

EVALUATION OF URBAN DRAINAGE SYSTEM BY SWMM : A CASE STUDY AT  
ALABA KULITO TOWN, SNNPRS, ETHIOPIA



MASTER OF SCIENCE THESIS

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HAWASSA UNIVERSITY, HAWASSA, ETHIOPIA

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EVALUATION OF URBAN DRAINAGE SYSTEM BY SWMM: A CASE STUDY AT  
ALABA KULITO TOWN, SNNPRS, ETHIOPIA

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**SCHOOL OF GRADUATE STUDIES**  
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Asmachew A.

November 2020, Hawassa University, Ethiopia

## DECLARATION

**I, Asmachew Abera Bemanjo**, declare that this thesis is my own original work and has not been taken from any other sources except such works which have been cited and acknowledged within the text, and that it has not been presented and will not be presented by me to any other university for similar or any other degree awards.

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## **LIST OF ABBREVIATIONS AND ACRONYMS**

BM	Block Maxima
m.a.s.l.	Meters Above Sea Level
CSA	Central Statistic Agency
DEM	Digital Elevation Model
DMC	Double Mass Curve
EPA	Environmental Protection Authority
ERA	Ethiopian Roads Authority
WHSD	World Harmonized Soil Database
SWMM	Stormwater Management Model
GPD	Generalized Pareto Distribution
IDF	Intensity Duration Frequency
JGHP	Joint Government and Humanitarian Partners
MoWEI	Ministry of Water Energy and Irrigation
MAE	Meteorology Agency of Ethiopia
MRLP	Mean Residual Life Plot
NMA	National Meteorological Agency
POT	Peaks-Over-Threshold
SCS-CN	Soil Conservation Service Curve Number
SNNPR	Southern Nations, Nationalities, and Peoples Region
SWC	Soil and Water Conservation
SWMM	Stormwater Management Model
UNEP	United Nations Environmental Program
UNISDR	United Nation International Strategy for Disaster Reduction
WMO	World Meteorological Organization

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## ABSTRACT

*The general objective of the study was Evaluation of Urban Drainage System of Alaba Kulito Town by Storm Water Management Model (SWMM). The study employed the collection of primary data like measuring the existing drainage size of the flood-prone region and asking stakeholders and secondary data which was obtained from National Meteorology Agency (NMA) of Ethiopian (33 years meteorological data), Ethiopian Map Agency (topographic map and soil map data), Ministry of Water, Irrigation and Energy (soil data) and Alaba Town Municipality (historical data of flood and organizational structure of city administration). 3.7% of Missed RF data of Alaba Kulito station was filled by Normal Ratio Method (NRM). The Generalized Pareto Distribution (GPD) method which was followed by Peak Over Threshold (POT) extreme value determination method was used to predict the probability of flood occurrence due to the best fit and approach of study. For analysis of hydrology and hydraulics done by the Soil Conservation Service Curve Number (SCS-CN)/rational method and Stormwater Management Model (SWMM). It has been found that Alaba-Kulito town is geographically nearly plain which was between class 1 and class 4 slope classification and located in foothill which contributes much amount of runoff to the town and some of the drainage lines are incapable to convey runoff generated from rural catchment. In addition to this, limited landscape based mitigation strategies in the study area with insufficiency of drainage canals, limited collector and feeder drainage lines, lack of awareness of community while disposing of household wastes together have worsened the impacts of flooding. The overall result of the study is terminated by distinguishing and pointing both structural methods: diverting the upper catchment(which shares more than 64% runoff load ), providing collector and feeder drainage lines through the flood-prone section of town and constructing a lined canal at the common outlet to Bilate river which is about 1.5km from ST.Gabriel church ; and non-structural managing systems depending on the degree of the flood. The peak runoff load of each junction and nodes are obtained by summing up the runoff magnitude of all upper contributing catchments and accordingly the outlet point received about 49.45m<sup>3</sup>/s and 29.1m<sup>3</sup>/s without diversion work and if diversion work was provided for 10 years return period respectively. The coefficient of correlation between simulated and estimated peak discharge becomes greater than 0.99.*

**Keywords:** Alaba Kulito Town, Urban Drainage, GPD, IDF, SWMM

# **1 INTRODUCTION**

## **1.1 Background of Study**

Natural hazards have caused considerable danger to the progress and development of the human population on the surface of the earth. Some of the natural hazards include volcanic eruption, earthquake, temperature extremes, hurricane, tropical cyclones and flooding. Among the natural hazards, floods hold the leading position in the world (Birehanu, 2018) and in practice, flooding occurs when the volume of water in a waterway exceeds the capacity of the channel.

Biplab and Aditya, (2012); defined flooding as a general and temporary condition of partial or complete inundation of normally dry land areas from the usual and rapid runoff of surface waters which may result from rainfall, rivers, ice melt and so on.

According to Teferi (2016), floods are caused by many factors such as heavy rainfall, highly accelerated snowmelt, severe winds over water, unusual high tide, tsunamis, or failure of dams, levees, retention ponds, or other structures that retained the water and can be intensified by increased amounts of impervious surface or hard ground cover that not allow the water to pass through.

Commonly; flooding is caused by rainfall intensity, duration, soil condition, nature of the topography, ground cover, antecedent moisture condition, climate change and other natural and manmade factors. Construction of buildings and roads without providing adequate side canals, main canals and appropriate outlets in the urban area leads to urban flooding as urbanization results in considerable changes in hydrological processes due to an increase in built-up and decrease in infiltration.

The Global Facility for Disaster Reduction and Recovery, (2019); reports that Ethiopia is one of the African countries which are experiencing flooding which occurs at irregular intervals and varies in duration, magnitude and the area affected by the flood and evidence shows that many flooding events occur due La Nina climate condition commonly preceded by drought effect of El Nino events. The section of the country like Dire Dawa, Afar, Somali, Harari, South Nations Nationalities and Peoples Regional State (SNNPRS), Tigray, Gambella, Amhara, Addis Ababa and Oromia regionals faces either riverine or flash flood at heavy kiremt rainy season (Surafel et al., 2019).

Even though many societies have recognized floods as unavoidable action of nature or acts of God to be tolerated, in modern times the interface between water and society has shaped a changing attitude as control over the physical environment has increased and technology and social organization have made possible successful management of natural resources (Huong and Pathirana, 2013). Therefore, the flood hazard which refers to the damaging force produced by flooding and which can affect the exposed location, building, people, critical facilities and transport lines located in an area that may be subjected to a hazard event can be minimized and managed by human intervention.

## **1.2 Statement of the Problem**

Vegetation loss, land degradation, soil erosion, overutilization of fuelwood and rainy season flooding and climate change are some of the environmental problems of the world. These problems can magnify flooding which is a global issue and destructive natural event affecting both rural and urban settlements as well as developed and developing countries.

As Myers (2016) and Taubenbock et al. (2011), cited in Nigussie and Altunkaynak (2019), flooding is known to result in both short-term and long-term impacts in various parts of the world by causing human life and property losses accounting for about 43% of the recorded events, which has been the most common natural disaster by far in the last 20 years.

Historically, Ethiopia had extensive experiences in both urban flooding and riverine flooding. The identified historical flood event years are 1988, 1996, 1997, 1998, 2004, 2005, 2006, 2010, 2012, 2013 and 2016 from which 1997, 2005 and 2013 are the most flood years with 16.7%, 15.7% and 13.9% area of the country affected by flood events respectively (Surafel et al., 2019).

Studies reveal that the increase in artificial surfaces due to urbanization increases flooding frequency and intensity because of the resulting poor infiltration and reduction in flow resistance and it is an accelerating trend throughout the world (Zhang et al., 2007). Likewise, the portion of permeable land of Alaba Kulito Town is changing to impermeable surfaces due to the increasing of population number and built-up which aggravates the occurrence of urban flooding with the combined effect of hydrologic condition and hydraulics of existing drainage system.

During intense rainfall of Kiremt season (July to September), flooding is suspected at the town section as the town is geographical located between Rekame Hill and Bilate River which has been

historically affecting the town resulting damage to properties and inundation of stormwater on flat areas malfunctions the road and reduce the aesthetic view of the town since the establishment of the town. Hence, the climatic change and expansion of urbanization are not able to be avoided, we need to control nature and to shape the environment we are living in.

It is obvious that relative to rural areas, urban environments have high concentrations of people and valuable assets. To sustain the development of the town, to keep these valuable assets, to save the life of people and to make the town less threat of flooding; we need to solve urban flood catastrophes and build a favorable town by doing such problem solving scientific researches.

This study is an initial effort to quantify the flood severity of design stormwater for both existing drainage systems and the future plans of flood controlling mechanisms at Alaba Kulito Town by SWMM. This model is selected as it is combined hydrologic and hydraulics software to simulate urban flooding and is widely favored by researchers for urban stormwater drainage systems (Dawe et al., 2019) than other models.

### **1.3 Objectives**

#### **1.3.1 General Objective**

The main objective of this research is to assess the flood hazard and drainage system of Alaba Kulito Town using Stormwater Management Model (SWMM).

#### **1.3.2 Specific Objectives**

- To develop intensity duration frequency (IDF) curve for return periods of 2, 5, 10, 25, 50 and 100 years.
- To assess the hydraulic capacity of the existing drainage system in the study area and to identify drainage lines which are insufficient to convey the stormwater.
- To identify the causes of flood at Alaba Kulito Town.
- To propose appropriate flood hazard mitigation methods.

### **1.4 Research Questions**

The following questions are formulated based on the above research objectives:

- What will be the intensity of rainfall for the different duration and return periods' of 2, 5, 10, 25, 50, and 100 years of Alaba Kulito Town?
- Are the existing drainage lines capable enough to convey generated runoff?

- What are the main causes of the flood of Alaba Kulito Town?
- What should be done to keep the town from flood hazards?

### **1.5 Significance of Research**

This study provides basic information for different concerned bodies including local communities, governmental and non-governmental organizations. This helps to formulate land use policy, to increase the size of the future design of ditches, to regulate any debris filled in existing ditches and catchment management of Alaba Kulito Town and the rural area contributing runoff to the town.

Moreover, this study will act as an input for the long term drainage and watershed management planning and flood control practices of Alaba Kulito Town. Data derived from the rainfall intensity-duration-frequency curves will be needed by hydrologists and engineers involving in the planning and design of water resources projects.

The present study therefore will provide useful design data for water resources development in the study area. This study employs different methodologies especially the SWMM which have an important contribution to the academic societies in making further studies and other related investigations.

### **1.6 Scope and Limitation of the Study**

The study is limited by producing the IDF curve of Alaba Kulito, evaluating existing drainage systems, identifying causes of flooding, and proposing proper flood mitigation methods. However; these are a wide concept and due to difficulty to cover the whole Kulito Town, only Lenda Ber and Mahal Arada Kebele (kebeles of Kulito town which suffers flooding more than the other kebeles) with a total area of 1717.46ha is considered in this study. The model result was simulated without calibration as there is no recorded data of runoff at outfall and junctions which is a common problem of developing countries as a whole and to adjust sensitive parameters of the SWMM model the peak discharge result from both Soil Conservation Service Curve Number method and Rational method are used.

## **2 LITERATURE REVIEW**

### **2.1 The Concept of Flooding**

Flooding occurs when channels are filled with water beyond their capacity and results in excess water pouring out of the natural or manmade waterway into the adjacent floodplain. Among many definitions of flooding (Biplab and Aditya, 2012), defined flooding as ‘Flooding is a general and temporary condition of partial or complete inundation of normally dry land areas from the usual and rapid runoff of surface waters which may results from rainfall, rivers, ice melt and so on’. Furthermore, Birehanu (2018), also defined flooding is ‘the accumulation of water within a water body and the overflow of excess water on to adjacent flood plains, or it is an overflow of inland or tidal waters, unusual and rapid accumulation of runoff or surface waters from any source’.

Flood occurs when water ways have no a capacity to hold additional water due to sedimentation or other related factors, and also it occurs when soil and vegetation cannot absorb or infiltrate all the water may come from rainfall or other sources (Gezahegn, 2013).

### **2.2 Forms of Floods in Ethiopia**

Flooding can be classified into several types. Out of those river floods and urban floods are the most common in Ethiopia. The flood types are explained below.

#### **2.2.1 River floods**

River floods can be further classified by the rapidity of flooding or by the magnitude of flooding. In the first classification; a flood can either be a flash flood or a normal flood. Flash floods occur in mountainous areas or in urban cities founded near mountains, with high slopes and shallow soil depths and are caused by high intense of rainfall. In the case of normal flood, the river rises gradually and gives time for the people before arrival of the flood (Fisha, 2015).

#### **2.2.2 Urban Flooding**

Hence, this research is specific to urban households’ vulnerability to flooding hazards in town, it is worth to discuss issues related to urban flooding. Flooding in urban areas happens when the size of drainage system is not able to convey the coming runoff generated from the upper sub-catchment contributing to the drainage and it vary considerably in size and duration. Field scale flooding is usually due to intense local storms where water and soil can flow straight off the land surface. With prolonged raining over wide areas channels are fed by a network of manmade

and/or natural drains and flows build up to the point where the normal channel overcome and water floods onto surrounding areas (Daniel, 2007).

In this regard, as Mulugeta (2016), cited in Selamawit (2018), the major causes of flooding especially in an urban environment are those drainage areas, constrictions, obstruction (bridges and culverts) debris contamination soil saturation, velocity, topography, ground cover and size are the major ones.

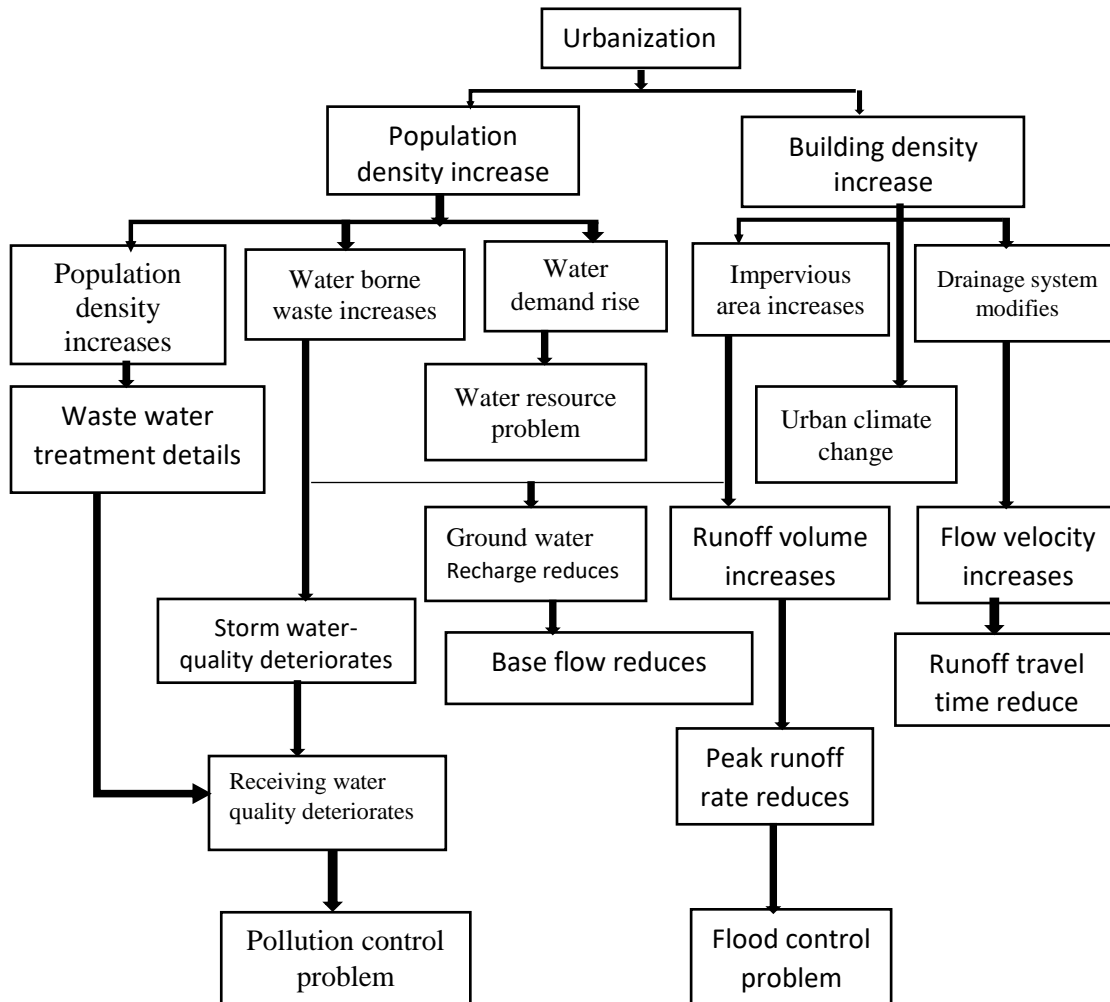


Figure 2.1: Hydrological impacts of urbanization

Source: [https://www.researchgate.net/figure/Causes-of-Urban-Flooding\\_fig2\\_309767636](https://www.researchgate.net/figure/Causes-of-Urban-Flooding_fig2_309767636)

Therefore, urban drainage systems are needed in all urban areas because of their interaction between human daily activity and the natural water circulation.

## **2.3 Flood History**

### **2.3.1 Flood History of the World**

According to O'Connor, et al., (2004), two complimentary types of world's largest flood information: first is a compilation of the world's largest known Quaternary floods (The Quaternary Period extends from about 1.8 million years ago to the present) are known only from geologic evidence and resulted from special circumstances during the course of Earth's history. Nevertheless, the records of such floods shed light on the great diversity and scales of flood-producing mechanisms and their particular settings on earth and over geologic time. The second source of information is historical measurements of the largest meteorological floods on the largest river basins in the world. These floods, which are more within the realm of day-to-day human experience, provide a background for discussing the geologic, climatologic, and physiographic settings of large meteorological floods on a global basis.

The report of O'Connor, et al., (2004), also reveals that the largest floods of the quaternary period evacuates about 500,000 m<sup>3</sup>/s of flood occurs during Quaternary time resulted from rapid release of water stored behind natural dams; Floods from Ice-Dammed Lakes of 1986 resulting from the failure of the dam formed by Hubbard Glacier across Russell Fiord, Alaska with estimated discharge of 105,000 m<sup>3</sup>/s; flood caused by volcanism of about 300,000 m<sup>3</sup>/s, which accompanied a subglacial volcanic eruption at Katla Iceland in 1918; Extreme ice-jam floods are most common in high altitude, north-flowing continental river systems like the Lena River and adjacent large river systems of northern Eurasia, and the MacKenzie River of North America; large meteorological floods of 1953, 1963, and 1976 on the Amazon River at Obidos, Brazil, and the 1870 flood of 110,000 m<sup>3</sup>/s on the Yangtze River of China and other flood events have been occurred.

### **2.3.2 Flood History of Africa**

Demographic distribution, hydro-climatic condition, and economic setting of African continent worsens the effect of flood on socio economic development and flood related losses in Africa have increased dramatically which needs analyzing of large, consistent, and reliable dataset of flood in Africa (Giuliano et al., 2015).

During the last 50 years, population growth and urban development of flood prone areas have been observed to cause a dramatic increase of flood induced fatalities and economical losses in Africa

(Dibaldassarre et al., 2010b). Many regions in Africa are presently faced with an increasing flood risk due to impending climate change and population growth. One useful mitigation strategy to decrease this risk would be to map it, so that urban planning, warnings systems and emergency response subsequently could be designed to reduce societal vulnerability. This is, however, not widely feasible on the African continent, as developing countries often lack access to the topography and discharge data required to produce high quality flood risk maps.

### **2.3.2.1 Torrential rains and flooding of 2009**

Torrential rains and flooding affected 600,000 people in 16 West African nations in September 2009 in which the worst hit countries were Burkina Faso, Senegal, Ghana and Niger (Giuliano et al., 2015). This event closely followed the 2007 floods that displaced more than a million people in Uganda, Ethiopia, Sudan, Burkina Faso, Togo, Mali and Niger, and claimed over 500 lives, and the 2008 flooding in Mozambique (United Nations, 2009).

### **2.3.2.2 The 2000 and 2013 Flood Events of Mozambique**

Mozambique has a long record of being affected by water related disasters and is ranked the most flood prone country in southern Africa (Venton et al., 2013). The reasons behind the many flood disasters are the region's exposure and vulnerability to tropical cyclone activity, intensive local rainfall and poor management of upstream reservoirs (INGC, 2003). The consequences of the flood was devastating, with reports of 800 casualties, 4500 000 people affected and widespread destruction to buildings and infrastructure (CRED, 2016). In monetary terms, the losses were estimated to a cost of 600 million US\$, a sum that at the time corresponded to 20 % of Mozambique's GDP (INGC, 2003).

The flood in 2013 had an inundation extent similar to the 2000 flood, but its dynamic character was very different (Venton et al., 2013). Heavy rain in January 2013 created a flood peak with a steep rise and fast recession. The overall duration of the flood disaster in 2013 was therefore significantly shorter than the flood in 2000, which was caused by multiple cyclones. The impact of the flood in 2013 was also different, since the experience 13 years earlier had motivated developments in the flood disaster management of the area.

### **2.3.3 Flood History of Ethiopia**

Owing to its topographic and altitudinal characteristics, flooding as a natural phenomenon is not new to Ethiopia. It have been occurring at different places and times with varying magnitude.

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

### **2.3.3.1 Flooding in Gambela Region**

The lowlands of Gambela region, located in the southwest of the country, are among the most flood prone areas and overflow of the Baro River create favorable conditions for crop cultivation and livestock keeping, but in the case of an extreme flood, it threatens the lives and livelihoods of the people with regard to the flood frequency and magnitude in the region has increased during the last decade (Abaya et al., 2009).

Gambela, which is likely to increase the frequency of extreme floods in in the future and Major recent floods in Gambela include those in 2006, 2007, 2010, 2011 and 2012, causing the loss of lives and livelihoods with Climate models predicting an increase in rainfall extremes over East (Alemseged et al., 2013).

### **2.3.3.2 Dire Dawa Town Flood History**

Dire Dawa lying at the foot of a mountain range is subjected to annual flooding by runoff from the mountain during torrential rains and the study reveals that from the records the last three major flood event occurred during in April 1981, 1994 and the flood which occurred in May 2005 has caused loss of 35 human lives as well as an estimated amount of 10 million birr damages to property (Daniel, 2007).

The flood history is changed and the most devastating flood in the history of Dire Dawa occurred on the fifth day of August, 2006 is the worst of its kind which resulted for the loss of more than 240 human lives and Property damages is also more than ever before (Ephrem, 2006). This catastrophe draws government attention for proper planning and immediate response for Dire Dawa Urban flood alleviation. The administration of the city in collaboration with neighboring regions has plans for watershed management programs to be implemented during the national water sector development program. The planned watershed management programs comprise construction of check-dams and weirs for water conservation and retaining floods, construction of

terraces along mountain slopes to reduce runoff and encourage ground water recharge, and re-forestation along the slopes draining towards Dire Dawa (Daniel, 2007).

### 2.3.3.3 The 2006 Flood Disaster in Ethiopia

Floods are relatively common in Ethiopia during the rainy season between June and September when heavy rains from the highlands flow into the low lands. On 6 August 2006, the Dechatu River burst its banks in the city of Dire Dawa near Somalia. Home to about 400,000 people, it is Ethiopia's sixth-largest city, about 525 kilometers east of the capital Addis Ababa. Entire buildings were destroyed in some cases and sweeping away homes, trees and fences. Altogether, the devastating floods left close to 20,000 people homeless in eastern Ethiopia. The Omo River, which flows into Kenya's Lake Turkana, burst its banks between 8 and 13 August 2006. Extensive flooding occurred in one district of south Omo zone with heavy loss of lives and property. Around 14 villages are flooded and cut off. 364 people are dead. Generally, more than 650 people are dead by floods in Ethiopia at 2006 (Fisha, 2015).

The statistical data is summarized as shown in table 2.1.

Table 2.1: Number of affected Population in Ethiopia at 2006 Kiremet

Region	Affected	Displaced	Casualties
Dire Dawa	9,927	9,927	256
SNNPR	106,666	28,775	368
Amhara	97,824	37,863	3
Somali	361,619	125,000	80
Oromia	20,156	3,392	10
Afar	42,100	4,050	-
Gambella	30,915	30,915	2
Tigray	582	582	-
Harari	3475	-	-
Total	674,479	241,699	719

The areal extent of flooding and location of flooded regions vary from year to year indicating that flood event in 2010 were most devastating affecting about 29 locations, and followed by 2004 and 2006 which affected 24 and 22 locations respectively (Surafel et al., 2019).

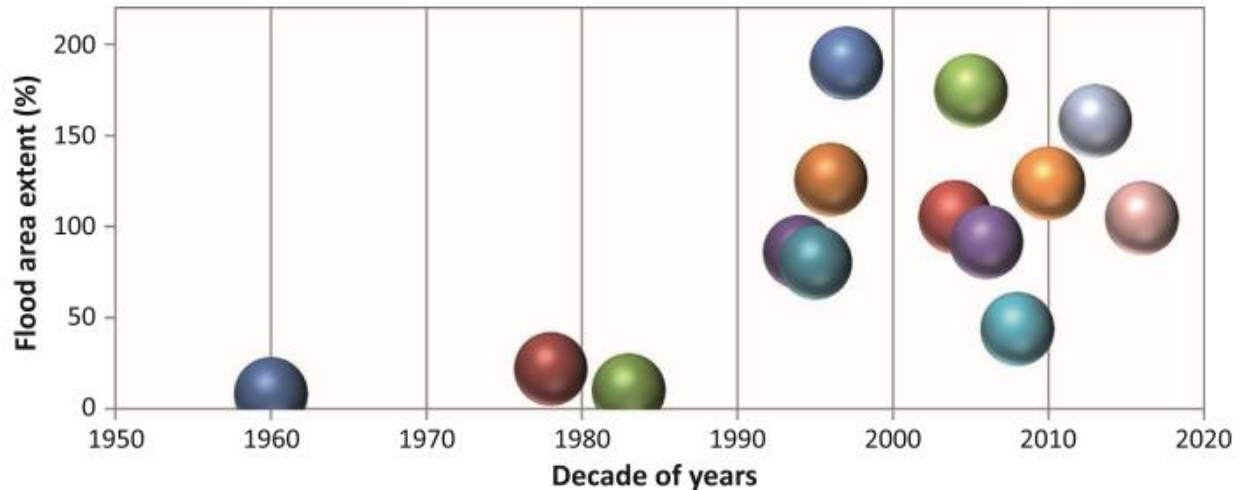


Figure 2.2: Frequency and area extent of flood events for last six decades (Surafel et al., 2019)

The country experiences two types of floods: flash floods and river floods. Flash floods are the ones formed from excess rains falling on upstream watersheds and gush downstream with massive concentration, speed and force (Joint Government and Humanitarian Partners, 2006). Often, they are sudden and appear unnoticed. Therefore, such flood often results in a considerable damage and becomes pronounced and devastating when they pass across or along human settlements and infrastructure concentration.

## 2.4 Factors Which Affects Intensity of Flooding

### 2.4.1 Landuse Change

Landuse change is defined to be any physical, biological or chemical change attributable to management, which may include conversion of grazing to cropping, change in fertilizer use, drainage improvements, installation and use of irrigation, plantations, building farm dams, pollution and land degradation, vegetation removal and so on (Nigatu, 2014).

Knowing and mapping of the changes in landuse/land cover could be taken as a good indicator of ecosystem health and can be considered as bench mark for land use/ land cover change detection in the future and it could be a pillar for different land use planning. Hence, it becomes important to undertake studies of land use/land cover changes to see the severity of the changes with time (Daniel, 2007).

With the current, economy and technology, getting aerial photographs that covers large area with a better resolution is becoming difficult. Thus, currently, more attention has been given to the use

of satellite imagery. Although it depends on the type of image data and purpose of study, the applicability of space borne remote sensing is becoming important. In general expression, catchment landuse/land cover changes influence the condition for transformation of precipitation into runoff by expanding impervious surfaces which lead to decrease infiltration rate and consequently increase the amount and rate of runoff (Sluiter, 2005).

#### **2.4.2. Climate Change**

Climate-related pressures and shocks already amount obviously in the lives of many of the world's people and mainly so in the lives of the poor. Occasions such as droughts and floods are often terrible experiences for those affected: they cause great loss of life, destroy countless livelihoods and leave millions of people devastated and Sub-Sahara Africa is considered to be most vulnerable to climate variability including flooding (Frederick, 2010).

The future trend of temperature and precipitation shows that there was change in climate when compared with observed temperature and precipitation i.e. temperature shows increasing trend and precipitation shows increasing and decreasing trend that clearly shows there is change in climate which resulting change in flood frequency from which climate change is considered as one of the biggest challenges of 21<sup>st</sup> century to the whole world will face with different impacts on many sectors among which impacts of climate change on water resources, especially on flood frequency of watershed (Gobena, 2016 ). Climate change will not only affect the economy but also increase the number of possible hazardous events to citizens.

#### **2.5 Management of Urban Flooding**

The harsh adverse impacts of flooding usually calls for managing flood disaster through appropriate systems and strategies is necessarily required especially to predict and evaluate possible flood that may occur in a particular area. This, in turn helps to reduce the potential damage caused by flooding hazards. It is always argued that to find out possible solutions to flooding problems, an understanding of the long-term factors that contribute to increase flooding are important including unplanned urbanization, soil erosion and deforestation.

Then to mitigate flooding hazards, it is important to adopt watershed-based management practices In addition to this, to mitigate flooding susceptibility, both the government and people have to adopt watershed-scale best management practices which includes floodplain zoning, planned urbanization, restoration of abundant channels, dredging of rivers and streams, increased

elevations of roads and village platforms, building of efficient storm sewer systems, establishment of buffer zones along rivers, conservation tillage, controlled runoff near construction sites, adjustment of life-style and crop patterns, good governance, and improvement in flood warning/preparedness systems (Gezahegn, 2013).

## **2.6 Impacts of Flooding on Communities**

An important fact of flooding is the destruction of the environment leading to a decrease in environmental quality including the existence of swamps which become breeding grounds for water-borne diseases; Destruction of homes, grain stores, social and economic infrastructures by flood waters; destruction of farmlands together with crops and farm animals; accumulation of massive quantities of silt on key community structures like water supply, sewage treatment, and others which paralyzes crucial life-support and ecosystem services (Frederick, 2010).

As White (1945), cited by Albert et al. (2016), flooding causes four types of impacts on communities: (a) damage to physical property; (b) interruption of the production of goods and services; (c) loss or impairment of human life; and (d) reoccupation and rehabilitation of flooded areas.

## **2.7 Hydraulic Models**

A hydraulic modeling is required to carry out the flood simulation to produce flood level at various locations along the flood plain. At present, one of the ways to study and understand the flood behavior is by generating the flood extent map, and flood propagation in rivers only or through water way can be described by one dimensional (1D) mathematical model but modeling of inundation of open floodplains shall be made by two dimensional (2D) models (Dawit, 2015). There are different flood modeling tools which have their own distinct model structure and solution procedures. Among these models HEC families, ARC GIS, MIKE 21 FM, 2D hydrodynamic, FLDWAV, ISIS, FLUCOMP, GFT, DUFLOW and MIKE11, SWMM and other tools are used to assess both rural and urban hydrology and hydraulic systems.

## **2.8 Possible Causes of Urban Flooding**

The causes of floods can be broadly divided into natural forces (climate factors), and human influences such as deforestation and urban development. Among the causes, climate related, mostly high rainfall is common causes of flood. Long duration of rainfall or high intensity of

rainfall events are the most common cause of flooding in Ethiopia urban areas. These events are usually associated with several days, weeks or months of continuous rainfall. Urbanization in particular has a direct impact on the magnitude and behavior of floods (Fisha, 2015).

Flooding in the urban areas may occur when heavy rainfall causes and leads to channels overtops their banks, drainage systems back up because they cannot convey the volume of water or are blocked by rubbish, sewers overflow because of illegal connections and the sewer system cannot cope with the increased volume (WMO, 2009).

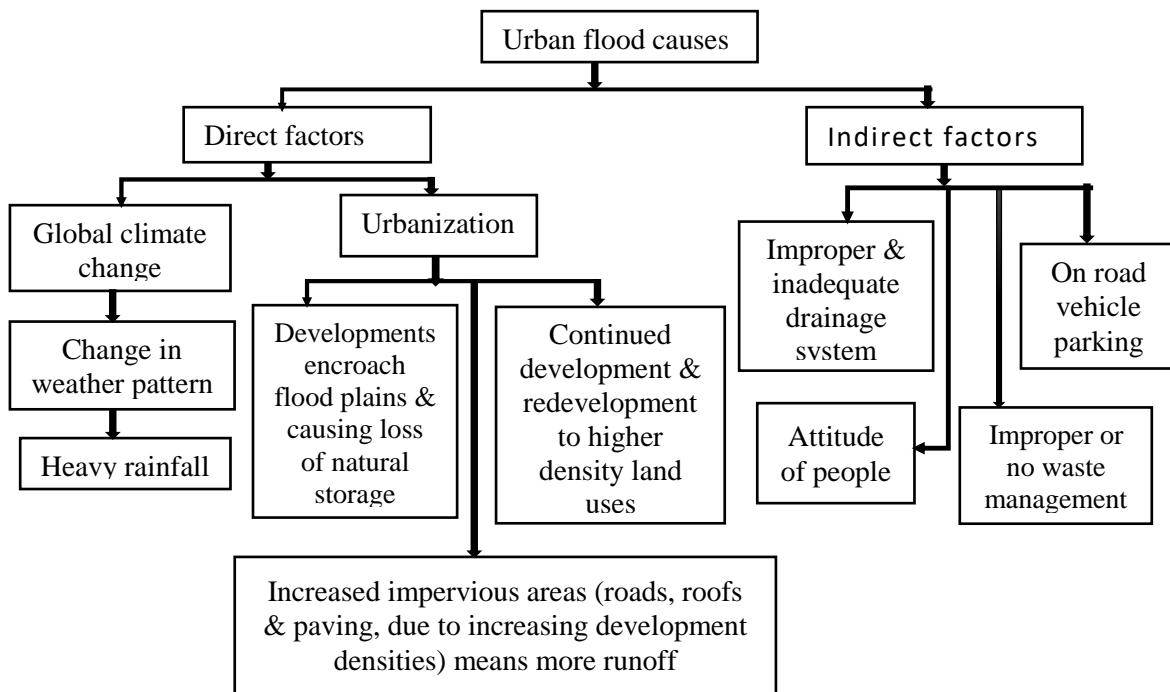


Figure 2.3: Causes of Urban Flooding

Source: [https://www.researchgate.net/figure/Causes-of-Urban-Flooding\\_fig2\\_309767636](https://www.researchgate.net/figure/Causes-of-Urban-Flooding_fig2_309767636)

## 2.9 Frequency Distribution and Development of IDF Curves

Since IDF curves describe the amount of rainfall in watershed area for a given period of time, the IDF relationship is one of the prerequisite statistics in water resources Engineering planning, development, management and to assess the vulnerability of hydraulic structures. The accessibility of accurate and long term rainfall data in the developing countries and lack of information are the major problem for IDF curves construction and it is known that the magnitude of an extreme rainfall event has an inverse relation to its occurrence frequency, therefore the severe rainfall events have less frequency compared to moderate rainfall events (Yassen, 2017).

The frequency analysis of rainfall data is to relate the magnitude of extreme events to their frequency of occurrence using the probability distribution. The IDF relationship is a mathematical relationship between the rainfall intensity, the duration, and the return period using extreme rainfall data. In order to establish the IDF relationships, essential information extracted from the existing rainfall data using statistical tools. To derive the information, probability distribution functions are applied to hydrological data such as normal, lognormal, exponential, Gamma, Pearson, Log-Pearson and extreme values distributions (Yassen, 2017).

### **2.10 Hydraulic Functioning of the Existing Drainage System**

The hydraulic capacity of a storm drain is controlled by its slope, shape, size and friction of flow resistance. Among several flow friction formulas which have been advanced with the relationship of capacity of flow with existing drainage canal parameters, Manning's equation is the most widely used formula for gravity and pressure flow (ERA, 2002).

### **2.11 Review on Evaluation of Hydrologic Response of Catchment**

Urban drainage is composed of an artificial system of sewers: pipes and other related infrastructures that collect and disposes all sorts of disposable water that an urban area can encounter. Seasonal inundations and water loggings are some of the main natural flood hazards that cities may faces. In nature, when rainwater falls on a natural surface, some water returns to the atmosphere through evaporation, or transpiration by plants; some infiltrates the surface and becomes groundwater; and some runs off the surface. In case of urban due to the reduction of infiltration capacity of land and insufficiency of drainage system, most of towns needs construction of additional drainage canals and appropriate management of drainage system.

Therefore, it is sounded to assess the drainage system and flood hazard of Alaba Kulito Town as it is among the towns which are facing drainage problem.

Some of the most widely used runoff estimation methods are (ERA, 2013): Rational Method; Natural Resource Conservation- Curve Number (NRC-CN) Method; Statistical analysis of stream data; and Regional regression equations..

The following table (table 2.2) Summarizes the application and limitations of the above flood estimation methods.

Table 2.2: Application and limitation of flood estimation methods

Method	Input data	Recommended Maximum area (km <sup>2</sup> )	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
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
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 -200
Empirical method	Catchment area, watercourse length, distance to catchment centroid (center), mean annual rainfall	No limitation, large areas	2 – 200
Statistical method	Historical flood peak records	No limitation, large areas	2–200 (depending on the record length)

Source: ERA drainage design manual, 2013

Of these possible hydrologic methods based on the available data, it should be noted that, at the present time, only the Rational and SCS methods are applicable to the whole country and the analysis of this paper relies on subcatchment area to choose either rational or SCS method. Therefore, among these methods Rational method and NRC-CR methods with SWMM model will be applied in this paper.

### 2.11.1 Rational Method

ERA manual of 2013 describes that the application of rational method of peak discharge estimation technique implies:

**Site visit:** - The physical parameters of rational methods are needed to be visited and collected to analyze the catchment response numerically

**Land use type identification:** - Paved and unpaved land should be detected throughout the study area

**Runoff coefficient (C) determination:** - Determination of weighted runoff coefficient plays significant role on final result.

**Time of concentration (Tc):** - Urban stormwater flow comprises both canal flow time and sheet flow time together.

**Determination of rainfall intensity (I):** - Calculated Tc and developed IDF curve helps to generate the value of corresponding catchments intensity.

**Peak discharge determination:** - Using the popular equation of rational method, peak discharge of a given area can be obtained.

### 2.11.2 SCS Curve Number Method

In 1954, the USDA Soil Conservation Service (SCS) developed a procedure, the Soil Conservation Service Curve Number (SCS-CN) method, to estimate direct runoff from storm rainfall for the design of conservation practices, watershed evaluation, and erosion control. This procedure was used by the SCS in project planning for the small watershed program and to estimate direct runoff from ungagged areas for engineering design of hydraulic structures (Bonta, 1997).

The hypothesis is proposed as:

$$\frac{F}{S} = \frac{Q}{P} \rightarrow 1 \text{ as } P \rightarrow \infty \quad (2.1)$$

Where; F = actual retention after runoff begins

S = potential maximum retention of a watershed

Q = runoff depth

P = rainfall depth (P > Q).

The potential maximum retention (S) is a constant for a particular storm because it is the maximum amount of water that can be held under existing conditions for a watershed. However, it varies from storm to storm because of the variation of the soil moisture content and position of the saturated soil profile, which in turn are mainly a function of precipitation. The retention (F) varies within a storm because it is the actual retention that occurs in a watershed during a particular storm.

The retention (F) can also be defined as the difference between P and Q at any point on the rainfall–runoff relationship curve:

$$F = (P - Q) \quad (2.2)$$

Equation 2.1 can be rewritten as:

$$\frac{P - Q}{S} = \frac{Q}{P} \quad (2.3)$$

Early versions of the runoff equation did not contain an initial abstraction term ( $I_a$ ), which represents interception, surface storage, and infiltration that occurs before runoff begins (SCS, 1985). This term was soon added, and the equation became:

$$\frac{P - I_a - Q}{S} = \frac{Q}{P - I_a} \quad (2.4)$$

Where  $I_a$  is initial abstractions and field experiment was used to estimate  $I_a$  in terms of S:

$$I_a = 0.2 * S \quad (2.5)$$

Substituting equation 2.5 into equation 2.4 Q resulted in:

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} \quad (2.6)$$

The potential maximum retention parameter (S) was represented by curve number (CN) to enable the rapid solution as equation 2.7:

$$S = \frac{25400}{CN} - 254 \quad (2.7)$$

### 2.11.3 Assessing Flood Hazard Using SWMM

SWMM is a rainfall-runoff typical model used for modeling the amount and quality of surface runoff from built-up areas and simulates urban flooding and drainage well and is widely favored by researchers (Dawe et al., 2019).

The application of SWMM model can be used in planning of dimensions of water catchment facilities for flood control such as retarding basins, mapping of flooded areas and natural drainage systems, planning of regulatory strategies to minimize drainage flow through evaluation of the

influence of inflow and infiltration on the flow discharge of the drainage system, and identifying sources of pollutant transport (Junaidi et al., 2018).

SWMM was originally developed in 1971 and it is widely used in rainfall-flood simulation and water pollution analysis in urban areas and has undergone several major upgrades with the latest EPA SWMM5 being one of the best representations of the global hydrological model (Dawe et al., 2019).

#### ❖ **Main components of SWMM**

SWMM displays result in the form of graphs, maps, tables, and reports of the following components:

**Catchment:** - A catchment is the area collecting water from nearby higher terrain surface, which is delineated by topographic contour lines. A catchment is usually described by its parameters (catchment area, the percentage of impervious area, average slope, the longest flow length and approximate shape).

**Nodes:** - are junctions to link the sewers. They also provide stormwater transition between surface and subsurface systems. Typical examples of nodes are manholes. Manholes should be provided at intersections of stormwater drains, junctions between different size of stormwater drains, where a stormwater drain changes direction/gradient and on long straight lengths.

**Links:** -transport flow in the system, and are often open channels or closed sewers with regular or irregular cross sections.

**Outlet:** -The most downstream component of the urban drainage system, which discharges the sewage from the system for receiving waters.

#### ❖ **Input Parameters of SWMM**

**Area:** This is the area bounded by the sub-catchment boundary. Its value is determined directly from maps or field surveys of the site or by using SWMM's Auto-Length tool when the sub-catchment is drawn to scale on SWMM's study area map.

**Width:** The width can be defined as the sub-catchment's area divided by the length of the longest overland flow path that water can travel. In applying this approach one must be careful not to include channelized flow as part of the flow path.

**Slope:** This is the slope of the land surface over which runoff flows and is the same for both the pervious and impervious surfaces.

**Imperviousness:** This is the percentage of the sub-catchment area that is covered by impervious surfaces, such as roofs and roadways, through which rainfall cannot infiltrate. Imperviousness tends to be the most sensitive parameter.

**Roughness Coefficient:** The roughness coefficient reflects the amount of resistance that overland flow encounters as it runs off of the sub-catchment surface. Since SWMM uses the Manning equation to compute the overland flow rate, this coefficient is the same as Manning's roughness coefficient  $n$ .

**Depression Storage:** Depression storage corresponds to a volume that must be filled prior to the occurrence of any runoff. It represents initial abstractions such as surface ponding, interception by flat roofs and vegetation, and surface wetting.

**Percent of Impervious Area without Depression Storage:** This parameter accounts for immediate runoff that occurs at the beginning of rainfall before depression storage is satisfied. It represents pavement close to the gutters that has no surface storage, pitched rooftops that drain directly to street gutters, new pavement that may not have surface ponding.

**Infiltration Model:** Three different methods for computing infiltration loss on the pervious areas of a sub-catchment are available in SWMM. They are the Horton, Green-Ampt and Curve Number models. There is no general agreement on which model is best. The Horton model has a long history of use in dynamic simulations, the GreenAmpt model is more physically-based, and the Curve Number model is derived from (but not the same as) the well-known SCS Curve Number method used in simplified runoff models. The SCS Curve Number model will be used in this scenario.

**Precipitation Input:** Precipitation is the principal driving variable in rainfall-runoff-quantity modelling. The volume and rate of stormwater runoff depends directly on the precipitation magnitude and physical parameters of the catchment.

#### ❖ Selection of Model

According to World Meteorological Organization (2009), it is often difficult to determine the relative advantages and disadvantages of models proposed for operational use and the selection of

a model for a specific hydrological situation has implications in water-resources planning, development, and management.

However, EPA SWMM is selected for this study due to the following reasons:-

- ◆ Is public domain software and freely copied
- ◆ Primarily developed for drainage systems of urban areas
- ◆ Combined hydrologic and hydraulics software.
- ◆ Simplicity and understand easily

Hence SWMM is the best assemblage of computer equipment and a set of computer programs for modelling urban flooding, with ability to entry and editing, storage, query and retrieval, transformation, analysis and display of the factors affecting flood hazard; SWMM hydrological model will be used to assess flood capacity and drainage system of the study area.

### 3 MATERIAL AND METHODS

#### 3.1 Description of the Study Area

##### 3.1.1 Location

Alaba Kulito (also known as Halaba Kulito, or Kuliito) is found between Shashamane and Wolayta Sodo Town and crossed by two cross country main federal asphalt roads at a distance of 313 km and 243 km from Addis Ababa, via Shashamane and Butajira/Hulbareg/Sankura/ towns, and also 63 km and 95 km from Shashamane and Hawassa Town respectively. Its absolute location is between 7°17'19" and 7°19'25" N of latitude and 38°4'10" and 38°6'17" E of longitude. The town sits on the left bank of the Bilate River, with an elevation of 1726 meters above the mean sea level.

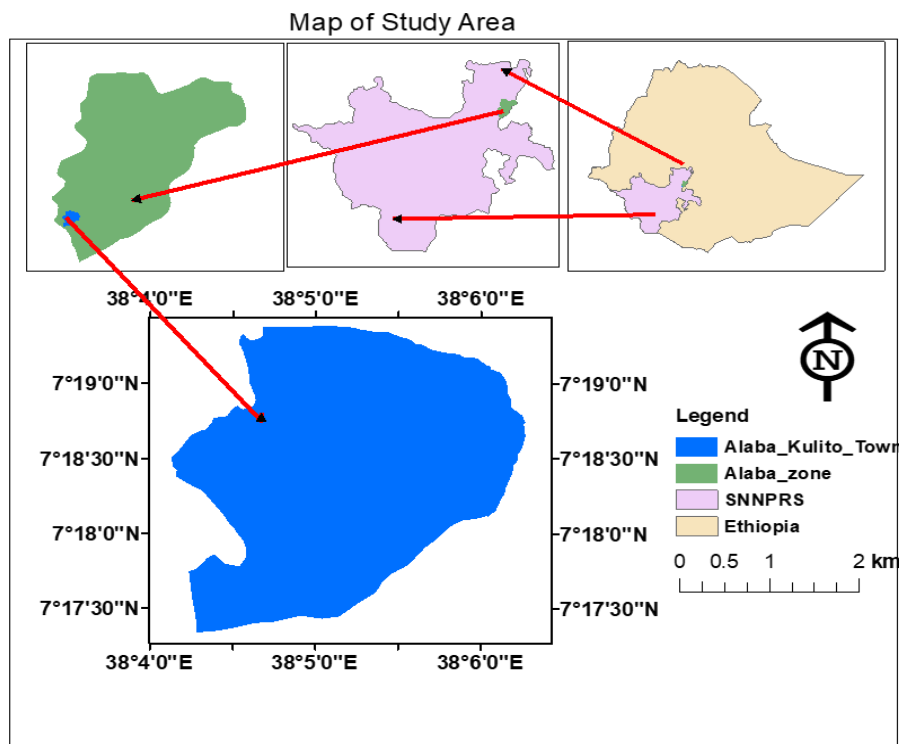


Figure 3.1: Map of Alaba Kulito Town

##### 3.1.2 History of Kulito town

The city was founded in 1880's as a village and the main factor for its foundation includes its location on the road of Addis Ababa and Shashemene, its central location to Walayita, Kambata, Hadiya and Arsie administrative areas, presence of trade and suitable land for the expansion of the town are the major ones. Hlaba Kulito was a town of Kembata Tembaro zone before 2002, a town of Alaba special wereda from 2002 up to 2018 and now it is a town of Alaba zone of the Southern Nations, Nationalities, and Peoples Region (SNNPR) since 2018.

Now a day; the Town has two sub-cities (Zobichame and Zalla) and five kebele’s administrations, named as Mehal Arada, Denebe Fama, Wanja Ber, Lenda Ber and Murasa kebele.

### 3.1.3 Climate

The climate is tropical in Alaba Kulito with a warm temperature (Mengesha et al., 1996). The average temperature and rainfall are 19.4 °C and 89.7mm respectively. Annual Precipitation here averages 1043 mm and the driest month is December having 23 mm of precipitation. The greatest amount of precipitation occurs in August, with an average of 154 mm.

February is the warmest month with an average of 20.6 °C. The lowest average temperatures in the year occur in August (18.4 °C). The monthly precipitation varies 131 mm between the driest month and the wettest month. The variation in temperatures throughout the year is 2.2 °C. The climatic data of kulito station is summarized in the following table (table 3.1) and the consecutive figures (figure 3.2 and figure 3.3).

Table 3.1: Climate data of Alaba Kulito Station

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
Avg. T° (°C)	19.6	20.6	20.6	20.3	19.8	19.1	18.5	18.4	19.1	19	18.7	18.6
Min. T° (°C)	11.1	12.3	12.6	13.2	12.7	12.8	13.2	12.9	12.8	11.2	10	9.1
Max. T° (°C)	28.2	28.9	28.6	27.5	27	25.5	23.8	24	25.5	26.9	27.5	28.1
RF (mm)	27.5	49.7	91.3	137.1	126.4	93.4	115.3	154.8	121.0	73.8	63.2	23.0

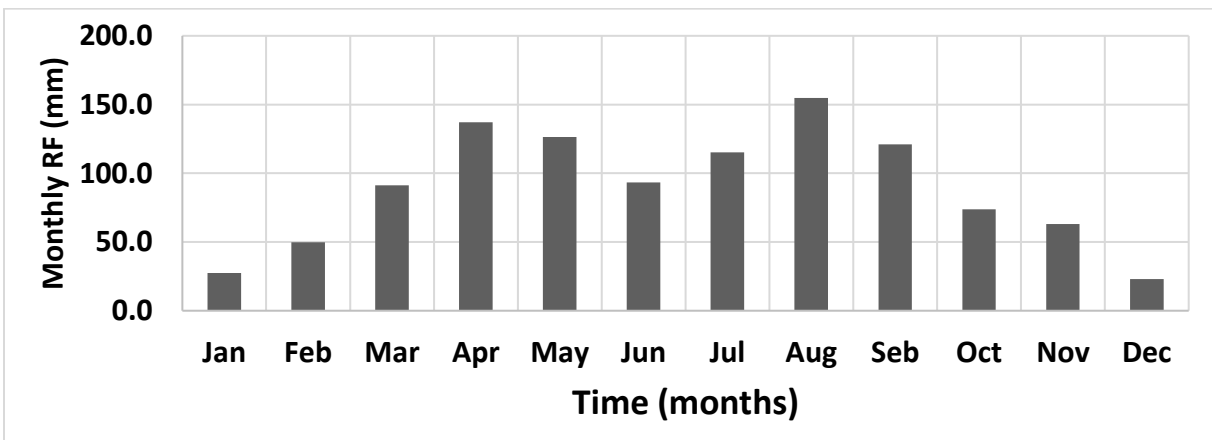


Figure 3.2: Average rainfall versus time curve of Alaba Kulito

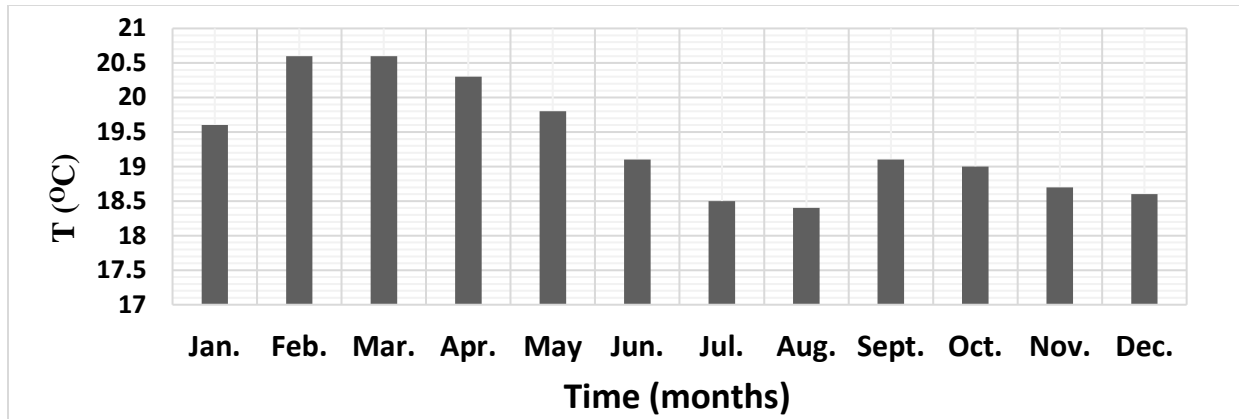


Figure 3.3: Temperature versus time curve of Alaba Kulito town

### 3.1.4 Geology

Dino formation which is evolved in quaternary period, following the subsidence of the main Ethiopia rift (MER) covers the Halaba Kulito area. This formation is made of ignimbrite, tuff, coarse pumice, water lain pyroclastic rocks with the interaction of laws trine sediments. The rocks around Halaba Kulito town are mainly ignimbrite. The drilling results in the vicinity of Halaba Kulito indicate the subsurface geology is composed of alternating layers of ignimbrite, tuff, volcanic ash and water laid pyroclastic. Basaltic hills of recent basaltic eruptions are found some 15 km to the west of Halaba Kulito. Around Halaba Kulito the thickness of this formation is not known. However, the drilling result of 300 meters deep well some 20 km northeast of Halaba Kulito (at Halaba Beshno) indicates that its thickness is over 300 meters. Because of Halaba Kulitos situation close to the western fault belt (WFB) of the central part of the main Ethiopia rift, there are some faults around Halaba Kulito trending North, East, South and West related to this fault system (Mengesha et al., 1996).

### 3.1.5 Road Coverage

There is about 127.07 km of road coverage out of which 13.36 km asphalt roads, 18.98 km cobblestone roads and 94.75 km Compacted Earth roads coverage in the town.

### 3.1.6 Organizational Structure of the City Administration

Currently, Halaba City is serving as one of the City State of SNNPR and the Center of Halaba Zone. The city has an autonomous power and duties that include customizing local development policies, rules and regulations, which provides efficient and effective administrative and judicial services, delivering socio-economic and municipal services, mobilizing and managing local

resources, promoting private businesses and creating a conducive environment for investments, etc.

Concerning its Governance structure, the regional government proclamations 'no 103/1999 allows Halaba city to organize city councils, city administrations, and judiciary bodies of the government. The city administration has incorporated two sub-cities, which are further divided into kebeles and villages structures under it.

As presented in the organizational structure of the city administration, the city's governance is a composition of the city council, executives and judicial bodies. The executive body: the chief executors (mayor) is responsible to coordinate, administer and regulate overall systems of governance including municipal functions. The Town has five kebeles administrations named as Mehal-Arada, Denebe-Fama, Wanja-Ber, Lenda-Ber and Murasa kebele. The presence of such an organizational structure of the city administration is paramount importance for the decentralization of power to different local administrative areas so the service charged to end-users would become easy and accessible by dwellers.

### **3.1.7 Vulnerability to flood**

Flooding has been a problem in Alaba Kulito Town having the natural topography, which varies from the mountainous (Rekame) to the flatlands. The catchment of Alaba Kulito Town is flat land and hence it is severely affected by flooding since the formation of the Town.

### **3.1.8 Soil Type**

The characteristics and conditions of the land can be provided by soil mapping. Knowing soil property enables us to manage the reaction of soil with human and natural interaction for water resource management practice and any Engineering works. Therefore, soil mapping or the process of delineating natural bodies of soils, classifying and grouping of the delineated soils in to map unit is needed for further use in this study.

According to FAO soil map classification of Ethiopia, ERA's manual (2002) and soil property information of World Harmonized Soil Database (WHSD) version 1.2 software; the soil type of the study area is classified as mollic andosol which is B hydrologic soil group. Group B soils are Silt loam, or loam Soils having a moderately low runoff potential due to moderate infiltration rates. These soils primarily consist of moderately deep to deep, moderately well to well-drained soils with moderately fine to moderately coarse textures (ERA, 2002).

### 3.1.9 Slope and Topography of the Study Area

Slope and topography describes the shape and relief of the land. The topography is a measurement of elevation, and the slope is the percent change in that elevation over a certain distance. Topography may be measured with lines that connect points representing the same elevation; while Slope is measured by calculating the difference in the elevation from one point to another divided by the lateral distance between those points.

Here; 20 by 20 DEM is used to calculate the slope of the study area which is reduced to six classes. According to FAO slope classification, the most portion of Alaba Kulito Town is between class 1 and class 4 with the remaining classes are found around Bilate River.

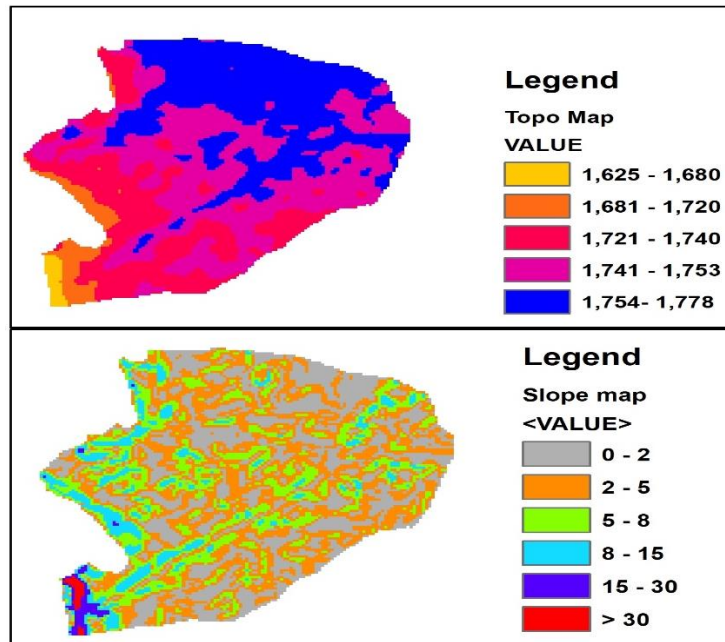


Figure 3.4: Slope and Topographic map of the Alaba Kulito Town

Table 3.2: FAO slope classification

Class of slope	Range of percentage of slope
Class 1	0 % - 2 % Slope
Class 2	2 % - 5 % Slope
Class 3	5 % - 8 % Slope
Class 4	8 % - 15 % Slope
Class 5	15 % - 30 % Slope
Class 6	30 % - 60 % Slope
Class 7	> 60 % Slope

### 3.1.10 Contour map of Alaba Kulito town

The surface feature of the land can be indicated by a map that shows elevation above sea level by means of contour lines. Figure 3.5 shows the contour map of Alaba Kulito Town. The town's highest altitude is 1770m and the lowest altitude is 1640m around Hospital and Bilate River respectively(10m contour interval is used).

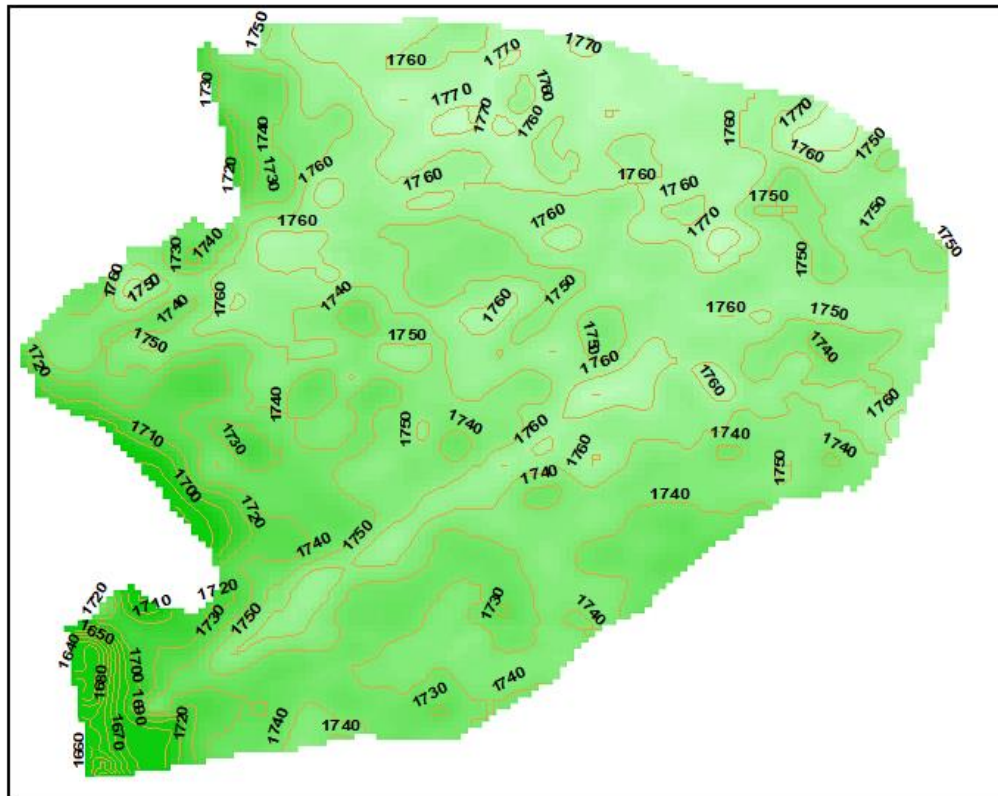


Figure 3.5: Contour map of Alaba Kulito Town

### 3.1.11 Land-Use Land-Cover Classes

Homogeneous land use land cover units are developed by performing land use and land cover studies. Sit visit, Google Earth pro, local knowledge, and any other relevant information helps us to differentiate the available data sources. Hence, based on the result of classification, twelve different types of land use/ land cover have been identified for the Alaba Kulito Town. These are named as: Commercial land, Institutional land, Asphalt road, Cobble-stone road, Compacted earthen road, Grassland, Forest land, expansion land, agricultural land Sub-urban (undeveloped) land and Open space (play-ground) land

As the basis of hydrologic impact evaluation and to introduce the current land use coverage of Alaba Kulito Town; land use classification is carried out.

For this study, the lack of clear image and similarity of pixels of one land use with another land use of buildups leads to overlapping of such land uses which enforces me to use Google Earth pro which is tedious and time-consuming than supervised classification. Here the shapefile of Alaba Kulito Town (study area) is imported to Google Earth pro and each land use polygon is created under created “new project” by giving different codes for each polygon and different colors for each land-use groups. Then after finishing each land uses under each category the project is saved in the form of “kml” which is further converted into ‘raster” in arc tool to do the remaining processing by ArcMap 10.4.1. The output of the classification looks as shown in the figure below with a clear legend. The classification is done based on the current view of the town. It requires the manual identification of the point of interest areas as a reference or ground truth within the images of Google Earth pro.

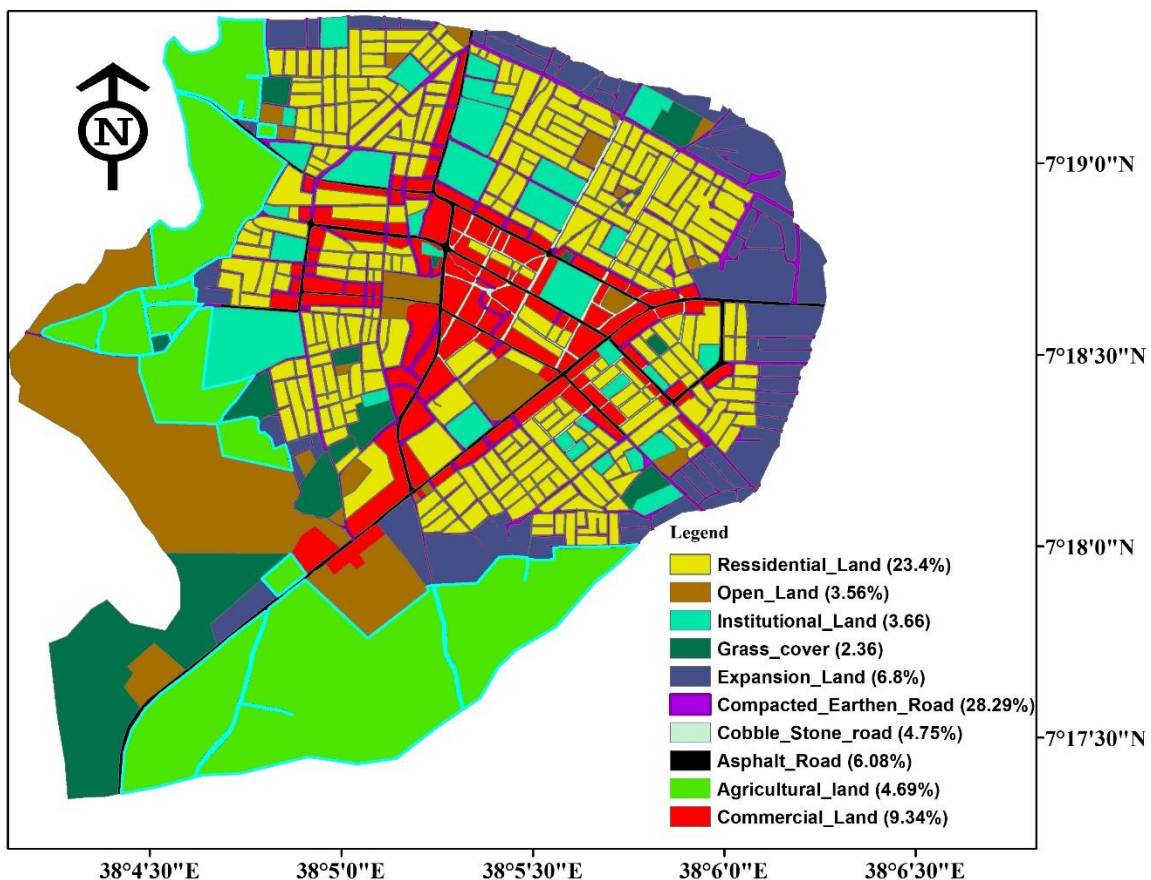


Figure 3.6: land-use map of Alaba Kulito Town

## **3.2 Data Collection and Analysis Methods**

### **3.2.1 Development of Intensity-Duration-Frequency (IDF) Curves**

As its name describes, Intensity-Duration-Frequency (IDF) curve is the relationship between rainfall intensity, rainfall duration, and return period as general and the curves for precipitation constitute a probabilistic tool and have proven useful in water resources management which can be obtained through frequency analysis of rainfall observations (Ombadi et al., 2018).

#### **3.2.1.1 Rainfall data**

Determination of frequencies and magnitudes of extreme rainfall events is very important for flood plain management and designs of hydraulic structures. However, the length of available records is not enough large to define heavy rainfall. In this case, magnitude-frequency analysis, fitting samples to a frequency distribution, permit the estimation of how often a specific event will occur or the frequency of events greater than those observed during the period records.

For this study, daily rainfall data for 33 years (from 1987-2019) is obtained from the National Meteorological Agency (NMA) of Ethiopia. For further hydraulic simulation works, the maximum daily precipitation was selected and used as an input, but it is not full which should be filled before using it for hydraulic and hydrologic analysis. Therefore, filling missed rainfall data is necessary.

#### **3.2.1.2 Filling missed Rainfall data**

Hence rainfall is an important part of the hydrological cycle, the accurate planning and management of water resources depend on the presence of consistent and exact precipitation data in meteorology stations. One of the first steps in any hydrological and meteorological study is accessing reliable data. However, precipitation data is frequently incomplete.

The incompleteness of precipitation data may be due to damaged measuring instruments, measurement errors and/or changes to instrumentation over time, a change in the measurement site, a change in data collectors, the irregularity of measurement, or severe topical changes in the climate of a zone (Mohammad et al., 2016).

The outcome of data analysis depends on the quality and completeness of data. To assess the suitability of the different methods for filling in missing data, daily precipitation data collected at four different stations were considered. To develop normal annual precipitation, it is better to have more than 33 years of recorded rainfall data. Even though stations found relatively nearby Alaba

kulito town are Aje, Shamena, Shone, Angecha and Shankura; there is no adequate data recorded to develop annual normal rainfall for those listed stations. Therefore, I am enforced to use other distant stations which have at least thirty years (33 years for this case) recorded rainfall data. Accordingly, the selected weather stations are Durame, Awassa, Hosana with Alaba Kulito being the main station found at Kulito town.

Among major classes of filling rainfall data (deterministic, stochastic, and artificial intelligence-based methods); deterministic approaches (arithmetic mean method, normal ratio method and inverse distance method) which are more suitable and mostly used due to their ease of implementation and computational efficiency of filling daily missed rainfall data was selected. Hence; the annual rainfall values are not within 10%, (for both Durame and Hosana), the stations are not evenly spaced, hilly regions and they are not near to each other (distant) it is not satisfactory to use the Inverse distance method and arithmetic mean method. Therefore, this paper considers the Normal Ratio Method (NRM) to fill missing precipitation data.

If the normal precipitations vary considerably then  $P_x$  is estimated by weighting the precipitation at various stations by the ratios of normal annual precipitation. The normal ratio method gives  $P_x$  as:

$$P_x = \frac{N_x}{3} \left[ \frac{P_1}{N_1} + \frac{P_2}{N_2} + \frac{P_3}{N_3} \right] \quad (3.1)$$

Where;  $P_x$  is the rainfall value of the weather station with missing precipitation

$P_1$ ,  $P_2$ , and  $P_3$  are the current rainfall values for station 1, station 2, and station 3 respectively.

$N_1$ ,  $N_2$ ,  $N_3$  and  $N_x$ , are the annual normal precipitation values for station 1, station 2, station 3 and station X respectively.

### 3.2.1.3 Checking consistency by Double-mass curve (DMC)

Double-mass curve (DMC) is composed of cumulative values of parameters plotted against one another over a certain time span which is used to study trends or possible changes in precipitation.

In the DMC, the cumulative values of relevant variables are plotted with the x- and y- coordinates (the average cumulative precipitation of neighboring stations is serially arranged in the reverse chronological order (i.e. the latest year getting the first entry) and plotted with the yearly

cumulative of precipitation of Alaba Kulito station for the corresponding years). If values of x and y axes are equally affected by external disturbances, then a DMC is a straight line; if not, slope breaks and presents additional information on the relationship between the studied variables.

The double mass curve technique was used to adjust precipitation records to take account of non-representative factors such as a change in location or exposure of rain gauge. Therefore, if a significant change is observed on the plot, it should be corrected by:

$$P'_x = P_x * \frac{M'}{M} \quad (3.2)$$

Where:  $P'_x$  = Corrected precipitation at station x.

$P_x$  = Original recorded precipitation at station x.

$M'$  = Corrected slope of the double mass curve.

$M$  = Original slope of the double mass curve.

#### **3.2.1.4 Homogeneity test**

Homogeneity analysis was used to separate a change in the statistical properties of the time-series data. The causes can be either natural or man-made including alterations to land use and relocation of the observation gauging station. Therefore in order to select the representative meteorological station for the analysis of areal rainfall estimation, checking the homogeneity of group stations is essential. The homogeneity of the selected gauging stations rainfall records were carried out by comparing the monthly average rainfall of each stations and all are found bimodal rainfall area.

#### **3.2.1.5 Determination of extreme value**

Erroneous selection of design rainfalls can cause significant problems for water infrastructure projects and flood mitigation works since the design rainfalls are an important input for the design of these projects. Therefore, there is a need to conduct a frequency analysis of extreme rainfall events.

In relation to extreme value determination of various return periods (frequency analysis), two methods are widely used and these approaches are used to build IDF curves: Annual Maximum Series (AMS) approach, also known as Block Maxima (BM) approach, and Partial Duration Series (PDS), also referred to as the Peak-over-Threshold approach (POT) method (Cook et al., 2020).

#### **❖ Block Maxima (BM) Extreme Value Determination Approach.**

The BM approach is the simplest and more commonly employed method for sampling original data for extreme analysis, but it has a very important shortcoming: Waste of data as only one data point is taken from each block and may cause loss of some important information, and also smaller sample sizes, which affect the accuracy of the parameter estimates. For these reasons, another approach to creating the sample series is considered: Peak over the threshold (POT), where all events above the determined threshold are included.

❖ **Peak over Threshold (POT) extreme value Determination Approach**

The POT method is more time consuming than BM, but has been found to yield better results in many studies for both extreme events of flood and drought (Yilmaz and Perera, 2015). An important advantage of the POT approach is that it produces larger sample sizes (data points), that enables one to use observed data more efficiently by considering more than one sample per year. Too high a threshold will generate few excesses and will lead to high variance of the estimate of the parameters by discarding too much data and too low a threshold will lead to an increase in bias as a result of the use of data that are not considered as being in the tails of the distribution. However, the number of extreme data points to be used for analysis depends on the selected threshold value; on average, 1.65–3.0 extreme events per year are considered for further analysis (Niguse et al., 2018).

A widely used method of determining the threshold value from a time series is the use of a plot called Mean Residual Life Plot (MRLP) (Hafid and Mohamed, 2019). The MRL plot displays the mean excess against a range of different threshold values. The mean residual life plot should be approximately linear above a threshold,  $u$ , at which the GPD provides a valid approximation to the excess distribution and the linearity is the basis for deciding the threshold value (Saeed & Abd Wahab, 2016). The mean of the excesses of the threshold,  $u$ , represented as  $(X - U|X > U)$ , can be estimated by the sample mean of the threshold excesses and, thus, the mean residual life plot is plotted by using the locus of points as:

$$\left\{ \left( U, \frac{1}{n_u} \sum_{i=1}^{n_u} (X_i - U) \right); U < X_{max} \right\} \quad (3.3)$$

Where;  $n_u$  is the number of observations that exceed threshold  $u$ .

### 3.2.1.6 Generalized Pareto Extreme Value Distribution Method

The probability density function for GPD with a shape parameter  $k \neq 0$ , a scale parameter  $\sigma$ , and a threshold (location) parameter  $\mu$ , is given as:

$$Y = f(x|k, \sigma, \mu) = \left(\frac{1}{\sigma}\right) \left(1 + k \frac{(x - \mu)}{\sigma}\right)^{-1 - \frac{1}{k}} \quad (3.6)$$

Range of  $x$ :  $\mu < x \leq \mu + \sigma/k$  if  $k > 0$ ;  $\mu \leq x < \sigma$  if  $k \leq 0$

Graphical and statistical methods are also used to test the goodness-of-fit of the distribution of the extreme rainfall values to the selected test distribution (GPD in this study). Among the several statistical tests available in the literature, the Kolmogorov-Smirnov (KS) statistic is the most frequently used; the most widely used graphical tests are the P-P and QQ plots (Mohamed, 2015). In this study, EasyFit 5.6 Professional software was used to develop these plots.

The KS test is a nonparametric supremum test based on the empirical cumulative distribution function (CDF). The empirical distribution function  $F_n$  for  $n$  number of iid observations of  $X_i$  is expressed as:

$$F_n(x) = \frac{1}{n} \sum_{i=1}^n I_{[-\infty, x]}(X_i) \quad (3.7)$$

Where:  $I_{[-\infty, x]}$  is the indicator function.  $I_{[-\infty, x]} = 1$  if  $X_i \leq x$  and  $I_{[-\infty, x]} = 0$  if  $X_i > x$ .

The Kolmogorov-Smirnov statistic ( $D_n$ ) is then computed as the largest vertical distance between the empirical CDF ( $F_n(x)$ ) and the expected CDF ( $F(x)$ ) as:

$$D_n = \sup_x |F_n(x) - F(x)| \quad (3.8)$$

Where:  $\sup_x$  is the supremum of the set of distances. The KS statistic is a procedure for testing whether two samples of a dataset are from the same distribution. In these statistics, the null hypothesis, which states that the two samples were drawn from the same distribution, is rejected if the p-value is less than the significance level.

The probability-probability or percent-percent (P-P) plot is a plot developed by plotting a variable's cumulative proportions against the cumulative proportions of the selected test distribution. Thus, it is used for investigating the closeness of the two cumulative proportions. The straighter the line formed by the P-P plot, the more the variable's distribution confirms to the fitted

distribution (GPD in this study). The quantile-by quantile (Q-Q) plot is a plot of the quantiles of a variable's distribution against the quantiles of the test distribution. It is used to demonstrate the match or mismatch between the observed values in the data and the estimated value given by the hypothesized fitted distribution (GPD in this study).

### 3.2.1.7 Detecting outliers

Outliers are data that appear outside the range of expected values due to recording errors. Outliers may indicate errors, data unrelated to the rest of the data set, or maybe perfectly valid data that indicates unusual hydrogeological conditions. The goal of outlier identification is to properly analyze the data to determine which outliers are representative of valid data points (and should be kept), and which outliers likely represent errors, and should be removed from the data set. The first step is to quantify the mean and how far the outlier is from the others. Calculate the lower and upper limit Z as the difference between the three times standard deviation and mean, and the summation of three times standard deviation and mean respectively (Plavsic et al., 2014).

If Z value deviates from the limit points, the value is far from the others. The most widely used method of detecting outlier is equation 3.4.

$$Z = 3 * \delta \pm \mu \tag{3.4}$$

Where: Z is limit value,  $\sigma$  is standard deviation and  $\mu$  is mean for N number of data set.

### 3.2.1.8 Disaggregation of daily rainfall of shorter duration

Design and analysis of drainage structures require a rainfall intensity duration relationship of shorter duration. But, rainfalls for shorter durations were not available for the stations located in the study area. The rainfall depths obtained from the gauging station are of 24hr duration depth. Consequently, a convenient method of converting 24-Hr rainfall depths into rainfall depths of different durations was adopted. Both maximum precipitation and statistical variables for durations of (5, 10, 15, 20, 25, 30, 60, 90, 120, 150 and 180) minutes were computed using the most widely used equation (equation 3.5) in East Africa (Fisha, 2015) and ERA (2013) suggests the following equation,

$$RR_t = \frac{t}{24} \left[ \frac{(b+24)^n}{(b+t)^n} \right] \tag{3.5}$$

Where:  $RR_t$  = rainfall ratio =  $R_t : R_{24}$

$R_t$  = rainfall in given duration "t" in hour

$R_{24}$  = rainfall in 24 hour and  $t$  = time in hour

Using  $b = 0.3$  and  $n = 0.92$  as suggested by ERA manual results are tabulated for rainfall durations 5, 10, 15, 20, 25 ... 180 minutes. Using the above formula the rainfall ratio (RRt) for different minutes of durations are computed in Table 3.9.

Table 3.3: Rainfall ratio (RRt) computation

t(min)	5	10	15	20	25	30	60	90	120	150	180
t(hr)	0.08	0.17	0.25	0.33	0.42	0.50	1.00	1.50	2.00	2.50	3.00
b+24	24.30	24.30	24.30	24.30	24.30	24.30	24.30	24.30	24.30	24.30	24.30
(b+24) <sup>n</sup>	18.83	18.83	18.83	18.83	18.83	18.83	18.83	18.83	18.83	18.83	18.83
b+t	0.38	0.47	0.55	0.63	0.72	0.80	1.30	1.80	2.30	2.80	3.30
(b+t) <sup>n</sup>	0.41	0.50	0.58	0.66	0.74	0.81	1.27	1.72	2.15	2.58	3.00
t/24	0.00	0.01	0.01	0.01	0.02	0.02	0.04	0.06	0.08	0.10	0.13
RRt	0.16	0.26	0.34	0.40	0.44	0.48	0.62	0.69	0.73	0.76	0.78

Using the values obtained in the table, the correlation ( $R_t = RR_t * R_{24}$ ) for each years of 24 hours rainfall in mm will be changed to (5, 10, 15, 20, 25, 30, 60, 90, 120, 150 and 180) minutes of rainfall.

While designing drainage works, IDF relationships are used for water resource management related Engineering projects. Proper development and prediction of IDF enable Engineers to design economically feasible and structurally safe and adequate to control flood. In this paper, peak over threshold (POT) extreme value determination technique was used and to compute the maximum design discharge, Generalized Pareto distribution (GPD) probability density function is selected which follows POT and mean residual life plot as it is practically used distribution function to determine maximum discharge of extreme events like drought and flood (Ashraful et al., 2018 and Hidalgo-Muñoz et al., 2010).

### 3.2.2 Assessment of Hydraulic Capacity of the Existing Drainage System

The interaction between natural water circulation and human activity around built-up areas needs a drainage system. The urban drainage system needs proper study and reasonable forecasting of runoff volume and drainage size of channels. The urban drainage systems can be completely artificial, or a combination of manmade sewer facilities and natural watercourses. The system can be represented as a network consisting of catchments and sub-catchments, nodes, links and outlets.

Therefore, data of catchment property and data of existing drainage canals are necessary to know the runoff volume generated from the catchment area and capacity of drainage lines.

### 3.2.2.1 Digital Elevation Model (DEM) data of the study area

DEM is consisting of a grid of square cells in which each square cells value representing the elevation of the surface of the land. It enables us to know the existing ground surface flow direction of water and selection of the location of the study area. For this study, 20x20 meter of DEM of USGS is obtained from Ethiopia Mapping Agency and the location of the study area was clipped by ArcGIS 10.4.1 data management tools.

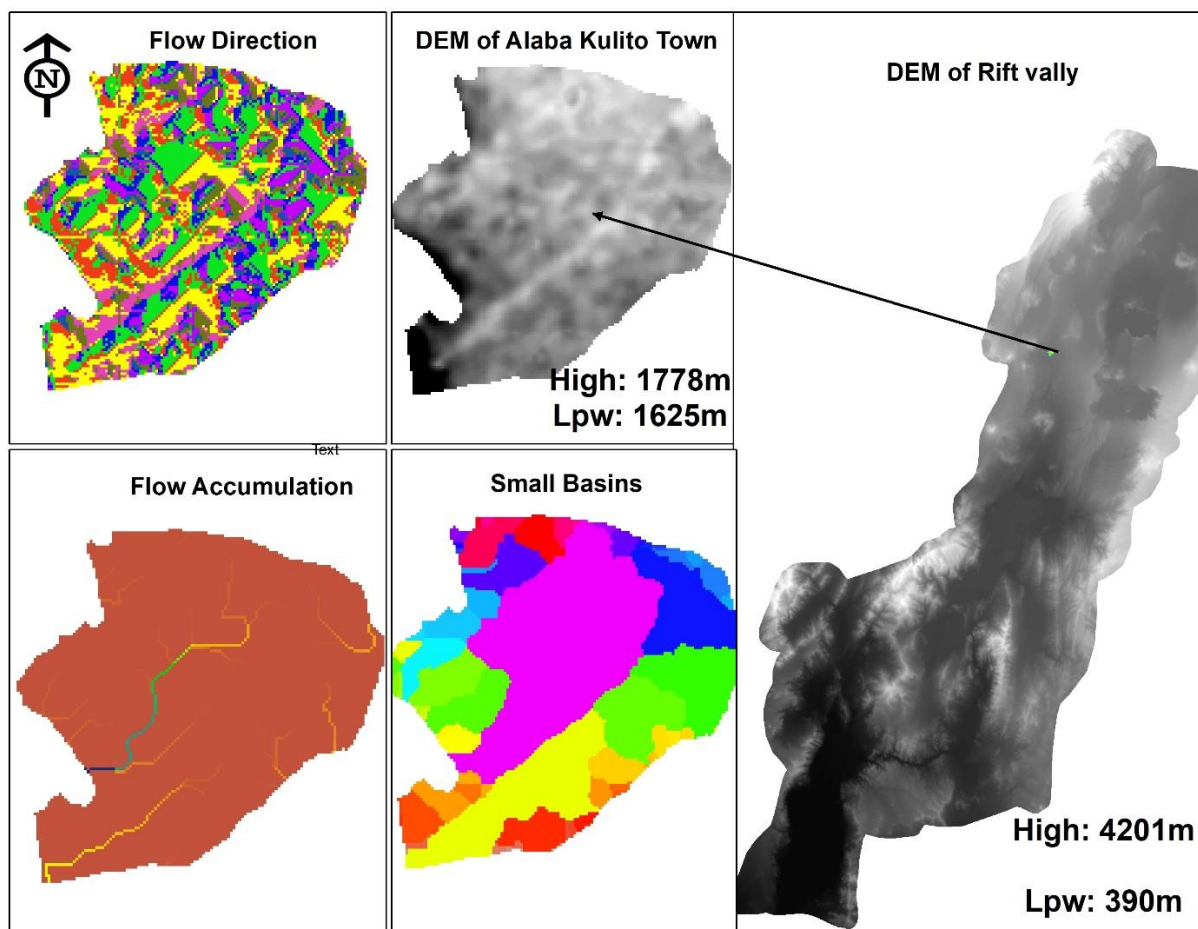


Figure 3.7: Clipped DEM for the Alaba Kulito Town

### 3.2.2.2 Soil Data

Soil data was taken from the Ministry of Agriculture to know the types of soil for the study area. The soil type of the Town is mollic andosol and the catchment out of the catchment contains dystric cambisol in which both are B hydrologic soil group.

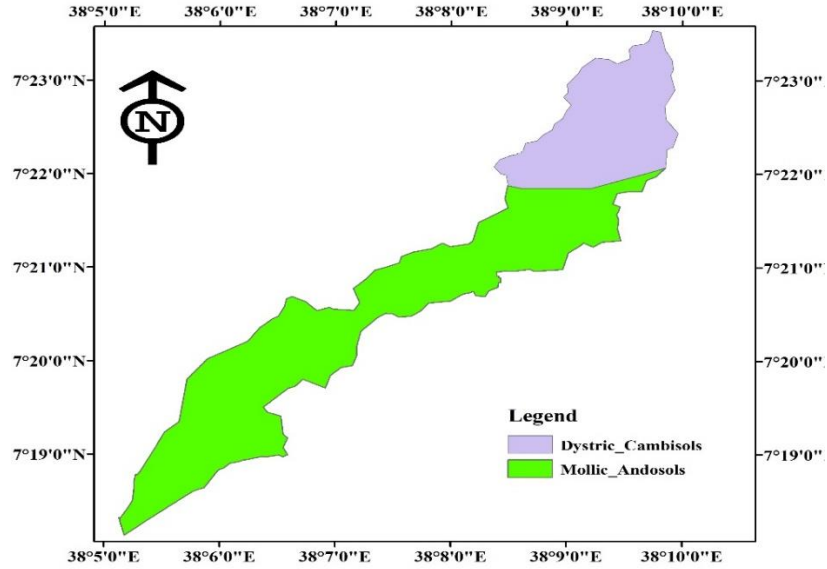


Figure 3.8: Soil map of study area

### 3.2.2.3 Drainage System Network Data

Data of the drainage system network for the drainage network is collected on-site by direct measuring. The following information of storm drain system are collected:

- ❖ Dimension, slope, type and location of links
- ❖ Dimension and location of manholes and junctions
- ❖ X, Y, Z Coordinates of junctions

The measured canal dimension helps us to analyze the hydraulic capacity of the existing drainage system to convey the runoff generated from sub-catchments by using the Manning method. Further, the output of EPA SWMM 5.1, rational equation (for less than 50 ha area) and soil conservation service curve number (SCS-CN) method (for more than 50 ha area) are used for comparison.

### 3.2.2.4 Rational Method

The rational formula estimates the peak rate of runoff at any location in the catchment area as a function of the catchment area, runoff coefficient, and rainfall intensity for a duration equal to the time of concentration (the time required for water to flow from the remote point of the basin/watershed to the outlet (interest) point. According to ERA Drainage Design Manual

(EDDM) (2013), this method estimates the peak runoff rate for small urban and rural watersheds of less than 50ha.

$$Q = \frac{CIA}{360} \tag{3.15}$$

Where: Q = maximum discharge for a return period equivalent to the design rainfall (m<sup>3</sup>/s)

C = Dimensionless runoff coefficient assumed to be a function of the watershed and often the frequency of the flood being estimated.

I = Rainfall intensity during the concentration-time (mm/hr)

A = area of the basin (ha).

Here a frequency factor C<sub>f</sub> is used to higher intensity storms which will require modification of coefficient C because of infiltration and other losses affect runoff. The rational formula by a frequency factor C<sub>f</sub> now becomes: (ERA manual, 2002).

$$Q = \frac{CfCIA}{360} \tag{3.16}$$

Where: the product of C<sub>f</sub> and C shall not exceed 1.0 and the cross-ponding values of C<sub>f</sub> are obtained from the ERA manual (table 3.4).

Table 3.4: Frequency factors for rational formula (ERA manual, 2002)

Recurrence Interval(years)	C <sub>f</sub>
5	1.0
10	1.0
25	1.1
50	1.2
100	1.25

Determination of watershed area, time of concentration, selection of appropriate runoff coefficient and computing peak discharge for the study area of the desired frequency by assuring consistency with the assumptions and limitations are remarkably needed for applying the rational method.

#### ❖ **Runoff Coefficient**

Runoff coefficient C is a dimensionless number representing the ratio of runoff to rainfall. The interaction of variables like depression storage, infiltration, soil type, ground cover, ground slope and antecedent moisture factors the value of runoff coefficient all together.

A weighted runoff coefficient is used when the catchment area contains a varying amount of land covers like urban areas which have varies land-use within small sub-catchment and it is determined as:

$$C_w = \frac{\sum C_i A_i}{A} \quad (3.17)$$

Where:  $C_w$  = weighted runoff coefficient,

$C_i$ , = runoff coefficient for land-cover type i,

$A_i$  = area for land-cover type i,

$A$  = total drainage area.

#### ❖ Rainfall Intensity

Once a particular return period has been selected for design and time of concentration calculated for the catchment area, the rainfall intensity can be determined from Rainfall-Intensity-Duration curves. The rainfall intensity can be determined from the IDF curve for a selected particular return period and time of concentration which has been calculated for the drainage area. Therefore, rainfall intensity (I) is the average rate of rainfall in (mm/hr) of selected return period to duration which equals to the time of concentration.

#### ❖ Time of Concentration

The basin time of concentration is the time required for flowing water to reach the most remote part of the drainage point of interest for discharge calculation. The rational method requires calculating the time of concentration for each design point of the drainage basin. For a specific drainage basin, the time of concentration consists of sheet flow time (an inlet time) plus the time of flow in a closed conduit or open channel to the design point which is obtained by velocity method (ERA, 2013).

#### ➤ Sheet flow travel time (overland flow)

Inlet time is the time required for runoff to flow over the surface to the nearest inlet and is primarily a function of the length of overland flow (l), the slope of the drainage basin (S), and surface cover ( $C_v$ ).

$$T_{c1} = 0.604 \left( \frac{C_{vi} * L_i}{S^{0.5}} \right)^{0.467} \quad (3.18)$$

Where:  $T_{c1}$  = time of concentration to inlet (hrs.)

$C_{vi}$  = roughness coefficient of land use

$L_i$  = flow length from the remotest point to the point of interest (km).

$S$  = Slope of the catchment (m/m)

➤ **Open channel (pipe) flow time**

The time of open channel flow can be estimated from the hydraulic properties of the conduit or channel. After first determining the average flow velocity in the pipe or channel, the travel time is obtained by dividing velocity into the pipe or channel length. Manning's Equation can be used to determine the velocity.

$$T_{c2} = \frac{L}{3600V} \quad (3.19)$$

Where:  $T_t$  = travel time (hr),  $L$  = length of the main drainage canal (m),  $V$  = flow velocity in the channel (m/s) and 3600 is the conversion factor from seconds to hours.

$$V = \frac{R^{2/3}S^{1/2}}{n} \quad (3.20)$$

Where:  $R$  = hydraulic radius (m)

$S$  = channel slope (m/m)

$n$  = Manning roughness coefficient.

Then to obtain the total time of concentration, the pipe or open channel flow time must be calculated and added to the inlet time.

❖ **Total time of concentration ( $T_c$ )**

$$(T_c = T_{c1} + T_{c2}) \quad (3.21)$$

### 3.2.2.5 Soil Conservation Service (SCS) Method

This method is developed by the United States Soil Conservation Service for calculating rates of runoff requires the same basic data as the Rational Method, but mainly useful for large catchments. However, The SCS method is more sophisticated in that it considers also the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate

that decreases during the storm, either real or fabricated, by subtracting infiltration and other losses from the rainfall to obtain effective precipitation.

### **Peak Discharge Estimation (qp)**

The following equation was used for the estimation of the peak discharge in the SCS method:

$$qp = (Q * A * qu) \quad (3.22)$$

Where: qp = peak discharge in m<sup>3</sup>/s

qu = unit peak discharge in (m<sup>3</sup>/s/km<sup>2</sup>)/mm

A = drainage area in km<sup>2</sup>

Q = depth of runoff in mm

### **❖ Rainfall-Runoff Equation**

The SCS method is based on a 24-hour storm event which has a Type II time distribution. The Type II storm distribution is a typical time distribution that the SCS has prepared from rainfall records and is applicable for interior rather than the coastal regions and appropriate for Ethiopia. A relationship between accumulated rainfall and accumulated runoff was derived by SCS from experimental plots for numerous hydrologic and vegetative cover conditions. The equation was developed for small catchment areas for which daily rainfall and catchment area data are ordinarily available. The equation is:

$$Q = \frac{(P - Ia)^2}{(P - Ia) + S} \quad (3.23)$$

Where: Q = Accumulated direct runoff in mm

P = Accumulated rainfall (potential maximum runoff) in mm

Ia = Initial abstraction including surface storage, interception, and infiltration prior to runoff in mm and S is Potential maximum retention in mm.

The relationship between I<sub>a</sub> and S was developed from experimental catchment area data. The empirical relationship used in the SCS runoff equation is:

$$I_a = 0.2S \quad (3.24)$$

S is related to the soil and cover conditions of the catchment area through the Curve Number (CN).

$$S = \frac{25400}{CN} - 254 \quad (3.25)$$

❖ **The ratio of Initial abstraction to Accumulated rainfall (Ia/P) parameter**

Ia/P is a parameter that is necessary to estimate peak discharge rates. P is the 24-hour rainfall depth for a selected return period and Ia denotes the initial abstraction. The 24-hour rainfall depth is taken from the frequency analysis result or from the ERA DDM rainfall region rainfall depth recommendations. For this particular study, the 24-hour rainfall depth was taken from the frequency analysis of GPD for Alaba Kulito town.

❖ **Runoff and Curve Number**

Runoff is rainfall excess or effective rainfall when the amount of rainfall exceeds the capability to infiltrate or 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. The Soil conservation service uses a combination of soil conditions and land use to assign a runoff factor (curve number). These runoff factors, called runoff curve numbers (CN), indicate the runoff potential of an area. The higher the CN, the higher is the runoff potential.

❖ **Unit peak discharge (qu)**

The unit peak discharge is obtained from the following equation, which requires the time of concentration (Tc) in hours and the initial abstraction to rainfall ratio (Ia/P) as input:

$$qu = \alpha * 10^{c_0 + c_1 \log t_c + c_2 (\log t_c)^2} \quad (3.26)$$

Where  $\alpha = 0.000431$  C<sub>0</sub>, C<sub>1</sub>, and C<sub>2</sub> = regression coefficients given in Appendix C (table C5) of EDDM, 2013 for various Ia/P ratios.

The combination of Ia/p parameter with rainfall type (I, IA, II and III) helps us to fix regression coefficients given in the table of EDDM, 2013. The SCS method is based on a 24-hr storm event which has a type II time distribution. The type II storm distribution is a typical time distribution which the SCS has prepared from rainfall records. It is applicable for the interior rather than the coastal regions and appropriate for Ethiopia (ERA, 2013).

**3.2.2.6 Manning's Formula for Hydraulic Analysis**

This method is used for major streams to compute the design flood levels at crossing sites after the discharges have been estimated by the hydrological methods of either the Rational or SCS method.

Manning's Formula was used to compute the average flow velocity (V) in any channel or natural stream with the uniform flow (ERA, 2013). Channel discharge quantity (Q) is the product of the mean channel velocity (V) and the area (A) of the channel. To determine the discharge (Q) of non-pressure pipes, canals and natural drainages, the following formula is used:

$$Q = 1.49AV \quad (3.27)$$

Where: Q= discharge in cubic meter per second ( $m^3/s$ )

A = Cross-sectional area in square meter ( $m^2$ )

V = Average flow velocity in meter per second ( $m/s$ )

The average flow velocity (V) in any channel or natural stream is calculated by:

$$V = \frac{1}{n} R^{2/3} S^{1/2} \quad (3.28)$$

Where: S = Channel slope in meter per meter ( $m/m$ )

R = hydraulic radius in meter (m)

n = manning's roughness coefficient

### 3.2.2.7 Modelling Using EPA SWMM5.1

#### ❖ Model Description

The Environmental Protection Agency Stormwater Management Model (EPA SWMM) is a dynamic rainfall-runoff simulation model used for a single event or long-term (continuous) simulation for urban area drainage systems. The collection of sub-catchments which receives precipitation are the runoff component on which SWMM operates. SWMM transports generated runoff through a system of pipes or channels as routing portion. During the simulation period, SWMM tracks runoff quantity generated in each sub-catchments, quantity of water in each pipe and channel, flow depth and flow rate (Lewis, 2010).

#### ❖ Steps of Using EPA SWMM 5.1

For the selected study area the following steps are generally used (Lewis, 2010)

- I. Specify a set of default and project properties to use
  - ◆ Default ID labels for nodes and links
  - ◆ Default sub-catchments properties like area, width, slope
  - ◆ Default node/link properties (node invert, conduit length,)

- II. Draw a network representing of physical components of the study area
  - ◆ Add a rainfall intensity for the study area
  - ◆ Add a sub-catchment
  - ◆ Add nodes (manholes of the study area)
  - ◆ Add links (channels connecting manholes of the study area)
- III. Edit the properties of an object that make up the system
- IV. Select a set of analysis options
- V. Run a simulation
- VI. View the result of a simulation

#### ❖ **Model setup**

The contributing catchment and sub-catchments of the study area should be divided to build a conceptual model. This helps us to gate information like area and slope of sub-catchments by reviewing the literature for typical values of previous and impervious manning's coefficient. The average maximum length and sub-catchment area is used to find the width parameter according to the equation below (Lee et al., 2017).

$$W = \frac{A}{L} \quad (3.29)$$

Where: A is the area of each sub-catchment (m<sup>2</sup>)

L is the average maximum length of each sub-catchment (m)

#### ❖ **Model Input Parameters**

The main parameters used under simulation of the SWMM model are parameters of sub-catchment, node (junctions and outfalls) and link.

**Link data:** - Invert elevation, maximum depth

Maximum depth is the distance from the ground surface to the bottom of the junction and invert elevation is the elevation of the bottom of the junction recorded and encoded from site.

**Conduit data:** - Shape, length, inlet and outlet node, roughness, maximum depth

The shape of the conduit is rectangular having different value of length, maximum depth and roughness of 0.02 is used for those drainage lines which are found along roadsides with concrete bed and 0.015 is used for remaining masonry beds.

**Sub-catchment data:** - Area in a hectare, slope in percentage, width in meter, rainfall intensity of 2, 5, 10, 25, 50 and 100 years return period, percentage of imperviousness, depth of depression storage (for both pervious and impervious area), manning roughness coefficient (for both pervious and impervious area) and outlet are used.

Table 3.5: Key Parameters and allowed ranges used for sensitivity analysis of SWMM analysis (Li et al., 2014)

Description	Symbol	Allowed Range
Manning's roughness coefficients for impervious areas	N-Impervious	0.005-0.05
Manning's roughness coefficient for pervious areas	N-Pervious	0.05-0.5
Depth of storage in impervious areas (mm)	D-store-Impervious	1.3-2.5
Depth of surface storage in pervious areas (mm)	D-store-Pervious	2.5-7.6

### 3.2.3 Identifying causes of Flood

#### ❖ observation of the study area

Field observation is made to look at the drainage pattern and current condition of drainage lines. Important pictures have been captured to strengthen the report by reasonable and logical photos. The drainage lines from minor drainages (around Alaba Kulito Zonal Hospital) up to outlet at ST. Gabriel church observed.

#### ❖ Interview of the concerned body

The concerned bodies like dwellers and head of Alaba Zone Municipality have been interviewed in addition to field observation to soundly point main causes of the historical flood of Alaba Kulito Town.

#### ❖ Analysis of drainage hydraulics and the hydrologic response of a catchment to rainfall

The existing drainage lines' capability to convey the generated runoff from the upper catchment is assessed by SWMM. This helps to judge the runoff load upon drainage lines is excess or tolerable.

### 3.2.4 Flood Mitigation Methods

#### 3.2.4.1 Trapezoidal canal design

Under the section of mitigation, two different trapezoidal sections are designed one at the upper side of the town as a diversion canal and one at the lower side of the town as an outfall canal. The lower canal which is the common outfall of both Lenda Ber and Mahal Arada Kebele is proposed to be a lined trapezoidal canal done by Manning's method and that of upper diversion canal is

proposed to be an earthen/erodible canal designed by maximum permissible velocity method of canal design.

The procedure to size the channel section consists of the following steps:

- I. For the specified channel material, determine the Manning roughness factor 'n', the maximum permissible velocity  $V_{max}$  and the side slope 'z'
- II. Compute the corresponding hydraulic radius 'R', from the Manning's equation, rearranged as:

$$R = \left( \frac{n * V_{max}}{\sqrt{S}} \right)^{3/2} \quad (3.30)$$

- III. Compute the required flow area ( $m^2$ ) from discharge ( $m^3/s$ ) and velocity (m/s)

$$A = \frac{Q}{V_{max}} \quad (3.31)$$

- IV. Compute the wetted perimeter P (m) as:

$$P = \frac{A}{R} \quad (3.32)$$

- V. Knowing the magnitudes of 'A' and 'P' and using the expressions for 'A' and 'P', solve for the flow depth 'y' and the bottom width 'b'
- VI. Both flow depth 'y' and bed width 'b' can be solved by using intermediate dimensionless number 'w' as:

$$W = \frac{Q}{R^2 * V_{max} * (2 * \sqrt{1 + z^2} - z)} \quad (3.33)$$

- VII. From step II and step VI, flow depth y (m) can be calculated as:

$$y = \frac{R * W}{2} \left( 1 - \sqrt{1 - \frac{4}{W}} \right) \quad (3.34)$$

- VIII. Bed width is calculated from the above parameters with side slope m as:

$$b = \frac{Q}{R * V_{max}} - 2 * y * \sqrt{1 + z^2} \quad (3.35)$$

- IX. Finally, twenty percent of flow depth ( $0.2*y$ ) is added as a freeboard and the erodible trapezoidal canal by maximum permissible velocity method is designed in such manner to divert runoff generated from a rural area into Bilate river at about 3.5 km distance from Alaba zonal hospital.

This method is based on the assumption that a channel will not be eroded if the average cross-sectional velocity in the channel does not exceed the maximum permissible velocity. Therefore, a channel cross-section is designed so that, under the design flow conditions, the cross-sectional average velocity remains below the maximum permissible value.

The data collection mechanisms and analysis methods are described in the above sections in detail and the general framework of research is shown in the figure below (figure 3.8).

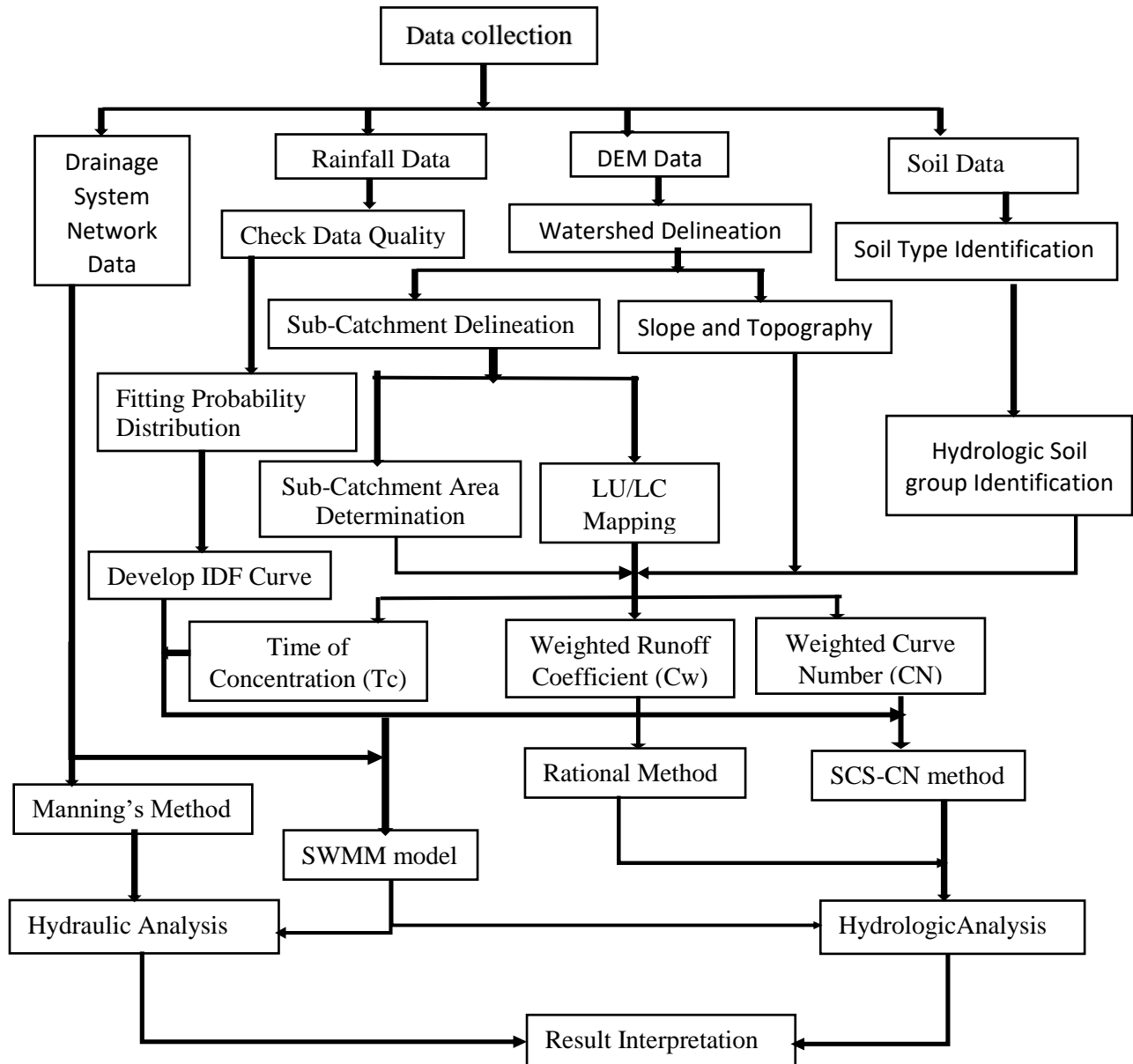


Figure 3.9: Research Framework

## 4 RESULTS AND DISCUSSION

### 4.1 Intensity Duration Frequency (IDF) Curve of Alaba Kuito Town

The main data used to develop IDF curve is rainfall data which is 33 (from 1987 up to 2019 E.C) years daily (24-hour) data obtained from Ethiopian Meteorological Agency (EMA).

#### 4.1.1 Rainfall Data

The thirty-three (33) years recorded daily rainfall data of the study area were obtained from the Ethiopian National Meteorological Service agency. Out of 12,052 days, about 445 days of rainfall data is not recorded (missed) for Alaba Kulito weather station which is about 3.7 percent of total days to be recorded. After filling missed rainfall data, monthly data is obtained by adding each successive daily data for each respective month and yearly is obtained by adding each successive month's data. Due to incompleteness of rainfall data three additional neighboring rainfall stations that are relatively near to the study area are used to fill missed data as presented in table 4.1 below.

Table 4.1: Spatial description of weather stations which are found near to Alaba Kulito

Name of stations		Alaba kulito	Durame	Hosana	Awassa
Geographic position	Latitude (N)	7.31058°	7.2°	7.5673°	7.065°
	Longitude (E)	38.09392°	37.95°	37.8538°	38.48306°
	Elevation (m)	1772	2000	2307	1694
Distance from A/ Kulito weather station (km)		0	20	38.8	50.8
Elevation(m) with respect to A/Kulito		0	228	535	-78
Annual normal rainfall		1043	1156	1173	965
Percentage of missed RF data		3.7	4.2	2.5	2.7

#### 4.1.2. Filling Missed Rainfall Data

After missed rainfall data are filled by using equation 3.1; each daily data of the months are added together for each consecutive month and each monthly data is further added together to get yearly data. Then the yearly and cumulative yearly data of the four stations is obtained as shown in appendix A.

#### 4.1.3 Checking consistency by Double-mass curve (DMC)

Here, the average cumulative precipitation of neighboring stations (Durame, Awassa, Hosana and Alaba Kulito) is consecutively arranged in the reverse chronological order and plotted with the yearly cumulative of precipitation of Alaba Kulito station for the corresponding years. The table

value of appendix A (table A1) is used to draw the Double Mass Curve (DMC) of figure 4.1 and figure 4.2.

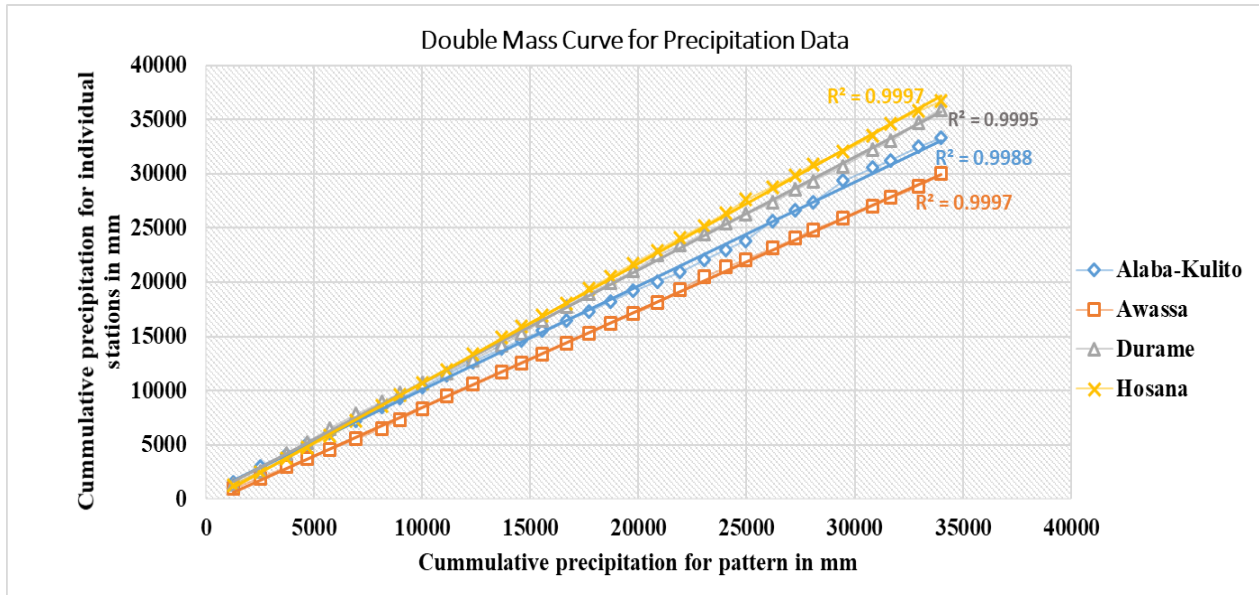


Figure 4.1: Double Mass Curve of rainfall data of individual stations

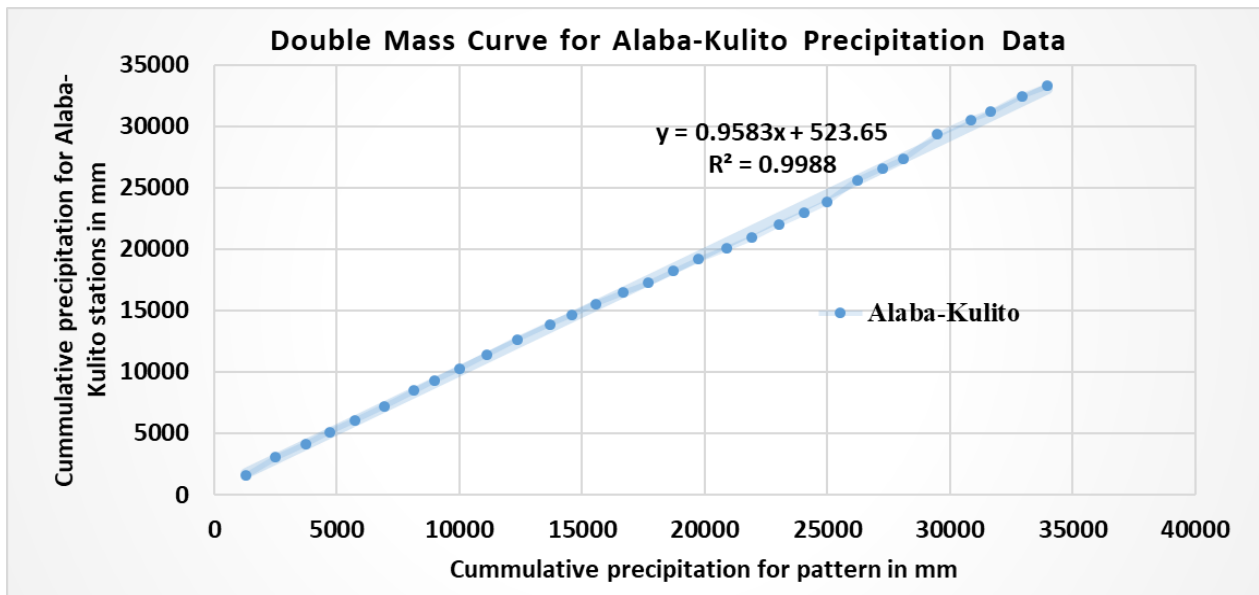


Figure 4.2: Double Mass Curve of rainfall data of Alaba-Kulito station

In this study, the result of the applied consistency checking method (double mas curve)shows that, all stations results in good consistency having a straight line and R-square values near to unit (0.9988, 0.9997, 0.9995 and 0.9997 for Alaba Kulito, hosanna, Durame and Awassa respectively).

#### 4.1.4 Homogeneity test

According to Homogeneity test analysis, the selected stations were plotted for comparison with each other. The result is displayed in the figure below (figure 4.3).

Table 4.2: Average monthly rainfall (mm) of 33 years of four stations

Months	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	
Stations	A/kulito	27.5	49.7	91.3	137.1	126.4	93.4	115.3	154.8	121.0	73.8	63.2	23.0
	Durame	24.0	45.3	86.2	148.8	154.3	113.3	158.6	160.3	141.6	97.3	44.7	23.9
	Awassa	26.1	35.9	75.0	111.7	125.8	103.3	122.1	122.7	118.6	72.8	38.2	25.9
	Hosana	26.8	43.1	99.4	141.7	149.5	127.7	152.3	176.7	155.5	72.5	31.3	26.6

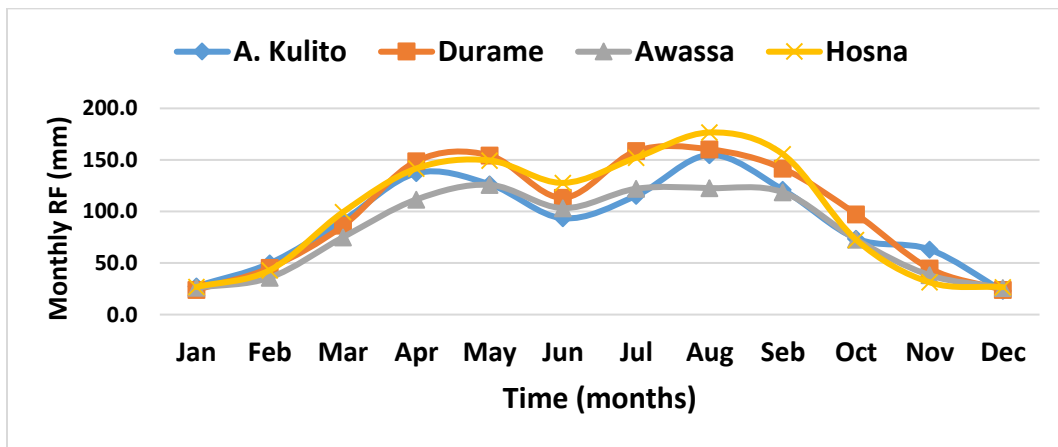


Figure 4.3: Homogeneity of the rainfall areas of the representative stations

This shows that the result of homogeneity analysis resulting Same-mode (bi-modal) and pattern of the stations are observed and hence group stations selected are homogenous since all shows likely similar patterns.

#### 4.1.5 Determination of extreme value

As mentioned in section 3.2.1.5; the exceedances magnitudes in the POT series is prepared to be GPD which is concerned in this study.

##### ❖ Peak Over Threshold Method

According to section 3.2.1.5; the maximum number of observations considered for further analysis are 99 (i.e. 33\*3) data sets. The data sorted in ascending order for maximum daily rainfall of possible maximum number (99) as stated above is shown in appendix A (table A3).

By using equation 3.3, the MRL plot can be developed as shown by figure 4.4.

Table 4.3: Mean residual versus threshold values

Threshold	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54
Mean Residual	12.8	12.3	12.8	12.8	13	12.7	12.3	12.1	11.6	11.2	10.6	10.9	12	11.2	11.5	11	11
Threshold	55	56	57	59	60	61	62	63	64	65	66	68	71	73	74	75	79
Mean Residual	10.7	11.5	12.8	12	11.7	11.5	11.2	9.6	9.9	8.9	8.6	10.3	7.3	6.2	6.2	6.2	3.7

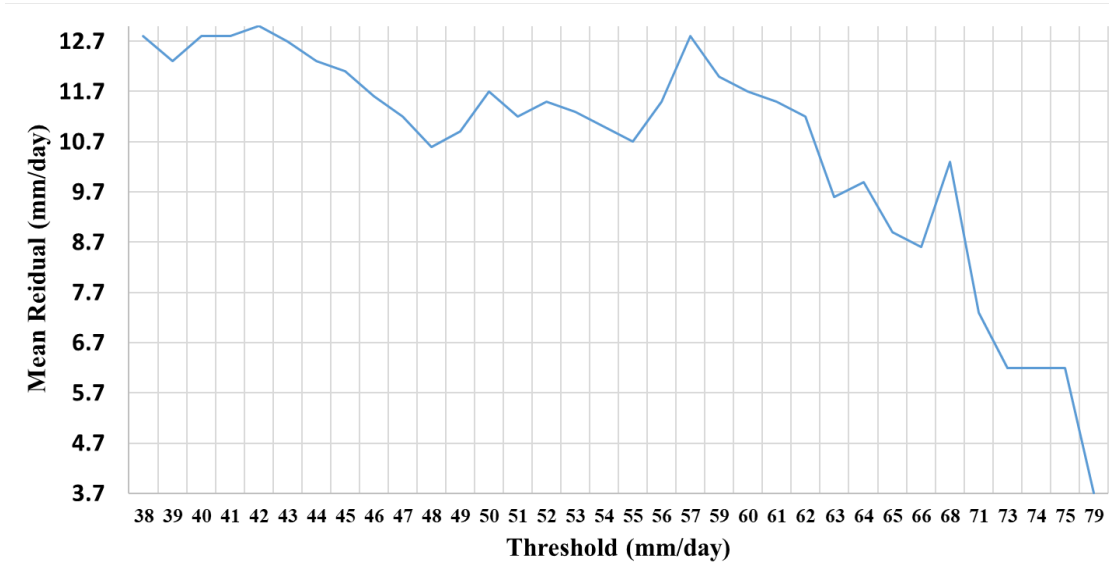


Figure 4.4: Mean Residual Life Plot (MRLP) of the reference period rainfall data

The graph (figure 4.4) shows that there is a sort of slight linear relationship between MRV and threshold in the range between (48 and 50), (55 and 57) and (66 and 68) relatively.

Here; by trial and error, values above 48.4 mm/day fits best the GPD ranking first, second and third-order for Chi-Squared, Kolmogorov Smirnov and Anderson Darling respectively than others.

As mentioned in section 3.2.1.6; GPD is used to run the process of possible maximum discharge determination.

#### 4.1.6 Detecting outlier

As mentioned in section 3.2.1.7; by using equation 3.7 outlier detection is summarized in the table below (table 4.4) for POT approach.

Table 4.4: Summary of outlier detection

Type of Ext. Value determination	N	Mean	Standard deviation	Lower limit	Upper limit	Rejected	
						value/s	Reason
POT	51	58.1	10.24	-28.26	89.7	None	(48.4 & 86) are within (-28.26 & 89.7)

Accordingly; out of 99 sample data (3\* number of years (33)), 48.2 mm/day is selected as the threshold value of POT based on MRLP and the remaining values (48.4mm/day up to 86mm/day which are 51 sample data) are used. Furthermore with  $\mu = 58.1$ ,  $\sigma = 10.24$  and  $N = 51$ , none of data deviates from range (-28.26 and 89.7). Remarkably, all values fall under the limit as illustrated in table 4.5 above. In the same manner, all data sets (33 years daily maximum RF data) lays between lower and upper limits. Therefore, the selected data can be used for the remaining hydraulic and hydrologic analyses.

#### 4.1.7 Generalized Pareto Distribution Method

Graphical and statistical methods are used to test the goodness-of-fit of the distribution of the extreme rainfall values to the selected test distribution (GPD in this study). As illustrated in section 3.2.1.6; the statistical method of testing goodness-of-fit is summarized in table 4.4 and graphical value is plotted in appendix A (figure A1) from which the following discussion is made. The output of Easyfit 5.6 software for GPD is summarized in table 4.5 below.

Table 4.5: Basic parameters obtained from the output of EasyFit 5.6 Professional Software

Distribution	Chi-Squared	Anderson Darling	Kolmogorov Smirnov	k	$\mu$	$\sigma$	Reject?	Sample size
Gen. Pareto	Rank	Rank	Rank	-0.0261	47.669	11.394	no	51(POT)
	1	2	3					

It can be seen from these plots that the empirical and model probability values and quintile values are very close to the 1:1 line (45° diagonal line), indicating that the selected threshold value is good enough and the GPD function has well fitted the extreme values. Moreover, the KS coefficient, which was found to be 0.07504, indicates that the GPD fits the extreme values above the selected threshold value as the KS value is less than the critical value at 0.01 (0.22386) significance level.

IDF curve is developed by combining the RR<sub>t</sub> value of the table (table 3.3) for different durations with 24 hr Rainfall depth of Alaba Kulito station for different return periods.

Table 4.6: Return period, percentage of probability and 24 hr RF depth (mm) of GPD

T	2	5	10	25	50	100
$P = 1 - \frac{1}{T}$	0.5	0.8	0.9	0.96	0.98	0.99
<b>R24 of A. Kulito of GDP</b>	55.49	65.626	73.131	82.845	90.041	97.108

The estimation results is shown in table 4.7 and the corresponding IDF curve is plotted (Figure 4.5).

Table 4.7: IDF values of GPD for Alaba Kulito station

		Intensity I(mm/hr)								
Duration (hr)		0.0833	0.1667	0.25	0.5	1	1.5	2	2.5	3
Duration (min)		5min	10min	15min	30min	60min	90min	120min	150min	180min
Return period (T)	2	105.2	87.8	77.7	53.5	34.2	25.3	20.2	16.9	14.5
	5	124.4	103.8	91.9	63.2	40.4	30.0	23.9	20.0	17.2
	10	138.6	115.7	102.4	70.4	45.1	33.4	26.7	22.2	19.1
	25	157.0	131.0	116.0	79.8	51.0	37.8	30.2	25.2	21.7
	50	170.6	142.4	126.1	86.7	55.5	41.1	32.8	27.4	23.5
	100	184.0	153.6	136.0	93.5	59.8	44.4	35.4	29.5	25.4
	1000	226.8	189.3	167.6	115.3	73.8	54.7	43.6	36.4	31.3

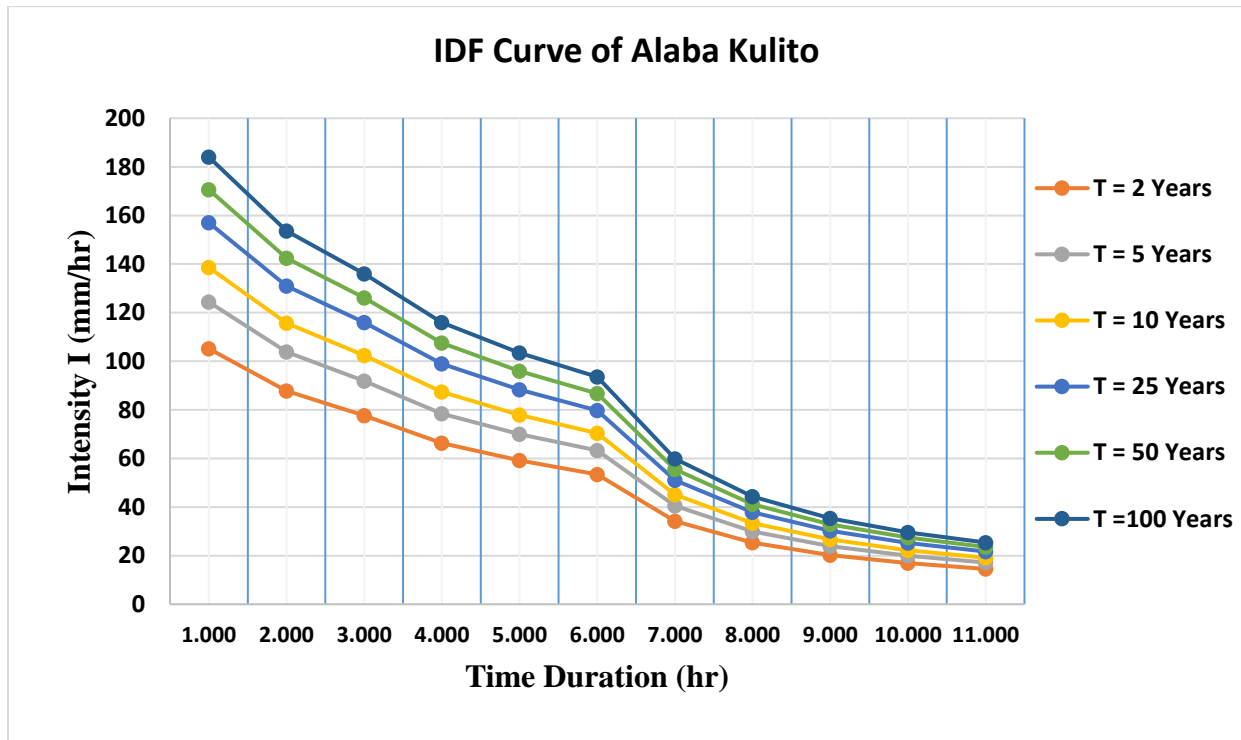


Figure 4.5: IDF curves of 33 years of rainfall data for Alaba Kulito by GPD

#### 4.1.8 ERA's IDF curve of Region B2

Alaba Kulito is found under region B2 according to ERA classification of region of precipitation in which three meteorological stations (Arbaminch, Sodo and Hawassa) are used as representative stations (ERA, 2013).

Table 4.8: Intensity for duration versus return period of region B2 developed by ERA

		Intensity I(mm/hr)								
Duration (hr)		0.0833	0.1667	0.25	0.5	1	1.5	2	2.5	3
Duration (min)		5min	10min	15min	30min	60min	90min	120min	150min	180min
Return period (T)	2	96.4	80.7	69.6	49.7	32.1	24	19.2	16.1	13.9
	5	122	102.2	88.2	62.9	40.7	30.3	24.3	20.4	17.6
	10	139	116.4	100.4	71.7	46.3	34.5	27.7	23.2	20
	25	160.5	134.5	116	82.8	53.5	39.9	32	26.8	23.1
	50	176.7	148	127.7	91.1	58.9	43.9	35.2	29.5	25.5
	100	192.9	161.6	139.4	99.5	64.3	48	38.5	32.2	27.8
	200	227.5	189.9	163.2	143.4	127.9	115.6	74.0	54.8	43.8

Based on this table the following curve is developed

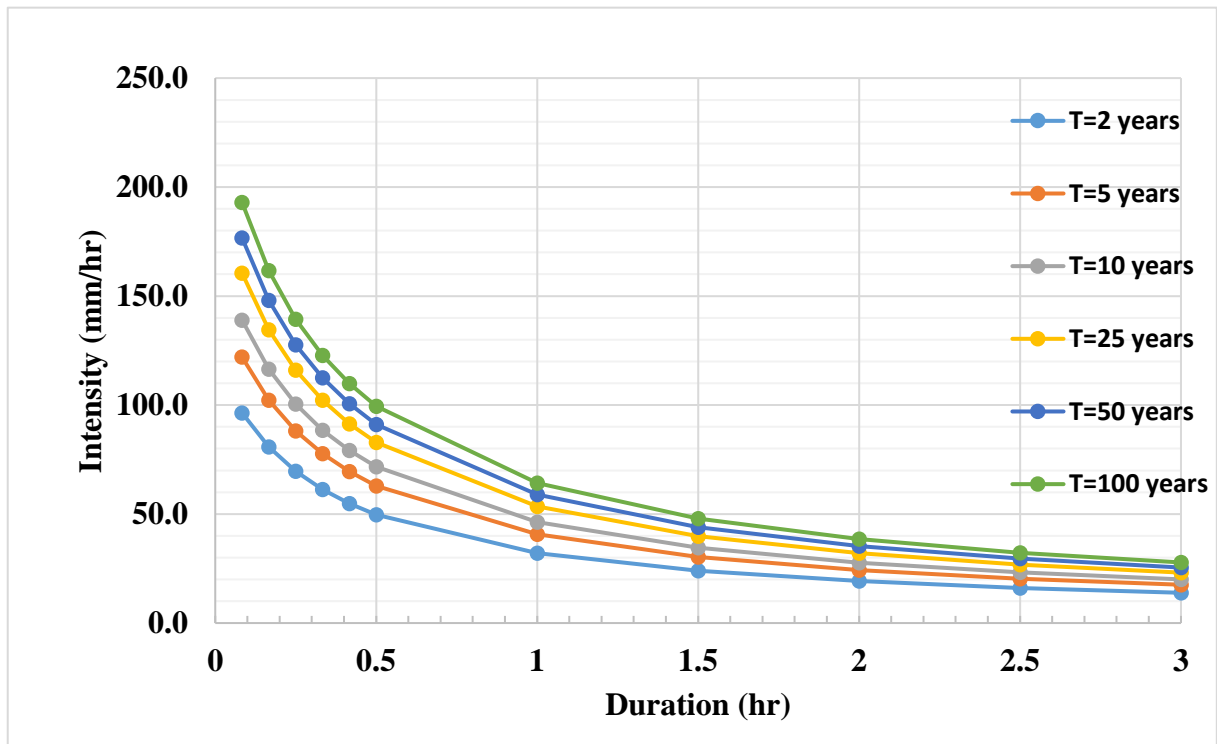


Figure 4.6: IDF of Alaba Kulito (region –B2) of ERA

As observed from consecutive IDF curves, it is marked that the rainfall intensity of ERA is greater than GPD for almost all return periods which is illustrated in appendix A (table A6). For further hydrological and hydraulics analysis/process IDF curves developed by GPD is used due to the best representation of the area than ERA’s IDF which is slightly magnified as it considers far stations (Arbaminch and Sodo) and not-updated rainfall data of before two decades (ERA, 2013).

## 4.2 Assessment of Hydraulic Capacity of Existing Drainage canal and Hydrologic Response of Catchment to Rainfall Intensity

### 4.2.1 Peak Discharge Computation and comparison with the drainage capacity

Peak discharge estimation is mandatory to design any hydraulic structures of conveyance systems like channels, culverts and bridges. As mentioned in section 3.2.2, both the rational method (for  $\leq 50$ ha) and Soil Conservation Service-Curve Number (SCS-CN) method (for  $> 50$ ha) with SWMM model simulation result are used to compute peak discharges and compared with Manning's result.

### 4.2.2 Property of Study Area

#### 4.2.2.1 Land-use map of the study area

Land-use type and its hydrological analysis help us to estimate the response of the catchment to the precipitation. As the basis of hydrologic impact evaluation, the land-use study was carried out with the help of Google Earth pro and ArcGIS 10.4.1 software with detail physical observation of the considerable study area including land-use types and existing drainage lines. The result obtained is illustrated as shown in figure 4.7 below.

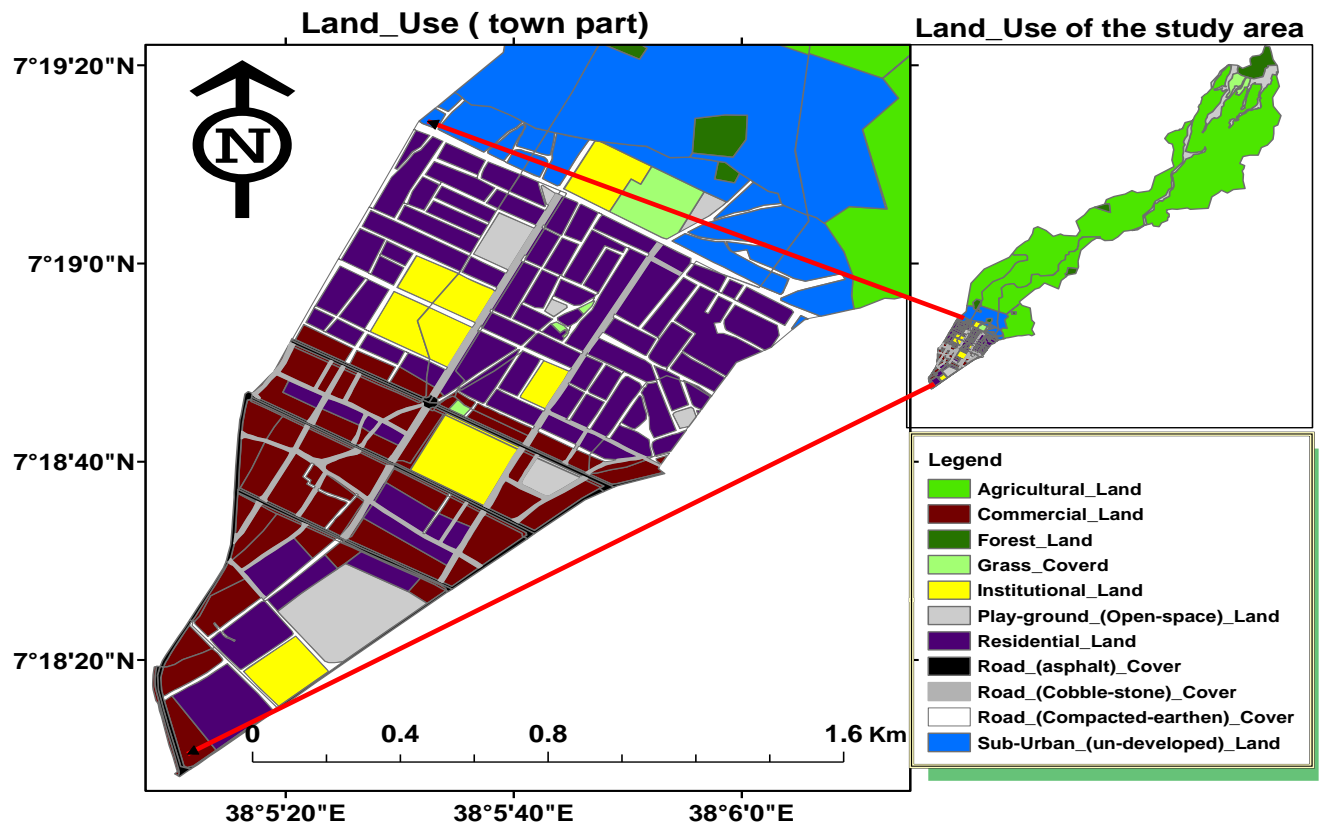


Figure 4.7: Study Area Land Use/Land Cover

According to the study; eleven land use/land covers are distinguished and coded as shown in table 4.9 with their respective area (ha), land-use percentage from coverage of study area (%), and representative runoff coefficients (C).

Table 4.9: Land Use Composition and Runoff coefficient of study Area

Land-Use/Cover	Land-use code	Area (Ha)	Percentage (%)	Runoff cof.(C)
Agricultural	LU-AG	1318.6	76.8	0.4
Commercial	LU-CM	37.0361	2.2	0.8
Forest	LU-FR	61.7704	3.6	0.3
Grass	LU-GR	23.8986	1.4	0.35
Institutional	LU-IT	16.9837	1.0	0.5
Play-ground (Open Space)	LU-OS	102.4137	6.0	0.3
Residential	LU-RS	66.32	3.9	0.5
Road (Asphalt)	LU-RA	5.5805	0.3	0.9
Road (Cobble-Stone)	LU-RC	7.64219	0.4	0.7
Road (Compacted Earth)	LU-RE	18.19991	1.1	0.45
Sub-Urban (Un-Developed)	LU-SU	59.0185	3.4	0.4
Sum		1717.46	100.00	

#### 4.2.2.2 Flow Direction and Drainage Network of Study Area

As urban drainage is factored by manmade drainage structures along and the sides of the roads from collector to the outlet through main conduits; well-connected drainage lines are necessarily needed in the urban drainage system to drain the collected runoff safely. Both Lenda Ber and Mahal Arada kebele are connected by the same drainage lines by which Lenda Ber being the upper catchment and drains toward outfall located at Mahal Arada around ST Gabriel Church.

The drainage network follows the geographical topography/contour with necessary modifications made according to the land-use and road pattern of the town. The existing drainage lines are used to divide sub-catchments where SC01 (sub-catchment 01) being the uppermost sub-catchment and SC30 (sub-catchment 30) is found at the lower with the coded links. Accordingly, there are 37 drainage lines with respective sub-catchments connected as shown in the figure below (figure 4.8).

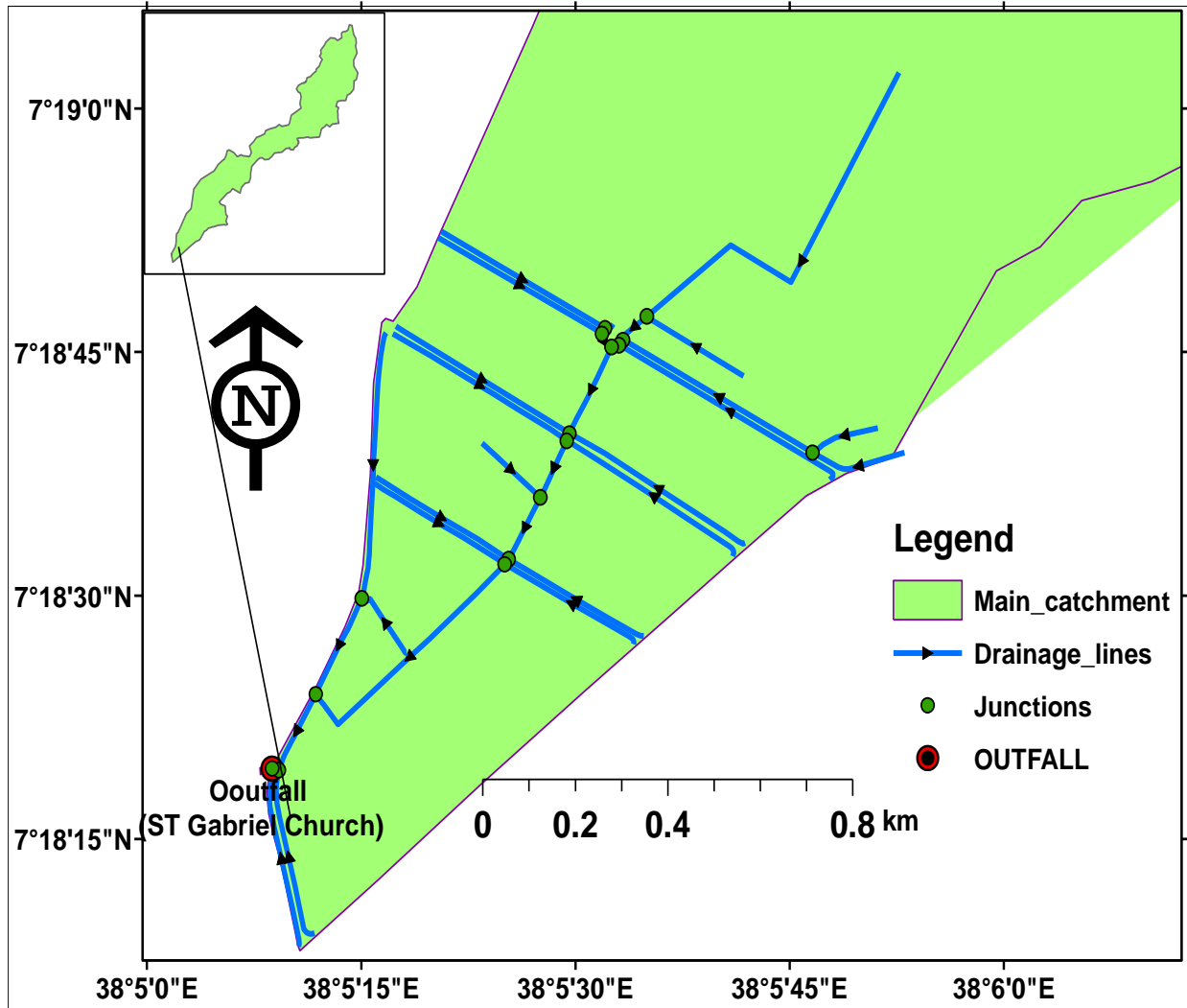


Fig 4.8: Flow Direction of links

#### 4.2.2.3 Sub-catchment, LandUse Compositions and Weighted Runoff Coefficient ( $C_w$ )

Initially, raw data from Elevation Model (DEM) was analyzed by ArcGIS10.4.1 terrain processing tool with an input stream network. The output 30 sub-catchments were obtained with the input stream network which is an existing link network of study area since the urban drainage is not as such a natural way, sub-catchments are done by the pattern of elevation differences following the consecutive drainage lines. LandUse Compositions and Weighted Runoff Coefficients of each sub-catchments are summarized in table 4.10 below.

The delineation was followed several steps such as DEM reconditioning, filling sink in DEM, assigning flow direction and flow accumulation. Initially, the area of each sub-catchment was calculated by Google Earth pro interface and imported as kml to ArcGIS View.

Table 4.10: Each Sub-catchment Land Use Compositions and Weighted Runoff Coefficient

Sub-Catchments	Composition of LU/LC for each Sub-catchments (ha)												Cw
	LU-AG	LU-CM	LU-FR	LU-GR	LU-IT	LU-OS	LU-RS	LU-RA	LU-RC	LU-RE	LU-SU	Sum	
SC01	1086.		52.55	17.42	2.72	92.61	15.34		0.72	4.79	33.09	1305.24	0.39
SC02	69.60	1.81	3.99	6.33	3.13		14.11	0.26		5.29	12.21	116.73	0.42
SC03	163.0	1.55	5.23	0.15			14.39	0.25	0.52	3.78	13.72	202.58	0.41
SC04		0.10			2.95	1.69	1.93	0.03	0.51	0.90		8.12	0.47
SC05		0.06					0.66	0.04	0.68	0.10		1.54	0.61
SC06		0.54			1.04		2.21		0.11	0.48		4.38	0.54
SC07		0.97				0.27	3.43		0.22	0.87		5.76	0.54
SC08								0.12	0.03	0.04		0.19	0.77
SC09		0.10			0.18	0.10		0.46	0.03			0.87	0.73
SC10		0.01						0.07				0.08	0.89
SC11		0.27						0.26	0.03			0.56	0.84
SC12		5.29					1.33	0.26	1.18			8.06	0.74
SC13		0.70							0.31			1.02	0.77
SC14		2.53			4.43	1.04		0.51	0.73			9.23	0.60
SC15		0.29						0.30	0.01	0.01		0.61	0.84
SC16		0.24						0.25	0.02			0.51	0.85
SC17		2.47						0.50	0.22			3.18	0.81
SC18		4.85						0.20	0.41	0.21		5.67	0.78
SC19		2.08							0.50	0.00		2.59	0.78
SC20		0.03					0.05		0.17			0.24	0.67
SC21		0.10					0.04		0.19			0.33	0.71
SC22		4.98					3.03	0.41	0.71	0.08		9.20	0.69
SC23		0.08					0.08	0.19				0.35	0.79
SC24		0.17						0.22	0.02	0.01		0.42	0.84
SC25		1.65					1.60		0.31			3.56	0.66
SC26		2.31			2.54	6.70	4.38	0.31		1.15		17.38	0.47
SC27		0.47						0.13				0.60	0.82
SC28		0.71						0.13		0.06		0.90	0.79
SC29		2.49					3.74	0.42		0.44		7.09	0.63
SC30		0.20						0.26				0.46	0.86
Sum	1318.6	37.04	61.77	23.90	16.98	102.41	66.32	5.58	7.64	18.20	59.02	1717.46	

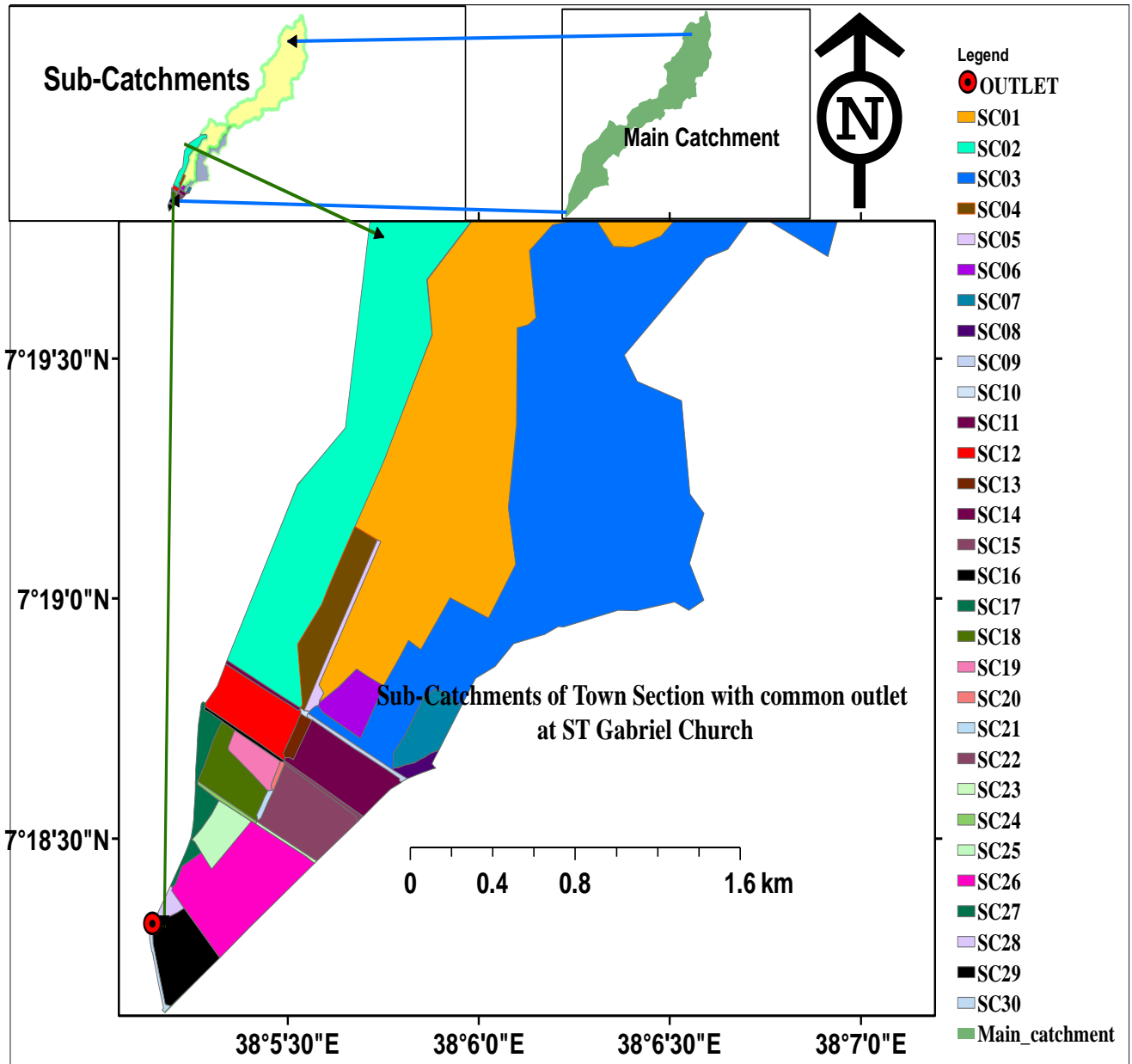


Figure 4.9: Sub-Catchments of the Study Area section

#### 4.2.3 Estimation off Peak Runoff by Rational Method

As stated in section 3.2.2.4, the peak runoff rate for small sub-catchment areas less than 50 ha are done by the rational method. As depicted in Table 4.10 above, about 27 sub-catchments are less than half a kilometer square.

Here is a sample calculation is done for sub-catchment twelve (SC12);

**Step 1.**Sub-catchment area of SC12:  $A_{sc12} = 8.06ha$  (from table 4.11)

**Step 2.** Longest flow path and elevation: length of overland flow is 0.185 km, length of the defined canal is 0.425km, elevation difference for overland flow is 1m.

**Step 3.** Catchment property: Hydrologic soil group is B, land cover is composed of five land-use type as illustrated in table 4.10 above, rainfall region is grouped under B2 but for analysis Intensity done by GPD is selected.

**Step 4.** Time of concentration:

i. For overland flow; for  $C_{v12} = 0.74$ ,  $L = 0.185\text{km}$  and  $S = 0.54054\%$

$$T_{c1} = 0.604 \left( \frac{C_{vi} * L_i}{S^{0.5}} \right)^{0.467} = 0.2753\text{hr}$$

ii. For canal flow;  $L = 425\text{m}$ ,  $V = 2.117\text{m/s}$  (done by manning equation)

$$T_{c2} = \frac{L}{3600V} = 0.05576\text{hr}$$

$$T_c = T_{c1} + T_{c2} = 0.05576\text{hr} + 0.2753\text{hr} = 0.3311\text{hr} = 20\text{min}$$

**Step 5.** Rainfall intensity: The rainfall intensity of different return period is done as shown in appendix A (table A5).

Accordingly; Intensity for sub-catchment 12 is found to be:  $I_2 = 66.48$ ,  $I_5 = 78.62$ ,  $I_{10} = 87.61\text{mm/hr}$ ,  $I_{25} = 99.25\text{mm/hr}$ ,  $I_{50} = 107.87\text{mm/hr}$  and  $I_{100} = 116.33\text{mm/hr}$ .

**Step 6.** The runoff coefficient were depended upon vegetation cover (if there is), land use type and inclination of respective sub-catchment slope and it is 0.74 for SC12.

**Step 7.** Peak flood,  $Q = C_f * C * I * A$  (all parameters by SI unit);

T (year)	2	5	10	25	50	100
$C_f$	1	1	1	1.1	1.2	1.25
I (mm/hr)	66.48	78.62	87.61	99.25	107.87	116.33
Q (m <sup>3</sup> /s)	1.10	1.30	1.45	1.81	2.14	2.41

Hydrological analysis done for the remaining sub-catchments is depicted in table 4.11 below.

Table 4.11: Peak Discharge by Rational Method

Return Period (T)			2	5	10	25	50	100
Frequency Factors (Cf)			1	1	1	1.1	1.2	1.25
SC	C	A(ha)	Q (m <sup>3</sup> /s)					
SC04	0.471	8.12	0.65	0.77	0.86	1.07	1.27	1.42
SC05	0.605	1.54	0.14	0.17	0.19	0.23	0.27	0.31

SC06	0.537	4.38	0.50	0.59	0.66	0.82	0.97	1.09
SC07	0.541	5.76	0.57	0.68	0.75	0.94	1.11	1.25
SC08	0.769	0.19	0.03	0.04	0.04	0.06	0.07	0.07
SC09	0.728	0.87	0.16	0.18	0.20	0.25	0.30	0.34
SC10	0.886	0.08	0.02	0.02	0.02	0.03	0.04	0.04
SC11	0.841	0.56	0.12	0.15	0.16	0.20	0.24	0.27
SC12	0.739	8.06	1.10	1.30	1.45	1.81	2.14	2.41
SC13	0.769	1.02	0.21	0.25	0.28	0.35	0.42	0.47
SC14	0.598	9.23	1.11	1.31	1.47	1.83	2.16	2.43
SC15	0.843	0.61	0.12	0.15	0.16	0.21	0.24	0.27
SC16	0.846	0.51	0.12	0.14	0.15	0.19	0.23	0.25
SC17	0.809	3.18	0.56	0.66	0.73	0.91	1.08	1.21
SC18	0.784	5.67	0.84	0.99	1.10	1.37	1.63	1.83
SC19	0.780	2.59	0.46	0.55	0.61	0.76	0.90	1.01
SC20	0.674	0.24	0.05	0.06	0.06	0.08	0.10	0.11
SC21	0.709	0.33	0.07	0.08	0.09	0.11	0.13	0.15
SC22	0.695	9.20	1.20	1.42	1.58	1.97	2.34	2.63
SC23	0.785	0.35	0.08	0.09	0.10	0.13	0.15	0.17
SC24	0.841	0.42	0.10	0.11	0.13	0.16	0.19	0.21
SC25	0.656	3.56	0.47	0.55	0.61	0.76	0.91	1.02
SC26	0.467	17.38	1.50	1.77	1.98	2.46	2.92	3.28
SC27	0.821	0.60	0.12	0.14	0.15	0.19	0.23	0.26
SC28	0.791	0.90	0.16	0.18	0.21	0.26	0.30	0.34
SC29	0.626	7.09	0.78	0.92	1.03	1.28	1.52	1.71
SC30	0.857	0.46	0.08	0.10	0.11	0.13	0.16	0.18
$Q(m^3/s) = C * C_f * A(ha) * I(mm/hr) / 360$								

#### 4.2.4 Estimation of Peak Runoff by SCS-CN Method

##### ❖ Area and Land-use/cover percentage

As the area of three sub-catchments (SC01, SC02 and SC03) is greater than 50ha, they are done by SCS-CN method. For each sub-sections, their respective catchment area (table 4.12), land-use percentage (table 4.12) and weighted runoff coefficient (table 4.13) are calculated as shown in each respective tables.

Table 4.12: Area and Land-use/cover percentage of Sub-Catchments done by SCS

Sub-cat chments		LU/LC percentage (%) for each sub-catchments											SUM
		LU-AG	LU-CM	LU-FR	LU-GR	LU-IT	LU-OS	LU-RS	LU-RA	LU-RC	LU-RE	LU-SU	
SC01	A (ha)	1086.00	0.00	52.55	17.42	2.72	92.61	15.34	0.00	0.72	4.79	33.09	1305.24
	LU(%)	83.20	0.00	4.03	1.33	0.21	7.10	1.18	0.00	0.06	0.37	2.54	100.00
SC02	A (ha)	69.60	1.81	3.99	6.33	3.13	0.00	14.11	0.26	0.00	5.29	12.21	116.73
	LU(%)	59.62	1.55	3.42	5.42	2.68	0.00	12.09	0.23	0.00	4.53	10.46	100.00
SC03	A (ha)	163.00	1.55	5.23	0.15	0.00	0.00	14.39	0.25	0.52	3.78	13.72	202.58
	LU(%)	80.46	0.77	2.58	0.07	0.00	0.00	7.10	0.12	0.26	1.86	6.77	100.00

❖ **Weighted Curve Number**

Weighted runoff coefficients are obtained by dividing the sum of the product of each sub-catchment area by LU percentage to the summation of land-use percentage. Curve number values are selected by considering soil group, slope, and vegetation coverage as depicted in appendix B (table B3).

Table 4.13: Weighted Curve Number

Sub- cat chments	Product of percentage of land-use/land-cover by curve number												Su m	CN <sub>w</sub>
	LU-type	LU-AG	LU-CM	LU-FR	LU-GR	LU-IT	LU-OS	LU-RS	LU-RA	LU-RC	LU-RE	LU-SU		
	CN	71	92	55	61	70	61	68	98	85	82	69		
SC01	LU (%)	83.2	0.0	4.0	1.3	0.2	7.1	1.2	0.0	0.1	0.4	2.5	100.0	69.47
	CN*L U%	5907.4	0.0	221.4	81.4	14.6	432.8	79.9	0.0	4.7	30.1	174.9	6947.3	
SC02	LU(%)	59.6	1.6	3.4	5.4	2.7	0.0	12.1	0.2	0.0	4.5	10.5	100.0	70.20
	CN*L U%	4233.4	142.7	188.2	330.9	187.4	0.0	822.0	22.2	0.0	371.6	721.7	7020	
SC03	LU(%)	80.5	0.8	2.6	0.1	0.0	0.0	7.1	0.1	0.3	1.9	6.8	100.0	70.67
	CN*L U%	5712.8	70.4	141.9	4.5	0.0	0.0	482.9	12.0	21.9	152.9	467.2	7066.5	

The 24 hr rainfall depth of table 4.6 and equation 3.18, 3.19 and 3.20 are used to calculate accumulated runoff (Q), initial abstraction (I) and maximum potential retention (S) respectively.

Table 4.14: Accumulated precipitation (P) in mm and Ia/P

cat chments	Number (CN)	S (mm)	Ia (mm)	Accumulated precipitation (P) in mm for return period of 2, 5, 10, 25, 50 and 100 respectively											
				55.5	65.63	73.1	82.8	90.04	97.11	55.5	65.6	73.1	82.85	90.041	97.11
				Accumulated direct rain fall (Q) in mm						Ia/P					
SC01	69.47	111.63	22.33	7.4	11.9	15.6	21.0	25.3	29.7	0.13	0.18	0.21	0.25	0.28	0.31
SC02	70.20	107.82	21.56	8.0	12.7	16.6	22.1	26.5	31.0	0.14	0.19	0.23	0.27	0.29	0.32
SC03	70.66	105.47	21.09	8.6	13.4	17.4	23.0	27.5	32.1	0.16	0.20	0.24	0.28	0.31	0.33

❖ **Unit peak discharge, ( $m^3/s/km^2$ )/mm**

Equation 3.21 is used to solve unit peak discharge which is the values of the ratio of initial abstraction to 24 hr rainfall depth(table 4.15).

Regression coefficients  $C_0$ ,  $C_1$  and  $C_2$  (from table 3.5) are used. The values which are not found in the table are interpolated as shown in Appendix B (table B4).

Table 4.15: unit peak discharge, ( $m^3/s/km^2$ )/mm

Sub-catchment	Tc (min)	Tc (hr)	Log (tc(hr))	qu ( $(m^3/s/km^2)/(mm)$ ) for return period of 2, 5, 10, 25, 50 and 100 respectively					
				2	5	10	25	50	100
SC01	69.46	1.16	0.064	0.136288	0.1289	0.1308	0.1206	0.1170	0.1123
SC02	51.22	0.85	-0.069	0.162883	0.1542	0.1487	0.1445	0.1401	0.1328
SC03	62.04	1.03	0.015	0.141505	0.1368	0.1303	0.1257	0.1206	0.1156

❖ **Peak discharge (qp)**

Equation 3.17 is used for estimation of the peak discharge in SCS method.

Table 4.16: Peak discharge (qp) of Sub-catchments done by SCS-CN

Sub-cat chments	Area ( $km^2$ )	Peak Discharge $qp = (Q * A * qu)$ in $m^3/s$						
		T	2	5	10	25	50	100
SC01	13.052	Q	7.27	11.68	15.39	20.69	24.92	29.29
		qu	0.136	0.129	0.131	0.121	0.117	0.112
		<b>qp</b>	<b>12.94</b>	<b>19.65</b>	<b>26.28</b>	<b>32.57</b>	<b>38.05</b>	<b>42.92</b>
		Q	7.98	12.59	16.46	21.95	26.31	30.81

SC02	1.167	qu	0.163	0.154	0.149	0.145	0.140	0.133
		<i>qp</i>	<i>1.51</i>	<i>2.25</i>	<i>2.84</i>	<i>3.68</i>	<i>4.28</i>	<i>4.75</i>
SC03	2.026	Q	8.72	13.55	17.57	23.25	27.75	32.37
		qu	0.142	0.137	0.130	0.126	0.121	0.116
		<i>qp</i>	<i>2.50</i>	<i>3.76</i>	<i>4.64</i>	<i>5.92</i>	<i>6.78</i>	<i>7.58</i>

#### 4.2.5 Estimation of Drainage Capacity by Manning Equation

The existing drainage canals of both Lenda Ber and Mahal Arada Kebele are considered in this study as these kebeles are prone to flood than the remaining three kebeles (Wanaja, Denebefama and Murasa Kebele). Most of the drainage facilities are open canals constructed by masonry and concrete along the main road, sub-main and local roads; whereas concrete made closed canals are found only along the main road of Hossana-Alaba Kulito-Sodo road way.

The physical parameters of existing canals like shape, type and size are used to analyze the hydraulic capacity of conduits under the study area. A tape meter is used to measure the depth and width of the canals. According to a field survey; all canals are open rectangular masonry except L20L, L21L, L22L, L23L and L23R which are closed links and found along the main asphalt road of Mehal Arada kebele.

In order to apply equation 3.22 and equation 3.23; perimeter, hydraulic radius, and slope of the terrain (appendix D) are derived from canal depth, canal width and an elevation difference of starting and ending of successive links. The values obtained by the manning equation is shown in Appendix D and under the section of adequacy analysis which is compared with the output of the SWMM model, and rational/SCS method.

#### 4.2.6 Simulation of Drainage Network with EPA SWMM 5.1.

In this study, the parameters inputted are from sub-catchment, node, conduit, and rain gage. The sub-catchment area was divided into 30 sub-catchments, 37 junctions, 37 conduits and 1 outfall as shown in the figure below (figure 4.10).

As there is no simulation made by any model for the study area, the peak discharge value of rational and SCS is used to adjust the sensitive parameters. Accordingly; D-store-impervious, D-store-pervious, N-impervious and N- pervious are adjusted to be 1.65, 3.01, 0.029 and 0.1 respectively. The simulation was done by using 3hours rainfall intensity with a 5minute time interval and the remaining parameter values of sub-catchment, link and node used for simulation are found in Appendix E (table E1, table E2 and table E3 respectively).



## Model outcomes

### ❖ Peak discharge of each sub-catchments for return period of 2, 10, 25, 50 and 100

The statistical analysis by SWMM is performed for the study area and peak discharge generated from sub-catchments is shown in the table below (Table 4.17).

Table 4.17: Peak discharge result of SWMM for return periods of 2, 5, 10, 25, 50 and 100 years

SC-Code	Q m <sup>3</sup> /s					
SC01	15.42	21.37	26.27	33.17	38.69	44.36
SC02	2.2	2.99	3.63	4.51	5.21	5.91
SC03	3.18	4.31	5.23	6.52	7.54	8.58
SC04	0.55	0.69	0.8	0.95	1.07	1.18
SC05	0.12	0.16	0.18	0.21	0.24	0.26
SC06	0.43	0.54	0.62	0.74	0.82	0.91
SC07	0.55	0.68	0.79	0.93	1.04	1.14
SC08	0.03	0.03	0.04	0.04	0.05	0.05
SC09	0.16	0.19	0.22	0.26	0.28	0.31
SC10	0.02	0.02	0.02	0.03	0.03	0.03
SC11	0.12	0.14	0.16	0.18	0.2	0.21
SC12	0.97	1.2	1.37	1.61	1.79	1.98
SC13	0.16	0.19	0.22	0.26	0.29	0.32
SC14	1.02	1.27	1.46	1.72	1.92	2.12
SC15	0.12	0.14	0.16	0.19	0.21	0.22
SC16	0.11	0.14	0.15	0.17	0.19	0.21
SC17	0.51	0.63	0.72	0.84	0.93	1.02
SC18	0.77	0.96	1.1	1.29	1.43	1.57
SC19	0.4	0.49	0.57	0.66	0.73	0.8
SC20	0.04	0.05	0.06	0.06	0.07	0.08
SC21	0.06	0.07	0.08	0.09	0.1	0.11
SC22	1.1	1.36	1.57	1.83	2.04	2.25
SC23	0.07	0.09	0.1	0.11	0.12	0.13
SC24	0.09	0.11	0.12	0.14	0.15	0.16
SC25	0.41	0.5	0.58	0.68	0.76	0.84
SC26	1.33	1.67	1.94	2.31	2.6	2.89
SC27	0.09	0.11	0.13	0.15	0.16	0.18
SC28	0.12	0.15	0.18	0.21	0.23	0.25
SC29	0.75	0.93	1.07	1.26	1.41	1.56
SC30	0.09	0.11	0.12	0.14	0.15	0.17

### ❖ Flooding Results

The flooding volume and flooding hours of flooded junctions for different return periods is summarized in the table below (Table 4.18).

Table 4.18: Flooded junctions for return period of 2, 5, 10, 25, 50 and 100 years

No. of flooded junctions	flooded nodes	Flooding hours	Flood volume (10 <sup>6</sup> ltr)	No. of flooded junctions	flooded nodes	Flooding hours	Flood volume (10 <sup>6</sup> ltr)	No. of flooded junctions	flooded nodes	Flooding hours	Flood volume (10 <sup>6</sup> ltr)
<b>2 Years return period</b>				<b>5 years return period</b>				<b>10 years return period</b>			
1	J01	2:55	114.9	1	J01	2:55	158.6	1	J01	2:55	194.6
2	J05	2:55	10.5	2	J05	2:55	16.7	2	J05	2:55	21.8
3	J11	2:51	15.7	3	J11	2:52	25.3	3	J09	0:30	0.1
4	J17	0:25	2.1	4	J17	0:25	5.7	4	J11	2:55	33.2
5	J18	0:05	0.0	5	J18	0:04	0.0	5	J17	0:25	8.9
6	J25	0:25	0.2	6	J21	0:15	0.0	6	J18	0:04	0.0
7	J26	0:20	15.4	7	J25	0:25	0.8	7	J21	0:14	0.0
8	J34	0:26	1.6	8	J26	0:18	19.8	8	J24	0:20	0.0
9	J36	0:24	7.7	9	J34	0:23	4.7	9	J25	0:25	1.4
<b>25 years return period</b>				10	J36	0:015	8.3	10	J26	0:17	22.3
1	J01	2:55	245.6	<b>50 years return period</b>				11	J34	0:21	7.5
2	J05	2:55	28.9	1	J01	2:55	286.5	12	J35	0:18	0.0
3	J09	0:30	0.5	2	J05	2:55	34.6	13	J36	0:01	8.7
4	J11	2:54	43.9	3	J09	0:30	0.9	<b>100 years return period</b>			
5	J17	0:26	13.5	4	J11	2:53	52.4	1	J01	2:55	328.7
6	J18	0:04	0.2	5	J17	0:25	17.1	2	J05	2:55	40.4
7	J21	0:12	0.0	6	J18	0:03	0.6	3	J09	0:30	1.3
8	J24	0:20	0.3	7	J21	0:12	0.0	4	J11	2:53	61.1
9	J25	0:25	2.3	8	J24	0:20	0.6	5	J17	0:25	20.8
10	J26	0:15	24.6	9	J25	0:25	3.1	6	J18	0:03	1.0
11	J31	0:20	0.0	10	J26	0:15	25.9	7	J21	0:11	0.0
12	J34	0:25	11.5	11	J31	0:20	0.2	8	J24	0:20	0.9
13	J35	0:16	0.0	12	J34	0:24	14.5	9	J25	0:20	4.0
14	J36	0:17	9.1	13	J35	0:15	0.1	10	J26	0:14	26.8
				14	J36	0:17	9.3	11	J31	0:20	0.3
								12	J34	0:23	17.6
								13	J35	0:14	0.1
								14	J36	0:16	9.5

As we can see from table 4.18, the number of junctions which are flooded is 9, 10 and 13 for a return period of 2, 5, and 10 years respectively and for remaining return periods (25, 50 and 100 years) 14 junctions are flooded with varying time and volume amount.

Among different paths of links path, J01-OUT1 and J26-OUT1 of 10 years return period is shown in the figure below in figure 4.11 and figure 4.12 respectively.

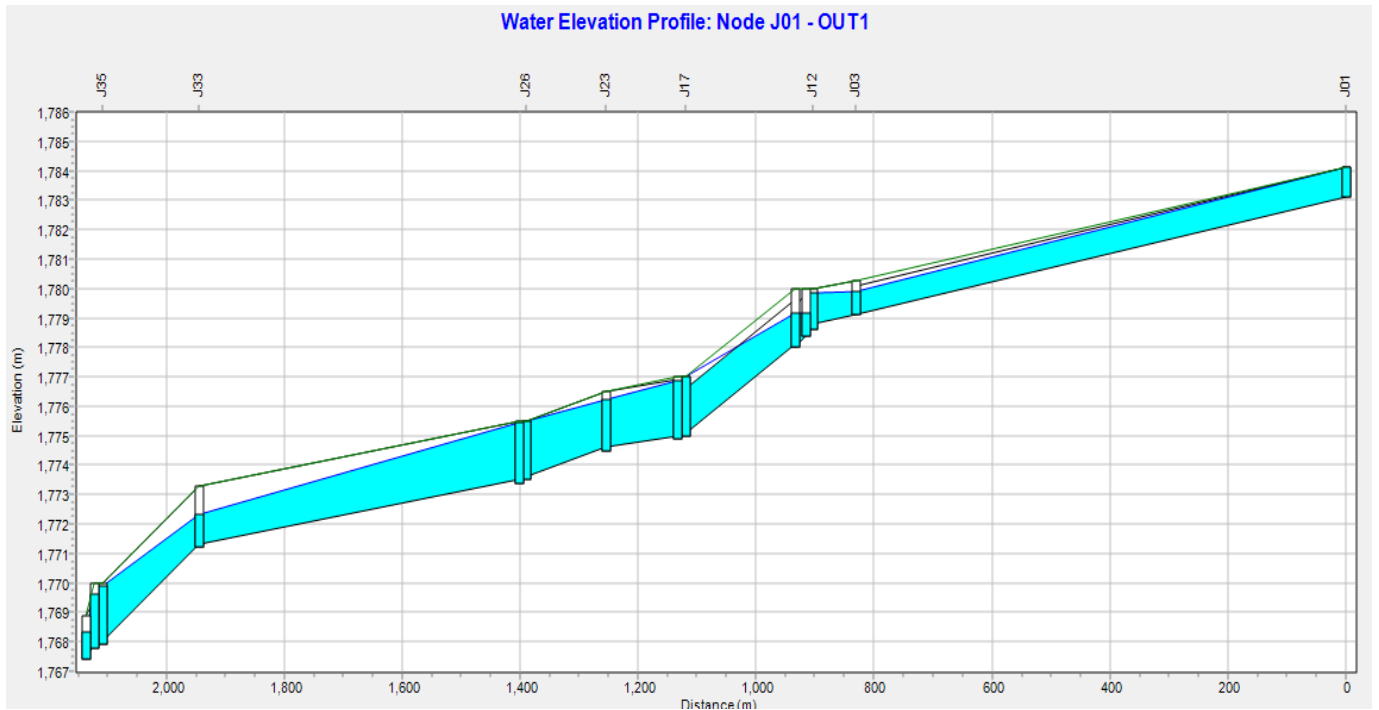


Figure 4.11: 10 years return period flooded junctions of path J01-OUT1

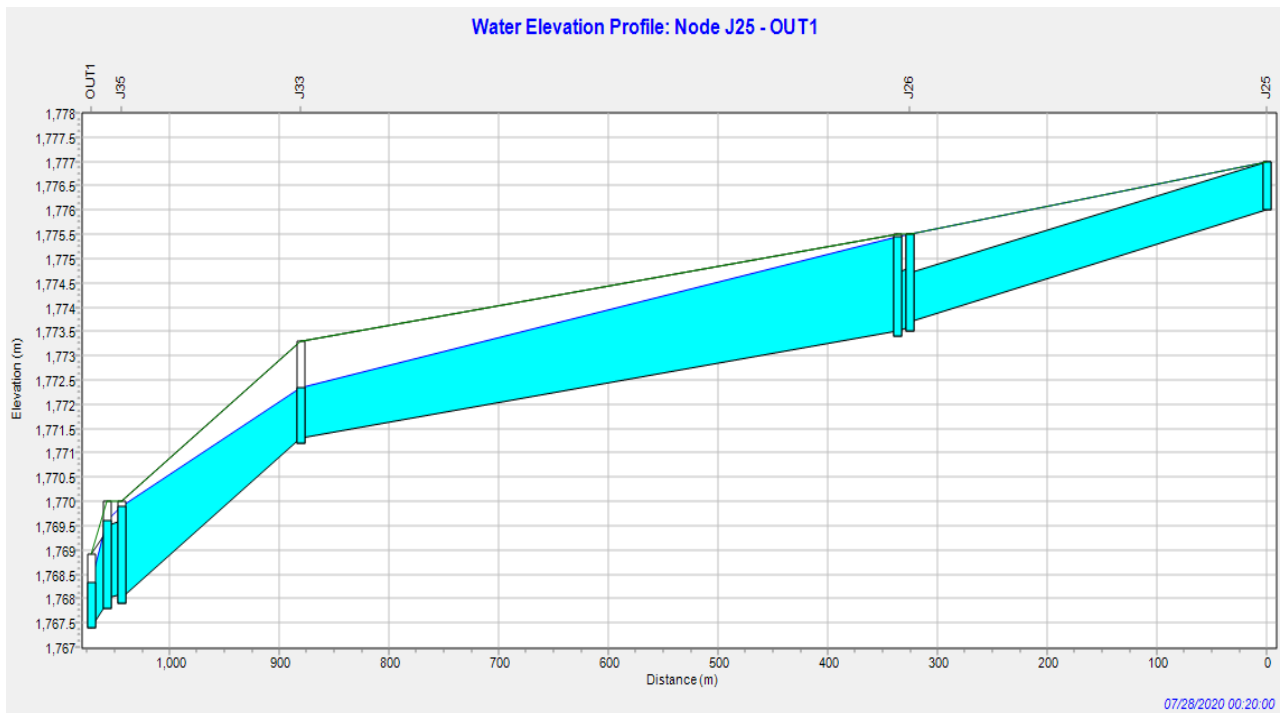


Figure 4.12: 10 years return period flooded junctions of path J26-OUT1

#### 4.2.7 Simulated and Estimated Peak Discharge Comparison

The peak discharge result of EPA SWMM 5.1 and rational/SCS method for a different return period of each sub-catchment shows nearly the same result which indicates that the sensitive parameters are adjusted in a well manner having correlation coefficient greater than 0.99 for return periods of 2, 5, 10, 25, 50 and 100 years as shown in the table below (table 4.19) and Appendix E (Figure E1).

Table 4.19: Peak discharge comparison for return period of 10, 25 and 50 years

SC Code	Generated Q <sub>10</sub> (CMS)		Generated Q <sub>25</sub> (CMS)		Generated Q <sub>50</sub> (CMS)	
	SCS/ Rational	SWMM	SCS/ Rational	SWMM	SCS/ Rational	SWMM
SC01	26.28	26.27	32.57	33.17	38.05	38.69
SC02	2.84	3.63	3.68	4.51	4.28	5.21
SC03	4.64	5.23	5.92	6.52	6.78	7.54
SC04	0.86	0.8	1.07	0.95	1.27	1.07
SC05	0.19	0.18	0.23	0.21	0.27	0.24
SC06	0.66	0.62	0.82	0.74	0.97	0.82
SC07	0.75	0.79	0.94	0.93	1.11	1.04
SC08	0.04	0.04	0.06	0.04	0.07	0.05
SC09	0.2	0.22	0.25	0.26	0.3	0.28
SC10	0.02	0.02	0.03	0.03	0.04	0.03
SC11	0.16	0.16	0.2	0.18	0.24	0.2
SC12	1.45	1.37	1.81	1.61	2.14	1.79
SC13	0.28	0.22	0.35	0.26	0.42	0.29
SC14	1.47	1.46	1.83	1.72	2.16	1.92
SC15	0.16	0.16	0.21	0.19	0.24	0.21
SC16	0.15	0.15	0.19	0.17	0.23	0.19
SC17	0.73	0.72	0.91	0.84	1.08	0.93
SC18	1.1	1.1	1.37	1.29	1.63	1.43
SC19	0.61	0.57	0.76	0.66	0.9	0.73
SC20	0.06	0.06	0.08	0.06	0.1	0.07
SC21	0.09	0.08	0.11	0.09	0.13	0.1
SC22	1.58	1.57	1.97	1.83	2.34	2.04
SC23	0.1	0.1	0.13	0.11	0.15	0.12
SC24	0.13	0.12	0.16	0.14	0.19	0.15
SC25	0.61	0.58	0.76	0.68	0.91	0.76
SC26	1.98	1.94	2.46	2.31	2.92	2.6
SC27	0.15	0.13	0.19	0.15	0.23	0.16
SC28	0.21	0.18	0.26	0.21	0.3	0.23
SC29	1.03	1.07	1.28	1.26	1.52	1.41
SC30	0.11	0.12	0.13	0.14	0.16	0.15

#### 4.2.8 Adequacy of Existing Drainage System

Here there are canals which carry the runoff from more than one sub-catchment. The canals conveying runoff from single sub-catchment (according to the sub-catchment classification that I made) are; L01R, L02R, L04R, L05L, L06L, L07L, L07R, L08R, L11R, L12R, L12L, L11L, L14L, L16L, L16R, L17L, L17R, L19R, L20L, L23L and L23R. The remaining canals carry the runoff which comes from the upper sub-catchments and for comparison purposes, their values are added together and compared whether they are capable enough to convey the coming runoff or not.

The canals hydraulic capacity is calculated by using Manning's equation and compared with the SWMM model output and SCS/Rational method of estimated peak runoff of the given sub-catchment contributing to their respective coded canals. The comparison for the return period of 10 and 25 is done for SWMM and SCS/Rational methods with hydraulic capacity results done by Manning's method as shown in table 4.20.

Table 4.20: Comparison of Manning's Peak discharge with output of SWMM and Rational/SCS Method

Conduit code	Contributing Sub-catchment/ junction	Q10 (CMS)		Q25 (CMS)		Manning's	Remark
		SCS/Rational	SWMM	SCS/Rational	SWMM		
L01R	J01	26.28	26.27	32.57	33.17	2.15	Inadequate
L02R	J02	0.66	0.62	0.82	0.74	2.13	adequate
L03L	J03	27.13	27.07	33.62	34.12	3.96	Inadequate
L04R	J10	0.04	0.04	0.06	0.04	4.01	adequate
L05L	J09	0.75	0.79	0.94	0.93	1.04	adequate
L06R	J11	3.64	4.46	4.68	5.48	2.74	Inadequate
L06L	J13	0.20	0.22	0.25	0.26	1.91	adequate
L07L	J05	4.64	5.23	5.92	6.52	1.54	Inadequate
L07R	J07	0.16	0.16	0.20	0.18	1.49	adequate
L08R	J04	0.86	0.80	1.07	0.95	1.75	adequate
C01	J06	5.50	6.03	6.99	7.47	3.92	Inadequate
C02	J08	10.14	11.26	12.91	13.99	5.26	Inadequate
C03	J12	30.76	31.53	38.30	39.60	9.80	Inadequate
L09L	J14	30.99	31.77	38.58	39.89	9.18	Inadequate
L10L	J15	41.41	43.25	51.84	54.14	13.41	Inadequate
L11R	J18	1.47	1.46	1.83	1.72	2.31	adequate
C04	J17	44.33	46.08	55.48	57.47	10.58	Inadequate
L12L	J16	1.45	1.37	1.81	1.61	2.24	Inadequate
L12R	J20	0.14	0.14	0.14	0.14	1.94	Inadequate
L11L	J19	0.16	0.16	0.21	0.19	2.28	adequate
L13L	J17	44.70	46.44	55.91	57.86	9.44	Inadequate

L14L	J22	0.61	0.57	0.76	0.66	2.18	adequate
L15L	J23	45.40	47.09	56.78	58.61	14.08	Inadequate
L16L	J24	1.10	1.10	1.37	1.29	1.17	adequate
L16R	J27	0.13	0.12	0.16	0.14	1.17	Inadequate
C05	J26	48.08	49.76	60.12	61.73	14.53	adequate
L17R	J28	1.58	1.57	1.97	1.83	1.59	adequate
L17L	J28	0.10	0.10	0.13	0.11	1.53	adequate
L18L	J29	50.28	51.92	62.87	64.29	14.48	Inadequate
L19R	J31	0.61	0.58	0.76	0.68	0.50	Inadequate
L20L	J30	0.73	0.72	0.91	0.84	6.68	adequate
L21L	J32	1.43	1.38	1.78	1.61	6.55	adequate
L22L	J33	51.92	53.48	64.91	66.11	23.92	Inadequate
L23R	J34	1.03	1.07	1.28	1.26	3.03	adequate
C06	J35	52.95	54.55	66.19	67.37	17.31	Inadequate
L23L	J36	0.11	0.12	0.13	0.14	0.81	adequate
L24	J37	53.06	54.67	66.32	67.51	19.43	Inadequate

As shown in the table 4.20, it is clear that there are drainage lines which are incapable to convey the runoff generated from both the town and rural area (out of the town). SC01 contributes the majority of flood as it emanated from ReKame hill and crosses about 10km of rural agricultural land and gets the inlet around Zonal Hospital of Alaba Kulito. Next to SC01; SC03 and SC02 hold successive ranks in which most of their runoff is also generated from the rural region near to suburban areas of North of Alaba Kulito Town.

### **4.3 Flood causes of Alaba Kulito Town**

#### **4.3.1 Incapable Drainage Structure to Convey the coming Runoff from Rural Area**

The hydrologic and hydraulic analysis of the study area results all main and some sub-main canals of the drainage network are not able to convey the peak discharge which can be produced possibly. This is due to the incapability of the size of links which are loaded to convey much runoff from rural area crossing both Lenda Ber (upper part) and Mahal Arada (lower part) through seriously connected drainage networks. The drainage lines which are not capable enough to convey even 2 year return period peak discharge are lines out of LO2R, L04R, L05L, L06L, L07R, L08R, L11R, L11L, L14L, L16L, L17R, L17L, L20L, L21L, L23R and L23L.

#### **4.3.2 Neglecting collector and feeder Drainage Structures**

In addition to the insufficiency of the existing drainage canals, the drainage service of the town is not well connected which means by, collector drainage services are neglected to allow the overcoming flood to pass arbitrarily through the town. This impedes the comfort of dwellers: by

inundating in depressions, by entering the individual blocks, by creating pondages which is safe for production of insects causing malaria and related diseases, hindering harmonious movement after heavy rain (in the town and upper catchment of Rekame hill) and related effects on both Lenda Ber and Mahal Arada kebele dwellers. The access roads are also acting as canals with washing the upper layer of soil (erosion) and aggravating flood occurrence.



Figure 4.13: Effect of neglecting collector Drainage Systems around Lenda Ber(left side) and Mahal Arada (right side)

#### **4.3.3 Blockage of drainages by dry waste materials and maintenance problem**

From field visit, some drainage lines of open and closed canals are field by dry wastes of households, shops and street debris materials which are able to reduce the capability of canals conveyance system by the amount of volume they occupied. Some canals are damaged which lets the chance of runoff to deviate from the conveyance line to the street and then to houses.



Figure 4.14: Blocked closed canal (right side), Blocked open canal (left side) and Damaged canal (middle)

The other problem observed along closed canals of main streets is insufficient opening and closure of manhole openings by mud and garbage materials which prevents the entrance of runoff from the street into drainage canal through provided manhole openings.



Figure 4.15: Manhole opening closed by garbage and less depth of opening

#### **4.4 Remedial Measures to be taken**

##### **4.4.1 Diverting the Upper Catchment Runoff to Bilate River**

###### **(a) Diverted and considered Catchment**

The sub-catchments which are reduced to diversion work are SC01 and SC03 while others are not affected due to diversion. Here both values of area and percentage coverage are changed. The area of SC01 and SC03 are reduced from 1305.24ha and 202.57ha to 272.309ha and 197.653ha respectively with total area reduced from 1717.46ha to 673.306ha that is the runoff of about 1044ha land can be reduced from flowing to town.

By doing this, runoff magnitude of SC01 and SC03 of 10 years return period can be reduced from 26.28 m<sup>3</sup>/s to 6.51 and 5.23 to 4.64 respectively which is 20.35 m<sup>3</sup>/s of total peak discharge load is diverted to Bilate River at the North-West of Alaba Kulito Town as shown in the figure below (figure 4.17).

The total load of sub-catchments which should be conveyed through corresponding conduits for both cases (without diversion work and with diversion work) enumerated in appendix F (table F1 and table F2).

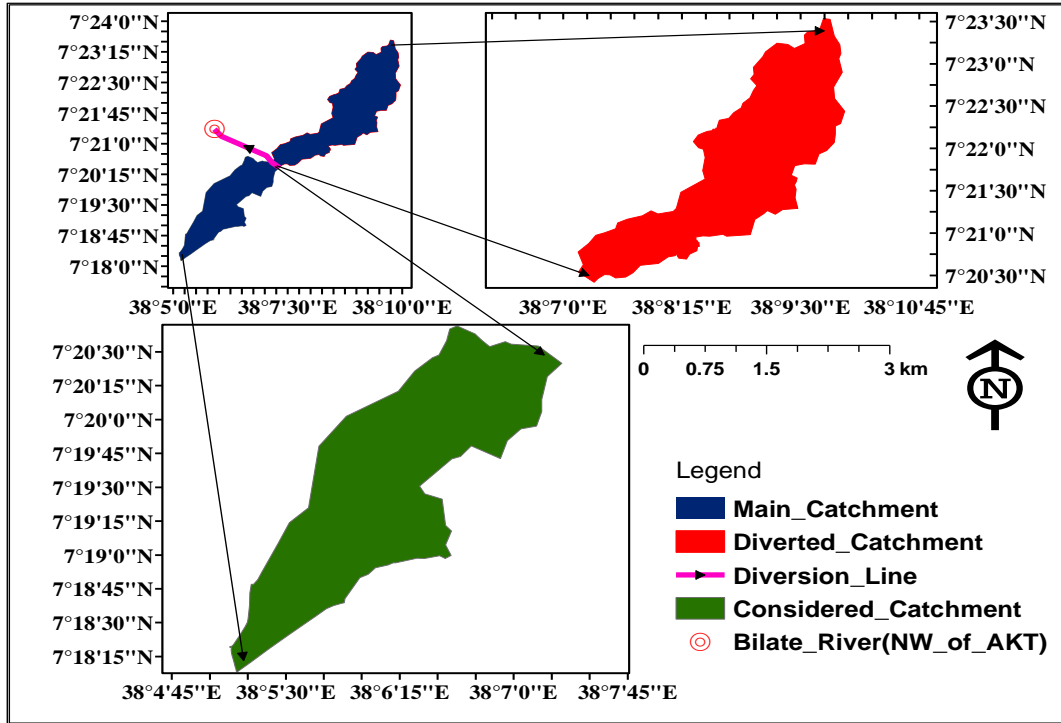


Figure 4.16: Diverted and considered Catchment of study area

The total difference which can be obtained by diverting the upper catchment is illustrated in the table below (Table 4.21).

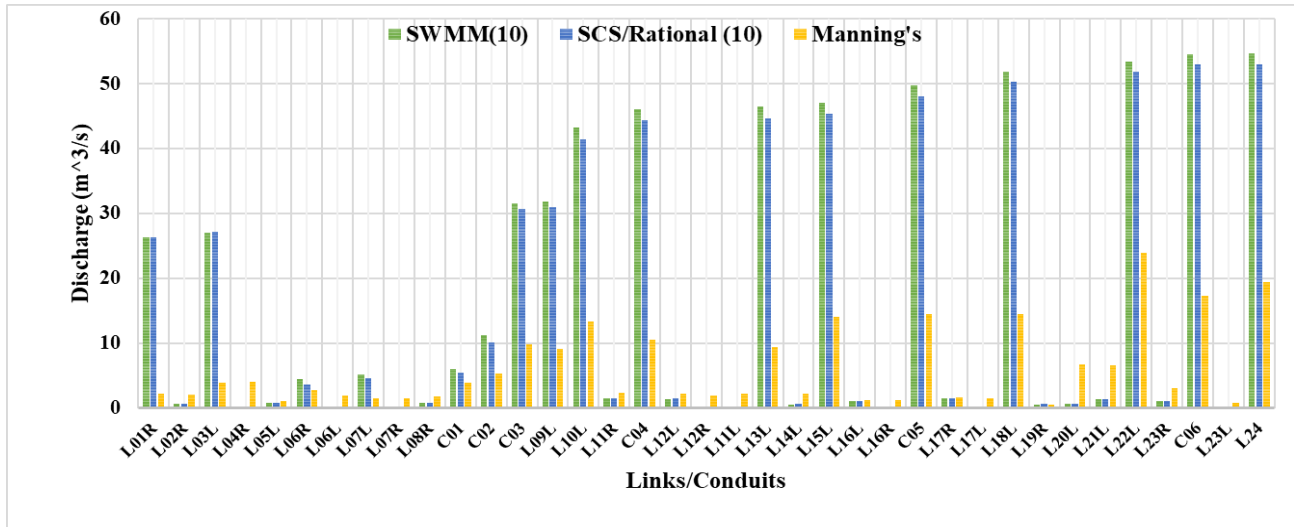
Table 4.21: Change of Peak discharge due to Diversion

Return period (yr)		2	5	10	25	50	100
SC01	$Q_o$ ( $m^3/s$ )	15.42	21.37	26.27	33.17	38.69	44.36
	$Q_r$ ( $m^3/s$ )	3.84	5.32	6.51	8.15	9.45	10.75
	$Q_d$ ( $m^3/s$ )	11.58	16.05	19.76	25.02	29.24	33.61
Ssc03	$Q_o$ ( $m^3/s$ )	3.18	4.31	5.23	6.52	7.54	8.58
	$Q_r$ ( $m^3/s$ )	2.8	3.82	4.64	5.77	6.66	7.57
	$Q_d$ ( $m^3/s$ )	0.38	0.49	0.59	0.75	0.88	1.01
Total $Q_d$		11.96	16.54	20.35	25.77	30.12	34.62
Diverted Discharge (%)		64.30	64.41	64.60	64.93	65.15	65.39

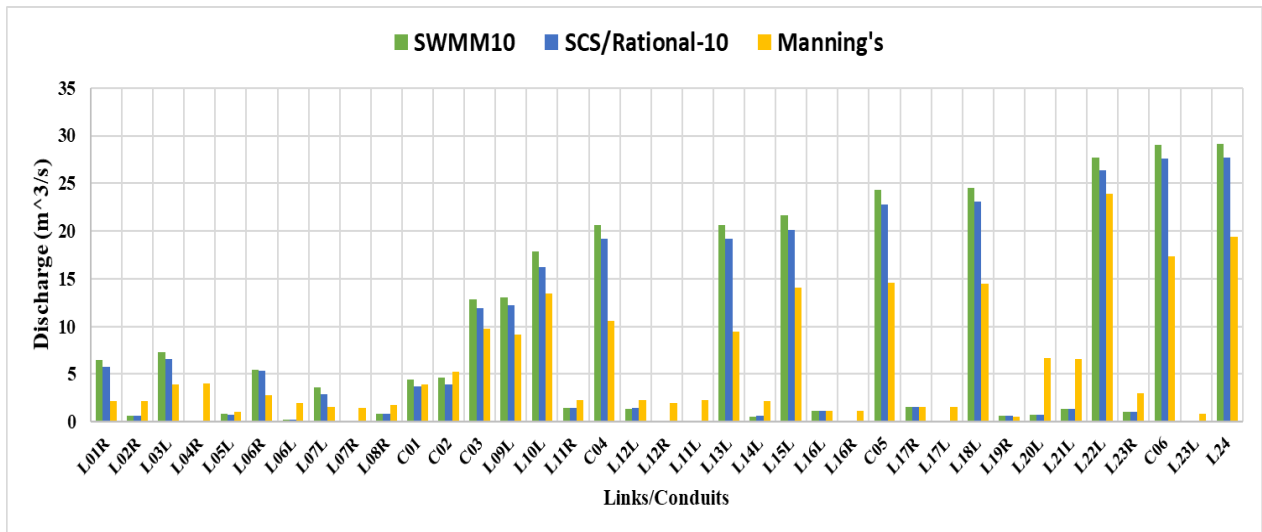
Where:-  $Q_o$ ,  $Q_r$  &  $Q_d$  are peak discharge of original, remaining and diverted respectively

Without diverting the upper catchment runoff, the existing canal capacity and peak discharge values to be conveyed deviates by much amount. As we have seen from table 4.21, diversion work shares more than 64% of runoff load which indicates that it is a sounded solution.

The following figure shows a clear change of runoff load on links due to proposed diversion work. The load difference for all conduits, for all methods (SWMM, SCS/Rational methods and Manning's method) and return periods (2, 5, 10, 25, 50 and 100 years) is done in Appendix F (table F1 and table F2). Figure 4.18 is plotted by using 10 years return period of runoff load done by SWMM and SCS/Rational method of discharge analysis comparing with the hydraulic capacity of existing drainage conduits.



a) Without diversion work



b) With diversion work is done

Figure 4.17: Discharge result comparison (a)without diversion work (b) with diversion

The amount of peak discharge (total  $Q_d$  of table 4.21) are used and canal dimensions are calculated by using the maximum permissible velocity method of erodible canals at which the design canal can function with compromised velocity without exposing to erosion.

➤ **Erodible/earthen trapezoidal channel design**

The proposed canal dimension based of each corresponding return period's peak discharge and canal dimension is shown in the table below (Table 4.22). As illustrated in section 3.2.4.1, the following table is developed by a series of steps and by using Appendix B; table B5, table B6 and table B7 for n, m and v respectively.

Table 4.22: Possible canal dimension of different return period for the diversion canal

$Q_T$	Q (m <sup>3</sup> /s)	Bed slope (m/m)	n	Side slope (H:V)	V (m/s)	R	A	W	Flow depth y(m)	bed width b(m)	free board y'(m)	Total depth Y(m)
Q <sub>2</sub>	11.960	0.01036	0.04	2 : 1	2.00	0.70	5.98	4.98	0.96	4.27	0.19	1.16
Q <sub>5</sub>	16.540	0.01036	0.04	2 : 1	2.00	0.70	8.27	6.89	0.85	8.09	0.17	1.01
Q <sub>10</sub>	20.350	0.01036	0.04	2 : 1	2.00	0.70	10.18	8.48	0.81	10.99	0.16	0.97
<b>Q<sub>25</sub></b>	<b>25.770</b>	<b>0.01036</b>	<b>0.04</b>	<b>2 : 1</b>	<b>2.00</b>	<b>0.70</b>	<b>12.89</b>	<b>10.74</b>	<b>0.78</b>	<b>15.02</b>	<b>0.16</b>	<b>0.93</b>
Q <sub>50</sub>	30.120	0.01036	0.04	2 : 1	2.00	0.70	15.06	12.55	0.76	18.20	0.15	0.92
Q <sub>100</sub>	34.620	0.01036	0.04	2 : 1	2.00	0.70	17.31	14.42	0.75	21.48	0.15	0.90

As the town found below the diversion canal is a permanent town and the flood occurrences aggravate for the future than the current time, it is logical to design for 25 years return Period of the design flood. The canal dimension increases with the considered return period and the layout of 25 years of design discharge is shown as the figure below (figure 4.18).

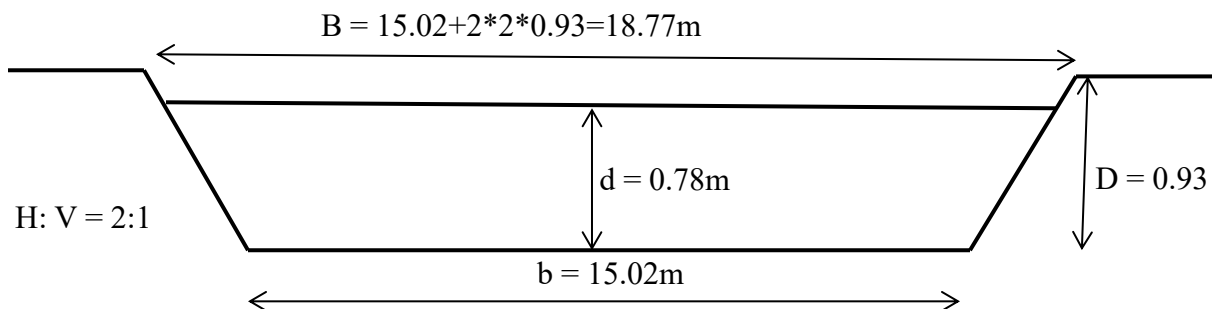


Figure 4.18: Layout of trapezoidal canal of Diversion canal

#### **4.4.2 Strengthening Soil Conservation and Afforestation Work at the upper catchment**

As runoff is the combined effect of the response of land-use /land-cover type, soil type, terrain or slope and the precipitation condition; there is the opportunity to change land cover of the land which is not irrigable and still it is possible to do soil conservation work on both cultivated and uncultivated land throughout the catchment by sustaining the started green legacy program in a strong manner. By doing so, the curve number and/or runoff coefficient magnitude can be reduced which is directly linked to runoff generation.

#### **4.4.3 Constructing Additional Drainage Canals**

As collector canals are the canals which collect and dispose the runoff to the main drainage canal, it is reasonable to provide such canals through the town. The upper Lenda Ber kebele (above Aymale Adebabay) needs a number of well-connected collector drainage canals to keep the town and dwellers from the adverse effects mentioned above. Similarly, both wings of the zonal hospital of Alaba Kulito town need adequate feeder drainage canal which can share the load from link coded as L01. Likewise, the Eastern part of Mahal Arada (including sides of the main road of Shashemene-Alaba Kulito-Sodo way) needs collector canals and main canals which can convey runoff from SC02 and roadsides drainage to Bilate River. By doing these the town can be kept from stormwater retention and environmental pollution with keeping an aesthetic view of the developing zonal city (ALaba Kulito Town).

#### **4.4.4 Adapting appropriate dry waste material removal**

As dry waste does have its own volume and occupies a place that is to be occupied by runoff; the presence of any dry waste in the drainage canal decreases the capability to convey the coming discharge. The existence of dry waste in the drainage canal can block the opening of the conveyance system. In addition to increasing the runoff volume.

Illegal dumping of solid wastes like plastics, chat remains, and other waste of households should not be disposed into stormwater drainage lines instead it should be collected separately and disposed into the prepared waste disposing place. Therefore, awareness should be created regarding to waste disposing and environmental protection, there must be a collaborated approach of appropriate dry waste removing mechanism among the dwellers and government body and individuals must be responsible for the aesthetic view of the environment that they are living.

In fact, Alaba Kulito Town administration do have appreciable dry waste collecting and removal trend. It do have a cart and manpower which is intended to clean the drainage canal. However, the town administration only cannot keep the whole drainage lines clean always. It should be an assignment of each dweller and town administration together with a cooperative manner.

#### 4.4.5 Connecting Outfall with Bilate River by Lined Trapezoidal canal

The high amount of runoff generated from the upper catchment is allowed to pass through the outfall located near to ST Gabriel Church. The peak discharge which should be conveyed through outfall is the summation of each sub-catchments i. e.  $Q_2 = 18.88\text{m}^3/\text{s}$ ,  $Q_5 = 24.57\text{ m}^3/\text{s}$ ,  $Q_{10} = 29.10\text{ m}^3/\text{s}$ ,  $Q_{25} = 35.25\text{ m}^3/\text{s}$ ,  $Q_{50} = 40.06\text{ m}^3/\text{s}$  and  $Q_{100} = 44.88\text{m}^3/\text{s}$ .

There is a naturally existing gully which is being eroded due to high flood coming from the upper catchment and eroding the bed and banks of the formed gully. It is observe that there are buildings and residents on sides of the gully which needs to be protected. Therefore, outfall starting from ST Gabriel Church up to Bilate River should be changed from natural earthen canal to manmade masonry trapezoidal canal which can save the assets and life on both sides of canal, the soil which is being eroded can be conserved and the stormwater can be removed into the natural river (Bilate River) in a safe manner.

For this case (non-erodible trapezoidal canal); Manning’s equation is used and to calculate the dimension of section factors ( $AR^{2/3}$ ) containing bed width (b in m), flow depth (d in m), side slope (z in m/m), wetted perimeter (p in m), freeboard which is 20 percent of flow depth (d’ in m), total depth (D in m) and top width (B in m) are done by using constants (manning’s roughness coefficient (n) and side slope) and longitudinal slope (S in m/m) from elevation difference and canal length with fixing bed width and solving for flow depth by iteration until the cross parameters agree to each other and able to convey the design discharge.

Table 4.23: Discharge of corresponding return periods of canal dimension of common outlet

b	d	z	A	P	R	$R^{2/3}$	n	S	V	Q	Comment	d'	D	B
<b><math>Q_2 = 18.88\text{m}^3/\text{s}</math></b>														
1	2	1	6	3.83	1.57	1.35	0.03	0.0141	5.35	32.08	too much	0.40	2.40	5.80
1	1	1	2	2.41	0.83	0.88	0.03	0.0141	3.49	6.99	too small	0.20	1.20	3.40
1	1.5	1	3.75	3.12	1.20	1.13	0.03	0.0141	4.48	16.79	slightly small	0.30	1.80	4.60
1	1.58	1	4.08	3.23	1.26	1.17	0.03	0.0141	4.62	18.84	O.K.	0.32	1.90	4.79
<b><math>Q_5 = 24.57\text{m}^3/\text{s}</math></b>														
1.2	2	1	6.40	4.03	1.59	1.36	0.03	0.0141	5.39	34.53	too much	0.40	2.40	6.00

1.2	1	1	2.20	2.61	0.84	0.89	0.03	0.0141	3.53	7.77	too small	0.20	1.20	3.60
1.2	1.7	1	4.93	3.60	1.37	1.23	0.03	0.0141	4.88	24.07	slightly Small	0.34	2.04	5.28
1.2	1.72	1	5.00	3.63	1.38	1.24	0.03	0.0141	4.91	24.57	O.K.	0.34	2.06	5.32
<b>Q<sub>10</sub> = 29.10m<sup>3</sup>/s</b>														
1.5	2	1	7.00	4.33	1.62	1.38	0.03	0.0141	5.46	38.21	too much	0.40	2.40	6.30
1.5	1.8	1	5.94	4.05	1.47	1.29	0.03	0.0141	5.12	30.40	slightly big	0.36	2.16	5.82
1.5	1.75	1	5.69	3.97	1.43	1.27	0.03	0.0141	5.03	28.61	slightly small	0.35	2.10	5.70
1.5	1.77	1	5.76	4.00	1.44	1.28	0.03	0.0141	5.06	29.14	O.K.	0.35	2.12	5.74
<b>Q<sub>25</sub> = 35.25m<sup>3</sup>/s</b>														
<b>2</b>	<b>2</b>	<b>0.5</b>	<b>6.00</b>	<b>3.12</b>	<b>1.92</b>	<b>1.55</b>	<b>0.03</b>	<b>0.0141</b>	<b>6.13</b>	<b>36.78</b>	<b>slightly much</b>	<b>0.40</b>	<b>2.40</b>	<b>4.40</b>
<b>2</b>	<b>1.8</b>	<b>0.5</b>	<b>5.22</b>	<b>3.01</b>	<b>1.74</b>	<b>1.44</b>	<b>0.03</b>	<b>0.0141</b>	<b>5.72</b>	<b>29.88</b>	<b>too small</b>	<b>0.36</b>	<b>2.16</b>	<b>4.16</b>
<b>2</b>	<b>1.9</b>	<b>0.5</b>	<b>5.61</b>	<b>3.06</b>	<b>1.83</b>	<b>1.50</b>	<b>0.03</b>	<b>0.0141</b>	<b>5.93</b>	<b>33.23</b>	<b>slightly small</b>	<b>0.38</b>	<b>2.28</b>	<b>4.28</b>
<b>2</b>	<b>1.96</b>	<b>0.5</b>	<b>5.83</b>	<b>3.09</b>	<b>1.88</b>	<b>1.53</b>	<b>0.03</b>	<b>0.0141</b>	<b>6.05</b>	<b>35.26</b>	<b>O.K.</b>	<b>0.39</b>	<b>2.35</b>	<b>4.35</b>
<b>Q<sub>50</sub> = 40.06m<sup>3</sup>/s</b>														
2.3	2.3	0.5	7.94	3.59	2.21	1.70	0.03	0.0141	6.73	53.39	too much	0.46	2.76	5.06
2.3	2.1	0.5	7.04	3.47	2.03	1.60	0.03	0.0141	6.34	44.62	slightly big	0.42	2.52	4.82
2.3	1.9	0.5	6.18	3.36	1.84	1.50	0.03	0.0141	5.94	36.69	slightly small	0.38	2.28	4.58
2.3	2	0.5	6.60	3.42	1.93	1.55	0.03	0.0141	6.14	40.55	O.K.	0.40	2.40	4.70
<b>Q<sub>100</sub> = 44.88m<sup>3</sup>/s</b>														
2.5	2.5	0.5	9.38	3.90	2.41	1.80	0.03	0.0141	7.11	66.68	too big	0.50	3.00	5.50
2.5	2	0.5	7.00	3.62	1.93	1.55	0.03	0.0141	6.15	43.06	slightly small	0.40	2.40	4.90
2.5	2.1	0.5	7.46	3.67	2.03	1.60	0.03	0.0141	6.35	47.34	slightly big	0.42	2.52	5.02
2.5	2.04	0.5	7.18	3.64	1.97	1.57	0.03	0.0141	6.23	44.75	O.K.	0.41	2.45	4.95

Since it is likely the same with the diversion canal, it shall be designed for 25 years return period and is illustrated in figure form as shown below (figure 4.19).

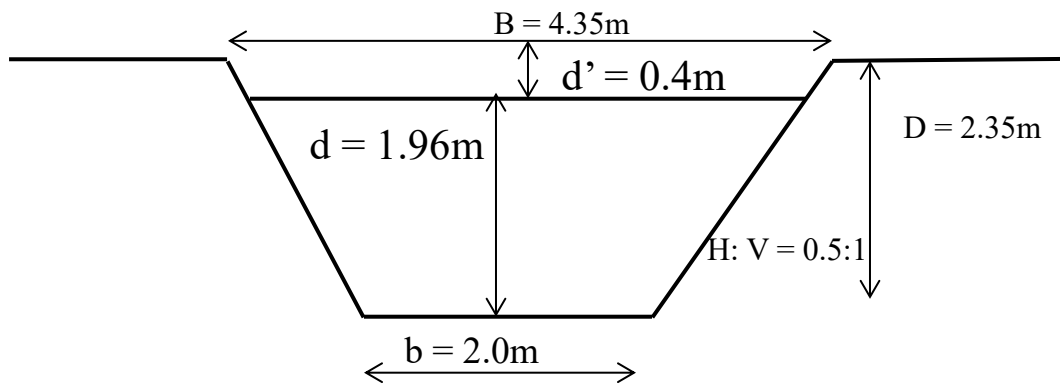


Figure 4.19: Layout of trapezoidal canal of common outlet canal

## **5 SUMMARY AND CONCLUSION**

### **5.1 Summary**

Regarding the thesis entitled as “Assessment of flood hazard and drainage system by SWMM in case of Alaba Kulito Town” the overall work can be summarized as:

- To assess flood hazard and drainage systems of Alaba Kulito Town both primary and secondary data were collected from institutions, fields and concerning bodies. The study was focused on Lenda Ber kebele and Mahal Arada kebeles which are the most flood-prone kebeles of Alaba Kulito Town.
- The 33 years daily rainfall data which was obtained from NMA of Ethiopian was the main input of the hydrological analysis model. But, the recorded data was incomplete and the missed rainfall data of the representative station (Alaba Kulito) was filled by NRM by using 33 years data of nearby stations (Hawassa, Durame, and Hosana meteorology stations).
- POT Extreme value determination method was preferred than BM method as POT has been found to yield better results in many studies for both extreme events of flood and drought. However, it was more time consuming than BM.
- GPD probability distribution method was selected based on the linearity of MRL plot which follows POT extreme value determination and Easyfit 5.6 software contained 51 daily maximum rainfall data samples.
- The IDF curve was developed by GPD method and compared with representative IDF of ERA for the rainfall region of B2 (developed by considering Arba Minch, Sodo and Hawassa meteorology stations).
- The adequacy of the existing drainage lines has been checked by measuring canal dimensions and analyzed by Manning’s equation and the result is compared with hydrologic analysis of peak discharge determination methods (SCS method and rational method) and SWMM simulation result.

### **5.2 Conclusion**

The general work of this thesis is concluded as:

- ❖ IDF was developed by GPD (in this paper) which can be used for Hydrologic analysis and any design of hydraulic structure around Alaba Kulito by water resource professionals and concerned institutions. Because of IDF curve of GPD best represents the study area as it is

developed by using long years RF data which is updated and uses the rainfall data of Alaba station and nearby stations, and that of IDF developed by ERA for Alaba Kulito is not updated and it is done by stations of Sodo, Arbaminch and Hawassa which are distant stations from Alaba Kulito, IDF done by GPD was used in this paper.

- ❖ The flood prediction made for all return periods of 2, 5, 10, 25, 50 and 100 years showed that most of the canals which carry runoff collected from upper catchments (more than one sub-catchment) and drainage lines which received runoff from wide sub-catchment coming from the rural area were totally incapable to convey the peak discharge.
- ❖ It has been found that insufficiency of existing canals size, canal length and canal number with inappropriate waste disposing system all together allows the generated runoff from a rural area (emanating from Rekame Hill) causes flood at lower catchment of Alaba Kulito Town during high intensity of rainfall with favorable antecedent moisture condition of the catchment.
- ❖ Runoff which is generating from the rural areas coming from Rekame Hill (North-West of Alaba Kulito Town) and passes through Alaba Kulito Town and then to Bilate River accounts for more than 64% of total runoff. Therefore it shall be diverted to Bilate river before it reaches the town at about 3km from Alaba Zonal Hospital.
- ❖ Adequate and enough drainage canals which receives runoff from the upper catchment and convey to outfall should be added at Lenda Ber kebele (at both wings of the existing canal from Hospital to Aymale Adebabay) and Eastward of Mahal Arada along the main road of Bus station to Sodo way and should be well connected to dispose of runoff in a safe manner. The existing common outlet (from ST. Gabriel to Bilate River) should be lined canal instead of natural waterway which is going to be gully due to high amount of runoff from the upper catchment which is allowed to pass through it.
- ✓ Finally, I recommend the Alaba Zone Municipality to construct drainage structures by using the currently developed IDF curve and should have to store data like dimensions of constructed canals, images and videos of flood events and hazard level of the flood to do a flood risk assessment. It is necessary to invest money, time, labor, knowledge and share the experience of dwellers, government body and non-governmental organizations altogether to keep the town from threat and to sustain life and property.

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## 7 APPENDICES

### APPENDIX A: ANALYSIS OF PRECIPITATION DATA

Table A1: Annual precipitation and Cumulative annual precipitation in mm for double mass curve

Year	Annual precipitation of stations					Cumulative annual precipitation of stations				
	A. Kulito	Awassa	Durame	Hosana	Mean	A. Kulito	Awassa	Durame	Hosana	Mean
1987	1550.3	958.7	1330.6	1280.5	1280.0	1550.3	958.7	1330.6	1280.5	1280.0
1988	1498.0	957.0	1255.1	1216.2	1231.6	3048.3	1915.7	2585.7	2496.7	2511.6
1989	1083.5	1082.0	1580.8	1199.7	1236.5	4131.8	2997.7	4166.5	3696.4	3748.1
1990	968.0	756.7	1047.9	1058.9	957.9	5099.8	3754.4	5214.4	4755.3	4706.0
1991	975.1	846.8	1277.3	1079.1	1044.6	6074.9	4601.2	6491.7	5834.4	5750.6
1992	1162.6	962.3	1326.7	1387.6	1209.8	7237.5	5563.5	7818.4	7222.0	6960.4
1993	1272.2	928.4	1117.2	1413.5	1182.8	8509.7	6491.9	8935.6	8635.5	8143.2
1994	806.1	861.5	812.8	920.4	850.2	9315.8	7353.4	9748.4	9555.9	8993.4
1995	975.6	1004.4	935.9	1160.7	1019.2	10291.4	8357.8	10684.3	10716.6	10012.5
1996	1160.5	1189.1	949.7	1168.2	1116.9	11451.9	9546.9	11634.0	11884.8	11129.4
1997	1193.5	1054.1	1190.4	1442.5	1220.1	12645.4	10601.0	12824.4	13327.3	12349.5
1998	1252.7	1148.3	1378.6	1556.4	1334.0	13898.1	11749.3	14203.0	14883.7	13683.5
1999	763.4	808.9	1066.0	1011.3	912.4	14661.5	12558.2	15269.0	15895.0	14595.9
2000	876.0	821.5	1162.2	991.9	962.9	15537.5	13379.7	16431.2	16886.9	15558.8
2001	944.2	1021.7	1323.7	1145.5	1108.8	16481.7	14401.4	17754.9	18032.4	16667.6
2002	796.0	919.3	1140.4	1346.4	1050.5	17277.7	15320.7	18895.4	19378.8	17718.1
2003	943.3	888.9	1053.5	1140.2	1006.5	18221.0	16209.6	19948.9	20519.0	18724.6
2004	992.6	897.7	1064.7	1185.1	1035.0	19213.6	17107.3	21013.6	21704.1	19759.6
2005	866.5	1002.6	1460.9	1179.0	1127.3	20080.1	18109.9	22474.5	22883.1	20886.9
2006	873.8	1197.9	939.8	1201.7	1053.3	20953.9	19307.8	23414.3	24084.8	21940.2
2007	1081.6	1152.7	1033.1	1098.8	1091.6	22035.5	20460.5	24447.4	25183.6	23031.7
2008	950.3	915.0	1003.8	1202.8	1018.0	22985.8	21375.5	25451.2	26386.4	24049.7
2009	845.8	703.7	853.8	1247.2	912.6	23831.6	22079.2	26305.0	27633.6	24962.3
2010	1800.4	1032.3	1024.7	1121.5	1244.7	25632.0	23111.5	27329.7	28755.1	26207.1
2011	956.0	922.9	1201.2	1124.5	1051.1	26588.0	24034.4	28530.9	29879.6	27258.2
2012	752.3	785.4	758.9	981.6	819.6	27340.3	24819.8	29289.8	30861.2	28077.8
2013	2017.6	1054.1	1373.7	1145.1	1397.6	29357.9	25873.9	30663.4	32006.3	29475.4
2014	1212.7	1154.6	1564.8	1541.5	1368.4	30570.6	27028.5	32228.2	33547.8	30843.8
2015	644.6	778.4	815.7	1070.0	827.2	31215.1	27806.9	33043.9	34617.8	31670.9
2016	1275.7	1001.9	1656.6	1133.1	1266.8	32490.8	28808.8	34700.5	35750.9	32937.7
2017	802.1	1210.4	1206.9	952.0	1042.9	33292.8	30019.2	35907.4	36702.9	33980.6
2018	1000.5	1180.6	1703.2	1454.8	1334.8	34293.3	31199.8	37610.6	38157.7	35315.4
2019	1180.0	1076.2	1932.9	1543.4	1433.1	35473.3	32276.0	39543.5	39701.1	36748.5

Table A2: Annual maximum daily precipitation for POT method

Annual maximum daily precipitation for POT method												
Rejected												
Number	1	2	3	4	5	6	7	8	9	10	11	12
P(mm)	38	38.2	38.3	38.4	38.5	39.4	39.5	39.5	39.5	39.5	39.5	39.6
Number	13	14	15	16	17	18	19	20	21	22	23	24
P(mm)	40	40	40	40	40.2	40.3	40.7	40.9	41	41	41.4	41.4
Number	25	26	27	28	29	30	31	32	33	34	35	36
P(mm)	41.7	42	42	42	42	42.2	42.4	42.4	42.8	43.2	43.5	43.5
Number	37	38	39	40	41	42	43	44	45	46	47	48
P(mm)	44.1	44.2	44.4	44.5	45.4	45.7	46.3	46.7	46.8	47.6	47.6	48.2
Accepted												
Number	49	50	51	52	53	54	55	56	57	58	59	60
P(mm)	48.4	48.8	49	49	49	49.3	49.3	50	50	50	50	50
Number	61	62	63	64	65	66	67	68	69	70	71	72
P(mm)	51	51	51.2	51.3	51.4	52	52.4	52.4	53.8	54	54.8	55
Number	73	74	75	76	77	78	79	80	81	82	83	84
P(mm)	55.3	55.4	55.7	55.7	56.4	56.6	56.7	57	59	59	60	60.5
Number	85	86	87	88	89	90	91	92	93	94	95	96
P(mm)	62	64	65.3	66.5	66.8	68	68	71.8	73.7	74.7	75.4	79.4
Number	97	98	99									
P(mm)	79.4	86	86									

Table A3: Rainfall Depth of 24 hour (mm) of GPD and ERA (region-B2)

Return period T (Years)	2	5	10	25	50	100
R24-Alaba Kulito of GPD	55.495	65.626	73.131	82.845	90.041	97.108
R24-B2(ERA)	55.26	69.95	79.68	92.03	101.29	110.61

Table A4: Generated Short Duration Rainfall (mm) by Generalized Pareto Distribution (GPD) method

t(min)	t(hr)	RRt	Rainfall in 24 hr (R24) for T of 2, 5, 10, 25, 50 &100 years respectively					
			55.495	65.626	73.131	82.845	90.041	97.108
			Short duration rainfall p(mm)					
5	0.083	0.158	8.8	10.4	11.5	13.1	14.2	15.3
10	0.167	0.264	14.6	17.3	19.3	21.8	23.7	25.6
15	0.250	0.340	18.9	22.3	24.9	28.2	30.6	33.0
20	0.333	0.398	22.1	26.1	29.1	33.0	35.8	38.7
25	0.417	0.444	24.6	29.1	32.5	36.8	40.0	43.1
30	0.500	0.482	26.7	31.6	35.2	39.9	43.4	46.8
35	0.583	0.513	28.5	33.7	37.5	42.5	46.2	49.8
40	0.667	0.540	29.9	35.4	39.5	44.7	48.6	52.4
45	0.750	0.562	31.2	36.9	41.1	46.6	50.6	54.6
50	0.833	0.583	32.3	38.2	42.6	48.3	52.5	56.6
55	0.917	0.600	33.3	39.4	43.9	49.7	54.1	58.3
60	1.000	0.616	34.2	40.4	45.1	51.0	55.5	59.8

65	1.083	0.630	35.0	41.4	46.1	52.2	56.8	61.2
70	1.167	0.643	35.7	42.2	47.1	53.3	57.9	62.5
75	1.250	0.655	36.4	43.0	47.9	54.3	59.0	63.6
80	1.333	0.666	37.0	43.7	48.7	55.2	60.0	64.7
85	1.417	0.676	37.5	44.4	49.4	56.0	60.9	65.6
90	1.500	0.685	38.0	45.0	50.1	56.8	61.7	66.5
95	1.583	0.694	38.5	45.5	50.7	57.5	62.5	67.4
100	1.667	0.702	38.9	46.1	51.3	58.1	63.2	68.1
105	1.750	0.709	39.4	46.5	51.9	58.8	63.9	68.9
110	1.833	0.716	39.7	47.0	52.4	59.3	64.5	69.6
115	1.917	0.723	40.1	47.4	52.9	59.9	65.1	70.2
120	2.000	0.729	40.5	47.8	53.3	60.4	65.6	70.8
125	2.083	0.735	40.8	48.2	53.8	60.9	66.2	71.4
130	2.167	0.741	41.1	48.6	54.2	61.4	66.7	71.9
135	2.250	0.746	41.4	49.0	54.6	61.8	67.2	72.4
140	2.333	0.751	41.7	49.3	54.9	62.2	67.6	72.9
145	2.417	0.756	41.9	49.6	55.3	62.6	68.1	73.4
150	2.500	0.761	42.2	49.9	55.6	63.0	68.5	73.9
155	2.583	0.765	42.4	50.2	55.9	63.4	68.9	74.3
160	2.667	0.769	42.7	50.5	56.3	63.7	69.3	74.7
165	2.750	0.773	42.9	50.7	56.5	64.1	69.6	75.1
170	2.833	0.777	43.1	51.0	56.8	64.4	70.0	75.5
175	2.917	0.781	43.3	51.3	57.1	64.7	70.3	75.8
180	3.000	0.785	43.5	51.5	57.4	65.0	70.6	76.2
P (mm) = R24 (mm)*RRt								

Table A5: Developed intensity (mm/hr) by Generalized Pareto Distribution (GDP) method

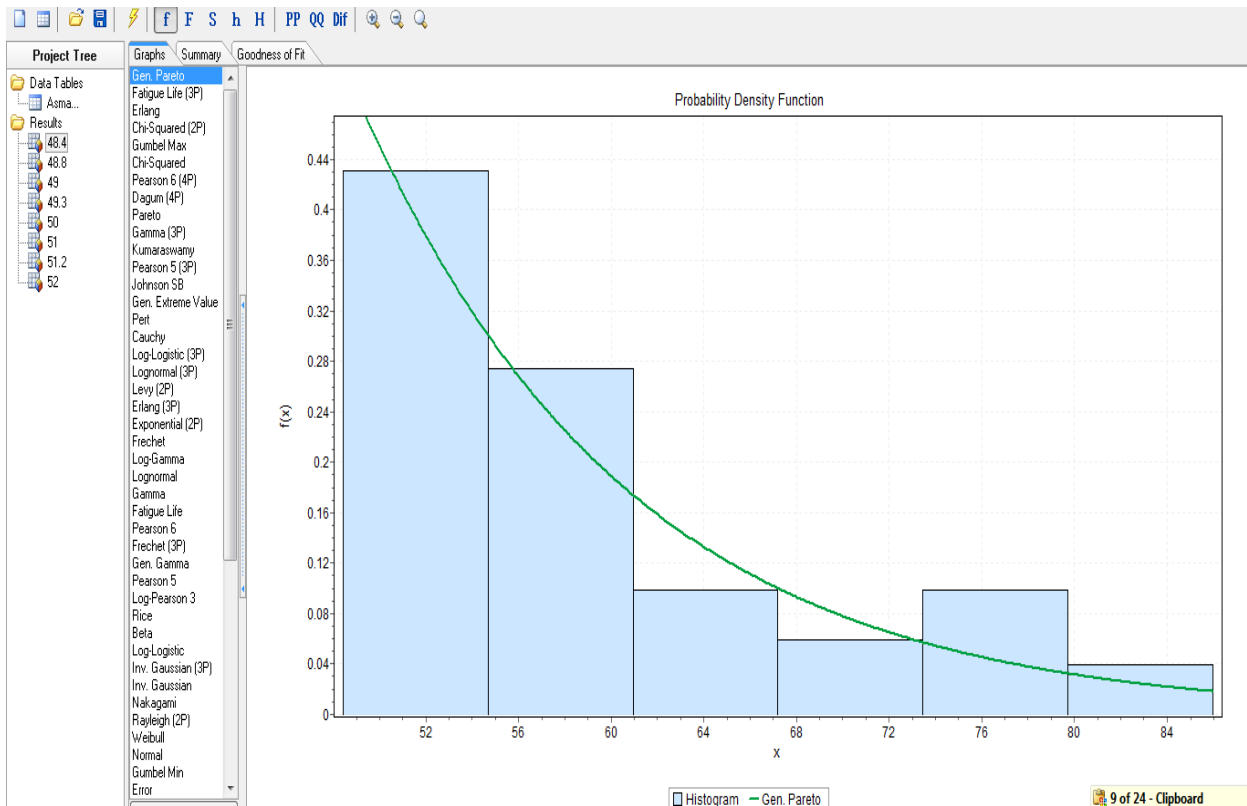
t(min)	t(hr)	RRt	Rainfall in 24 hr (R24) for T of 2, 5, 10, 25, 50 &100 years respectively					
			55.495	65.626	73.131	82.845	90.041	97.108
			INTENSITY I(mm/hr)					
5	0.083	0.158	105.2	124.4	138.6	157.0	170.6	184.0
10	0.167	0.264	87.8	103.8	115.7	131.0	142.4	153.6
15	0.250	0.340	75.5	89.2	99.4	112.6	122.4	132.0
20	0.333	0.398	66.3	78.4	87.3	98.9	107.5	116.0
25	0.417	0.444	59.1	69.9	77.9	88.3	96.0	103.5
30	0.500	0.482	53.5	63.2	70.4	79.8	86.7	93.5
35	0.583	0.513	48.8	57.7	64.3	72.8	79.2	85.4
40	0.667	0.540	44.9	53.1	59.2	67.0	72.9	78.6
45	0.750	0.562	41.6	49.2	54.8	62.1	67.5	72.8
50	0.833	0.583	38.8	45.9	51.1	57.9	62.9	67.9
55	0.917	0.600	36.3	43.0	47.9	54.3	59.0	63.6
60	1.000	0.616	34.2	40.4	45.1	51.0	55.5	59.8
65	1.083	0.630	32.3	38.2	42.6	48.2	52.4	56.5
70	1.167	0.643	30.6	36.2	40.3	45.7	49.7	53.6

75	1.250	0.655	29.1	34.4	38.3	43.4	47.2	50.9
80	1.333	0.666	27.7	32.8	36.5	41.4	45.0	48.5
85	1.417	0.676	26.5	31.3	34.9	39.5	43.0	46.3
90	1.500	0.685	25.3	30.0	33.4	37.8	41.1	44.4
95	1.583	0.694	24.3	28.8	32.0	36.3	39.5	42.5
100	1.667	0.702	23.4	27.6	30.8	34.9	37.9	40.9
105	1.750	0.709	22.5	26.6	29.6	33.6	36.5	39.4
110	1.833	0.716	21.7	25.6	28.6	32.4	35.2	37.9
115	1.917	0.723	20.9	24.8	27.6	31.2	34.0	36.6
120	2.000	0.729	20.2	23.9	26.7	30.2	32.8	35.4
125	2.083	0.735	19.6	23.2	25.8	29.2	31.8	34.3
130	2.167	0.741	19.0	22.4	25.0	28.3	30.8	33.2
135	2.250	0.746	18.4	21.8	24.2	27.5	29.9	32.2
140	2.333	0.751	17.9	21.1	23.5	26.7	29.0	31.3
145	2.417	0.756	17.4	20.5	22.9	25.9	28.2	30.4
150	2.500	0.761	16.9	20.0	22.2	25.2	27.4	29.5
155	2.583	0.765	16.4	19.4	21.7	24.5	26.7	28.8
160	2.667	0.769	16.0	18.9	21.1	23.9	26.0	28.0
165	2.750	0.773	15.6	18.5	20.6	23.3	25.3	27.3
170	2.833	0.777	15.2	18.0	20.1	22.7	24.7	26.6
175	2.917	0.781	14.9	17.6	19.6	22.2	24.1	26.0
180	3.000	0.785	14.5	17.2	19.1	21.7	23.5	25.4
I (mm/hr) = P (mm)*t(hr)								

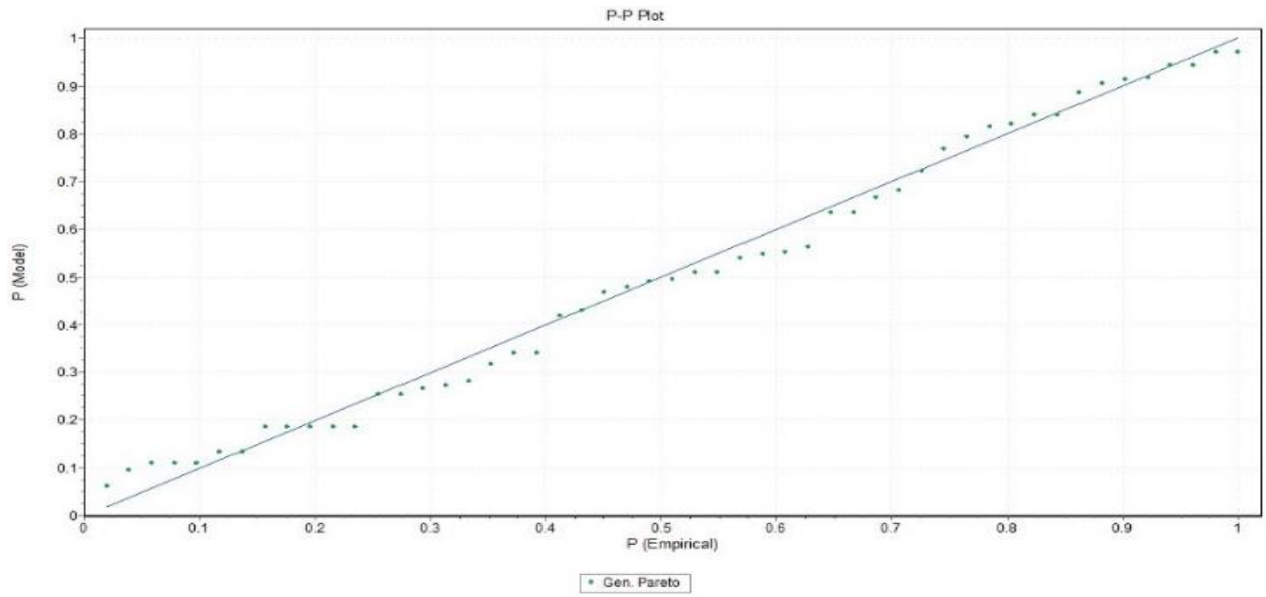
Table A6: Comparison of IDF curves developed by GPD and ERA (B2) of Alaba station

#	Return period (years)								
	2			5			10		
Duration (min)	IDF-ERA (a)	IDF-GPD (b)	Rank	IDF-ERA (a)	IDF-GPD (b)	Rank	IDF-ERA (a)	IDF-GPD (b)	Rank
5	104.73	105.17	b > a	132.57	124.37	a > b	151.01	138.59	a > b
10	87.39	87.77	b > a	110.62	103.79	a > b	126.01	115.66	a > b
15	75.13	77.69	b > a	95.10	91.88	a > b	108.33	102.38	a > b
20	65.98	66.26	b > a	83.52	78.36	a > b	95.14	87.32	a > b
25	58.89	59.14	b > a	74.54	69.93	a > b	84.91	77.93	a > b
30	53.22	53.45	b > a	67.37	63.21	a > b	76.74	70.44	a > b
60	34.05	34.20	b > a	43.10	40.44	a > b	49.10	45.06	a > b
90	25.24	25.35	b > a	31.95	29.97	a > b	36.40	33.40	a > b
120	20.15	20.23	b > a	25.50	23.92	a > b	29.05	26.66	a > b
150	16.81	16.88	b > a	21.28	19.96	a > b	24.24	22.25	a > b
180	14.45	14.51	b > a	18.29	17.16	a > b	20.84	19.13	a > b
#	Return period (years)								
	25			50			100		

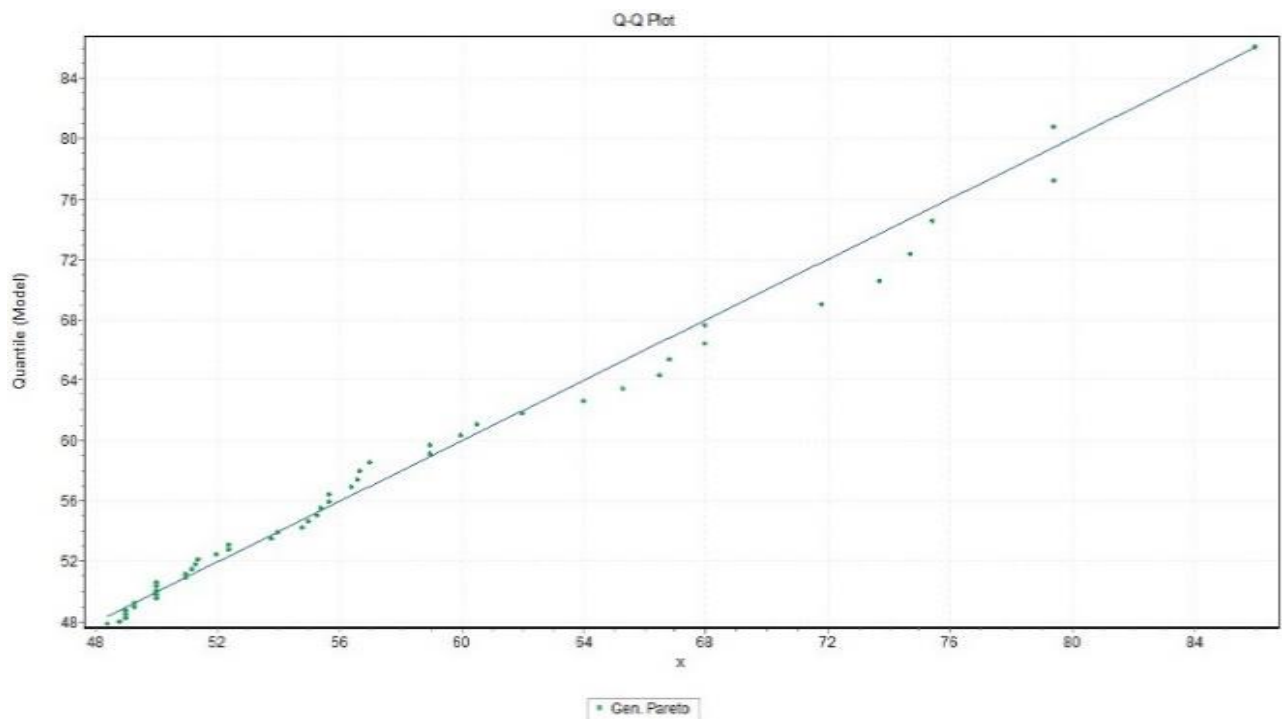
Duration (min)	IDF-ERA (a)	IDF-GPD (b)	Rank	IDF-ERA (a)	IDF-GPD (b)	Rank	IDF-ERA (a)	IDF-GPD (b)	Rank
5	174.41	157.00	a > b	191.96	170.64	a > b	209.62	184.03	a > b
10	145.54	131.03	a > b	160.18	142.41	a > b	174.92	153.59	a > b
15	125.12	115.98	a > b	137.71	126.06	a > b	150.39	135.95	a > b
20	109.88	98.92	a > b	120.94	107.51	a > b	132.07	115.95	a > b
25	98.07	88.28	a > b	107.93	95.95	a > b	117.87	103.48	a > b
30	88.64	79.80	a > b	97.56	86.73	a > b	106.54	93.53	a > b
60	56.71	51.05	a > b	62.41	55.48	a > b	68.16	59.84	a > b
90	42.04	37.84	a > b	46.27	41.12	a > b	50.52	44.35	a > b
120	33.55	30.20	a > b	36.93	32.82	a > b	40.32	35.40	a > b
150	28.00	25.20	a > b	30.81	27.39	a > b	33.65	29.54	a > b
180	24.07	21.67	a > b	26.49	23.55	a > b	28.93	25.40	a > b



(a)



(b)



(c)

**Figure A1:** Diagnosis plots for the reference period POT fit (51 data points from 48.4mm/day – 86mm/day) of the GP distribution: (a) best-fitting of GPD, (b) PP plot, (c) QQ plot.

## APPENDIX B: CONSTANTS USED TO HYDROLOGIC/HYDRAULIC ANALYSIS

Table B1: Recommended Runoff Coefficient C for Urban Catchment

Type of Drainage Area	Runoff Coefficient (C)
<b>Business</b>	
Commercial areas	0.70-0.95
Neighborhood areas	0.50-0.70
<b>Residential</b>	
Single-family areas	0.30-0.50
Multi-units, detached	0.40-0.60
Multi-units, attached	0.60-0.75
Suburban	0.25-0.40
Apartment dwelling areas	0.50-0.70
<b>Industrial</b>	
Light areas	0.50-0.80
Heavy areas	0.60-0.90
Parks, Cemeteries	0.10-0.25
Playgrounds	0.20-0.40
Railroad yard areas	0.20-0.40
Unimproved areas	0.10-0.30
<b>Lawns</b>	
Sandy soil, flat, <2%	0.05-0.10
Sandy soil, average 2 to 7%	0.10-0.15
Sandy soil, steep, >7%	0.15-0.20
Heavy soil, flat,<2%	0.13-0.17
Heavy soil, average 2 to 7%	0.18-0.22
Heavy soil, steep, >7%	0.25-0.35
<b>Streets</b>	
Asphaltic	0.70-0.95
Concrete	0.80-0.95
Brick	0.70-0.85
Drives and Walks	0.75-0.85
Roofs	0.75-0.95

Table B2: Recommended Runoff Coefficient C for Rural Catchment

Factor	Description	Runoff Coefficient
Average slope of catchment (Cs)	< 3.5% Flat	0.05
	3.5% -10% Soft to moderate	0.1
	10% - 25% Rolling	0.15
	25% - 45% Hilly	0.2
	> 45% Mountainous	0.25
Permeability of soil (Cp)	Well drained soil/ sand & gravel	0.05
	Fair drained soil/ sand & gravel with fine	0.1
	Poor drained soil/silt	0.15

	Impervious soil/clay, organic silt	0.25
	Water-logged /black cotton soil	0.5
	Rock	0.4
Vegetation (Cv)	Dense forest/thick bush	0.05
	Sparse forest/dense grass	0.1
	Grassland/scrub	0.15
	Cultivation	0.2
	Space grassland	0.25
	Barren	0.3
$C = C_s + C_p + C_v$		

Table B3: Runoff curve Number for selected Agricultural, Sub-Urban and Urban land use

Land use description	Hydrologic Soil Group				
	A	B	C	D	
Cultivate land without conservation treatment	72	<b>81</b>	88	91	
Cultivate land with conservation treatment	62	<b>71</b>	78	81	
Pasture land with poor condition	68	<b>79</b>	86	89	
pasture land with good condition	39	<b>61</b>	74	80	
Meadow with good condition	30	<b>58</b>	71	78	
wood or forest land with poor cover, no mulch	45	<b>66</b>	77	83	
wood or forest land with good cover	25	<b>55</b>	70	77	
Open space with good condition (>75% grass cover)	39	<b>61</b>	74	80	
Open space with fair condition (50% to 75% grass cover)	49	<b>69</b>	79	84	
Commercial & business area (85% impervious)	89	<b>92</b>	94	95	
Industrial districts (72% impervious)	81	<b>88</b>	91	93	
<b>Residential</b>					
Average lot size in acre	average % impervious	77	<b>85</b>	90	92
< 1/8	65				
1/4	38	61	<b>75</b>	83	87
1/3	30	57	<b>72</b>	81	86
1/2	25	54	<b>70</b>	80	85
1	20	51	<b>68</b>	79	84
paved parking lots, roofs, driveways		98	<b>98</b>	98	98
paved streets with curbs and storm sewers		98	<b>98</b>	98	98
paved streets with gravel		76	<b>85</b>	89	91
paved streets with dirt		72	<b>82</b>	87	89
Developing urban, no vegetation, pervious area		77	<b>86</b>	91	94

Table B4: Interpolated regression coefficients value

Ia/p	0.100	0.130	0.139	0.148	0.163	0.165	0.185	0.194	0.196	0.206	0.218
c <sub>0</sub>	2.553	2.540	2.536	2.532	2.525	2.524	2.516	2.512	2.511	2.522	2.501
c <sub>1</sub>	-0.615	-0.616	-0.617	-0.617	-0.618	-0.618	-0.618	-0.619	-0.619	-0.615	-0.620

$c_2$	-0.164	-0.157	-0.166	-0.153	-0.149	-0.149	-0.144	-0.142	-0.141	-0.164	-0.136
<b>Ia/p</b>	<b>0.230</b>	<b>0.244</b>	<b>0.250</b>	<b>0.258</b>	<b>0.280</b>	<b>0.289</b>	<b>0.300</b>	<b>0.310</b>	<b>0.320</b>	<b>0.330</b>	<b>0.350</b>
$c_0$	2.496	2.489	2.487	2.483	2.474	2.470	2.465	2.456	2.447	2.437	2.419
$c_1$	-0.620	-0.621	-0.621	-0.621	-0.622	-0.623	-0.623	-0.622	-0.620	-0.619	-0.616
$c_2$	-0.133	-0.130	-0.129	-0.127	-0.122	-0.120	-0.117	-0.111	-0.105	-0.100	-0.088

Table B5: Manning's roughness coefficient (n) for open channels

Channel types	Manning's n	Channel types	Manning's n	Channel types	Manning's n
lined channels		Excavated or dredged		Natural Channels	
Asphalt	0.013-0.017	Earthn, Straight &Uniform	0.02-0.03	Fairy Regular	0.03-0.07
Brick	0.012-0.018	Earth, Winding, Fairly uniform	0.025-0.04		
Concrete	0.011-0.020	Rock	0.03-0.045		
Rubble or Riprap	0.020-0.035	Unmaintained	0.05-0.14	Irregular section	0.04-0.100
Vegetal	0.030-0.40				

Table B6: Maximum Canal Side Slope m

Soil type	Side slope (H:V)
Sand, Soft Clay	3.0:1
Sandy Clay, Silt Loam, Sandy Loam	2.0:1
Fine Clay, Clay Loam	1.5:1
Heavy Clay	1.0:1
Stiff Clay with Concrete	0.5 to 1:1
Lined Canals	1.5:1

Table B7: Suggested Maximum Permissible Channel Velocities of erodible canals

Channel Material	Vmax (m/s)
Fine sand	0.6
Coarse sand	1.2
Fine gravel	1.8
Sandy silt	0.6
Silt clay	1
Clay	1.8
Bermuda grass on sandy silt	1.8
Bermuda grass on silt clay	2.4
Blue grass on sandy silt	1.5
Blue grass on silt caly	2.1
Sedimentary rock	3
Soft Sandstone	2.4
Soft Shale	1
Igneous or Hard Metamorphic rock	6

## APPENDIX C: HYDROLOGIC ANALYSIS SHEETS BY RATIONAL MEHOD

Table C1: Rainfall Ratio Computation of Sub Catchments for Rational Method of Q Determination

SC	t(min)	t(hr)	b+24	(b+24)^n	b+t	(b+t)^n	t/24	RRt
SC04	23.38	0.390	24.300	18.826	0.690	0.711	0.016	0.430
SC05	28.87	0.481	24.300	18.826	0.781	0.797	0.020	0.474
SC06	14.49	0.241	24.300	18.826	0.541	0.569	0.010	0.333
SC07	20.13	0.335	24.300	18.826	0.635	0.659	0.014	0.399
SC08	11.63	0.194	24.300	18.826	0.494	0.522	0.008	0.291
SC09	9.96	0.166	24.300	18.826	0.466	0.495	0.007	0.263
SC10	7.34	0.122	24.300	18.826	0.422	0.453	0.005	0.212
SC11	7.93	0.132	24.300	18.826	0.432	0.462	0.006	0.224
SC12	19.87	0.331	24.300	18.826	0.631	0.655	0.014	0.397
SC13	6.75	0.113	24.300	18.826	0.413	0.443	0.005	0.199
SC14	16.43	0.274	24.300	18.826	0.574	0.600	0.011	0.358
SC15	10.03	0.167	24.300	18.826	0.467	0.496	0.007	0.264
SC16	7.10	0.118	24.300	18.826	0.418	0.449	0.005	0.207
SC17	13.99	0.233	24.300	18.826	0.533	0.561	0.010	0.326
SC18	19.16	0.319	24.300	18.826	0.619	0.643	0.013	0.389
SC19	11.91	0.198	24.300	18.826	0.498	0.527	0.008	0.295
SC20	4.57	0.076	24.300	18.826	0.376	0.407	0.003	0.147
SC21	4.85	0.081	24.300	18.826	0.381	0.411	0.003	0.154
SC22	19.21	0.320	24.300	18.826	0.620	0.644	0.013	0.390
SC23	6.33	0.106	24.300	18.826	0.406	0.436	0.004	0.190
SC24	6.90	0.115	24.300	18.826	0.415	0.445	0.005	0.202
SC25	16.88	0.281	24.300	18.826	0.581	0.607	0.012	0.364
SC26	19.84	0.331	24.300	18.826	0.631	0.654	0.014	0.396
SC27	10.70	0.178	24.300	18.826	0.478	0.507	0.007	0.276
SC28	13.31	0.222	24.300	18.826	0.522	0.550	0.009	0.316
SC29	22.00	0.367	24.300	18.826	0.667	0.689	0.015	0.418
SC30	15.83	0.264	24.300	18.826	0.564	0.590	0.011	0.351

$$RRt = \frac{t}{24} \left( \frac{(b+24)^n}{(b+t)^n} \right) \dots \text{dimensionless number}$$

Table C2: Accumulated rainfall (P) in mm for return periods of 2, 5, 10, 25, 50 and 100 respectively

SC	t(min)	t(hr)	RRt	Rainfall of 24 hr (R24)					
				55.495	65.626	73.131	82.845	90.041	97.108
SC04	23.38	0.390	0.430	23.878	28.237	31.466	35.645	38.741	41.782
SC05	28.87	0.481	0.474	26.289	31.088	34.644	39.245	42.654	46.002
SC06	14.49	0.241	0.333	18.481	21.855	24.355	27.590	29.986	32.339
SC07	20.13	0.335	0.399	22.162	26.207	29.204	33.084	35.957	38.779
SC08	11.63	0.194	0.291	16.146	19.093	21.277	24.103	26.197	28.253
SC09	9.96	0.166	0.263	14.589	17.253	19.226	21.780	23.671	25.529
SC10	7.34	0.122	0.212	11.773	13.922	15.514	17.575	19.101	20.601
SC11	7.93	0.132	0.224	12.449	14.721	16.405	18.584	20.198	21.784
SC12	19.87	0.331	0.397	22.013	26.031	29.008	32.861	35.716	38.519
SC13	6.75	0.113	0.199	11.065	13.085	14.581	16.518	17.952	19.361
SC14	16.43	0.274	0.358	19.869	23.496	26.183	29.661	32.238	34.768
SC15	10.03	0.167	0.264	14.655	17.330	19.312	21.877	23.778	25.644
SC16	7.10	0.118	0.207	11.483	13.579	15.132	17.142	18.631	20.093
SC17	13.99	0.233	0.326	18.104	21.408	23.857	27.026	29.373	31.678
SC18	19.16	0.319	0.389	21.600	25.543	28.464	32.245	35.045	37.796
SC19	11.91	0.198	0.295	16.392	19.384	21.601	24.470	26.596	28.683
SC20	4.57	0.076	0.147	8.150	9.637	10.739	12.166	13.223	14.260
SC21	4.85	0.081	0.154	8.549	10.110	11.266	12.763	13.871	14.960
SC22	19.21	0.320	0.390	21.628	25.577	28.502	32.288	35.092	37.847
SC23	6.33	0.106	0.190	10.539	12.463	13.888	15.733	17.099	18.441
SC24	6.90	0.115	0.202	11.238	13.289	14.809	16.776	18.233	19.664
SC25	16.88	0.281	0.364	20.174	23.857	26.585	30.116	32.732	35.301
SC26	19.84	0.331	0.396	21.997	26.013	28.988	32.838	35.691	38.492
SC27	10.70	0.178	0.276	15.297	18.090	20.159	22.837	24.820	26.768
SC28	13.31	0.222	0.316	17.563	20.769	23.145	26.219	28.496	30.733
SC29	22.00	0.367	0.418	23.179	27.410	30.545	34.602	37.607	40.559
SC30	15.83	0.264	0.351	19.456	23.008	25.639	29.044	31.567	34.045
$P(mm) = R24(mm)*RRt$									

Table C3: Intensity of rainfall for each return period versus duration of sub-catchments

Sub-Catchment code	Time (min)	Time (hr)	Return Period(T)					
			2	5	10	25	50	100
			Intensity (mm/hr)					
SC04	23.385	0.390	61.264	72.45	80.73	91.46	99.40	107.20
SC05	28.871	0.481	54.635	64.61	72.00	81.56	88.65	95.60
SC06	14.486	0.241	76.549	90.52	100.88	114.28	124.20	133.95
SC07	20.128	0.335	66.063	78.12	87.06	98.62	107.19	115.60
SC08	11.627	0.194	83.32	98.53	109.80	124.38	135.19	145.80
SC09	9.961	0.166	87.87	103.92	115.80	131.18	142.58	153.77
SC10	7.343	0.122	96.19	113.75	126.76	143.60	156.08	168.33
SC11	7.930	0.132	94.19	111.38	124.12	140.61	152.82	164.82
SC12	19.866	0.331	66.48	78.62	87.61	99.25	107.87	116.33
SC13	6.754	0.113	98.30	116.24	129.54	146.75	159.49	172.01
SC14	16.428	0.274	72.57	85.81	95.63	108.33	117.74	126.98
SC15	10.028	0.167	87.68	103.69	115.55	130.90	142.26	153.43
SC16	7.099	0.118	97.06	114.77	127.90	144.89	157.47	169.83
SC17	13.991	0.233	77.64	91.81	102.31	115.90	125.97	135.86
SC18	19.157	0.319	67.65	80.00	89.15	100.99	109.76	118.37
SC19	11.907	0.198	82.60	97.68	108.85	123.31	134.02	144.54
SC20	4.569	0.076	107.02	126.56	141.03	159.77	173.64	187.27
SC21	4.847	0.081	105.82	125.14	139.45	157.97	171.69	185.17
SC22	19.206	0.320	67.57	79.90	89.04	100.87	109.63	118.23
SC23	6.332	0.106	99.87	118.10	131.60	149.08	162.03	174.75
SC24	6.895	0.115	97.78	115.64	128.86	145.98	158.66	171.11
SC25	16.883	0.281	71.70	84.79	94.48	107.03	116.33	125.46
SC26	19.839	0.331	66.53	78.67	87.67	99.31	107.94	116.41
SC27	10.698	0.178	85.80	101.46	113.07	128.08	139.21	150.13
SC28	13.305	0.222	79.20	93.66	104.37	118.23	128.50	138.59
SC29	22.001	0.367	63.21	74.75	83.30	94.36	102.56	110.61
SC30	15.829	0.264	73.75	87.21	97.19	110.10	119.66	129.05
$I \text{ (mm/hr)} = P \text{ (mm)} / t \text{ (hr)}$								

Table C4: Discharge by Rational Method in m<sup>3</sup>/s

Return Period (T)			2	5	10	25	50	100
Frequency Factors (Cf)			1	1	1	1.1	1.2	1.25
SC	C	A(ha)	Q (m <sup>3</sup> /s)					
SC04	0.471	8.12	0.65	0.77	0.86	1.07	1.27	1.42
SC05	0.605	1.54	0.14	0.17	0.19	0.23	0.27	0.31
SC06	0.537	4.38	0.50	0.59	0.66	0.82	0.97	1.09
SC07	0.541	5.76	0.57	0.68	0.75	0.94	1.11	1.25
SC08	0.769	0.19	0.03	0.04	0.04	0.06	0.07	0.07
SC09	0.728	0.87	0.16	0.18	0.20	0.25	0.30	0.34
SC10	0.886	0.08	0.02	0.02	0.02	0.03	0.04	0.04
SC11	0.841	0.56	0.12	0.15	0.16	0.20	0.24	0.27
SC12	0.739	8.06	1.10	1.30	1.45	1.81	2.14	2.41
SC13	0.769	1.02	0.21	0.25	0.28	0.35	0.42	0.47
SC14	0.598	9.23	1.11	1.31	1.47	1.83	2.16	2.43
SC15	0.843	0.61	0.12	0.15	0.16	0.21	0.24	0.27
SC16	0.846	0.51	0.12	0.14	0.15	0.19	0.23	0.25
SC17	0.809	3.18	0.56	0.66	0.73	0.91	1.08	1.21
SC18	0.784	5.67	0.84	0.99	1.10	1.37	1.63	1.83
SC19	0.780	2.59	0.46	0.55	0.61	0.76	0.90	1.01
SC20	0.674	0.24	0.05	0.06	0.06	0.08	0.10	0.11
SC21	0.709	0.33	0.07	0.08	0.09	0.11	0.13	0.15
SC22	0.695	9.20	1.20	1.42	1.58	1.97	2.34	2.63
SC23	0.785	0.35	0.08	0.09	0.10	0.13	0.15	0.17
SC24	0.841	0.42	0.10	0.11	0.13	0.16	0.19	0.21
SC25	0.656	3.56	0.47	0.55	0.61	0.76	0.91	1.02
SC26	0.467	17.38	1.50	1.77	1.98	2.46	2.92	3.28
SC27	0.821	0.60	0.12	0.14	0.15	0.19	0.23	0.26
SC28	0.791	0.90	0.16	0.18	0.21	0.26	0.30	0.34
SC29	0.626	7.09	0.78	0.92	1.03	1.28	1.52	1.71
SC30	0.857	0.46	0.08	0.10	0.11	0.13	0.16	0.18
$Q(m^3/s) = C * Cf * A(ha) * I(mm/hr) / 360$								

Table C5: SCS peak Discharge Method Coefficients (Ia/P)

Rainfall type	Ia/P	CO	C1	C2	Rainfall type	Ia/P	CO	C1	C2
I	0.1	2.3055	-0.514	-0.117	<b>II</b>	<b>0.1</b>	<b>2.553</b>	<b>-0.615</b>	<b>-0.164</b>
I	0.2	2.235	-0.504	-0.09	<b>II</b>	<b>0.3</b>	<b>2.465</b>	<b>-0.623</b>	<b>-0.117</b>
I	0.3	2.106	-0.457	-0.028	<b>II</b>	<b>0.35</b>	<b>2.419</b>	<b>-0.616</b>	<b>-0.088</b>
I	0.4	1.877	-0.322	-0.058	<b>II</b>	<b>0.4</b>	<b>2.364</b>	<b>-0.599</b>	<b>-0.056</b>
I	0.5	1.679	-0.069	0.000	<b>II</b>	<b>0.5</b>	<b>2.203</b>	<b>-0.516</b>	<b>-0.013</b>
IA	0.1	2.033	-0.316	-0.138	III	0.1	2.473	-0.518	-0.171
IA	0.2	1.919	-0.282	-0.070	III	0.3	2.396	-0.512	-0.133
IA	0.25	1.838	-0.256	-0.026	III	0.35	2.355	-0.497	-0.120
IA	0.3	1.727	-0.198	-0.026	III	0.4	2.307	-0.413	-0.111
IA	0.5	1.634	-0.091	-0.000	III	0.5	2.178	-0.368	-0.095



Figure C1: Manning's coefficient ( $n = 0.020$  (lower) and  $0.015$  (upper) are taken

**APPENDIX D: EXISTING DRAINAGE CANNELS CAPACITY BY MANNING EQUATION**

Conduit code	Area (m <sup>2</sup> )	Slope (%)	n	R(m)	V (m/s)	Q(m <sup>3</sup> /s)
L01R	1.000	0.361	0.020	0.3333	1.4443	2.152
L02R	0.930	0.422	0.020	0.3252	1.5358	2.128
L03L	1.170	0.834	0.020	0.3503	2.2697	3.957
L04R	1.500	0.242	0.015	0.4054	1.7947	4.011
L05L	0.618	0.336	0.020	0.2422	1.1253	1.035
L06R	0.694	0.976	0.015	0.2550	2.6487	2.737
L06L	0.646	0.562	0.015	0.2504	1.9851	1.911
L07L	0.740	0.251	0.015	0.2701	1.3962	1.539
L07R	0.700	0.278	0.015	0.2593	1.4286	1.490
L08R	0.595	1.000	0.020	0.2479	1.9732	1.749
C01	0.900	0.833	0.015	0.3333	2.9258	3.923
C02	0.805	2.500	0.015	0.2683	4.3853	5.260
C03	1.200	2.500	0.015	0.3750	5.4815	9.801
L09L	1.960	1.093	0.020	0.4667	3.1448	9.184
L10L	2.250	1.613	0.020	0.5000	4.0003	13.411
L11R	0.989	0.463	0.020	0.3130	1.5683	2.311
C04	2.000	0.714	0.015	0.5000	3.5494	10.577
L12L	0.945	0.471	0.020	0.3150	1.5879	2.236
L12R	0.850	0.471	0.020	0.2982	1.5311	1.939
L11L	0.972	0.469	0.020	0.3115	1.5742	2.279
L13L	3.200	0.331	0.020	0.5714	1.9796	9.439
L14L	0.848	0.617	0.020	0.2904	1.7227	2.177
L15L	3.200	0.735	0.020	0.5714	2.9524	14.077
L16L	0.706	0.309	0.020	0.2522	1.1089	1.167
L16R	0.706	0.313	0.020	0.2522	1.1158	1.174
C05	2.400	0.833	0.015	0.5455	4.0628	14.529
L17R	0.680	0.615	0.020	0.2537	1.5720	1.593
L17L	0.653	0.625	0.020	0.2511	1.5732	1.530
L18L	4.000	0.405	0.020	0.6667	2.4288	14.475
L19R	0.490	0.132	0.020	0.2333	0.6874	0.502
L20L	1.276	1.183	0.015	0.3376	3.5160	6.685
L21L	3.120	0.097	0.015	0.5612	1.4098	6.554
L22L	4.000	1.014	0.015	0.6667	4.0132	23.919
L23R	2.700	0.030	0.015	0.5294	0.7538	3.032
C06	1.766	1.429	0.015	0.7500	6.5776	17.310
L23L	1.000	0.029	0.015	0.3333	0.5457	0.813
L24	1.766	1.800	0.015	0.7500	7.3833	19.431

**APPENDIX E: SWMM INPUT PARAMETERS AND CORRELATION VALUE OF SIMULATED AND ESTIMATED PEAK DISCHARGE**

Table E1: Sub-Catchment Property

Name	Outlet node	Area (ha)	Width (m)	Slope (%)	Imperv. %	N-Imperv	N-Perv	Dstore.Imperv (mm)	Dstore.perv (mm)
SC01	J01	1305.24	1155.08	1.91	6	0.029	0.1	1.65	3.01
SC02	J05	116.73	323.36	0.66	10	0.029	0.1	1.65	3.01
SC03	J11	202.58	386.60	0.65	10	0.029	0.1	1.65	3.01
SC04	J04	8.12	109.76	0.81	42	0.029	0.1	1.65	3.01
SC05	J03	1.54	21.07	0.55	58	0.029	0.1	1.65	3.01
SC06	J02	4.38	224.81	1.03	48	0.029	0.1	1.65	3.01
SC07	J09	5.76	215.84	0.37	55	0.029	0.1	1.65	3.01
SC08	J10	0.19	22.35	1.19	64	0.029	0.1	1.65	3.01
SC09	J13	0.87	385.86	0.88	75	0.029	0.1	1.65	3.01
SC10	J14	0.08	23.09	0.88	95	0.029	0.1	1.65	3.01
SC11	J07	0.56	561.04	2.00	75	0.029	0.1	1.65	3.01
SC12	J16	8.06	435.90	0.54	65	0.029	0.1	1.65	3.01
SC13	J15	1.02	234.57	2.31	70	0.029	0.1	1.65	3.01
SC14	J18	9.23	423.38	1.38	55	0.029	0.1	1.65	3.01
SC15	J19	0.61	225.27	0.74	80	0.029	0.1	1.65	3.01
SC16	J20	0.51	389.90	2.31	80	0.029	0.1	1.65	3.01
SC17	J30	3.18	303.08	0.95	79	0.029	0.1	1.65	3.01
SC18	J24	5.67	219.90	1.55	74	0.029	0.1	1.65	3.01
SC19	J22	2.59	186.00	2.16	76	0.029	0.1	1.65	3.01
SC20	J21	0.24	153.00	1.87	60	0.029	0.1	1.65	3.01
SC21	J23	0.33	203.56	1.25	35	0.029	0.1	1.65	3.01
SC22	J25	9.20	375.65	0.82	63	0.029	0.1	1.65	3.01
SC23	J28	0.35	219.81	2.50	75	0.029	0.1	1.65	3.01
SC24	J27	0.42	419.36	2.00	75	0.029	0.1	1.65	3.01
SC25	J31	3.56	140.26	1.57	58	0.029	0.1	1.65	3.01
SC26	J33	17.38	538.21	0.62	41	0.029	0.1	1.65	3.01
SC27	J32	0.60	93.09	1.56	70	0.029	0.1	1.65	3.01
SC28	J33	0.90	92.41	0.52	69	0.029	0.1	1.65	3.01
SC29	J34	7.09	239.63	1.69	54	0.029	0.1	1.65	3.01
SC30	J36	0.46	139.73	3.03	75	0.029	0.1	1.65	3.01

Table E2: Conduit Properties

Conduit code	Conduit Type/Shape/Surface	Inlet node	Outlet node	length (m)	Width (m)	Depth (m)	Slope (%)	Roughness	Location/Near to
L01R	Open/ Rectangular/Masonry	J01	J03	831	1.00	1.00	0.361	0.020	Haymale Mewcha
L02R	Open/ Rectangular/Masonry	J02	J03	237	1.00	0.93	0.422	0.020	Haymale Mewcha
L03L	Open/ Rectangular/Masonry	J03	J12	71.9	1.00	1.17	0.834	0.020	Edget Jerba
L04R	Open/ Rectangular/Concrete	J10	J11	207	1.20	1.25	0.242	0.015	Shashene Mewcha
L05L	Open/ Rectangular/Masonry	J09	J11	149	0.65	0.95	0.336	0.020	Dashn Bank
L06R	Open/ Rectangular/Concrete	J11	J12	461	0.68	1.02	0.976	0.015	Menaharya
L06L	Open/ Rectangular/Concrete	J13	J14	534	0.68	0.95	0.562	0.015	Menaharya
L07L	Open/ Rectangular/Concrete	J05	J06	398	0.74	1.00	0.251	0.015	Hayat Dabo Megagerya
L07R	Open/ Rectangular/Concrete	J07	J08	396	0.70	1.00	0.278	0.015	Hayat Dabo Megagerya
L08R	Open/ Rectangular/Masonry	J04	J06	20	0.70	0.85	1.000	0.020	Haymale Adebabay
C01	Closed/ Rectangular/Concrete	J06	J08	12	1.50	0.60	0.833	0.015	Haymale Adebabay
C02	Closed/ Rectangular/Concrete	J08	J15	36	0.70	1.15	2.500	0.015	Haymale Adebabay
C03	Closed/ Rectangular/Concrete	J12	J14	12	1.20	1.00	2.500	0.015	Haymale Adebabay
L09L	Open/ Rectangular/Masonry	J14	J15	18.3	1.40	1.40	1.093	0.020	Haymale Adebabay
L10L	Open/ Rectangular/Masonry	J15	J17	186	1.50	1.50	1.613	0.020	Haymale Adebabay
L11R	Open/ Rectangular/Masonry	J18	J17	432	0.86	1.15	0.463	0.020	Jemal Res. House
C04	Closed/ Rectangular/Concrete	J17	J21	14	2.00	1.00	0.714	0.015	Dubay one fashion
L12L	Open/ Rectangular/Masonry	J16	J17	425	0.90	1.05	0.471	0.020	Dubay one fashion
L12R	Open/ Rectangular/Masonry	J20	J21	425	0.85	1.00	0.471	0.020	Ommo Micro finance
L11L	Open/ Rectangular/Masonry	J19	J17	426	0.86	1.13	0.469	0.020	Jemal Res. house
L13L	Open/ Rectangular/Masonry	J17	J23	121	1.50	1.90	0.331	0.020	lowee Condominium
L14L	Open/ Rectangular/Masonry	J22	J23	162	0.80	1.06	0.617	0.020	Deneke Molla Res. House
L15L	Open/ Rectangular/Masonry	J23	J26	136	1.50	1.90	0.735	0.020	Gubae Egziabiher Church
L16L	Open/ Rectangular/Masonry	J24	J26	324	0.66	1.07	0.309	0.020	Ker Tej House/Enkutatash mgb

L16R	Open/ Rectangular/Masonry	J27	J29	320	0.66	1.07	0.313	0.020	Ker Tej House/Enkutatash mgb
C05	Closed/ Rectangular/Concrete	J26	J29	12	2.00	1.20	0.833	0.015	Green View Hotel
L17R	Open/ Rectangular/Masonry	J28	J26	325	0.68	1.00	0.615	0.020	Green View Hotel
L17L	Open/ Rectangular/Masonry	J28	J29	320	0.68	0.96	0.625	0.020	Green View Hotel
L18L	Open/ Rectangular/Masonry	J29	J33	543	1.60	2.00	0.405	0.020	Mender 39 Bridge
L19R	Open/ Rectangular/Masonry	J31	J32	152	0.70	0.70	0.132	0.020	Eden Wuha Akefafay
L20L	Closed/ Rectangular/Concrete	J30	J32	507	0.88	1.45	1.183	0.015	Commercial Bank/Market
L21L	Closed/ Rectangular/Concrete	J32	J33	207	1.56	2.00	0.097	0.015	Zagol Nedaj Madeya
L22L	Closed/ Rectangular/Concrete	J33	J35	163	1.60	2.00	1.014	0.015	Halaba View Hotel
L23R	Closed/ Rectangular/Concrete	J34	J35	335	1.50	1.80	0.030	0.015	Nafkot Pension
C06	Closed/ Circular/Concrete	J35	J37	14	Dia = 1.5		1.429	0.015	ST Gabriel
L23L	Closed/ Rectangular/Concrete	J36	J37	345	1.00	1.00	0.029	0.015	Nafkot Pension
L24	Closed/ Circular/Concrete	J37	OUT1	14	Dia = 1.5		2.857 1	0.015	ST Gabriel

Table E3: Node property

Name	Type	X(m)	Y(m)	Max. Depth(m)	Invert elvn. (m)
J01	Junction	400433.08	808923.33	0.90	1783.1
J02	Junction	400098.7	808349.21	0.90	1780.1
J03	Junction	399890.72	808462.62	0.90	1779.1
J04	Junction	399818.82	808448.95	0.80	1779.2
J05	Junction	399449.83	808624.76	1.00	1780
J06	Junction	399802.64	808440.4	1.00	1779
J07	Junction	399444.03	808624.76	1.00	1780
J08	Junction	399795.58	808430.67	1.20	1778.8
J09	Junction	400386.16	808250.65	0.50	1783
J10	Junction	400443.59	808204.41	0.50	1783
J11	Junction	400249.54	808204.17	0.80	1782.2
J12	Junction	399840.04	808419.54	1.40	1778.6
J13	Junction	400282.54	808152.41	1.00	1782
J14	Junction	399832	808409	1.60	1778.4
J15	Junction	399815	808405	2.00	1778
J16	Junction	399351.64	808446.03	1.00	1778
J17	Junction	399725.54	808241.99	2.00	1775
J18	Junction	400102.12	808032.42	1.00	1779
J19	Junction	400072.24	808008.67	1.00	1778.5
J20	Junction	399344.81	808430.9	1.00	1779

J21	Junction	399717	808227	2.10	1774.9
J22	Junction	399535.87	808224.11	1.00	1776
J23	Junction	399662	808121	1.50	1774.5
J24	Junction	399309.48	808159.57	1.00	1776
J25	Junction	399883.61	807857.68	1.00	1776
J26	Junction	399593	808004	1.50	1773.5
J27	Junction	399305.15	808148.07	1.00	1775
J28	Junction	399861.84	807842.56	1.00	1776
J29	Junction	399585	807995	1.60	1773.4
J30	Junction	399330.84	808432.07	1.60	1777.4
J31	Junction	399374.92	807827.55	1.00	1773
J32	Junction	399278	807932	1.60	1772.4
J33	Junction	399177	807746	1.80	1771.2
J34	Junction	399176.01	807296.81	1.00	1768
J35	Junction	399100	807606	2.10	1767.9
J36	Junction	399139.93	807271.58	0.80	1768.2
J37	Junction	399085	807609	2.20	1767.8
OUT	Outfall	399072.18	807607.69		1767.4

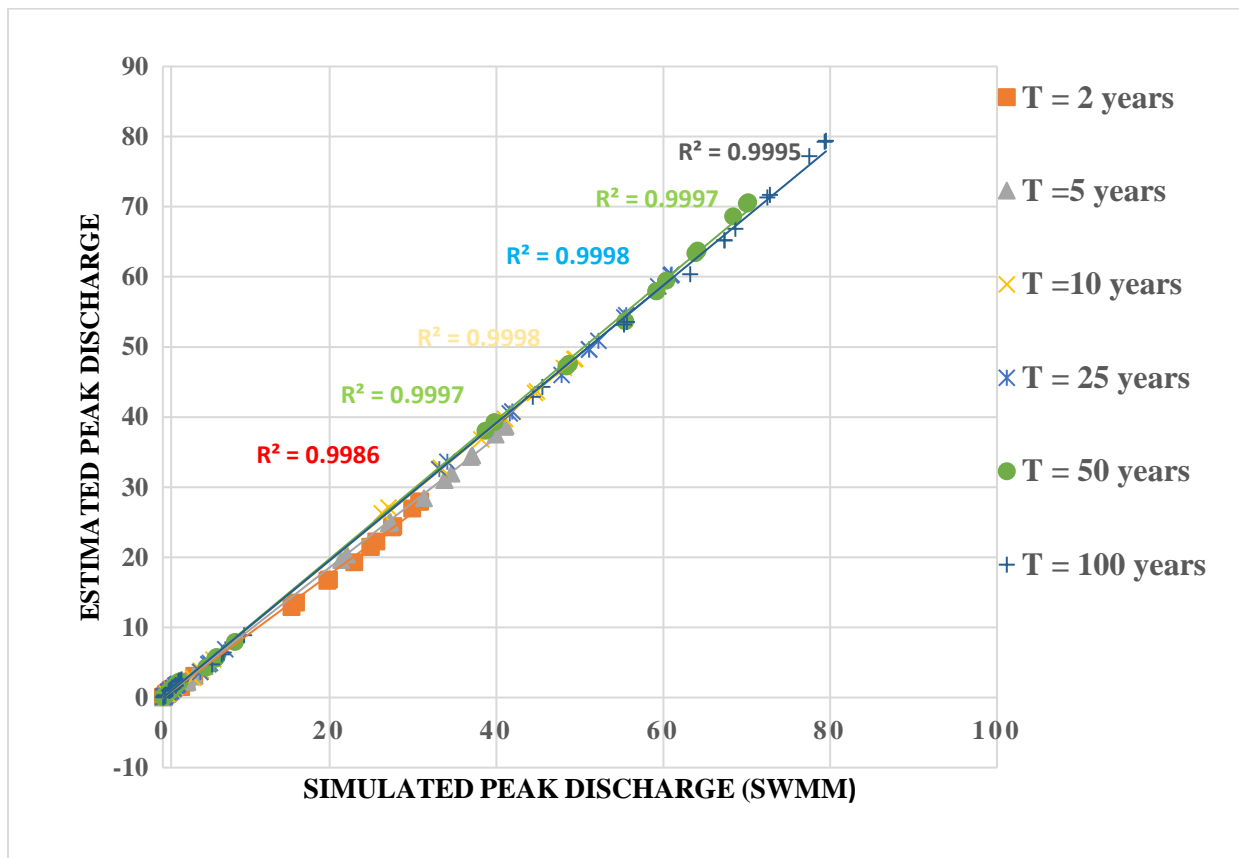


Figure E1: Correlation Value of Simulated and Estimated Peak Discharge for different return period

## APPENDIX F: COMPARISON OF DISCHARGE

Table F1: Peak discharge (m<sup>3</sup>/s) Comparison without Diversion Work

Conduit code	Q2		Q5		Q10		Q25		Q50		Q100		MANNINGS
	SWMM	SCS/RAT	SWMM	SCS/RAT	SWMM	SCS/RAT	SWMM	SCS/RAT	SWMM	SCS/RAT	SWMM	SCS/RAT	
L01R	15.42	12.94	21.37	19.64587	26.27	26.28	33.17	32.57092	38.69	38.05	44.36	42.9188	2.15
L02R	0.43	0.50	0.54	0.591703	0.62	0.66	0.74	0.82165	0.82	0.97	0.91	1.094442	2.13
L03L	15.97	13.58	22.07	20.40458	27.07	27.13	34.12	33.62448	39.75	39.30	45.53	44.32215	3.96
L04R	0.03	0.03	0.03	0.039657	0.04	0.04	0.04	0.055069	0.05	0.07	0.05	0.073352	4.01
L05L	0.55	0.57	0.68	0.677128	0.79	0.75	0.93	0.940273	1.04	1.11	1.14	1.252449	1.04
L06R	3.76	3.11	5.02	4.473518	6.06	5.44	7.49	6.914512	8.63	7.96	9.77	8.905991	2.74
L06L	0.16	0.16	0.19	0.1833	0.22	0.20	0.26	0.254534	0.28	0.30	0.31	0.339041	1.91
L07L	2.2	1.51	2.99	2.252935	3.63	2.84	4.51	3.679946	5.21	4.28	5.91	4.746588	1.54
L07R	0.12	0.12	0.14	0.146045	0.16	0.16	0.18	0.202801	0.2	0.24	0.21	0.270132	1.49
L08R	0.55	0.65	0.69	0.769453	0.8	0.86	0.95	1.068478	1.07	1.27	1.18	1.423218	1.75
C01	2.75	2.16	3.68	3.022389	4.43	3.70	5.46	4.748424	6.28	5.54	7.09	6.169806	3.92
C02	2.87	2.28	3.82	3.168434	4.59	3.86	5.64	4.951225	6.48	5.78	7.3	6.439938	5.26
C03	19.73	16.68	27.09	24.8781	33.13	32.56	41.61	40.539	48.38	47.26	55.3	53.22814	9.80
L09L	19.91	16.86	27.3	25.08338	33.37	32.79	41.9	40.82405	48.69	47.59	55.64	53.60784	9.18
L10L	22.94	19.29	31.31	28.43511	38.18	36.86	47.8	46.02981	55.45	53.68	63.25	60.38681	13.41
L11R	1.02	1.11	1.27	1.314697	1.46	1.47	1.72	1.825614	1.92	2.16	2.12	2.431727	2.31
C04	24.93	21.50	33.78	31.05148	41.01	39.77	51.13	49.66295	59.16	57.99	67.35	65.22617	10.58
L12L	0.97	1.10	1.2	1.301667	1.37	1.45	1.61	1.80752	1.79	2.14	1.98	2.407626	2.24
L12R	0.11	0.12	0.14	0.136784	0.15	0.15	0.17	0.189941	0.19	0.23	0.21	0.253003	1.94
L11L	0.12	0.12	0.14	0.147755	0.16	0.16	0.19	0.205175	0.21	0.24	0.22	0.273294	2.28
L13L	24.93	21.50	33.78	31.05148	41.01	39.77	51.13	49.66295	59.16	57.99	67.35	65.22617	9.44
L14L	0.4	0.46	0.49	0.547378	0.57	0.61	0.66	0.760099	0.73	0.90	0.8	1.012456	2.18
L15L	25.6	22.26	34.6	31.94139	41.95	40.76	52.21	50.89869	60.36	59.45	68.66	66.87219	14.08
L16L	0.77	0.84	0.96	0.987849	1.1	1.10	1.29	1.371746	1.43	1.63	1.57	1.827174	1.17
L16R	0.09	0.10	0.11	0.113336	0.12	0.13	0.14	0.157381	0.15	0.19	0.16	0.209632	1.17
C05	27.47	24.29	36.92	34.34869	44.62	43.45	55.33	54.24151	63.83	63.42	72.48	71.32485	14.53
L17R	1.1	1.20	1.36	1.419447	1.57	1.58	1.83	1.971071	2.04	2.34	2.25	2.625478	1.59
L17L	0.07	0.08	0.09	0.090591	0.1	0.10	0.11	0.125796	0.12	0.15	0.13	0.167561	1.53
L18L	27.63	24.47	37.12	34.55261	44.84	43.67	55.58	54.52469	64.1	63.75	72.77	71.70204	14.48
L19R	0.41	0.47	0.5	0.550678	0.58	0.61	0.68	0.764682	0.76	0.91	0.84	1.01856	0.50
L20L	0.51	0.56	0.63	0.656392	0.72	0.73	0.84	0.911478	0.93	1.08	1.02	1.214093	6.68
L21L	0.92	1.02	1.13	1.207069	1.3	1.35	1.52	1.676159	1.69	1.99	1.86	2.232653	6.55
L22L	29.88	26.98	39.92	37.53204	48.08	46.99	59.41	58.66197	68.39	68.66	77.52	77.21293	23.92
L23R	0.75	0.78	0.93	0.922249	1.07	1.03	1.26	1.280652	1.41	1.52	1.56	1.705836	3.03
C06	30.75	27.92	41	38.63874	49.33	48.23	60.88	60.19876	70.03	70.48	79.33	79.25993	17.31
L23L	0.09	0.08	0.11	0.095689	0.12	0.11	0.14	0.132876	0.15	0.16	0.17	0.176992	0.81
L24	30.84	28.00	41.11	38.73443	49.45	48.33	61.02	60.33164	70.18	70.64	79.5	79.43693	19.43

Table F2: Peak discharge (m<sup>3</sup>/s) Comparison if Diversion Work is Done

Conduit code	Q2		Q5		Q10		Q25		Q50		Q100		MANNINGS
	SWMM	SCS/RAT.	SWMM	SCS/RAT.	SWMM	SCS/RAT.	SWMM	SCS/RAT.	SWMM	SCS/RAT.	SWMM	SCS/RAT.	
L01R	3.84	2.96	5.32	4.49	6.51	5.77	8.15	7.45	9.45	8.70	10.75	9.90	2.15
L02R	0.43	0.50	0.54	0.59	0.62	0.66	0.74	0.82	0.82	0.97	0.91	1.09	2.13
L03L	4.39	3.60	6.02	5.25	7.31	6.62	9.1	8.50	10.51	9.95	11.92	11.30	3.96
L04R	0.03	0.03	0.03	0.04	0.04	0.04	0.04	0.06	0.05	0.07	0.05	0.07	4.01
L05L	0.55	0.57	0.68	0.68	0.79	0.75	0.93	0.94	1.04	1.11	1.14	1.25	1.04
L06R	3.38	3.05	4.53	4.39	5.47	5.33	6.74	6.78	7.75	7.80	8.76	8.73	2.74
L06L	0.16	0.16	0.19	0.18	0.22	0.20	0.26	0.25	0.28	0.30	0.31	0.34	1.91
L07L	2.2	1.51	2.99	2.25	3.63	2.84	4.51	3.68	5.21	4.28	5.91	4.75	1.54
L07R	0.12	0.12	0.14	0.15	0.16	0.16	0.18	0.20	0.2	0.24	0.21	0.27	1.49
L08R	0.55	0.65	0.69	0.77	0.8	0.86	0.95	1.07	1.07	1.27	1.18	1.42	1.75
C01	2.75	2.16	3.68	3.02	4.43	3.70	5.46	4.75	6.28	5.54	7.09	6.17	3.92
C02	2.87	2.28	3.82	3.17	4.59	3.86	5.64	4.95	6.48	5.78	7.3	6.44	5.26
C03	7.77	6.65	10.55	9.64	12.78	11.94	15.84	15.28	18.26	17.75	20.68	20.03	9.80
L09L	7.95	6.82	10.76	9.84	13.02	12.17	16.13	15.56	18.57	18.09	21.02	20.41	9.18
L10L	10.98	9.26	14.77	13.19	17.83	16.24	22.03	20.77	25.33	24.17	28.63	27.19	13.41
L11R	1.02	1.11	1.27	1.31	1.46	1.47	1.72	1.83	1.92	2.16	2.12	2.43	2.31
C04	12.97	11.47	17.24	15.81	20.66	19.15	25.36	24.40	29.04	28.48	32.73	32.03	10.58
L12L	0.97	1.10	1.2	1.30	1.37	1.45	1.61	1.81	1.79	2.14	1.98	2.41	2.24
L12R	0.11	0.12	0.14	0.14	0.15	0.15	0.17	0.19	0.19	0.23	0.21	0.25	1.94
L11L	0.12	0.12	0.14	0.15	0.16	0.16	0.19	0.21	0.21	0.24	0.22	0.27	2.28
L13L	12.97	11.47	17.24	15.81	20.66	19.15	25.36	24.40	29.04	28.48	32.73	32.03	9.44
L14L	0.4	0.46	0.49	0.55	0.57	0.61	0.66	0.76	0.73	0.90	0.8	1.01	2.18
L15L	13.64	12.22	18.06	16.70	21.6	20.14	26.44	25.64	30.24	29.95	34.04	33.67	14.08
L16L	0.77	0.84	0.96	0.99	1.1	1.10	1.29	1.37	1.43	1.63	1.57	1.83	1.17
L16R	0.09	0.10	0.11	0.11	0.12	0.13	0.14	0.16	0.15	0.19	0.16	0.21	1.17
C05	15.51	14.26	20.38	19.11	24.27	22.83	29.56	28.98	33.71	33.91	37.86	38.13	14.53
L17R	1.1	1.20	1.36	1.42	1.57	1.58	1.83	1.97	2.04	2.34	2.25	2.63	1.59
L17L	0.07	0.08	0.09	0.09	0.1	0.10	0.11	0.13	0.12	0.15	0.13	0.17	1.53
L18L	15.67	14.43	20.58	19.31	24.49	23.05	29.81	29.26	33.98	34.25	38.15	38.50	14.48
L19R	0.41	0.47	0.5	0.55	0.58	0.61	0.68	0.76	0.76	0.91	0.84	1.02	0.50
L20L	0.51	0.56	0.63	0.66	0.72	0.73	0.84	0.91	0.93	1.08	1.02	1.21	6.68
L21L	0.92	1.02	1.13	1.21	1.3	1.35	1.52	1.68	1.69	1.99	1.86	2.23	6.55
L22L	17.92	16.95	23.38	22.29	27.73	26.37	33.64	33.40	38.27	39.15	42.9	44.01	23.92
L23R	0.75	0.78	0.93	0.92	1.07	1.03	1.26	1.28	1.41	1.52	1.56	1.71	3.03
C06	18.79	17.89	24.46	23.40	28.98	27.61	35.11	34.94	39.91	40.97	44.71	46.06	17.31
L23L	0.09	0.08	0.11	0.10	0.12	0.11	0.14	0.13	0.15	0.16	0.17	0.18	0.81
L24	18.88	17.97	24.57	23.49	29.1	27.71	35.25	35.07	40.06	41.13	44.88	46.24	19.43