ADDIS ABABA UNIVERSITY

INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

Investigation of Flooding Problems in Urban Drainage System: the case at Zenebe Werk in Addis Abeba, Ethiopia

By:

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A thesis submitted to the school of graduate studies of Addis Abeba University in partial fulfilment of the requirement for degree of Master of Science in Hydraulics Engineering

Advisor: Dr. Yenesew Mengiste.

June, 2017

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External examiner	Signature	Date

Declaration

I, Habtamu Alemu, declares that this thesis, which I submit to the school of Graduate studies of Addis Ababa University in Partial fulfilment of degree of Masters of Science in Hydraulics Engineering, is my original work and that it has not been presented and will not be presented, in whole or in part, by me to any other university for similar or any other degree award. Additionally, I reasonably ensure that the work is original and to the best of my knowledge and has not been taken from other sources except where such works have been cited and acknowledged within the text.

Habtamu Alemu

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June, 2017

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Acronyms

AACRA	Addis Ababa City Road Authority
AAOS	Addis Ababa Observatory Station
ARI	Average Recurrence Interval
DEM	Digital Elevation Model
ECDF	Empirical cumulative Distribution Function
ERA	Ethiopian Road Authority
GDEM	Global Digital Elevation Model
GoF	Goodness of Fit
IDF	Intensity-Duration-Frequency

Abstract

Roads are among the basic infrastructures such that drainage structures provided by the sides of roads should be well designed and constructed to the standard and managed properly such that the road serve the intended purposes. The major intention of an urban storm water management system is to guarantee storm water generated from developed catchments causes least nuisance, danger and damage to people, property and the environment. The general objective of this study is to investigate the storm drainage problem of Zenebewerk (located in West Addis Ababa, Ethiopia) and forward problem identification approach with recommendation of possible engineering remedial measures which can be applicable to similar problems in the city. The study area frequently gets flooded with small rain fall intensity and results in traffic congestion, asphalt erosion and inconvenience on the day to day socio economic activity of the society. Primary and secondary data were collected, analyzed and presented using Microsoft-excel, AutoCAD, ArcGIS, tables, graphs and percentages. The catchment that contribute flow was delineated and stream network in the catchment was generated using DEM and the delineation made more precise by field survey. Frequency analysis of rainfall computed using Gumbel and log Pearson Type III probability distribution functions and the later found to fit the actual data using goodness of fit test. Intensity duration curve developed to help analyze rain fall and the consequent peak runoff for different return periods using rational method. Dimensions of the existing drainage structures are checked as a problem identification approach for confirming capacity of the existing drainage structures. The main causes of flooding were found to be surface flows from the study catchment area are not collected and disposed properly; most of the inlets by the sides of the road closed by sediments; cross slopes of the asphalt road are not proper such that asphalt sheet flow does not escape to inlets; and big runoff from neighboring catchment forced to flow upward and discharged over the asphalt. Therefore, for the road to serve the purpose effectively, all the structures including drainage structures provided by the sides of the roads should be well designed and constructed to the standard and managed properly.

1 Introduction

1.1 Back Ground

A flood is an excess of water (or mud) on land that's normally dry and is a situation wherein the inundation is caused by high, flow, or overflow of water in an established watercourse, such as a river, stream, or drainage ditch; or ponding of water at or near the point where the rain fell. Flood is a duration type event; that is, it can strike anywhere without warning. This occurs when a large volume of rain falls within a short time. Flooding can be categorized in to two based on duration and location. According to duration: Slow-Onset Flooding, Rapid-Onset Flooding or Flash Flooding. According to location: Coastal Flooding, Arroyos Flooding, River Flooding and Urban Flooding.

The primary aim of an urban storm water management system is to ensure storm water generated from developed catchments causes minimal nuisance, danger and damage to people, property and the environment. This requires the adoption of a multiple objective approach, considering issues such as Ecosystem health, both aquatic and terrestrial; Flooding and drainage control; Public health and safety; Economic considerations; Recreational opportunities; Social considerations and Aesthetic values (Queensland Urban drainage manual, 2007)

Drainage systems are needed in developed urban areas because of the interaction between human activity and the natural water cycle. This interaction has two main forms: the abstraction of water from the natural cycle to provide a water supply for human life, and the covering of land with impermeable surfaces that divert rainwater away from the local natural system of drainage. These two types of interaction give rise to two types of water that require drainage. The first type, *wastewater*, is water that has been supplied to support life, maintain a standard of living satisfy and the needs of industry. The second type of water requiring drainage, *storm water*, is rainwater (or water resulting from any form of precipitation) that has fallen on a built-up area. (David Butler and John W. Davies, 2004). The latter is the focus of this study.

With urbanization, impermeability increases with the increase in impervious surfaces (i.e. residential houses, commercial buildings, paved roads, parking lots, etc.), drainage pattern changes, overland flow gets faster, flooding and environmental problems such as land degradation increases. It is a crucial problem facing the existing and future road infrastructure. In spite of these problems, drainage facilities in most urban centers of the country are nearly absent or at a lower coverage. (Ministry of Works and Urban Development, 2008) Urban areas and their development strongly depend on the two key tasks of urban water management: supply of high quality potable water and disposal of wastewater and storm water. These services are central for the human wellbeing as well as for the economic development of urban settlements. Storm water drainage systems are required in developed urban areas for the safety of the road and proper environmental condition.

Drainage is one of the most important factors to be considered in the road design, construction and maintenance. When a road fails, whether it is concrete, asphalt or gravel, inadequate drainage is often a major factor to be considered. Poor drainage is often the main cause of road damages and problems with long term road serviceability. Though provision of proper road surface drainage systems have such a great importance for the urban road to give the intended use and thereby contribute to the overall development of a nation, in particular in road sector, the practice of the construction of proper integrated drainage structures did not get due attention in our country. Therefore the problems and achievements on the design, construction and maintenance of surface road drainage systems need to be assessed to provide remedial measures for the better performance of the road infrastructure. Insufficient urban storm water drainage facility represent one of the most common sources of compliant from the citizens in many towns of Ethiopia and this problem is getting worse and worse with the ongoing high rate of urbanization in different parts of the country, especially in Addis Ababa. (Getachew K. W, *et al* 2011) Storm drainage design is an integral component in the design of highway and transportation networks. Drainage design for highway facilities must strive to maintain compatibility and minimize interference with existing drainage patterns, control flooding of the roadway surface for design flood events, and minimize potential environmental impacts from highway related storm water runoff. (Dabara I. D., O. *et al*, 2012). Storm water collection systems must be designed to provide adequate surface drainage. Traffic safety is intimately related to surface drainage. Surface drainage is a function of transverse and longitudinal pavement slope, pavement roughness, inlet spacing, and inlet capacity (Dabara I. D. O, *et al*, 2012)

Service life of infrastructure can be highly reduced by improper drainage system. It can be seen from rainy season which lasts from July to September where the highways are covered by surface water. This water accelerates the deterioration rate of the roads and results in economic loss. The flooding of the highways is the result of improper drainage system of the roads and poor integration of road and urban storm water drainage network (Ewnetu, 2013).

This research is necessary for providing solution for the flooding problem of the study area as it has been a problem for the nearby community since the inauguration of the ring road project.

1.2 Statement of the Problem

In view of the aforementioned scientific perspectives and eye view assessment, along the ring road, in many points of the roundabout, there is a common flooding problem which creates a sever traffic congestion especially in rainy season. The flooding do also affect the asphalt pavement by eroding & creating a number of potholes and this will result in longer period impact even after a rain. Besides, the traffic congestion & car accidents, it hinders the movement of pedestrians as the flood flows over the walk way. The impact is too much on the socio – economic activity of the area for it affects their day to day deeds. Zenebe Werk, the study area, is one of the sites having this problem in the city of Addis Ababa. Addis Ababa City Road Authority (AACRA) is the responsible body for managing, repairing and maintaining the roads in the city. The authority is investing a lot of money for maintaining pothole damages caused by flooding and cleaning blocked drainages. However, the problem remains the same every year.

In general, the problem that one can observe at this study area is very serious and this problem occurred since the inauguration of the ring road, fourteen years ago. It is an important source of complaint of the city people, especially the nearby community. Hence, this study investigated the causes of flooding and recommended possible engineering solution.

The following pictures which are taken on August 7, 2016 shows the extent of the problem.

Figure 1: Left- Storm Water escaping into a river through an illegally widened inlet at the side of bridge. Right: Both main asphalt road and walk way flooded

In the left figure, narrow inlet by the side of the bridge is widened illegally in order to release more water to the river. However, this damaged the structural element of the bridge which has negative impact on life span of the bridge. On the figure at the right, one can see that the walkway at the bridge is flooded. This is a common phenomenon for the nearby community

Figure 2: Left: A lady moving across the roundabout flooded with water. Right: A young man arranging a stone as Foot Bridge to pass over the storm flow, for sprinting purpose.

It is very common to see several boulder stones on the asphalt once the flood drained. These stones are being causing vehicle transportation difficult. On the other hand, especially ladies are leaving their shoes behind and losing it while moving across the flood. This is again common phenomenon for the community when rain comes.

Figure 3: Left: A young man jumping to escape from flood flowing over roundabout using stone as a tool to sprint. Right: Mini-bus going across the flood at the roundabout.

Most of the time ladies wait until the flood depth decreases for they are afraid of jumping using the boulder stones. But, young boys use the stone to cross the flood but still they get socked below knee. Some drivers dare to cross the flood and remain there for long time as the car would come across technical problem.

Figure 4: Left: A bus partially sunk in the flood at the bridge. Right: Pick-up car trying to escape out from the flood at the bridge

It is very common phenomena to see long rows of vehicles when it rains. This may remain for hours based on the duration and intensity of rainfall. Most of the time cars get technical problem while moving inside the flood and this aggravates the traffic congestion. Only one car moves at a time when crossing the flood otherwise the following car would get difficulty to cross. This again worsens the traffic jam.

1.3 Research Questions

i. What are basic problems of drainage systems of the study area?
ii. Was the intensity of rain fall and the consequent runoff properly analyzed?
iii. Was special consideration paid at the sag point of the road?

1.4 Objective of the Study

1.4.1 General Objective

The study is bounded to identification of causes of flooding of the study area by storm water and recommendation of possible engineering solution to the problem.

1.4.2 Specific Objectives

The specific objectives of the study are:

- Estimate runoff generating from the catchment area.
- Identify the real cause of flooding in the study area.
- Develop possible remedial engineering solution for the flooding problem.

1.5 Scope of the Study

This study is geographically limited to Zenebe Werk area- Kolfe Keranio Sub city, Addis Ababa, Ethiopia. The study is bounded to identification of causes of flooding of the study area by storm water and recommendation of possible engineering solution to the problem. The study does delineate catchments contributing flow, frequency analysis of forty year daily rain fall data, produce IDF curve and estimates peak discharges for different recurrence intervals using rational method. The Engineering design aspect of roads as well as storm water drainage networks in the catchment are not part of this study.

1.6 Thesis Outline

The thesis is made up of five chapters. The first chapter introduces the thesis. The second chapter presents knowledge from literatures that are reviewed in a way that explains the theoretical background and the gaps in other works. The third chapter is all about the

Methodology. The study area, data collection, methods used and hydrological analysis are included in this chapter. Hydrological results and discussions are presented in the fourth chapter. The last chapter, the fifth chapter, presents conclusion, remedial measures and recommendation based on the results obtained.

2 Literature Review

2.1 Storm Drainage Conditions of Addis Ababa

Proper storm water drainage system problem occurs in the city, Addis Ababa, especially at Gotera- Welo Sefer road, Saris- Gotera road and on the ring road (Dessalegn, 2011) and in Addis Ketema Sub-city (Dagnachew, 2011).This literatures reveal that no doubt on the existence of drainage problem in the city, especially on areas where the studies referring to.

Congested traffic due to flooding of roads after small depth of rainfall, erosion of pavements resulting in reduction of service life of road infrastructure and impact of road flooding on nearby community are consequences of poor drainage system in the area.

These problems would be solved if good design, construction and maintenance of drainage infrastructures were practiced. In addition, smaller inlet spacing, higher inlet efficiency and frequent maintenance would alleviate the flood problem totally (Dessalegn, 2011)

The pattern of urbanization and modernization in Ethiopia has meant increase densification along with urban infrastructure development. The combined effect of this results in higher rain drop intensity and consequently accelerated and concentrated runoff. Inadequate integration between road and urban storm water drainage infrastructure provision and poor management, significant proportion of the study area is exposed to flooding hazards/risks. This has resulted in negative impacts on urban storm water drainage provision and management. The major causes of flooding was found to be the blockage of urban storm water drainage lines along with inadequate/poor integration between road and urban storm water drainage infrastructures. The paper recommended improvement in the integration of road and urban storm water drainage infrastructure and integrated solid waste management to prevent over flowing of flood as a result of blockage of drains (Dagnachew, 2011).

2.2 History of Urban Drainage Development

Natural hydrological processes would have prevailed; there might have been floods in extreme conditions, but these would not have been made worse by human alteration of the surface of the ground. Artificial drainage systems were developed as soon as humans attempted to control their environment. Archaeological evidence reveals that drainage was provided to the buildings of many ancient civilizations such as the Mesopotamians, the Minoans (Crete) and the Greeks (Athens). Historical accounts show that the objectives of the drainage systems were to collect rainwater, prevent flooding, and convey wastes. At the time, planning and design were limited, people used to able to find the systems that met their objectives after trial-and-error modifications. Despite the lack of optimization and the use of trial-and-error construction methods, numerous ancient urban drainage systems can be rated very successful.

Urban drainage was firmly established as a vital public works system in the early parts of the twentieth century. Engineers continued to improve design concepts and methods. During the second half of the twentieth century regulatory elements were spread in the United States, Europe, and other locations addressing urban drainage issues. Computer modeling tools advanced the methods used to design and analyze urban drainage systems. Regulations, monitoring, computer modeling, and environmental concerns have altered the perspective of urban drainage from a public health and nuisance flooding concern during the first half of the twentieth century into a public health and nuisance flooding with additional concerns for ecosystem protection and urban sustainability (S.K. Garg,2005)

Communities worldwide are yet searching for innovative techniques to capture, detain, and use rainwater within the watershed instead of constructing massive drainage structures. Many communities are developing watershed-wide storm water quality management plans to meet the dual objectives of flood prevention and water quality control. Urban drainage has indeed expanded significantly during the past few decades beyond a technical challenge to drain the urban area rapidly to include the consideration of social, economic, political, environmental, and regulatory factors (David Butler and John W. Davies, 2004).

2.3 Sustainability in Urban Planning

According to the United Nations, the world's population has recently become predominantly urban, and the total world urban population is projected to double by 2050. Because of the urbanization trend, the U.N. Habitat has reasoned that "the millennium ecosystem goals will be won or lost in cities!" (UN HABITAT, 2013). This reality has made different disciplines such as civil engineering, urban planning and architecture to focus on developing and testing new theories, strategies, and best practices to enhance the sustainability of cities. To tackle economic, social and environmental sustainability in an integrated way experts from design and planning disciplines are demanded to engage in interdisciplinary practice with economists, social scientists, biologists and other engineers. Beyond sustainability research and practice also engages the stakeholders and decision makers, in a genuine and meaningful manner, throughout a continuous, interactive, and iterative process of urban planning and design (Tress *et al*., 2005)

Unlike the historical drainage systems that would aim to collect and dispose of water as quickly as possible which have an adverse effect on the receiving infrastructure nowadays an alternative approach is being utilized. This alternative approach aims to reduce the volume of water discharged to the receiving body of water. It will do so by encouraging the infiltration of water into the ground, and by providing water storage within the drainage system to attenuate the peak discharge. Hence downstream receiving infrastructure do not have to cope with a greater volume of flow and a higher peak discharge any more. A number of techniques are available for achieving this which are collectively known as Sustainable Drainage Systems (Ahern, 1995)

Water is essential for all life because it is the universal solvent that transports and redistributes nutrients and pollutants across entire watersheds. But urbanization itself threatens future water quality and management for urban uses in most cities of the world. Therefore, sustainability of water resources is the central challenge for sustainable urbanism

Urban planning and design can play a key role to preserve, protect, restore and reuse the full spectrum of water uses that cities depend on, including: drinking water provision; wastewater collection, treatment, disposal, and reuse; storm water management; and innovative, more holistic systems to create a new urban hydrological cycle (Natural Resources Defense Council, 2014).

To achieve sustainability and resilience in cities, urban infrastructure must be reconceived and understood as a means to improve and contribute to sustainability. Arguably, achieving sustainability will depend on significant innovations. In the 21st century, much of the infrastructure of the developed world will be replaced or rebuilt, and even more infrastructure will be needed to service the rapidly expanding cities of the developing world (Nelson, 2004). This can be viewed as an opportunity, the magnitude of global infrastructure development represents an extraordinary opportunity to redirect and reconceive the process of urbanization from one that is inherently destructive to one that is sustainable in specific terms.

Innovative urban planning and design is possible where ambitious goals for multiple aspects of sustainability can be adopted to guide and focus the design process, including: a sustainable urban hydrology model, zero net energy use, a mix of urban uses, inclusion of biodiversity, and providing a healthy environment for people. The planning and design disciplines have developed a significant body of knowledge with respect to sustainable water resources, and more recently have begun to use cities as laboratories to test new practices and thereby promote innovation (M.H Raghunath,2008)

Urban planning is inherently a strategic process because, rather than employing tactics to respond to urban changes it attempts to understand and proactively manage the elements and forces that causes change. By definition planning is proactive, but not all planning is strategic. For urban planning to be strategic, it requires integration of interdisciplinary knowledge to define strategic goals consistent with political expectations, economic factors, and the reality of the existing landscape condition. Strategic urban planning requires a particular blending and integration of knowledge, vision, creativity, and political skills. (Ahern, 1995).

2.4 How Urbanization Affects the Water Cycle

The water cycle, also known as the hydrological cycle, is the continuous exchange of water between land, water bodies, and the atmosphere. The world's total water resource estimated as 1.36 x 10^8 M ha-m. Of these global water resources, about 97.2% is salt water mainly in oceans and only 2.8% is available as fresh water at any time on the planet earth (H.M Raghunath, 2008). When precipitation falls over the land, it follows various routes. Some of it evaporates, returning to the atmosphere, some seeps into the ground, and the remainder becomes surface water, traveling to oceans and lakes by way of rivers and streams.

Impervious surfaces associated with urbanization alter the natural amount of water that takes each route as topography of land changes by urbanization. The consequences of this change are a decrease in the volume of water that percolates into the ground, and a resulting increase in volume and decrease in quality of surface water. These hydrological changes have significant implications for the quantity of fresh, clean water that is available for use by humans, fish and wildlife.

With natural groundcover, 25% of rain infiltrates into the aquifer and only 10% ends up as runoff (H.M Raghunath, 2008). .As imperviousness increases, less water infiltrates and more and more runs off. In highly urbanized areas, over one-half of all rain becomes surface runoff, and deep infiltration is only a fraction of what it was naturally .The increased surface runoff requires more infrastructure to minimize flooding. Natural waterways end up being used as drainage channels, and are frequently lined with rocks or concrete to move water more quickly and prevent erosion. In addition, as deep infiltration decreases, the water table drops, reducing groundwater for wetlands, riparian vegetation, wells, and other uses. Consequently, as the land cover and land use changes during urbanization so does the water cycle. Scientific measures need to be considered in order to sustain the water cycle while urbanization is taking place (H.M Raghunath, 2008).

2.5 Components of a Drainage System

A complete storm drainage system design includes consideration of both major and minor drainage systems. The minor system consists of curbs, gutters, ditches, inlets, access holes, pipes and other conduits, open channels, pumps, detention basins, water quality control facilities, etc. The minor system is normally designed to carry runoff from 10 year frequency storm events.

The major system provides overland relief for storm water flows exceeding the capacity of the minor system. This usually occurs during more infrequent storm events, such as the 25-, 50-, and 100-year storms. The major system is composed of pathways that are provided – knowingly or unknowingly -for the runoff to flow to natural or manmade receiving channels such as streams, creeks, or rivers.

Usually, storm drainage design efforts have focused on components of the minor system with little attention being paid to the major system. Although the more significant design effort is still focused on the minor system, lack of attention to the supplementary functioning of the major storm drainage system is no longer acceptable (urban Drainage Design Manual, 2001)

2.6 Storm Water Collection

Storm water collection is a function of the minor storm drainage system which is accommodated through the use of roadside and median ditches, gutters, and drainage inlets. Roadside and median ditches are used to intercept runoff and carry it to an adequate storm drain. These ditches should have adequate capacity for the design runoff and should be located and shaped in a manner that does not present a traffic hazard. If necessary, channel linings should be provided to control erosion in ditches. Where design velocities will permit, vegetative linings should be used (Thomas N, 2003).Gutters are used to intercept pavement runoff and carry it along the roadway shoulder to an adequate storm drain inlet. Curbs are typically installed in combination with gutters where runoff from the pavement surface would erode fill slopes and/or where right-of-way requirements or topographic conditions will not permit the development of roadside ditches. Pavement sections are typically curbed in urban settings. Parabolic gutters without curbs are used in some areas.

Inlets are the receptors for surface water collected in ditches and gutters, and serve as the mechanism whereby surface water enters storm drains. When located along the shoulder of the roadway, storm drain inlets are sized and located to limit the spread of surface water on to travel lanes. Drainage inlet locations are often established by the roadway geometries as well as by the intent to reduce the spread of water onto the roadway surface. Generally, inlets are placed at low points in the gutter grade, intersections, crosswalks, cross-slope reversals, and on side streets to prevent the water from flowing onto the main road (Ahern, 1995).

2.7 Storm Water Conveyance

Storm drains are that portion of the storm drainage system that receive runoff from inlets and conveys the runoff to some point where it is discharged into a channel, water body, or other piped system. Storm drains can be closed conduit or open channel. They may consist of one or more pipes or conveyance channels connecting two or more inlets. Access holes, junction boxes and inlets serve as access structures and alignment control points in storm drainage systems. Critical design parameters related to these structures include access structure spacing and storm drain deflection. Spacing limits are often dictated by maintenance activities. In addition, these structures should be located at the intersections of two or more storm drains, when there is a change in the pipe size, and at changes in alignment (horizontal or vertical). In areas where gravity drainage is impossible or not economically justifiable, storm water pump stations are often required to drain depressed sections of roadways. Detention/ retention facilities are used to control the quantity of runoff discharged to receiving waters. A reduction in runoff quantity can be achieved by the storage of runoff in detention/retention basins, storm drainage pipes, swales and channels, or other storage facilities.

Outlet controls on these facilities are used to reduce the rate of storm water discharge (David Butler and John W. Davies, 2004).This concept should be considered for use in highway drainage design where existing downstream receiving channels are inadequate to handle peak flow rates from the highway project, where highway development would contribute to increased peak flow rates and aggravate downstream flooding problems, or as a technique to reduce the size and associated cost of outfalls from highway storm drainage facilities.

2.8 Causes of flooding

A flood is an excess of water (or mud) on land that's normally dry and is a situation wherein the inundation is caused by high flow, or overflow of water in an established watercourse, such as a river, stream, or drainage ditch; or ponding of water at or near the point where the rain falls. Flooding is a duration type event. A flood can strike anywhere without warning, occurs when a large volume of rain falls within a short time. In general flooding are categorized in to two: according to duration (as Slow-onset, Rapid-onset and flash flooding) and according to location (Coastal Flooding, Arroyos Flooding, River Flooding and Urban Flooding). The urban area is paved with roads, houses etc and the discharge of heavy rain can't absorbed into the ground due to drainage constraints leads to flooding of streets, underpasses, low lying areas and storm drains .Causes of urban flooding are either natural or human. The natural causes are heavy rainfall or flash floods, lack of lakes and silting. On the other hand, human causes are:

Population pressure: Because of large amount of people, more materials are needed, like wood, land, food, etc. This aggravates overgrazing, over cultivation and soil erosion which increases the risk of flooding.

Deforestation: Large areas of forests near the rivers/catchment of cities are used to make rooms for settlements, roads and farmlands and is being cleared due to which soil is quickly lost to drains. This raises the drain bed causing overflow and in turn urban flooding.

Urbanization: leads to paving of surfaces which decreases ground absorption and increases the speed and amount of surface flow. The water rushes down suddenly into the streams from their catchment areas leading to a sudden rise in water level and flash floods. Unplanned urbanization is the key cause of urban flooding. Various kinds of depression and low lying areas near or around the cities which were act as cushions and flood absorbers are gradually filled up and built upon due to urbanization pressure. This results in inadequate channel capacity causing urban flooding.

Poor Water and Sewerage Management: Old drainage and sewerage system has not been repaired nor it is adequate now (Er. Pareva, 2005)

Urban flooding is specific in the fact that the cause is a lack of drainage in an urban area. A lot of the sewerage and drainage network would be old and its condition may be unknown. They cannot cope with the volume of water or are blocked by rubbish and by non-biodegradable plastic bags. Sewers overflow because of illegal connections and the sewer system cannot cope with the increased volumes. As new developments cover previously permeable ground, the amount of rainwater running off the surface into drains and sewers increases dramatically. Developments encroach floodplains, obstructing floodways and causing loss of natural flood storage. Continued development and redevelopment to higher density land uses by high land costs. The proportion of impermeable ground in existing developments is increasing as people build patios and pave over front gardens. Increased impervious areas such as roads, roofs and paving, due to increasing development densities means more run-offs. Some of the major hydrological effects of urbanization are:

(1) Increased water demand, often exceeding the available natural resources;

(2) Increased wastewater, burdening rivers and lakes and endangering the ecology;

(3) Increased peak flow;

(4) Reduced infiltration and

(5) Reduced groundwater recharge, increased use of groundwater, and diminishing base flow of streams. According to natural hydrological phenomena, due to increased impervious area precipitation responds quickly reducing the time to peak and producing higher peak flows in the drainage channels (Debu Mukherja. July 2016).

3 Methodology

3.1 Study Area

Addis Ababa was established in 1886 by Emperor Minilik II. Addis Ababa is one of the oldest and largest cities in Africa, being the capital of a non-colonized country in Africa. Addis Ababa has been playing a historic role in hosting the regional organizations such as the Organization of African Unity / African Union, and the Economic Commission for Africa, which contributed to the decolonization of African countries, and later bringing Africa together.

Addis Ababa is one of the rapidly urbanizing cities of the world. Its geographic location is in the center of Ethiopia, combined with lack of development policies in other urban centers have given the capital the majority of social and economic infrastructure in the country. As a result, it has been a melting pot to hundreds of thousands of people, coming from all corners of the country in search of better employment opportunities and services. Because of its rapid population growth, Addis Ababa, one of the fast growing cites in Africa, is facing critical challenges. Among the challenges of the city are high rate of unemployment as well as housing and transportation shortage (UN-HABITAT- Addis Ababa Urban Profile, 2008).

Poor urban storm drainage systems are typical problem of the city and one would observe this problem along the roads of the city even when there is light rain fall intensity. The ring road in Addis Ababa is part of the cities urbanization process that is intended to solve the transportation problem of the capital.

3.1.1 Location

Addis Ababa lies 9°1′ 48″ N latitude and 38°44′ 24″ E longitude. The city is located at the heart of the country, at an altitude ranging from 2,100 meters at Akaki in the south to 3,000 meters at Entoto Hill in the North. This makes Addis Ababa the third highest city in the world, after La Paz and Quito in Latin America. The city occupies a total area of 540 Sq.Km. (Source: Socio-Economic Profile of Addis Ababa, 2013)

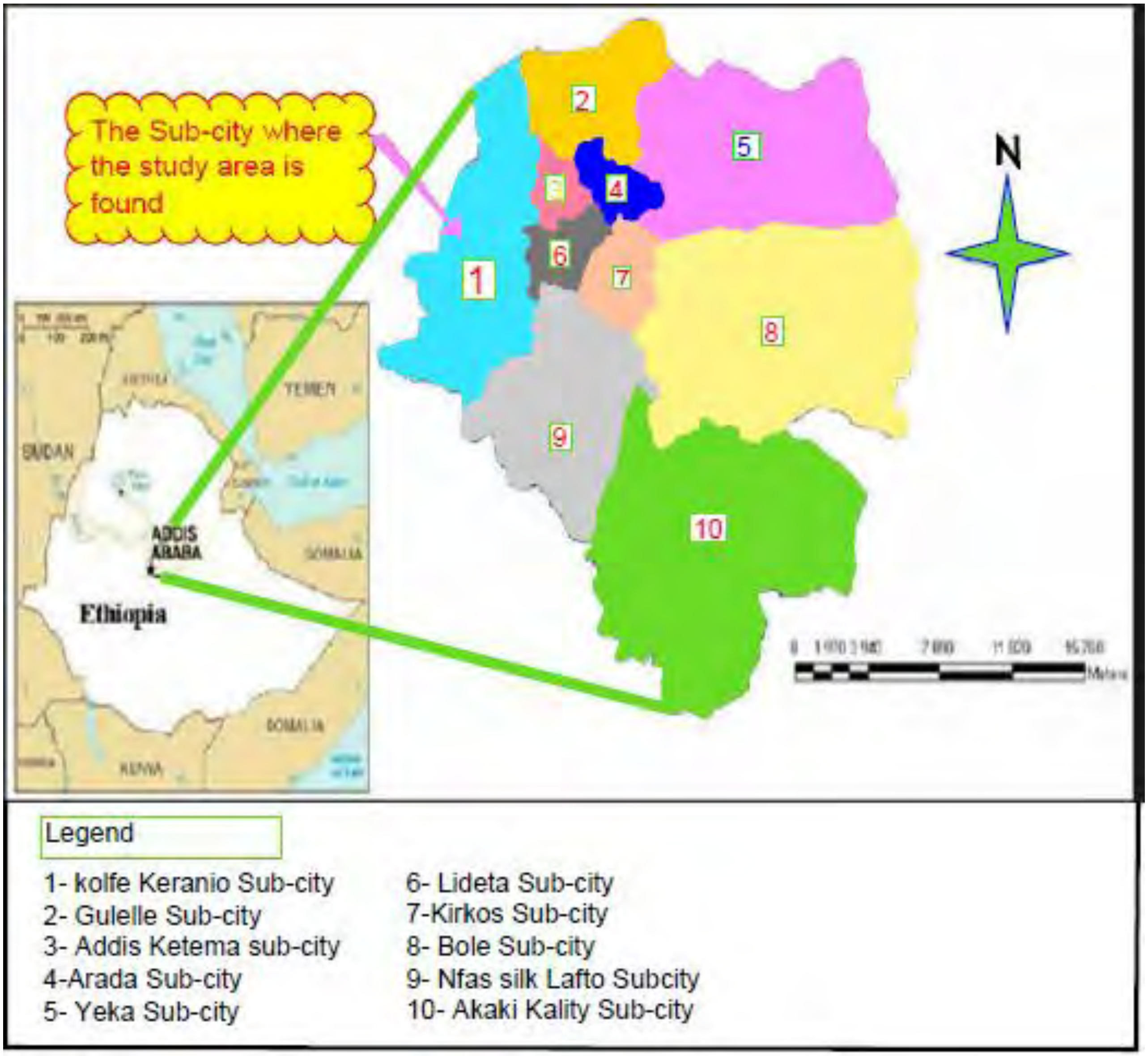


Figure 5: Map of Addis Ababa showing all the ten Sub-cities

The study area, Zenebe Werk Roundabout, is specifically located in the Kolfe Keranyo sub-city which is named as 1 in the figure above. It is located in the West of Addis Abeba. The study area is part of the ring road. Most of the ring road parts have storm water flooding problems- such as Zenebewerk round about, Emperial round about, Road along Lebu roundabout to Germen roundabout and Atana Tera roundabout. As the study area is one of the locations (on the ring road) being affected by flooding, its location on the ring road is depicted in the following figure.

3.1.2 Climate

Because Addis Ababa is located around the equator its temperature stays nearly constant month to month with no more than $10°C$ change and a temperate climate due to its high-altitude location in the subtropics. The average minimum and maximum temperature of each month as well as average monthly rainfall depth are presented in the following graphs.

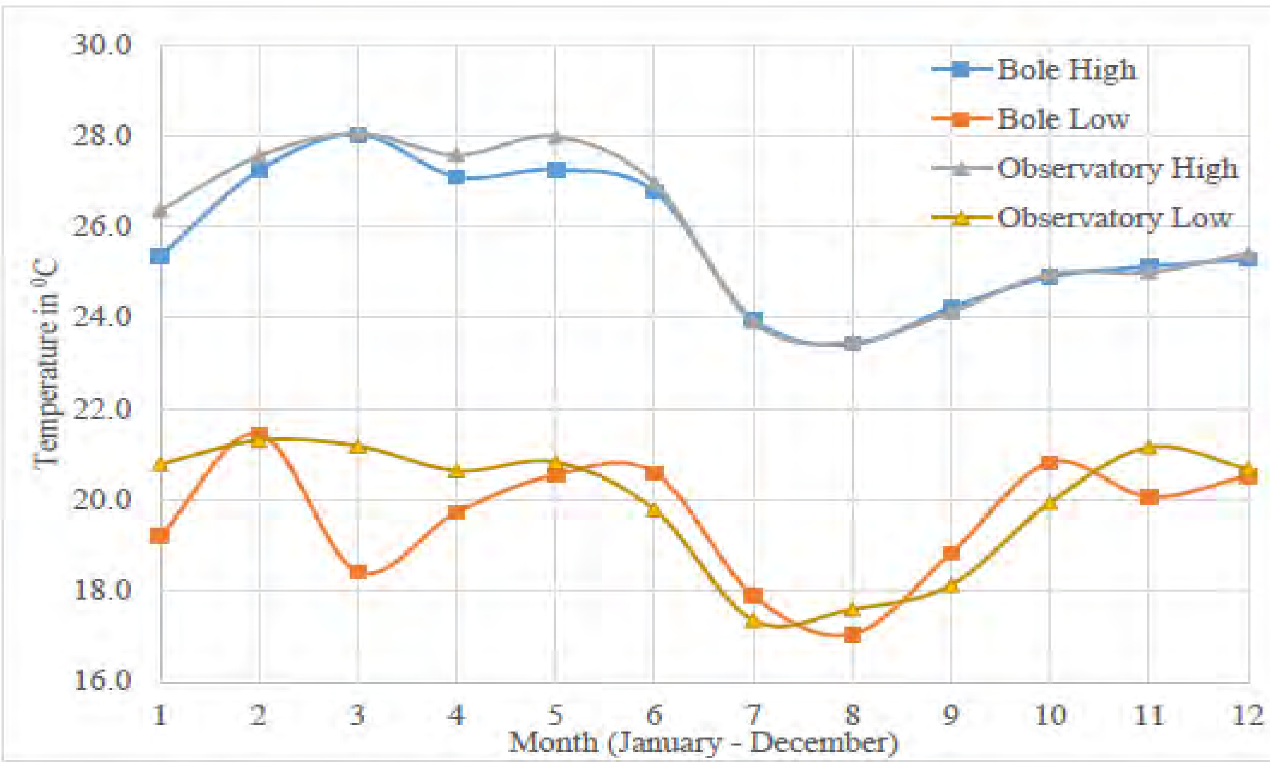


Figure 6: Average High and Low Temperature of Bole and Observatory Gauging Stations (Anteneh, 2015)

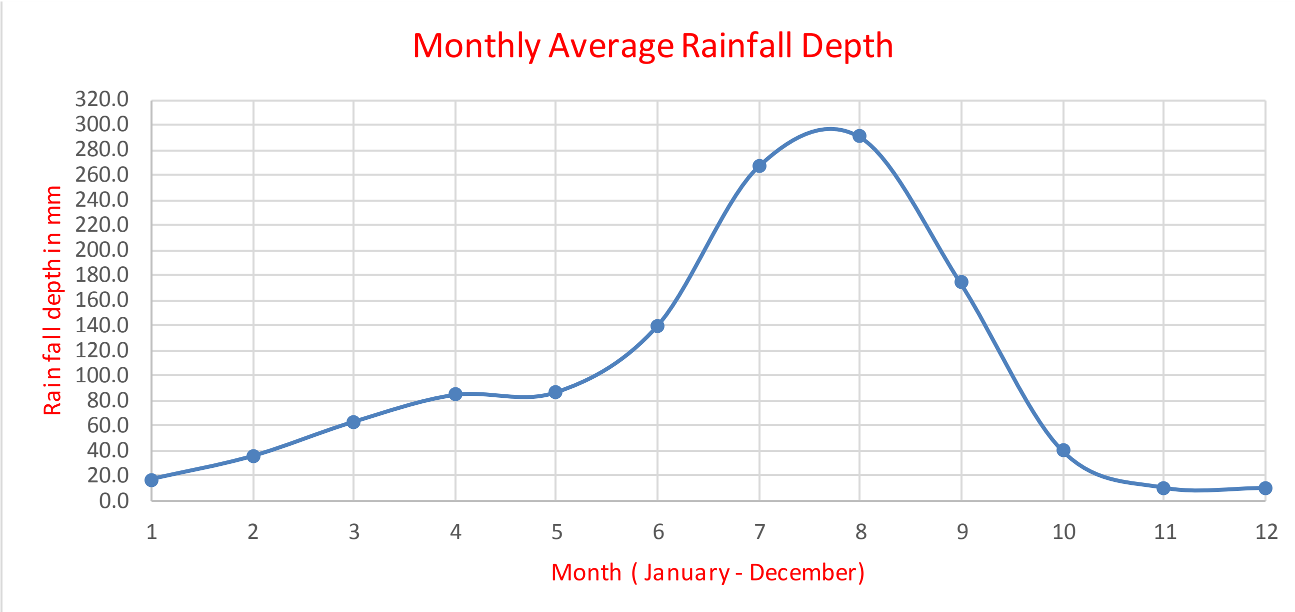


Figure 7: Average monthly Rain Fall Depth of Observatory Stations (1976 to 2015)

Addis Ababa has a prominent rainfall peak during the summer season which is from June to September. It also exhibits a substantial amount of rainfall during February to May. The average monthly rainfall graph is produced using a forty year (from1976 to 2015) daily rainfall data from Addis Ababa Observatory station. This rainfall data is utilized for rainfall frequency analysis in this study. As it can be seen from the graphs above, the highest temperature is measured in March (about 28℃) where as the lowest temperature is measured in August (about 17℃). Similar, highest rainfall is measured in August (About 290mm) and the lowest rain fall is measured in December (about 10mm).

3.1.3 Demography

Addis Ababa, the capital of Ethiopia, is the industrial, commercial and cultural center of the country. Being the seat of various regional and international institutions, it is an important political and diplomatic hub of Africa. With a population of more than 2.7 million (CSA 2008), Addis Ababa is the largest city in Ethiopia. According to the 2007 population and housing census of the Central Statistical Agency (CSA), the city accounts for 30% of the nation's urban population and is ethnically diverse. 52.4% of the population is female, a slightly higher rate than the national ratio (51%). Addis Ababa also has the highest number of female headed households (38.5%) in Ethiopia (Addis Ababa City Public Expenditure Review, 2010)

3.1.4 Land Use and Land Cover

Addis Ababa have land use ranges from agricultural to high density commercial areas. Information about changes in land use/cover provides valuable insights while planning future natural resource management strategies. Remotely sensed data, serve as an effective tool for deriving this kind of information. Landsat Thematic Mapper images of 1987 and 1999 were used to extract land use/cover change of the city of Addis Ababa and the surrounding area. Analysis of the multitemporal Landsat images has clearly revealed the loss of forest to urban and residential sprawl within the city limit and the surrounding area (Anteneh 2015). The implication of land use land cover change to paved land/ surface is that the storm runoff as well as the rainfall intensity increases and as a result the city requires well integrated drainage system. Urbanization process increases the amount of runoff decreasing the percolation of rain water in to the ground thereby the water cycle would be affected resulting in high rainfall intensity. The following figures illustrate that Addis Ababa is in a high rate of urbanization such that a well-integrated drainage system must be implemented in order to control the consequent high runoff.

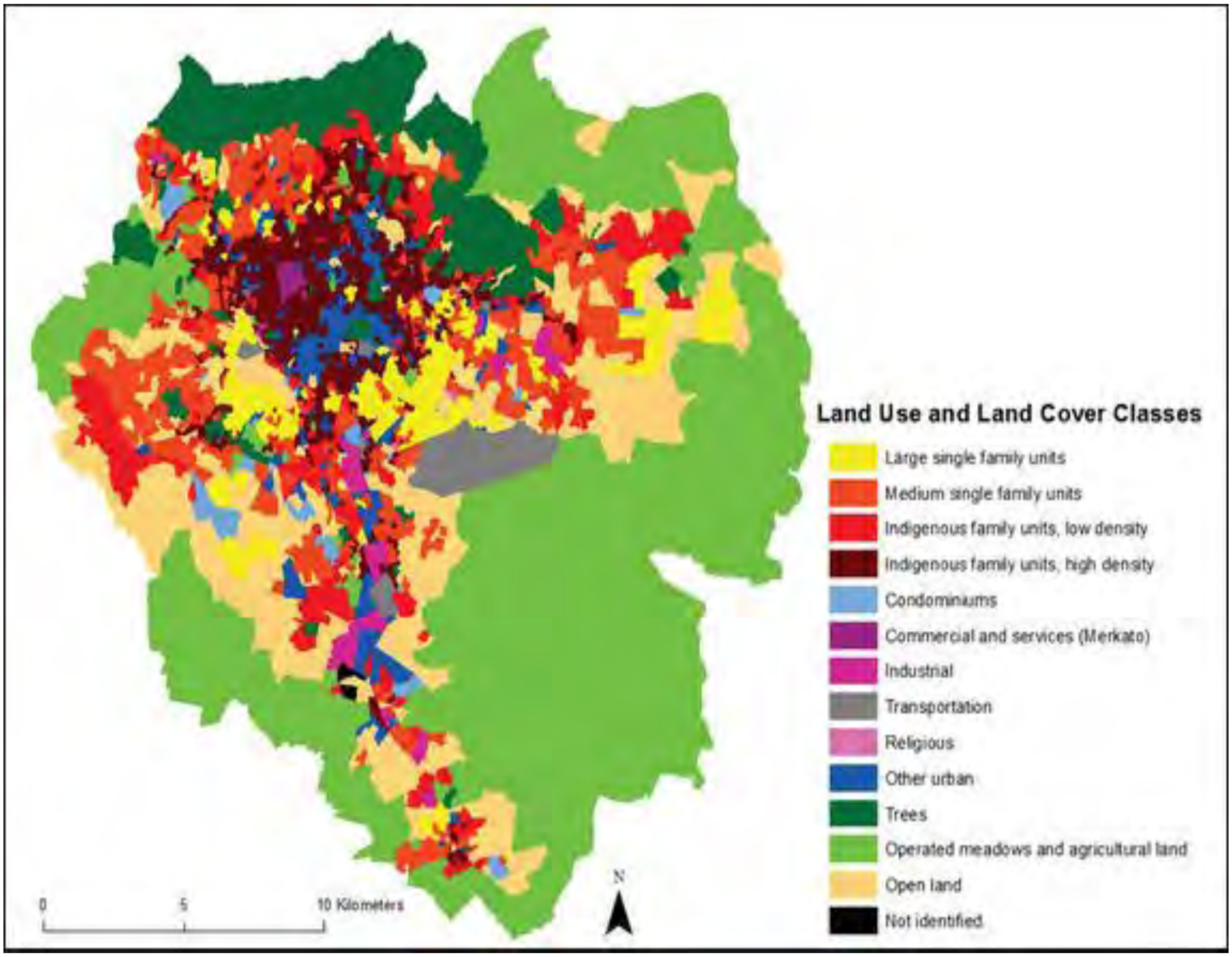


Figure 8 : Land use land cover classification of Addis Ababa

(Source: http://www.ee.co.za, 2010)

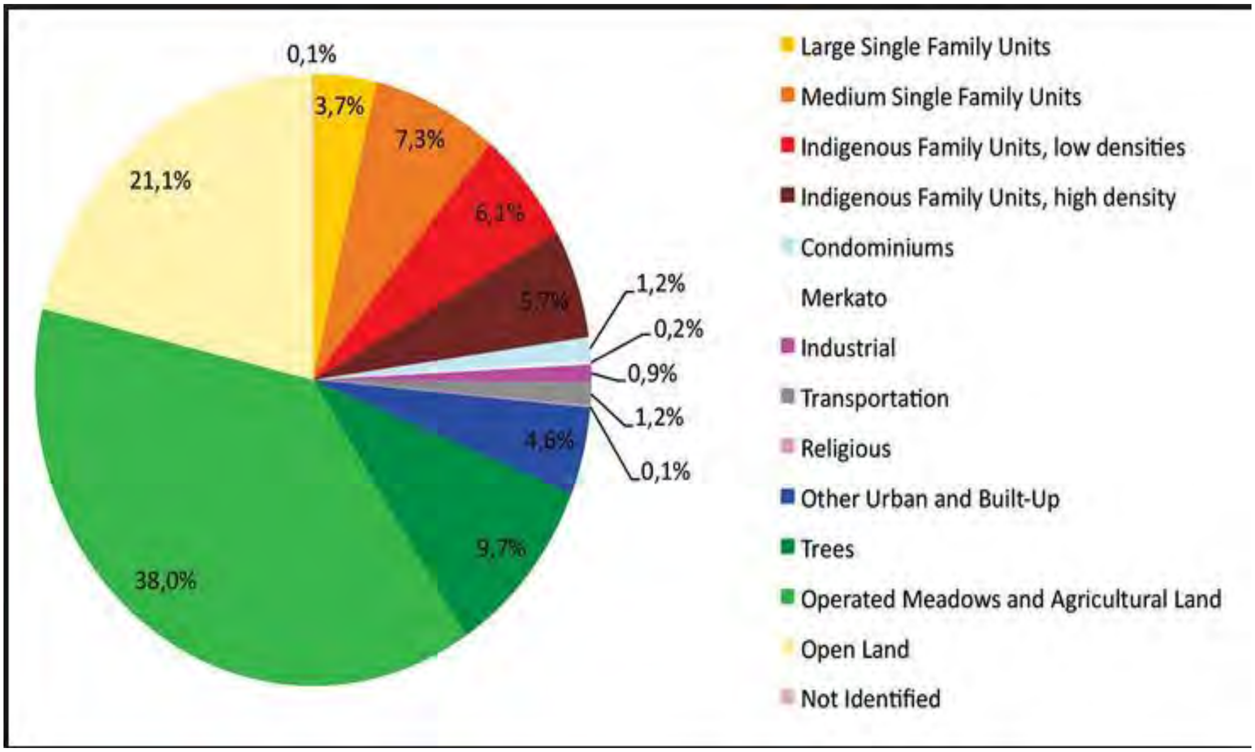


Figure 9: Land use and land cover composition of Addis Ababa

(Source: http://www.ee.co.za, 2010)

Following the land use and land cover dynamics analysis, theFfigure-10 below consists of the 1986, 2000 and 2010 classified land use and land cover maps of Addis Ababa city. The darker color indicates built-up areas and the darkness decreases consecutively for forest cover, grasslands, and croplands. If we see the 1986 land use and land cover map of the city, there was forest dominated cover land in the Northern and Northwestern part while crop and grasslands dominated the Southern, Southeastern, and Northeastern part. The built-up area was only concentrated on the central part. The 2000 and 2010 land use and land cover maps clearly show the expansion of built-up areas and shrinkage of forest lands.

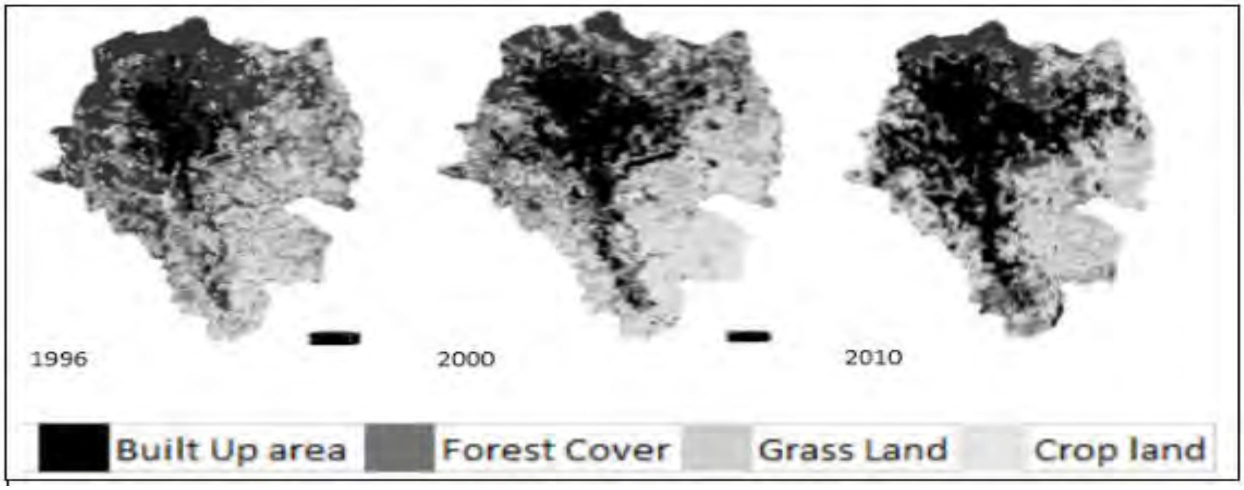


Figure 10*:* land use/Land cover maps of Addis Ababa city in 1986, 2000 and 2010.

(http://www.efdinitiative.org/sites/default/files/leulsegged_kasa_paper_presented_for_10th_international_conference_on_Ethiopian_economy.pdf)

3.1.5 Location of Flooding Area

The study area is located in the kolfe Keranio Sub-city and is part of the ring road which is shown in the Figure-11 below. This flooding area, Zenebe Werk, is located in the Google earth map below showing the stretch of four routes emerging from the roundabout.

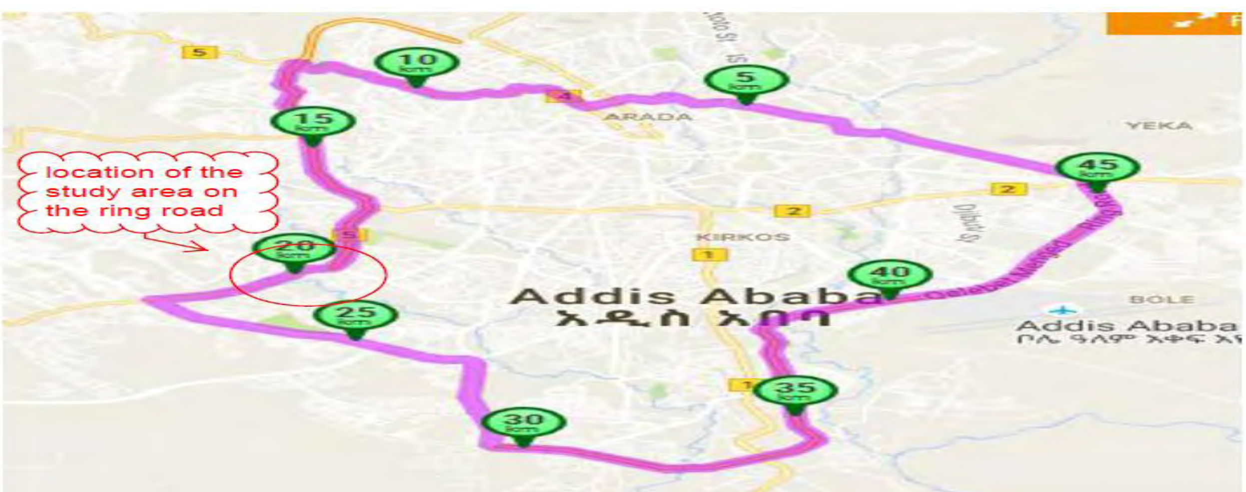


Figure11: Map of Ring Road showing the study area

(Source: www.bikely.com/maps/bike- path/Addis-Ababa-Ring-Road, 2009)

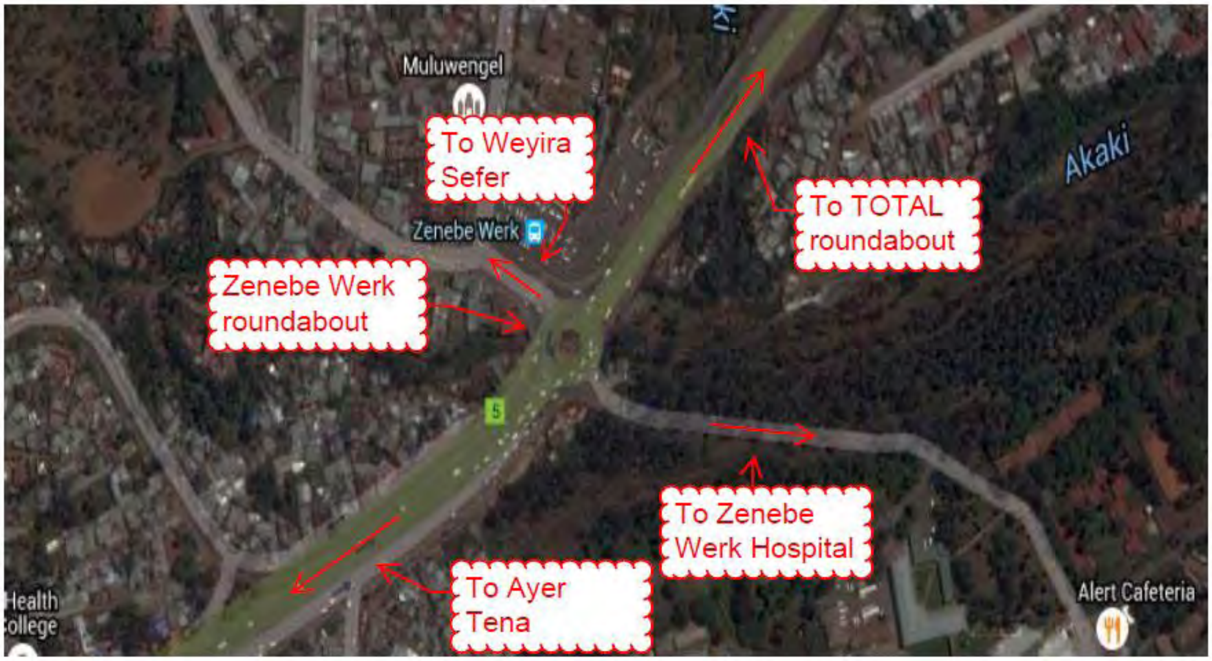


Figure 12: Google Earth map showing the study area

3.1.6 Selection of Study Focus Areas

AACRA (Addis Ababa city Road Authority) provided me with thirteen selected road flood problem areas in the city as per my application. Out of these sites, I selected Zenebe Werk area for the problem is more serious compared to the other. The main flood problem areas in Addis Ababa as depicted by the Authority are Zenebewerk round about, Jemmo-2, Mebrat hail (Lebu roundabout), Welo sefer, Beherawi, Piassa, Alem Bank, along the new Light Rail way. (From Ayat to Tor Hailoch and From Giorgis to Kality), Goro, Sumit, Merkato, Asko, Emperial round about.

3.2 Data Collections

3.2.1 Rain Fall Data

Rainfall data for Addis Ababa city can be obtained from two important observatory stations: Addis Ababa Observatory station and Bole International Airport observatory Station. There are other rainfall gaging stations in the city such as Ayer Tena, Kolfe keraniyo School, Yekatit 23 school, Abyssinia School, Medhaniyalem School, Kality, Akaki,Asko, Kotebe TTC and Entoto. Some of these stations are aged not more than ten years and the other stations are actually old but with a lot of missing data therefore the data are not complete enough to use for frequency analysis. Those with ten years old recording history have a lot of missing data. Consequently, this study used rainfall data from the Addis Ababa observatory station for it has forty years (from 1976 to 2015) of daily rain fall depth record as well as the station is very close to the study area. Therefore the data at this station is used to develop intensity-duration-curve (IDF) for rain fall analysis.

The rainfall data is used to develop IDF curves so that rational method can be used to estimate peak flow. For a specific time of concentration, duration of rainfall is assumed the same and its intensity is estimated from the developed IDF curve. Consequently, this study develop IDF curve for estimation of runoff using rational method. Moreover, compares this IDF curve with the one given in the drainage design manual by Addis Ababa City Road Authority (AACRA).

Rain fall of a place can be completely defined if the intensities, durations and frequencies of the various storms occurring at that place is known. Whenever, an intense rain occurs, its magnitude and duration is generally known from the meteorological readings. Thus, at a given station, the magnitude of the isolated rains of various durations are generally known. This available data can be used to determine frequencies of various rains.

3.2.2 Elevation Data

Since the analysis that are to be done are primarily elevation based, topography of the study area is important. Accuracies of *elevation maps* may vary greatly. More precise outcomes are obtained by project specific surveying. Project specific surveying may include satellite imagery or field surveying. Because elevation of the whole catchment is required for this project area, obtaining project specific elevation map is not possible. Rather global elevation maps from satellite imagery are readily available in a raster and digital elevation format. There are different providers of digital elevation models both open source and commercial.

Advanced Space-borne Thermal Emission and Reflection Radiometer (ASTER) Global Digital Elevation Model (GDEM) is one kind of elevation data. ASTER GDEM was developed jointly by Ministry of Economy, Trade and Industry of Japan and the United States National Aeronautics and Space Administration (NASA). Second version of the elevation data with 30 by 30 meter resolution released in 2011 is openly available on different web sites including https://earthdata.nasa.gov. It is the source of elevation data for this thesis. However, as the study area is small, the data obtained from the website was revised and cross checked by field survey using GPS and levelling machine. There is prominent difference but the model helped to outline the general outline of the topography.

3.2.3 Land Use Land Cover Data

The other type of data required is the land use land cover of the study area. The land use land cover data will be an essential input for the calculation of run off coefficient in the determination runoff rate using rational method. The runoff coefficient of a surface is dependent on the land use and land cover of the area. For the study area is relatively small, categorization of different land use was done by field survey and with help of Google earth map. The land use land cover for the catchment area grouped in to Mixed Residences, Government compounds, colleges, Asphalt roads, Cobble stone roads, Commercial Areas, school, Health Center and play grounds. As there is no available data to name an area as commercial or mixed residence, the categorization is judgmental. The areas for each category are calculated with the help of AUTO CAD (Having collecting coordinates by GPS).

3.2.4 Soil Type Data

Another type of data required is the soil type of the study area. It is an essential input for the calculation of runoff coefficient. The runoff coefficient of a surface will vary with soil type of the area, because some kind of soil absorb more water than the other. For this study soil type is not significant in determining the runoff coefficient for the study area is urban (most of the surface is plastered).

3.3 Methods Used

3.3.1 Estimation of Runoff

3.3.1.1 Hydrological Modeling Using ArcGIS Arc Hydro Tools

Every hydrological modelling starts from delineating streams and watersheds, and obtaining watershed properties such as area, slope, flow length, flow direction, stream network, etc. Traditionally this was being done manually by using contour maps. Now the time have changed and the availability of digital elevation models (DEM) and computer programs have made it possible to perform automated analysis. Arc Hydro tool of ArcGIS is utilized in this research for Hydrological Modelling. The strong mathematical capability and its compatibility with vast Hydrological Modelling software is the main reason for choosing ArcGIS.

Because of these tools standard processes that are crucial for hydrological modelling but are similar for different projects do not have to be time consuming. One of such tool is the terrain pre-processor. It is called terrain pre-processor because it is the primary step that will be used as input for many other hydrological modelling tools.

The steps in terrain pre-processing are to Import raw data, make raw DEM depression less, Determine Flow Direction, Compute Flow Accumulation grid, Define Stream, Segmentation of Steam, Catchment Grid Delineation, Catchment Polygon Processing and Drainage Line Processing.

The above nine steps despite their great importance for hydrological and hydraulic analysis are computer intensive and require little input from the user. The user only needs to provide the raw digital elevation model at the beginning and the required number of cells or area threshold for stream definition. Hence the whole step can be done at once by creating a model with two parameters i.e. Raw DEM and the area in square kilometer. For this study a model is developed which asks for the two parameters and perform the terrain processing. The model is developed in Model Builder of ArcGIS application. The results of the processing are put in the same directory with the input Raw DEM. The model looks as follows.

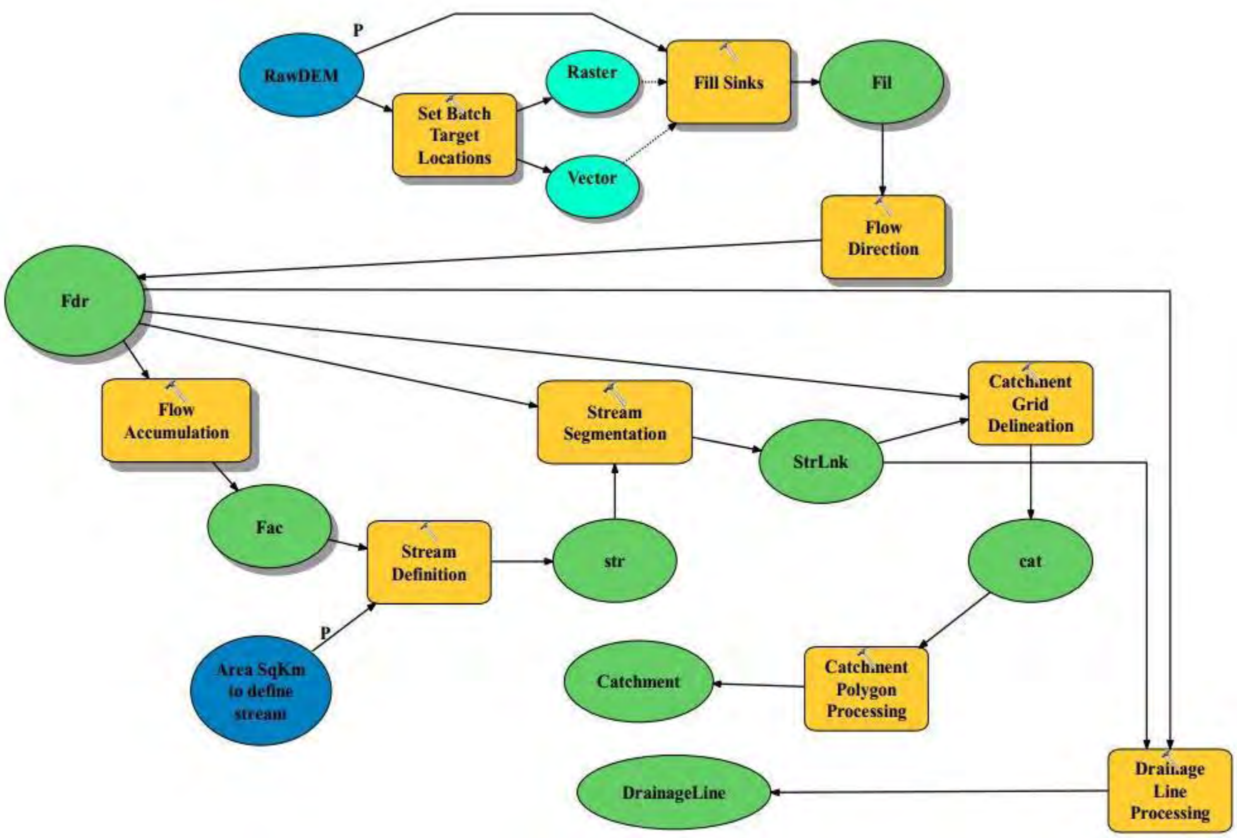


Figure 13: Custom ArcGIS Model for Terrain Processing (Anteneh ,2015)

3.3.1.2 Catchment Delineation for Study Area

In the previous sub section, the methods employed to determine the overall watershed make up as well as the flow directions have been detailed. The flow directions and the drainage lines obtained show the major and minor streams according to the definition made for streams. In order to focus into specific area, the local catchment draining to the specific point must be obtained. This is simple task as all the parameters required to delineate sub watershed i.e. flow direction grid, steam grid, catchment and ad-joint catchment are already generated by the model developed in the previous section. All that need to be done in order to get the local catchment is to select the specific point using the Arc Hydro Extension tool of the ArcGIS and the result will be displayed automatically. In order to verify the model result field survey by GPS was done. Coordinates at the ridge of the catchment are laid on AUTO cad to calculate the areas. Slope and flow length characteristics of the catchment are also worked out from the AUTO cad drawing as well as from contour map. Similarly, the flow direction are determined from the ground elevation depicted on the contour map.

3.3.1.3 Stream Network Verification

After the drainage lines of the project area are determined by running the terrain processor model they have to be verified against existing natural steams. The verification is required because there may be some inconsistencies between the actual streams and streams obtained from terrain processing due to inaccuracy of digital elevation model data. If vector data of natural streams were available, the DEM would be made to agree with the streams to make the result more accurate. Actually, data accuracy is achieved by overlaying topographic map and land use map of the area and comparing them. This method is acceptable by different literatures under circumstances. For the area is small, field survey was made throughout the catchment to cross check whether the physical situation agree with the digital elevation model data. As a result, small deviation between the two due to inaccuracy of digital elevation model was identified and then adjusted.

3.3.1.4 Rain Fall Frequency Analysis

The size of an extreme event is contrariwise related to its frequency of occurrence. Very sever events occurs less frequently than more moderate events. The objective of frequency analysis of hydrologic data is to relate the magnitude of extreme events to their frequency of occurrence through the use of probability distributions.

Forty years daily rainfall data are available from AAOS for the calculation of frequency analysis of a storm of given magnitude. This daily rainfall data is summarized to annual maximum 24hr-rainfall as shown in the Table-1 below. The design rainfall can be determined using frequency analysis. The historical rainfall data available is a 24hr duration rainfall hence an appropriate IDF reduction method need to be used to obtain rainfall intensities of shorter durations. The IDF reduction formula suggested in Ethiopian Road Authority Drainage Design Manual 2013 is used in this study.

Table 1: Annual Maximum 24hr rain fall (mm) at AAOS, 1976 -2015

year	1970	1980	1990	2000	2010
0		36.3	39.6	37.1	44.6
1		58.0	47.3	96.3	55.8
2		41.4	51.4	29.5	36.4
3		50.1	53.5	54.4	47.2
4		55.4	57.0	44.2	65.4
5		43.2	85.3	58.6	47.8
6	48.6	83.8	67.0	70.9	
7	59.4	56.8	46.3	64.0	
8	93.5	35.5	78.3	53.3	
9	50.6	49.2	37.4	54.7	

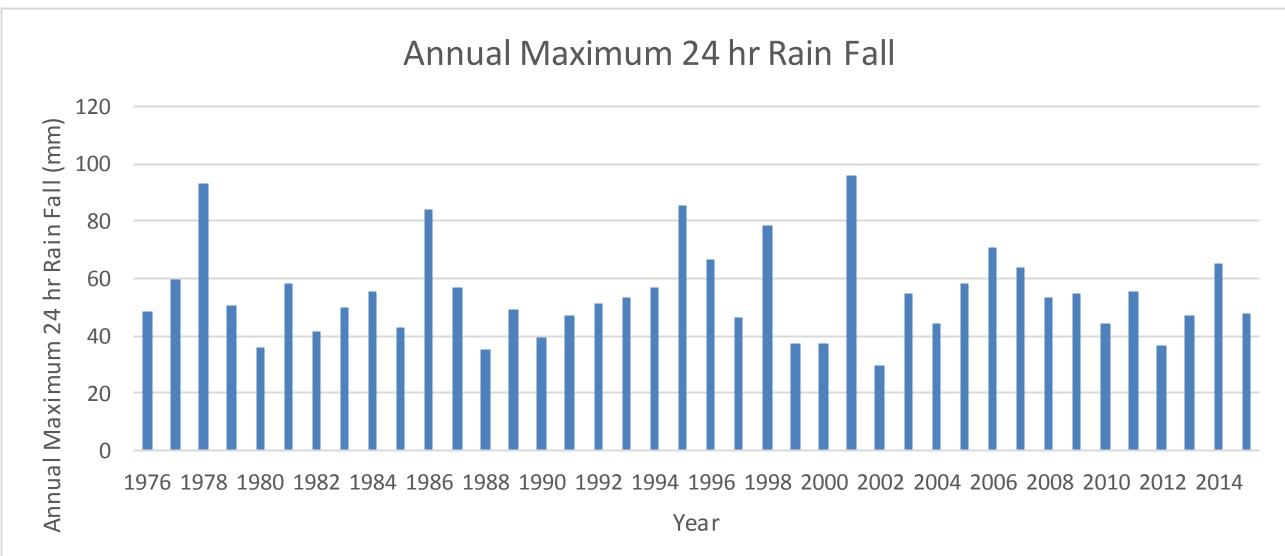


Figure 14: Annual Maximum 24 hour Rain fall (mm) at AAOS, 1976-2015

Any probability distribution can be used as a model but the reliability of the distribution is checked by the goodness of fit tests. Gumbel and Log Pearson Type III methods are used as suggested by Ethiopian Drainage Design Manual (ERA, 2013) and their goodness of fit is analyzed below.

The limitations of these statistical methods lie on the assumption that are made in using them. First rainfall is assumed not to be affected by climatic trends. While this may be true in the last century, now in the 21st century the relation between climate change and rainfall intensity is becoming more apparent. Second annual maximum series are considered as sample of random and independent events (The Hydrology Subcommittees, 1981). Historical data are used to drive sample, then the sample is used to estimate a population which will be used in making projections of the magnitude and frequency of rainfall (Chow, et al., 2012). Hence, for reliable estimates for extreme hydrological event, long term data series is required.

I. **Gumbel:** - Also known as Extreme Value Distribution Type I or double-exponential distribution of extreme values. It is based on the assumption that the cumulative frequency distribution of the largest values of samples drawn from a large population can be described by the following equation:

$$F(x) = \exp\left[-\exp\left(-\frac{x-u}{\propto}\right)\right], \qquad -\infty \leq x \leq \infty$$

$$\text{Where}, \propto = \frac{\sqrt{6}S}{\pi}\ , S = standard\ deviation\ \& \qquad u = \bar{X} - 0.5772 \propto$$

The methods employed in Ms. Excel are as follows

- Annual extreme values are obtained using =max(∀) function, where ∀ represents all record of 24 hour daily rainfall obtained from Ethiopian Meteorological Agency as mentioned in data section.
- Average of yearly maximum rainfall data is obtained using =average () function
- Standard deviation of the data set is obtained using =stdev () function
- The constant α of the data set is calculated using $\propto = Stdev(\quad) * \sqrt{\frac{6}{\pi}}$
- The constant u of the data set is calculated using $u = average(\ )-0.5772\alpha$

- Using equations $Y_T = -\ln\left[\ln\left(\frac{T}{T-1}\right)\right]$ and $X_T = \mu + \alpha Y_T$ where y is reduced variant, T is return period and X is rainfall. The result is tabulated for different return period (see Index- Table 11).

II. Log Pearson Type III: - is a three-parameter gamma distribution with a logarithmic transform of the variable. Mean, standard deviation, and coefficient of skew ness are the three necessary parameters that are necessary to describe the distribution.

The methods employed in excel are as follows

- The logarithms of annual maximum values are calculated,$Y = \log X$
- The average (=average ()), standard deviation (=stdev ( )) and standardized skew (=skew ( )) of the data set are obtained using the respective excel functions.
- Coefficient k to be used in calculating K_T is obtained as ${skew()}/{6}$
- Coefficient Z to be used in calculating K_T is obtained using excels = $Normsin$ (1/T) function where T is the desired return period. Or, the value of Z corresponding to an Exceedence probability of p(p=1/T) can be calculated by finding the value of an intermediate variable w:

  $w = \left[\ln \frac{1}{p^2}\right]^{1/2}$ $\quad (\, 0 < p \leq 0.5)$ Then calculate Z using the approximation,

$$Z = w - \frac{2.515517 + 0.802853w + 0.10328w^2}{1 + 1.432788w + 0.189269w^2 + 0.001308w^3}$$

- Coefficient K_T is calculated as

  $$K_T = Z + (Z^2 - 1)K + \frac{1}{3}(Z^3 - 6Z)K^2 - (Z^2 - 1)K^3 + ZK^4 + \frac{1}{3}K^5$$

  Y_T is calculated using Y_T=$average$( )+ K_T∗$stdev$( ) ; That is $Y_T = \bar{y} + k_T * S$
- Magnitude of precipitation with a return period of T,X_T is obtained by transformingY_T. (See index- Table 11)

  x_T=10^{Y_T}

Goodness of Fit Test

The goodness of fit (GOF) tests measure the compatibility of a random sample with a theoretical probability distribution function. These tests show how well the selected distribution fits to data. There are three most commonly used GOF tests. These tests are the Anderson-Darling, the Kolmogorov-Smirnov, and the Chi-Squared tests. In all three tests a parameter or statistic unique to each method is calculated for the required distribution types and these distributions are ranked based on their parameter values.

1. **Kolmogorov-Smirnov Test**

This test is used to decide if a sample comes from a hypothesized continuous distribution. It is based on the empirical cumulative distribution function (ECDF). Assume that we have a random sample $X_1 \ldots \ldots \ldots . X_n$ from some continuous distribution with CDF F(x). The empirical CDF is denoted by

$$F_n(X) = \frac{1}{n}[Number\ of\ observations \leq X]$$

The Kolmogorov-Smirnov statistic (D) is based on the largest vertical difference between F(x) and$F_n(x)$. It is defined as

$$D_n = {}^{sup}_{\ \ x}|F_n(x) - F(x)|$$

When comparing different distribution, lower statistics means better fit.

2. **Anderson-Darling Test**

The Anderson-Darling procedure is a general test to compare the fit of an observed cumulative distribution function to an expected cumulative distribution function. This test gives more weight to the tails than the Kolmogorov-Smirnov test.

The Anderson-Darling statistic (A^2) is defined as

$$A^2 = -n - \frac{1}{n}\sum_{i=1}^{n}(2i-1).[\ln F(X_i) + \ln(1 - F(X_{n-i+1}))]$$

The hypothesis regarding the distributional form is rejected at the chosen significance level (alpha) if the test statistic,A^2 is greater than the critical value obtained from a table. When comparing different distribution, lower statistics means better fit.

3. Chi-Squared Test

The Chi-Squared test is used to determine if a sample comes from a population with a specific distribution. This test is applied to binned data, so the value of the test statistic depends on how the data is binned. Although there is no optimal choice for the number of bins (k), there are several formulas which can be used to calculate this number based on the sample size (N). For example, Easy Fit employs the following empirical formula:

$$k = 1 + \log_2 N$$

The data can be grouped into intervals of equal probability or equal width. The first approach is generally more acceptable since it handles peaked data much better. The Chi-Squared statistic is defined as,

$$X^2 = \sum_{i=1}^{k} \frac{(O_i - E_i)^2}{E_i}$$

Where O_i is the observed frequency for bin i, and Ei

$$E_i = F(X_2) - F(X_1)$$

Where F is the CDF of the probability distribution being tested, and x1, x2 are the limits for bin i. When comparing different distribution, lower statistics means better fit.

Easy Fit 5.6 Professional software is used for testing goodness of the recommended Log Pearson-III and Gumbel Methods.

3.3.1.5 Development of Intensity- Duration- Frequency Curve (IDF Curve)

Rain fall of a place can be completely defined if the intensities, durations and frequencies of the various storms occurring at that place are known. Whenever an intense rain occurs, its magnitude and duration is generally known from meteorological readings. This available data can be used to determine the frequencies of the various rains. Such frequency data for storms of various durations can be represented by Intensity-Duration-frequency (IDF) curves. An IDF curve is a plot of average rainfall intensity (rainfall depth is averaged over the duration i.e.

$$\frac{rainfall\ depth}{duration} = average\ rainfall\ intensity\ in\ that\ duration)$$

and the duration in minutes (S. K Garg, 2005)

However, when short time duration rainfall data is not available intensity of a short time rainfall long time rainfall would be calculated using reduction formula.

The rainfall depths obtained from gauging station are of 24hr duration depth. Design and analysis of drainage structures require rainfall intensity duration relationship of shorter duration. Because rainfall data of shorter duration is unavailable, appropriate IDF derivation for shorter duration is required. Ethiopian Road Authority (ERA) Drainage Design Manual of 2013 suggests the following equation for calculation of shorter duration rainfall from 24hour duration rainfall.

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

Where:

R_{Rt} = Rainfall depth ratioR_t: R_{24}

R_t= Rainfall depth in a given duration t (hours)

R_{24} = 24hr rainfall depth

Coefficients b = 0.3 and n = 0.78 - 1.09

The methods employed to develop IDF curve for the shorter duration events using the above equations are as follows.

- Using the trend line equation obtained from Log Pearson type III method of frequency analysis, i.e. y = 11.847ln(x) + 37.521(See Index-Figure 23) where y is 24-hour rainfall depth (R_{24}) of a return period x under consideration, R_{24} is calculated for 2, 5, 10, 25, 50 and 100 year return period. Rearranging the above equation gives

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

Intensity (mm/hr.) $I_t = \frac{Rt}{t}$ Substituting in the above equation gives

$$I_t = \frac{R_{24}(b+24)^n}{24(b+t)^n} \text{ ,} where, I_t\ (\frac{mm}{hr}), t\ (\ hours), R_{24}\ (\ mm)$$

- Using b = 0.3 and n = 0.92 as suggested by ERA 2013 manual results are tabulated (see Index -Table 12) for rainfall durations 10, 20, 30 … 180 minutes.
- The resulting table is graphed for each return period. That is IDF curve is developed using reduction formula.

3.3.1.6 Run off Size Determination Using Rational Method

Runoff estimation can be performed by either statistical methods or deterministic methods. Statistical methods are based on historical gauging records to estimate the probability of occurrence of a given event. Runoff from a given site may be subject to changes by urbanization and drainage improvements. Statistical methods have no parameter to account for these changes and that is their limitation. Unlike statistical methods deterministic methods are based on a cause-effect consideration of the rainfall-runoff processes. Rational method is one of deterministic methods. Rational formula is recommended for size of drainage area less than 50 hectares (ERA drainage manual 2013). This method is used in this research to calculate the peak runoff as the size of the study area is within the permissible limit.

The idea behind the rational method is that for a *spatially* and *temporally* uniform rainfall intensity (i) which continues indefinitely, the runoff at the outlet of a catchment will increase until the time of concentration (t_c), when the whole catchment is contributing flows to the outlet. This takes the assumption that peak runoff does not result from more intense rainfall of shorter duration. The peak runoff is given by the following expression:

$$Q_p = 0.278CiA$$

Where Q_p =peak runoff in m^3/s

C = runoff coefficient

i = rainfall intensity in mm/hr

A = catchment area in $(Km)^2$

- The run off coefficient depends on the runoff quantities of the catchment and varies from 0.2 to 0.9. This can also be given by the ratio;

$$C = \frac{actual\ Discharge}{Theoretical\ Discharge} = \frac{peak\ run\ off\ rate}{rain\ fall\ rate}$$

- The intensity of rainfall, i, is equal to the design intensity or critical intensity of rainfall,i_c, corresponding to the time of concentration , t_c ,for the catchment for a given recurrence interval T. The design intensity of rainfall i (=i_c) can be found from the intensity-duration-frequency (IDF) curves for the catchment corresponding to t_c and T. If IDF curves are not available for the catchment and a maximum precipitation P,(cm) occurs during a storm period of t_R hours, then the design intensity I (= i_c) can be obtained from the equation

$$i_c = \frac{P}{t_R}\left(\frac{t_R+1}{t_c+1}\right)$$

When the time of concentration, t_c is not known, $i_c = \frac{P}{t_R}$ (H.M Raghunath, 2008)

However, for this study an IDF curve is developed and the design intensity can be read for corresponding time of concentration and return period.

3.3.1.6.1 Determination of Run off Coefficient, C

As runoff coefficient is the ratio of actual discharge from a catchment to that of theoretical discharge(if all rainfall occurs as runoff without any loss through evaporation, infiltration to subsurface, interception by trees or other structures on the earth surface etc) from the same catchment, it depends on percent imperviousness, slope, ponding characteristics of the surface, character or condition of the soil, rain fall intensity, proximity of the water table, degree of soil compaction, porosity of the sub soil and vegetation. The more the surface is impervious the higher the runoff will be as the infiltration decreases. Slope of the ground surface tends to give time for the rain drop either to stay or to flow. The higher the slope the lesser time the drop has thereby the drop flow down increasing the runoff and vice versa. If the soil is wet by previous precipitation and the infiltration rate is decreased or come to almost zero, the rain after such condition of soil shall join the runoff as the infiltration is negligible or so. The rainfall intensity directly affects the runoff coefficient. If for example, the intensity is greater than the rate of infiltration, no matter the soil condition (Dry or wet) runoff generates. On the other hand, if the intensity is much less than the infiltration rate, the tendency the all rain infiltrate to the ground increases such that the runoff coefficient value affected. The more the water table is closer to the surface, the soil condition become wet through capillary action so that infiltration decreases thereby increasing the runoff coefficient. The degree of compaction of the surface has direct impact on the porosity of the soil. The lesser the porosity is the lesser the infiltration rate and vice versa. Therefore, runoff coefficient value increases with high porosity and decreases with low porosity. In this case porosity refers to interconnected pores. Vegetation cover reduces the impact of rain drop on the ground and intercepts some of the rain on its leaves and branches letting them to evaporate. Rain drops that reach on the ground do not easily flow as the vegetation cover interrupts the flow giving much time for infiltration. This directly decreases the runoff coefficient.

A reasonable coefficient must be chosen to represent the integrated effects of all these factors. The runoff coefficient is the most important variable in the rational method of rainfall to runoff transformation. A weightage method is employed to obtain the representative runoff coefficient i.e. the individual areas multiplied by their specific runoff coefficient and their values added together and divided by the cumulative area. (Ven Te Chow, et al, 2012)

$$C_w = \frac{\sum_{I=1}^{n} C_i A_i}{\sum_{i=1}^{n} A_i}$$

Where, $C_w = weighted\ C;\ C_i = runoff\ coefficient\ for\ area\ A_i$

3.3.1.6.2 Determination of Time of Concentration (T_c)

The time of concentration is the time required for runoff to flow from the most remote point of the basin to the location being analyzed. In Rational Method time of concentration is used to determine the rainfall duration which will result in maximum runoff. The required design rainfall intensity is then established from IDF curve for the required recurrence interval.

Manning's equation can be used for road and railway side drainages as well as cross drainages because the channel cross section is defined and known. But Manning's equation is not used to calculate time of concentration for sub basins as the flow is sheet flow upstream and later the flow collects and flow in a channel. .

Flow path from the upstream sub basin to the outlet is well defined because of the volume of the water, hence the equation developed by United States Soil Conservation Service as suggested by literature is used.

$$T_c = \left(\frac{0.87L^2}{1000Sav}\right)^{0.385} \quad \text{Kirpiche's formula}$$

Where T_c is time of concentration (hours), L is hydraulic length of catchments measured along flow path from the catchment boundary to point where flood needs to be determined (km) and $S_{av}$is average water shade slope in (m/m).

Different slopes along the hydraulic length of the catchment were calculated from field survey data. The weighted average of these slope is taken to calculate (estimate) the time of concentration.

$$S_w = \frac{\sum_{i=1}^{n} S_i L_i}{\sum_{i=1}^{n} L_i}$$

Where, S_w is weighted average, $S_i\, slope\ for\ length\ L_i$

As rain-fall duration tends towards zero, the rainfall intensity tends towards infinity. Because the rainfall intensity/duration relationship is accessed by assuming that the duration is equal to the time of concentration, small areas with exceedingly short times of concentration could result in design rainfall intensities that are unrealistically high. To minimize this likelihood, use a minimum time of concentration of 10 minutes when using the coefficients presented in the Hydrology document. As the duration tends to infinity, the design rainfall intensity tends towards zero. Usually, the area limitation of 50 hectares should result in design rainfall intensities that are not unrealistically low. However, if the estimated time of concentration is extremely long, such as may occur in extremely flat areas, it may be necessary to consider an upper threshold of time or use a different hydrologic method (ERA drainage Design Manual, 2013)

3.3.2 Possible Engineering Solutions Approach

The usual way to approach to a solution is to review and evaluate the design analysis report and the as-built drawing for both the road side drainage as well as the road geometry. If the design analysis is accurate as per the design theory and procedure and the construction is as per the then design , then, cross check variables such as runoff coefficient and rain fall intensity whether they are changed through the process of urbanization, global warming or climate change. For this study, neither design analysis nor as-built drawing is available at the stakeholder, Addis Ababa City Road Authority (AACRA). That is, comparison between the existing one and the design is not possible. Therefore, the procedure to approach to a solution is to collect actual data of size and slope of the side drainage ditch; length, width and cross slope of the asphalt; size of the catchment; land use land cover data; estimate rainfall intensity and analyze the run off produced within the catchment. This is done in two scenarios. First, check whether the side drainage accommodates the runoff from the asphalt area only. Because, AACRA's manual recommends the size of drainage to be designed to accommodate the runoff from the asphalt area only. The second scenario is to check whether the existing drainage accommodate runoff from both the asphalt and the catchment.

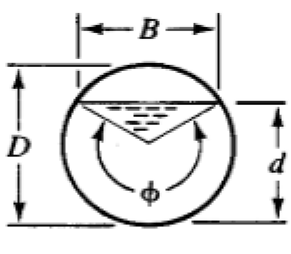
To assess whether the side drainage accommodate runoff from the asphalt, the maximum flow is at the outlet for the span (one lane only) from Ayer-Tena to Zenebe Werk round about is at the round bout. Estimate the peak runoff for ten year recurrence interval using rational method and then determine drainage size using Manning's formula.

In order to check drainage size whether it can accommodate the flow from the asphalt, take the out let of the asphalt from Ayer-Tena to Zenebe Werk roundabout is at the roundabout. Length and width of the asphalt are 530m and 10m respectively. Time of concentration is the time calculated for the catchment. For ten year recurrence interval, read the intensity from IDF curve. The next step is to estimate the runoff size by Rational Formula, $Q = 0.278CIA$.Once Q is estimated, diameter of the drainage size can be calculated using manning's formula (for 80% full flow) which is given by

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

Where, n= manning's roughness coefficient, A= flow area, R= hydraulic radius, and Sf is slope of channel bed or hydraulic gradient. Therefore, the existing drainage size can be assessed this way for its proper size.

Table 2: Formula for area and hydraulic radius of flows in a circular open channel

Description	Area of flow (A)	Hydraulic Radius (R)
 Circle	$\frac{1}{8}(\emptyset - \sin\emptyset)D^2$	$\frac{1}{4}\left(1 - \frac{\sin\emptyset}{\emptyset}\right)D$

On the other hand, the existing drainage size would be checked whether it does have the capacity to enclose both the flow from the catchment as well as from the asphalt. The size of flow from the asphalt is known from the above procedure. Whereas, the flow size from the catchment at the outlet (at the roundabout) would be estimated in a similar manner as done for the flow from the asphalt. Consequently, the existing drainage size shall be checked whether it can accommodate the sum of the two flows from the asphalt and the catchment. Velocity of flow must be checked to be between the permissible ranges. Smaller values of velocity causes settling of suspended particles and larger values scour walls of the pipe.

Note that the existing storm water drainage size is 80cm (diameter = 80cm) made of concrete circular pipe all along the asphalt side. Similarly, this procedure shall be applied for the sub-catchment 02 in order to evaluate whether the existing drainage size is enough or not to accommodate the runoff from the asphalt or from both the asphalt and the catchment.

4 Result and Discussion

4.1 Catchment Delineation and Stream Network Generation

4.1.1 Catchment Delineation

The delineated map of the catchment area are shown in the Figures 15 & 16 below. The Field survey have been done in order to identify the ridges of the catchment for the resolution on GDEM was not that much satisfactory to identify the divide line of the catchment for the area is small. The catchment properties such as slope, area, flow length and flow directions was worked out by using GPS as the model resolution would not give accurate result for the catchment area is small.

Table 3: Catchment Properties

S.No	Catchment Properties	Sub-Catchment-01	Sub-Catchment-02
1	Average Slope(m/m)	0.023	0.017
2	Area (m2)	215,834.5	127,282.7
3	Flow length (km)	0.68	0.54

The flood at the round about as well as at the bridge (Sag point) at 50m distance from Zenebewerk roundabout to Total roundabout comes from three locations.

i. The flow from sub-catchment-01 and from asphalt road (runoff – from Ayer Tena roundabout to Zenebewerk roundabout): This covers 680m of flow length and the asphalt width is 20m (each lane). The flow from sub-catchment 01 partially comes to the interceptor asphalt road and then flows to the roundabout.

ii. The flow from sub-catchment-02: The outlet of this sub-catchment is just at the roundabout and the flow length is 540m.

iii. From asphalt runoff flow (from total roundabout to the bridge nearby Zenebe Werk roundabout): The flow length along this asphalt road is about 490m with 20m width of each lane.

Therefore, as the area of each sub-catchment is less than 50 hectare, rational method is used to estimate peak flood discharge at the catchment outlet. (ERA drainage manual 2013)

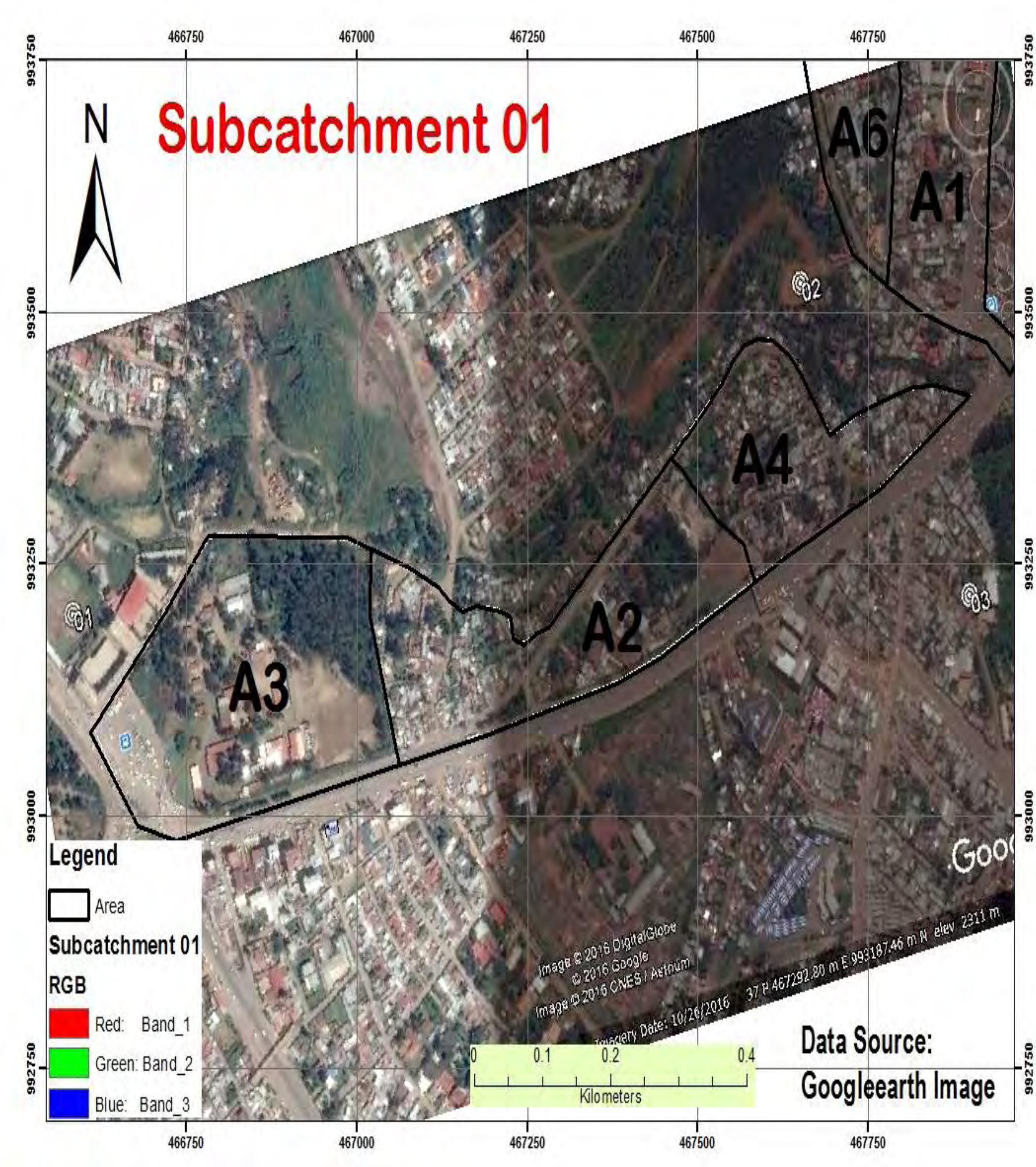


Figure 15: Shape of sub-catchment 01

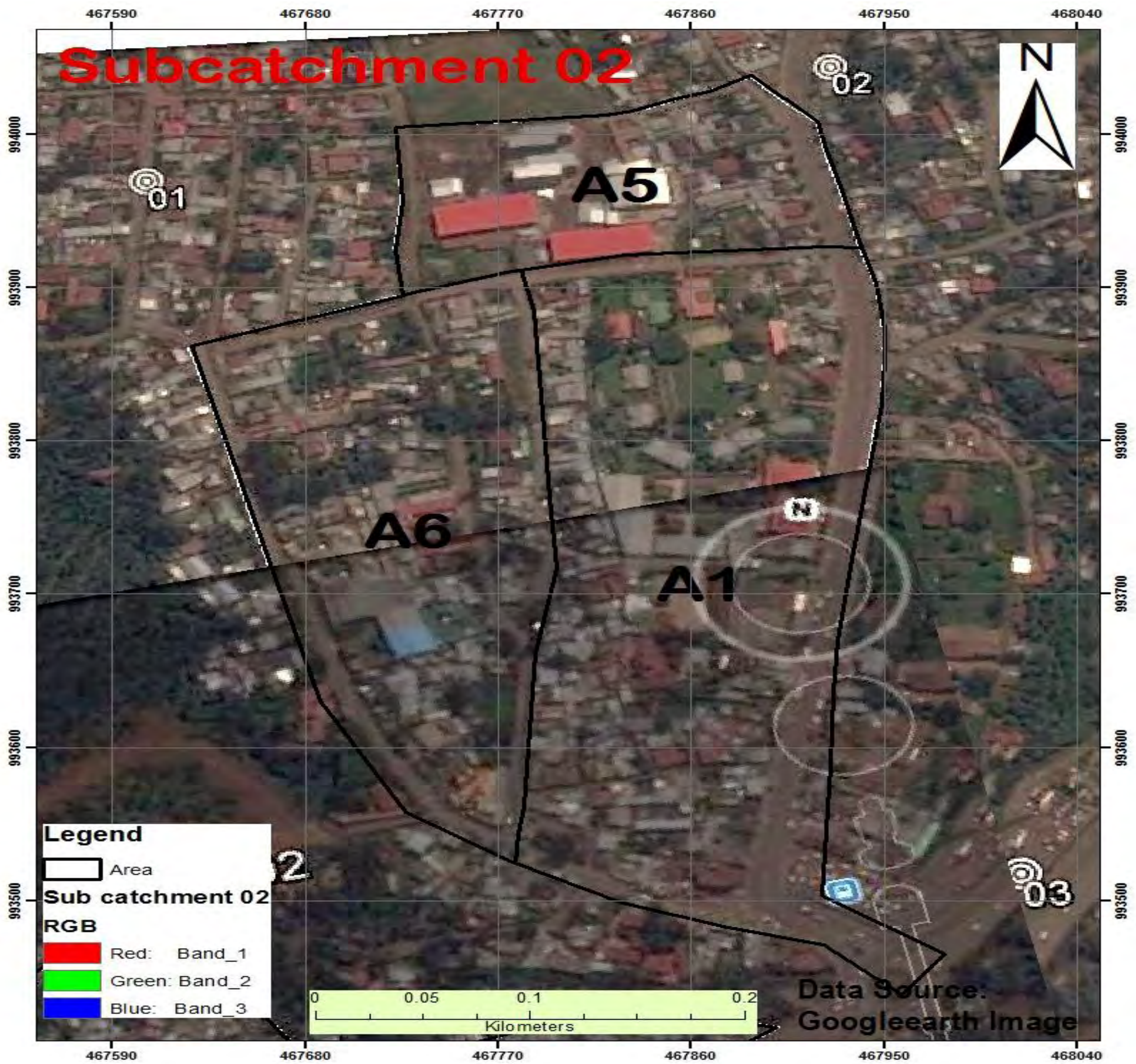


Figure 16: Shape of sub-catchment 02

4.1.2 Stream Network Generation

Two streams cross the Ayer Tena -Total route at Zenebe Werk roundabout as well as at the sag point (the bridge). The two sub catchments at the right of this roundabout when facing towards Ayer Tena (standing at the roundabout) are of great interest of this study.

Because this is urban area the stream network and catchment delineation have multiple limitations. The first limitation is the accuracy of elevation data used for modelling. The accuracy of DEM used is not good enough to correctly represent the complex urban terrain. The second limitation is in urban environment like this, runoff flows not only on surface but it also flows through manmade drainage structures such as roadside ditches. Therefore, unless built structures are incorporated in high accuracy, DEM stream networks will lack accuracy. However, using DEM the catchment is delineated and shown in the figure below. The three runoff contributing sub-catchments for which time of concentration and peak discharges are calculated are inside this catchment. The Zenebewerk roundabout and the bridge approximate locations are marked in the figure for better understanding of the study area. As it can be seen from the figure, one of the brunching stream is passing under the roundabout. The two streams join and go together once they pass by both the roundabout and the bridge.

With the limitations in mind the catchment delineation can be taken as worst case scenario. All runoff generated from the catchment do not flow towards the outlet because of the limitation that in urban environment runoff flows not only on surface but it also flows through manmade drainage structures such as roadside ditches.

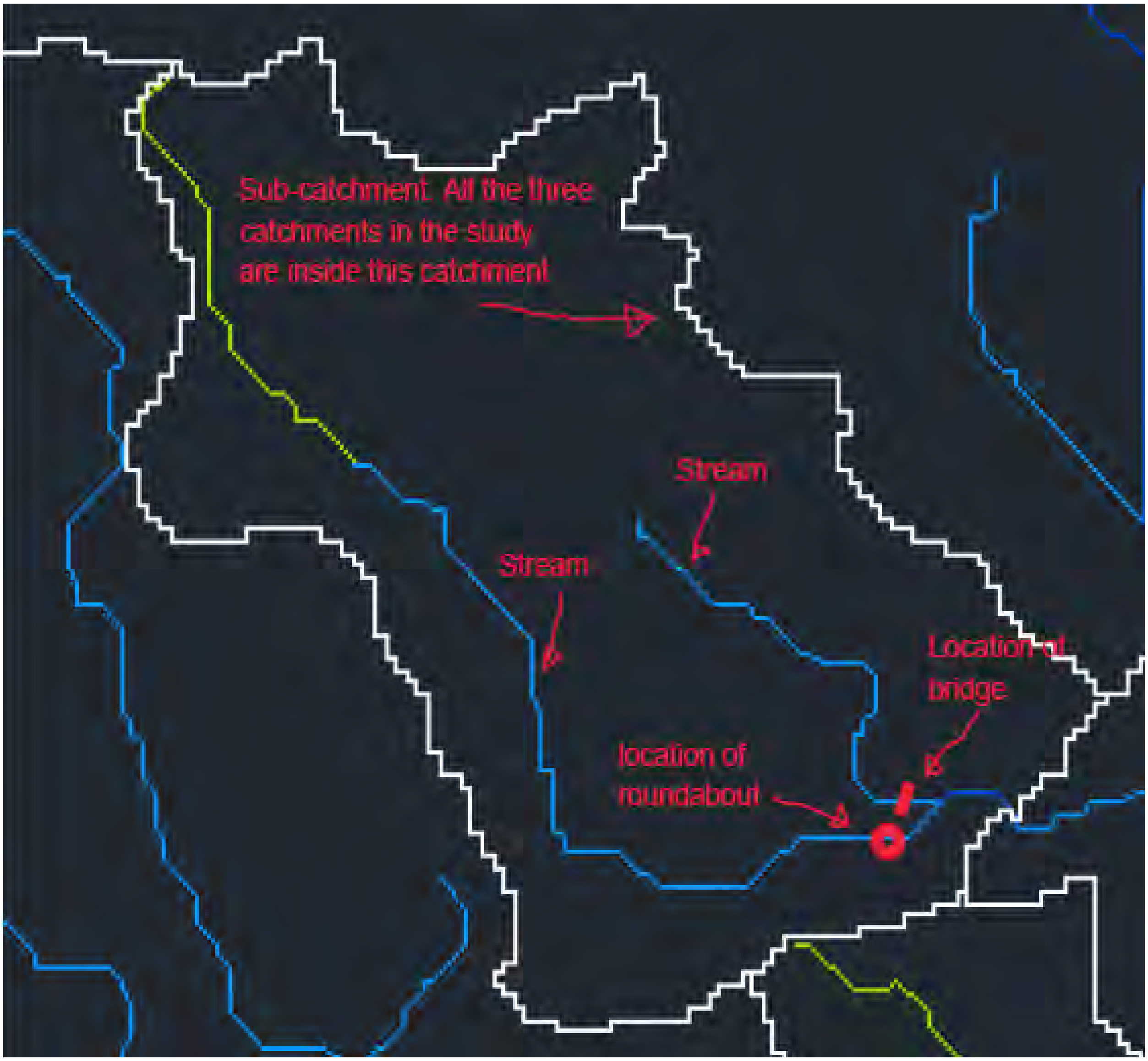


Figure 17: Stream network in the catchment

4.2 Rainfall and the Consequent Run off

4.2.1 Rainfall Frequency

The rainfall frequency analysis is done using both Gumbel and log Pearson type III methods as recommended by ERA manual 2013. The result obtained is tabulated in the following table.

Table 4: Yearly Extreme Series frequency Analysis

ARI	Extreme Rain Fall depth (mm)	
	Gumbel	Log Pearson Type III
2	52.05	45.03
5	65.94	57.13
10	75.15	65.27
25	86.77	75.75
50	95.4	83.71
100	103.96	91.82

As can be seen from the table that the extreme rain fall values from Log Pearson Type III are all smaller than that of the Gumbel values. In order to identify which distribution fits to the theoretical probability distribution, GOF test conducted using Easy Fit 5.6 professional software and the log Pearson Type III distribution fits for the statistical value for all the three different test methods is lesser than that of the Gumbel values as tabulated below. That is, Log Pearson-III method have proved to be good fit in all the three tests compared to the Gumbel's Method. The statistics for both methods are calculated and the ranking is given.

Table 5: Goodness of Fit of Log-Pearson III and Gumbel Methods

Distribution	Kolmogorov-Smirnov		Anderson-Darling		Chi-Squared	
	statistic	Rank	statistic	Rank	statistic	Rank
Log Pearson III	0.06464	1	0.23623	1	1.8868	1
Gumbel	0.07046	2	0.2588	2	2.2517	2

A graph Return period Vs. Precipitation (See Index Figure 23) for both distributions is produced and Log Person III fits for the distribution with R^2=0.9993. Therefore, both methods show that the log Pearson type III distribution fits with the rainfall data used for this study. Accordingly, the Log-Pearson III is chosen for further analysis.

4.2.2 Intensity – Duration – Frequency Curves

The IDF curve is developed from a 24-hour rainfall data of 40 years duration i.e. from 1976 to 2015, obtained from Ethiopian Meteorological Agency - gauge located around Black Lion Hospital, Addis Ababa. Reduction equation as depicted in the methodology section have been applied. Consequently, the following IDF curve has been produced. The data obtained for production of IDF curve is the result of calculations using reduction formula and it is tabulated below. Then, using the data in the table the IDF curve has been produced us shown below.

Table 6 : Duration of Rainfall against their corresponding Average Intensities

Duration (Min)	**Intensity (mm/hr)**					
	T=2	**T=5**	**T=10**	**T=25**	**T=50**	**T=100**
10	71	90	103	120	132	145
20	54	68	78	90	100	110
30	43	55	63	73	81	88
40	36	46	53	61	68	74
50	31	40	46	53	59	64
60	28	35	40	47	52	57
70	25	31	36	42	46	51
80	22	29	33	38	42	46
90	21	26	30	35	38	42
100	19	24	27	32	35	39
110	18	22	26	30	33	36
120	16	21	24	28	31	33
130	15	20	22	26	29	31
140	14	18	21	24	27	30
150	14	17	20	23	25	28
160	13	16	19	22	24	26
170	12	16	18	21	23	25
180	12	15	17	20	22	24

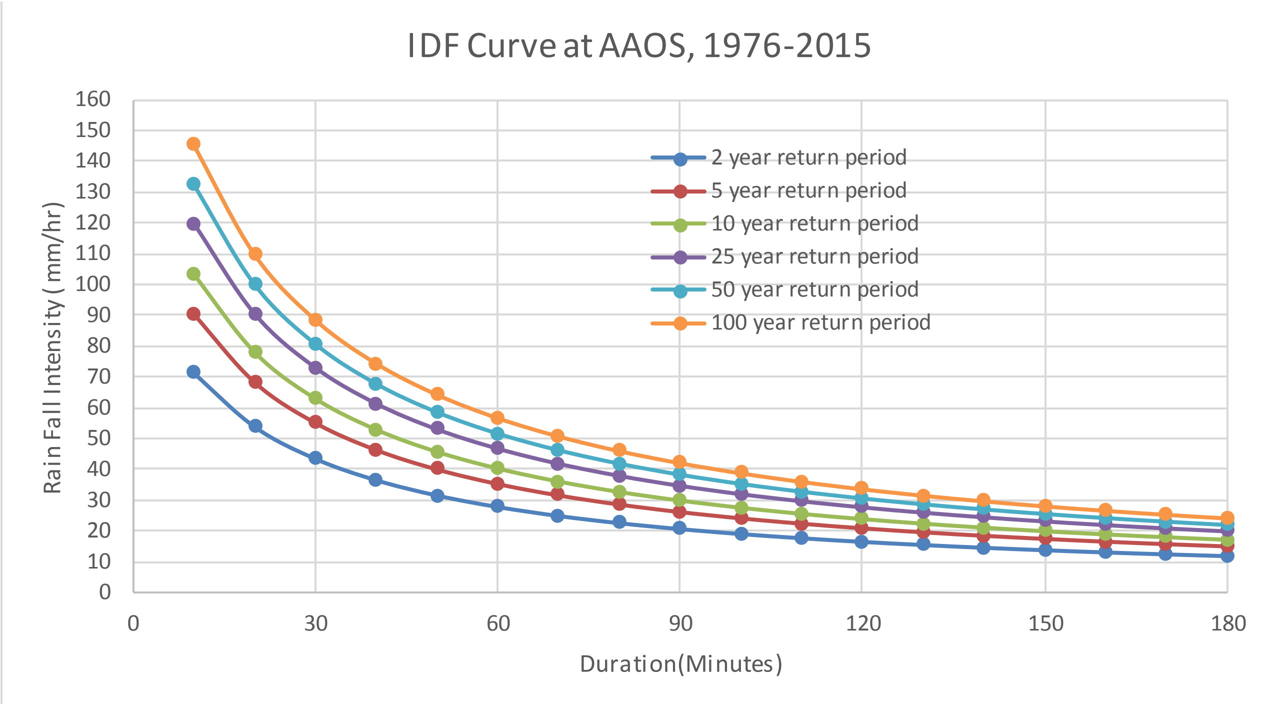


Figure 18: IDF Curve for Addis Ababa from AAOS, 1976-2015

On the other hand, IDF curve of Addis Ababa have previously been developed by AACRA and presented in its Drainage Design Manual, 2004. Duration of the rain fall data used by the authority to produce the IDF curve is 26 years only. For the release year of the manual is November, 2004, the latest record they would use is 2003 or earlier. The IDF curve developed by AACRA is presented here for comparison.

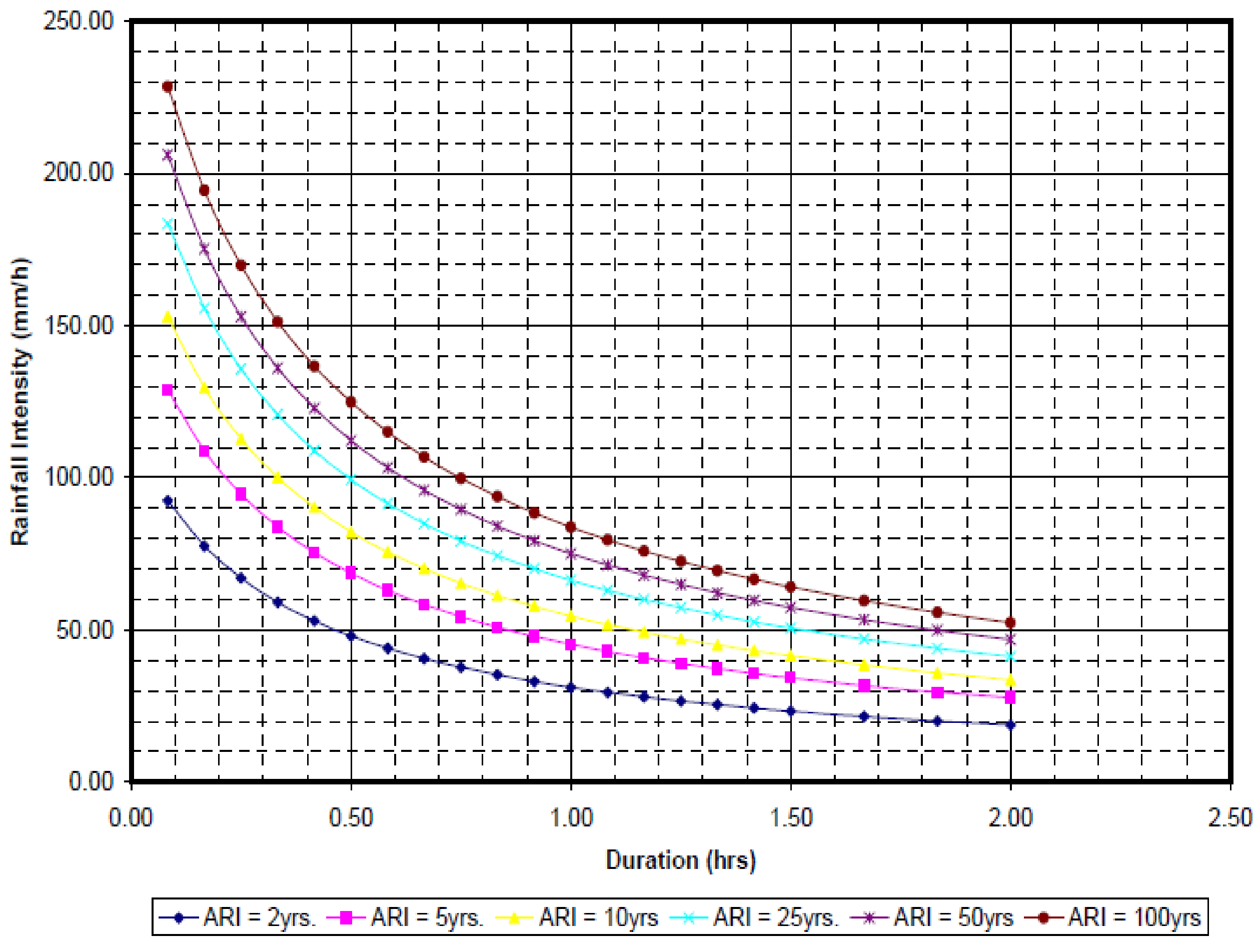


Figure 19: IDF curve of Addis Ababa from AAOS (AACRA 2004)

In the following table, comparison of IDF curve from self-analysis (IDF developed in this study) and AACRA-at AAOS (2004) is presented. The comparison is only for duration up to 120 minutes for the AACRA IDF maximum duration is 2 hours (120 minutes) only.

Table 7: Comparison of IDF curve results

ARI (yrs)	Duration(min)									
	10		30		60		90		120	
	This study IDF (mm/hr)	AACRA IDF (mm/hr)	This study IDF (mm/hr)	AACRA IDF (mm/hr)	This study IDF (mm/hr)	AACRA IDF (mm/hr)	This study IDF (mm/hr)	AACRA IDF (mm/hr)	This study IDF (mm/hr)	AACRA IDF (mm/hr)
2	**71**	75	**43**	49	**28**	31	**21**	23	**16**	19
5	**90**	104	**55**	70	**35**	45	**26**	35	**21**	27
10	**103**	125	**31**	82	**40**	55	**30**	42	**24**	33
25	**120**	150	**40**	100	**47**	67	**35**	50	**28**	42
50	**132**	175	**25**	113	**52**	74	**38**	57	**31**	48
100	**145**	183	**31**	124	**57**	84	**42**	64	**33**	52

It can be seen from the two IDF curves (the one developed in this study and that of IDF curve given in the AACRA drainage design manual) and the above table that the IDF curve of Addis Ababa developed by AACRA from AAOS have difference with the one developed in this study. The IDF curve from the road authority manual used rain fall data before the year 2003 (the length of duration 26 years). Whereas this study used a 40 year daily rain fall data from 1976 to 2015. Notice that each values in the Table-7 above for self-study are smaller than that of the correspondent values of AACRA IDF. It would be said that the use of AACRA's IDF curve is safe with regard to design purposes but it could be uneconomical.

The following are possible causes of IDF curve change.

- ✓ The recorded rainfall data length of duration can cause the changes. The rain fall data duration for this study is 40 years (Daily rainfall) from 1976 to 2015. Whereas that of the AACRA's is 26 years. But the range of the year is not mentioned. The latest data would be in the year 2003 as the manual was produced in 2004. The longer the duration is the better is the result.
- ✓ The climatic trend change due to global warming produce the change as the rain fall intensity, duration and time of occurrence be highly affected. (Raul Rodriguez, *et al*, 2013)

- ✓ As urbanization (increasing in Ethiopia especially in Addis Abeba) affects the hydrological water cycle and consequently the intensity and duration of rainfall can be affected.

4.2.3 Rational Method

Weighted run off coefficient for both sub catchments are calculated and tabulated below. Run off coefficient values for different land use land covers are referenced from Addis Ababa City Road Authority Drainage design manual November 2004. The areas for each category of land use land cover are calculated with the help of AUTO CAD having collecting coordinates by GPS. As it can be seen from the land use land cover composition table below, the weighted run off coefficient for sub-catchment 01 and 02 are 0.62 and 0.66 respectively. This implies that 62%& 66% of the rainfall rate shall be measured as peak run off rate when spatially and temporally continuous rain falls for at least a duration equal to the time of concentration from each corresponding catchment. These values are within the expected value for weighted run off coefficient of urban (Ven Te Chow *et al*, 2012)

Table 8*:* Land use composition and weighted runoff coefficient for sub-catchment-01 &02

Sub-catchment	Land use	Area (m^2)	Percentage (%)	Runoff Coefficient (C)	Weighted Average (C)
01	Mixed Residences	109,860	49.9	0.65	**0.62**
	Government compounds	28,274	12.1	0.50	
	colleges	27,411	13.7	0.55	
	Asphalt roads	20,073	10.3	0.90	
	Cobble stone roads	15,540	6.2	0.70	
	Commercial Areas	3,022	2.4	0.70	
	school	2,590	1.0	0.75	
	Health Center	431	0.4	0.60	
	Play grounds	8,633	4.0	0.30	
	Total=	**215,834**			

02	Mixed Residences	65423	51.4	0.65	**0.66**
	Government compounds	12856	10.1	0.5	
	colleges	12983	10.2	0.55	
	Asphalt roads	16292	12.8	0.9	
	Cobble stone roads	10564	8.3	0.7	
	Commercial Areas	2036	1.6	0.7	
	school	5219	1.4	0.75	
	Health Center	255	0.2	0.6	
	Play grounds	1655	1.3	0.3	
	Total=	**127,283**			

Time of concentration for the two sub-catchments as well as for the asphalt road extending from Total Roundabout to the bridge nearby Zenebewerk roundabout are calculated using Kirpiche's formula and presented in the table below. The time of concentration for areas contributing flows are 0.212, 0.21 and 0.21 hours respectively. Therefore, the bigger duration shall be taken as a value for time of concentration of the study area for the reason that within this period all the catchment contribute flow to the outlet or the Zenebewerk round about, point of interest of this study. Therefore, 0.212 hour (12.72 minutes) shall be considered as time of concentration of the study area. Based on this, the design intensity shall be read from IDF curve for corresponding recurrence intervals.

Table 9 : Time of concentration

Areas Contributing flow	**Length, L(Km)**	**Sav (m/m)**	**Time of concentration (Hours)**
Sub- catchment 01	0.68	0.023	0.212
Sub- catchment 01	0.54	0.017	0.21
Asphalt Road extending from Total to the Bridge	0.49	0.019	0.21

Length of flow as well as average slope for calculation of time of concentration were obtained by field survey. As the area is relatively smaller, field survey result is more reliable than DEM. The calculation for time of concentration and peak run off is shown in Table 24 in the appendix.

Frequency analysis of the extreme rainfall events analyzed using Log Pearson type III Distribution and then peak flood discharge is calculated using rational method. The result of peak flow rate against average recurrence interval presented in the table below.

Table 10: Peak Discharge against Average Recurrence Interval

ARI(Average Recurrence Interval)- years	Exceedence probability	Peak discharge (m3/sec)
2	50%	4.36
5	20%	5.54
10	10%	6.33
25	4%	7.34
50	2%	8.11
100	1%	8.90

Usually, as per AACRA standard, road side drainages are designed for 10 year recurrence interval peak discharges for they are minor drainage systems (AACRA drainage design manual, 2004) and based on the design storm guideline in the table below. In this case the 10 year recurrence interval peak discharge is 6.33m3/sec with 10% Exceedence probability (frequency). This magnitude of flood actually varies (or increases) as urbanization is highly promoted in the city such that either the existing road side drainage or a different intercepting drainage channel parallel to the existing one should at least accommodate to carry and dispose this flood. AACRA design storm selection Guideline provide the following chart for selection of an average recurrence interval for a specific drainage structure.

Table 11: Design Storm Selection Guideline

Road way classification	Exceedence probability	Return Period
Urban Principal Arterial System	4-2%	25-50 years
Urban Minor Arterial System	4%	25 years
Urban Collector street System	10%	10 years
Urban Local street System	20-10%	5-10 years

(Source: AACRA Drainage design manual, 2004)

4.3 Real Causes of Flooding

The cause for flooding problem at the study area are construction (or design) and drainage poor management. The basic causes of flooding of the study area are listed under:

- ✓ The runoff from the three sub-catchments: Subcatchment-01, subcatchment-02 and the asphalt road that stretches from Ayer-Tena to Total roundabouts (See Figure 18)
- ✓ Most of Inlets by the sides of the asphalt road are blocked by debris and sediments so that sheet flow over the asphalt flow to the Zenebewerk roundabout and the sag point (at the bridge) rather than relieving into the inlet and then to the drainage and finally to streams. To alleviate these problems, AACRA penetrated the median wall at the bottom at about every 3m distance however, these holes are again closed by debris. The purpose of these holes to transfer the runoff from one lane to another and then to the other road side of inlets. But the inlets themselves are blocked by sediments and still the flow is on the surface of the asphalt. On the other hand, if the inlets be cleaned they could not afford to the discharge runoff from the other lane as they were designed to threat runoff from one lane only.

Figure 20: Blocked inlets by the side of roads

- ✓ In most parts of the asphalt road, the cross slope of the asphalt road is not proper as it does not guide the flow to the inlets. This might be construction or design problem. This could not be confirmed as the design report and as-built drawing are not available from AACRA.
- ✓ Big run off is joining the asphalt road (See figure 18) from neighboring catchment. This flow is reversed and made to flow over the asphalt road.

Figure 21: Run off location that comes from neighboring catchment

The following real pictures would partially demonstrate the extent of flooding in the study area. Furthermore, it shows up negative consequent of flooding on the nearby community and people using the road. These pictures were taken on August 7, 2016 just after a high rain fall.

4.4 Possible Remedial Engineering Solutions Approach

4.4.1 Evaluation of the Existing Road Side Storm Drainage Size

Table 12: Evaluation of the existing drainage size for its capacity

S.No	Description	Q (m3/s) for T=1oyear	Drainage size required (m)	Remark
1	**Scenario-1** (C=0.9; I=94.8mm/hr; A=0.01km2 for the length is 530m and width is 20m)	0.25	0.29	Drainage size is calculated using Manning's formula. The value of n=0.012 for concrete and that of Sf=0.02
2	**Scenario-2** (C=0.62; I=94.8mm/hr; A=0.22km2)	3.84	0.80	

As it can be seen from the table that the existing drainage (diameter=0.8m) can accommodate the runoff from the asphalt for the required diameter is 0.29m only. Moreover, it can also accommodate the flow from both the asphalt and the catchment for it needs about 0.80m diameter of pipe. Note that the calculated drainage size is required at the outlet; however, for simplicity of construction the same size of concrete pipe can be placed all along the asphalt length.

4.4.2 Remedial Solutions

- ✓ The run off from the two sub-catchments should be collected in a sub catchment and connected to the existing road side storm water drainage such that it later joins the stream. That is, inlet along the drainage lie must be provided not only for the asphalt flow but also for the flow from the catchments.

- ✓ The inlets should be cleaned regularly such that they could at least give relief for some flood quantity even though the asphalt cross slope at some length of the road was not done (the construction or Design) properly.
- ✓ A lot of flood is being discharged over the asphalt from neighboring catchment as shown in Figure 18. This flood should soon be guided in a channel and disposed to the river. This gives some relieve to the flooding problem.
- ✓ At the sag point of the bridge (both left and right sides) inlets are made being piercing the bridge slab. This may result in loss of life as well as lesser life time of the bridge. These holes should be repaired the soonest possible. Then, temporary solution might be provided by demolishing the curb (which is not structural element of the bridge) on the left side (facing to Zenebewerk roundabout) of the bridge such that flooding would be minimized

5 Conclusion and Recommendation

5.1 Conclusion

- ✓ The rain fall runoff transformation analysis was worked out using rational Method for the total area of the catchment ($0.37km^2$) contributing the flow is less than 50 hectare, Consequently, the ten year recurrence interval peak discharge found to be 6.33m3/sec. This peak discharge is estimated based on the present existing land use land cover condition. The future land use land cover condition determines whether this discharge size increase, decrease or remain as it is. Therefore, any design modification for the drainage system or the road must consider the future land use land cover situation.

- ✓ The cause for flooding problem at the study area are construction and/or design problems and drainage poor management. The construction and/or design problems are cross slopes of the asphalt road which do not guide the flow over the asphalt to inlets, the surface runoff from the two sub-catchments have no inlets to join the drainage and the reversed runoff from the neighboring catchment. In addition, blockage of inlets with sediments aggravate the flooding.

5.2 Recommendation

- ✓ As-built drawing and design analysis for the road and drainage system of the study area is not available. For such kind of studies to be complete (even for future maintenance and repair) responsible bodies must record such significant data.
- ✓ The present as well as the future land use land cover map is not available. This is an important element in the analysis of flooding. This study did physical survey to collect the present land use land cover of the catchment to calculate runoff coefficient of the study area thereby the peak flood estimated accounts the current situation only.
- ✓ A 24 hour rainfall data is utilized for production of IDF curve. If instantaneous rainfall data were available, the IDF curve would be more accurate.
- ✓ A forty year 24 hour rain fall data was used for analysis and estimation of peak run off rate. This peak run off rate can be more precise if longer rain fall data were available.
- ✓ If there were standard to categorize an area as commercial or residential, the value of runoff coefficient can be made more accurate.
- ✓ If the similar work is to be done in the city for areas with flooding problem, the approach and methodology followed is applicable
- ✓ At the sag point of the bridge, a lot of flood accumulates for the inlets are not big enough to discharge the flow to the river. AACRA made big hole (piercing the bridge slab) on both left and right sides of the bridge in order to dispose the flow vertically down to the river. However the problem still exists. The hole affects the structural strength of the bridge and consequently may result in human life loss as well as shorter life time of the bridge. Solutions (whether short term or long term) better be justifiable. As the design analysis of the road is not available it could not be possible to check whether the sag point was given special attention or not. But the existing flooding problem tells that inlets at the sag point are not capable to relief the flow out. Consequently, the design need to be revised proper solution

should be coined soon in order to safeguard the wellbeing of the bridge as well as to improve the socioeconomic activity of the community who are using the road

- ✓ According to natural hydrological phenomena, due to increased impervious area precipitation responds quickly reducing the time to peak and producing higher peak flows in the drainage channels. Therefore, the design and construction (road as well as drainage) need to be revised as urbanization is highly growing in the city.

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7 Appendices

Table 13: Yearly Extreme series frequency analysis Calculations

Year	Annual Maximum 24-hr rain fall in mm(X)	Y=log(X)	Year	Annual Maximum 24-hr rain fall in mm(X)	Y=log(X)
1976	48.6	1.687	1996	67	1.826
1977	59.4	1.774	1997	46.3	1.666
1978	93.5	1.971	1998	78.3	1.894
1979	50.6	1.704	1999	37.4	1.573
1980	36.3	1.560	2000	37.1	1.569
1981	58	1.763	2001	96.3	1.984
1982	41.4	1.617	2002	29.5	1.470
1983	50.1	1.700	2003	54.9	1.740
1984	55.4	1.744	2004	44.2	1.645
1985	43.2	1.635	2005	58.6	1.768
1986	83.8	1.923	2006	70.9	1.851
1987	56.8	1.754	2007	64	1.806
1988	35.5	1.550	2008	53.3	1.727
1989	49.2	1.692	2009	54.7	1.738
1990	39.6	1.598	2010	44.6	1.649
1991	47.3	1.675	2011	55.8	1.747
1992	51.4	1.711	2012	36.4	1.561
1993	53.5	1.728	2013	47.2	1.674
1994	57	1.756	2014	65.4	1.816
1995	85.3	1.931	2015	47.8	1.679

1. Gumbel Method			Calculations		
			Return Period (T)	Y-T	X-T(mm)
$\bar{X}$	54.64		2	0.37	52.05
Sx	15.72		5	1.50	65.95
α	12.26		10	2.25	75.15
u	47.56		25	3.20	86.77
			50	3.90	95.40
			100	4.60	103.96

2. Log Pearson Type III method			**Calculations**						
$\bar{y}$	1.721		Return Period (T)	1/T	w	z	K-T	Y-T	X-T(mm)
Sy	0.118		2	0.5	1.177	-1.010067E-07	-0.05483	1.65	45.03
Cs	0.330		5	0.2	1.794	8.414567E-01	0.820966	1.76	57.13
k	0.055		10	0.1	2.146	1.281729E+00	1.311358	1.81	65.27
			25	0.04	2.537	1.751077E+00	1.859214	1.88	75.75
			50	0.02	2.797	2.054189E+00	2.227067	1.92	83.71
			100	0.01	3.035	2.326785E+00	2.567464	1.96	91.82

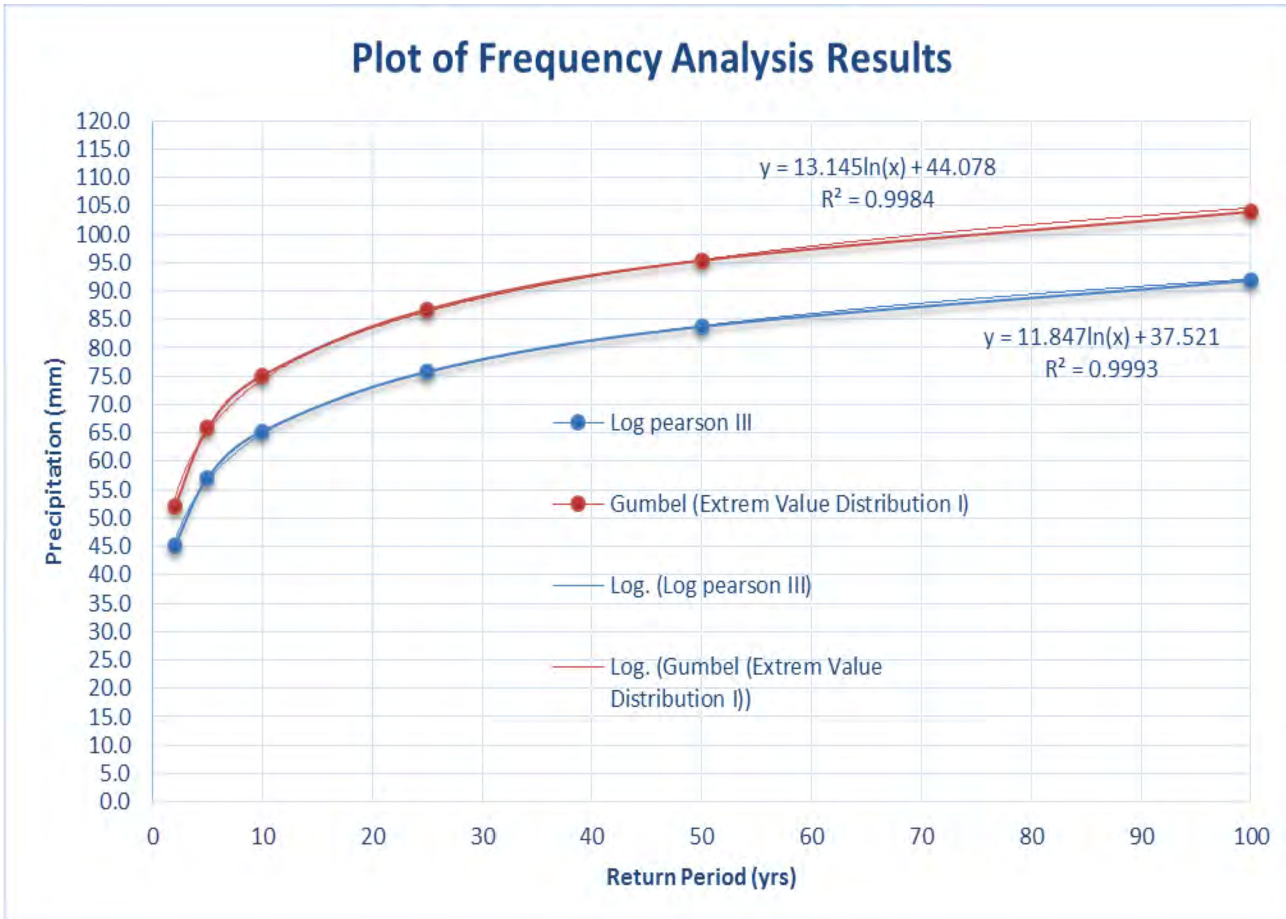

Figure 22: Plot of Frequency analysis Results

Table 14: Calculation of Rain fall of shorter duration using results from Log Pearson Type Ⅲ distribution method.

Duration (mints)	Return period in years (T)					
	2	5	10	25	50	100
	Annual Maximum 24-hr rain fall in mm(X) R-24					
	45.0	57.1	65.3	75.75	83.7	91.8
10	71	90	103	120	132	145
20	54	68	78	90	100	110
30	43	55	63	73	81	88
40	36	46	53	61	68	74
50	31	40	46	53	59	64
60	28	35	40	47	52	57
70	25	31	36	42	46	51
80	22	29	33	38	42	46
90	21	26	30	35	38	42
100	19	24	27	32	35	39
110	18	22	26	30	33	36
120	16	21	24	28	31	33
130	15	20	22	26	29	31
140	14	18	21	24	27	30
150	14	17	20	23	25	28
160	13	16	19	22	24	26
170	12	16	18	21	23	25
180	12	15	17	20	22	24

Rain fall of Shorter Duration (Log Pearson Ⅲ)

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

$$I_t = \frac{R_{24}(b+24)^n}{24(b+t)^n}$$

Where, n=0.92 & b=0.3

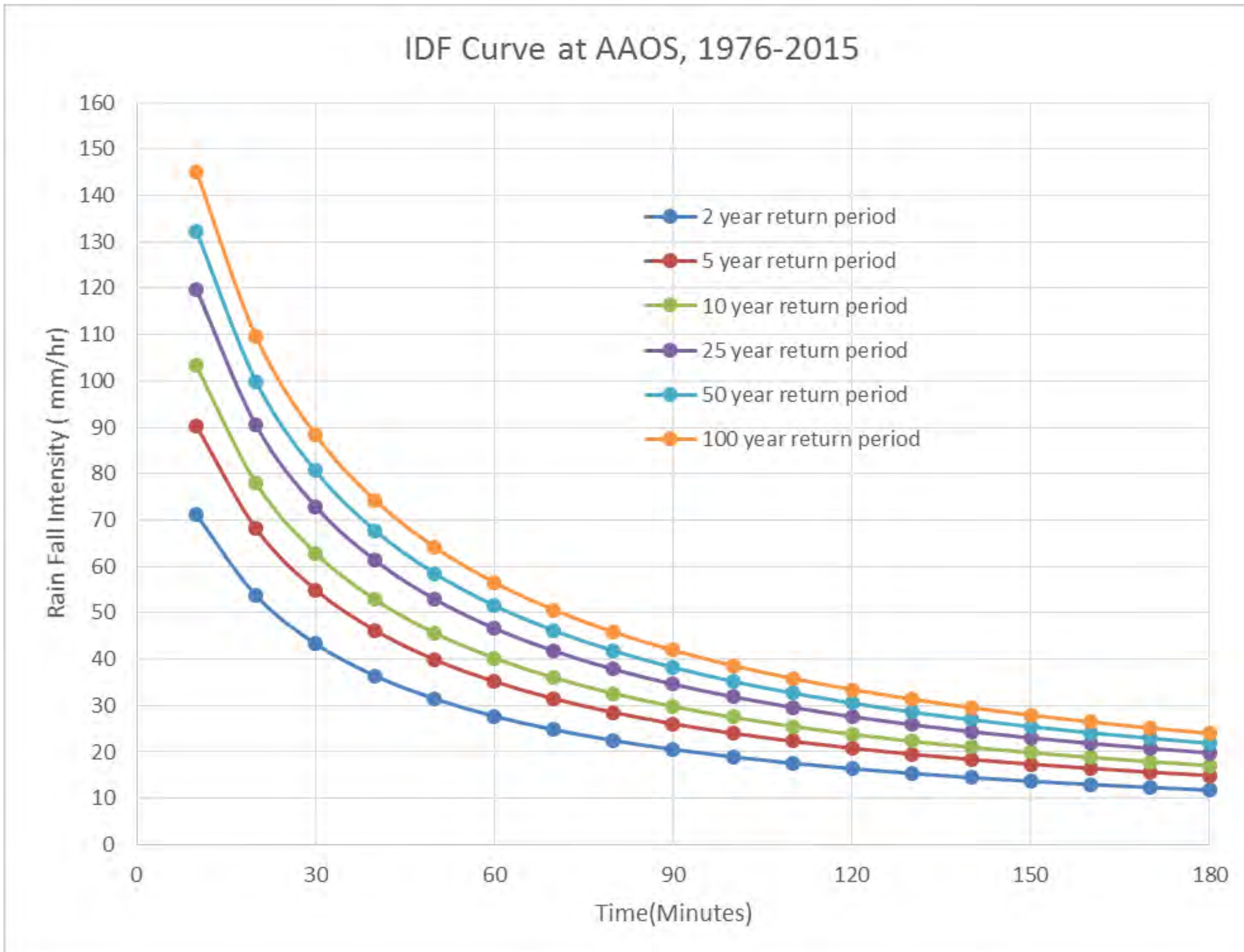

Figure 23: Intensity-Duration-Frequency curve at AAOD, 1976-2015

Table 15 Calculation for Estimation of Peak floods for different Return Periods

Input Data

Areas contributing flow	Length, L (Km)	Sav(m/m)	C (Weighted runoff coefficient)	A(km2)
Sub-catchment-01	0.68	0.023	0.62	0.22
Sub-catchment-02	0.54	0.017	0.66	0.13
Asphalt Road extending from Total to the Bridge	0.49	0.019	0.9	0.020

Calculations

Areas contributing flow	$T_c=\left[\frac{0.87L^2}{1000\,S_{av}}\right]^{0.385}$, (in hour)	Ic(mm/hr), from IDF curve						Qp(m3/s)=0.278*C*Ic*A					
		T=2yr	T=5yr	T=10yr	T=25yr	T=50yr	T=100yr	T=2yr	T=5yr	T=10yr	T=25yr	T=50yr	T=100yr
Sub-catchment-01	0.212	65.3	82.9	94.8	109.9	121.5	133.2	2.5	3.1	3.6	4.2	4.6	5.1
Sub-catchment-02	0.210	65.6	83.2	95.2	110.4	122.0	133.8	1.6	2.0	2.3	2.6	2.9	3.2
On Asphalt from TOTAL	0.210	65.6	83.2	95.2	110.4	122.0	133.8	0.3	0.4	0.5	0.5	0.6	0.7
							Total Sum	**4.36**	**5.54**	**6.33**	**7.34**	**8.11**	**8.90**

Result

Return Period in years	Peak Run off (m3/s)
2	**4.36**
5	**5.54**
10	**6.33**
25	**7.34**
50	**8.11**
100	**8.90**