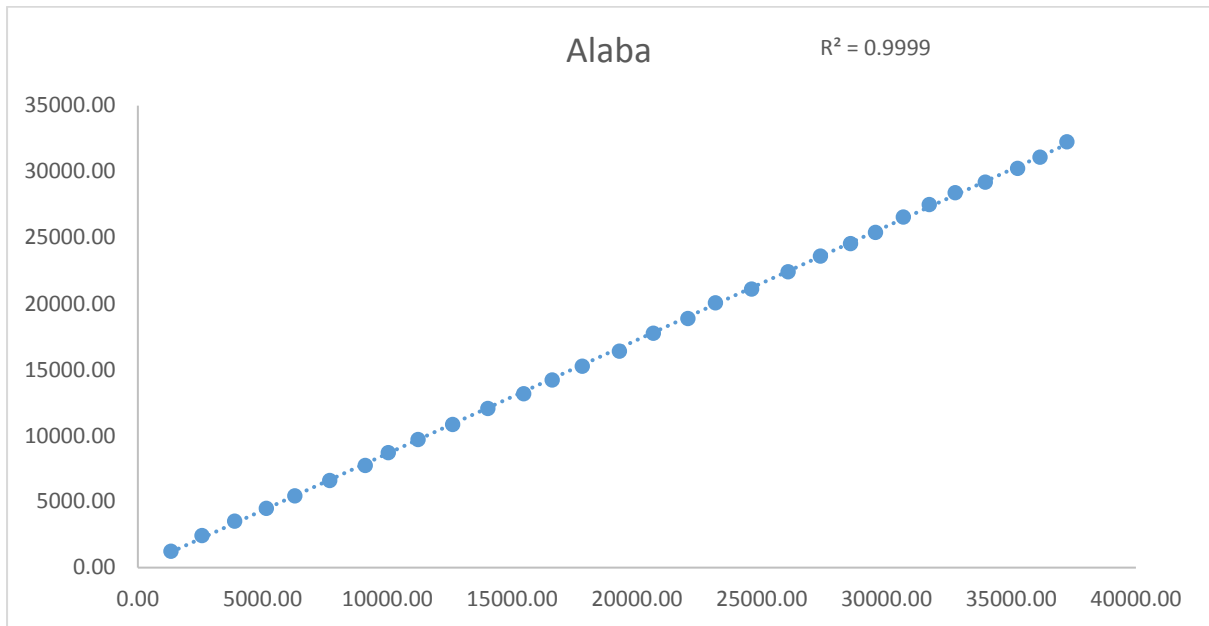
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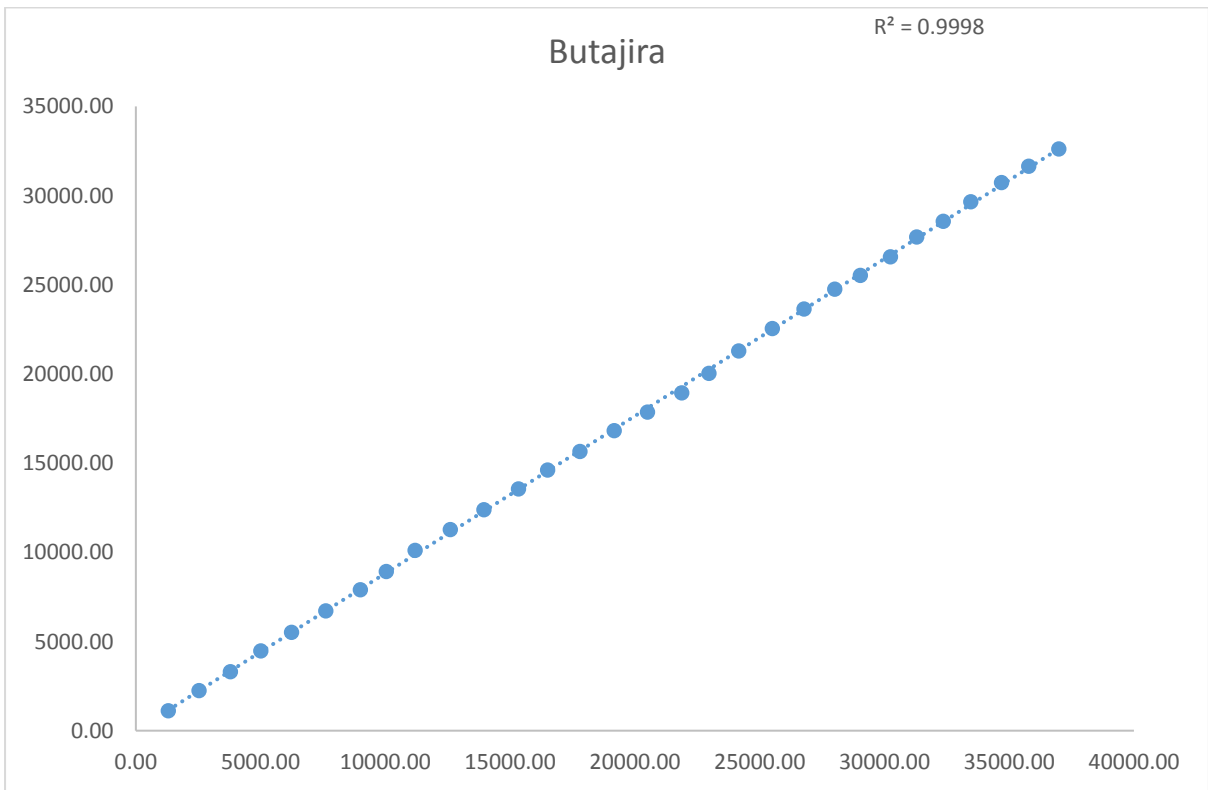
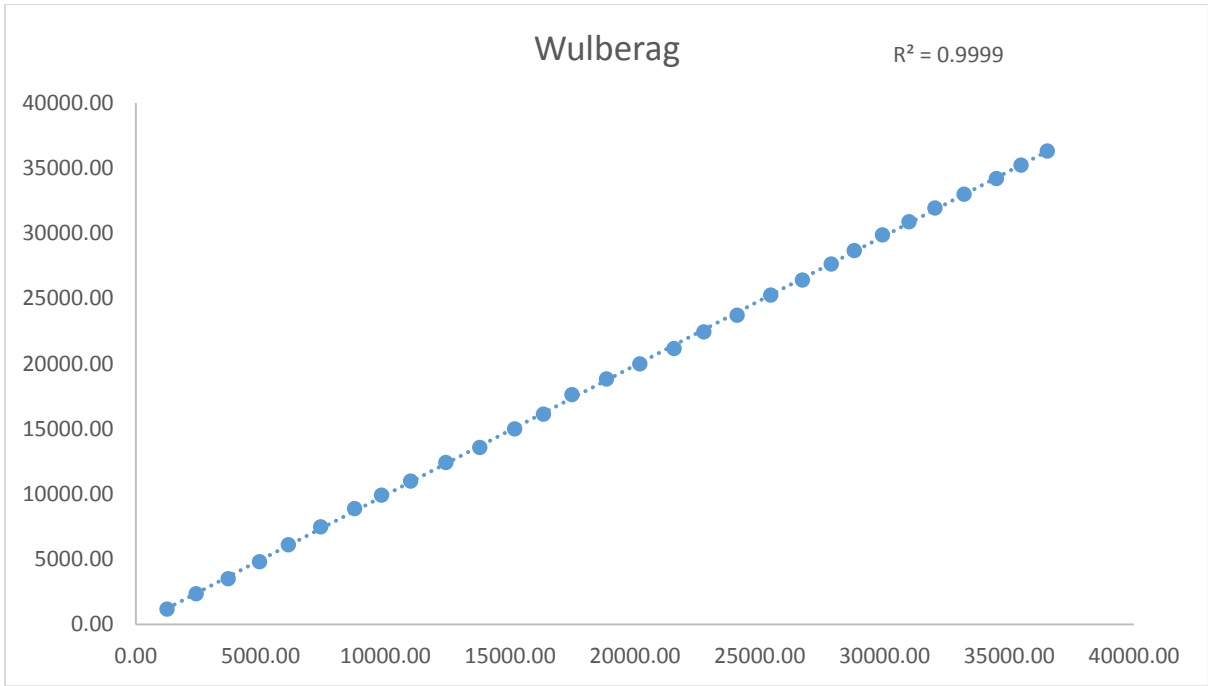
Appendix

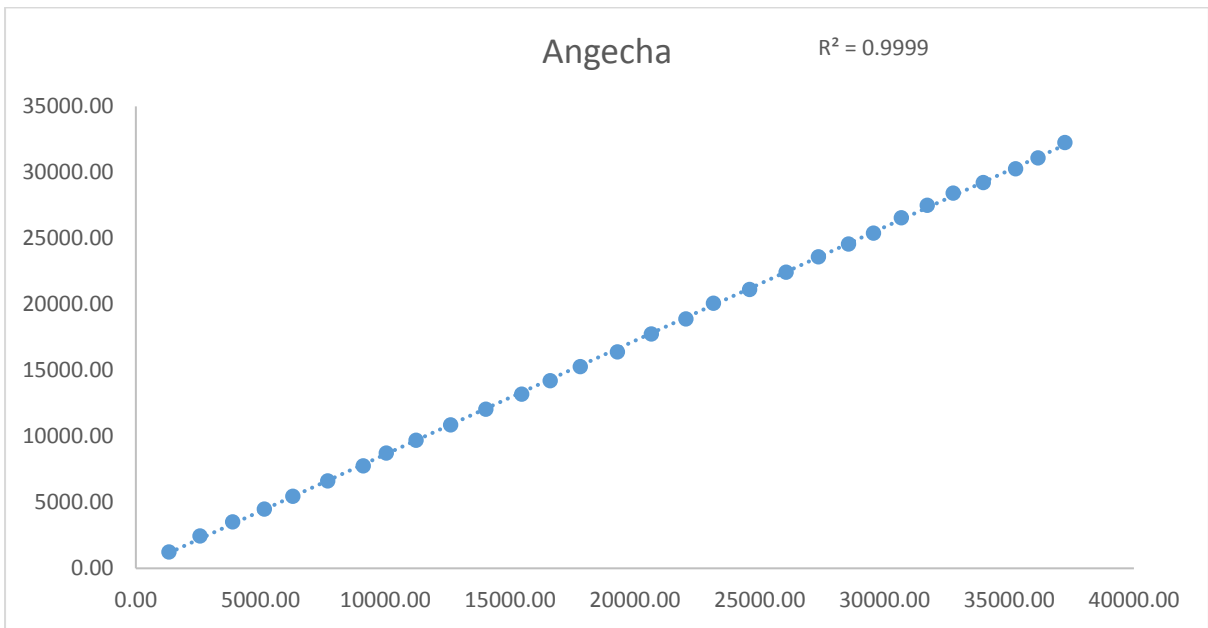
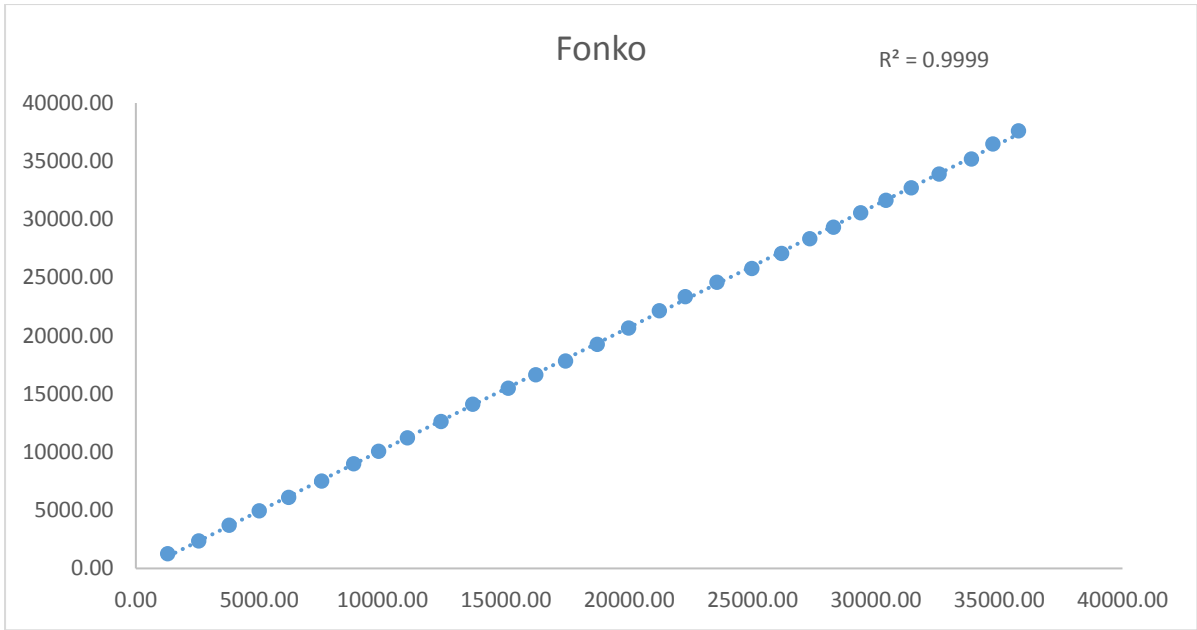
I. Annual rainfall data in mm

year	Alaba	Angecha	Hossana	Wulbarge	Butajira	fonko
1987	1240.24	1240.565	1152.45	1174.072	1123.11	1262.97
1988	1198.4	1267.1	1216.2	1171.92	1124.5	1111.6
1989	1083.5	1388.88	1199.7	1171.649	1049.2	1336.3
1990	968	1345.59	1058.9	1282.897	1172.4	1243.68
1991	965.0655	1450.964	1053.418	1304.83	1044.771	1137.95
1992	1162.6	1351.9	1387.6	1392.7	1199.904	1404.81
1993	1144.98	1416.026	1413.5	1378.426	1184.355	1511
1994	967.32	1234.743	1016.317	1041.881	1031.2	1069.08
1995	975.6	1439.802	1160.7	1084.105	1168.8	1135.32
1996	1160	1605.734	1168.2	1427.9	1178.96	1406
1997	1193.5	1408.5	1298.25	1152.3	1116.27	1498.41
1998	1129.492	1614.12	1089.48	1417.635	1167.92	1377.7
1999	1030.59	1380.922	1011.3	1136.2	1054.4	1153.5
2000	1051.2	1431.119	1190.28	1496.7	1038.72	1180.2
2001	1133.04	1685.32	1145.5	1200	1167.308	1421
2002	1353.2	1334.8	1346.4	1163.6	1043.91	1406.1
2003	1131.96	1538	1189.291	1176.5	1069.02	1493.1
2004	1191.12	1476.039	1092.111	1279.2	1096.08	1196.4
2005	1039.8	1383.55	1296.9	1272.24	1249.423	1237.9
2006	1310.7	1459.6	1081.53	1530.8	1257.84	1203.7
2007	1179.021	1487.338	988.92	1155.9	1106.712	1270.7
2008	950.3	1420.791	1179.718	1249.6	1106.321	1268.5
2009	847.5456	1229.849	1104.142	1010.315	781.1888	1009
2010	1153.685	1224.3	998.1	1212.2	1025.066	1236.6
2011	956	1292.594	1011.709	1016.63	1115.54	1064.4
2012	902.76	1141.408	981.6	1054.331	890.3042	1082.8
2013	807.333	1313.01	1022.08	1056.038	1084.112	1169.8
2014	1039.59	1311.401	1054.034	1196.511	1073.185	1295.484
2015	836.8442	1159.455	983.2626	1020.734	927.947	1298.947
2016	1167.265	1473.718	963.135	1097.924	967.7117	1130.5

II. DOUBLE MASS CURVE







III. River cross-sectional data

Upstream River cross section

11		10		9		8		7		6	
x	y	x	y	x	y	x	y	x	y	x	y
0	1738.138	0	1736.968	0	1736.335	0	1741.986	0	1734.669	0	1737.319
2.92	1735.38	10	1736.278	10	1734.112	10	1739.641	7.58	1728.56	2.34	1736.21
10	1728.227	12.21	1735.45	20	1734.009	20	1737.376	10	1727.059	5.97	1734.57
20	1727.38	20	1732.487	30	1733.992	30	1736.198	20	1725.258	11.56	1732.34
30	1727.639	30	1730.113	40	1733.979	34.7	1733.4	29.98	1725.15	18.44	1731.07
40	1727	40	1729.746	50	1728.567	40	1730.389	40	1725.063	22.73	1730.93
50	1736	50	1729.464	60	1727.807	50	1728.213	49.89	1725.02	25.58	1729.88
60	1736.639	60	1729.54	70	1728.119	60	1727.91	60	1727.947	29.74	1728.62
		70	1735.865	80	1728.161	70	1727.864	70	1728.203	40	1725.86
		80	1737.637	90	1728.768	80	1724.908	80	1731.407	50	1724.778
				100	1732.61	90	1730.784	90.04	1734.98	60	1725.114
				110	1732.793	100	1731.65			67.73	1725.45
				120	1734.575	110	1732.845			74.13	1727.23
				130	1736.249	120	1734			82.34	1729.06
				140	1737.698	123.79	1736.72			87.27	1730.47
				150	1739	130	1740.81			91.95	1731.67
				160	1741.717					94.94	1732.71
										99.22	1734.13
										102.99	1735.54
										108.57	1736.59

IV. Downstream River cross-section

5	4	3	2	1
x	y	x	y	x
0	1731.602	0	1727.981	0
7.34	1730.49	10	1726.104	10
11.24	1729.95	20	1724.383	20
15.94	1729.25	30	1721.371	30
21.04	1729.09	40	1722.357	40
27.06	1729.02	50	1724.343	50
31.71	1727.59	60	1727.262	60
35.39	1726.1			70
42.08	1723.86			80
50.91	1723.53			90
60	1723.528			100
68.57	1723.76			
76.88	1726.47			
81.82	1728.17			
85.17	1729.13			
89.74	1729.61			
91.67	1730.03			
97.79	1730.83			
100	1730.83			

V. Soil data

Diam (mm)	Mass of Empty Sieve (gm)	Mass of Sieve +Soil Retained (gm)	Soil Retained (gm)	% Retained	Cumm. % Retan.	% Finer (Passing)
37.5	440.5	442.5	2	0.13	0.13	99.87
25	453.5	688.5	235	15.34	15.47	84.53
19	444	500.5	56.5	3.69	19.16	80.84
12.5	447	543.5	96.5	6.30	25.46	74.54
9.5	584.5	641.5	57	3.72	29.18	70.82
4.75	431.5	548.5	117	7.64	36.81	63.19
2.36	387.5	487	99.5	6.49	43.31	56.69
2	556	587.5	31.5	2.06	45.37	54.63
1.18	354.5	441	86.5	5.65	51.01	48.99
0.6	323.5	484.5	161	10.51	61.52	38.48
0.425	294	400	106	6.92	68.44	31.56
0.3	302	396.5	94.5	6.17	74.61	25.39
0.15	282.5	394.5	112	7.31	81.92	18.08
0.075	256	311	55	3.59	85.51	14.49
Pan	254	476	222	14.49	100.00	0.00
Total Weight			1532			

VI. Daily Rainfall data

