ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITIUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIROMENTAL ENGINEERING

# EVALUATION OF RING ROAD DRAINAGE STRUCTURE

# (CASE STUDY MEGENAGNA TO BOLE ROAD)

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## A Thesis in Hydraulics Engineering

By Yisake Desalegn

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Addis Ababa

## A Thesis Submitted in partial Fulfillment of the requirements for the Degree of Master of Science

The undersigned have examined the thesis entitled '**Assesment of Ring Road Drainage Structure (Case Study Megenagna to Bole Road)'**, presented by **YISAKE DESALEGN**, a candidate for the degree of Master of Science and hereby certify that it is worthy of acceptance

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| Chair person | Signature | Date |

# UNDERTAKING

I certify that, this research work entitled **'Assessment of Ring Road Drainage Structure (Case Study of Megnagna to Bole Road)'** is my own work performed under close supervision of Dr.Daneal F. The work has never been presented elsewhere for assessment. However I have been used different reference materials, which has been properly captioned/acknowledged.

Signature of student

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# Abstract

The Purpose of the study is to assess the Performance of Drainage Structures of the Ring Road from Megenagna to Bole Round About and gives a mitigation measures regarding the drainages structure along the Road. An exploratory method of research has been used in the study. The study uses both primary and secondary instruments of data collection. The primary data collection instruments are observation, measuring the geometry (width and height) of existing cross drainage structures on the site, photographs that show the existing drainage structure conditions. The secondary data sources are Rainfall data, soil type, land use, geological map, and DEM data and QGIS 3.16 (open-source software) uses for watershed delineation of the catchment and extraction of other important geometric characteristics (i.e length of longest flow path, slope, elevation of the river and area of the catchment). Hydrological analyses have been carried out by using Soil Conservation Service (SCS) curve number Method and Rational method, the hydraulic analysis computed using the HEC RAS (open-source software) that to determine water surface elevations at all locations of interest for either a given set of flow data (steady flow simulation). The next step is to check the adequacy of the existing drainage system that suite the site conditions for the computed discharge, thus based on the hydraulics result, the major river exist around imperial not functioned well for the 100-return period, so shall be replaced with the new Bridge as span of 15m and 3.5m, which will accommodate the incoming 100-year discharge. The minor river exists between Ayat hospital and Imperial Round about, it is not having a significant flow and Can be retain as it is, only required maintenance and clearing of the channel. Regarding to the storm drainage all the inlets are not functioned properly due to complete damage of inlets and blockages of by rubbish/dribs. Therefore, this study recommends to maintain all the damaged inlets and clear the drainage system that has been blocked by rubbish/ dribs.



**Key words**: Hydrological analysis, SCS, Rational method, Hydraulic analysis, Steady flow simulation ,100-return period

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## Acronyms and abbreviations

| | |
|---|---|
| AACRA | Addis Ababa City Road Authority |
| AACRA DDM | Addis Ababa City Road Authority Drainage Design Manual |
| AASHTO | American Association of State Highway Transport Officials |
| AMC | Antecedent Moisture Condition |
| C | Coefficient of Runoff in Rational Formula |
| Cf | Coefficient of return period |
| CN | Curve Number |
| DEM | Digital elevation model |
| ERA | Ethiopian Roads Authority |
| ERADDM | Ethiopian Roads Authority Drainage Design Manual |
| EG | Energy Grade line Elevation |
| GIS | Geographical Information System |
| GPS | Global Positioning System |
| Ha | Hectare |
| HEC | Hydrologic Engineering Centre |
| HEC RAS | Hydrological Engineering Centre River Analysis System |
| HFL | High Flood Level |
| HSG | Hydrological Soil Group |
| IDF | Intensity Duration Frequency |
| Km | Kilometer |
| $Km^2$ | Square Kilometer |
| M | meter |
| ‘n’ | Manning’s Co-efficient |
| NMSA | National Meteorological Services Agency |
| SCS | Soil Conservation System |
| T | Time of Return |
| Tc | Time of Concentration |
| USACE | United States Army Corps of Engineers |
| USGS | United States Geological Survey |
| US NRCS | United States Natural Resources Conservation Service |
| US SCS | United States Soil Conservation Service |
| UTM | Universal Transfer Mercator |
| WS | Water Surface Elevation |

# Chapter One

## 1. Introduction

### 1.1 General back ground

Road Drainage structures are an essential component of the Road that help to collect, transport, and dispose of surface water (i.e. the water that comes from on the road way, near the roadway or flowing through the streams).

An efficient drainage system provides to dispose surface water efficiently from the road as quickly as possible. This achieved by a system consisting of the following components:

- Gutter and curbs collect Road surface water and dispose to major storm.
- Culverts which lead the water from the side drains under the road to the other lower side.
- Bridges which allow the road to cross rivers and streams in a controlled manner throughout the seasons.

The road selected for this study (Bole Roundabout to Megenagna Road) which is 4.2km asphalt concert (AC) the road is part of the road of Haile Garement to Megenagna with a total length of 16km.

The region of the study area has a significant amount of rain fall, the mean annual rainfall of Addis Ababa city varies between 600mm - 1600mm. Thus, the existence of proper drainage system along the road is mandatory unless the water lay on the road. According to **Ekblad, J. and Isacsson, U. 2006** the existence of excess water or moisture within the roadway would have a negative impact on the engineering properties of the materials which it was constructed.

The major problem along the ring road has high flooding problem due to unexpected flooding form Ambesa garage and the neighboring area, may be due to change in land use of the area ,in addition to this most of the inlet type around this section is grate inlet which is not well functioned (blocked by rubbish ) ,beside all junctions has storm drainage facility has manmade problem and lack of routine maintenance therefore it needs to be detail investigation of hydrologic and hydraulics analysis for giving an appropriate solution. As mentioned in the above, this study comprised of hydrological and hydraulics analysis, the hydrological analysis includes frequency analysis of rainfall and determination of the peak rate of discharge. The hydraulic analysis includes evaluating the existing opening sizes of the drainage based on the peak rate of runoff.

The frequency analysis carried out using Microsoft excel by the methods of Gumble and Log Pearson Type III methods. Log Pearson type III is the most fitting based on the results of test of goodness which is done by coefficient of determination(R2) , the value of $R^2$ for log Pearson type III 0.875 ,which is greater than that of Gumbles method the value of $R^2$ value 0.764 (refer Appendix ). The data that has been used the last 31 years of daily rainfall (from 1980-2020).

Based on Ethiopian Drainage Design Manual (ERA, 2013) the most widely method of runoff estimation

• Rational Method;

• NRCS Runoff Curve Number Methods;

• Statistical analysis of stream data; and

• Regional regression equations

From the above specified runoff calculation method, this study has used SCS graphical peak discharge methods and rational method based on the conditions they satisfy.

The other important parameter for rain falls runoff analysis is to study the properties of catchment area (i.e Area of the catchment, Length of the river, slope of the river, land use of the study area, types of the soil etc.). Delineation of the catchment area carried out using QGIS, Catchment Area 1 total area has equal to 5.8788 $km^2$, so it is reasonable to choose SCS graphical Methods. By the same, catchment Area 2 total area has equal to 0.306 $km^2$, So that it is very practical to select rational method for runoff estimation. The SCS (NRCS) runoff calculation method uses a combination of soil type and land cover to estimate runoff factor, however the runoff factor for rational method is directly selects from the table.

The other important parameters for determination of runoff is time of concentration. There are different empirical formulas to calculate time of concentration, however this study used the summation of time concentration for overland flow and define water course flow using kerby formula which is very practical for relatively flat slope, the slope is calculated according to the 10 -85-slope (us geological survey) method

The hydraulic analyses for this study carried out by using the HEC RAS computer program. The program computed water surface elevation on the basis of energy equation is called standard step method. Finally based on the results of hydrologic and hydraulic analysis, the paper gives a recommendation for the opening of the watercourses along the ring road.

## 1.2 Statement of the problem

The major problem exist in this Road segment has high flooding problem due to unexpected flooding form Ambesa Garage road Side and neighboring compound area during rainy season due to this, the floods overtop the road and the Road not execute the intended function.

## 1.3 Questions of the research study

The following fundamental questions that are addressed in this study:

- What are the major causes of drainage problems in the study area?
- Are the hydraulic capacities of the different drainage elements of the road adequate?
- What mitigation measures can improve the drainage problems?

## 1.4 Objective

### 1.4.1 General objective

The general objective of the study is to evaluate the performances of the existing road drainage structures and to propose mitigation measures on Bole Roundabout to Megenagna Road.

### 1.4.2 Specific objective

- To study the Rain fall - Runoff of the drainage area.
- To Evaluate the hydraulic performance of the existing drainage structure
- Determination of appropriate sizes of waterway opening for watercourses

## 1.5 Significance of the study

The result of this study may help filling of the gaps by identifying the problems through detail investigations of the existing road drainage structure and contribute solution for proper drainage system in the referenced road.

## 1.6 Scope and limitation of the study

The scope of this study specifically focuses, rainfall analysis, determination of peak rate of runoff for the specified catchments areas, computation of water surface elevation and determination of appropriate sizes of waterway opening.

Structural considerations, such as the design requirements to support loads(bridge slab thickness), are not addressed in this study and furthermore the hydraulic analyses, which is carried out using HEC RAS program to perform only steady state simulation (i.e determination of water surface profile).

# Chapter Two

## 2. Literature review

### 2.1 Road drainage structures

Road Drainage structures are an essential component of the Road that help to collect, transport, and dispose of surface water and storm water in properly.

Surface water: -any kind of water that appears on the surface of the earth in a diffused state, with no permanent source of supply or regular course for a considerable time; as distinguished from water appearing in water courses, lakes, or ponds. (ERA, 2013)

Storm water: is rainwater which has fallen on a built-up area that comes from precipitation, if storm water were not removed appropriately, it would cause awkwardness, damage, flooding, and further a cause of health issues. It contains some pollutants, originating from rain, the air, or the catchment surface (David Butler, Christopher Digman, Christos Markopoulos, and John W. Davies, 1959)

Drainage is vital pillar for road infrastructure and therefore drainage design cannot be undertaken in separation from the geometric design of the road (ERA, 2013).

Road Drainage design covers many disciplines, of which two are hydrology and hydraulics (ZIYADA BASENA, 2017). Frequency analysis of rainfall and determination of the peak discharge is hydrology analysis. Determination of the drainage structures with appropriate sizes of waterway opening for the various watercourses that is to collect, pass, divert and remove from the pavement structure is hydraulic design.

A hydraulic investigation of both the upstream and downstream reaches of the watercourse is necessary to determine the best location, size, and elevation of the proposed cross drainage structure, whether a culvert or a bridge. In the hydraulic design the structures shall have a minimum capacity of holding the expected flood in the design return period, thus the flood plain has not a significant adverse around the vicinity.

### 2.2 Purposes of road drainage structures

The main functions of road drainage system are safe passage of vehicles during the peak design storm event by optimal removal of flood and preventing of ponding on the road surface. The following preliminary criteria must full fill on the road; the drainage structures become perform their function satisfactorily.

1. surface drainage

The storm drainage facilities should be disposing the surface water of the carriage way using the surface drainage.

(a) Cross falls should be a minimum of 2.5% on carriageways, with increased crossfalls of up to 5.0% on hard shoulders.

(b) Longitudinal gradients should not be less than 0.5% on curbed roads.

(c) Drainage Inlets are the receptors for surface water collected in ditches and gutters, and serve as modulators whereby in between gutters and major storm drains. When positioned sideways or the edge of the road, drainage inlets are sized and located to limit the spread of surface water onto travel lanes. The term "inlets," as used here including to all types of inlets such as grate inlets, curb inlets, slotted inlets, etc.

Drainage inlet locations are frequently depending on by the road geometries as well as the spread of water onto the road surface. Generally, inlets are located at low elevation points in the gutter grade, intersections, crosswalks, cross-slope reversals, and on side streets to avoid the water moving into the main road. Additionally, inlets are placed upgrade of bridges to prevent drainage onto bridge decks and downgrade of bridges to prevent the flow of water from the bridge onto the road surface

d) Major Storm drains are part of the storm drainage system that collects discharge from inlets and transports the runoff to some point where it is removal place (a channel, water body, or other piped system). Storm drains can be closed conduit or open channel; they consist of one or more pipes or conveyance channels connecting two or more inlets

2. Cross drainage

In steep terrain, culvert capacity is usually governed by inlet control. The flowing of water through culvert, the entrance conditions affect the capacity of culverts subject to inlet control. The entrance condition that has a significant impact on inlet control are the geometry of the opening, the wing walls, head walls, the angle of wing walls & head walls and the protection of the culvert in to the headwater pond.

Pipe roughness, outlet conditions such as tail water level do not has an impact on the flow capacity of culverts operating under inlet control. When the culvert barrel is under performance of conveying of water as much flow as the inlet opening will accept the outlet, control occurs (FHWA, 2001).

## 2.3 Types of road drainage structures

It has various types of road drainage structure such as storm drainage facilities (curbs, gutters, drains) and cross drainage structures (i.e culverts and bridges). The placement and hydraulic capacities of road drainage structures should be designed to minimize damage to adjacent property and protected a low degree of risk of traffic disruption by flooding. Different types of Road drainage structures are working in the drainage systems,

- Storm drainage facilities, used to collect the runoff of the carriageway and surrounding areas and direct it to the channels (ERA, 2002).
- Culverts convey flows under road cross-section
- Bridges are structures that transport traffic over waterways (ERA, 2002)

### 2..3.1 Storm drainage

Road storm drainage facilities are basic component of the road drainage system that help adequately drains the water in the carriageway by gathering of water from the carriage way and transport through the pipe or channel and minimizes flooding and erosion for properties of adjacent to the right-of-way (ERA, 2013).

According to ERA, 2013 Drainage manual, Storm drainage facilities comprise of curbs, gutters, storm drains, channels and culverts.

**Gutters and Curbs are** Storm drainage facilities that are used for as surface drainage of the road carriage way. Surface drainage is accompanied by transverse pavement slope and longitudinal slope, pavement roughness, inlet spacing, and inlet capacity (ERA, 2013).

**Gutters** are used to capture pavement runoff and carry it along the roadway shoulder to an adequate storm drain inlet. The pavement (surface runoff) slowly moves to gutter by shaping the carriageway with a camber or a cross slope. Gutters and curbs form a triangular channel and carry surface discharge equal to or less than the design flow discharge. Gutters with curbs available in 0.3-to-1-meter widths.

ERA drainage manual recommends Gutter cross slopes may be equivalent to that of the pavement slope or may be considered as a steeper cross slope, usually 80 mm per meter steeper than the shoulder. AASHTO geometric guidelines state that an 8% slope is a common maximum cross slope.

**Curbs** are outside edge of the road and the key purposes of the curbs are Facilitate the surface runoff within the road and dispose properly to storm drain inlet. Curbs are typically installed in combination with gutters where runoff from the pavement surface would erode fill slopes and/or where right-of-way requirements or topographic conditions will not permit the development of roadside ditches. Pavement sections are typically curbed in urban settings. Parabolic gutters without curbs are used in some areas

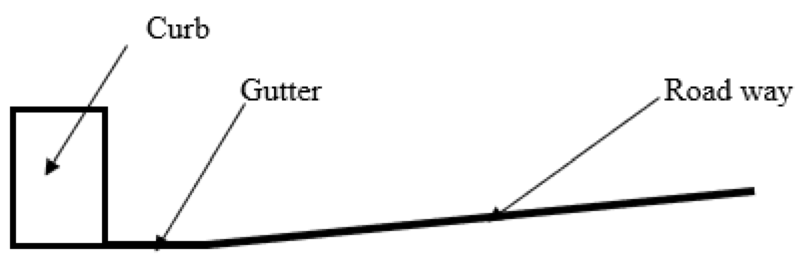



Figure 2.1 Gutter and curb

Source ERA DDM manual

### 2.4.1 Culverts

**Culverts** are the most common cross drainage structure used on roads. The main function of culvert is a conveyance of surface water and stream flow from the uphill side of the road to the lower side where it can be safely discharged. Besides to the hydraulic purpose, a culvert must carry self-weight (dead load) and road traffic weight (live load). Culvert design, therefore, includes both hydraulic and structural design.

There are basically two types of culverts, depending on their function:

- Stream culverts, allowing water from streams and canals to cross under beneath the road,
- Relief culverts, which divert water from the high side to the lower side of the road, relieving the side drains of water which cannot be discharged by miter drains.

The selection of the most appropriate type of culvert is dependent on a range of factors including economics, site conditions, and environmental considerations (ERA, 2013). However, Box culverts are the most widely used culvert type

#### 2.4.1.1 Hydraulic performance of culverts

The most significant reflection in culvert hydraulics is whether the flow is subject to inlet or outlet control (ERA, 2013).

For inlet control there are two separates hydraulics regime exist, depending on whether the inlet is submerged or not submerged. Outlet control exist in long culverts, culverts placed on level (flat) grades and with high tail water depths. In designing culverts, the type of control is computed by accepting the bigger headwater depths calculated for both inlet control and outlet control.

#### 2.4.1.2 In let control

The culvert is categorized under inlet control the following requirements may full filled

- The level of water under culvert barrel is bigger than that of the inlet (head water depth)
- Water level in the head or ponding at the entrance and the entrance conditions, such as the entrance type, existence and angle of headwalls and wing walls, and the projection of the culvert into the headwater pond.
- The inlet submerged and the outlet not submerged

When the culvert flows under inlet control, the roughness and length of the culvert barrel and the outlet conditions (including the depth of tail water) has not affected in computing culvert capacity. The head of water depth slightly decrease while increasing the slope of the culvert, and can normally be neglected for conventional culverts flowing under inlet control.

#### 2.4.1.3 Outlet control

With outlet control the culvert flow is restricted to the discharge which can pass through the conduit for a given level of water in the outlet channel (tail water level)

- A tail water depth equal to 80% or more of the height of the culvert barrel/cell will usually indicate outlet control,
- Culverts flowing with outlet control can flow with the culvert barrel full or with the barrel part-full for all of the culvert length.
- With outlet control, and both the inlet and the outlet submerged the culvert flows full and under pressure. The culvert, also, can flow full over part of its length with part-full flow at the outlet.

### 2.3.3 Bridges

The main functions of bridges are carrying highways, railroads, and utilities over surface waters. Commonly, a bridge is defined as having a span of 20 feet (6m) or more, as contrasting to a culvert (Santosh Kumar Garg ). If a bridge is not designed appropriately with respect to the design flow, overtopping and flooding will occur, leading to public hazards, erosion damage, and possible structural failure.

#### 2.3.3.1Linear water way of bridge

Linear water way of the bridge is total length of the bridge from abutment to abutment. The Hydraulic capacity of the bridge directly connected the bridge water way width.

Based on ERA DDM in a given design discharge the shorter span bridge have a higher velocity through the opening and it makes higher scour at the bridge foundation and further more creates more back waters in the upstream.

Therefore, the bridge design must provide enough capacity to:

• Eliminate extreme backwater effects on the upstream

• Eliminate extreme velocity and shear stress within the bridge waterway.

Hydrologist/hydraulic Engineers recommend having adequate free board for their bridges since reducing of back water effects.

The linear water way of a bridge across a purely alluvial stream is generally kept equal to its regime width (w) as given by lacy. The lacy's regime width of the stream is taken equal to its wetted perimeter (P) is given by the equation

$L = 4.75\ Q^{0.5}$

Streams which are semi alluvial (sides rigid and bed alluvial) or purely rigid the water way would be determined the width of the stream from edge to edge when the flood rises high flood level (H.F.L)

L = W = Width of the stream (in H.F.L)

The following main factors that have a significant influence on the hydraulic capacities of the bridges

- The bridge length (the bridge water way width)
- The alignment of the road (the road profile)

- Elevation (height) of low cord
- Channel stability
- Height of free board

#### 2.3.3.2 Bridge design principles

Based on FHWA during Bridges design the following general requirements must full filled

- The maximum back water allowed 0.5m, unless exceeding of the limit can be justified by hydraulic conditions
- The design should not significantly change the natural flow conditions.
- A definite clearance should be set based on the size conditions to allow for passage of debris.
- Degradation or aggradations of the river, local scour and contraction shall be estimated as part of the design
- Foundation of the bridges positioned below the total scour depth

#### 2.3.3.3 Bridges hydraulics condition

Bridge hydraulics conditions explained for the various types of flow conditions that exist at bridge crossing.

#### 2.3.3.4 Free surface bridge flow

In this type of flow the bridges low cord is not submerged, based on HEC RAS hydraulics reference manual classified into three flow conditions (i.e. class A, Class B, Class C) depending on the flow regime.

**Class A**:- under this category all of the regime (the upper stream ,downstream and the bridge water way) is sub critical. Furthermore, the flow satisfies the conditions of gradually varied flow (i.e steady state flow)

**Class B**:-flow pass a critical depth at the bridges water way, super critical flow exist at downstream after a short distant of control section (critical depth)

Mostly upstream and downstream flow becomes sub critical flow, hydraulic jump will often exist when within the bridges water way or short distant downstream of the bridges

R.M critical flow exist the flow moves from sub critical to super critical.

**Class C:** in this type of flow all the regime under super critical flow that is elevation of critical depth greater than the water surface elevation. This type of flow occurs in rare condition.

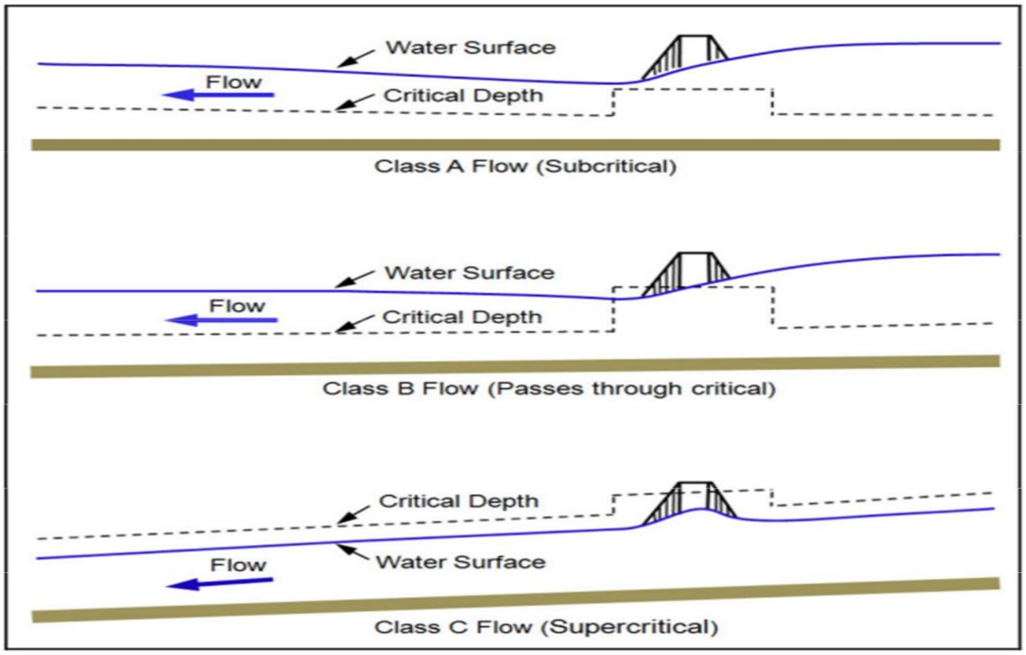



Figure 2.2: Free-surface bridge flow classes A, B, and C

(Source: USACE 2010c).

### 2.3.3.5 Overtopping flow

This type of flow condition moves through across the bridge deck. It is represented by broad crested weir. Overtopping flow is a combined flow of free surface bridge flow and submerged deck flow in the bridge water way, during overtopping flow occurs, the engineer shall be control how much flow is going through the bridge opening water way and how much water flow over the bridge deck.

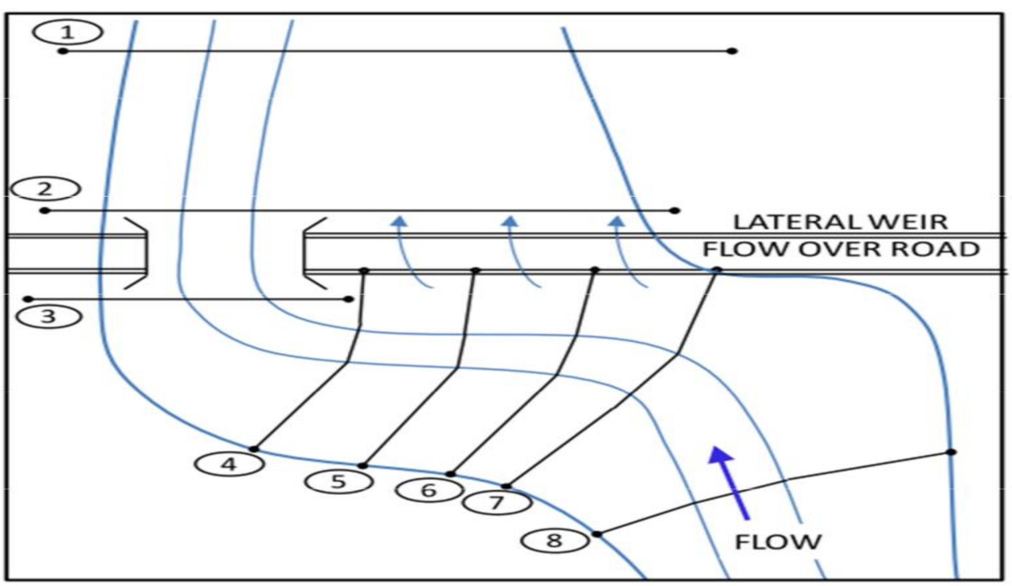



Figure 2.3: Lateral weir flow over road

(Source: Hydraulic Design Series Number 7, 2012).

#### 2.3.3.6 Flow submerging the bridge low chord

The water surface is above the elevation of the highest bridge low cord. It is represented by orifice flow. The first orifice flow exists when the low chord is totally submerged only at the upstream edge and this type of orifice flow is considered as free-flowing, and thus not affected by tail water. The second type of orifice flow exists when the upstream and downstream edges of the highest point of low chord submerged. This type of flow is evaluated using a formulation for a tail water-controlled orifice (FHWA 1978).

## 2.4 Hydrologic analysis

The main purpose of hydrologic study is to determine the peak flood of road-crossing rivers or streams so as to determine the size of drainage structures along the road alignment. Hydrologic design is the procedure of evaluating the influence of hydrologic events on a water resource system and selecting standards for the key variables of the system so that it will accomplish sufficiently (V.T.Chow, etal, 2004). In doing hydrologic analysis for drainage structures there are numerous variable issues that affect floods. ERA, DDM 2002 states some of the factors that need to be recognized and considered on individual site by site basis:

- Rainfall amount and storm distribution
- Catchment area size and  Ground cover
- Type of soil
- Slopes of terrain and stream(s)
- Antecedent moisture condition
- Future catchment area development potential.

Hydrologic analysis requires some parameters which must be extracted from the watershed (Catchment Area, Length of longest flow path, ground cover and soil map etc). The first step for doing hydrologic analysis extracting of hydrological parameters from the watershed. Previously these parameters were extracted manually from topographic maps, field surveys, or aerial photographs which were a labor-intensive activity. So that there is a need for determining this parameter with the help of computer system capable of assembling, storing, manipulating, and displaying geographically referenced information. QGIS 3.16 and QGIS 3.18 has been used for extraction of hydrological parameters, the expected peak rate of runoff for the catchment areas estimated by soil conservation service (**SCS**) graphical method and Rational for catchment Area 1 and catchment Area 2 respectively, based on the conditions that they satisfy.

Some of flood run off computation methods that has been approved by ERA:

- Rational method shall be used only for catchment areas less than 80 hectares (0.8 $km^2$ ).
- SCS and other unit hydrograph methods for catchment areas greater than 80 hectares;
- Catchment area regression equations shall be used for all routine designs at sites where applicable;
- Gumble or Log Pearson III analyses shall preferably be used for all routine designs if there is at least 10 years of continuous recorded discharge data for 10-year discharge estimates and 25 years recorded discharge data for 100-year discharge estimates;
- Suitable computer programs such as HYDRAIN's HYDRO, HEC HMS,

### 2.4.1 Rational method

It is one of the most usually used equations for computation of peak discharge for drainage area less than or equal to 80ha. The peak discharge for the rational method occurs when the entire watershed is contributing. AASHTO, 2014 also states that with few exclusions, runoff approximations for drainage design are performed by using rational methods. In order to make use of these methods, information on the intensity, duration, and frequency of rainfall for the locality of the project must be established. V.T. Chow and his friends on their applied hydrology book stated that the idea behind the rational method is: If a rainfall of intensity i begins instantaneously and continues indefinitely, the rate of runoff will increase until the time of concentration tc, when all of the watershed is contributing to flow at the outlet. The product of rainfall intensity i and watershed area A is the inflow rate for the system and the ratio of this rate to the rate of peak discharge Qp (which occurs at time tc) is termed the runoff coefficient C ($0 < C < 1$).

This is expressed in the rational formula:

$$QP = 0.278CIA$$

Where Qp = the peak discharge for the required return period ($m^3$ /s)

C = the runoff coefficient

I = the rainfall intensity for a required return period of duration equal to critical storm duration (mm/hr) and A = the drainage watershed area ($km^2$)

### 2.4.2 SCS peak discharge method

The peak discharge method was formulated by the United State Department of Agriculture (USDA) Soil Conservation Service (SCS now NRCS), it uses the same basic data as NRCS: catchment area, a runoff factor (curve number), time of concentration, soil type, land use and rainfall. The SCS peak discharge method use rainfall time distributions which are used for to transform excess precipitation into peak runoff

According to Technical Released 55(USDA 1986) the SCS graphical peak discharge estimation method only used the weighted Average CN is greater than 40 and the total catchment area less than 2000 acers ($8km^2$)

The SCS peak discharge has an interactions with watershed characteristics and metrological event, some of the key factors that have significant impact to SCS peak discharge are summarized below

- Rainfall : the total amount of rainfall that occurred in the catchment area increased the peak discharge also increased.
- Rainfall duration : for the same amount of storm occurred in the catchment, then the shorter storm duration has high peak discharge rather than longer periods.
- Size of watershed: increasing water shed area increasing the peak rate of runoff.
- Shape of waatershed : the more compacte watershed ,has more peak rate of rumoff
- Slope : the stepper slope should gives the higher discharge
- Land use: the catchment area coverd with more of vegetative produce the lower discharge ,on the other hand the area covered with impervious surface produce the higher peak discharge etc.

#### 2.4.2.1 Rainfall distribution

These rainfall distributions formulated by the SCS now nkown as the natural resource conservation service (NRCS) as an averages of rainfall pattern,the storm designed for small catchment area and it enables to determine the peak discharge from 24 hr excess percepitations of acritical duration called time of concentration.

The NRCS storm distribution formulated from historical storm event.The rainfall time distribution is a 24 hour cumulative rainfall distributions with step time interval of 0.1 hour which are used to design water related projects.

According to ERA DDM,2013 recommends NRCS rainfall distriubiutions type II for determining of the peak rate runoff ,by the reason of having high rate of runoff.

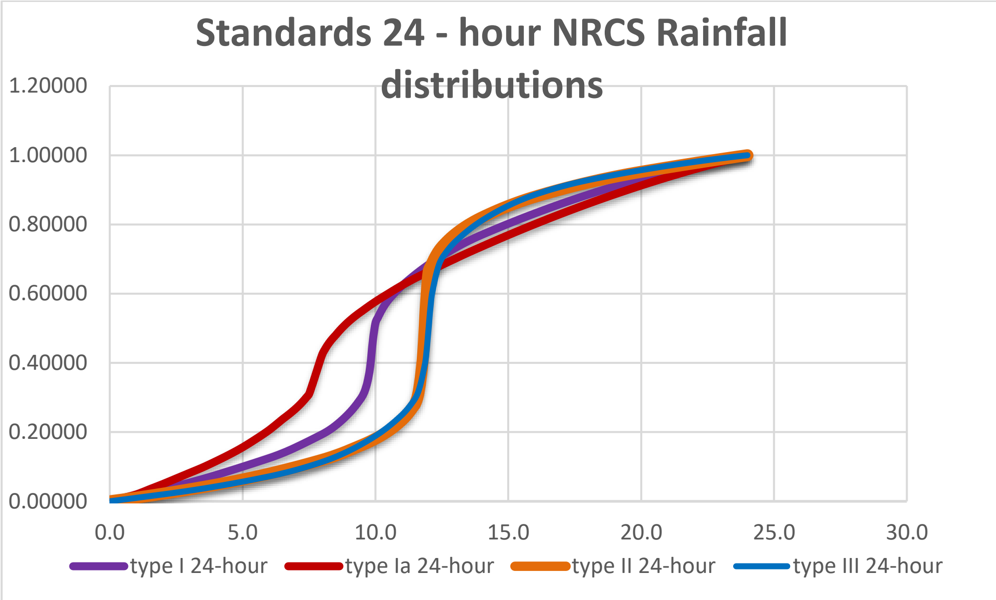



Figure 2.4 NRCS Rainfall distributions

Table 2.0 Regression coefficients for NRCS Rainfall distributions Type I and IA

| Rainfall type I | | | |
|---|---|---|---|
| Ia/P | C1 | C2 | C3 |
| 0.1 | 2.3055 | -0.5143 | -0.1175 |
| 0.2 | 2.23537 | -0.5039 | -0.0893 |
| 0.25 | 2.18219 | -0.4849 | -0.0659 |
| 0.3 | 2.10624 | -0.457 | -0.0284 |
| 0.35 | 2.00303 | -0.4077 | 0.01983 |
| 0.4 | 1.87733 | -0.3227 | 0.05754 |
| 0.45 | 1.76312 | -0.1564 | 0.00453 |
| 0.5 | 1.67889 | -0.0693 | 0 |

| Rainfall type IA | | | |
|---|---|---|---|
| Ia/P | C1 | C2 | C3 |
| 0.1 | 2.0325 | -0.3158 | -0.1375 |
| 0.2 | 1.91978 | -0.2822 | -0.0702 |
| 0.25 | 1.83842 | -0.2554 | -0.026 |
| 0.3 | 1.72657 | -0.1983 | 0.02633 |
| 0.5 | 1.63417 | -0.091 | 0 |

Table 2.1 Regression coefficients for NRCS Rainfall distributions Type II and III

| Rainfall type II | | | |
|---|---|---|---|
| Ia/P | C1 | C2 | C3 |
| 0.1 | 2.55323 | -0.6151 | -0.164 |
| 0.3 | 2.46532 | -0.6226 | -0.1166 |
| 0.35 | 2.41896 | -0.6159 | -0.0882 |
| 0.4 | 2.36409 | -0.5986 | -0.0562 |
| 0.45 | 2.29238 | -0.5701 | -0.0228 |
| 0.5 | 2.20282 | -0.516 | -0.0126 |

| Rainfall type III | | | |
|---|---|---|---|
| Ia/P | C1 | C2 | C3 |
| 0.1 | 2.47317 | -0.5185 | -0.1708 |
| 0.3 | 2.39628 | -0.512 | -0.1325 |
| 0.35 | 2.35477 | -0.4974 | -0.1199 |
| 0.4 | 2.30726 | -0.4654 | -0.1109 |
| 0.45 | 2.24876 | -0.4131 | -0.1151 |
| 0.5 | 2.17772 | -0.368 | -0.0953 |

#### 2.4.2.2 Catchment area

A catchment area is involves of all land area that contributes water to the outlet during rain, a catchment area (watershed area) extracted from; DEM data's using different extractions software (i.e QGIS,GIS..etc) and also topographic maps of the study area and to carried out field vist (field survey) are very important for delineation of the watershed. A field survey very crucial for existing or planned drainage system if the natural drainage system have been changed. These changes might make substantial variations in the size and slope of the drainage areas and further more may have an impact for the existing or the planned drainage system.

#### 2.4.2.3 Channel length

Channel length is very important in hydrologic design and analysis computation, according to Richard H.McCuen the cannel length computational schemes classified in to two

1. The distance measured throughout the main channel from the watershed outlet to the end of the channel (longest flow path)

2. According to USGS The distance that measured between two points located in 10% of the channel distance and 85% of the distance along the channel from the outlet, which is denoted as $L_{10\text{-}85}$, therefore USGS active length of the channel is 0.75 L

#### Over land slope

Channel slope defined as change in elevation with respect to the distance along the principal flow path (longest flow path), from the hydraulically most distant point to the point of out let

$S = \Delta E/L$

**channel slope**: it is used for determination of defined water coarse flow, the channel slope either determined by the average slope or 10 – 85 (USGS method) the average slope method determined in graphically based on the balance of areas above and below the line of the average slope (ERA DDM,2013). Alternatively, 10-85 (USGS method) determined by the rate of change elevation at 10% of the channel distance and 85% of the channel distance divide by the distance in between (10%-85%) which is 0.75L

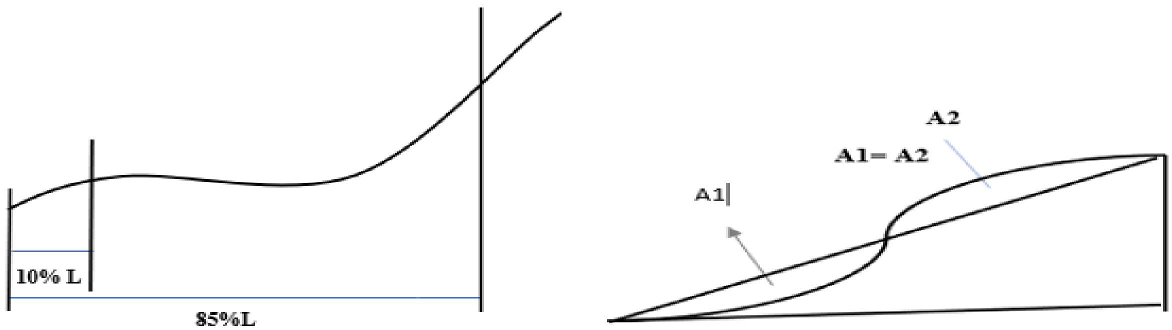



Figure 2.5 (a )10 - 85-slope Figure according to "US Geological survey

2.5 (b): Slope according to weighted average area method

### 2.4.2.4 Land use/land cover

Land use is the catchment area that broadly divided in to two main groups (agricultural and non-agricultural), Agricultural Land use like forest, grass and open space. Non-agricultural land covers such as industrial, commercial, residential, recreational ground and road. Land treatment applied mostly to agricultural land use, and it comprises mechanical practices such as contouring or terracing and agricultural land management practices such as rotation of crops. The SCS uses a combination of soil type and land- cover to deal a runoff factor for the specified area. These runoff factors are commonly known as curve numbers (CN) that give the runoff potential of the catchment area. The higher the CN, the higher is the runoff potential.

### 2.4.2.5 Hydrologic soil groups

The rainfall runoff of the catchment significantly affects by the engineering characteristics of the soil, which is the infiltration rate different from one soil type to others. The SCS has divided into hydrologic soil groups (HSG's) based on infiltration rates, which are A, B, C, and D, HSG's are very critical for determination of curve numbers. The four soil groups that are defined by SCS as follows:

**Group A**: The soils classified under this Group have high infiltration rates with low runoff potentials, even if the soils completely saturated. This type of soils consists of more than 90 % comprised by Gravel and sand and the rate of transmissivity of water (greater than 0.30 in/hr). **Group B:** this type of soils has moderate infiltration rates and moderate runoff, the soil comprised of from fine to moderately coarse textures (e.g shallow loess and sandy loam). These soils have a moderate rate of water transmissivity (0.15 to 0.30 in/hr).

**Group C:** the soils have low infiltration rates with high rate of runoff and the soils comprised of with moderately fine to fine texture (clay loams and soils with low organics). These soils have a low rate of water transmissivity (0.05 to 0.15 in/hr).

**Group D**: these soils have low infiltration rates and high runoff potential. the soils have more plastic clays and saline soils due to this the soils have a high swelling potential; these soils have a very low rate of water transmissivity within the soil (0 to 0.05 in/hr).

Table 2.1: Typical Hydrologic Soils Groups for Ethiopia

| I | **Soil Types** | **Hydrologic Soil Group** |
|---|---|---|
| Ao | Orthic Acrisols | B |
| Bc | Chromic Cambisols | B |
| Bd | Dystric Cambisols | B |
| Be | Eutric Cambisols | B |
| Bh | Humic Cambisols | C |
| Bk | Calcic Cambisols | B |
| Bv | Vertic Cambisols | B |
| Ck | Calcic Chernozems | B |
| E | Rendzinas | D |
| Hh | Haplic Phaeozems | C |
| Hl | Luvic Phaeozems | C |
| I | Lithosols | D |
| Jc | Calcaric Fluvisols | B |
| Je | Eutric Fluvisols | B |
| Lc | Chromic Luvisols | B |
| Lo | Orthic Luvisols | B |
| Lv | Vertic Luvisols | C |
| Nd | Dystric Nitosols | B |
| Ne | Eutric Nitosols | B |
| Od | Dystric Histosols | D |
| Oe | Eutric Histosols | D |
| Qc | Cambric Arenosols | A |
| Rc | Calcaric Regosols | A |
| Re | Eutric Regosols | A |
| Th | Humic Andosols | B |
| Tm | Mollic Andosols | B |
| Tv | Vitric Andosols | B |
| Vc | Chromic Vertisols | D |
| Vp | Pellic Vertisols | D |
| Xh | Haplic Xerosols | B |
| Xk | Caloic Xerosols | B |
| Xl | Luvic Xerosols | C |
| Yy | Gypsic Yermosols | B |
| Zg | Gleyic Solonchaks | D |
| Zo | Orthic Solonchaks | B |

(Source: Ministry of Agriculture)

### 2.4.2.6 Runoff factors

Runoff is rainfall excess or effective rainfall - the amount by which rainfall exceeds the capability of the land to infiltrate or otherwise retain the rainwater. The principal physical catchment area characteristics affecting the relationship between rainfall and runoff are land use, land treatment, soil types, and land slope.

Land use is the catchment area cover, and it includes both agricultural and nonagricultural uses. Items such as type of vegetation, water surfaces, roads, roofs, etc. are all part of the land use. Land treatment mostly applies to farming land, and it consist of mechanical practices for instance contouring or terracing and farming land management practices such as rotation of crops. To determine runoff factor for specific catchment area, SCS uses a combination of soil conditions and land use. This runoff factors, called runoff curve numbers (CN), indicate the runoff potential of an area. The catchments have a higher CN, the catchment area will have been higher runoff potential

### 2.4.2.7 Runoff curve numbers

The main aspects that govern the CN are the hydrologic soil group, Land use, hydrologic condition, and Antecedent Moisture condition. Tables 2.2 typical CN values for urban areas respectively, which taken from ERA DDM

Urban Area CNS (Table 2.2) were formulated for typical land cover interactions based on particular expected percentage of impervious area. These CN values were formulated depend on the following assumptions

- All pervious urban surfaces are the same as to grassland with in good hydrological conditions
- All impervious surfaces have a CN of 98 and all the impervious surface areas are directly connected to the drainage system

Some assumed percentage of impervious area and the corresponding land cover are as shown below in table

**Curve numbers for hydrologic soil groups**

Table 2.2: Runoff Curve Numbers- Urban Areas1

| **Cover description** | **Curve numbers for hydrologic soil groups** | | | | |
|---|---|---|---|---|---|
| **Cover type and Hydrologic condition** | **Average % impervious area2** | **A** | **B** | **C** | **D** |
| Open space (lawns, parks, cemeteries, etc.) | | | | | |
| Poor condition (grass cover <50%) | | 68 | 79 | 86 | 89 |
| Fair condition (grass cover 50 % to 75%) | | 49 | 69 | 79 | 84 |
| Good condition (grass cover >75%) | | 39 | 61 | 74 | 80 |
| Impervious areas: | | | | | |
| Paved parking lots, roofs, driveways, etc. | | | | | |
| (excluding right-of-way) | | 98 | 98 | 98 | 98 |
| Streets and roads: | | | | | |
| Paved; curbs and storm drains (excluding right-of-way) | | 98 | 98 | 98 | 98 |
| Paved; open ditches (including right-of-way) | | 83 | 89 | 92 | 93 |
| Gravel (including right-of-way) | | 76 | 85 | 89 | 91 |
| Dirt (including right-of-way) | | 72 | 82 | 87 | 89 |
| Desert urban areas: | | | | | |
| Natural desert cover | | 63 | 77 | 85 | 88 |
| Urban districts: | | | | | |
| Commercial and business | | 89 | 92 | 94 | 95 |
| Industrial | 85 | 81 | 88 | 91 | 93 |
| Residential districts by average lot size: | 72 | | | | |
| 0.05 hectare or less | 65 | 77 | 85 | 90 | 92 |
| 0.1 hectare | 38 | 61 | 75 | 83 | 87 |
| 0.135 hectare | 30 | 57 | 72 | 81 | 86 |
| 0.2 hectare | 25 | 54 | 70 | 80 | 85 |
| 0.4 hectare | 20 | 51 | 68 | 79 | 84 |
| 0.8 hectare | 12 | 46 | 65 | 77 | 82 |
| Developing urban areas | | | | | |
| Newly graded areas (pervious areas only, no vegetation) | | 77 | 86 | 91 | 94 |

Source ERA DDM- 2013

#### 2.4.2.8 Time of concentration

The time of concentration Tc is well defined as the time spent for water to travel the longest flow path from the hydraulically most distant point in a drainage area to the point of out let (interest point) or time at which all parts of the watershed begin contributing to the runoff from the watershed area.

Various methods or equation to calculate time of concentration the most popular Tc equations is The Kirpich equation was established from the data of six agricultural catchment area in Tennessee, USA (the size of watershed ranging from from (0.4 ha to 45 ha), with well-defined water coarse and the slopes of drainage area ranging from 3% to 10% (Viessman and Lewis 1996).

The other popular time of concentration equation was developed by Hathaway, the formula established on the basis of data from very small watersheds, the size of watershed less than 1.8ha, the slopes of the drainage area were less than 1% and storm runoff was dominated by surface flow (MaCuen 1989). the summation of  Over Land Time of concentration and the channel time of **concentration** gives an appropriate total time concentration of the runoff at the interest point.

#### A. Time of concentration for over land flow

Overland flow is the type of flow there is no clearly defined water course (channel) that exists in the upper reaches of the catchment area, the flow depth is very small and occurred in flat area (the slope is very flat). It is only appropriate where the slope is fairly even. ERA-DDM ,2013 recommends Kerby formula to calculate time of concentration of the overland flow that shown below.

$$Tc = \left(\frac{2.2 * n * l}{s^{0.5}}\right)^{0.324}$$

n  = maninings coefficient

L = length of over land flow in feet

S = slope

### Overland flow length

According to ERA DDM Selection of overland flow length the following criteria must fulfilled

- The overland flow path is not necessarily perpendicular to the contours shown on available mapping.
- Overland flow paths in less of 60 m in urban areas and 120 m in rural areas.

### B. Time of concentration for defined watercourses

The flow moves in defined watercourse, forming a channel flow. ERA – DDM, recommended empirical formula for determining the time of concentration in natural channels using the kirpich method

$$T_c = \left(\frac{0.87L^2}{1000S_{av}}\right)^{0.385}$$

L= length of over land flow in km

$S_{av}$ = average slope

**Summary of runoff estimation method**

Table 2.3 Types of runoff estimation method
Source ERA,DDM

| Method | In put Data | Recommended maximum area(km 2 ) | Return period of flood that could be determined (years) |
|---|---|---|---|
| **Rational Method** | Catchment area, watercourse length, average slope, catchment characteristics, rainfall intensity | <0.5 | 2 – 200, PMF |
| **SCS Method** | Catchment area, watercourse length, length to catchment centroid (center), mean annual rainfall, veg. type ,soil cover and synthetic regional unit hydrograph | 0.5 to 65 | 2 – 200, PMF |
| **Synthetic Hydrograph Method** | Catchment area, watercourse length, length to catchment centroid (center), mean annual rainfall, veg. type and synthetic regional unit hydrograph | 0.5 to 5000 | 2-200PMF |
| **Empirical methods** | Catchment area, watercourse length, distance to catchment centroid (center), mean annual rain fall | No limitation large areas | 2 – 200, PMF |
| **Statistical method** | Historical flood peak records | No limitation large areas | 2 – 200 (depending on the record length) |

### 2.4.3 Design rain fall for shorter duration

The depth of gauged rain fall data that obtain from Ethiopian metrological agency in a basis of 24 hr duration. However, the drainage structures for design and analysis may require shorter duration rain fall data, so that appropriate IDF derivation for shorter duration is required. ERA -DDM (2013) recommend the following equation

$$R_{Rt} = \frac{\mathbf{t\,(b+24)^n}}{\mathbf{24\,(b+t)^n}}$$

RRt = Rainfall depth ratio Rt: R24

Rt = Rainfall depth in a given duration t

R24 = 24hr rainfall depth Coefficients b = 0.3 and n = 0.78 - 1.09

Using Log Pearson type III distribution method of frequency analysis, a 24-hour rainfall depth is calculated for different return periods (2, 5 ,10, 25, 50 and 100) year. Rearranging the above equation gives the intensity rainfall for the required time of concentration.

$$R_{Rt} = \frac{\mathbf{t\,(b+24)^n}}{\mathbf{24\,(b+t)^n}} = \frac{\mathbf{Rt}}{\boldsymbol{R24}}$$

$$\mathbf{R_t = \frac{t\,(b+24)^n}{24\,(b+t)^n} * R_{24}}$$

We know that intensity $I = \frac{\mathbf{Rt}}{\boldsymbol{t}}$

$$\text{Then } I_{t\,=} = \frac{(b+24)^n}{24\,(b+t)^n} * R_{24}$$

Using b = 0.3 and n = 0.92 as suggested by ERA manual

## 2.5 Hydraulics study

Generally, the Hydraulic investigation is carried out by manning formula. The hydraulic physical properties of the river have an influence on the peak discharge of the river, such as slope of the stream, velocity of flow, roughness of the stream, cross sectional area of the stream and shape of the stream. The most widely used equation to solve uniform open channel flow calculations is Manning’s equation. This method is used for the design flood levels at crossing sites after the design discharges have been estimated by the one of appropriate hydrological methods. Accordingly, the following Manning’s equation can be used for high-water computations in the hydraulic design of drainage structures:

$Q = \frac{1}{n} A R^{2/3} S^{0.5}$

Q = Discharge in [m3/sec]

R = Hydraulic mean depth [m] = A/P

A = Cross-sectional flow area [m2 ]

P = Wetted perimeter [m]

S = Longitudinal bed slope [%]

n = Manning's coefficient

Uniform open channel flow which is required for use of the Manning equation occurs for a constant discharge through a channel with constant slope, size, shape, and roughness. This situation exists when there is partially full flow. Uniform flow occurred in partially full pipe flow which has a constant discharge through a pipe of unchanging diameter, surface roughness and slope. Under these conditions the water depth become constant (unchanging) throughout the pipe length. S is the slope of the hydraulic grade line of the flowing of water. For uniform flow, the depth of flow through the length of pipe is constant, so the slope of the hydraulic grade line is the same as the slope of the water surface. The common flow condition in urban drainage pipes is part-full flow. The presence of the free surface must be taken into account in hydraulic computations. In uniform steady gravity flow, equilibrium exists along a part-full pipe or channel. The energy consumed by friction between the liquid and the pipe wall is in balance with the fall along the pipe length. If pipe slope could be increased for the same flow-rate, additional energy would be available to the flow, resulting in higher velocity and lower depth.

### 2.5.1 Identifying model boundary conditions

The basic role of hydraulic engineer in the design/analysis of the drainage structure (like bridge) is to select representative boundary conditions for hydraulic analysis. The following boundary condition as shown in below

- The upstream condition is specified: - for known or computed up stream water surface level, for this condition the flow will be supercritical flow.
- The downstream condition is specified: - For known water surface elevation in the downstream and for this condition the flow will be subcritical flow
- Mixed flow conditions: - The downstream and upstream condition is specified

Based on ERA DDM the boundary condition specified based on identifiable hydraulic controls or on other reliable information. There are several types of hydraulic controls; these include slope breaks where critical depth occurs (from flat to steep in the downstream direction), diversion dams, bridges, roads and other structures.

The water surface elevation determining from gauge data or an observed high-water mark, however this information is not available, the modeling model boundary should be fixed to a location where the uncertainty does not affect the result at the crossing site (ERA, DDM 2013)

### 2.5.2 Normal depth and energy slope

The normal depth is referred to as the equilibrium depth. Since depth of flow and velocity are constant when conditions are uniform, and pressure at the surface is atmospheric, the normal depth occurs EGL (energy grade line) and HGL (hydraulic grade line) are parallel to the bed, and the HGL coincides with the water surface. (David Butler, 2004 ) .Regarding to normal depth the flow characteristics such as the flow depth and the flow velocity constant throughout the channel path. Mostly this is not occurs in natural rivers, however it can be a realistic estimate for creating boundary conditions in many situations. Using the channel invert for determination of bed slope shall be avoided in natural channels because the channel bed elevation can vary with short distances.

The advanced estimation is to use the floodplain slope that extract from a topographic map. Using energy slope or normal depth computes water surface elevation within the desired slope. Normal Depth Calculation For a constant discharge flowing through a channel with the same bottom slope, cross-sectional shape and size, and Manning roughness coefficient, the depth of flow will be constant and it is called the normal depth. The process of computing the normal depth for open channel flow is the same as for gravity flow through partially full pipe flow

### 2.5.3 Critical depth

Critical depth is a relatively well-defined boundary condition when a control structure produces a sudden drop in the channel. Critical depth in natural channels is unusual except in steep, bedrock or boulder-bed channels. In HEC-RAS (USACE 2010c) critical depth is defined as the minimum total energy.

In a natural channel, total energy includes the energy correction coefficient, $\alpha$, so roughness and flow distribution impact the determination of critical depth. Critical depth should be confirmed as reasonable before using it as a boundary condition in natural channels.

### 2.5.4 Computer hydraulics analysis program

In now days computer based hydraulic analysis is required for design of bridges and culverts, for computer modeling hydraulic design/analysis needs a decision for choosing the flow model. There are some flow model types, these are gradually varied flow (steady state flow), unsteady state flow, two dimensional steady flows and two-dimensional unsteady flow.

#### 2.5.4.1 HEC RAS

HEC RAS is an abbreviation for Hydrological Engineering Center for River Analysis System. It is part of the US Army Corps of Engineers. HEC RAS is the most appropriate and widely used freely available software program, in which to be determine the water surface profile of the river.

The US Army Corps of Engineers has made several computer programs for giving a solution regarding to water flow problems namely HEC1, HEC2, etc. HEC2 manipulated the water surface profile for a gradually varied flow. The HEC1 and HEC2 program do not have a user interface.

Later, the US Army Corps of Engineers develop a graphic user interface program, with interactive input of data and visualization of results. This program version is called HEC-RAS. The latest form of HEC-RAS, Version 6.2 of the River Analysis System (HEC-RAS) is available in open source. The other new form of RAS is GEO RAS; it is manipulating water surface elevation in 3D form; however, it is not open source

According to HEC RAS 6.0 reference manual, the program computed four one dimensional river analysis components as follow

1. computations of water surface profile for steady flow
2. unsteady flow simulation
3. Quasi unsteady or fully unsteady flow movable boundary sediment transport components (1D & 2D)
4. Water quality controls

From the four fundamental one dimensional river analysis component, this study focused only steady flow analysis for computations of water surface profile.

Steady flow water surface profile enables to model sub critical super critical and mixed flow regime water surface profile, the basic computational procedure is based on one dimensional energy equation, energy loss determines by manning's equation.

The momentum equation applied for rapidly varied flow; the flow is rapidly varied flow the regime will be mixed flow conditions.

HEC RAS has good integration system with spatial data and mapping system (HEC RAS mapper)

# Chapter three

## 3 Research methodologies

### 3.1. General description of study area

#### 3.1.1. Location

It is well aware that, Addis Ababa is capital city of Ethiopia and the third economic and diplomatic center in the world, its locations between latitude of $8^{0}50'11''$ - $9^{0}05'29''$North and longitude of $38^{0}39'40''$ - $38^{0}54'57''$East on Universal Transverse Mercator projection. The capital lays at the foot of Mount Entoto which is 3400 meters above sea level and extends south wards to its lowest point near to 2000 meters above sea level around Akaki i.e. south most edge of the city (Antene Zewdu,September 2015)

The road selected for this study is part of Megenagna to Haile Garemeent Ring road and it starts from megenagna and ends at Bole (refer figure 3.0).

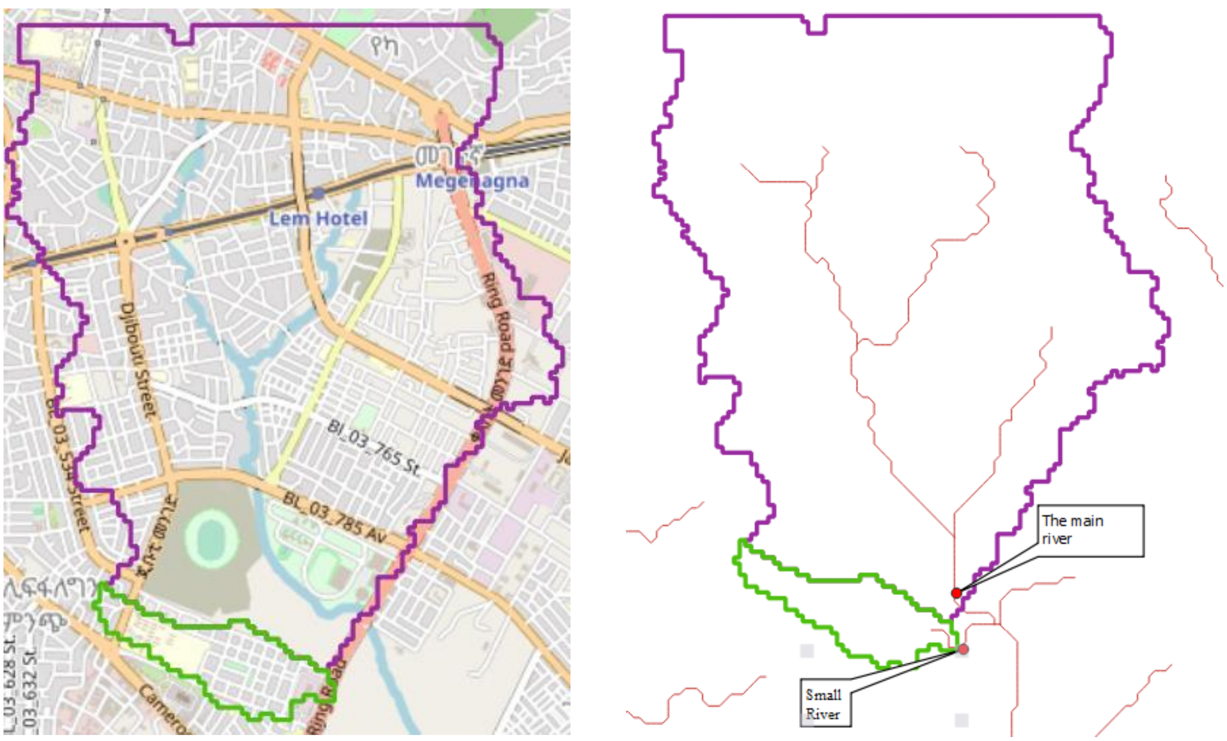



Figure 3.0 Location of the study area

**A. Cross drainage structure**

Megenagn to Bole road is 4.2km in length and it is crossed by two rivers, the first one is major river crossing around Imperial Round About and the second one is medium stream identified on Imperial Roundabout junction, both of the rivers photo as shown below, and there is no other minor crossing structure found on the reaming road part. (Refer figure 3.1)

<table>
<tr><td></td><td>Photo#1:- Around Imperial Stadium X:- 477800 Y:- 994762 around IMPERIAL roundabout</td></tr>
<tr><td></td><td>Photo#2:- Around Imperial Stadium X:- 467835 Y-993244</td></tr>
</table>

Figure 3.1 Bridges at Imperial and Ayat

## B. Strom drainage facility along the road

Through detail site investigation especially Imperial roundabout has high flooding problem due to unexpected flooding form Ambesa garage road Side and neighboring compound Area, in addition to this most of the inlet type around this section is grate inlet which is not well functioned, blocked shown below in figure 3.2 Beside all junctions has storm drainage facility has manmade problem and inlet type problem, lack of routine maintenance shown below

|  |  |
|---|---|
| **Fully closed Imperial Junction MH inlet Manmade problem** | **Un-functional Grate Inlet MH on Ring road Of Imperial Junction lack of maintenance** |

Figure 3.2 storm Drainage along the ring road

### 3.1.2. Climate

It well known that Addis Ababa is located around the equator its temperature stays nearly constant and no significant variation season to season. The average minimum and maximum temperature of each month is presented in the following table. The data is obtained from Ethiopian Meteorological Agency

Table 3.0 temperature of Addis Ababa

| | Jan | Feb | Mar | Apr | May | June | July | Aug | Sep | Oct | Nov | Dec |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Min | 8.3 | 9.5 | 10.9 | 11.4 | 11.3 | 10.8 | 11 | 11 | 10.2 | 8.3 | 8.1 | 7.8 |
| Max | 22.3 | 23.6 | 23.7 | 23.4 | 22.9 | 21.3 | 20 | 19.8 | 20.1 | 20.9 | 21.6 | 21.5 |
| Avg | 15.1 | 16.4 | 17.2 | 17.2 | 17 | 15.7 | 14.8 | 14.7 | 14.8 | 14.8 | 14.7 | 14.4 |

### 3.1.3 Rainfall

Based on the rainfall data retrieved from Addis Ababa Observatory Station, the mean annual rainfall of Addis Ababa city varies between 600mm - 1600mm. Rainfall data that has been obtained is referred to estimate the amount of rainfall of the city in general and that of the project areas in particular.

The mean monthly rainfall varies in the range of 8.0mm – 288.6mm with major rainfall records in the months of June, July, August and September and insignificant records from November to January.

Table 3.1: Average Monthly Rainfall Data of Addis Ababa Observatory Station

| Month | Jan | Feb | Mar | Apr | May | Jun | Jul | Aug | Sep | Oct | Nov | Dec | Year |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Rainfall (mm) | 14.5 | 34.9 | 65.9 | 90.2 | 85.6 | 134.1 | 267.4 | 288.6 | 168.9 | 35.0 | 8.7 | 8.0 | 1158.1 |
| *Source : National Metrological Agency of Ethiopia; Addis Ababa City Observatory Station (1981E.C – 2011E.C)* | | | | | | | | | | | | | |

According to the rainfall record, the mean annual rainfall measured is about 1158.1mm in which the areas receive about 74.2% of this rainfall from June to September. The following charts and tables show values and trends of measurements

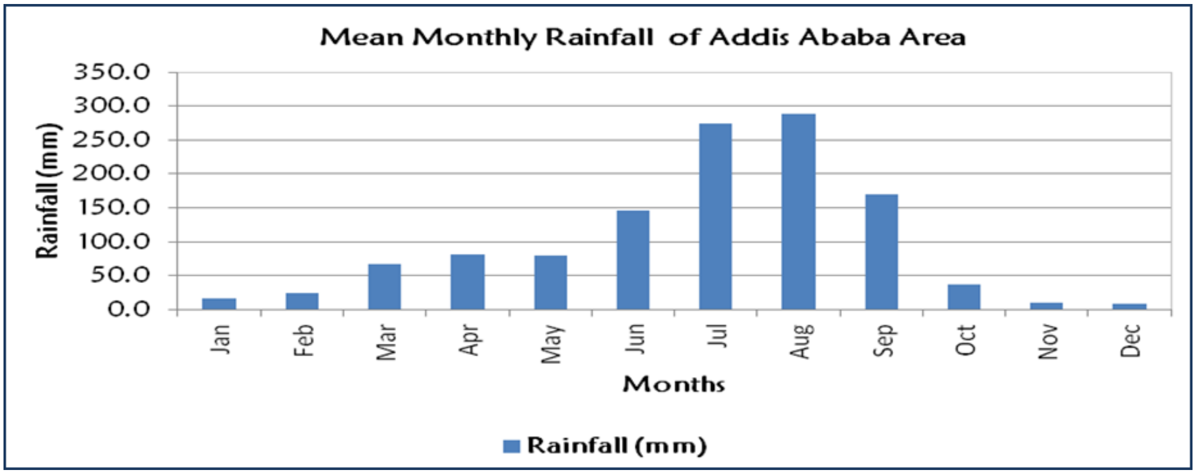



Figure 3.3 mean monthly rainfall of Addis Ababa

## 3.2 Data collection and analysis

### 3.2.1 Data collection

Different data has been collected such as rainfall, topographic maps, satellite imagery, and site findings (actual site data). The data were used for the Hydrology and hydraulics study.

### Rainfall data, topographic maps and other

Rainfalls, Satellite imaginary, geomorphology & soil, land use, topographic maps were collected.

- Soft copy of topography 1:50,000 topographic maps for the project road.
- 31 years of daily rain fall data at Addis Ababa station acquired from Ethiopia Metrological Agency
- Digital Elevation Model 30mX30m resolution.
- Geomorphology/soil and land use/cover of the project area were acquired.
- Surveying data (as built data )

### 3.2.2 Data analysis

#### 3.2.2.1 Hydrology analysis

The hydrological study has been undertaken in order to compute and evaluate peak discharges for all watercourses crossing the Ring Road. Calculation of these peak discharge values enabled the Evaluation of the existing hydraulic opening sizes and types of waterway required. The hydrological analysis was undertaken using available maps and digital Elevation model and together with the data acquired from Meteorological Services Agency. Additional ground information has been gathered from site visits and different sources.

#### 3.2.2.2 Rainfall frequency analysis

The major goal of frequency analysis of hydrologic data is to connect the magnitude of extreme events to their frequency of occurrence through the use of probability distributions (Chow, et al., 1988). It has been well known that for determining of Rain fall frequency analysis it should be required previous long-term rain fall data for the project area. In this study being used 31 years of daily rainfall from (1990-2020)

Any probability distribution can be used as the model however, the most well-known statistically analyses types are Generalized Extreme Value, Log Pearson-type III and Gumbel's Methods. Gumbel and Log Pearson Type III methods are used as suggested by Ethiopian Drainage Design Manual (ERA, 2013) so that the design rainfall frequency analysis for this study can be determined by log Pearson type III and Gumbles method's and the test of goodness of the data measured by the coefficient of determination ($R^2$), the greater value of $R^2$ have best distribution fitting.

#### 3.2.2.3 Estimating missing rainfall data

By various reason such as the absence of observer or instrumental problem rainfall data record infrequently are missed. In such case to estimate the missing data by using the nearest neighboring station rainfall data. There are various types of estimating missing rainfall data. Some of them are arithmetic means method, normal ration method, inverse distance weighing method and regression method. Arithmetic mean method can be used to fill the missing data when the normal annual precipitation is below or equal to 10% of the station which data are being recorded. The normal ration method is used when the normal annual precipitation any of index station different from that of precipitation station by more than 10%. (Adugna Ejeta,2019)

**Arithmetic mean method**

PX = (P1+P2+P3….PN)/N where Px = precipitation of the missed recorded

P1, P2, P3 and Pn are precipitation of the corresponding index station. N = number of index station In this study the missing data estimated directly has been taken the nearest neighboring station (i.e from kotebe rainfall station)

### 3.2.2.4 Design frequency or return Period

Design Frequency or return period is indicative of the frequency with which a certain magnitude of rainfall/runoff occurs in that period. The number of times a flood of a specified extent can be projected to occur on average over a long period. Design frequency can be expressed with probability. The probability of being equal or exceeded in any year can be defined by the following expression.

$$P(X \geq X_T) = \frac{1}{T}$$

Where: P = Frequency Exceedance.

T= Occurrence of design flood exceeded or equaled once (Return period), in years

Since it is not economically feasible to design a structure for the maximum runoff a watershed is capable of producing, a design frequency must be established according to the cost, potential flood hazard to property, expected level of service, and budgetary constraints, and considering the magnitude and risk associated with damages from inundation.

The recurrence interval values shown in the table below are used for hydraulic design of drainage facilities based on the ACCRA DDM recommendation.

Table 3-2: Design Return period selection based on Roadway Classification

| Roadway Classification | Exceedance Probability | Return Period |
|---|---|---|
| Urban Principal Arterial System | 4%-2% | 25-50 Year |
| Urban Minor Arterial System | 4% | 25 year |
| Urban Collector Street System | 10% | 10 year |
| Urban Local Street System | 20%-10% | 5-10 year |

In this study the road is categorized under the road classification of urban principal Arterial system so, the rainfall return period should be 25-50 year.in this case the return period to be 50 year.

Table 3.3: Design Return period selection based on type of structure

| Type of Structure | Design Flood Return Period (Years) | Check Review Flood (Years) |
|---|---|---|
| Drains in Residential Areas, Gutters and Inlets | 2 | 5 |
| Side Drains and Ditches | 10 | 25 |
| Culvert, Pipe Span <2 m | 10 | 25 |
| Culvert, 2m <Span < 6 m | 25 | 50 |
| Short Span Bridge, 6m < Span < 15 m | 50 | 100 |
| Medium Span Bridge, 15 < Span < 50 m | 50 | 100 |
| Long Span Bridge, Spans > 50 m | 100 | 200 |

The Drainage cross structure, which is exist around Imperial Round About, the width is 11m. Thus, based on the above table 3.3 the existing structure classified as short span bridge. Consequently, the design flood return period will be 50 years and check /review flood return period will be 100 years.

### 3.2.3 Methods of design flood computation

As per AACRA DDM and FHWA drainage Design manual -HEC-22 recommendations the design discharges are computed using

- Rational Method for catchments area equal to or less than 0.8 km2 and
- Soil Conservation Services (SCS) method for catchments area greater than 0.8 km2

In this study the design discharge computation carried out by U.S. Soil Conservation Services (SCS) graphical method and rational method depending on the size of catchments area as per ERA Drainage Design Manual and AACRA DDM recommendation finally verified by HEC HMS

Accordingly, Catchment Area 1 total area has 5.7888 km$^2$, the design discharge computed using soil conservation services (SCS) peak discharge method and catchment Area 2 has total area 0.306km$^2$, the design discharge computed by Rational method.

#### 3.2.3.1 Rainfall – runoff models

There are various recognized methods and approaches for hydrologic/hydraulic design of drainage facilities. The choice among the different methods depends on the availability of hydro-meteorological data required by those models and their appropriateness and applicability in the particular area of interest.

In areas like Ethiopia where there is scarcity of hydro-meteorological data, it is very difficult and often impossible to use most of the state-of-the art models found commercially. Therefore, it is essential to adapt some empirical formulas, which reasonably and safely enable to estimate the flood associated with the required return period.

For the purpose of flood estimation, a number of methods have been considered for further review as to their suitability and applicability. These are:

- Rational Method (RM)
- Soil Conservation Service (SCS) graphical method
- HEC HMS

### Phases of analysis of the study

The thesis will focus on evaluation of Ring road drainage structures on Megenagna to Bole. Thus, the data's gathered and analyzed critically on the above section of this chapter, The main steps that are used to solve the specific objectives of this study are;

- ✓ Gather and Check the quality of data
- ✓ Design rainfall analysis is carried out using Log Pearson type III ,it has higher R2 value
- ✓ Delineate water shed using DEM and Q GIS
- ✓ Develop current Land use (land cover) of the area using QGIS
- ✓ Download Soil Map for the study area from w,w,w FAO /UNESCO Soil map of the world/Africa/Ethiopia, then extract the soil map of the area using QGIS
- ✓ Compute curve number for the study area.
- ✓ To determine time of concentration
- ✓ To compute peak runoff Estimation of (using SCS method, Rational method based on the catchment area size)
- ✓ Re compute (verify) the peak runoff using HEC HMS Model
- ✓ To Evaluate the hydraulic performance of the existing drainage structure using HEC RAS computer modeling
- ✓ Determination of appropriate sizes of waterway opening for watercourses

# Chapter four

## 4. Result and discussion

## 4.1 Hydrology analysis for Catchment area1

### 4.1.1 Catchment area delineation

Before extraction of the watershed of the studied area, it has been done detail investigation through site reconnaissance and desk investigation.

- Site reconnaissance: -the main goal of site investigation is to identify the ground condition, which have a direct or an indirect impact on the project development.it creates better understanding of the site by looking around the whole corridor of the study area, thus it would be provide the safe and the most economical design for the project
- Desk investigation: is an office work which is including to collect and review of information regarding to the project. For the case of delineation of the catchment area on this study used topographic maps, open street maps, satellite maps and contour maps are source of desk investigation

The catchment areas of watershed on the whole route corridor including stream slope, stream length were delineated from DEM data of 30mx30m resolution using QGIS 3.1.6-3.18 which is an open source software's. The topographic satellite DEM data 30mx30m resolution, which is directly obtain from USGS (w.w.w. earthexplorer.usgs.gov)

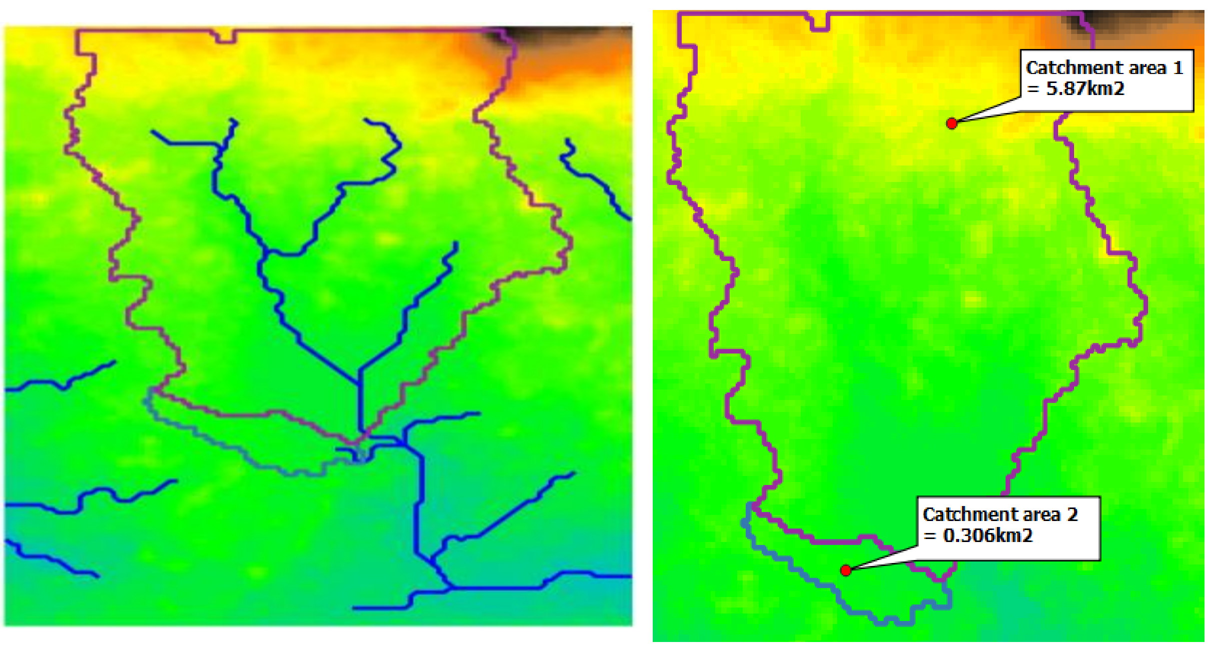



Figure 4.0 Catchment of the study area

### 4.1.2 SCS Graphical peak discharge method

The graphical (peak discharge) method used for determining of peak rate of runoff, the peak discharge method was formulated from hydrograph analysis using TR 20 “Computer Program for Project Formulation—Hydrology” (SCS 1983)

When using SCS graphical method for determining of peak rate of runoff, if the calculated value of Ia/P is fall outside the range, then the limiting value should be used. In connection to this the ratios fall in between, use linear interpolations (TR-55urbun hydrology for small watershed)

Based on ERA DDM the following procedure outlines the SCS graphical method for estimating peak discharge.

- Determine the watershed area in hectares (km2);
- Determine the time of concentration, with consideration for future characteristics of the watershed;
- Determine the soil type, soil group, land use and curve number of the watershed area; determine the hydrologic region, check the AMC and convert the CN value if required to wet or dry condition;
- Determine the 24hr rainfall depth and calculate the Ia/p ratio;
- According to Technical Release 55 Urban Hydrology for Small Watersheds the peak discharge of SCS graphical method qp = qu*A*Pe *F
- qu = $\boldsymbol{\alpha} * \mathbf{10}^{\boldsymbol{co+c1logtc+c2}(\boldsymbol{logtc})^2}$

#### 4.1.2.1 Land use/land cover

As per Ethiopia Land use and Land Cover Data, Addis Ababa and its neighboring areas land cover dramatically changed from day to day due to different activity on urbanization,
Land use of the City and Its Watershed Area is highly covered With Residential, Commercial building, Recreational places including Park and Vegetation cover.(see figure 4.1)
In connection to this, SCS uses a combination of and land-use (ground cover) and soil conditions to assign a runoff factor to an area. These runoff factors, called runoff curve numbers (CN), indicate the runoff potential of an area. If growing of urbanization, all of the natural ground cover become impervious and the water not infiltrates through the impervious area, this effect to rise the surface runoff .The current land cover of the study area analyses is carried out using Qgis3.18 open source software, after it has been obtained of Ethiopian current land cover from (w.w.w. geo fabric.de)

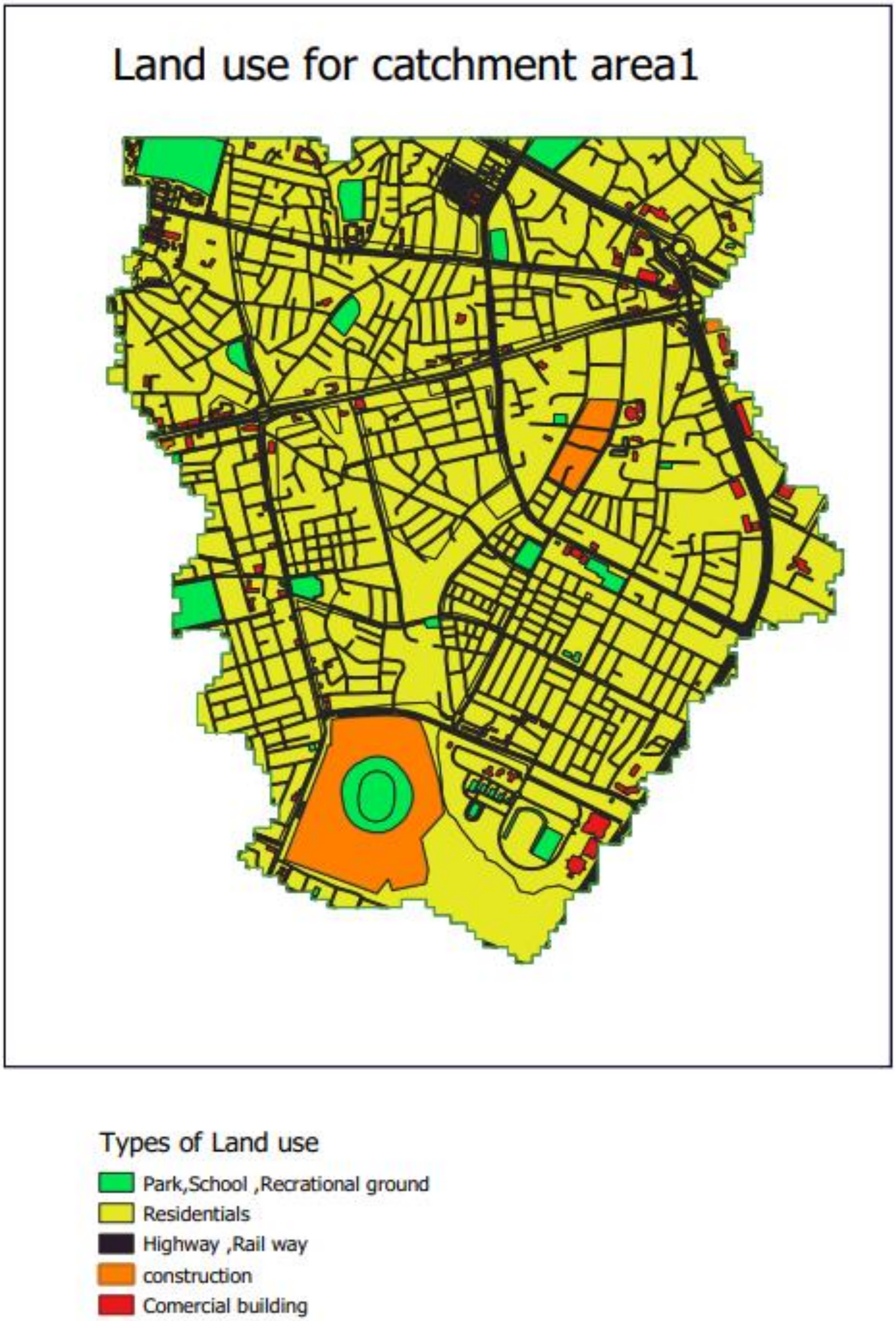



Figure 4.1 Land use for catchment area 1

### 4.1.2.2 Impervious Surface for residential area

In general principle the runoff is directly connected to the land cover of the area, if the land cover is highly built-up area (impervious), all forms precipitation to flow overland as runoff instead of infiltrating into the ground (Shuster, et al., 2005; SCS, 1986; Brody, et al., 2007; Schuler, et al., 2009)

In numerous scientific journals the impervious surface presented as either total impervious surface (TIA) or effective impervious surface (EIA).

TIA is usually presented as a percent of impervious area from total land area. TIA is agood indicator of percent impervious for the specified area, however TIA is not sufficiently account for the impervious area that are completely isolated from drainage networks. Thus, TIA may not directly contribute to increasing surface runoff volume, if the areas are isolated to the drainage systems. (for instance, a precipitation that fall in a roof draining only on a natural landscape area for infiltration, so that the precipitation which fall on roof not being a runoff),

Effective impervious surface (EIA): An effective impervious surface (EIA) is directly hydraulically connected to a drainage system, and thus the precipitation fall on the surface area can contribute to the total runoff volume of the drainage system. On the other hand, an ineffective impervious surface is placed to be hydraulically detached from a drainage system and may not contribute to the total area runoff volume.

Data is aback bone of the research study, which enables to answer the research questions. There is different sampling technique for data collection among those, this study has been used cluster sampling method. Cluster sampling is the whole study area is classified into groups, then subsequently a random sample is taken from these clusters(wilison,2010)

In this study the total sample area divided into five major groups in the study area (i.e North, South, East, West and Central) of the study area, from these groups, the lots size area selects randomly from this clustered groups to estimate the total percent of impervious area for the residential land use.

Table 4.0 Total impervious for residential area

| Sample Lot N.o | Total area | Impervious area (TIA) | Pervious area | % impervious |
|---|---|---|---|---|
| 1 | 5,018 | 2,547 | 2,471 | 50.76 |
| 2 | 7,961 | 4,960 | 3,001 | 62.30 |
| 3 | 9,022 | 8,342 | 680 | 92.46 |
| 4 | 21,564 | 10,238 | 11,326 | 47.48 |
| 5 | 7,576 | 1,805 | 5,771 | 23.83 |
| Total | 51,141 | 27,892 | 23,249 | 54.54 |

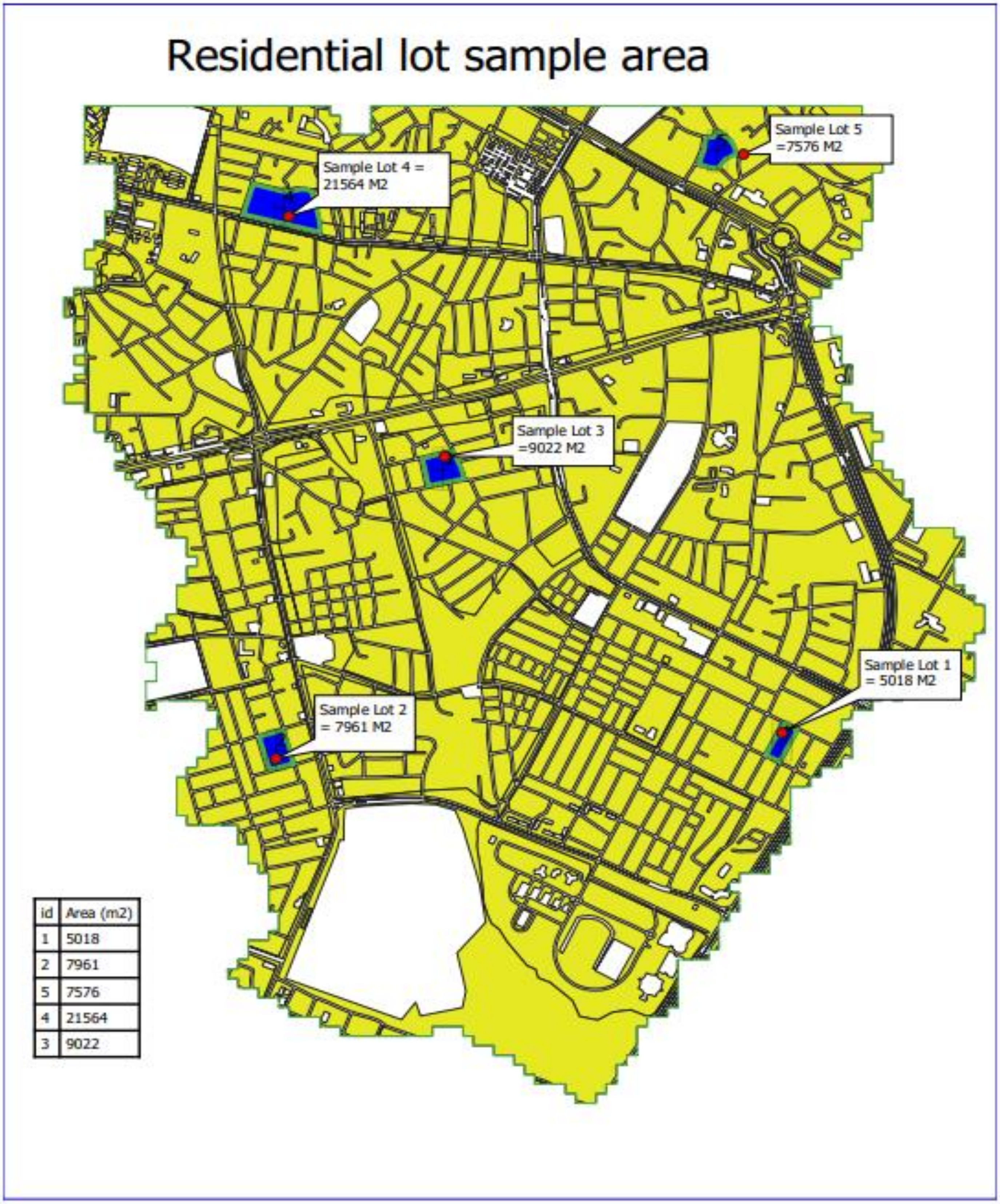



Figure 4.2 Samples lots sizes for residential

### 4.1.2.3 Hydrologic soil groups

Soil properties influence the relationship between rainfall and runoff by affecting the rate of infiltration (ERA,2013). The SCS has divided soils into four hydrologic soil groups based on infiltration rates (Groups A, B, C, and D). In Ethiopia, 17 major soil units have been identified (EMA,1988). The type of soil at the study area is Eutric nitosols ,this types of soils classified under the group of soli class B (refer Table 2.2 Typical Hydrologic Soils Groups for Ethiopia)

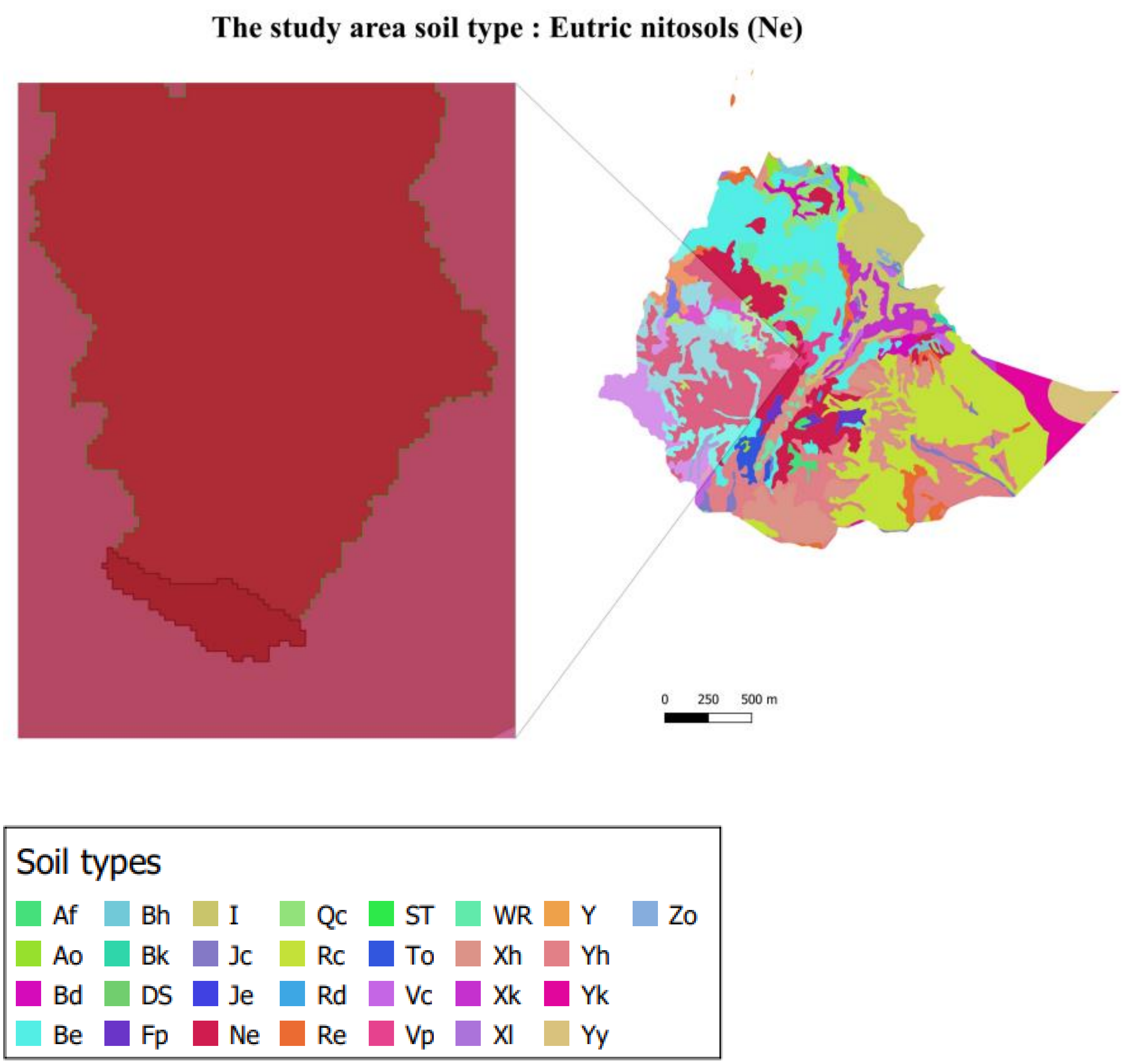



Figure 4.3 soil types for the study area

### 4.1.2.4 Curve number

The curve number is dimensionless parameter indicating the runoff response characteristics of a drainage basin. In the curve number approach, the runoff is connected to land use, land treatment, hydrological condition, and hydrological soil group and antecedent soil moisture condition in the catchment area.

To determine the appropriate CN value, various tables can be used. Firstly, there are tables (table 2.1) relating the value of CN to land use, hydrological conditions and hydrological soil group. However the table 2.1 fails to assign the curve number CN for residential land use type due to the table were formulated for typical land cover interactions based on particular assumed percentage of impervious area. So that for this study To determine the CN for residential land use type by computing the composite CN, the following assumptions are made for calculating the composite CN

- All pervious urban surfaces are the same as to grassland with in good hydrological conditions , the soil group of the study area is classified under group B ,thus the CN equal to 61
- All impervious surfaces have a CN of 98 and all the impervious surface areas are directly connected to the drainage system

Therefore the composite CN for the residential determined as shown below

Table 4.1 composite CN for residential area

| N.o | Type | Curve number | Area | Curve number *Area |
|---|---|---|---|---|
| 1 | Impervious | 98 | 27,892 | 2,733,416 |
| 2 | Pervious | 61 | 23,249 | 1,418,189 |
| | Sum | | | 4,151,605 |
| Representative curve number for residential area | | | | 81 |

To estimate the curve number for particular for this study area taking the following procedure

- Assign a hydrological soil group to each of the soil units found in the drainage area
  The type of soil at the study area is Eutric nitosols, this type of soil classified under the group of soli class B (refer table 2.2)
- Prepare land use map for the catchment area A1 as shown figure
- Calculate the weighted average CN value according to the areas they represent and the average (representative) CN for the study area

Table 4.2 Average curve number of Area 1

| Type of land use | curve number | Area (m2) | Area* CN | Representative curve number of the area |
|---|---|---|---|---|
| Commercial | 92 | 142,622 | 13,121,224 | 84 |
| Construction | 86 | 267,250 | 22,983,500 | |
| Recreation ground (fair condition) | 69 | 208,194 | 14,365,386 | |
| Residential | 81 | 4,379,788 | 355,549,359 | |
| Road | 98 | 881,036 | 86,341,528 | |
| Total area (m2) | | 5,878,890 | 492,360,997 | |

- The average curve number should be changed into wet or dry condition according to the rainfall region based on the table below. The rainfall region for the study area fall under wet region the CN to be 93. see the rainfall distribution under the appendix

Table 4.3 Conversion table average AMC into Dry and wet (source Soil Conservation Service 1972)

| CN AMC II (Average curve number) | CN AMC I (dry conditions) | CN AMC III (Wet conditions) | CN AMC II (Average curve number) | CN AMC I (dry conditions | CN AMC III (Wet conditions) |
|---|---|---|---|---|---|
| 100 | 100 | 100 | 58 | 38 | 76 |
| 98 | 94 | 99 | 56 | 36 | 75 |
| 96 | 89 | 99 | 54 | 34 | 73 |
| 94 | 85 | 98 | 52 | 32 | 71 |
| 92 | 81 | 97 | 50 | 31 | 70 |
| 90 | 78 | 96 | 48 | 29 | 68 |
| 88 | 75 | 95 | 46 | 27 | 66 |
| 86 | 72 | 94 | 44 | 25 | 64 |
| 84 | 68 | 93 | 42 | 24 | 62 |
| 82 | 66 | 92 | 40 | 22 | 60 |
| 80 | 63 | 91 | 38 | 21 | 58 |
| 78 | 60 | 9 | 36 | 19 | 56 |
| 76 | 58 | 89 | 34 | 18 | 54 |
| 74 | 55 | 88 | 32 | 16 | 52 |
| 72 | 53 | 86 | 30 | 15 | 50 |
| 70 | 51 | 85 | 25 | 12 | 43 |
| 68 | 48 | 84 | 20 | 9 | 37 |
| 66 | 46 | 82 | 15 | 6 | 30 |
| 64 | 44 | 81 | 10 | 4 | 22 |
| 62 | 42 | 79 | 5 | 2 | 13 |
| 60 | 40 | 78 | 0 | 0 | 0 |

Based on the result, the Average curve number for the study area 84, according to ERA DDM the study area (Addis Ababa) fall under the area located in WET region of the country. Thus, the average curve number changed to wet region as per the ERA DDM, so that the catchment area 1 CN= 93.

#### 4.1.2.5 Time of concentration

The rainfall intensity used in the rational method is determined from the time of concentration (Tc). Tc is defined as the time required surface runoff water to flow hydraulically from the remotest point of the catchment to the point of exit.

ERA DDM 2013 recommends to use USGS method to determine time of concentration, According to USGS the time of concentration for over land flow and defined water coarse flow computed as follow.

**Time of concentration for over land flow**

Overland flow travel time is combination flow of sheet flow travel time and concentrated flow travel time. The overland flow is occurring after completion of saturation of crust. Overland flow occurs in small, flat drainage area, where there is no clearly defined watercourse. The runoff becomes thin layers flowing slowly over the fairly ground surface. ERA DDM 2013 is suggested to use the Kerby formula for the calculation of Tc when the drainage area slope is fairly even

According to ERA-DDM 2013 Overland flow paths to be in less of 60 m in urban areas and 120 m in rural areas, in relation to this the maximum sheet flow length in TR-55 was reduced from 300 feet to 100 feet in the recently developed Windows TR-55 software system. (USDA, NRCS, National Water and Climate Center, 2001) This is therefore, it is reasonable to use 30m for overland flow length.

$$Tc = \left(\frac{2.2 * n * l}{s^{0.5}}\right)^{0.324}$$

L = hydraulic length of catchment, measured along flow path from the upper catchment boundary the water flow is concentrated to before entrance of the main channel (feet) = 98.425 feet = 30m

Manning roughness coefficient for overland flow = 0.011 (see the table below)

S = Slope of the catchment = $\frac{\Delta H}{1000L}$ =0.00067

Over land $T_c$ = 0.10123 hr

The selection of manning's roughness is based on the overall catchment area surface description, the study area more of comprised by imperviousness surface so that it is very reasonable to use manning's coefficient (n) = 0.011

Table 4.4 Roughness coefficients (manning's n) for overland flow

**Table RO-6: Roughness Coefficients (Manning's $n$) for Overland Flow (USDA NRCS – TR-55 1986)**

| Surface Description | $n$[1] |
|---|---|
| Smooth surfaces (concrete, asphalt, | |
| gravel, or bare soil) . . . . . . . . | 0.011 |
| Fallow (no residue) . . . . . . . . . . . . . | 0.05 |
| Cultivated Soils: | |
| Residue cover ≤ 20% . . . . . . | 0.06 |
| Residue cover > 20% . . . . . . | 0.17 |
| Grass: | |
| Short grass prairie . . . . . . . . | 0.15 |
| Dense grasses[2] . . . . . . . . . | 0.24 |
| Bermuda grass . . . . . . . . . . | 0.41 |
| Range (natural) . . . . . . . . . . . . . . . . | 0.13 |
| Woods:[3] | |
| Light underbrush . . . . . . . . . | 0.40 |
| Dense underbrush . . . . . . . . | 0.80 |

## Time of concentration for defined water coarse flow

L = hydraulic length of catchments, measured along flow path from the catchment boundary to the point where the flood needs to be determined (km) = 3.396 km

$$T_c = \left(\frac{0.87L^2}{1000S_{av}}\right)^{0.385}$$

**Average slope**: - In this study the average slope for the longest flow path computed by 10-85 (USGS recommendations) from the outlet

Sav = average slope (m/m) $= \frac{H85 - H10}{1000 * 0.75 * L}$

H 0.10L = elevation height at 10% of the length of the watercourse (m) = 2351m

H 0.85L = elevation height at 85% of the length of the watercourse (m) = 2322.05m

Sav = average slope (m/m) = 0.01178

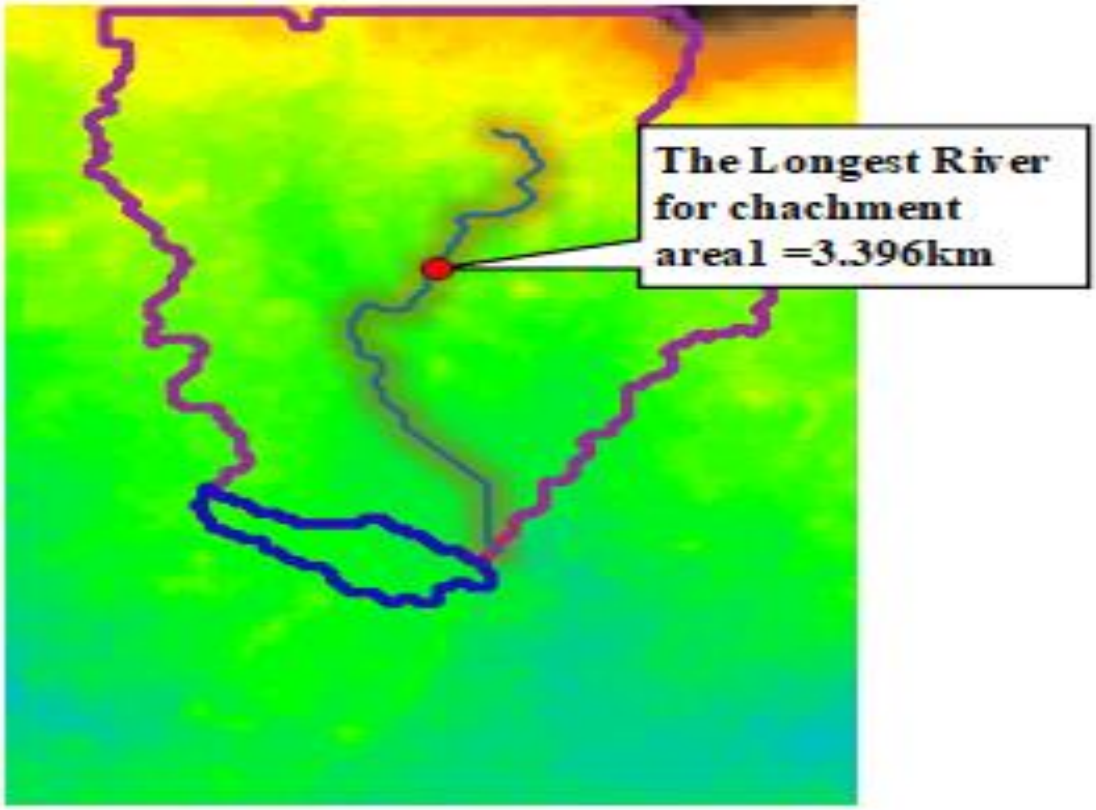



**Figure 4.4 the longest river for Catchment Area1**

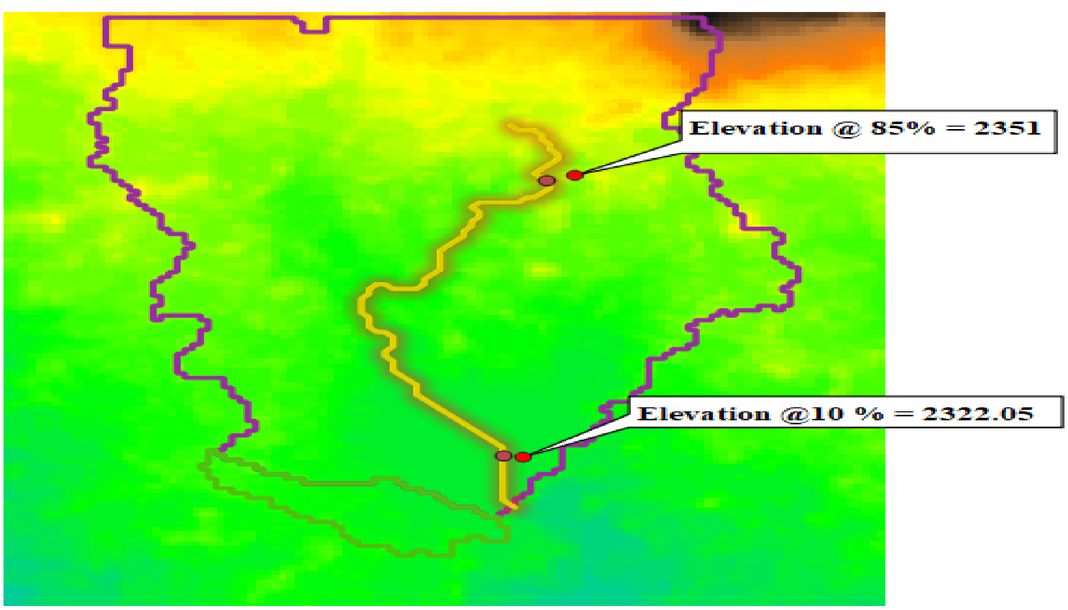
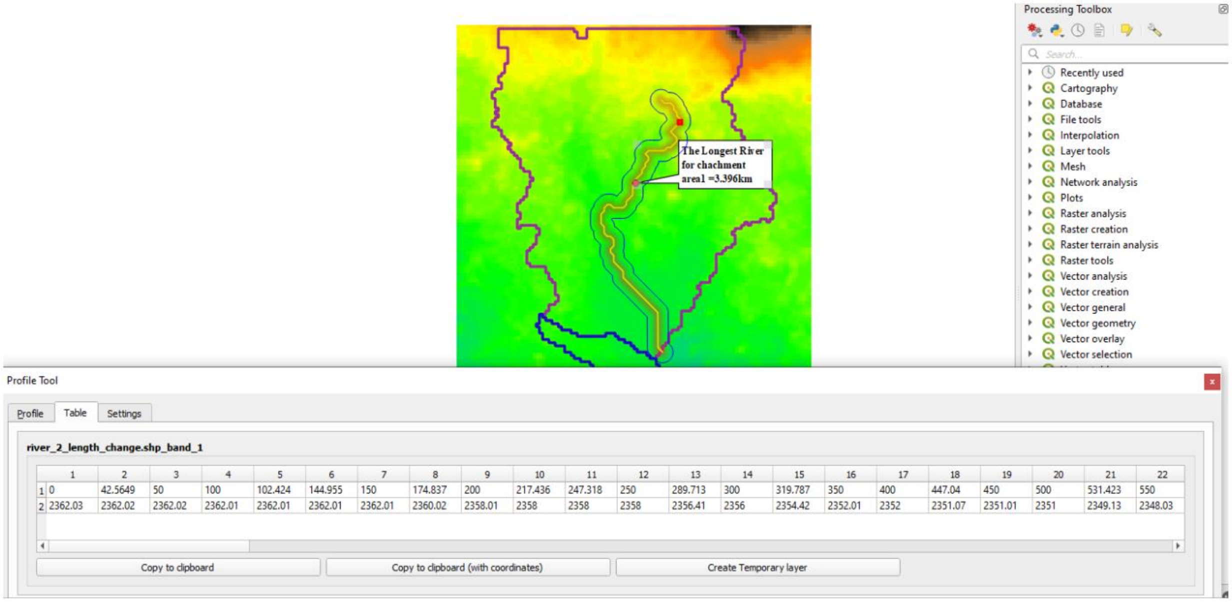



Figure 4.5 Elevation at 10-85% for River1




| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 | 21 | 22 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 0 | 42.5649 | 50 | 100 | 102.424 | 144.955 | 150 | 174.837 | 200 | 217.436 | 247.318 | 250 | 289.713 | 300 | 319.787 | 350 | 400 | 447.04 | 450 | 500 | 531.423 | 550 |
| 2 | 2362.03 | 2362.02 | 2362.02 | 2362.01 | 2362.01 | 2362.01 | 2362.01 | 2360.02 | 2358.01 | 2358 | 2358 | 2358 | 2356.41 | 2356 | 2354.42 | 2352.01 | 2352 | 2351.07 | 2351.01 | 2351 | 2349.13 | 2348.03 |

Figure 4.6 Station vs elevation for river1

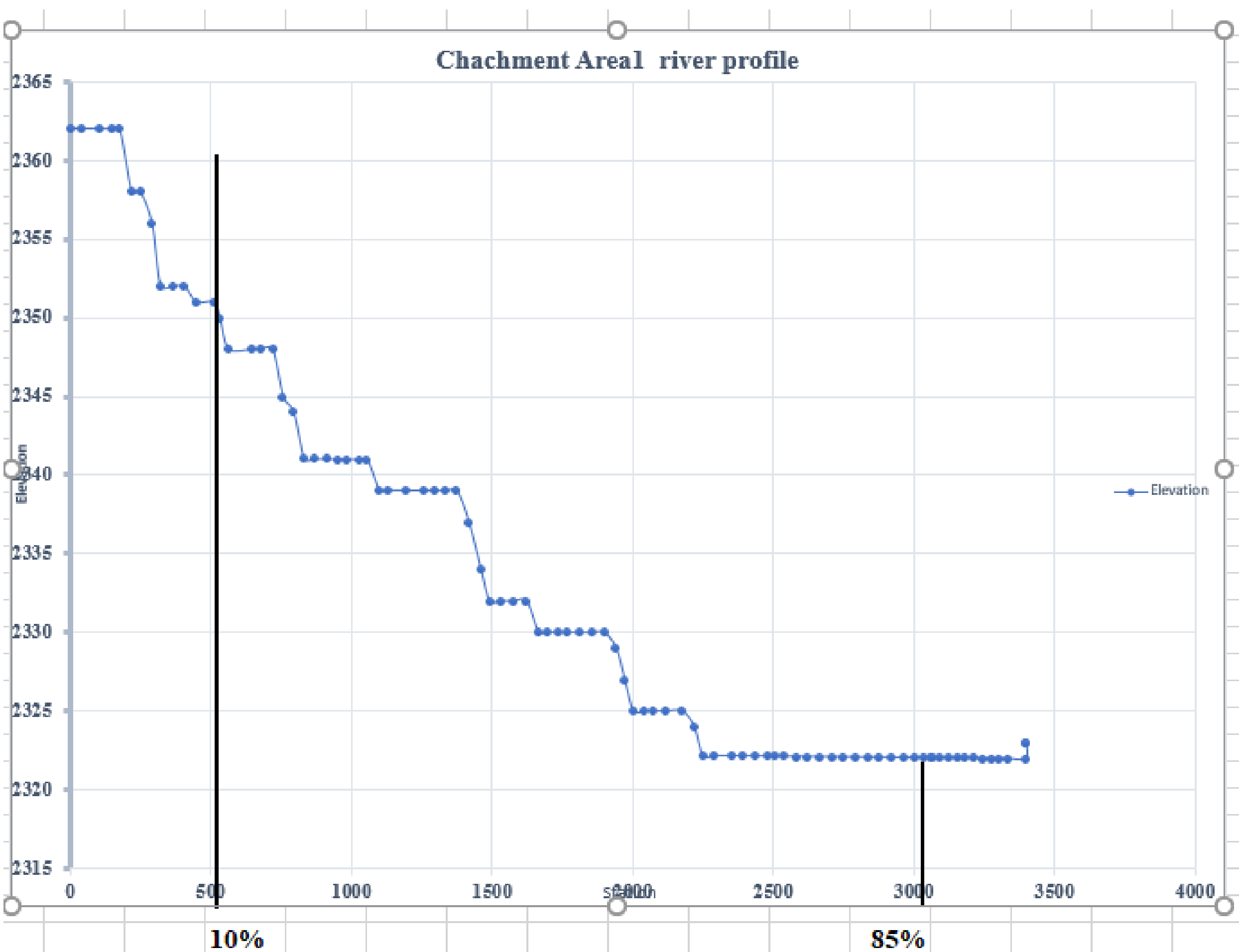



Figure 4.7 Elevation profile for river1

Tc = time of concentration (hours) for defined water coarse

Tc =$\left(\frac{0.872*L^2}{1000*Sav}\right)^{0.385}$ = 0.94hr

Total time of concentration =T.C overland flow + T.C defined water coarse = 1.01hr

#### 4.1.3 The design rainfall (P)

A number of numerous methods are employed to estimate the maximum precipitation, among these approaches ERA DDM 2013 recommended to use Log Pearson type III and Gumbels methods. Accordingly, the design rain fall prediction was done by using both Gumbel and Log Pearson III method , the data has been analyzed by Microsoft excel for forecasting the maximum and the goodness of test of the data.

##### 4.1.3.1 Test of goodness

Probability distribution fitting has been done using Gumbles and log Pearson type III using the previous 31 years of recorded data, then the test of goodness of the data carried out by coefficient of determination test ($R^2$ ), using the relation between the observed and the predicted precipitations. The result of $R^2$ for log Pearson type III is 0.875 and Gumbles 0.764 (refer appendix), thus based on the result of coefficient of determination test ($R^2$), The Log Pearson type III was selected for further analysis. The value the predicted rainfall (using Log Pearson type III) and the Ia/p with different return period for a given Ia = 6.93 shown in table 4.5

Table 4.5 Design rainfall with return period

| Return period | 2 | 5 | 10 | 25 | 50 | 100 |
|---|---|---|---|---|---|---|
| P | 42.02 | 59.78 | 77.91 | 110.32 | 143.29 | 185.96 |
| Ia/p | 0.09 | 0.06 | 0.05 | 0.03 | 0.03 | 0.02 |

##### 4.1.3.2 Excess rainfall (Pe)

The peak Run off determination has been doing by using the SCS method, on the basis of SCS method the total precipitation P is divided into three components Initial abstraction Ia, Continuous abstraction Fa and excess rainfall Pe

Initial abstraction is part of rainfall that is used for filling surface depressions, It is determined by as 20% of the total maximum retention (S) of the catchment area. Thus, Ia = 0.2S.

Continuous abstraction (Fa ) :-After initial abstraction is fulfilled, the other part of rainfall amount is used to percolate as continuous abstraction (Fa). The technique to calculate Fa by using the total amount of rainfall (P) and the maximum retention (S), the continuous abstraction is given by the following equation:

$$Fa = S*\frac{(P-Ia)}{(P-Ia+s)}$$

So that the accumulated excess rainfall Pe is expressed by the following equation

Pe = P-Ia-Fa

Pe = accumulated excess rainfall

P = total amount of rain fall

Ia= initial abstraction including surface storage, interception, and infiltration prior to runoff, mm

S = potential maximum retention, mm

**Ia=0.2*S**

$$\mathbf{S=\frac{25400}{CN}-254}$$

CN= Curve Number (i.e calculated earlier) CN = 93

Therefore

$$\mathbf{S = \frac{25400}{CN}-254 = 34.6}$$

**Ia=0.2*S = 6.93**

Table 4.6 Excess rainfall

| Return period | 2 | 5 | 10 | 25 | 50 | 100 |
|---|---|---|---|---|---|---|
| P | 42.02 | 59.78 | 77.91 | 110.32 | 143.29 | 185.96 |
| Ia/p | 0.09 | 0.06 | 0.05 | 0.03 | 0.03 | 0.02 |
| **Fa = S*((P-Ia))/((P-Ia+s))** | **12.74** | **14.25** | **15.20** | **16.21** | **16.81** | **17.30** |
| **Pe=P-Ia-Fa** | **25.46** | **41.71** | **58.89** | **90.29** | **122.65** | **164.83** |

#### 4.1.3.3 Unit peak discharge

Unit peak discharge is the unit discharge of drainage area for one millimeter depth of excess rainfall. In most case the unit peak discharge directly read from the corresponding time of concentration in the rainfall distribution graph (see figure 4.8 ). However, this study used the empirical method, which is an exponential function derived from using time of concentrations and the regression coefficients. SCS Regression coefficients directly read from table 4.8, which is presented with different types of rainfall distributions

qu = $\alpha * 10^{co+c1logtc+c2(logtc)^2}$

qu = unit peak discharge

C0, C1, C2 = regression cofficent

**α = 0.00431 (SI unit conversion factor)**

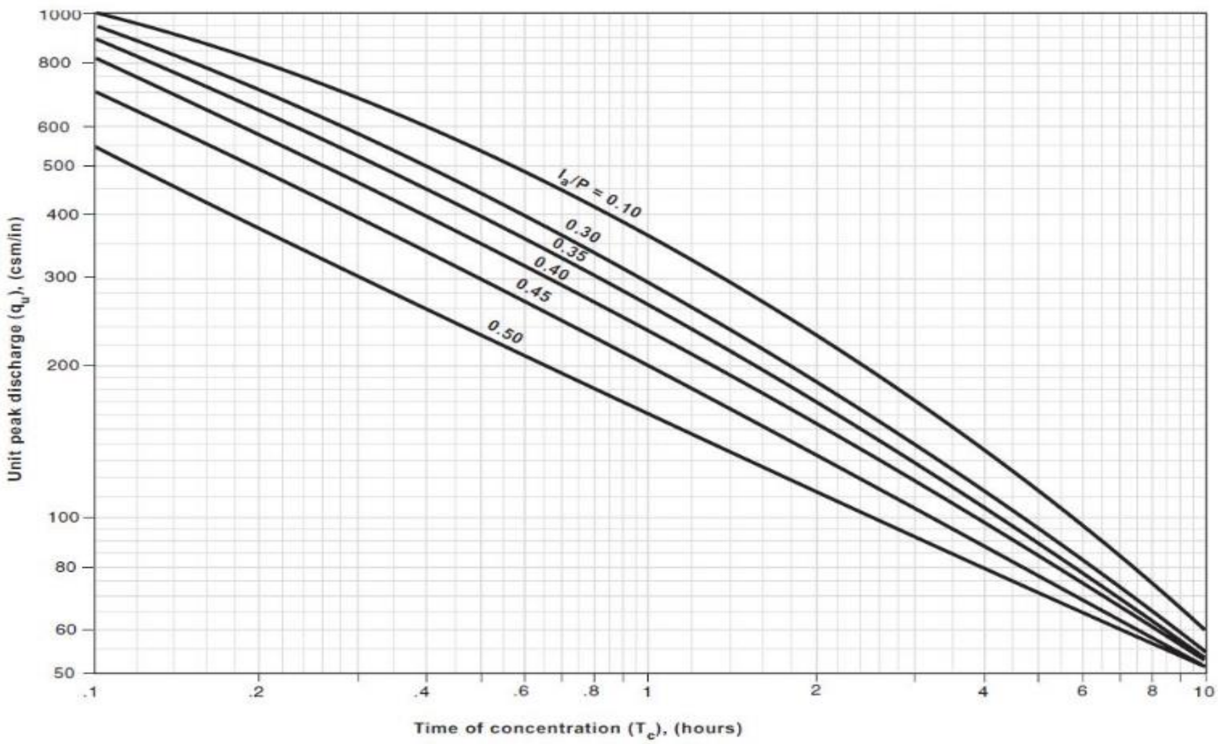



Figure 4.8 TR55 TypeII rainfall distribution unit peak discharge vs time of concentration

For determination the unit peak discharge, first it should be select appropriate NRCS rainfall distribution types, in this study type II rainfall distribution used. The other important factor to determine the unit peak discharge is the ratio of Ia/p Parameter. The Ia/P parameter is tabulated with different return period see the table 4.7

Table 4.7 Ia/P parameter

| Return period | 2 | 5 | 10 | 25 | 50 | 100 |
|---|---|---|---|---|---|---|
| P | 42.02 | 59.78 | 77.91 | 110.32 | 143.29 | 185.96 |
| Ia/p | 0.09 | 0.06 | 0.05 | 0.03 | 0.03 | 0.02 |

The ratio of Ia/P that shown in the table 4.7 is out of the range. The range of Ia/P from 0.1 up to 0.5 see figure 4.8 TR 55 Type II rainfall distribution and table 4.8.

However, based on TR55 urban hydrology recommendations the value of Ia/P uses the limiting value. For type II rainfall distributions the limiting value of Ia/p with respective value of regression coefficient as show in table 4.8 marked as in yellow color

Table 4.8 regression coefficient

| Rainfall type II | | | |
|---|---|---|---|
| Ia/P | C1 | C2 | C3 |
| 0.1 | 2.55323 | -0.6151 | -0.164 |
| 0.3 | 2.46532 | -0.6226 | -0.1166 |
| 0.35 | 2.41896 | -0.6159 | -0.0882 |
| 0.4 | 2.36409 | -0.5986 | -0.0562 |
| 0.45 | 2.29238 | -0.5701 | -0.0228 |
| 0.5 | 2.20282 | -0.516 | -0.0126 |

### 4.1.4 Determination of peak flood for catchment area 1

After detail investigation of rainfall analysis, watershed delineation from DEM using Q GIS, identification of land cover, determination of curve number, computation of time of concentration and finding of unit peak discharge using rainfall distribution type II, finally concluded by computation of peak discharge in the catchment area 1

The following equation were used for the estimation of the peak discharge in SCS graphical method

qp = qu*A*Pe

Where qp = peak discharge, m3/s

A = catchment area ( $km^2$)

Pe (Q) = depth of runoff

qu = $\alpha * 10^{co+c1logtc+c2(logtc)^2}$

qu = unit peak discharge, m3/s/km2/mm

Where Co, C1 and C2 = regression coefficients

**α = 0.00431 (SI unit conversion factor)**

**Tc =1.05 hr**
**A =5.87889 km2**

The computed peak discharge with different return period presented in table 4.9

Table 4.9 peak discharge for different return period

| Return period | Ia/P | P | Co | C1 | C2 | Q | qu=α*10^(C0+C1logtc+c2*(logtc)²) | qp =qu*A*Q |
|---|---|---|---|---|---|---|---|---|
| 2 | 0.09 | 42.02 | 2.55 | -0.62 | -0.16 | 25.46 | 0.15 | 23 |
| 5 | 0.06 | 59.78 | 2.55 | -0.62 | -0.16 | 41.71 | 0.15 | 37 |
| 10 | 0.05 | 77.91 | 2.55 | -0.62 | -0.16 | 58.89 | 0.15 | 53 |
| 25 | 0.03 | 110.32 | 2.55 | -0.62 | -0.16 | 90.29 | 0.15 | 81 |
| 50 | 0.03 | 143.29 | 2.55 | -0.62 | -0.16 | 122.65 | 0.15 | 110 |
| 100 | 0.02 | 185.96 | 2.55 | -0.62 | -0.16 | 164.83 | 0.15 | 148 |

### 4.1.3 Verifying the peak discharge by HEC HMS

HEC –HMS is a software formulated by the United States Army Corps of Engineers, in order to compute the rate of runoff at the interest points. the program execute hydrologic simulation such as surface runoff in the basin, determination of discharge in the channel and dam flood flows etc.

In hydrology the law of conservation of mass worked, that is the total incoming precipitations under the catchment area is the sum of infiltration of water through the soil and the excess precipitations left on the surface. In HEC HMS the actual infiltration determined by loss method, under this program there are twelve different types of loss method, among these this study has been selected the SCS curve number method, which is an incremental loses for event simulations.

Initial abstraction (Ia) is the amount of precipitations that must fall before excess precipitations, HEC HMS automatically perform the initial abstraction

The curve number should be a composite curve number otherwise determine the curve numbers for each sub basin. The composite curve number that has been used in HEC HMS model is calculated in the previous section for further information see the table

The actual excess rainfall transformed into runoff by transform method. From a total of nine different transform method this study uses SCS unit hydrograph method. Using the standard unit hydrograph, the peak flow occurred at 37.5% of the unit runoff, due to this the peak rate of factor PRF will be 484.

The flood routing is carried out by Muskingum. The design rainfall familiarized into the HEC HMS BY meteorological model. In this case, the runoff will be estimated for the 2, 5, 10, 25, 50 and 100 year return period.

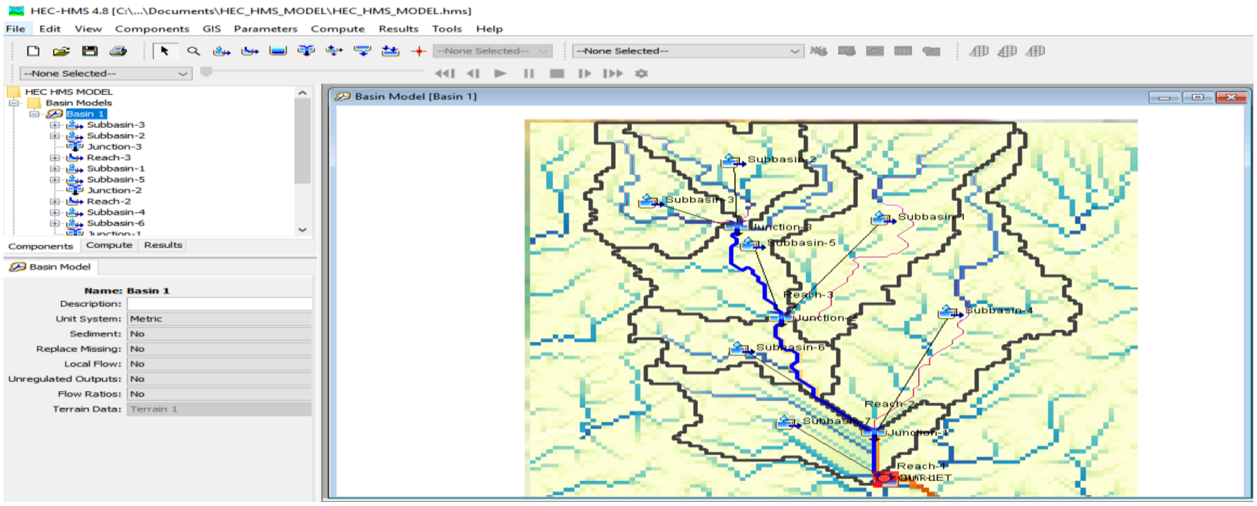



Figure 4.9 sub basin in HEC HMS

## A. Sub basin Lag time

HEC HMS automatically divide the watershed area into reasonable sub basins, in this case the there are seven sub basins under the watershed area.

For each sub basin computed the lag time, which is very critical for hydrological simulation of the model see the table below.

Table 4.10 sub basin lag time

| Sub basin | Longest flow path length (km) | Longest flow path slope | Basin slope | Tc= $L^{0.8}\frac{(\frac{1000}{CN}-9)^{0.7}}{441Y^{0.5}}$ | D/2 =0.1333333*Tc | Lag time=0.6*Tc+D/2 |
|---|---|---|---|---|---|---|
| Subbasin-3 | 1773.82 | 0.02255 | 0.06651 | 31 | 4 | 23 |
| Subbasin-2 | 1559.74 | 0.03013 | 0.08607 | 25 | 3 | 18 |
| Subbasin-1 | 2910.99 | 0.05358 | 0.09743 | 38 | 5 | 28 |
| Subbasin-5 | 2487.79 | 0.02612 | 0.07971 | 37 | 5 | 27 |
| Subbasin-4 | 3236.91 | 0.05126 | 0.08255 | 45 | 6 | 33 |
| Subbasin-6 | 1918.86 | 0.01716 | 0.06836 | 33 | 4 | 24 |
| Subbasin-7 | 1808.09 | 0.0188 | 0.04202 | 40 | 5 | 29 |

## B. Percent of Impervious

The impervious area is highly inter related to the runoff potential of the drainage area. Impervious surfaces, and other urbanization development, reduce the penetration of precipitation into the earth. Accordingly to Coastal Change Analysis Program (C-CAP) studied by NOHA the land cover classified into four major groups based on the percent impervious surface.

The C-CAP land use classification pattern comprises four groups that are defined by variances in the density of built up or impervious surfaces area within each lots. The C-CAP suggests to use these developed classes were scaled by class specific impervious surface coefficients. Based on types of land cover the percent of impervious coefficient as follows

Table 4.11 C-CAP impervious percent of coefficient

| Types of land cover (C-CAP Class Name) | Descriptions | Impervious percent of coefficient |
|---|---|---|
| Developed ,High intensity | The built-up areas account for 80 up to 100 percent of the total cover, this class includes heavily built-up urban centers and large constructed surfaces. the land uses such as commercial, asphalt, concrete | 0.8503 |
| Developed,medium Intensity | The built-up areas from 50 to 79 percent of the total area. This class commonly includes multi and single-family housing areas. | 0.5768 |
| Developed,Low Intensity | The built-up areas from 21 to 49 percent of the total area, this sub class commonly includes singe family housing areas, especially in rural neighborhoods, but may all types of land use | 0.2929 |
| Developed,Open space | The built-up areas less than 20 percent of the total cover, contains mixture of some constructed materials, but mostly managed grasses or low-lying vegetation planted in developed areas for recreation, erosion control or aesthetic purposes, | 0.0941 |

The total percent of an impervious for this study computed based on the C-CAP land use classification pattern presented in below. The total percent of an impervious for the catchment area is 60.7% see the table

Table 4.12 Representative impervious for catchment area 1

| Types of land use | % Impervious | Area (m2) | % Impervious *Area | Representative Impervious % |
|---|---|---|---|---|
| Commercial | 85.030 | 142,622 | 12,127,149 | 60.7 |
| Construction | 57.680 | 267,250 | 15,414,980 | |
| Recreation ground | 9.410 | 208,194 | 1,959,106 | |
| Residential | 57.680 | 4,379,788 | 252,626,172 | |
| Road | 85.030 | 881,036 | 74,914,491 | |
| Total area (m2) | | 5,878,890 | 357,041,897 | |

### C. Routing Models

In this section the models compute a downstream hydrograph based on the upstream hydrograph as a boundary condition, by solving the basic equation of continuity and momentum equations. Under HEC HMS model there are five different types of routing models, among those this study used a Muskingum routing model.

The Muskingum routing model solve the weighted difference between inflow and outflow multiplied by the travel time k.the Muskingum routing model as follows

$S_t = KO_t + KX(I_t – O_t) = K(XI_t +(1-X)O_t)$

K is travel time of the flood wave through routing reach, X is dimensionless weight (o <= X<=0.5)

If X = 0.5, equivalent weight is set to inflow and out flow thus, a uniformly wave distribute through the reach and having well defined channels. Channels with mild slopes and the flows going out of the parameter x will be approach to zero (HEC HMS technical reference manual)

Travel time of flood wave formulated as follows (HEC HMS user manual)

$K = L/V_w$

$V_w = (1.33\text{-}1.67)\ Vav$

L = reach length ,$V_w$ = wave velocity ,Vav = manning's velocity

Table 4.13 Reach travel time (K)

| Reach | Reach length in m | Reach slope | L1 | L2 | H | A | P | Average velocity in hr | Wave velocity in hr | Travel time (k) in hr |
|---|---|---|---|---|---|---|---|---|---|---|
| Reach-3 | 989.12 | 0.01413 | 23 | 29 | 1.3 | 37.7 | 35.3 | 3.7026 | 4.9 | 0.055793827 |
| Reach-2 | 1268.53 | 0.00628 | 18 | 26 | 1.8 | 46.8 | 34.4 | 2.8995 | 3.9 | 0.091374618 |
| Reach-1 | 414.85 | 0.00739 | 11 | 11 | 2.5 | 27.5 | 16.0 | 3.6758 | 4.9 | 0.023571651 |

In the same, the dimensionless x has been estimated by HEC HMS technical user manual the equation as shown below

$$X = \frac{1}{2}\left(1 - \frac{Q}{B * S * C * \Delta X}\right)$$

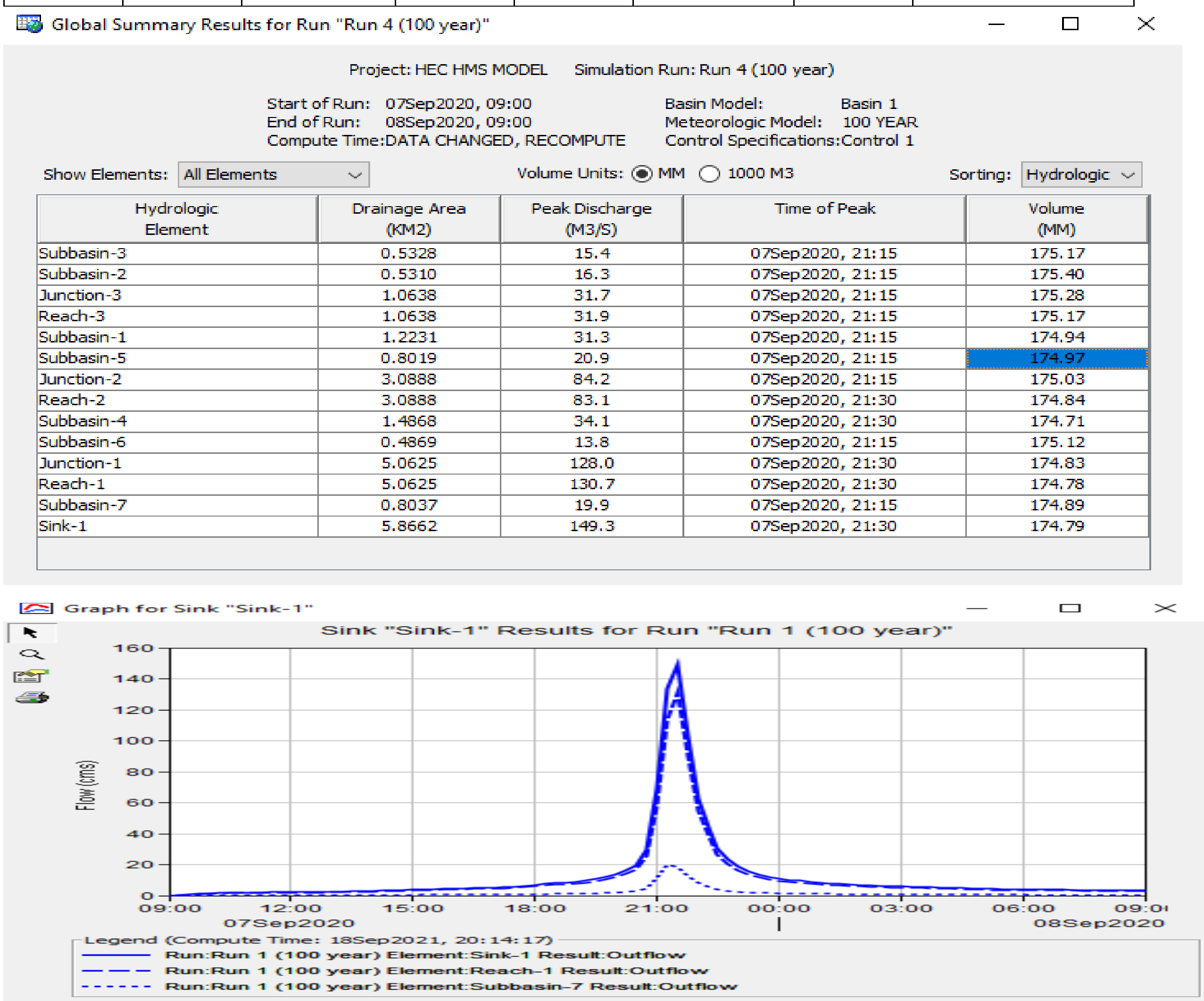
Qo = a reference flow from the inflow hydrograph

B = top width of flow area, So = friction slope or bed slope, C =flood wave speed, Δx = reach length

According to (ponce, 1983) the reference flow is the average value of the hydrograph, it is the mid-way between the base flow and the peak flow, after the value of k (travel time estimated) ,x can be computed by trial error. In this case, the first trial value of x is 0.2 and the reference inflow taken as half of the first trial result output, then the next trial is tabulated in the table 4.14 as follow

Table 4.14 Dimensionless x

| Trial ,No | Reach | Reach Lngth | Top width | Slope | wave velocity | Qo =Q/2 | Dimensionless (x) |
|---|---|---|---|---|---|---|---|
| 1 | Reach-3 | 989.12 | 29 | 0.01413 | 4.9245 | 15.6500 | 0.4961 |
| | Reach-2 | 1268.53 | 26 | 0.00628 | 3.8563 | 40.0000 | 0.4750 |
| | Reach-1 | 414.85 | 11 | 0.00739 | 4.8888 | 73.0500 | 0.2785 |
| | | | | | | | |
| 2 | Reach-3 | 989.12 | 29 | 0.01413 | 4.9245 | 15.9500 | 0.4960 |
| | Reach-2 | 1268.53 | 26 | 0.00628 | 3.8563 | 41.5500 | 0.4740 |
| | Reach-1 | 414.85 | 11 | 0.00739 | 4.8888 | 74.6500 | 0.2736 |


Global Summary Results for Run "Run 4 (100 year)"

Project: HEC HMS MODEL Simulation Run: Run 4 (100 year)

Start of Run: 07Sep2020, 09:00 Basin Model: Basin 1
End of Run: 08Sep2020, 09:00 Meteorologic Model: 100 YEAR
Compute Time:DATA CHANGED, RECOMPUTE Control Specifications:Control 1

Show Elements: All Elements Volume Units: MM 1000 M3 Sorting: Hydrologic

| Hydrologic Element | Drainage Area (KM2) | Peak Discharge (M3/S) | Time of Peak | Volume (MM) |
|---|---|---|---|---|
| Subbasin-3 | 0.5328 | 15.4 | 07Sep2020, 21:15 | 175.17 |
| Subbasin-2 | 0.5310 | 16.3 | 07Sep2020, 21:15 | 175.40 |
| Junction-3 | 1.0638 | 31.7 | 07Sep2020, 21:15 | 175.28 |
| Reach-3 | 1.0638 | 31.9 | 07Sep2020, 21:15 | 175.17 |
| Subbasin-1 | 1.2231 | 31.3 | 07Sep2020, 21:15 | 174.94 |
| Subbasin-5 | 0.8019 | 20.9 | 07Sep2020, 21:15 | 174.97 |
| Junction-2 | 3.0888 | 84.2 | 07Sep2020, 21:15 | 175.03 |
| Reach-2 | 3.0888 | 83.1 | 07Sep2020, 21:30 | 174.84 |
| Subbasin-4 | 1.4868 | 34.1 | 07Sep2020, 21:30 | 174.71 |
| Subbasin-6 | 0.4869 | 13.8 | 07Sep2020, 21:15 | 175.12 |
| Junction-1 | 5.0625 | 128.0 | 07Sep2020, 21:30 | 174.83 |
| Reach-1 | 5.0625 | 130.7 | 07Sep2020, 21:30 | 174.78 |
| Subbasin-7 | 0.8037 | 19.9 | 07Sep2020, 21:15 | 174.89 |
| Sink-1 | 5.8662 | 149.3 | 07Sep2020, 21:30 | 174.79 |



Figure 4.10 HEC HMS peak discharge for 100 years return period

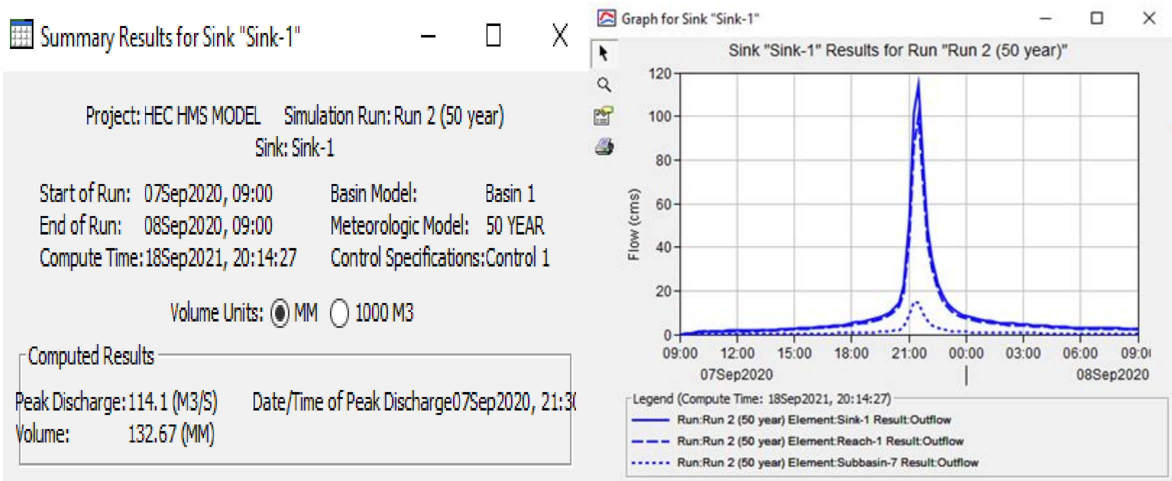



Figure 4.11 HEC HMS peak discharge for 50 years return period

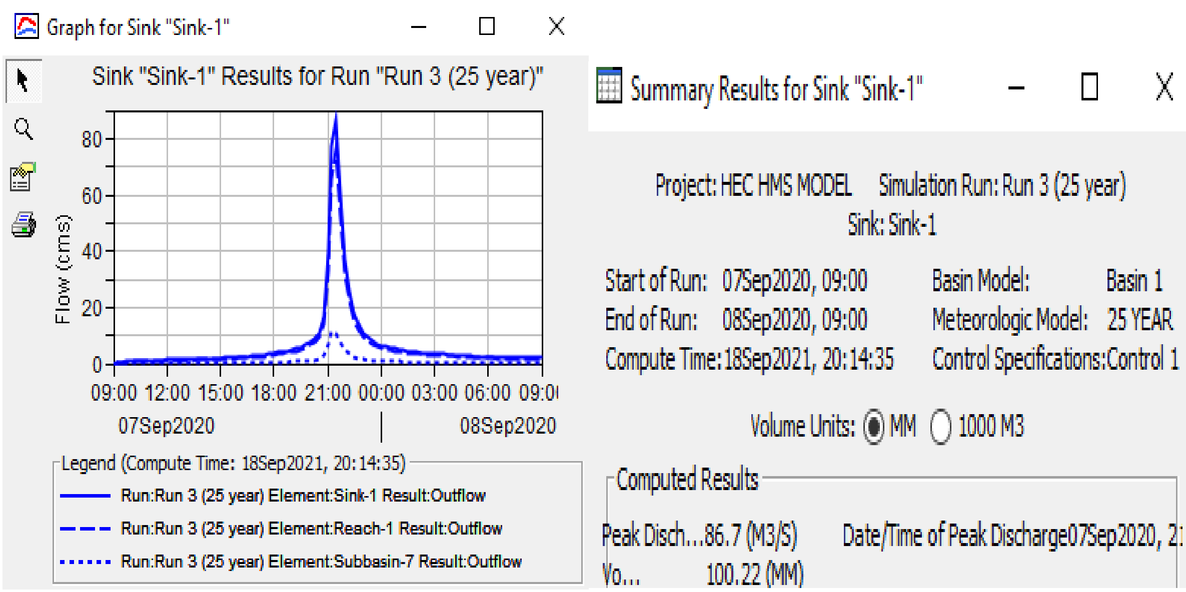



Figure 4.12 HEC HMS peak discharge for 25 years return period

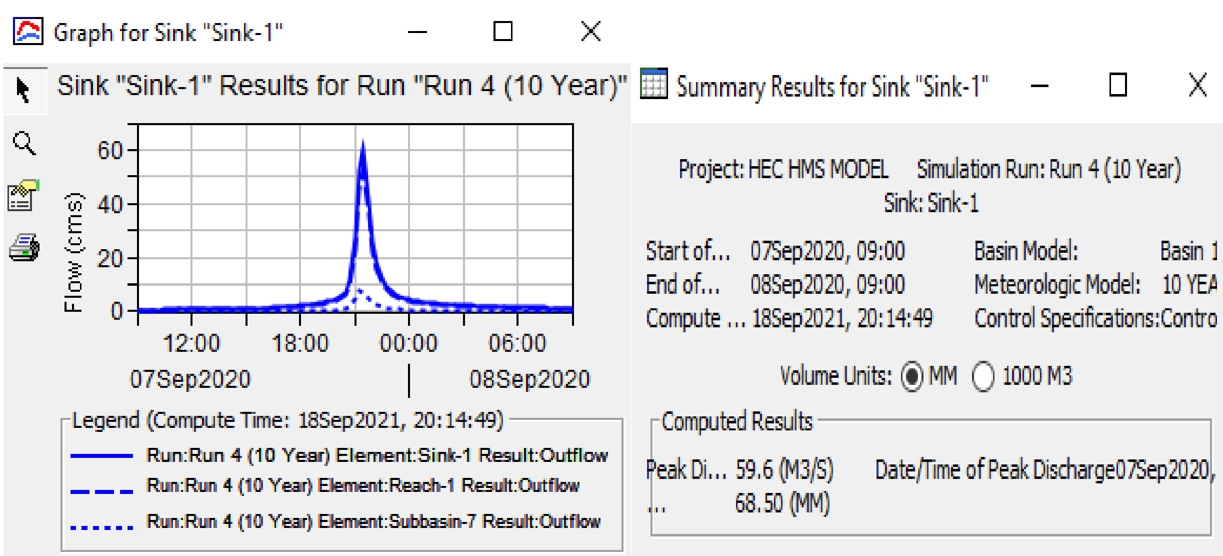



Figure 4.13 HEC HMS peak discharge for 10 years return period

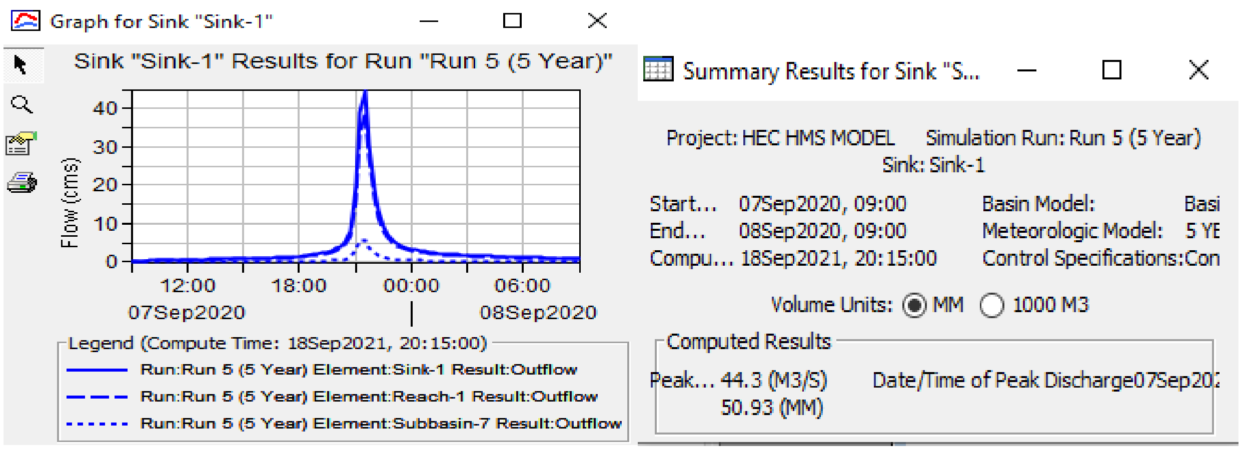



Figure 4.14 HEC HMS peak discharge for 5 years return period

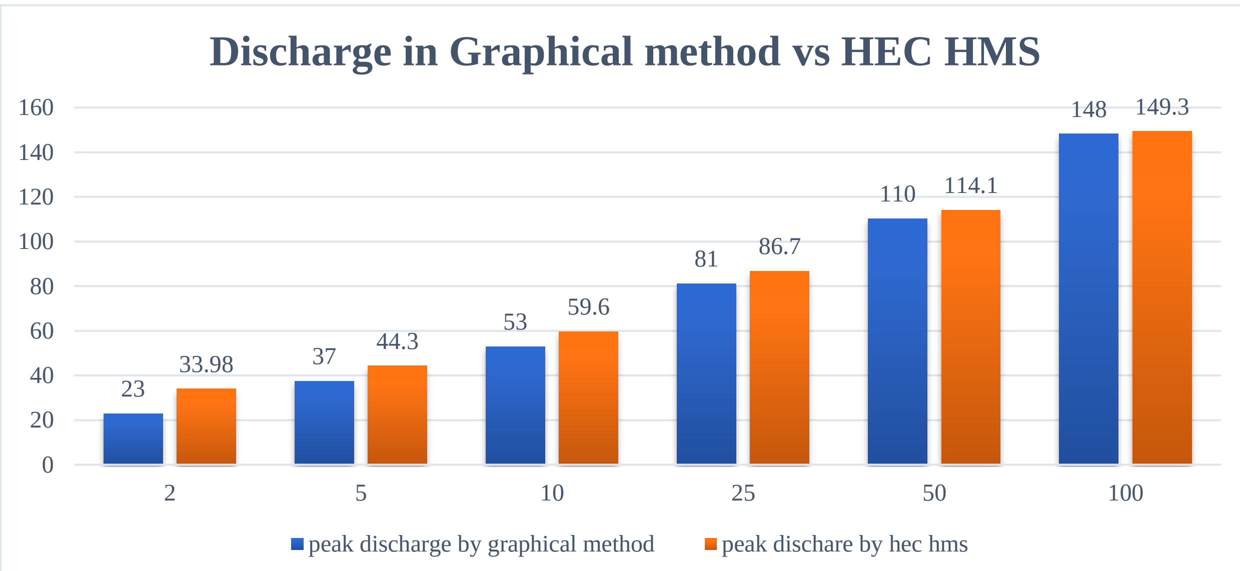



Figure 4.15 peak discharge comparison graphical vs HEC HMS

## 4.2 Hydrology analysis for catchment area2

### 4.2.1 Rational method

The Rational Method is most accurate for estimating the design storm peak runoff for areas up to 80 ha (0.8 km2). The method can be applied to small catchments if they do not exceed 0.8 km2 as per AACRADDM. The consequences of applying the Rational Method to larger catchments is to produce an over estimate of discharge and a non-Conservative design.

#### A. Selection of Runoff Coefficients, C

The runoff coefficient, C, characterizes the combined effects of penetration of water into the ground, detention storage, retention, flow routing, and interception, all of which an influence for the time of distribution and peak rate of runoff. The excess precipitation that runs off be subject to on the relative imperviousness of the ground surface. For different covering of land use ERA DDM formulated the runoff coefficient C (table 4.15) ,for accurate determination of runoff for a given area ,it should be selected reasonable runoff coefficient C

Regarding to this study the catchment area mostly covered by single family residential house, thus reasonable runoff coefficient will be an average of ( 0.3 – 0.5) that is 0.4

Table 4.15 runoff coefficient

| Description o Area | Runoff coefficient |
|---|---|
| Business Downtowns area | 0.7-0.95 |
| neighborhood areas | 0.5-0.7 |
| Residential: Single-family areas | 0.3-0.5 |
| Residential: Multi units, detached | 0.4-0.6 |
| Residential: Multi units, attached | 0.6-0.75 |
| Suburban | 0.25-0.4 |
| Residential (0.5 hectare lots or more) | 0.3-0.45 |
| Apartment dwelling areas | 0.5-0.7 |
| Industrial: Light areas | 0.5-0.8 |
| Industrial: Heavy areas | 0.6-0.9 |
| Parks, cemeteries | 0.1-0.25 |
| Playgrounds | 0.2-0.4 |
| Railroad yard areas | 0.2-0.4 |
| Unimproved areas | 0.1-0.3 |

## B. The frequency factors

As per ERA DDM, the frequency factor is used to magnify the less frequent storms, i.e. storms with recurrence interval greater than 10yr. Table 4.16 shows the frequency factor values.

Table 4.16 Recurence cofficent

| **Recurrence interval (years)** | **Cf** |
|---|---|
| 5 | 1 |
| 10 | 1 |
| 25 | 1.1 |
| 50 | 1.2 |
| 100 | 1.25 |

### 4.2.1.1 Time of concentration for catchment area 2

Time of concentration for over land flow determined by kerby formula, which is very reasonable for even slope

$$Tc = \left(\frac{2.2*n*l}{s^{0.5}}\right)^{0.324}$$

Time of concentration (Tc) in minute

Manigns cofficent (n) =0.011

Slope of the river bed (s) = 0.000633333

Length of overland flow recommended by USGS = 30 m =98.425 feet

$$Tc = \left(\frac{2.2*n*l}{s^{0.5}}\right)^{0.324}$$

Tc = 4.367 minute

Time of concentration for define water coarse

$$\mathbf{Tc} = \left(\frac{0.872*L^2}{1000*Sav}\right)^{0.385}$$

Length of the river (l) = 207 m

Tc = 5.41 minute

Total time of concentration =9.77 minute = 10 minute

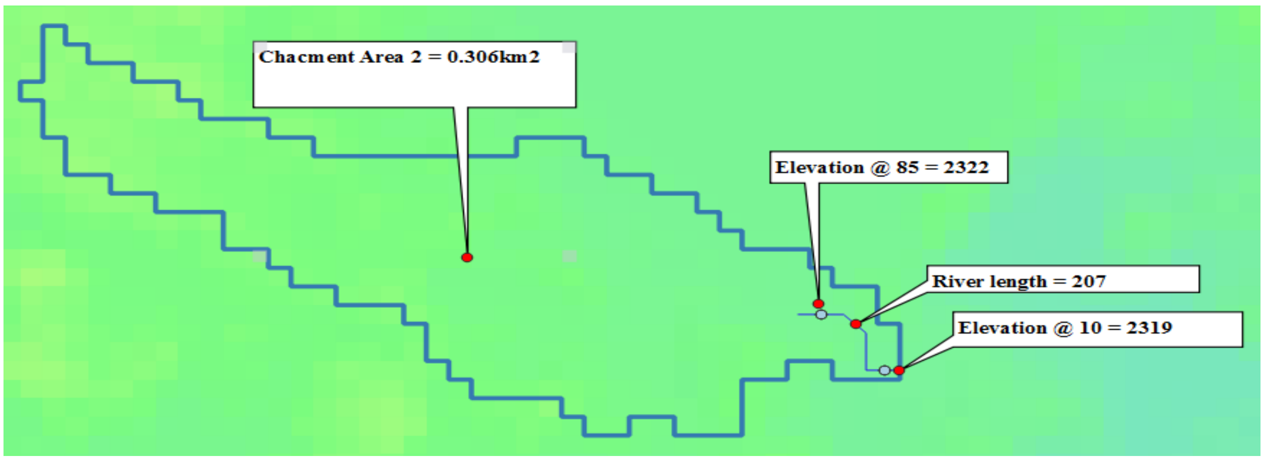



Figure 4.16 catchment area 2

**Average rainfall intensity (I):-** The rainfall intensity, I, is obtained from an intensity-duration-frequency (IDF) curve which constructed for specific location area for different return period and read the rainfall intensity for each return period for the computed time of concentration

However, in this study the computed total time of concentration for catchment area 2 is short, thus it has been used the empirical formula for rainfall intensity, which is formulated to determine the rainfall intensity for short period of time, under the assumption that the duration is equal to the time of concentration for the catchment area. The computed intensity as follow in the table 4.17

$$\mathbf{I_{t} =} = \frac{(b+24)^n}{24\ (b+t)^n} * \mathbf{R_{24}}$$

R24 = design precipitations for 24 hr in mm ,t = time of concentration (hr) ,b = 0.3 and n = 0.92 as suggested by ERA DDM 2013

Table 4.17 Rain fall intensity

| Return period | P | $I_t = \frac{(b+24)^n}{24\,(b+t)^n} * R_{24}$ |
|---|---|---|
| 2. | 42.02 | 3.86 |
| 5. | 59.78 | 5.49 |
| 10. | 77.91 | 7.15 |
| 25. | 110.32 | 10.12 |
| 50. | 143.29 | 13.15 |
| 100. | 185.96 | 17.07 |

#### 4.2.1.2 Rain fall runoff transformation

There are various recognized methods and approaches for estimation of runoff. The choice among the different methods depends on the availability of hydro-meteorological data required by those models and their appropriateness and applicability in the particular area of interest

In areas like Ethiopia where there is scarcity of hydro-gauged river data. Therefore, it is essential to adapt some empirical formulas, which reasonably and safely enable to estimate the flood associated with the required return period.

To estimate the amount of runoff generated from the catchment area 2 shown in table below , the rain fall runoff transformation carried out by rational methods.

The Rational formula is expressed as:

$$\mathbf{Q = 0.278 * C * Cf * I\ A}$$

Where:

Q = Maximum rate of runoff, $m^3/s$

C = Runoff coefficient

I = Rainfall intensity for a period equal to the time of concentration and for design return period, mm/hr

A = catchment area tributary to the design location, $km^2$

A = 0.31KM2

Table 4.18 Peak discharge for catchment area 2

| **Return period** | **2** | **5** | **10** | **25** | **50** | **100** |
|---|---|---|---|---|---|---|
| **Cf** | **1** | **1** | **1** | **1.1** | **1.2** | **1.25** |
| **C** | **0.4** | **0.4** | **0.4** | **0.4** | **0.4** | **0.4** |
| **I (mm/hr)** | **66.0** | **93.9** | **122.4** | **173.3** | **225.1** | **292.2** |
| **Q (m3/sec)** | **2.3** | **3.2** | **4.2** | **6.6** | **9.3** | **12.6** |

# Chapter five

## 5 Hydraulics analyses

In the previous chapter, the runoff of the catchment area is determined. The next step is to check the adequacy and sustainable of the existing drainage system that suite the site conditions for the peak discharge. This section deals with review of waterway and selection of appropriate drainage structure type.

Among from the specific objective, one of the main tasks is to evaluate the existing opening sizes of the drainage structures from the rate of flood runoff (discharge) that has been passing through the bridge.

This technique arranges the hydraulic features of the stream influencing the maximum discharge, such as velocity of flow, slope of the stream, cross sectional area of the stream and shape and roughness of the stream. This method will be used for major streams to compute the design flood levels at crossing sites after the Review discharges have been estimated by the hydrological methods of SCS Method.

## 5.1 Methodologies on hydraulics analysis

The main steps that are used to solve the specific objectives of hydraulics analysis for this study are;

- Import terrain data (30 m resolution DEM of the specified area ) into RAS Mapper in a gridded format, in order to be used to make a terrain model such as (*.flt); GeoTIFF (*.tif) format; ESRI grid files etc.
- Create the projection file using Q GIS 3.18 on the desired projection
- Import the river and digitalization with RAS Maper
- Draw the cross section perpendicular to the river start from the upstream to down stream
- Using google satellite map under RAS Map layer checked the specified projection file laid on the right place.
- After verifying the cross-section place, review and correct the geometry of the cross section using graphical cross section editor under HEC RAS.
- Select appropriate type of manning's (n) coefficient
- Assign the flow state (in this case the flow state is steady flow)
- Using the flow regime, boundary condition and peak discharge information, run the model and identify the water surface elevation.

- Based on the water surface elevation proposed the new water opening

### 5.1.1HEC-RAS hydraulic model

The main objective of the HEC – RAS program for a given set of flow data (in this case the flow is steady state flow) is to compute water surface elevations at all locations of interest in Subcritical, supercritical, and mixed flow regime. HEC RAS program solving Energy equation with an iterative procedure called the standard step method for determination of all water surface profile from one cross section to others.

From the most important hydraulic parameter slope, roughness coefficients (manning's n), expansion and contractions coefficients are very important parameter for hydraulic modelling.

### 5.1.2 Basic data requirement

The data needed to perform these computations are divided into the following categories: Geometric data; Steady flow data. Geometric data are the main essential information for computing any of the analyses executed by HEC – RAS. The basic geometric data comprises of creating the connectivity of the river system; cross section data (whether extracts from DEM or assembling the data from surveyor), reach lengths, Hydraulic structure data's (bridge water way width and depth of bridge) which are also considered geometric data.

**A. River system schematic**

The schematic defines how the various river reaches are connected, as well as establishing a naming convention for referencing all the other data. The rivers delineation for this study is carried out using google earth and HEC RAS. During extraction of the river, it has been used google satellite map in RAS Mapper and google earth software which enables to access aerial and satellite imaginary. The river schematic shown below in figure 5.1 and 5.2

Figure 5.1 river 1 and river 2 From google satellite map

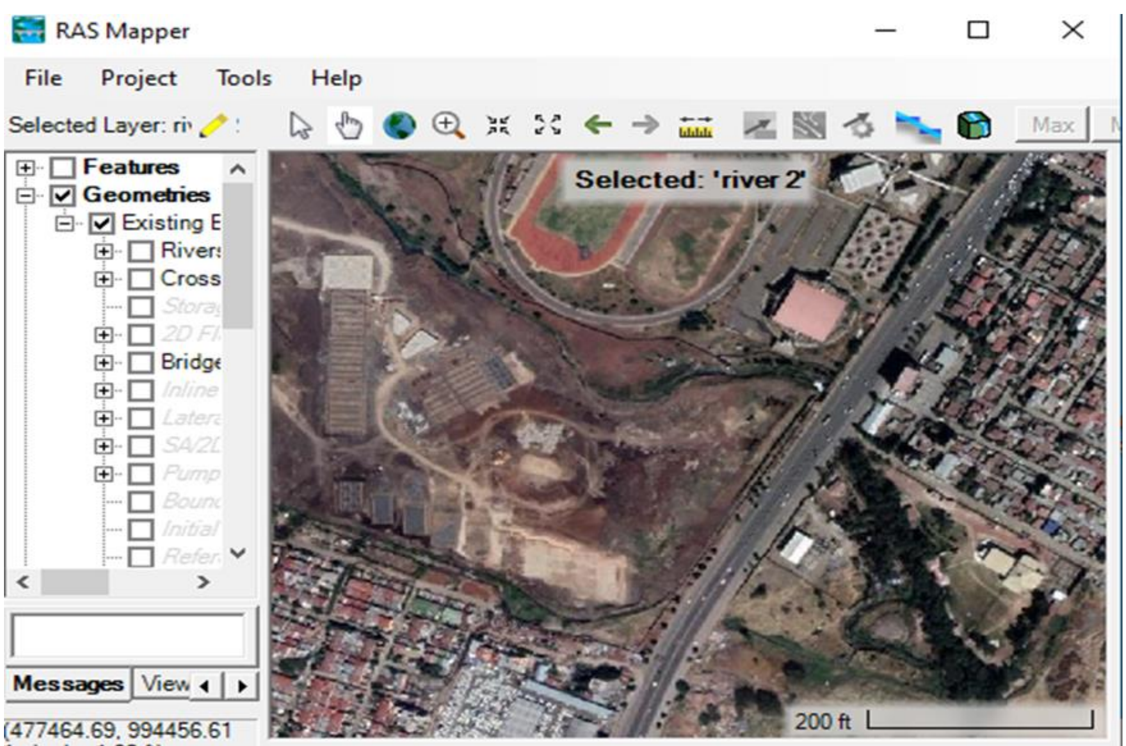



Figure 5.2 river 1 and river 2 From RAS Mapper

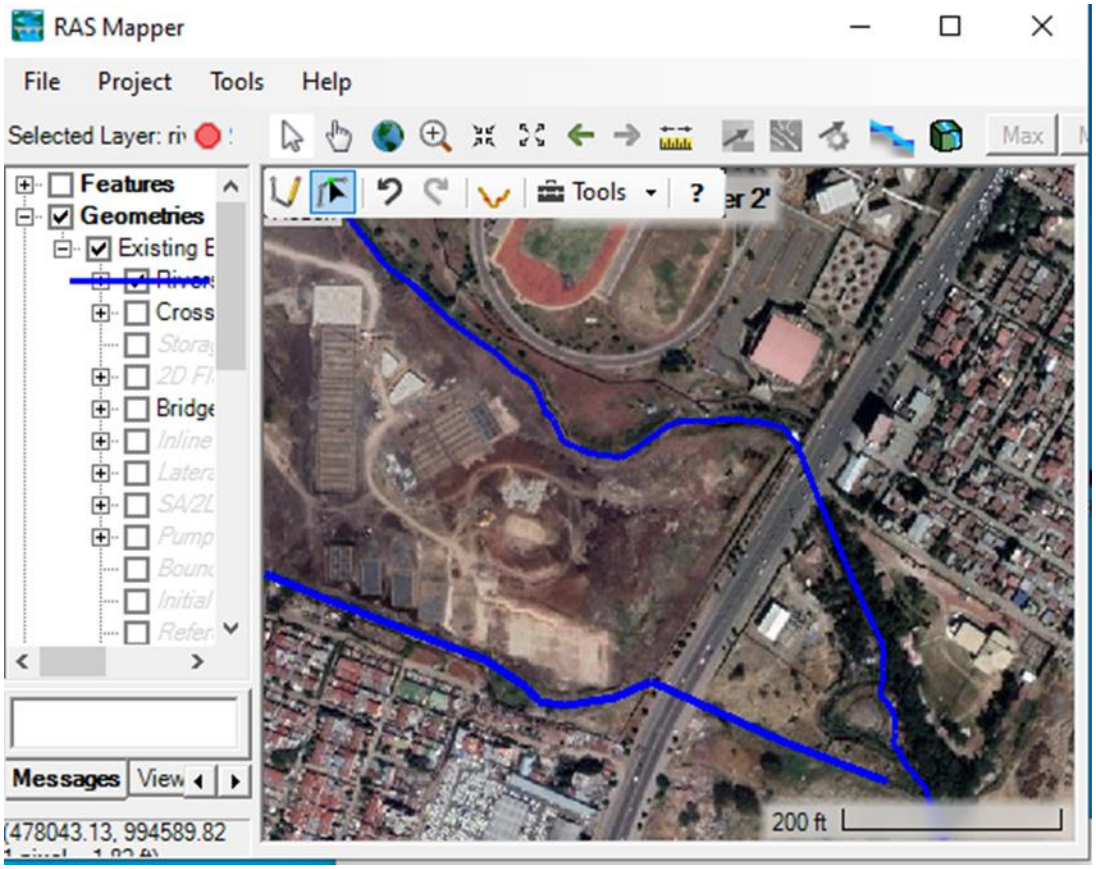



Figure 5.3 river 1 and river 2 Schematic diagram in RAS Mapper

### B. Cross section geometry

**A** good model needs adequate data, with a precise representation of model. River model also needs enough data for the given river channel geometry. An accurate illustration of the river geometry, it would be possible to predict accurately the river water level along the channel

In this section, presents extractions of cross section data that is hydraulically fit for hydraulic model. The extraction/obtaining of river cross section data can be acquired from google earth software, using surveying and extracting from satellite DEM (Ujas Pandaya,Anant Patel,Dhruvesh Patel ,2017)

In this study the cross-section data of the river acquired by extraction of satellite DEM data using HEC RAS model.

Boundary geometry for the analysis of flow in natural stream is specified in terms of ground surface profiles (cross section) and the measured distance between them (reach lengths). Cross sections are located at intervals along a stream to characterize the flow carrying capability of the stream and its adjacent floodplain. The following main tasks were carried out:

- Cross sections of the river: - in this study the river cross section data were extracted directly from bare earth LIDAR DEM (30m resolution UTM 32637 North) of the study area
- Projection (PRJ):- Created using QGIS 3.18 shape file in the desired projection.

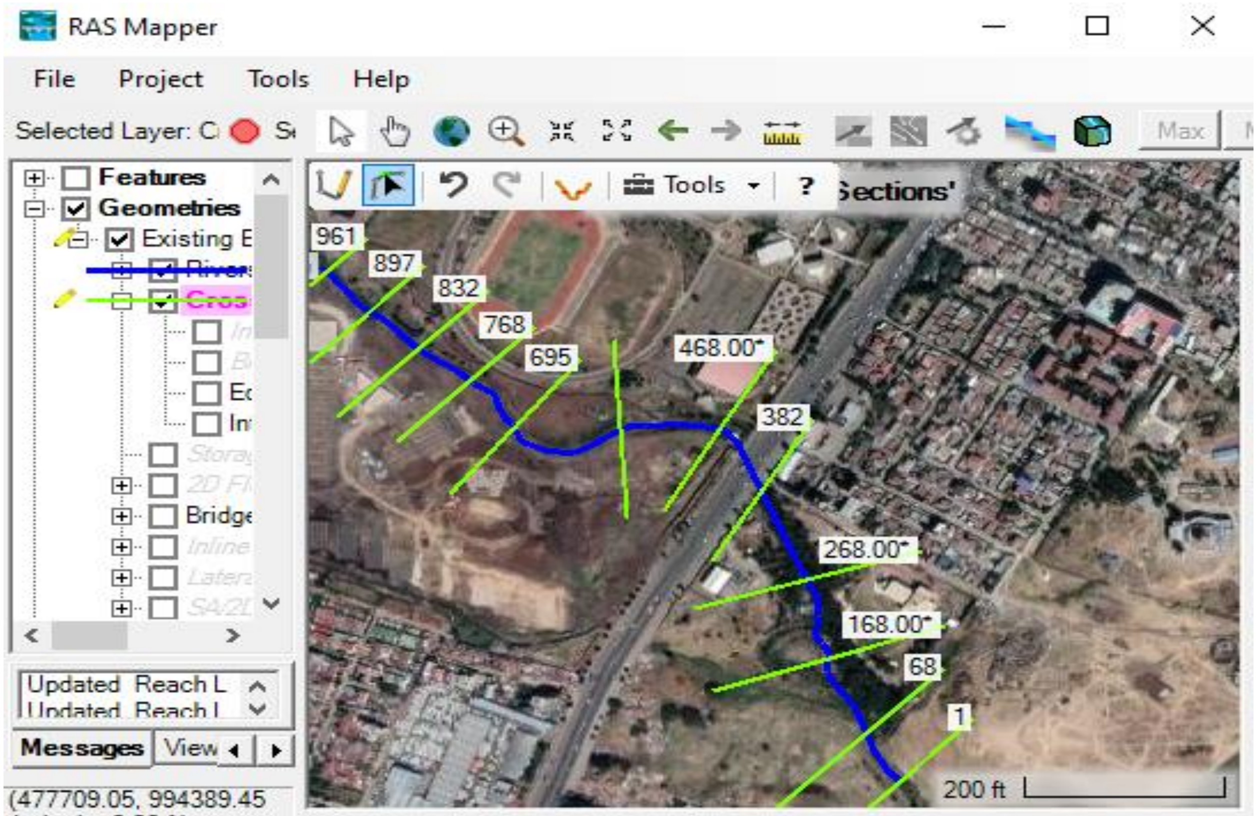



Figure 5.4 River 1 schematic in RAS mapper

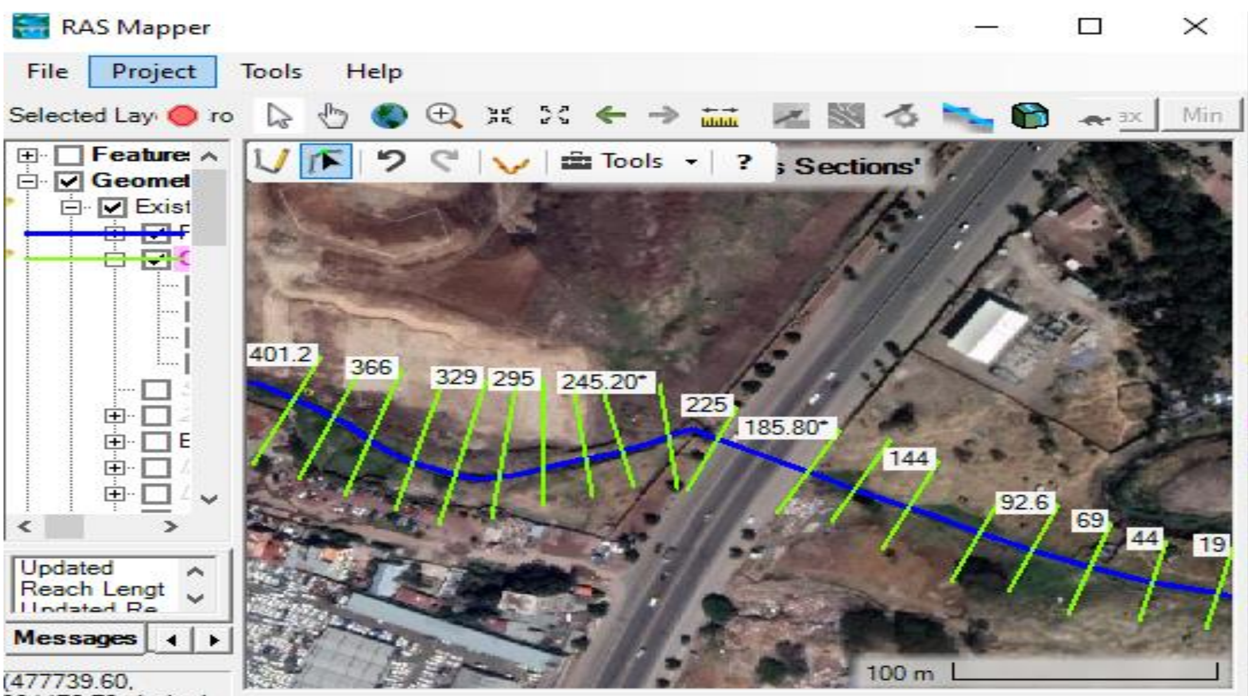



Figure 5.5 River 2 schematic in RAS mapper

**C. Reach length**

The measured distances between cross sections are referred to as reach lengths. The reach lengths for the left overbank, right over bank and channel are specified on the cross-section data, HEC RAS set automatically the reach length.

**D. Roughness coefficients**

HEC RAS program determines the main channel portion slope and subdivided into different portion for using roughness coefficients (manning's n) based on the following criteria:

- If the main channel side slope stepper than 5H:1V (in this study the side slope not greater than 5H:1V, thus have been used homogeneous roughness coefficient for the entire channel.

Manning's 'n' is influenced by several factors and its selection is very hard for natural channels, depending on the site visiting and Pictures of channels, select appropriate type of manning's using n (normal) = 0.05 (see the figure as shown below) and select the appropriate roughness for natural channel stream coefficient from the table 3.16

Figure 5.6 Scattered brush and heavy weeds along river channel

Table 5.0 roughness coefficient (n) for different conditions

(Source ERA-DDM 2013)

| NATURAL STREAMS | | | | |
|---|---|---|---|---|
| **1 Minor streams (top width at flood stage < 30 m)** | | | | |
| a. Streams on Plain | | minimum | normal | maximum |
| 1 | Clean, straight, full stage, no rims or deep pools | 0.025 | 0.03 | 0.033 |
| 2 | Same as above, but more stones and weeds | 0.03 | 0.035 | 0.04 |
| 3 | Clean, winding, some pools and shoals | 0.033 | 0.04 | 0.045 |
| 4 | Same as above, but some weeds and stones | 0.035 | 0.045 | 0.05 |
| 5 | Same as above, lower stages, more ineffective | | | |
| | slopes and sections | 0.04 | 0.048 | 0.055 |
| 6 | Same as 4, but more stones | 0.045 | 0.05 | 0.06 |
| 7 | Sluggish reaches, weedy, deep pools | 0.05 | 0.07 | 0.08 |
| 8 | Very weedy reaches, deep pools, or floodways | 0.075 | 0.1 | 0.15 |
| | with heavy stand of timber and underbrush | | | |
| b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages | | | | |
| 1 | Bottom: gravel, cobbles, and few boulders | 0.03 | 0.04 | 0.05 |
| 2 | Bottom: cobbles with large boulders | 0.04 | 0.05 | 0.07 |
| **2 Flood Plains** | | | | |
| a. Pasture, no brush | | minimum | normal | maximum |
| 1 | short grass | 0.025 | 0.03 | 0.035 |
| 2 | high grass | 0.03 | 0.035 | 0.05 |
| b. Cultivated area | | | | |
| 1 | No crop | 0.02 | 0.03 | 0.04 |
| 2 | Mature row crops | 0.025 | 0.035 | 0.045 |
| 3 | Mature field crops | 0.03 | 0.04 | 0.05 |
| c. Brush | | | | |
| 1 | Scattered brush, heavy weeds | 0.035 | 0.05 | 0.07 |
| 2 | Light brush and trees in winter | 0.035 | 0.05 | 0.06 |
| 3 | Light brush and trees, in summer | 0.04 | 0.06 | 0.08 |
| 4 | Medium to dense brush, in winter | 0.045 | 0.07 | 0.11 |
| 5 | Medium to dense brush, in summer | 0.07 | 0.1 | 0.16 |
| d. Trees | | | | |
| 1 | Dense willows, summer, straight | 0.11 | 0.15 | 0.2 |
| 2 | Cleared land with tree stumps, no sprouts | 0.03 | 0.04 | 0.05 |
| 3 | Same as above, but with heavy growth of spouts | 0.05 | 0.06 | 0.08 |
| 4 | Heavy stand of timber, a few down trees,little undergrowth, flood stage below branches | 0.08 | 0.1 | 0.12 |
| 5 | Same as above, but with flood stage reaching branches | 0.1 | 0.12 | 0.16 |

**E. Site collected data**

In the Ring Road did not identified any designed and as built drawing for the Major Drainage Crossing Structure, therefore collect each data by measuring from site for each crossing structure.

**F. 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.

The flow Regime: is answer for the question, through which flow type the model to be run. The flow types are as follows

- Sub critical
- Super critical
- Mixed flow

Boundary conditions: is the border that the model to be run, regarding to the boundary conditions, flow regime and availability of data are very important for specifying of the boundary conditions.

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

## 5.2 Hydraulics output result

The hydraulic analysis is carried out using HEC RAS model; the existing bridges at Imperial intersection and small bridge exist between Ayat Hospital and Imperial Round About analysis presented as shown below

### 5.2.1 Bridge at imperial intersection

Through detail site investigation and hydrological analysis and hydraulics analysis the existing bridge will serve properly for 50 years return period flood, however the bridge will not serve flood which exist for 100 years return period flood due to change in land use and land cover of the area. (see the existing bridge figure for 50 years ,100years flood)

The input Data collected from the actual site measurement, ERA DDM-2013 and AACRA DDM-2004. The Existing Bridge has a span of 11m and Depth of 3m as shown below

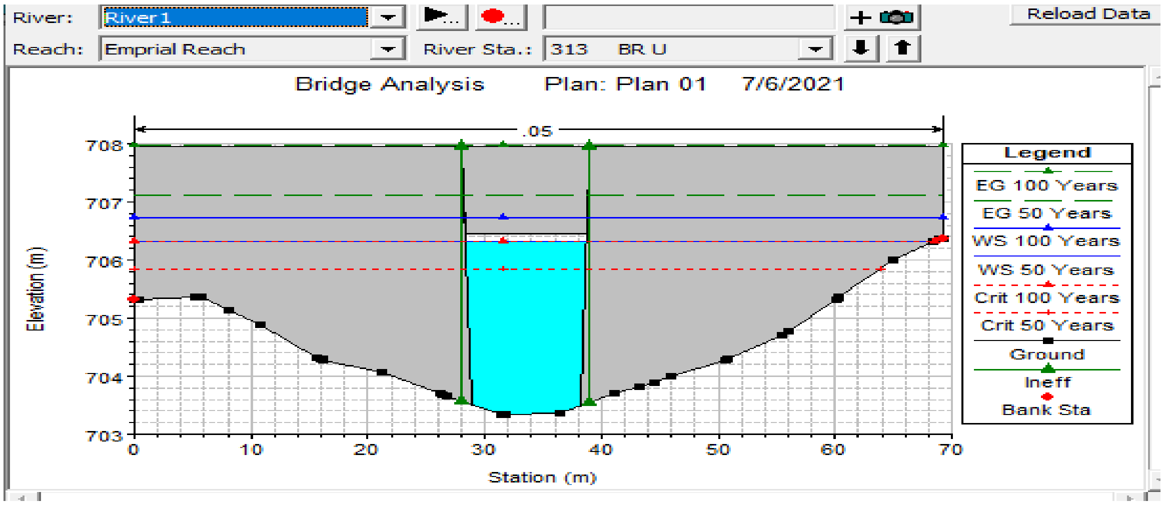



Figure 5.7 Existing upstream water profile section for 50 and 100 return period

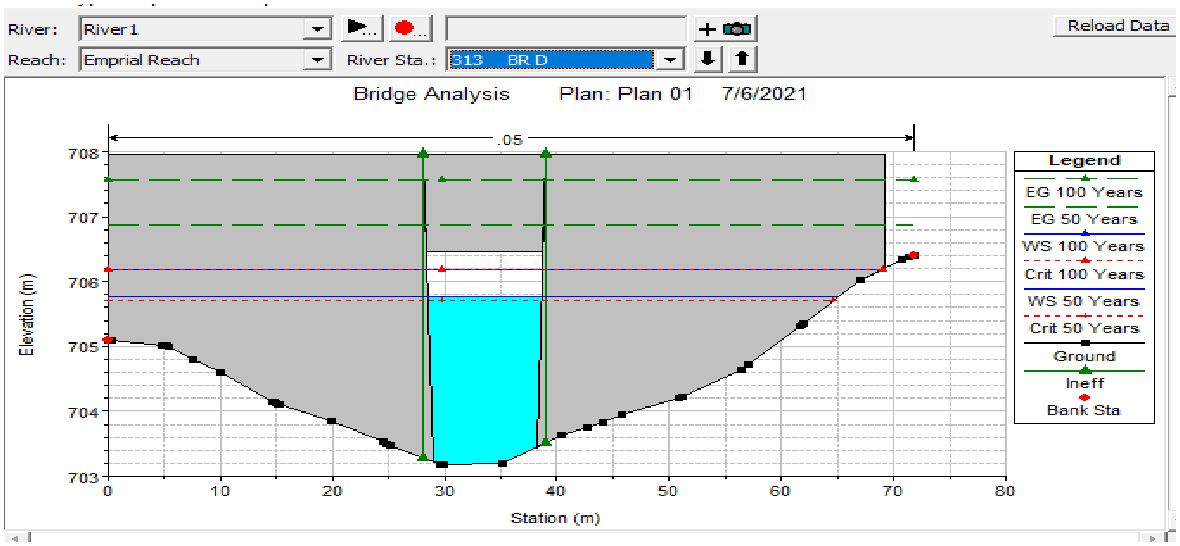



Figure 5.8 Existing Downstream water profile section for 50 and 100 return period

### 5.2.2 Bridge between ayat hospital and imperial round about

In this study has only one Medium Drainage Crossing Structure between Ayat Hospital and Imperial Round About. The Drainage of this Area is from the Compound and area field, has no significant flow. The existing bridge has span of 6m and Depth of 2.5m, is hydraulically sufficient and adequate, retain as it is.

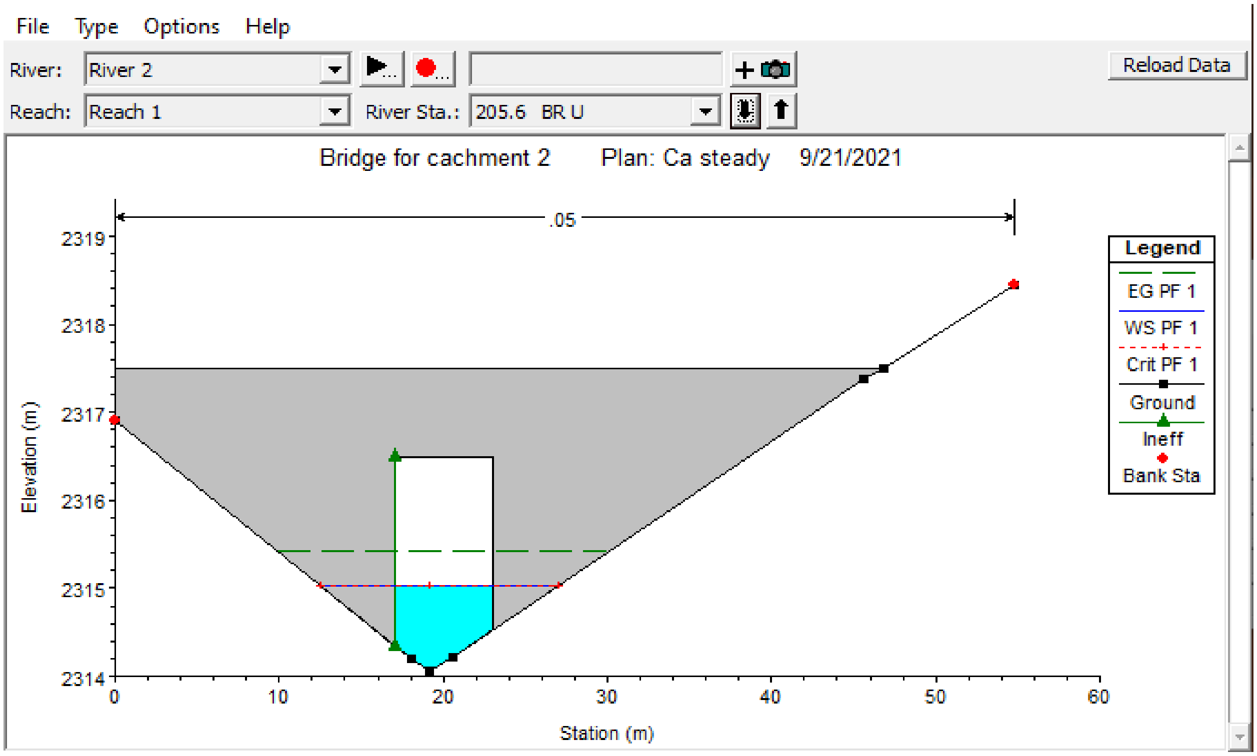



Figure 5.9 Existing upstream water profile section for 50 and 100 return period

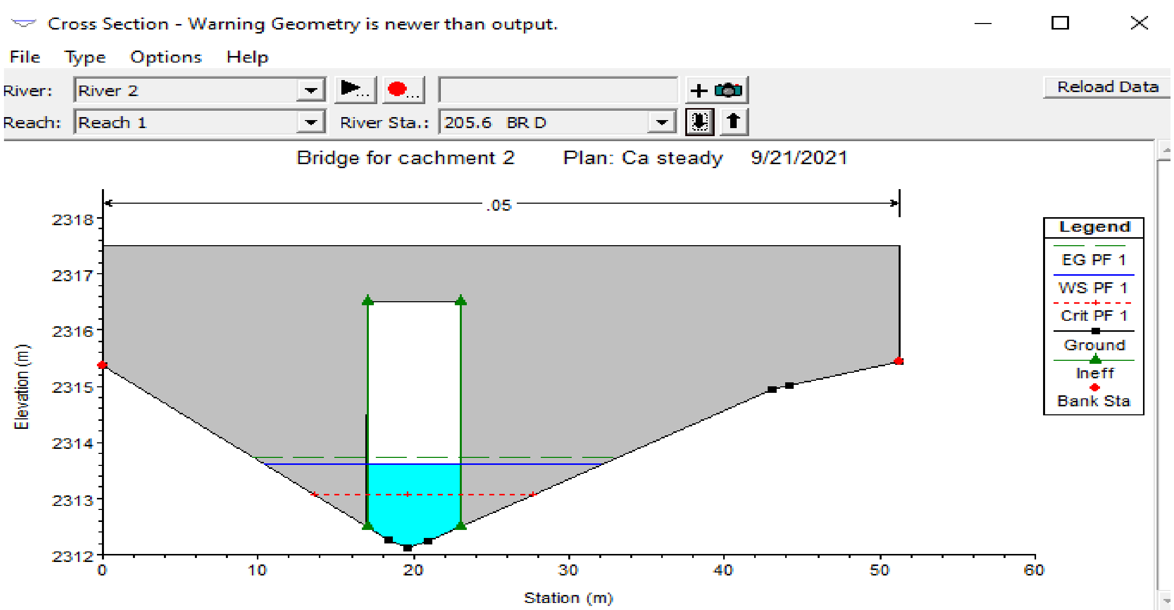



Figure 5.10 Existing Downstream water profile section for 50 and 100 return period

### 5.2.3 Storm Drainage

The main goal of pavement drainage is to keep a safe driving surface by providing minor storm and major storms. The minor storm disposes the water from the road, major storm acts as a conduit,

As it is observed during site investigation, the storm drainage facilities along the road are not function properly because of different reasons such as blockages of the inlet by debris, damage and lack of routine maintenance, improper usage of openings, and others. The tabulated reasons for improper functioning of storm drainage facilities are illustrated in the photo below

Figure 5.11 Damaged inlets and inlets clogged by rubbish

# 6 Conclusion and recommendations

## 6.1 Conclusion

Through detail site visit and desk analysis of hydrology/ hydraulic, one major River crossing and One Medium stream Crossing Structure only Identified around Imperial intersection.

Especially the major River required Detail Hydrological and Hydraulic assessment and analysis has been conducted on the U/S watershed of the area. It is noted that the land use and Land Cover dramatically changed and the Amount of runoff expected to be increased from the Ring road Design Period. Therefore, the existing bridge has been replaced with the New Bridge Structure, which will accommodate the incoming Q100 year discharge.

In-addition to this, storm drainage facility (inlet) are poorly functioned. The following recommendation has been done after evaluating section by section the drainage condition and Problem.

## 6.2 Recommendation

Based on detail site visit, Desk study, Detail Hydrology and Hydraulic analysis result the following specific Recommendation has been forwarded for each drainage type.

A. **Major river crossing bridge**.

Bridge structure around Imperial roundabout not Functioned Well for the 100-return period, so shall be replaced with the new Bridge as span of 15m and 3.5m, which will accommodate the incoming Q100 year discharge. in addition to this, the approach river Channel bed shall be Paved to increase its efficiency of crossing structure.

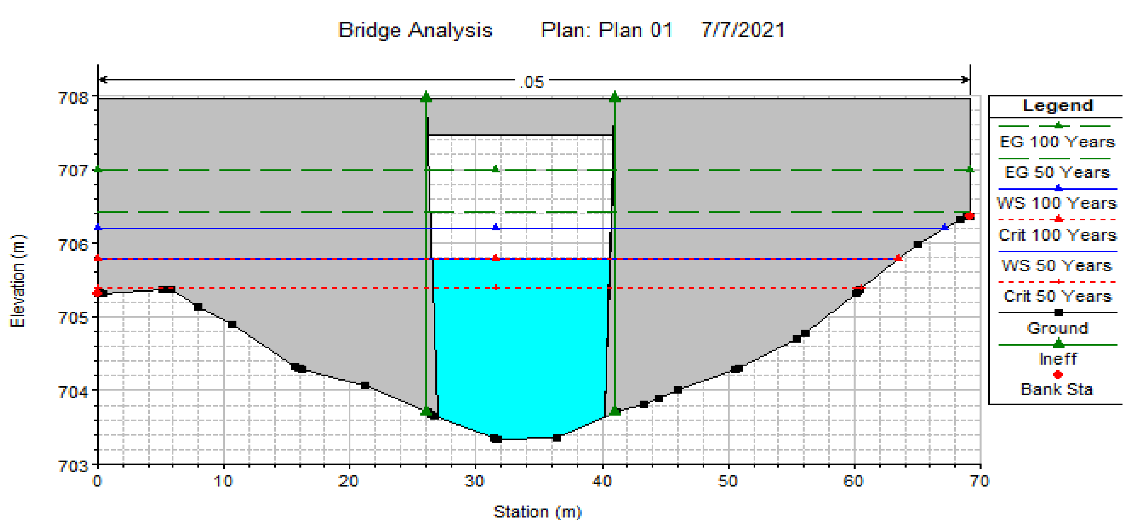



Figure 6.0 upstream Replaced bridge at imperial

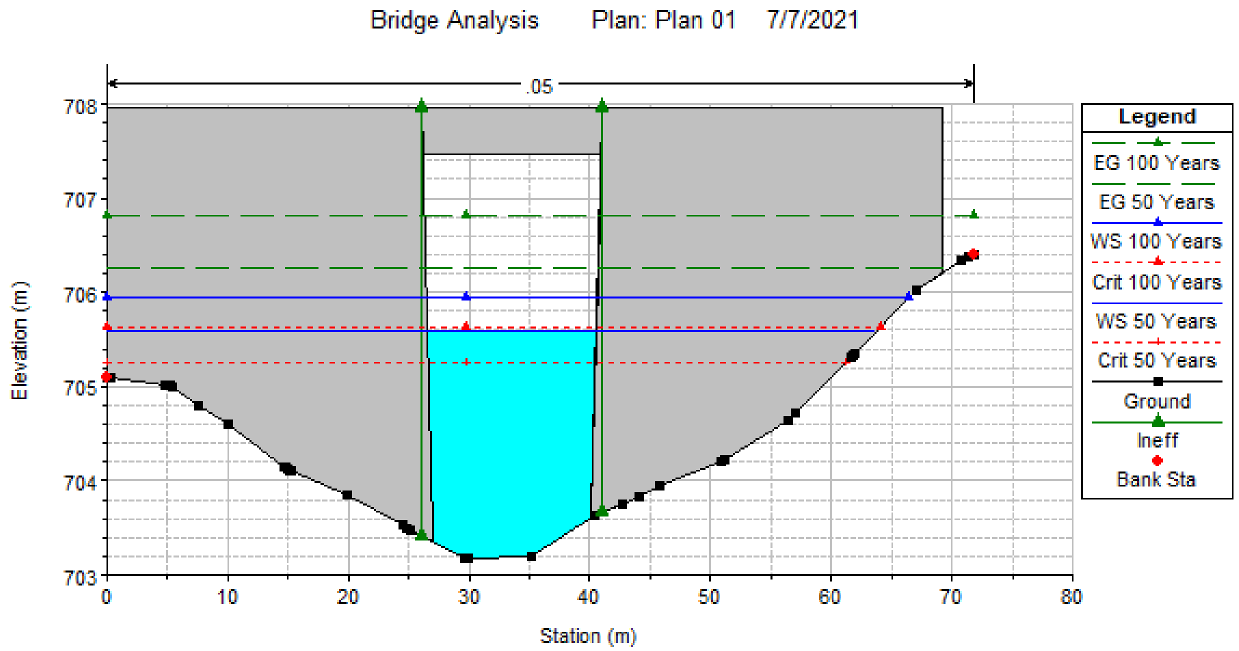



Figure 6.1 Downstream Replaced bridge at imperial

## B. Minor drainage crossing.

The medium crossing structure around between Ayat hospital and Imperial Round about, and as per the hydrologic and hydraulic Analysis it is not have a significant flow and Can be retain as it is , only required maintenance and clearing of the channel.

Figure 6.2 minor Drainage between Ayat hospital and Imperial

### C. Strom Drainage.

During site Reconnaissance most of the in lets are not functioned properly due to complete damage of inlets, blockages of by rubbish/dribs, dismantle of the inlets covering (skimmer), lack of routine maintenance and clearing, due to this the surface runoff become lay on the road and the pavement drainage system not being execute the intended function, thus the following recommendations has been forwarded

- Maintenance: - Through age some inlets are vandalized, in addition to this, most of the inlets are dismantle their cover (skimmer), thus the damaged inlets and skimmer should be changed by new ones.
- Clearing: -the type of inlets installed in the road were grate inlets, this type of inlets the major disadvantage is that they may be clogged by floating trash or debris, therefore it needs proper and continuous clearance throughout the drainage system.

## References

AACRA, 2003. Drainage Design Manual. 1st ed. Addis Ababa: Addis Ababa City Road Authority.

American Association of State Highways and Transportation Officials (AASHTO), ( 1991). Model Drainage Manual.

American Association of State Highways and Transportation Officials (AASHTO), ( 2014). Model Drainage Manual.

Antene ,Z., 2015.Intergrated Urbun Drainage System ;the case of Ayat to megenagn LRT Route (Master's thesis). Addis Ababa university institute of Technology

Adugn Ejeta ,2019. Sustainable urbun storm water management and drainage system modeling in small urbun catchment : case study of sebeta town (Master's thesis). Addis Ababa university institute of Technology

Belete.B ,Yilma .S ,Assefa .M (2014)Surface Water and Groundwater Resources of Ethiopia:

Potential and challenges of water resources Development

Beniyam.A. (2016). Assessiment of storm water drainage system in kemise town Addis Ababa university institute of Technology

Coastal Change Analysis Program (C-CAP) studied by NOHA

David Buter ,Christopher Digman,Christos Mrkopolouse and John w. Davies (Barbosa et al., 2012) : Urbun Drainage system: Design and Operation

David Butler, 2004 Jornal on urban drainage

EMA (1988).Ethiopian mapping Authority

Ethiopian Roads Authority. (2002). Drainage Design Manual . Addis Ababa.

Ethiopian Roads Authority. (2013),Drainage Design Manual,. Addis Ababa: .

Ekblad, J. and Isacsson, U. 2006. Influence of water on resilient properties of coarse granular materials

FHWA (2001) . Federal Highway Administration manual

FHWA (2001) . Federal Highway Administration manual

FUPCoB, F. U. (2008),Urban Storm Water Drainage Design Manual. Addis Ababa.

HEC RAS ,2006 Hydrologic Engineering center River Analysis System Reference manual

McCuen (1989) High way hydrology, 2nd edition

NRCS (2002), Natural resource conservation service manual

H.M.Ranghunath (2006), Hydrology: Principles, Analysis, Design; New Age International Publisher, Revised Second Edition

Richrd H.Mc Cuen(2004). Hyrdrologic analysis and design, 3rd edition

(Shuster, et al., 2005; SCS, 1986; Brody, et al., 2007; Schuler, et al., 2009). Flood inundation risk reduction using Smart Growth development

Santosh Kumar Garg 19th edition Irrigation Engineering and hydraulic structure

Tiza Michel Toryila (2016). The Effects of Poor Drainage System on Road Pavement

The World Bank Group, (July 2015) .Enhancing Urban Resilience

V.T.Chow, D. M. (1988 ). Applied Hydrology,

Viess man and Lewis(1996). Introduction to Hydrology, 4th Edition

Yilma, S.,(2005,et al). Engineering Hydrology. Addis Ababa.

Ziyada Basena ,2017 investigation on the cause of high way flooding in Addis Ababa; case study Lebu Roundabout-jemo and Mikael –sebeta)

USACE (EM 2000-1-2).(2016) United States Army Corps of Engineers Emergency flood fight training manual

USDA, NRCS – TR-55 (1986) United States Development of Agriculture and Natural Resource Conservation Service Technical release 55

USDA, NRCS, National Water and Climate Center, 2001

USGS (w.w.w. earthexplorer.usgs.gov)

(Ujas Pandya, Anant Patel, Dhruvesh), (2017) Patel River cross section delineation from the google earth for development of 1D HEC RAS model A case of Sabarmati river, Gujarat, India

Appendices

I.Frequency analysis using Gumbel method

| No. | Year | Percepitation |
|---|---|---|
| 1 | 1990 | 37 |
| 2 | 1991 | 59.6 |
| 3 | 1992 | 44.3 |
| 4 | 1993 | 40.6 |
| 5 | 1994 | 38.2 |
| 6 | 1995 | 64.7 |
| 7 | 1996 | 52 |
| 8 | 1997 | 37.3 |
| 9 | 1998 | 60.1 |
| 10 | 1999 | 37.8 |
| 11 | 2000 | 47 |
| 12 | 2001 | 32.4 |
| 13 | 2002 | 28.6 |
| 14 | 2003 | 34.6 |
| 15 | 2004 | 29 |
| 16 | 2005 | 44.5 |
| 17 | 2006 | 61.7 |
| 18 | 2007 | 71.2 |
| 19 | 2008 | 37.2 |
| 20 | 2009 | 51.2 |
| 21 | 2010 | 54.4 |
| 22 | 2011 | 36.9 |
| 23 | 2012 | 64.7 |
| 24 | 2013 | 42.6 |
| 25 | 2014 | 27.2 |
| 26 | 2015 | 60.5 |
| 27 | 2016 | 33 |
| 28 | 2017 | 47.9 |
| 29 | 2018 | 60 |
| 30 | 2019 | 49.2 |
| 31 | 2020 | 213.2 |

| | | |
|---|---|---|
| N (Number of Years) | 31 | |
| Mean | 51.56774194 | |
| Standard Devation (Sd) | 32.36985622 | |
| A | Sd*(6/π)^0.5 | 44.74572729 |
| U | mean - 0.5772*α | 25.74050814 |
| | 32.36985622 | |
| Yt - | Ln(Ln(T/T-1)) | |
| Xt | u + α*Yt | |

| T | Yt | Xt |
|---|---|---|
| 2 | 0.366512921 | 42.14039534 |
| 5 | 1.499939987 | 92.85641375 |
| 10 | 2.250367327 | 126.4348309 |
| 25 | 3.198534261 | 168.8612499 |
| 50 | 3.901938658 | 200.3355912 |
| 100 | 4.600149227 | 231.5775309 |

II.Frequency Analysis using Log Pearson III

| Year | Max Percepitation | Log Y | (LogY-Log avg)^3 |
|---|---|---|---|
| 1990 | 37.0 | 1.5682 | -0.0011 |
| 1991 | 59.6 | 1.7752 | 0.0011 |
| 1992 | 44.3 | 1.6464 | 0.0000 |
| 1993 | 40.6 | 1.6085 | -0.0002 |
| 1994 | 38.2 | 1.5821 | -0.0007 |
| 1995 | 64.7 | 1.8109 | 0.0027 |
| 1996 | 52.0 | 1.7160 | 0.0001 |
| 1997 | 37.3 | 1.5717 | -0.0010 |
| 1998 | 60.1 | 1.7789 | 0.0013 |
| 1999 | 37.8 | 1.5775 | -0.0008 |
| 2000 | 47.0 | 1.6721 | 0.0000 |
| 2001 | 32.4 | 1.5105 | -0.0041 |
| 2002 | 28.6 | 1.4564 | -0.0099 |
| 2003 | 34.6 | 1.5391 | -0.0023 |
| 2004 | 29.0 | 1.4624 | -0.0091 |
| 2005 | 44.5 | 1.6484 | 0.0000 |
| 2006 | 61.7 | 1.7903 | 0.0017 |
| 2007 | 71.2 | 1.8525 | 0.0060 |
| 2008 | 37.2 | 1.5705 | -0.0010 |
| 2009 | 51.2 | 1.7093 | 0.0001 |
| 2010 | 54.4 | 1.7356 | 0.0003 |
| 2011 | 36.9 | 1.5670 | -0.0011 |
| 2012 | 64.7 | 1.8109 | 0.0027 |
| 2013 | 42.6 | 1.6294 | -0.0001 |
| 2014 | 27.2 | 1.4346 | -0.0132 |
| 2015 | 60.5 | 1.7818 | 0.0014 |
| 2016 | 33.0 | 1.5185 | -0.0036 |
| 2017 | 47.9 | 1.6803 | 0.0000 |
| 2018 | 60.0 | 1.7782 | 0.0012 |
| 2019 | 49.2 | 1.6920 | 0.0000 |
| 2020 | 213.2 | 2.3288 | 0.2845 |

| | |
|---|---|
| N (Number of years) | 31 |
| Mean | 1.67 |
| Standard Devation | 0.17 |
| K | 0.73 |
| ∑(LogY-Log avg)^3 | 0.25 |
| cs | 1.90 |

| cs | 1 | 2 | 5 | 10 | 25 | 50 | 100 |
|---|---|---|---|---|---|---|---|
| 2 | -0.99 | -0.307 | 0.609 | 1.302 | 2.219 | 2.912 | 3.605 |
| 1.9 | -1.037 | -0.294 | 0.627 | 1.31 | 2.207 | 2.881 | 3.553 |

| T | z | kt | Yt | Xt (Expected Rain fall) |
|---|---|---|---|---|
| 2 | 0 | -0.294 | 1.6216 | 41.8411 |
| 5 | -0.84162 | 0.627 | 1.7766 | 59.7909 |
| 10 | -1.28155 | 1.31 | 1.8916 | 77.9121 |
| 25 | -1.75069 | 2.207 | 2.0426 | 110.3056 |
| 50 | -2.05375 | 2.881 | 2.1561 | 143.2360 |
| 100 | -2.32635 | 3.553 | 2.2692 | 185.8532 |

III.Test of goodness for Gumbel

| Year | Observed Rainfall | Rank | P excedence | T | YT = -Ln(Ln(T/T-1) | Xt |
|---|---|---|---|---|---|---|
| 1990 | 213.2 | 1 | 0.03125 | 32.00 | 3.45 | 180.1 |
| 1991 | 71.2 | 2 | 0.0625 | 16.00 | 2.74 | 148.4 |
| 1992 | 64.7 | 3 | 0.09375 | 10.67 | 2.32 | 129.5 |
| 1993 | 64.7 | 3 | 0.09375 | 10.67 | 2.32 | 129.5 |
| 1994 | 61.7 | 5 | 0.15625 | 6.40 | 1.77 | 105.1 |
| 1995 | 60.5 | 6 | 0.1875 | 5.33 | 1.57 | 96.1 |
| 1996 | 60.1 | 7 | 0.21875 | 4.57 | 1.40 | 88.3 |
| 1997 | 60 | 8 | 0.25 | 4.00 | 1.25 | 81.5 |
| 1998 | 59.6 | 9 | 0.28125 | 3.56 | 1.11 | 75.3 |
| 1999 | 54.4 | 10 | 0.3125 | 3.20 | 0.98 | 69.7 |
| 2000 | 52 | 11 | 0.34375 | 2.91 | 0.86 | 64.4 |
| 2001 | 51.2 | 12 | 0.375 | 2.67 | 0.76 | 59.5 |
| 2002 | 49.2 | 13 | 0.40625 | 2.46 | 0.65 | 54.9 |
| 2003 | 47.9 | 14 | 0.4375 | 2.29 | 0.55 | 50.5 |
| 2004 | 47 | 15 | 0.46875 | 2.13 | 0.46 | 46.2 |
| 2005 | 44.5 | 16 | 0.5 | 2.00 | 0.37 | 42.1 |
| 2006 | 44.3 | 17 | 0.53125 | 1.88 | 0.28 | 38.2 |
| 2007 | 42.6 | 18 | 0.5625 | 1.78 | 0.19 | 34.3 |
| 2008 | 40.6 | 19 | 0.59375 | 1.68 | 0.10 | 30.4 |
| 2009 | 38.2 | 20 | 0.625 | 1.60 | 0.02 | 26.6 |
| 2010 | 37.8 | 21 | 0.65625 | 1.52 | -0.07 | 22.8 |
| 2011 | 37.3 | 22 | 0.6875 | 1.45 | -0.15 | 19.0 |
| 2012 | 37.2 | 23 | 0.71875 | 1.39 | -0.24 | 15.1 |
| 2013 | 37 | 24 | 0.75 | 1.33 | -0.33 | 11.1 |
| 2014 | 36.9 | 25 | 0.78125 | 1.28 | -0.42 | 7.0 |
| 2015 | 34.6 | 26 | 0.8125 | 1.23 | -0.52 | 2.7 |
| 2016 | 33 | 27 | 0.84375 | 1.19 | -0.62 | |
| 2017 | 32.4 | 28 | 0.875 | 1.14 | -0.73 | |
| 2018 | 29 | 29 | 0.90625 | 1.10 | -0.86 | |
| 2019 | 28.6 | 30 | 0.9375 | 1.07 | -1.02 | |
| 2020 | 27.2 | 31 | 0.96875 | 1.03 | -1.24 | |

| | | |
|---|---|---|
| N (NUMBER OF YEARS) | 31 | |
| MEAN (Ẋ) | 51.56774194 | |
| STANDARD DEVIATION (Sd) | 32.36985622 | |
| α | Sd*(6/π)^0.5 | 44.7 |
| u | Ẋ - 0.5772*α | 25.7 |
| | | |
| Yt - | Ln(Ln(T/T-1)) | |
| Xt | u + α*Yt | |

| | |
|---|---|
| Yn (Average value of the variante) | 0.546962993 |
| Snc(standard devation of the variant) | 1.148818918 |
| Ẋ (average value of the observed rainfall) | 51.56774194 |
| sd (standard devation of the observed rainfall) | 32.36985622 |
| Cofficent of Determination ($R^2$ ) | 0.761855271 |

IV.Test of goodness Log pearsom typeIII

| YEAR | Observed rainfall | RF descendimg | Log RF | Rank | P excedence | P2 | w | z | Kt | Yt | Xt |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1,990 | 37.00 | 213.2 | 2.33 | 1.00 | 0.03 | | 2.63 | 1.86 | 2.61 | 2.11 | 129.08 |
| 1,991 | 59.60 | 71.2 | 1.85 | 2.00 | 0.06 | | 2.35 | 1.53 | 1.90 | 1.99 | 97.82 |
| 1,992 | 44.30 | 64.7 | 1.81 | 3.00 | 0.09 | | 2.18 | 1.32 | 1.48 | 1.92 | 83.30 |
| 1,993 | 40.60 | 64.7 | 1.81 | 3.00 | 0.09 | | 2.18 | 1.32 | 1.48 | 1.92 | 83.30 |
| 1,994 | 38.20 | 61.7 | 1.79 | 5.00 | 0.16 | | 1.93 | 1.01 | 0.96 | 1.83 | 68.13 |
| 1,995 | 64.70 | 60.5 | 1.78 | 6.00 | 0.19 | | 1.83 | 0.89 | 0.78 | 1.80 | 63.43 |
| 1,996 | 52.00 | 60.1 | 1.78 | 7.00 | 0.22 | | 1.74 | 0.78 | 0.62 | 1.78 | 59.71 |
| 1,997 | 37.30 | 60.0 | 1.78 | 8.00 | 0.25 | | 1.67 | 0.67 | 0.49 | 1.75 | 56.67 |
| 1,998 | 60.10 | 59.6 | 1.78 | 9.00 | 0.28 | | 1.59 | 0.58 | 0.37 | 1.73 | 54.12 |
| 1,999 | 37.80 | 54.4 | 1.74 | 10.00 | 0.31 | | 1.53 | 0.49 | 0.26 | 1.72 | 51.94 |
| 2,000 | 47.00 | 52.0 | 1.72 | 11.00 | 0.34 | | 1.46 | 0.40 | 0.17 | 1.70 | 50.04 |
| 2,001 | 32.40 | 51.2 | 1.71 | 12.00 | 0.38 | | 1.40 | 0.32 | 0.08 | 1.68 | 48.36 |
| 2,002 | 28.60 | 49.2 | 1.69 | 13.00 | 0.41 | | 1.34 | 0.24 | 0.00 | 1.67 | 46.87 |
| 2,003 | 34.60 | 47.9 | 1.68 | 14.00 | 0.44 | | 1.29 | 0.16 | -0.08 | 1.66 | 45.53 |
| 2,004 | 29.00 | 47.0 | 1.67 | 15.00 | 0.47 | | 1.23 | 0.08 | -0.15 | 1.65 | 44.31 |
| 2,005 | 44.50 | 44.5 | 1.65 | 16.00 | 0.50 | | 1.18 | 0.00 | -0.21 | 1.64 | 43.20 |
| 2,006 | 61.70 | 44.3 | 1.65 | 17.00 | 0.53 | 0.47 | -1.23 | -0.08 | -0.27 | 1.63 | 42.18 |
| 2,007 | 71.20 | 42.6 | 1.63 | 18.00 | 0.56 | 0.44 | -1.29 | -0.16 | -0.33 | 1.62 | 41.25 |
| 2,008 | 37.20 | 40.6 | 1.61 | 19.00 | 0.59 | 0.41 | -1.34 | -0.24 | -0.39 | 1.61 | 40.38 |
| 2,009 | 51.20 | 38.2 | 1.58 | 20.00 | 0.63 | 0.38 | -1.40 | -0.32 | -0.44 | 1.60 | 39.57 |
| 2,010 | 54.40 | 37.8 | 1.58 | 21.00 | 0.66 | 0.34 | -1.46 | -0.40 | -0.49 | 1.59 | 38.81 |
| 2,011 | 36.90 | 37.3 | 1.57 | 22.00 | 0.69 | 0.31 | -1.53 | -0.49 | -0.53 | 1.58 | 38.11 |
| 2,012 | 64.70 | 37.2 | 1.57 | 23.00 | 0.72 | 0.28 | -1.59 | -0.58 | -0.58 | 1.57 | 37.45 |
| 2,013 | 42.60 | 37.0 | 1.57 | 24.00 | 0.75 | 0.25 | -1.67 | -0.67 | -0.62 | 1.57 | 36.83 |
| 2,014 | 27.20 | 36.9 | 1.57 | 25.00 | 0.78 | 0.22 | -1.74 | -0.78 | -0.66 | 1.56 | 36.25 |
| 2,015 | 60.50 | 34.6 | 1.54 | 26.00 | 0.81 | 0.19 | -1.83 | -0.89 | -0.70 | 1.55 | 35.71 |
| 2,016 | 33.00 | 33.0 | 1.52 | 27.00 | 0.84 | 0.16 | -1.93 | -1.01 | -0.74 | 1.55 | 35.21 |
| 2,017 | 47.90 | 32.4 | 1.51 | 28.00 | 0.88 | 0.13 | -2.04 | -1.15 | -0.77 | 1.54 | 34.75 |
| 2,018 | 60.00 | 29.0 | 1.46 | 29.00 | 0.91 | 0.09 | -2.18 | -1.32 | -0.80 | 1.54 | 34.34 |
| 2,019 | 49.20 | 28.6 | 1.46 | 30.00 | 0.94 | 0.06 | -2.35 | -1.53 | -0.83 | 1.53 | 34.03 |
| 2,020 | 213.20 | 27.2 | 1.43 | 31.00 | 0.97 | 0.03 | -2.63 | -1.86 | -0.83 | 1.53 | 33.93 |

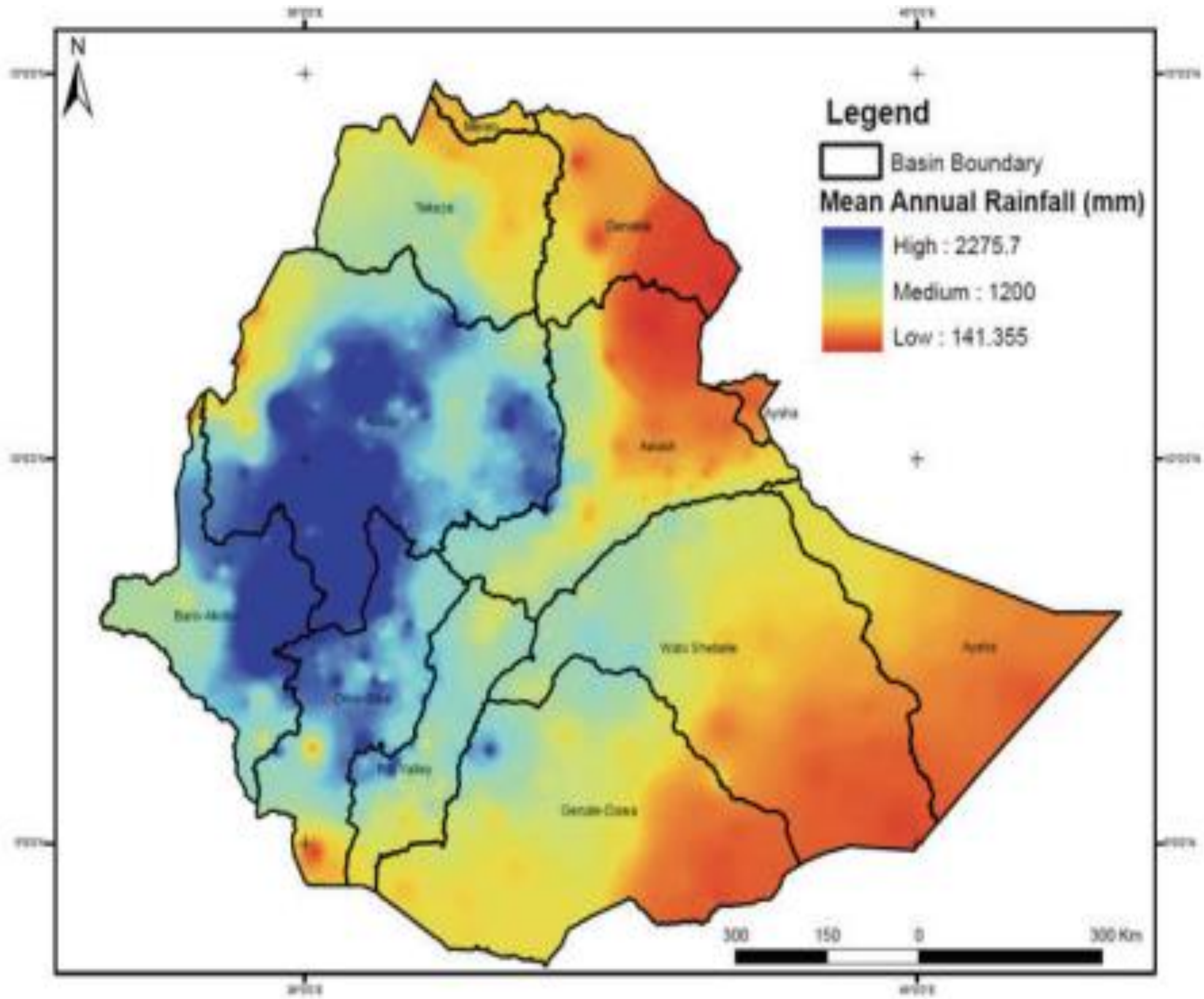
| | |
|---|---|
| Sy (Standrd devation of the Log RF) | 0.16833 |
| Yn (acerage value of Log RF) | 1.671092 |
| Cs (skew of Log RF) | 1.902859 |
| k (Cs/6) | 0.317143 |
| Cofficent of Determination ($R^2$ ) | 0.875 |

V. Frequency factor K

**Frequency Factors K for Gamma and log-Pearson Type III Distributions (Haan, 1977, Table 7.7)**

| | Recurrence Interval In Years | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| **WEIGHTED** | **1.0101** | **2** | **5** | **10** | **25** | **50** | **100** | **200** |
| **SKEW COEFFICIENT** | **Percent Chance (>=) = 1-F** | | | | | | | |
| **Cw** | **99** | **50** | **20** | **10** | **4** | **2** | **1** | **0.5** |
| **3** | -0.667 | -0.396 | 0.42 | 1.18 | 2.278 | 3.152 | 4.051 | 4.97 |
| **2.9** | -0.69 | -0.39 | 0.44 | 1.195 | 2.277 | 3.134 | 4.013 | 4.904 |
| **2.8** | -0.714 | -0.384 | 0.46 | 1.21 | 2.275 | 3.114 | 3.973 | 4.847 |
| **2.7** | -0.74 | -0.376 | 0.479 | 1.224 | 2.272 | 3.093 | 3.932 | 4.783 |
| **2.6** | -0.769 | -0.368 | 0.499 | 1.238 | 2.267 | 3.071 | 3.889 | 4.718 |
| **2.5** | -0.799 | -0.36 | 0.518 | 1.25 | 2.262 | 3.048 | 3.845 | 4.652 |
| **2.4** | -0.832 | -0.351 | 0.537 | 1.262 | 2.256 | 3.023 | 3.8 | 4.584 |
| **2.3** | -0.867 | -0.341 | 0.555 | 1.274 | 2.248 | 2.997 | 3.753 | 4.515 |
| **2.2** | -0.905 | -0.33 | 0.574 | 1.284 | 2.24 | 2.97 | 3.705 | 4.444 |
| **2.1** | -0.946 | -0.319 | 0.592 | 1.294 | 2.23 | 2.942 | 3.656 | 4.372 |
| **2** | -0.99 | -0.307 | 0.609 | 1.302 | 2.219 | 2.912 | 3.605 | 4.298 |
| **1.9** | -1.037 | -0.294 | 0.627 | 1.31 | 2.207 | 2.881 | 3.553 | 4.223 |
| **1.8** | -1.087 | -0.282 | 0.643 | 1.318 | 2.193 | 2.848 | 3.499 | 4.147 |
| **1.7** | -1.14 | -0.268 | 0.66 | 1.324 | 2.179 | 2.815 | 3.444 | 4.069 |
| **1.6** | -1.197 | -0.254 | 0.675 | 1.329 | 2.163 | 2.78 | 3.388 | 3.99 |
| **1.5** | -1.256 | -0.24 | 0.69 | 1.333 | 2.146 | 2.743 | 3.33 | 3.91 |
| **1.4** | -1.318 | -0.225 | 0.705 | 1.337 | 2.128 | 2.706 | 3.271 | 3.828 |
| **1.3** | -1.383 | -0.21 | 0.719 | 1.339 | 2.108 | 2.666 | 3.211 | 3.745 |
| **1.2** | -1.449 | -0.195 | 0.732 | 1.34 | 2.087 | 2.626 | 3.149 | 3.661 |
| **1.1** | -1.518 | -0.18 | 0.745 | 1.341 | 2.066 | 2.585 | 3.087 | 3.575 |
| **1** | -1.588 | -0.164 | 0.758 | 1.34 | 2.043 | 2.542 | 3.022 | 3.489 |
| **0.9** | -1.66 | -0.148 | 0.769 | 1.339 | 2.018 | 2.498 | 2.957 | 3.401 |
| **0.8** | -1.733 | -0.132 | 0.78 | 1.336 | 1.993 | 2.453 | 2.891 | 3.312 |
| **0.7** | -1.806 | -0.116 | 0.79 | 1.333 | 1.967 | 2.407 | 2.824 | 3.223 |
| **0.6** | -1.88 | -0.099 | 0.8 | 1.328 | 1.939 | 2.359 | 2.755 | 3.132 |
| **0.5** | -1.955 | -0.083 | 0.808 | 1.323 | 1.91 | 2.311 | 2.686 | 3.041 |
| **0.4** | -2.029 | -0.066 | 0.816 | 1.317 | 1.88 | 2.261 | 2.615 | 2.949 |
| **0.3** | -2.104 | -0.05 | 0.824 | 1.309 | 1.849 | 2.211 | 2.544 | 2.856 |
| **0.2** | -2.178 | -0.033 | 0.83 | 1.301 | 1.818 | 2.159 | 2.472 | 2.763 |
| **0.1** | -2.252 | -0.017 | 0.836 | 1.292 | 1.785 | 2.107 | 2.4 | 2.67 |
| **0** | -2.326 | 0 | 0.842 | 1.282 | 1.751 | 2.054 | 2.326 | 2.576 |
| **-0.1** | -2.4 | 0.017 | 0.846 | 1.27 | 1.716 | 2 | 2.252 | 2.482 |
| **-0.2** | -2.472 | 0.033 | 0.85 | 1.258 | 1.68 | 1.945 | 2.178 | 2.388 |
| **-0.3** | -2.544 | 0.05 | 0.853 | 1.245 | 1.643 | 1.89 | 2.104 | 2.294 |
| **-0.4** | -2.615 | 0.066 | 0.855 | 1.231 | 1.606 | 1.834 | 2.029 | 2.201 |
| **-0.5** | -2.686 | 0.083 | 0.856 | 1.216 | 1.567 | 1.777 | 1.955 | 2.108 |
| **-0.6** | -2.755 | 0.099 | 0.857 | 1.2 | 1.528 | 1.72 | 1.88 | 2.016 |
| **-0.7** | -2.824 | 0.116 | 0.857 | 1.183 | 1.488 | 1.663 | 1.806 | 1.926 |
| **-0.8** | -2.891 | 0.132 | 0.856 | 1.166 | 1.448 | 1.606 | 1.733 | 1.837 |
| **-0.9** | -2.957 | 0.148 | 0.854 | 1.147 | 1.407 | 1.549 | 1.66 | 1.749 |
| **-1** | -3.022 | 0.164 | 0.852 | 1.128 | 1.366 | 1.492 | 1.588 | 1.664 |
| **-1.1** | -3.087 | 0.18 | 0.848 | 1.107 | 1.324 | 1.435 | 1.518 | 1.581 |
| **-1.2** | -3.149 | 0.195 | 0.844 | 1.086 | 1.282 | 1.379 | 1.449 | 1.501 |
| **-1.3** | -3.211 | 0.21 | 0.838 | 1.064 | 1.24 | 1.324 | 1.383 | 1.424 |
| **-1.4** | -3.271 | 0.225 | 0.832 | 1.041 | 1.198 | 1.27 | 1.318 | 1.351 |
| **-1.5** | -3.33 | 0.24 | 0.825 | 1.018 | 1.157 | 1.217 | 1.256 | 1.282 |
| **-1.6** | -3.88 | 0.254 | 0.817 | 0.994 | 1.116 | 1.166 | 1.197 | 1.216 |
| **-1.7** | -3.444 | 0.268 | 0.808 | 0.97 | 1.075 | 1.116 | 1.14 | 1.155 |
| **-1.8** | -3.499 | 0.282 | 0.799 | 0.945 | 1.035 | 1.069 | 1.087 | 1.097 |
| **-1.9** | -3.553 | 0.294 | 0.788 | 0.92 | 0.996 | 1.023 | 1.037 | 1.044 |
| **-2** | -3.605 | 0.307 | 0.777 | 0.895 | 0.959 | 0.98 | 0.99 | 0.995 |
| **-2.1** | -3.656 | 0.319 | 0.765 | 0.869 | 0.923 | 0.939 | 0.946 | 0.949 |
| **-2.2** | -3.705 | 0.33 | 0.752 | 0.844 | 0.888 | 0.9 | 0.905 | 0.907 |
| **-2.3** | -3.753 | 0.341 | 0.739 | 0.819 | 0.855 | 0.864 | 0.867 | 0.869 |
| **-2.4** | -3.8 | 0.351 | 0.725 | 0.795 | 0.823 | 0.83 | 0.832 | 0.833 |
| **-2.5** | -3.845 | 0.36 | 0.711 | 0.711 | 0.793 | 0.798 | 0.799 | 0.8 |
| **-2.6** | -3.899 | 0.368 | 0.696 | 0.747 | 0.764 | 0.768 | 0.769 | 0.769 |
| **-2.7** | -3.932 | 0.376 | 0.681 | 0.724 | 0.738 | 0.74 | 0.74 | 0.741 |
| **-2.8** | -3.973 | 0.384 | 0.666 | 0.702 | 0.712 | 0.714 | 0.714 | 0.714 |
| **-2.9** | -4.013 | 0.39 | 0.651 | 0.681 | 0.683 | 0.689 | 0.69 | 0.69 |
| **-3** | -4.051 | 0.396 | 0.636 | 0.66 | 0.666 | 0.666 | 0.667 | 0.667 |

VI Spatial variability of the mean annual rainfall in Ethiopia




**Source: Scientific journal on Surface Water and Groundwater Resources of Ethiopia: Potentials and Challenges of Water Resources Development**