JIMMA UNIVERSITY

SCHOOL OF POST GRADUATE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIRONMENTAL ENGINEERING

MASTERS OF SCIENCE PROGRAM IN HYDRAULIC ENGINEERING

ASSESSMENT OF URBAN STORM WATER DRAINAGE :(A CASE STUDY OF MIZAN- AMAN TOWN, SOUTHERN ETHIOPIA)

BY
SINTAYEHU TEKA

A THESIS SUBMITTED TO THE SCHOOL OF GRADUATE STUDIES OF JIMMA UNIVERSITY IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTERS OF SCIENCE IN HYDRAULIC ENGINEERING

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MAIN ADVISOR: Dr. ZEINU AHMED (PhD.)

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February, 2020

Jimma, Ethiopia

DECLARATION

I, **Sintayehu Teka** declare that this thesis entitled: ASSESSMENT OF URBAN STORM WATER DRAINAGE:(A CASE STUDY OF MIZAN- AMAN TOWN, ETHIOPIA)" is my own original work. It has not been submitted for similar or any other degree award in any other University. All the sources I have used or quoted have been indicated and acknowledge by complete reference.

_______________ _______________

Signature Date

APPROVAL SHEET

The undersigned certify that the thesis entitled: "ASSESSMENT OF URBAN STORM WATER DRAINAGE:(A CASE STUDY OF MIZAN- AMAN TOWN, ETHIOPIA" is the work of Sintayehu Teka. It has been accepted and submitted for examination with my approval as university advisor in partial fulfillment of the requirements for Degree of Master of Science in Hydraulic Engineering.

Main-Advisor: Dr. Zeinu Ahmed ______________ Signature ______________ Date

Co -advisor: Mr. Walabuma Oli ______________ Signature ______________ Date

As member of Board of Examiners of the MSc.Thesis Open Defense Examination, we certify that we have read, evaluated the thesis prepared by Sintayehu Teka and examined the candidate. We recommend that the thesis could be accepted as fulfilling the thesis requirement for the Degree of Master of Science in Hydraulic Engineering.

______________ External Examiner	______________ Signature	______________ Date
______________ Internal Examiner	______________ Signature	______________ Date
______________ Chair person	______________ Signature	______________ Date

ABSTRACT

One of the many complex problems resulting from increased urbanization is related to management of storm water from urban areas. Mizan Teferi town is surrounded by high forest which receives highest annual rainfall in every year. Due to this, severe floods occur frequently in every rainy season in some parts of the town. But the Existing drainage system is inadequate and it is not properly managed to carry the surface runoff. Therefore, this study focuses on assessment of urban storm water drainage conditions of Mizan Teferi town and then to propose the appropriate drain size with effective drainage capacity for the study area. Daily precipitation data, Land use land cover (LU/LC) and topographic data were the input data used for this study. The daily precipitation data from year 1995 to 2017 was obtained from the National Metrological Agency of Ethiopia.LU/LC and topographic data were obtained from Mizan Teferi town municipality service offices. Field observation was under taken in order to identify the existing drainage system problems of the town. The rational formula was used to estimate the peak discharge of the catchment which requires runoff coefficient, catchment area and rainfall intensity as an input data. The study area was delineated in to 10 sub-catchments using Arc GIS 10.1. The existing drainage outlets of the town were followed to delineate the catchments. The runoff coefficient for each sub-catchment was computed form the LU/LC of each plot of land. The Intensity Duration Frequency (IDF) curve was developed for 2, 5,10,25,50 and 100 years of return period. The visual information indicates that the observed major problems of the drainage system are Overflow from the channel, clogging of the channel by sediment, grasses and domestic solid waste. The performance value indicates that only 18.75% of the existing channels are ranked under very god condition. The capacity of the existing channels was computed using the Manning roughness formula and it was compared with the 25 years return period peak discharge. The result shows that only 30% of the existing channels can accommodate the 25 years return period.70% of the channels dimensions need modification to accommodate the 25 years return period peak discharge. Based on this, appropriate channel dimension has been designed based on the 25 years return period peak discharge for each catchment. The result indicates that, the existing peak discharge has occurred at catchment 4 which was 13.17 m^3/s whereas the 25 years return period of catchment 4 will be 20.339 m^3/s which requires to modify the channel depth, bottom width, and side slope. The finding can be used as an input data for Mizan Teferi town municipality for decision regarding modification and management of the existing drainage system. Finally, it is recommended to continuously monitor the existing or the modified drainage systems of the town.

Key words: Existing drainage system, IDF curve, Mizan Teferi town, Sub- catchments

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1. INTRODUCTION

1.1 Back ground of the study

Though water is very essential for all life on the earth, it can also cause devastation through erosion and flooding. Due to the development of infrastructures as a result of urbanization, the surface runoff water greatly flows over road surface in town and damages the road section.

The contributed runoff water, thus, need to be safely disposed to the rivers/outlet channels so that the functional utility of the road infrastructures maintained and there by avoid the damages which otherwise occurred to the road and property (Belete, 2011).

Storm water drainage networks in cities are usually designed to effectively collect and convey excess surface runoff in order to prevent urban flooding (Adisu, 2017). Often most of them face reduction of functionality and capacity for transferring the runoff flow, and their level of service reduces due to degradation in time, improper maintenance, inappropriate design, aging, sedimentation and siltation, increase in materials' roughness, and structural deterioration (Qianqian, 2014).

In addition, urban development and climate change exacerbate the situation because, such phenomena are followed by increase in runoff volume and peak flow rates. Despite development over the years, it remains a significant challenge to design an effective functioning drainage system. In particular, impacts due to climate change and urbanization have been widely acknowledged, which could entail a substantial increase in the frequency and magnitude of urban flooding in many regions of the world (Willems *et al*., 2012). Even, when there is a drainage system with acceptable functionality, the design capacity of the system is in adequate for extreme events and flood occurrence (Eyosias, 2018).

In urban areas, impermeability increases because of the increase in impervious surfaces. This in turn changes the drainage pattern, increases overland flow resulting in flooding and related environmental problems. The impact of this is severe on spatial structures like road and socio economic impact. This is because, flooding and its related environmental problems like sheet and gulley erosion, surface inundation tends to affect road services and its life span. Given the significance not only in socio-economic development, but also a path way for the location of other infrastructure, issues that affect its performance and longevity are critical areas of research (Dagnachew, 2011).

In the design of highway/access road, highway storm water drainage structures are extremely important component. Provision of adequate drainage is an important factor in the location and geometric design of highways.

Adequate level of service can be acquired by properly designing them. Initial cost, design life, and the risk of loss of use of the road way for a time due to runoff exceeding the capacity of the drainage structure, need to be considered in the design (Biniyam, 2016).

Asphalt road, Cobble stone road and gravel road are the major road types which are found in Mizan Teferi town. According to Ethiopian Road Authority, the road is a low volume road and it is categorized under design standard six (DS6) or design class two (DC2). The study area is characterized by extended and large volumes of rainfall. According to ERA geometric design manual 2011 for low volume roads, DC2 low volume roads carry 25-75 vehicles per day and the road is classified under feeder road. Among dense access road for some part was providing Trapezoidal drainage system. At some stations, the drainage structures are lacking even if they are required for the drainage purpose.

Many Studies done on drainage system of Mizan Teferi town were planned to reviewed and learn a gap regarding to the storm water drainage system problem. Unfortunately, there are no studies done before on the assessment of the storm water drainage system in Mizan Teferi town. Therefore, the main objective of this study is to assess the urban storm water drainage problems and to suggest design modification based on the existing problems of the drainage system.

1.2 Statement of the problem

Urban drainage systems are generally designed to drain out surface runoff from urban areas during storm events. However, storm water exceeding the drainage capacity can cause urban flooding and result in traffic interruption, socio-economic losses and health issues. An increase in impervious land use leads to more surface runoff, faster runoff concentration and higher peak flow rate (Stewart and Hytiris, 2018). Thus, there is an increasing need to improve drainage capacity to reduce flooding in rapidly urbanizing areas. Conventionally, the improvement of drainage capacity relies on expanding and upgrading the existing storm drainage system (Adisu, 2017).

Lack of Urban Storm water drainage (USWD) management represent one of the most common sources of compliant from the residents in many urban centers of Ethiopia, and this problem gets worse and worse with the rate of urbanization (Dagnachew,2011). In addition to increased densification and impermeability of the urban landscape, the planning as well as implementation of storm water protecting structures is insufficient.

Mizan Aman town is geographically situated in a region that is influenced directly by the southwest monsoon. It is surrounded by high forest which receives highest annual rainfall in every year. Since storm water increases due to the rapid growth of urbanization, severe floods occur frequently in every rainy season in some parts of the town. But, little part of the town is covered by adequate drainage system.

In the last decades, little numbers of urban drainage structures are constructed in the town. But visual information indicates that most of them were designed under inadequate hydrological data condition and hydraulic analysis. Presently, base failure, Depressions, Shoving, Edge crack, Shoulder erosion, Abetment damage, silted drainage ditches and flooding are some of the major problems that have been observed in the Mizan- Teferi. The urban drainage of these structures failed to deliver the design yields due to the major problems such as; sediments, less capability and failure-stability problems. Therefore, in this study, the existing overall drainage system condition and its performance were assessed and appropriate drainage system has been modified and recommended.

1.3 Objectives

1.3.1 General objective

The general objective of this study is to assess the storm water drainage problem of Mizan-Teferi Town.

1.3.2 Specific objective

1. To assess the existing condition and problems related to storm water drainage of Mizan Teferi town.
2. To assess the performance of existing drainage systems of the town.
3. To modify and recommend appropriate capacity and type of drainage structure for the town.

1.4 Research questions

1. What are existing condition and problems related to storm water drainage of Mizan Teferi town?

2. What is the performance of existing drainage systems of the town?
3. What is the appropriate recommendation?

1.5 Scope of the study

This study specifically focuses on the assessment of the existing drainage system performance and problems. It relies on the design modification of the existing drainage system of the town. But this study does not include structural design of all types of drainage structures except proposing the type and size of required drainage facilities hydraulically.

1.6 Significance of the study

The benefit that will be draw from this study may contribute to current efforts by governments and other concerning body to solve the problem of drainage schemes that contribute for better service coverage. It can also help to understand the problems of damage and preserve the structures by avoiding further deteriorations for taking correct measures as well as to reduce any inconvenience and disruption to travel due to over flow of water on the main road because of flooding.

The concerned body of Mizan - Teferi town will use it as reference while they are preparing their annual plans in relation to spatial and financial plans for roads and urban storm water drainage infrastructure. They can also use it as a further reference to fill the existing gap between road and urban storm water drainage demand and supply.

2. LITERATURE REVIEW

2.1 Storm Water Runoff

Storm water runoff is the direct response of a watershed to precipitation and includes the surface and subsurface runoff that enters a ditch, storm drain, stream or other concentrated flow during and following the precipitation. Runoff that occurs on surfaces before reaching a channel is also called non-point source pollution.

In urban areas storm water is generated by rain runoff from roofs, roads, driveways, footpaths and other impervious or hard surfaces. Poorly managed storm water can cause problems on and off site through erosion and the transportation of nutrients, chemical pollutants, litter and sediments to waterways. Well-managed Storm water can replace imported water for uses where high quality water is not required, such as garden watering (Hatt, B, *et al*., 2004).

Floods generally develop over a period of days, when there is too much rainwater to fit in the rivers and water spreads over the land next to it (the floodplain). However, they can happen very quickly when lots of heavy rain falls over a short period of time. These flashfloods occur with little or no warning and cause the biggest loss of human life than any other type of flooding (Vent cow, 1988).

2.2 Flood and Flooding

Flooding is a natural process and part of the hydrological cycle of rainfall, surface and groundwater flow and storage. Floods occur whenever the capacity of the natural or manmade drainage system is unable to cope with the volume of water generated by rainfall (Butler and Parkinson, 2009).

Floods vary considerably in size and duration (Heimhuber, 2013). In the design of roadway drainage structures, floods are usually considered in terms of peak runoff. For drainage facilities, which are designed to control volume of runoff, like detention facilities, or where flood routing through culverts is used, then the entire discharge hydrograph will be of interest.

The analysis of the peak rate of runoff, volume of runoff, and time distribution of flow is fundamental to the design of drainage facilities. Errors in the estimates will result in a structure that is either undersized and causes more drainage problems or oversized and costs more than necessary.

In the hydrologic analysis for a drainage facility, it must be recognized that many variable factors affect floods. Some of the factors which need to be recognized and considered on an individual site by site basis includes rainfall amount and storm distribution, drainage area characteristics (size, shape and orientation), ground cover, type of soil, type of terrain and stream(s), antecedent moisture condition, storage potential (over bank, ponds, wetlands, reservoirs and channel types (Kokeb,2016).

2.3 Hydrologic Methods

For the hydraulic evaluation of drainage systems of channels, storm drains, culverts, and median drains, the peak flows and hydrographs are determined by appropriate Hydrologic methods. The peak flow from a drainage basin is a function of the basin's physiographic properties such as size, shape, slope, soil type, land use, as well as climatic factors such as selected rainfall intensities (Desalegn,2011). Therefore, any hydrologic method should incorporate the basic physiographic properties and climatic factors of the watershed. The two major types of hydrologic methods mostly used for design of urban storm drainage structures are rational method and (NRCS) or SCS Method (ERA, 2008).

2.3.1 Rational Method

The Rational method is one of the most commonly used simplified models for road storm drainage, is primarily based on the concept that the peak discharge from a watershed will always occur when the rain lasts long enough at its maximum intensity to enable all portions of the basin to contribute to the flow (Biniyam, 2016).

It is an empirical formula relating runoff to rainfall intensity which is expressed in the Eq. (2.1).

$$Q = 0.00278CIA \qquad 2.1$$

Where; Q is peak flow (m3/s), A is drainage area (hectares), C is runoff coefficient (weighted) and I is rainfall intensity (mm/hr)

The Basic Assumptions involved in rational method are (1) the peak rate of runoff (Q) at any point is a direct function of the average rainfall intensity (I) for the time of concentration (Tc) to that point. (2) The recurrence interval of the peak discharge is the same as the recurrence interval of the average rainfall intensity. (3) The time of concentration is the time required for the runoff to become established and flow from the most distant point of the drainage area to the point of discharge.

2.3.2 US Natural Resources Conservation Service (NRCS) or SCS Method

Techniques developed by the U.S. Natural Resources Conservation Service (NRCS), formerly the U.S. Soil Conservation Service (SCS) for calculating rates of runoff require the same basic data as the Rational Method. However, it is more sophisticated in that it considers also the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm (Subermaniya, 2008).

In this method, the major factors that affect runoff generation; soil type, land use and treatment, surface condition and antecedent moisture condition are incorporated in a single CN parameter.

In this method, the precipitation excess is a function of cumulative precipitation, soil type, land use/cover and antecedent moisture. Considering the initial loss and the potential maximum retention, the precipitation excess can be calculated. The maximum retention and the basin characteristics are related through the curve number by Eq. (2.2).

$$Q = \begin{cases} \frac{(P-I_a)^2}{(P-I_a+s)} & \text{for } p > I_a \\ 0 & \text{for } P < I_a \end{cases} \qquad 2.2$$

Where: P= is the precipitation (mm), S= is the soil maximum retention (mm), I_a =is all loss before runoff begins. It includes water retained in surface depressions, water intercepted by vegetation, evaporation and infiltration.

To remove the necessity for an independent estimation of I_a , a linear relationship between I_a and S was suggested by Mishra and Singh, (2013) as Eq.(2.3).

$$I_a = s\lambda \qquad 2.3$$

Where: λ= is an initial abstraction ratio. The values of λ vary in the range of 0 to 0.3 and have been documented in a number of studies encompassing various geographic locations (Mishra and Singh, 2013). Through studies of many small agricultural catchments, I_a was found to be approximated by the empirical Eq.(2.4)

$$I_a = 0.2s \qquad 2.4$$

By removing I_a as an independent parameter, a combination of S and P to produce a unique runoff amount can be approximated as Eq. (2.5) and Eq. (2.6):

$$Q = \frac{(P-0.2S)^2}{(P+0.8S)} \qquad 2.5$$

$$S(mm) = \frac{25400}{CN} - 254 \qquad 2.6$$

Where: CN denotes the curve number for each soil types of the sub-basin. Other variables were defined in Eq. (2.2).

Two types of hydrographs are used in the NRCS procedure, unit hydrographs and dimensionless hydrographs. A unit hydrograph represents the time distribution of flow resulting from 25.4 mm of direct runoff occurring over the watershed in a specified time. A dimensionless hydrograph represents the composite of many unit hydrographs. The dimensionless unit hydrograph is plotted in non-dimensional units of time versus time to peak and discharge at any time versus peak discharge.

Steep slopes, compact shape, and an efficient drainage network tend to make lag time short and peaks high; flat slopes, elongated shape, and an inefficient drainage network tend to make lag time long and peaks low.

The NRCS method is based on a 24-hour storm event, which has a certain storm distribution. To use this distribution, it is necessary for the user to obtain the 24-hour rainfall value for the frequency of the design storm desired.

Several techniques have been developed and are currently available to engineers for the estimation of runoff volume and peak discharge using the NRCS methodology. Some of the commonly used of these methods are: the NRCS Technical Release 55 (TR-55), U.S. Army Corps of Engineers HEC-1 Model, the NRCS TR-20 Model and the NRCS TR-55 (Eyosiyas, 2011).

Some of these models utilize computer programs/rainfall-runoff simulation models which use a storm hydrograph, runoff curve number and channel features to determine runoff volumes as well as unit hydrographs to estimate peak rates of discharge.

2.4 Return periods

A flood peak does not occur with any fixed pattern in time or magnitude. Time intervals between floods vary. Return period is the average of these inter-event times between flood events. Large floods naturally have large return periods and vice versa. The definition of the return period may not involve any reference to probability. However, a relationship between the probability of occurrence of a flood and its return period can be justified. A given flood with return period T may be exceeded once in T year (Adugna, 2011).

The selection of return period for a given hydraulic structure is dependent on the size, type and useful life of the structures. Table 2.1 shows different return period for different types of structures.

Table 2. 1: Recurrence interval and Land use/Land cover description Source (ERA, 2011).

Land Use/ Facility Description	Recurrence Interval
Residential, recreational, open space	2 years/5 years
Commercial/Business, dense residential, small detention/retention facilities	10 years
Main collector drainage lines cross drain pipes of collector roads/a highway. Culverts under local and collector streets and small embankments. Also, pipes along a highway that conveys runoff to the disposal point or a waterway.	20 years/25 years
Bridges/large culverts along major arterials or high ways, any drainage or flood protection facility/dam along rivers, or other relatively larger water bodies.	50 years to 200 years

2.5 Rainfall Intensity

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

Rainfall Intensity is a function of geographic location, design exceedance frequency (or return interval), and storm duration. The magnitude of the will be Carried out by using return period maps and intensity-duration-frequency (IDF) Curves. The IDF relationship is a mathematical relationship between the rainfall intensity, the duration, and the return period (the annual frequency of exceedance).

2.6 Time of Concentration (Tc)

The time of concentration is the time required for the runoff to become established and flow from the most distant point of the drainage area to the point of discharge. Different factors affect time of Concentration in different way some of this are:

a) Surface Roughness: One of the most significant effects of development on flow velocity is less retardance of flow. That is, undeveloped areas with very slow and shallow overland flow through vegetation become modified by development; the flow is then delivered to streets, gutters, and storm sewers that transport runoff downstream more rapidly. Travel time through the watershed is generally decreased.

b) Channel Shape and Flow Patterns: In small watersheds, much of the travel time results from overland flow in upstream areas. Typically, development reduces overland flow lengths by conveying storm runoff into a channel as soon as possible. Since channel designs have efficient hydraulic characteristics, runoff flow velocity increases and travel time decreases.

c) Slope: Slopes may be increased or decreased by development, depending on the extent of site grading or the extent to which storm sewers and street ditches are used in the design of the storm water management system. Slope will tend to increase when channels are straightened and decrease when overland flow is directed through storm sewers, street gutters, and diversions.

d) The method of Time of Concentration: Water moves through a watershed as sheet flow, street/gutter flow, pipe flow, open channel flow, or some combination of these. Sheet flow is sometimes commonly referred to as overland flow. The type of flow that occurs is a function of the conveyance system and is best determined by field inspection, review of topographic mapping and subsurface drainage plans.

2.7 Types of storm water drainage system

The primary purpose of road drainage structure is to serve as conveyance structures preventing water from pooling on the roadway surface. Effective drainage structures prevent overland runoff from reaching the roadway, as well as drain water from the road surface. (Anteneh, 2015).

A drainage system will include all the components needed to ensure that the substructure is properly drained, and may be formed of components such as, open ditches, closed ditches with pipe drains, Drainage through storm water drainage pipes, Channels and culverts (Kokeb, 2016).

Provision shall be made to remove runoff from streets into drainage channels, watercourses, and pipe systems at low points and at intervals that will assure that ponding of storm water on streets does not occur for long durations. The maximum depth of storm water flow on any street shall not exceed 0.3 m, with a maximum flow velocity of 2 m/s (ERA,2013).

For storms greater than the design storm of the minor drainage system (i.e. a storm event with a return period in excess of 5 years), streets could be designed to temporarily convey flow as part of the major drainage system. The flow conveyance capacity of street shall be determined using the Manning Equation, with a Manning's resistance coefficient of 0.013 (asphalt surfaces) or 0.015 (concrete surfaces).

For storms up to and including the 5-year-return-period storm, the Designer must ensure that, for all roads, a travelled way of adequate width is maintained to ensure the safe passage of all vehicles in both directions. For residential streets and local collector streets, the Designer must ensure that during storms up to and including the major design storm (1.2 times the 100-year-return-period storm), the depth and spread of flow does not exceed the curb height and does not exceed the right-of-way width (ERA,2011).

For major collector streets and arterial streets (emergency access routes), the Designer must ensure that during storms up to and including the major design storm (1.2 times the 100-year return-period storm), a travelled way of adequate width is maintained to ensure the safe passage of vehicles in both directions (ERA,2011).

2.8 Functions of storm water drainage system

One of the drainage system's functions is to collect surface water and/or ground water and direct it away, thereby keeping the ballast bed drained. The drainage system must also protect the substructure from erosion, from becoming sodden, and from losing its load-bearing capacity and stability (Alejo and Ayodele, 2018). Another main objective of storm sewer is to protect Public health and safety, Environmental protection and Sustainable development.

Storm water drain networks in cities are usually designed to effectively collect and convey excess surface runoff in order to avert urban flooding (Gouri and Srinivas, 2015). Often most of them face reduction of functionality and capacity for transferring the runoff flow, and their level of service reduces due to degradation in time, improper maintenance, inappropriate design, aging, sedimentation and siltation, increase in materials' roughness, and structural deterioration (Negin *et al.,* 2016).

Drain and Sewer systems are provided in order to prevent spread of disease by contact with fecal and other waterborne waste, to protect drinking water sources from contamination by waterborne waste and to carry runoff and surface water away while minimizing hazards to the public. Additionally, the impact of drain and sewer systems on the receiving waters shall meet the requirements of any national or local regulations or the relevant authority (Biniyam, 2016).

2.9 Hydraulics of Storm Drainage Systems

2.9.1 Design Frequency

A design frequency shall be selected commensurate with the facility cost, volume of traffic, potential flood hazard to property, expected level of service, strategic considerations, and budgetary constraints, as well as the magnitude and risk associated with damages from larger flood events.

With long highway routes having no practical detour, where many sites are subject to independent flood events, it may be necessary to increase the design frequency at each site to avoid frequent route interruptions from floods. When selecting a design frequency, potential upstream land use which could reasonably occur over the anticipated life of the drainage facility shall be considered (ERA, 20011).

2.9.2 Hydraulic Capacity

The hydraulic capacity of a storm drain is controlled by its size, shape, slope, and friction resistance. Several flow friction formulas have been advanced which define the relationship between flow capacity and these parameters. The most widely used formula for gravity and pressure flow in storm drains is Manning's Equation (ERA, 2008).

2.9.3 Hydraulic Design Elements

General principles relating to channels, culverts, bridges, and other storm drainage elements are (1) the design of artificial drainage channels or other facilities should consider the frequency and types of maintenance expected and make allowance for access by maintenance equipment. (2) A stable channel is an important aspect for a proper functioning of highway drainage structures. (3) The range of design channel discharges shall be selected and based on Geometric Design Standards, consequences of traffic interruptions flood hazard risks, economics, and local site conditions. (4) Coordination with Ministry of Water Resources shall have high priority in the planning of highway facilities (Anteneh,2015).

2.10 Previous studies

Even though there is no study conducted on the road drainage system of Mizan Teferi town, there are a couple of studies which were conducted on different town of Ethiopia. The following are some of the privies studies that were conducted on road drainage system of Ethiopian town.

Habtamu, 2017 investigated the storm drainage problem of Zenebewerk (located in West Addis Ababa, Ethiopia), The author has stated that the main causes of flooding over the road and elsewhere were found to be surface flows from the study catchment area which were not collected and disposed properly; most of the inlets by the sides of the road were closed by sediments; cross slopes of the asphalt road were not proper such that asphalt sheet flow did not escape to inlets; and big runoff from neighboring catchment forced to flow upward and discharged over the asphalt.

The author has finally recommended that, for the road to serve its 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.

Eyosias, 2018 assessed the Performance of drainage systems of Debere Berhan town in amhara region of Ethiopia using SWMM5 Model. The occurrence of Street flooding and over topping drainage system problems during rainy season in the town was the major initial problems for the study. The model simulation has also proved the initial Hypothesis of the author.

Biniyam, 2016 studied the storm water drainage system of Kemise town which was found in amhara region of Ethiopia. Based on the author's finding, the drainage system of the town was insufficient at different area. The existing drainage convening capacity is only 19% at different catchment.

Improper construction alignment problem was another problem identified for the existing drainage system. The author has proposed appropriate mitigation measures for the remaining (81%) of new drainage system in order to serve the area from different negative effect and drainage structures for the future purposes sustainably. Periodic cleaning and modification of slope was recommended by the author for the existing drainage system.

Takhellambam, 2016 Investigated the Urban Drainage System of Sululta City in Ethiopia. The existence of Flooding and water logging were the initial problem of the study for the author. Based on the study, only 37.12 % of the total length of the road in the city was covered by drainage facilities. This indicated that the drainage system in the Sululta city was inadequate.

Common people were suffering due to lack of drainages facility. Finally, the author has recommended that the Sululta City Administration has to plan out to solve the drainages problems and to facilitate adequate drainage system.

Tamene and Getachew, 2015 assessed the road surface water drainage problems and its net-work integration systems in Ginjo Guduru Kebele of Jimma town. From the study, it was proofed that the road surface drainage system was found to be inadequate due to insufficient road profile, insufficient drainage structures provision, improper maintenance and lack of proper interconnections between the road and drainage infrastructures thereby resulting damages to road surface material and flooding in the area. Therefore, the findings of different authors indicate that the major part of Ethiopian city road drainage system has inadequate capacity. The performance of the existing road drainage system in Ethiopia is poor.

The studies indicate that improper construction of road drainage system, sediment deposition, construction alignment problem and lack of improper and periodic maintenance are the major problems of the existing road drainage system of Ethiopian town. Proper design and construction, periodic cleansing and maintenance of road drainage system of Ethiopian city are the common recommendations agreed by the authors.

3. MATERIALS AND METHODS

3.1 Description of the study area

3.1.1 Location and climatic conditions of the study area

Mizan-Teferi city is located in the Southern Nations, Nationalities and People's Region. It is one of the zonal city. The city is suited at a distance of 568 and 836 kilometers south west of Addis Ababa and Hawassa respectively. The city is also found at a distance of 50 and 230 km from Tepi and Jimma respectively. The city boarded by seven peasant Association north by Garkin, north east and east by Kosokol, in the south by Shonga, Zemika and south west by Mashimbaye.

Geographically, Mizan Teferi town is suited 06°04′80″ north latitude and 35°02′6″, east longitude. Figure 3.1 shows the location map of the study area. The climatic condition ranges from sub-humid warm to hot which has average daily temperature of 26.65 °C and annual maximum rainfall of 848 mm.

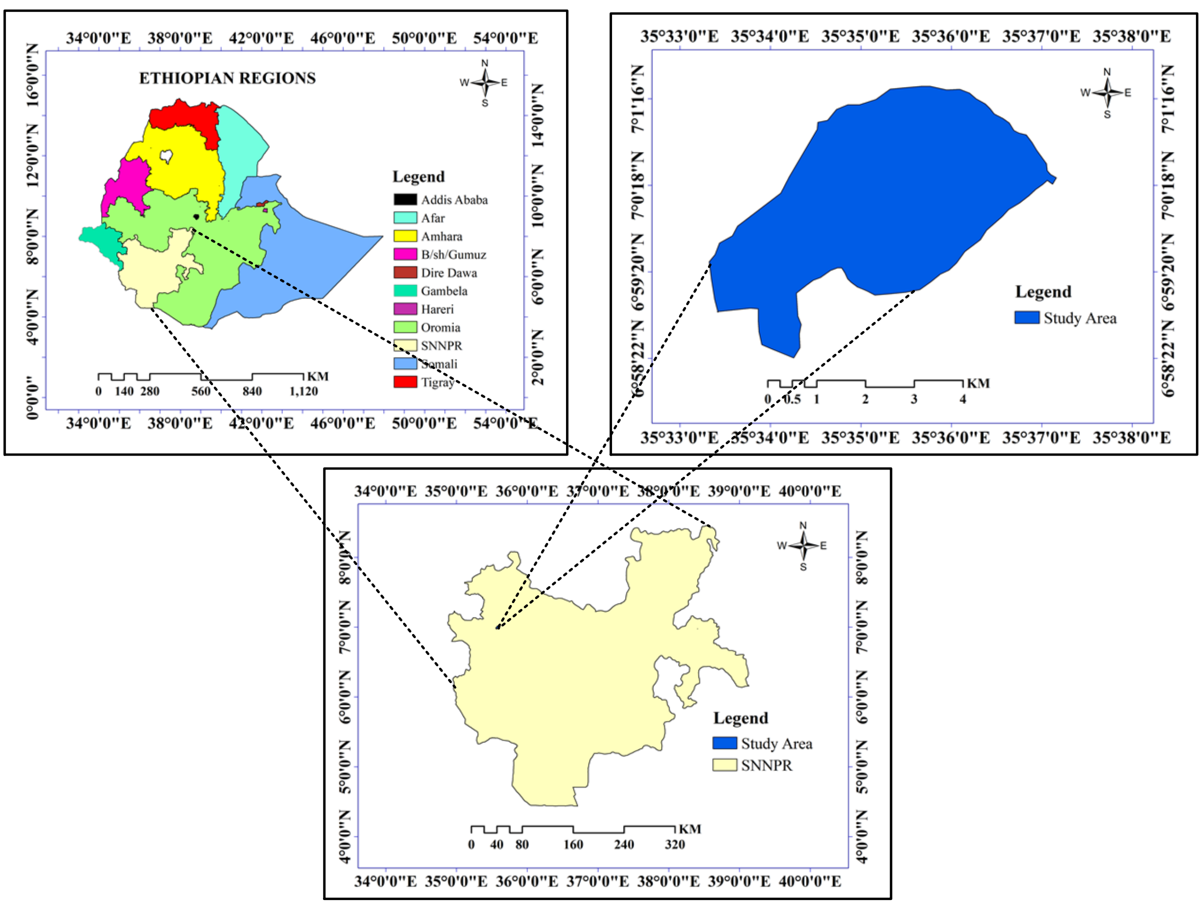

Figure 3. 1: Location map of the study area

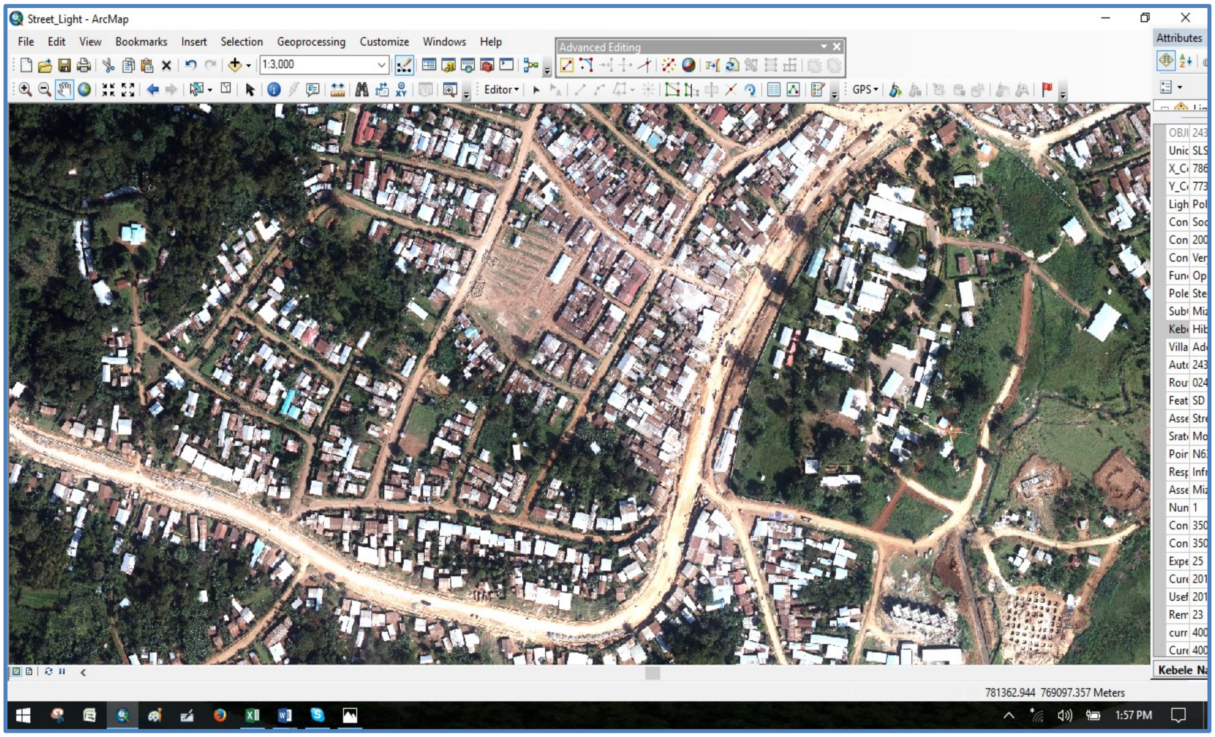

Figure 3. 2: Satelite image of Mizan-Teferi sub- City in Bench Maji Zone.

3.1.2 Population Size of the City

According to the recent population and housing census results (2007 CSA preliminary), the total population size of the city is79,581 from which male account 40,678 and female 38,903. The estimated population growth rate is 4.8percent. Regarding age structure, 36 % of the total population of the city falls within the age bracket of 0-14 years. The majority of the population (62 %) belongs to the working age group (between 15- 64), and the group with 65 years and above represent only1 percent. From the total population 48.10% are males and 51.90 % are females. The age15-64 years are economically active age group. The Dependency Ratio used to measure the economic burden that the population must be carry. From the last many years' experience the computation.

3.1.3 Economic Activity and Industrial Areas of the City

Mizan-Teferi city is one of the business areas in SNNPR acts as commercial center. The city functions mainly as a market and commercial center along all direction. Economic activities are a mix of administrative functions, local trade and commerce, plus a small amount of agro processing.

The city is the central commercial site for both agricultural and industrial products. Specially, fruit, spices and coffee are transshipped to national central markets from Mizan- Teferi city.

When one looks in to the migration status of Mizan- Teferi city. According to the economic activities of Mizan- Teferi city, with increasing size of city, some farming areas are engulfed and changed into built up area.

Around the city there is main coffee production areas and high supply of coffee to the central market. Commercial activities highly concentrated along the highway and feeder roads. There are one market places; at the center of city with a number of shops. And also open market every day as mini market. In these markets various kinds of fruits come from neighbor rural Kebele, and finished goods from central market of the Mizan- Teferi city.

Mizan Teferi has recently established industrial area that far 3 Kilo meter from the center of city. The site has a minimal leveling, power, water and telecommunication lines which is suitable for investors.

The city of Mizan- Teferi city like other Ethiopian urban centers is characterized by, lack/shortage of basic urban infrastructure and services. To have a sustainable infrastructure management, Asset management plan provides an integrative approach that links project-based capital investment planning with long term operation and maintenance needs.

It will also enable the city to build the knowledge of its infrastructure asset base, and improve capital investment planning, whilst at the same time creating effective strategies for long-term operation and maintenance of their infrastructures and services.

3.1.4 LU/LC and soil type of the city

Mizan- Teferi has urban agricultural land with dense habitation of small farms which are supported by very productive fruit and coffee, is the main cash crop. Soils are fertile and the sub-region enjoys a plentiful and extended rainy season. Urban agriculture is also the dominant land use of the study area.The analysis of LU/LC indicates that 53.15 % of the study area is used by urban agriculture. Figure 3.3 shows the LU/LC of the study area.

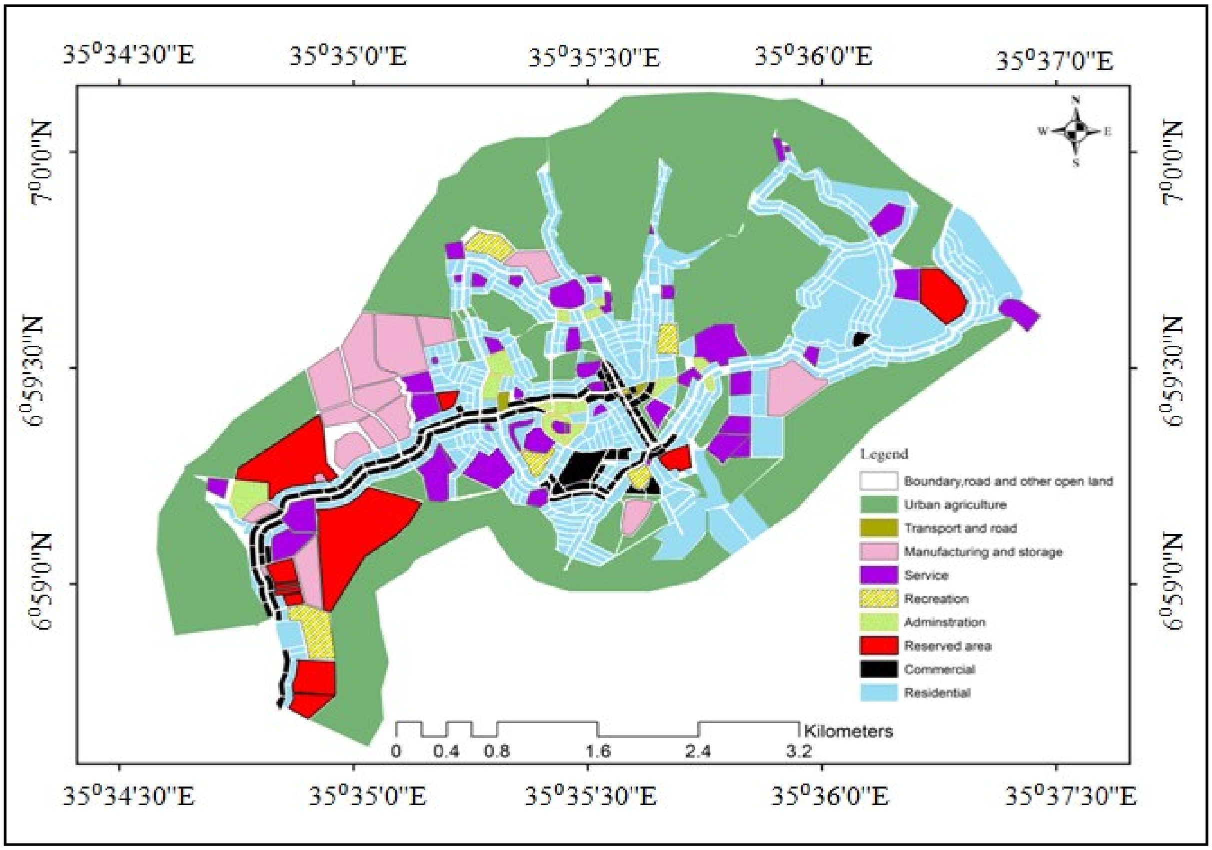

Figure 3. 3: LU/LC map of the study area

The only soil type of the study area is Dystric nitisols which belongs to HSG B. Figure 3.4 also shows the soil type of the study area which was extracted from the soil map of bench wereda.

The most dominant soil type in this study is Nitosols. Nitosols accommodates deep, well-drained, red, tropical soils with diffuse horizon boundaries and a subsurface horizon with more than 30% clay and moderate to strong angular blocky structure elements that easily fall apart into characteristic shiny, polyhydric (nutty) elements. Nitosols are strongly weathered soils but far more productive than most other read tropical soils. Generally, it is considered to be fertile soils in spite of its low level of available phosphorus and normally low base status.

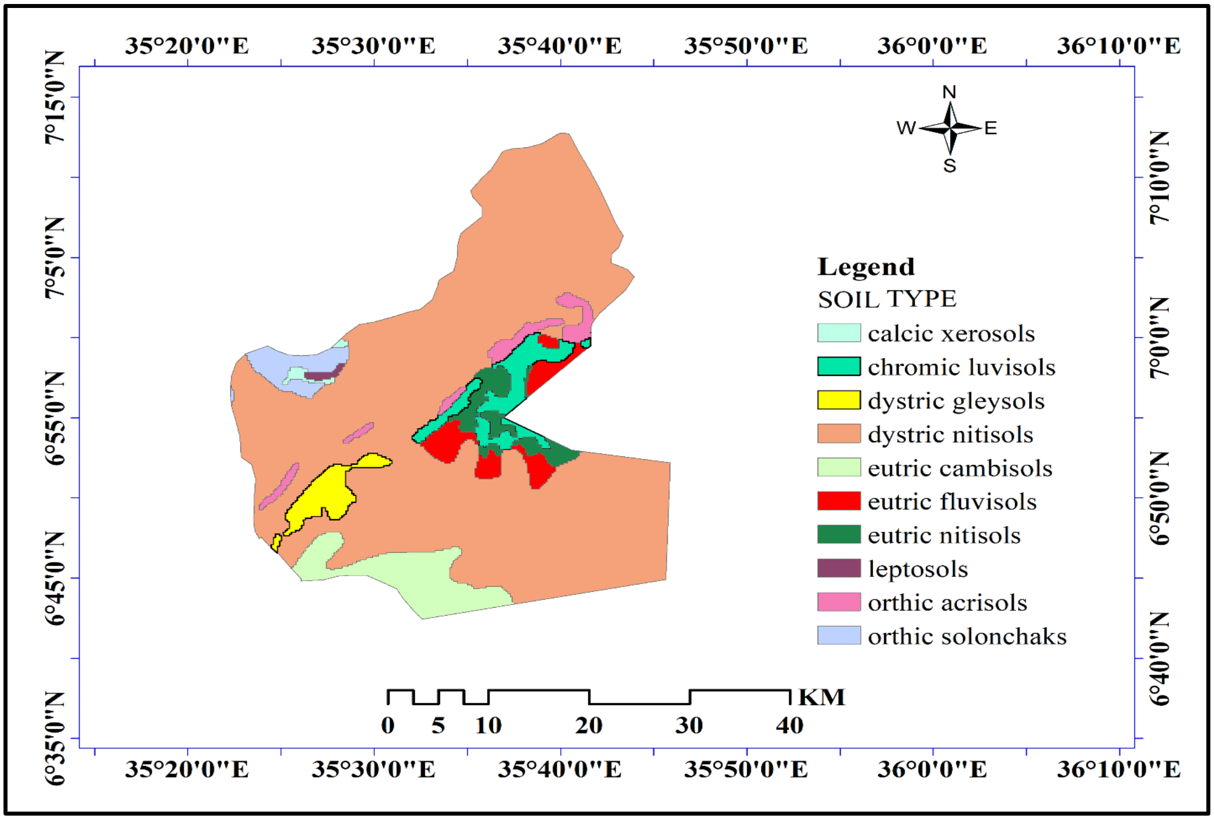

Figure 3. 4: Soil type of bench wereda

3.2 Material used during the study

Major types of software used for this study were ArcGIS, Excel spreadsheet and easy fit version 5.6. Additionally, Microsoft excel sheet and XLSTATE were used. Arc GIS version10.1 which is public domain software was developed by ESRI. It was released in June 2012. It was used for spatial data analysis and terrain preprocessing. Easy fit was used to select the best fit probability distribution of the daily precipitation data.

Excel spread sheet was used to calculate and analyze the runoff coefficient, the time of concentration, the intensity at different return period and the peak discharge vales for different return period. Additional software used during this study were Microsoft excel sheet for time series data analysis and XLSTATE2018 to fill the missing Hydro-metrological data.

3.3 Study design

The overall frame work of the methodology followed throughout the study is shown in Figure 3.5.

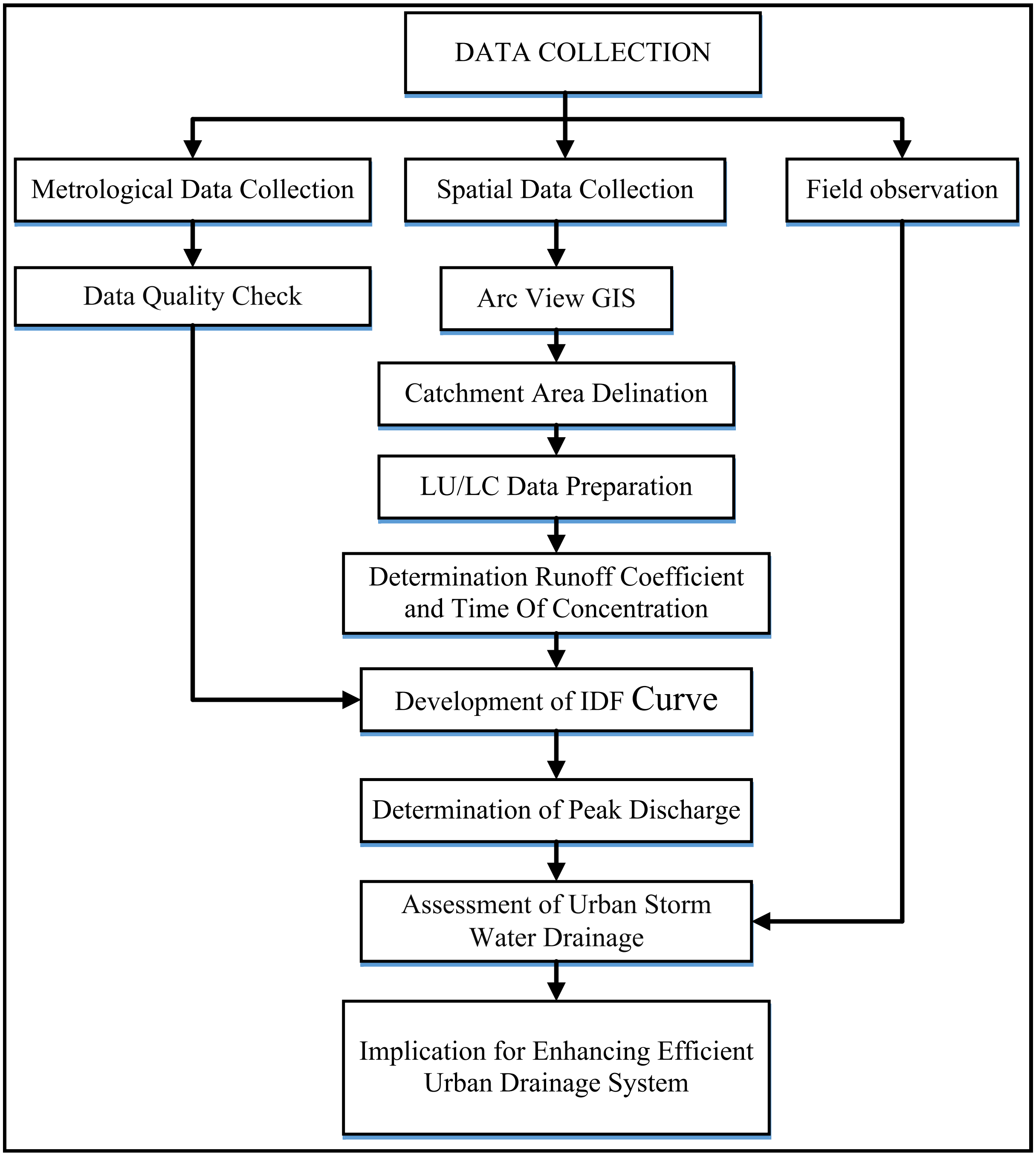

Figure 3. 5: Simplified representation of the study procedure

3.4 Data collection and analysis

3.4.1 Data collection

Input data used for this study is categorized into two. These are spatial data and physical data. Soil, LU/LC and DEM are categorized under spatial data, whereas metrological data can be categorized under physical data (Atnafu and Niguse, 2015).

The daily metrological data like minimum temperature, maximum temperature, precipitation, and relative- humidity and sunshine hours were collected from the NMAE. Metrological stations in and around the catchment were selected based on the number of missed data. A station having 85% of full recorded data were selected for this study. Metrological data from 1985 to 2017 for 4 stations were from Tepi, Bonga, Aman and Mizan teferi.

Field Observation has been carried out to observe the drainage infrastructures and its problems. The subjects of field observations were drainage lines and networks, land use of the city, streams in the City and waste management.

3.4.2 Metrological data analysis

a) Filling missing rainfall data

The continuity of the record may be broken with missing data due to failure of the observer to take reading at regular interval, vandalism of the recording gauges and instrumental failure. The rainfall data taken from the seven stations has missing data ranging from 4.4% to 13.5%. Therefore, it is required to estimate these missing records before undertaking further data analysis.

The arithmetic average, normal ratio and linear regression method was used to fill the missing rainfall data. When the average annual precipitation at the adjacent gauges differed from the average annual precipitation at the considered gauge by less than 10%, arithmetic average method was used. In this method, missing data is obtained by computing the arithmetic average of the rainfall data recorded nearest to the considered gauge. Mathematically, arithmetic average method can be expressed Eq. (3.1) (Sebarmanya, 2008).

$$p_x = \frac{p_1 + p_2 + \cdots p_m}{M} \qquad 3.1$$

If the average annual rainfall at any of the adjacent gauges and the considered gauge is greater than 10% a normal ratio method was used. Mathematically, normal ratio method can be expressed as Eq. (3.2):

$$p_x = \frac{N_x}{M}\left[\frac{P_1}{N_1} + \frac{P_2}{N_2} + \frac{P_3}{N_3} + \cdots + \frac{P_m}{N_m}\right] \qquad 3.2$$

Where: N1, N2, N3 & Nm represent the average annual rainfall at station 1, 2, 3 & m respectively. P1, P2 & Pm observed daily precipitation data for station 1, 2, 3 & m respectively, Nx = represents the average annual rainfall at the missing station, Px =

represents the required daily precipitation value at the missing station, M=represents the number of station. The estimated data is considered as a combination of parameters with different weights from the surrounding gauges.

Linear regression is used for modeling relationship between scalar dependent variable denoted by Y and one independent parameter denoted by X. Model that depends on linearly on their unknown parameters are easier to fit than model that are linearly related to their parameter because the statistical parameters are easier to determine (Tesfaye and Chane,201). Mathematically, the correlation coefficient can be expressed as:

$$R^2 = \frac{a\sum Y + b(\sum XY) - \frac{1}{N}(\sum Y)(\sum Y)^2}{\sum Y^2 - \frac{1}{N}(\sum Y^2)} \qquad 3.3$$

a Denotes the slope of the linear equation $Y = aX + b$ 3.4

$$a = \frac{\sum Y - b\sum X}{N} \qquad 3.5$$

b Denotes the y intercept of the above equation which is given by

$$b = \frac{N(\sum XY) - (\sum X)(\sum Y)}{N(\sum X^2) - (\sum Y^2)} \qquad 3.6$$

X and Y denotes the two neighboring gauges having missed and observed daily rainfall N denotes the number of observed daily rainfall from the two neighboring gauges. If the there is no observed rainfall data simultaneously at more than three stations and if the correlation coefficient (R^2) of the two station's rainfall data is greater than 0.7, linear regression method was used. The computation of arithmetic average, normal ratio and linear regression was done using Microsoft excel sheet and XLSTAT.

b) Consistency test for Rain fall data

If the characteristic of the recorded data has not changed with time, it is called consistent record. However, if the characteristics of the recorded data vary with time, it is called inconsistent record. Inconsistency may occur due to change in observation procedures, exposure of the gauge and land use. Adjustment of measured data is important to obtain a consistent data. It involves the estimation of effect rather than missing value (Tufa and Hailu, 2011).

Double mass curve is one of widely accepted method. It is used to check the consistency of a long-term trend test of hydrological and metrological data. The method is based on the fact that a plot of two cumulative data having the same recorded period exhibits a straight line as long as the proportionality between the two remains unchanged (Atnafu and Niguse, 2015). Therefore, a double mass curve analysis was selected for this study to test the consistency and adjust an inconsistent data.

According to Tesfaye and Chane (2011), a double-mass curve is a graph of the cumulative rainfall at the rain gauge of interest versus the cumulative rainfall of one or more gauges in the region with similar hydro-meteorological occurrences.

If a rainfall record is a consistent estimator of the hydro- meteorological occurrences over the period of record, the double-mass curve will have a constant slope. A change in the slope of the double mass curve indicates changes in the characteristics of the recorded values.

Hence, the record needs to be adjusted with either the early or later period of record by changing the values. After that, the slope of the resulting double-mass curve will be straight line. The rainfall records of a given station x are adjusted by multiplying the recorded values of rainfall by the ratio of slopes of the straight lines before and after change in environment. Mathematically, it can be expressed as;

$$Y2 = \left(\frac{S2}{S1}\right) Y1 \qquad 3.7$$

Where: Y2 = corrected precipitation at station x, Y1 = original recorded precipitation at station x, S2 = slope of double mass curve to be corrected, S1= original slope of double mass curve.

Based on the result rainfall data of Tepi and Bonga stations were consistent which have R^2 values of 0.999. The rainfall data at Mizan and Mizan Teferi were relatively inconsistent with R^2 values of 0.986 and 0.972 respectively. By double mass curve analysis, R^2 value was adjusted to 0.998 for both stations. Figure 3.6 shows the consistency test result for all stations.

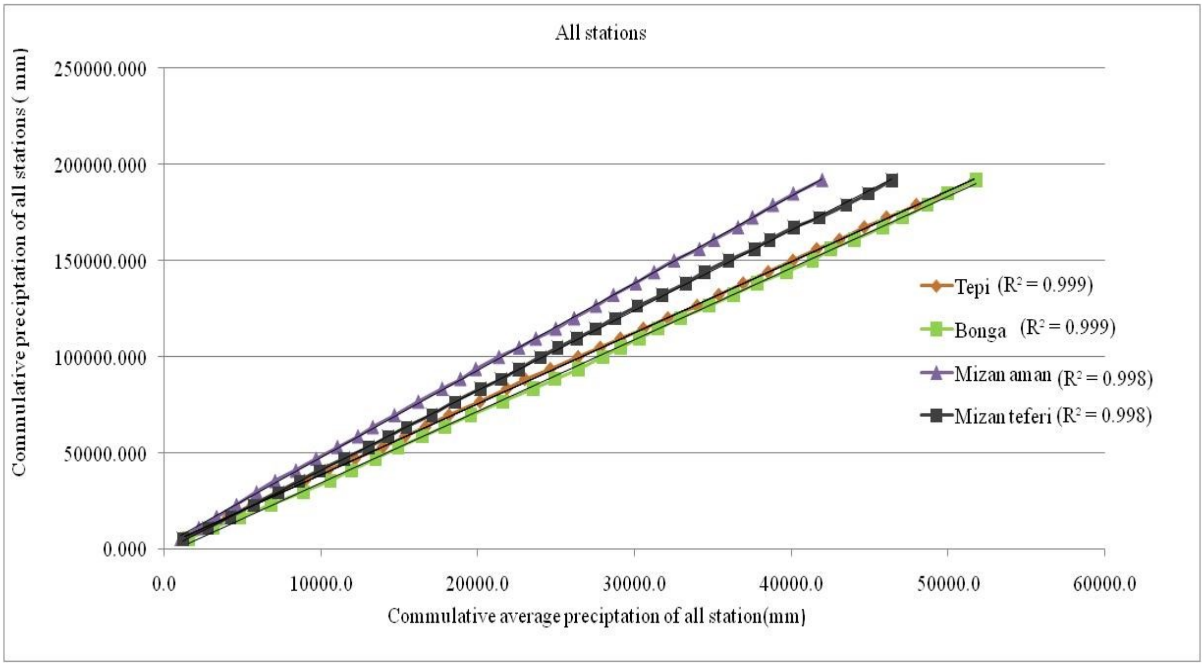

Figure 3. 6: Consistency test graph of stations

c) Outlier test for Rain fall data

Outliers are data points that deviate from the trend of remaining data. This may be due instrumental and personal errors or due to extreme metrological events (Raes, 2006).

When low outliers are definitely mistaken measurement, those outliers can be rejected and the problem will be converted in to one of missing data treatment. However, high outlier adjustment is generally considered a better procedure than excluding it from the sample except when a doubt exists that it is caused by measurement error (Ponce, 1989).

Keeping high outliers in the sample and treating them as an ordinary value seem to be a preferred approach because it is believed that they carry important information. On the other hand, keeping the high outliers may lead to over estimation of runoff.

According to (Raes., 2006).an option that would combine keeping the information of higher outliers and showing consideration for effects of higher outliers would be to censor higher outliers. This can be done by replacing them by some threshold value that keeps their information so that the effect on the sample statics is reduced.

The Grubbs and Beck (1972) test was used to detect outliers. This method was selected due to its simplicity and reasonable precision. The Grubbs and Beck equation used to test higher and lower outlier is given by Eq. (3.8) and Eq. (3.9).

$$X_H = \overline{X} + K_N S \qquad 3.8$$

$$X_L = \overline{X} - K_N S \qquad 3.9$$

Where: X_H = the higher outlier value, X_L = the lower outlier value, $\overline{X}$ is the mean of the logarithm of the sample, S = the standard deviation of the logarithm of the sample, K_N = critical deviate for sample size N. It is tabulated for various sample size and significance level (mostly 10% is used for outlier test). K_N value at 10% significance level was proposed by (Pilon et al., 1985) as Eq. (3.10).

$$K_N = -3.62201 + 6.28446N^{1/4} - 2.49835N^{1/2} + 0.491436N^{3/4} - 0.0379 \qquad 3.10$$

According to this test, sample values greater than X_H were considered as high outliers while those less than X_L, were considered as low outliers.

Based on the analysis, all precipitation data values were found within the limits of high and low outlier threshold values. The outlier test result graph for Aman station was shown in Figure 3.7. The remaining stations outlier test graphs were attached in Appendix-1.

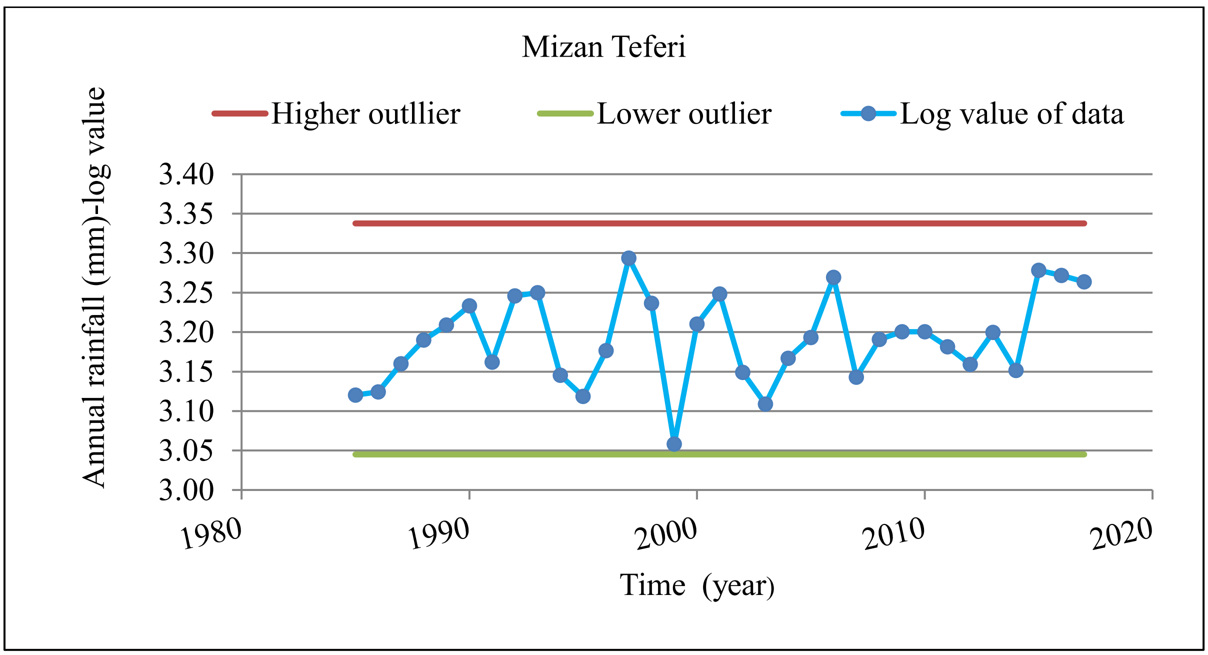

Figure 3. 7: Outlier test result graph

d) Homogeneity test for Rain fall data

Homogeneity test is necessary to detect the variability of the data. Homogeneity of a time series data indicates that the measurements of the data are taken at a time with the same instruments and environments. However, it is a hard task when dealing with rainfall data because there might be a certain error due to change in measurement and observational procedure, environment characteristics, and the location of measuring device.

There are various methods used for homogeneity test, but the most preferred methods are, standard homogeneity test and measuring cumulative deviation from the mean. XLSTATE can be used for standard homogeneity test, while Rainbow software can be used to measure the cumulative deviation from the mean (Raes *et al*., 2006).

For this study, homogeneity of the rainfall data was checked using Rainbow software. This software is designed to carry out frequency analysis and to test the homogeneity of climatic and hydrological data. It tests the homogeneity of a given data set based on the cumulative deviation from its mean (Raes et al., 2006).

$$S_k = \sum_{i=1}^{K}(X_i - \overline{X}) \qquad K=1, 2, 3..., n \qquad 3.11$$

Where: S_k = is the cumulative deviation, K = is the number of year, Xi = is series of rainfall data, $\overline{X}$ = is the mean of rainfall data

The initial value of S_k (for k=0) and the last value of S_k (for k=n) are equal to zero. When plotting the S_k's (also called a residual mass curve) changes in the mean are easily detected. For a record Xi above normal the S_k=i increases while for a record below normal, Sk=i decreases. For a homogenous record one may expect that the S_k's oscillate around zero since there is no regular pattern in the deviations of the Xi's from their average value X.

To test the homogeneity of the data set, the cumulative deviations are often rescaled. This is obtained by dividing the Sk's by the sample standard deviation value (s). By evaluating the maximum (Q) or the range (R) of the rescaled cumulative deviations from the mean, the homogeneity of the data of a time series can be tested. The equation can be expressed in Eq. (3.12).

$$\mathrm{Q} = \max\left[\frac{\mathrm{S_K}}{\mathrm{s}}\right] \qquad 3.12$$

$$\mathrm{R} = \max\left(\frac{\mathrm{S_K}}{\mathrm{s}}\right) - \min\left(\frac{\mathrm{S_K}}{\mathrm{s}}\right) \qquad 3.13$$

Where; Q = maximum cumulative deviation, R = the range of cumulative deviation, s = the sample standard deviation.

High values of Q or R are an indication that time series data is not from the same population and the fluctuations are not purely random.

The cumulative deviation versus time series graph of annual maximum daily rainfall for Mizan Aman station was drawn using rainbow software (Figure.3.8). In this graph, the vertical-axis is rescaled and lines representing various probabilities with which the homogeneity of the data can be rejected were plotted.

The graph indicates that the rescaled Sk's fluctuated around zero, and they were far off the lines where the homogeneity is rejected. Hence, the precipitation time series data were considered as homogeneous.

The homogeneity statistics menu Mizan teferi station (Figure 3.9) indicates that the cumulative deviation and maximum of cumulative deviation at 90%, 95% and 99% were not rejected. This also indicates the homogeneity of the precipitation time serious data.

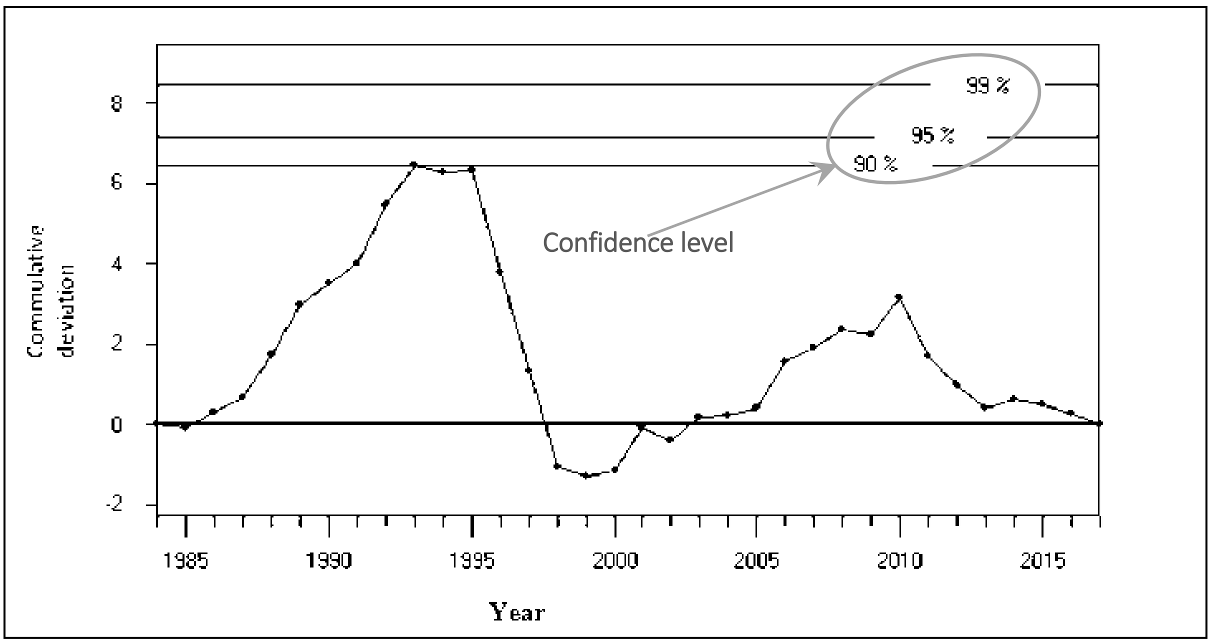

Figure 3. 8: Rescaled cumulative deviation from mean for annual rainfall of Mizan aman station

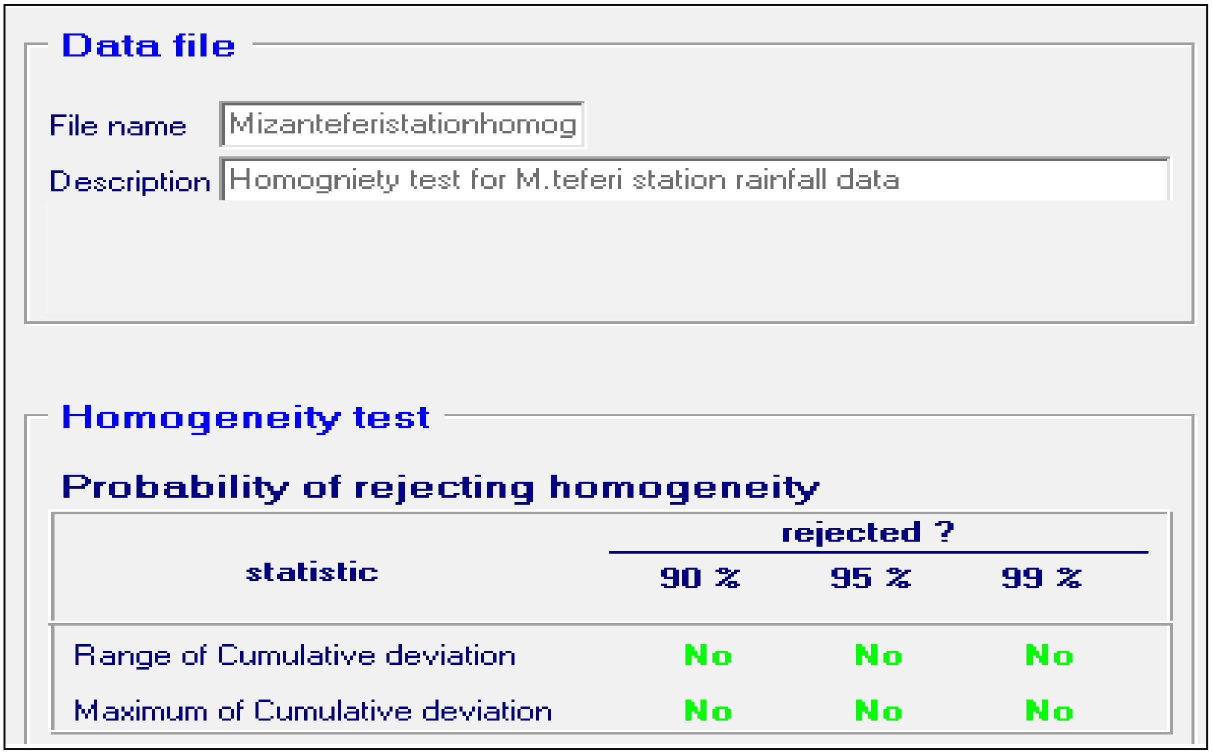

Figure 3. 9: Statistics showing probability of rejecting homogeneity

3.5 Frequency analysis of Rainfall data

Storms of high intensity and varying durations occur from time to time. However, the probability of these heavy rainfalls varies with locality. The first step in designing any hydraulic Engineering projects dealing with flood control is to determine the probability of occurrence of a particular extreme rainfall (Adugna, 2011). This information is determined by the frequency analysis of point rainfall data. Therefore, proper Estimation of extreme rainfall depths or intensities is also required for the design of drainage or sewer systems.

Several distribution functions can be selected to estimate extreme rainfall during the considered period. For this study, the best fit probability distribution function was selected using easy fit version 5.6 software.

The annual maximum daily rainfall data from year 1985 to 2017 was analyzed using different PDF by the software. The output of the analysis is the rank of different PDF based on their goodness of fit.

The result indicates that The Gumbel distribution function, which is skewed probability distribution function, was the first PDF that fits the data. (ERA, 2013) has also suggested that the Gumbel and Log Pearson Type III methods are best fit PDF for the frequency analysis of annual maximum daily rainfall data (Habtamu, 2017). The primary reason is Gumbel has a fixed value of skew. Therefore, for this study, the frequency analysis was conducted by using the Gumbel method.

3.6 Basic concept of IDF curve

IDF curve is a plot of rainfall intensity for different duration (Anila, 2013). Design of flood protecting structures require short duration (ranging from several minutes to hours) of rain fall data. This is due to the occurrence of flood for short period of time.

The rainfall depths obtained from gauging station are of 24 hr 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. ERA, 2013 has suggested the following reduction equation for any given time from 24-hour rainfall depth.

$$I_t = \frac{R_{24}(b+24)^n}{24(b+t)^n} \tag{3.14}$$

Where: I_t = Intensity (mm/hr), t =the required short duration (min), R24 = 24hr rainfall depth (mm), b and n are reduction constants which depends up on the topography of the area usually b = 0.3 and n = 0.92 are recommended (Habtamu, 2017).

According to (ERA, (2013), development of IDF curve for different return period for the required duration is the basic step for design of surface drainage system using rational method. For this study, the design storm obtained from frequency analysis was used to plot the IDF curve of Mizan -Teferi town.

In actual case, the probability of occurrence of flood will ranges from 2 years to 100 years. Miraf, 2011 has also stated the design of storm drainage should consider a minimum return period of 2 years and a maximum return period of 100 years. In between this the selection of return period should be based on the life span of the structure. Considering this, An IDF curve for 2, 5,10,25,50 and 100 years return period was developed for short term durations between 12 and 180 minutes at 15 minutes’ interval.

3.7 Estimation of peak discharge using rational method

Flood peak discharges estimation is the most essential step for storm channel design. The rational method provides the most reliable results when applied to small, developed watersheds and particularly to roadway drainage design. The method is recommended for size of drainage that ranges from 200 to 300 hectares (Mahari, 2015).

3.7.1 Advantage and disadvantage of rational method

The Rational Method is an adequate method for approximating the peak rate and total volume of runoff from a design rainstorm in a given catchment. The greatest drawback to the Rational Method is that it normally provides only one point on the runoff hydrograph. When the areas become complex and where sub-catchments come together, the Rational Method will tend to overestimate the actual flow, which results in over sizing of drainage facilities.

The Rational Method provides no direct information needed to route hydrographs through the drainage facilities. One reason the Rational Method is limited to small areas is that good design practice requires the routing of hydrographs for larger catchments to achieve an economic design. Another disadvantage of the Rational Method is that with typical design procedures one normally assumes that all of the design flow is collected at the design point and that there is no water running overland to the next design point. However, this is not the fault of the Rational Method but of the design procedure.

From the parameters of the rational formula only the area 'A' can be precisely defined by measuring the area of the sub-catchment. The intensity 'i' for the sub-catchment depends on the analysis of point rainfall data, time of concentration etc.

The most cumbersome process is the estimation of runoff coefficient 'C' which comprises so many characteristics on the sub-catchment like land use pattern, antecedent precipitation, soil moisture, infiltration, ground slope, surface and depression storage, shape of the drainage area, over land flow velocity and this coefficient can vary with time also.

The formula's main limitation is the use of C because the original tables were not created through the calibration of runoff coefficients in experimental basins but instead through consultation with experts. This approach can be ineffective considering that the estimation of C for observed data can be particularly variable and different from the classified values (Anteneh, 2015).

Furthermore, the soil use and soil type classification is limited and restricted to a few classes and, for each class; C is given as a range of values. This last point is pivotal because it introduces 'subjectivity' into the estimation of Q_p, and it is difficult to manage this subjectivity in practical use since the choice of C value cannot be related to a specific physical or conceptual hypothesis (Desalegn, 2011).

One of the most serious limitations of the rational method is that it does not take into consideration the real storm pattern. Thus the time variation of the rate of rainfall and the variation in area and velocity contributing the flow are therefore not accounted.

Although the Rational Formula has several drawbacks, it is reliable and surprisingly accurate considering the paucity of input information. It is the most applied equation in practical hydrology due to its simplicity and the effective compromise between theory and data availability.

The success of the Rational Formula is likely due to multiple factors. First, the formula is easy to apply and can be solved without the use of a computer. Second, the input data are easy to obtain, as the formula only requires the intensity–duration–frequency (IDF) curves, Tc and C. Finally, the formula is not entirely empirical.

The only physical catchment variable that can be assumed without measurement is the runoff coefficient which ranges from 0 to 1.

The SCS-CN method of run of estimation requires the assumption of curve number. In addition to the assumption, the method only considers the curve number for estimation of the peak discharge (Miraf, 2011).

Therefore, for this study, the rational method was selected to estimate the peak discharge of the study area. This method estimates the peak runoff using the Eq. (2.1) the rational formula.

Spatial data (LU/LC, soil and topographic) data analysis and preparation, Development of IDF curve for different short durations and return period, Determination of Time of concentration, Determination of weighed runoff coefficient and computation of design flow (Peak discharge) are the basic procedures involved in estimation of peak discharge using rational method.

3.7.2 Determination of Runoff Coefficient(C)

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).

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.

Therefore, a reasonable runoff 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. For this study, a weighted method is employed to obtain the representative runoff coefficient that can be expressed as: (Vent Chow 1988).

$$C_w = \frac{\sum_{i=1}^{n} C_i A_i}{\sum_{i=1}^{n} A_i} \qquad 3.15$$

Where, Cw is the weighted runoff coefficient and Ci is the runoff coefficient for individual area (Ai). Table 3.1 shows the ranges of runoff coefficients recommended by ERA (2013) to be taken during the estimation of peak discharge using rational formula.

Table 3. 1: The Rational Method of Runoff Coefficient for different LU/LC types (ERA, 2013)

LU/LC type	Runoff Coefficient(C)
Business: Downtown areas	0.7-0.95
Neighborhood areas	0.5-0.7
Residential: Single-family	0.3-0.5
Residential: Multi units, detached	0.4-0.6
Residential: Multi units, attached	0.6-0.75
Suburban	0.25-0.4
Residential (0.5 hectares lots or more)	0.3-0.45
Apartment dwelling areas	0.5-0.7
Industrial: Light areas	0.5-0.8
Heavy areas	0.6-0.9
Parks, cemeteries	0.1-0.25
Playgrounds	0.2-0.4
Railroad yard areas	0.2-0.4
Unimproved areas	0.1-0.3

3.7.3 Determination of Time of Concentration (Tc)

Different empirical equations of time of concentration could be used for inner and peripheral areas of urban centers. But the most preferred equation for urban drainage design are the airport or federal aviation administration Methods which could preferably be used for inner areas (for the developed areas of urban centers) and the SCS method, for cultivated areas.ERA,2013.

The equation for the airport and federal aviation administration method of Tc estimation is given by Eq. (3.16) and Eq. (3.17) respectively.

$$T_c = \frac{3.64(1.1 - C)L^{0.83}}{H^{0.33}} \quad 3.16$$

$$T_c = \frac{1.8(1.1 - C)l^{0.5}}{S^{0.333}} \quad 3.17$$

Where: Tc is the Time of Concentration (hr), L is the Flow length from the remotest point to the point of interest(km), H is the elevation difference (m), C is the rational method runoff coefficient, l is the length of overland flow(ft), and S is the surface slope (%).

The Airport formula is used when the land is covered more than 75% by impervious layer. The Federal Aviation Administration method was developed from air field drainage data assembled by the corps of Engineers. The method is intended for use on airfield drainage problems, but has been used frequently for overland flow in urban basins.

For this study, since the study area is mainly urban. There is no cultivated area or an area suitable for agriculture. Therefore, more than 75% percent of the study area is impervious due to this; the Airport formula is used for the calculation of time of concentration for each catchment (Habtamu, 2017).

4. RESULT AND DISCUSSION

4.1 Drainage Catchment Delineation

The drainage area contributing to the system being designed and the drainage sub area contributing to each inlet point must be measured. The outline of the drainage divide must follow the actual watershed boundary, rather than commercial land boundaries, as may be used in the design of sanitary sewers. The drainage divide lines are influenced by pavement slopes, locations of downspouts and paved and unpaved yards, grading of lawns, and many other features introduced by urbanization.

The rational method of peak discharge estimation is more effective if small size of the catchment area was taken. Because, over sizing of the catchment area will overestimate the peak discharge of the catchment. Considering this, the total study area was divided into 10 sub-catchments following the existing drainage outlets of the area. The outlet of each catchment was fixed following the elevation contour line of the area. Table 4.1 shows the area, flow length (longest flow path and the elevation difference of the outlet and the initial of the flow length for each catchments which was measured using Arc GIS 10.1.

Table 4. 1: Catchment characteristics of the study area

Catchment	Area(Hectare)	Longest flow path(Km)	Starting elevation(m)	Outlet elevation(m)
1	104.378	1.27	1420.046	1416.53
2	96.423	2.119	1439.00	1416.53
3	118.258	2.983	1441.77	1420.046
4	112.275	1.732	1442.333	1404.00
5	87.314	1.565	1408.176	1404.00
6	64.223	1.366	1442.333	1420.046
7	116.762	2.874	1554.00	1416.54
8	54.356	0.889	1442.333	1416.54
9	99.0451	1.206	1416.54	1412.00
10	45.927	0.417	1422.058	1416.53

Figure 4.1 also shows the delineated catchments outlets and the direction of each catchment. Catchment 1 and 10, 3 and 6,4and 5 are fixed to have the same outlet based on the elevation information and road alignment of the area.

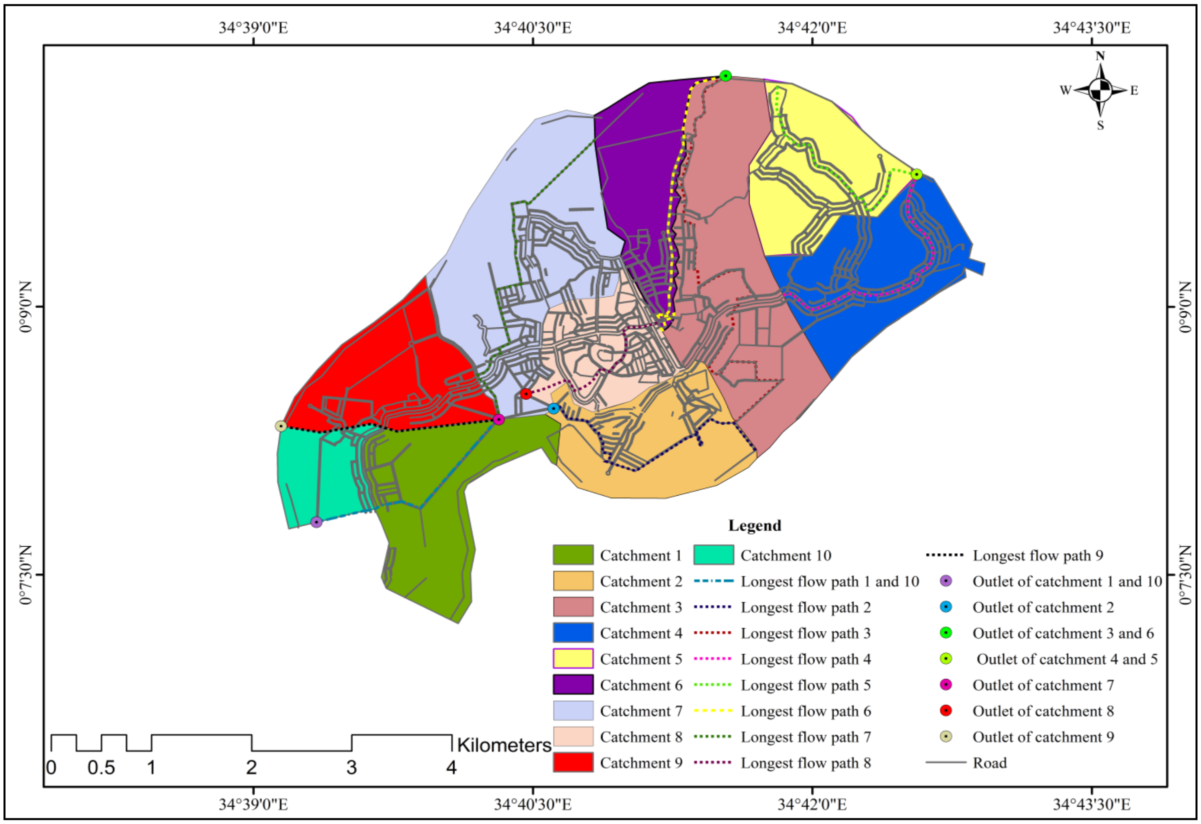

Figure 4. 1: Catchment characteristics and road alignment of the study area

4.2 The newly developed IDF curve of Mizan Teferi station

An Ideal IDF curve for 32 years of 24 hr rainfall data is developed for 2,5,10,25,50, and 100 years return period respectively. The selection of return period was based on the maximum expected life span of drainage structures.

Figure 4.2 shows the IDF curve of Mizan Teferi town. Eq. 3.14 which is the Reduction equation described in the methodology section of this document was used to drive the intensity for a duration that starts from 12 minutes up to 180 minutes at 15 minutes' interval. Then the result of intensity at different duration was drawn using Microsoft excel sheet.

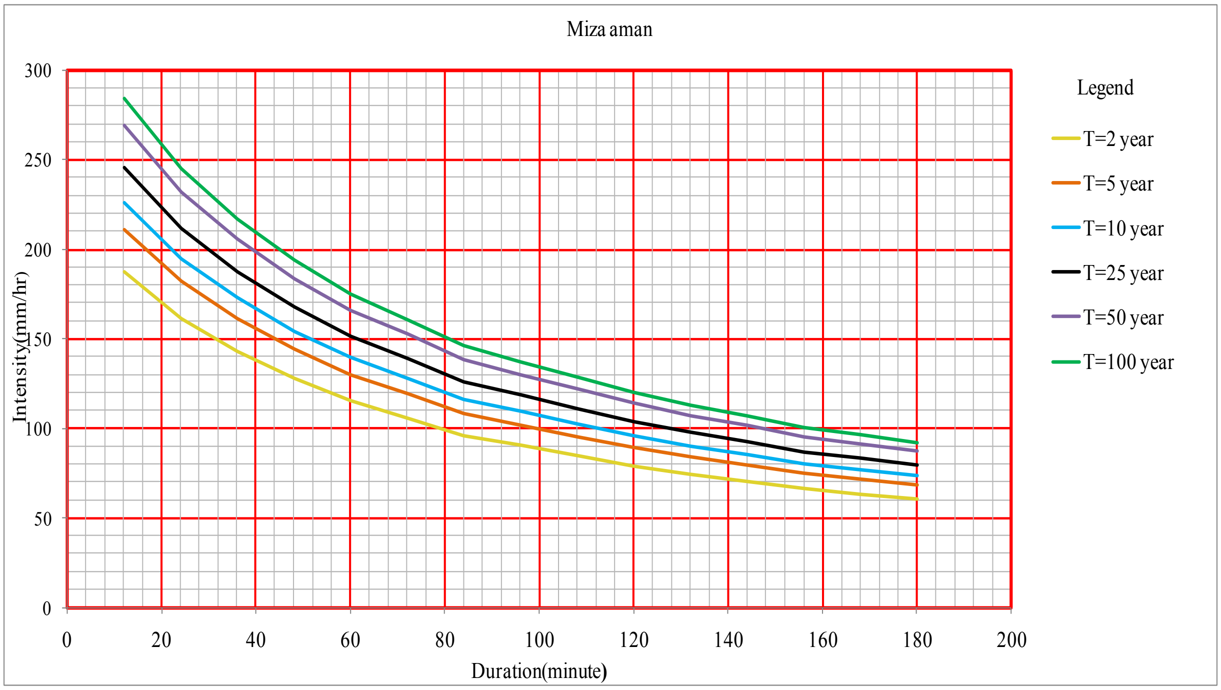

Figure 4. 2: IDF curve of Mizan Teferi town

4.3 Comparison of newly developed IDF curve with an IDF curve developed by ERA

The Ethiopian road authority have been subjected to statistical techniques to develop the information needed from hydrologic analyses.

Based on the result, the country was divided regions. Namely, A1, A2, A3, B1, B2, C. Appendix -3 shows the different regions of Ethiopia based on the rainfall pattern. The Table part of appendix- 3 also shows the metrological stations that fells in different regions. Using the statistical analyses, rainfall intensity-duration curves have been developed by ERA for commonly used design frequencies.

This curve was compared with the new developed IDF curve for safe and economical design of the drainage facilities. The comparison is also very important to adjust the possible cause of technical errors during the development of the new IDF curve.

From the rainfall distribution pattern (Appendix-2), Mizan Teferi station is found in region B1. Therefore, the IDF curve of region B was drawn based on the tabular information of time series data of intensity for different durations for region B which was obtained from ERA drainage design manual.

The result indicates that there is a minor difference between the newly developed and the region B of the IDF curve. There are many possible causes which brought the difference. Three of them are mentioned as follow.

(1) The length of recorded data. For this study, the length of the recorded data was 32 years (From 1985 to 2017) while the data obtained from ERA drainage design manual is not specified. (2) The difference in Duration between the two IDF curves was another possible cause to identify differences. The IDF curve developed by ERA has a maximum duration of 120 minutes but the newly developed IDF curve has a maximum duration of 180 minutes. (3) Now a day, the rainfall distribution pattern is changing radically due to climate and LU/LC change, and rapid increase in urbanization.

Therefore, the IDF curve of the different regions will be modified from past to present. Figure 4.3 shows the graphical representation of the IDF curve of Region B and the newly developed IDF curve. The figure indicates that there a similar trend between the two curves

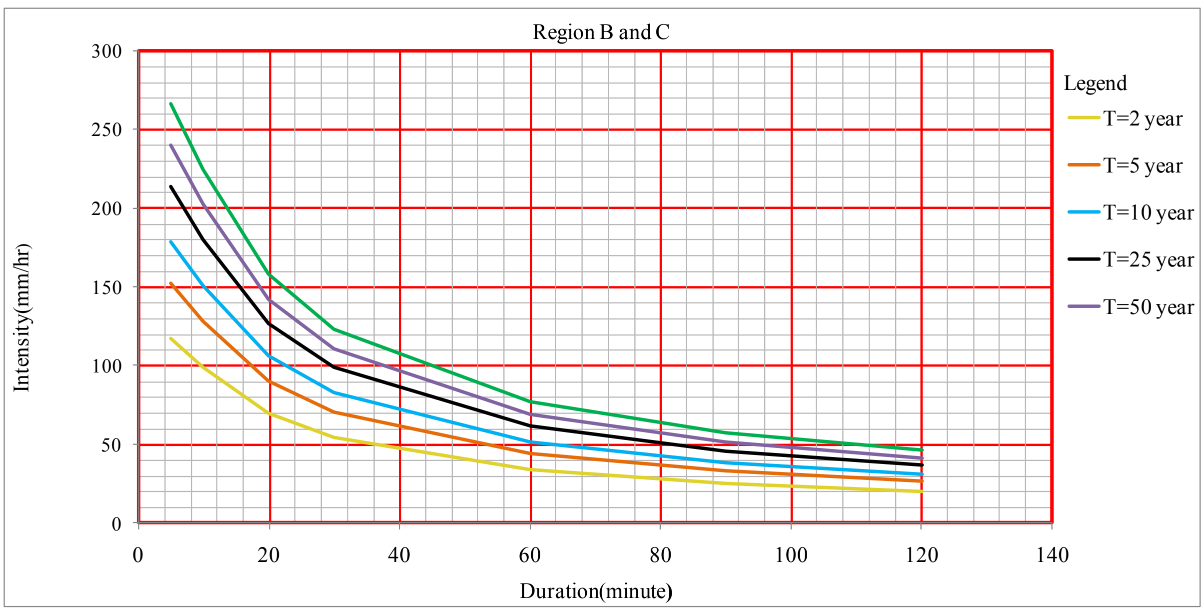

Figure 4. 3: IDF curve of region B and C

4.4 Computation of Weighted Runoff Coefficient

The runoff coefficient is one of the most importan catchment parameter that can be determined from the Land use land cover of the the catchment in the absence of detail phsical information of the area. It is often desirable to develop a composite runoff coefficient based on the percentage of different types of LU/LC in the catchment area.

For this study, the runoff for each LU/LC of the catchment was asigned by measuring the area occupied by different types of LU/LC for each catchment.Then the weighted value of runoff coeffcient was computed using Equation 3.15 for each catchment.Table 4.2 shows the area of each LU/LC occupied by catchment 1 and the weighted runoff coeffcient.

The result shows that the weighted runoff coeffcient for catchment 1 is 0.368. This implies that 37 % of the rainfall rate shall be measured as peak run off rate when continuous rain falls occurs for at least a duration equal to the time of concentration of the catchment. The weighted runoff coeffcient for the remaining catchment was computed using the same procedure and attached in Appendix-3 of this document.

Table 4. 2: LU/LC composition and runoff coeffcient of sub catchment 1

Land use type	Area(sq Km)	Percentage	Runoff coefficient(C)
Urban agriculture	0.7409	70.981	0.325
Residential(Mixed)	0.0430	4.12	0.525
Services	0.1012	9.7	0.8
Reserved	0.1587	15.204	0.25
Total area	1.0438	100.005	Weighted C value=0.368

4.5 Computation of Time of Concentration

The time of concentration for each catchment was calculated using Eq. (3.16). The main input parameters for the calculation of Tc were longest flow path of the catchment, the elevation difference between the initial point of the longest flow path and the Outlet of the catchment and the runoff coefficient.

The topographic information of the catchment (Longest flow path and Elevation) was obtained from the shape file of the master plan of the town. The topographic information was carefully extracted using Arc GIS 10.1.

Table 4.3 shows the time of concentration for each catchment. Based on the airport formula, the effect of longest flow path on the time of concentration is larger than other parameters. Therefore, from table 4.3 a catchment area with longest flow path has higher time of concentration.

The value of time of concentration was used to obtain the intensity value from the newly developed IDF curve for a given return period from Table 4. 3.

Table 4. 3: Time of concentration value of the delineated catchments

Catchment	Area (Hectare)	Longest flow path(Km)	Starting elevation(m)	Outlet elevation(m)	C	Tc(Min)
1	104.378	1.27	1420.046	1416.53	0.3679	128.392
2	96.423	2.119	1439.00	1416.53	0.3917	106.472
3	118.258	2.983	1441.77	1420.046	0.3809	143.004
4	112.275	1.732	1442.333	1404.00	0.4809	75.508
5	87.314	1.565	1408.176	1404.00	0.4012	144.268
6	64.223	1.366	1442.333	1420.046	0.3699	74.155
7	116.762	2.874	1554.00	1416.54	0.3793	75.426
8	54.356	0.889	1442.333	1416.54	0.5850	49.473
9	99.0451	1.206	1416.54	1412.00	0.4252	113.049
10	45.927	0.417	1422.058	1416.53	0.3383	43.877

The value of Time of concentration for different return period with respect to the intensity value is drawn and shown in Fig 4.4. The graph indicates that the value of intensity decreases when the time of concentration increases.

The trend is similar to the IDF curve. But the curve is slightly straight in the case of Tc and intensity graph which indicates that the decrease in Tc for all return period is gradual. The duration of rainfall is then set equal to the time of concentration and is used to estimate the design average rainfall intensity for each return period.

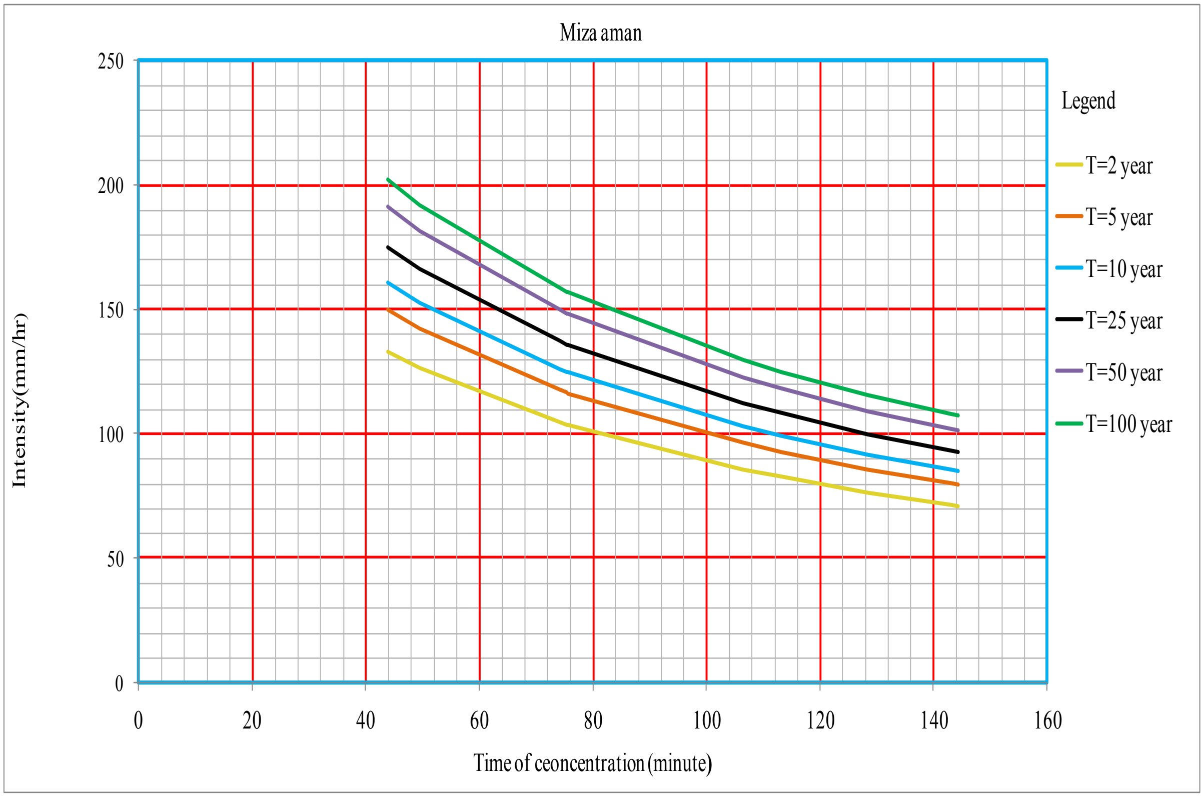

Figure 4. 4: Graphical representation of intensity and time of concentration.

4.6 Computation of Peak Discharge

The peak discharge that will generate from each catchment is calculated using the rational method. All the required parameters required by rational method are determined. Using Eq. (2.1) the Peak runoff generated from each catchment is calculated to fix the dimensions of the drainage structures. The ERA recommendation for design of urban minor Arterial system, urban collector street system, and urban local street system is between 25-50 years, 25 years, 10 years and 5-10 years.

The exceedance probability of 50 and 100 years return period is 2% and 1% respectively. Therefore, for this study, the design discharge with a return period of 25 years is considered to design appropriate dimension of the drainage system. But the value of discharge for the remaining return periods can be used as a reference for the design of large flood protecting structures in the area. Table 4.4 shows the value of 25 years return period peak discharge for each catchment.

Table 4. 4: The 25 year return period of peak discharge

Catch ment	Area (Hectare)	Runoff coefficient(C)	Time of concentration (Tc)	Intensity(mm/hr)	Peak discharge(m^3/s)
1	104.378	0.3679	128.392	99.7737	10.651
2	96.423	0.3917	106.47	112.0609	11.766
3	118.258	0.3809	143.004	93.16891	11.667
4	112.275	0.4809	75.508	135.5033	20.339
5	87.314	0.4012	144.268	92.6126	9.019
6	64.223	0.3699	74.155	136.9553	9.045
7	116.762	0.3793	75.426	135.5913	16.694
8	54.356	0.5850	49.473	165.8329	14.659
9	99.0451	0.4252	113.049	108.1917	12.667
10	45.927	0.3383	43.877	174.7111	7.546

4.7 Existing problems related with storm water drainage.

A, lack of regular maintenance and clearing of the channel

There is no regular maintenance of road and drainage structures as it was investigated during site observation. Most of the side drain ditch is full of garbage and sediment at many places which obstruct the normal flow of water in the channel.

Some drain ditches are also covered totally with grasses and shrubs and thus not giving the desired function for which it was constructed. Some of them were also totally covered by sediment deposition as indicated in Figure 4.5.

Figure 4. 5: An open channel type clogged by grass and silt

B, Inadequate slope provision

Adequate longitudinal slope provision based on the soil type, incoming discharge and nature of the road surface is the major task during the design of urban drainage system. If adequate slope is not provided for the drainage system, the incoming flood does not translate to the outlet efficiently. This is the major cause of over flooding, sediment deposition and reproduction of disease transferring insects like mosquito.

Figure 4.6 indicates that insufficient slope provision is one of the major problems of the town**.** The incoming flood did not translate to the outlet which becomes stagnant and the color has changed.

Figure 4. 6: Stagnant flood that occurs du tot insufficient slope provision

C, Lack of proper waste disposal mechanisms in the town

Solid waste problem is directly related with the existing problem of the town. Due to lack of proper waste removal mechanisms, Domestic solid wastes are carelessly thrown into the channels. This will also clog the channel and creates bad smell.

Figure 4.7 shows one of these major problems. In other side, there are large water shade areas that drain into and across the city due to the topographic nature of the city. This releases huge volume of storm water into the city core. Whereas, the capacity of the existing drainage lines at the core and intermediate areas is highly reduced by accumulation of solid waste and silt.

Figure 4. 7: A typical ditch covered by domestic solid wastes.

D. Improperly designed channels

In some area of the town, channels are designed improperly. There are some channels that have inefficient capacity. According to field observation made, some of the side drain ditches were constructed for nothing as there is no inlet or opening to collect storm water from the adjacent surrounding area or road. In some cases, the inverted levels of the ditches were above the elevation of the adjacent surrounding area and thus water cannot enter to the ditch. Some of them are not carefully covered or the top cover is damaged (Figure 4.8).

Figure 4. 8: Improperly designed drainage system

4.8 Measures to be taken

Due to poor maintenance and lack of periodic cleaning, Improper provision of slope, dimension and shape, inadequate drainage infrastructure, absence of drainage facility in the city, lack of proper waste disposal system causes majorly over flow of the channel, property losses, erosion of the road disposal of domestic non easily recycled waste into the river where the outlet of the drainage channel is going to meet. It also causes different types of environmental related impacts such as water born disease, breeding site for vectors like mosquito and water pollutions.

Proper design and construction of drainage structures are vital components for road structure to function without traffic interruption. Appropriate hydrological analysis of the catchment area, where the drainage structure will be constructed and appropriate hydraulic parameters should be determined.

A drainage structure must be designed to carry allowable recurrence interval of flood. Otherwise, accidental flood may damage by under estimated (low peak runoff) construction or over topping storm runoff on the surface of drainage facility and road surface almost in every year. The existing drainage structure in study area are designed improperly to carry out peak runoff and not properly managed by municipality even to reduced pollution and health related problem.

To eradicate all to above problem, the channel design should be based on scientifically well-known and accepted hydrologic method of the estimated peak discharge, there should be proper maintenance and cleaning mechanism, waste management should be modernized.

4.9 Performance Assessment of the existing drainage system

Due to the existence of different problems on urban drainage system, it is necessary to check the performance to suggest appropriate recommendation. Based on the information obtained from Mizan Teferi town municipality, the problems of major channels were identified. Based on the problem, a rank has been assigned for each channel.

Channel which is relatively free from over flow during rainy seasons, sediment, domestic waste and grass clogging, sufficient slope, crack and score are categorized under very good. While channels having minor clogging by sediment domestic wastes and grass but if they freely discharge water to the downstream with no over flow, are categorized under good condition. Channels that fall under fair conditions are those which are clogged have small sign of over flow and have insufficient slope but they can discharge water to the downstream. The channel which are categorized under poor ranks, are those which have design problem, over flow and alignment problem, inadequate slope and clogging problem either of domestic waste, sediment or grass. Table 4.5 shows the drainage condition of the study area.

Table 4. 5: Drainage conditions of the study area (UIIDP, 2012)

Kebele _Name	Village Name	Construction Year	Drain Condition
Shesheka	Selami	2007	good
Kometa	Selami	2007	Poor
Kometa	Megenegna	2004	good
Hibrete	Dedebit	2007	Very good
Hibrete	Dedebit	2007	Poor
Kometa	Selami	2007	Very good
Shesheka	Selami	2008	Poor
Shesheka	Gebiya Meda	2005	Fair
Shesheka	Gebiya Meda	2006	good
Shesheka	Gebiya Meda	2007	Fair
Shesheka	Gebiya Meda	2005	good
Shesheka	High school	2006	good
Shesheka	Geteri technology	2006	Poor
Shesheka	Geteri technology	2008	Fair
Addiss ketema	Tesfa	2005	good
Addiss ketema	Ambasodor	2005	Very good

The percentage of the performance values are worked out for each rank and shown in Figure 4.9. Based on the result, 37.5%, 25% and 18.75% of the channels are categorized under good, poor and fair condition. Only 18.75% is categorized under very good condition.

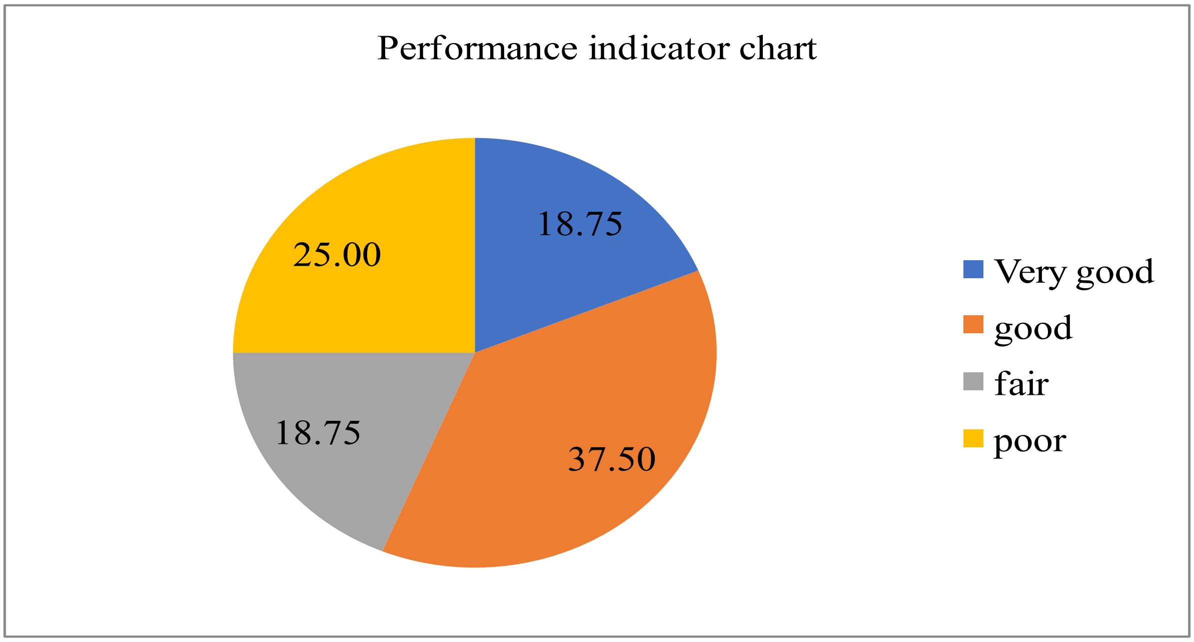

Figure 4. 9: The performance condition of the existing drainage system of Mizan Teferi town

4.10 Determination of the existing capacity of the channels

As discussed under section 4.9 of this document, the existing drainage system of the town has different problems. But the major drainage problem is the over flow of the flood from the channel. Major consequences of over flow from the channel are erosion of the road, damage of personal property and creation of small pond on the road or outside the main road which creates an opportunity for production of disease transmitting insects.

Major solution for the problem is appropriate estimation of peak discharge and design of the drainage system based on the peak discharge.

Therefore, for this study, the adequacy of the existing drainage dimension was evaluated based on the peak discharge of 2 years, 5 years, 10 years and 25 year return period. The existing channel type is Trapezoidal. The dimensions of larger channels which are silt free and free from any other defects in each catchment were measured directly. These channels were also the main storm water collector which conveys the storm to the outlet.

Manning equation was used to estimate the maximum capacity of the cannels based on its existing dimensions. All parameters, except the manning roughness coefficient, which are used to estimate the discharge for the existing capacity was obtained by direct measurement from the field. The slope was obtained from the topographic map of the area and compared with the computed value using Arc GIS.

The result indicates that the maximum discharge estimated discharge is generated in catchment 8 of the study area (Table 4.6). Since manning formula does not estimate the total discharge from a catchment, the formula is independent of the catchment area. But it is a function of channel dimension and longitudinal slope of the channel.

Therefore, the Peak discharge should be carefully determined using Different hydrologic analysis methods, and the channel dimension must be fixed based on the design discharge.

In reality, the peak discharge should occur at the greater size of the catchment. The total discharge generated from catchment 3 was estimated about, 7.56 m^3/s. because the catchment has a gentle slope compared to catchment 7 this, makes a little bit difference in the expectation of the reality.

Table 4. 6: The capacity of existing drainage system of Mizan Teferi town

Catchment	Area (Hectare)	Bottom Width (m)	Top Width (m)	Depth (m)	Slope	Peak discharge (m^3/s)
1	104.378	1.50	1.90	1	0.003	4.66
2	96.423	1.50	1.90	0.85	0.011	7.08
3	118.258	1.50	1.90	1	0.007	7.56
4	112.275	1.50	1.90	1	0.022	13.17
5	87.314	1.50	1.90	1	0.003	4.57
6	64.223	1.50	1.90	1	0.016	11.31
7	116.762	1.50	1.90	1	0.048	19.36
8	54.356	1.50	1.90	0.85	0.029	11.71
9	99.0451	1.50	1.90	0.85	0.004	4.22
10	45.927	1.50	1.90	0.85	0.013	7.91

4.11 Comparisons of the existing channel capacity with the 25 years return period peak discharge

Table 4.7 shows the existing discharge with the 25 years return period peak discharge to check the adequacy of the provided dimension to carry the 25 years return period. This is an easy task because the peak discharge of each catchment for different return period is already computed.

It is also possible to check the adequacy of existing dimension of the channel for 2,5, 10,50 and 100 years return period. But in this study since a 25 years return period is considered for drainage design of the study area, the existing channel capacity is compared with 25 years return period peak discharge.

Table 4. 7: Comparison of the existing capacity of the drainage system with the peak discharge of 25 years return period

Catchment	Area (Hectare)	Existing Peak discharge (m^3/s	25year return period peak discharge((m^3/s)	remark
1	104.378	4.66	10.651	Needs Modification
2	96.423	7.08	11.766	Needs Modification
3	118.258	7.56	11.667	Needs Modification
4	112.275	13.17	20.339	Needs Modification
5	87.314	4.57	9.019	Needs Modification
6	64.223	11.31	9.045	Adequate
7	116.762	19.36	16.694	Adequate
8	54.356	11.71	14.659	Needs Modification
9	99.0451	4.22	12.667	Needs Modification
10	45.927	7.91	7.546	Adequate

The result indicates that the dimensions provided at catchment 6, 7 and 10 are adequate. Numerically, 70% of the channels should be modified to accommodate the peak discharge of the 25 years return period. Only, 30% of the channels can accommodate the 25 year return period peak discharge.

4.12 Modification of the existing channel design

As discussed earlier the 25 years return period peak discharge is greater than the 2, 5, and 10 years return period because the intensity will increase as the return period increases (Figure 4.3). Also the Ethiopian road authority drainage design guide line suggests using 25 years return period for urban drainage design with smaller catchment area. Therefore, the hydraulic design of the channel is based on the peak discharge of the 25 years return period.

The dimensions of the channel are computed using the manning formula which is the well-known open channel design formula. The slope of the channel is calculated earlier. Manning roughness coefficient is assumed to be 0.011 for all catchments of the channel. The most economical trapezoidal channel with side slope of 60 degrees, hydraulic radius equal to half of the channel depth and bottom width equal to twice of the channel depth is considered to fix the channel dimension.

Table 4.8 indicates the dimension of the main collector drain in each catchment. If there are n numbers of drains, the total discharge of the catchment will be divided into n and their dimension will be fixed accordingly.

Based on the manning formula, channel dimension is a function of design discharge, manning roundness coefficient longitudinal slope. From the result the maximum depth of the channel is for catchment 9 having a design discharge of 99.0451m^3/s and a longitudinal channel slope of 0. 004.This is apparent as the longitudinal slope is gentler than other channel that can be designed in the remaining catchments.

This is the channel dimension for 25 years return period of maximum flood. Therefore, the dimension of the channel should be modified to protect both the road erosion and property loss from the over flow of the channel.

The result indicates that the greatest value of the peak discharge was 20.339 m^3/s that will be generated from catchment 4. The existing peak discharge of catchment 4 was 13.17 m^3/s which requires the channel depth, bottom width, and side slope from1m,1.5m and 63.5 degree to 1.1m,1.3m and 60 degrees. Similar to this, the existing dimension has been modified to the new dimensions using the 25 years return period peak discharge.

Table 4. 8: New dimension of the channel

Catchment	Area (Hectare)	Peak discharge(m^3/s)	n	slope	Depth(m)	Bottom width(m)	Top width(m)
1	104.378	10.651	0.011	0.003	1.3	1.5	3.02
2	96.423	11.766	0.011	0.011	1.1	1.2	2.44
3	118.258	11.667	0.011	0.007	1.1	1.3	2.60
4	112.275	20.339	0.011	0.022	1.1	1.3	2.60
5	87.314	9.019	0.011	0.003	1.2	1.4	2.85
6	64.223	9.045	0.011	0.016	0.9	1.0	2.04
7	116.762	16.694	0.011	0.048	0.9	1.0	2.09
8	54.356	14.659	0.011	0.029	0.9	1.1	2.19
9	99.0451	12.667	0.011	0.004	1.3	1.5	3.04
10	45.927	7.546	0.011	0.013	0.9	1.0	1.98

5. CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

The urban drainage system of the town is one of the greatest infrastructures that need a careful attention from the concerned bodies. Now a day the issue of urban drainage system of the town has been given little attention which brought several urban related problems. Therefore, carful assessment of the existing drainage system and proposing appropriate design modification was conducted to minimize the problem

Inadequate slope provision, Lack of periodic cleaning and maintenance, insufficient channel dimension which are designed by rough estimation of the peak discharge from the catchment and lack of waste disposal mechanisms are the major problems of the existing drainage system of the town. Awareness at the community level for drainage system is poor. Since some peoples intentionally throw solid waste into existing drains and caused the water to be stagnant. Road damage by from the over flow of the channels, property losses during high over flow, reproduction of disease transmitting insects are the main effects from the existing drainage problem.

The performance of the existing drainage system was assessed based on the major identified problem. Based on the result, 18.75%, 37.5%, 25% of the channels are categorized under very good, fair, good and poor conditions respectively.

Only 30% of the existing dimension of the channels can accommodate the peak discharge for the 25 years return period. 70% of the channels should be modified to accommodate the peak discharge of the 25 years return period. Therefore, to minimize, the overflow problem of the channel, appropriate channel dimensions and longitudinal slope has been recommended considering the 25 years return period.

5.2 Recommendations

There should be proper maintenance if there is damage and periodic clearing mechanism. Throwing of domestic solid waste is the major problem which is not given much attention. To alleviate this problem, it is better to teach the community to increase their awareness on waste disposal mechanism. In addition to this, Construction of small waste container at suitable interval throughout the road is necessarily.

Appropriate design modification of the drainage system should be conducted by the Mizan Teferi city municipality. This is important to protect the road from the scouring and erosion from the over flow of the channel and to save the losses of human life and property. Modifying the design also saves the cost of periodic maintenance and clearing, the damage of the drainage system by huge amount of flood. This study should be considered as an input and reference for the concerned body to modify and construct the new dimension of the drainage channels.

Estimation of peak discharge from the catchment requires carful assessment of hydro metrological and catchment parameters. Due to the spatial and temporal variability of both hydro metrological and catchment parameters, it requires carful work to estimate the peak discharge of the catchment at minimum error. To achieve this, it better to estimate the peak discharge by more than one hydrological method.

Most government owned infrastructures in general and urban drainage system in particular lack continuous follow up after the end of the construction to keep the long term sustainability of the structure. This has caused to make the structure nonfunctional before its design period and which needs a huge investment cost to reconstruct it. Therefore, continuous follow up is needed to make the operating life of the longer.

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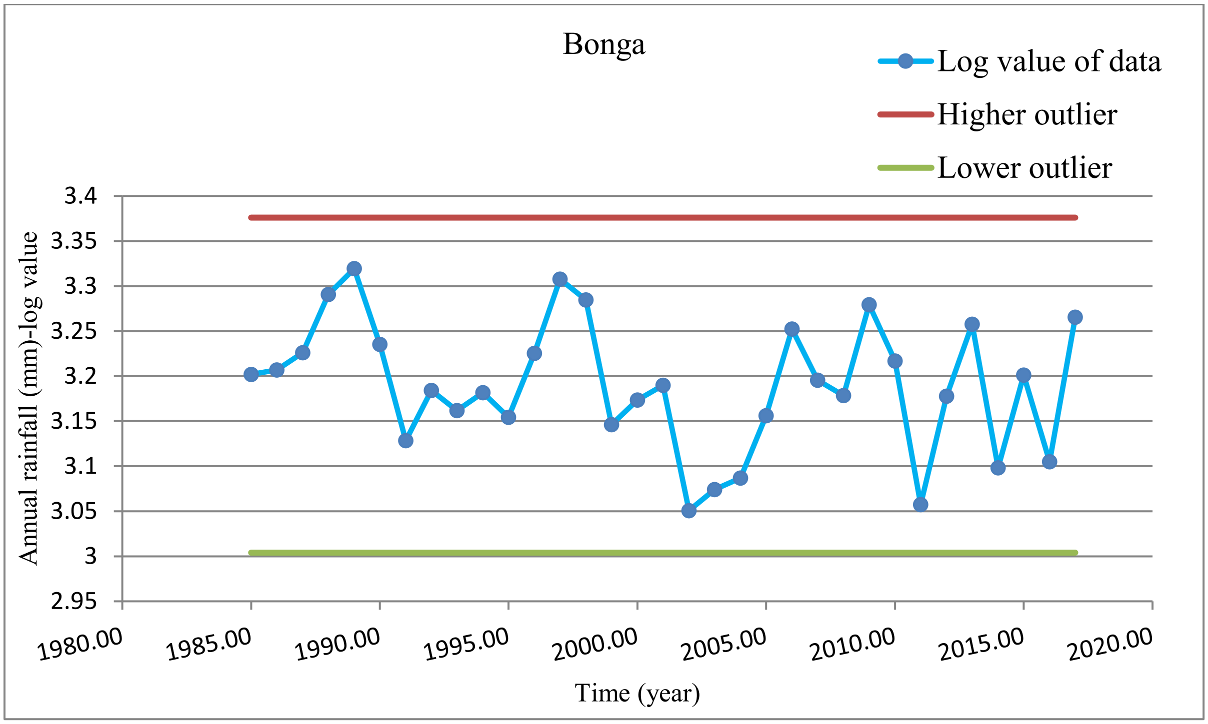
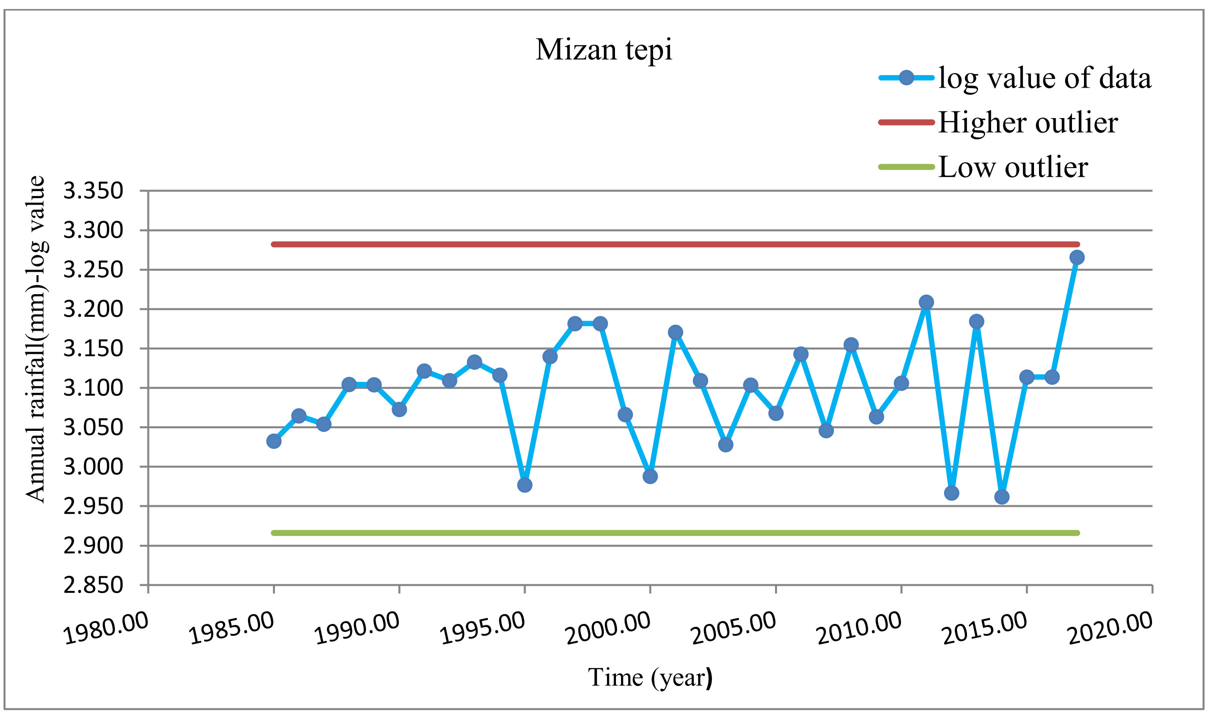
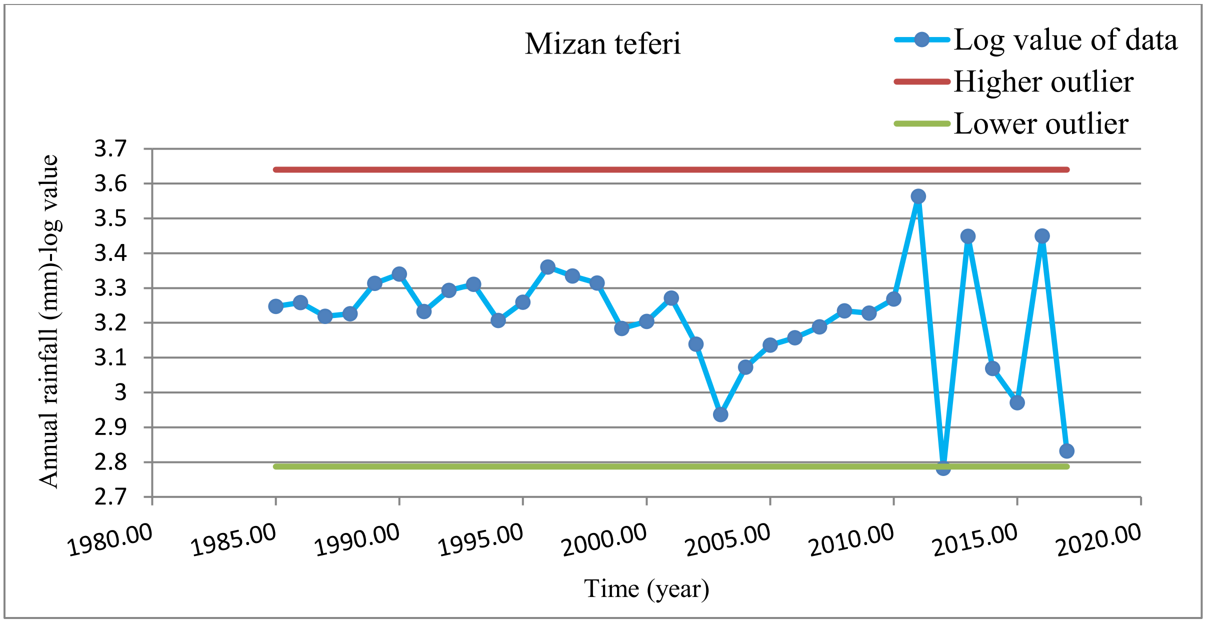
APPENDIXES

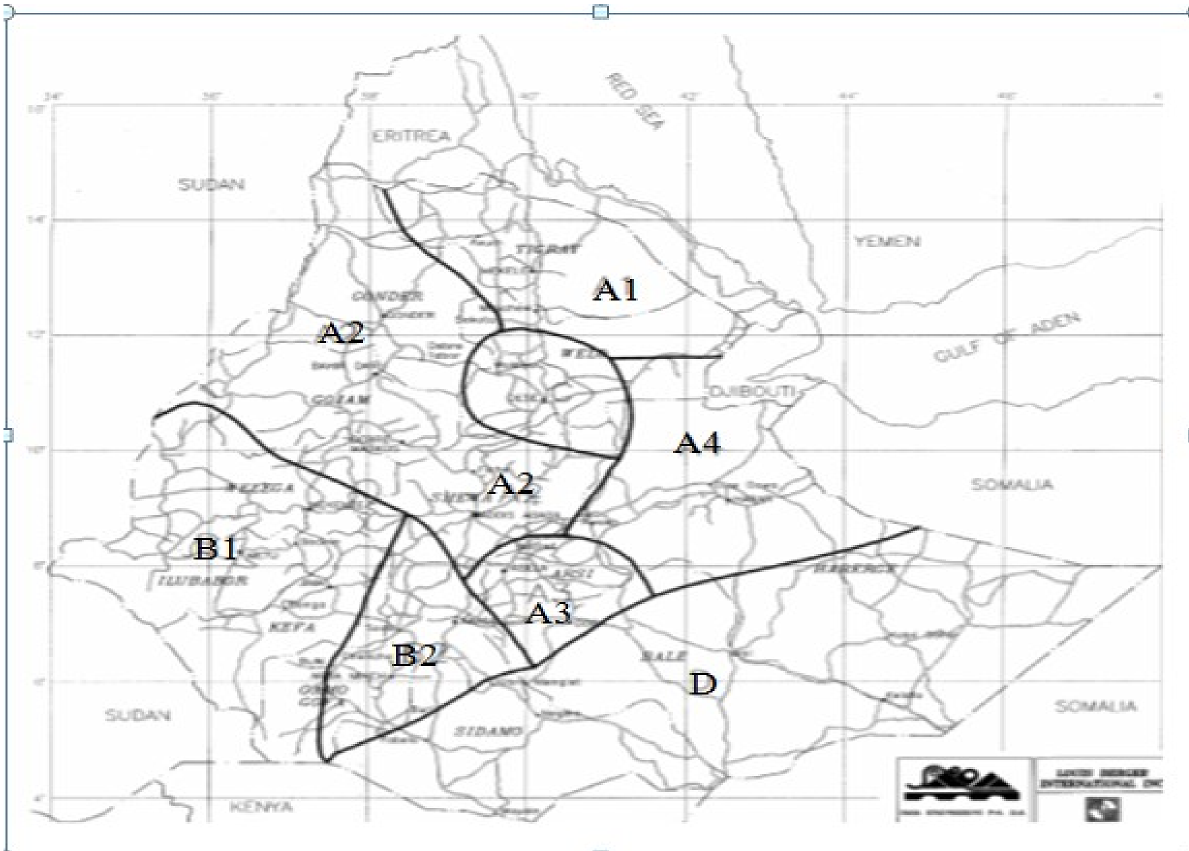
Appendix- 1: Outlier test result







Appendix-2 Rainfall regions of Ethiopia



Meteorological Region	Station	Years of Record	Meteorological Region	Station	Years of Record
A1	Axum	18	B	Bedele	19
	Mekele	35		Gore	45
	Maychew	24		Nekempte	27
A2	Gondar	40		Jima	45
	Debre Tabor	22		Arba Minch	11
	Bahir Dar	35		Sodo	28
	Debre Markos	44		Awasa	26
	Fitche	25	C	Kombolcha	46
	Addis Ababa	33		Woldiya	23
A3	Nazareth	40		Sirinka	17
	Kulumsa	31	D1	Gode	29*
	Robe/Bale	19		Kebri Dihar	38
A4	Metehara	28	D2	Kibre Mengist	24
	Dire Dawa	46		Negele	45
	Mieso	35		Moyale	18

Appendix-3 Land use and Land cover composition of the study area

Catchment 2

Land use type	Area(sq Km)	Percentage	Runoff coefficient
Urban agriculture	0.633434	65.7	0.325
Residential(Proposed)	0.2362	24.5	0.525
Services	0.0098	1.0164	0.8
Manufacturing	0.0470	4.874	0.65
Reserved	0.0378	3.92	0.25
Total area	0.964234	100.0104	Weighted C value=**0.391721356**

Catchment 3

Land use type	Area(sq Km)	Percentage	Runoff coefficient
Urban agriculture	0.992937	83.954	0.325
Services	0.13204	11.166	0.8
Administration	0.019237	1.627	0.75
Recreation	0.038366	3.245	0.2
Total area	1.18258	100	Weighted C value=**0.380893872**

Catchment 4

Land use type	Area(sq Km)	Percentage	Runoff coefficient
Urban agriculture	0.4484745	39.945	0.325
Residential(Proposed)	0.528342	47.058	0.525
Services	0.145935	13.000	0.8
Total area	1.1227515	100.000	Weighted C value=**0.480855971**

Catchment 5

Land use type	Area(sq Km)	Percentage	Runoff coefficient
Urban agriculture	0.60805	69.640	0.325
Residential(Proposed)	0.21602	24.741	0.525
Services	0.04907	5.62	0.8
Total area	0.87314	100.000	Weighted C value=**0.401175928**

Catchment 6

Land use type	Area(sq Km)	Percentage	Runoff coefficient
Urban agriculture	0.659171	82.168	0.325
Residential(Proposed)	0.113932	14.202	0.525
Services	0.016557	2.064	0.8
Administration	0.01257	1.567	0.75
Total area	0.80223	100.000	Weighted C value=**0.369866466**

Catchment 7

Land use type	Area(sq Km)	Percentage	Runoff coefficient
Urban agriculture	0.960441	82.257	0.325
Residential(Proposed)	0.082503	7.066	0.525
Services	0.016557	1.418	0.8
Manufacturing	0.069344	5.939	0.65
Administration	0.038775	3.321	0.75
Total area	1.16762	100.001	Weighted C value=**0.379282515**

Catchment 8

Land use type	Area(sq Km)	Percentage	Runoff coefficient
Residential(Proposed)	0.347053	63.848	0.525
Services	0.116078	21.355	0.8
Administration	0.048744	8.968	0.75
Recreation	0.031685	5.829	0.2
Total area	0.54356	100	Weighted C value=**0.584958836**

Catchment 9

Land use type	Area(sq Km)	Percentage	Runoff coefficient
Urban agriculture	0.375144	37.876	0.325
Residential(Proposed)	0.024754	2.500	0.525
Services	0.020163	2.036	0.8
Manufacturing	0.238339	24.064	0.65
Administration	0.064367	6.500	0.75
Reserved	0.267684	27.027	0.25
Total area	0.990451	100	Weighted C value=**0.425225125**

Catchment 10

Land use type	Area(sq Km)	Percentage	Runoff coefficient
Urban agriculture	0.428716	93.347	0.325
Commercial	0.030554	6.653	0.85
Total	0.45927	100	Weighted C value=**0.338305463**

Appendix-4 the peak discharge generated from different catchment for 2, 5, 10, 50 and 100 years return period

T=2 years

Catchment	Area (Hectare)	Runoff coefficient(C)	Time of concentration(Tc)	Intensity(mm/hr)	Peak discharge(m^3/s)
1	104.378	0.3679	128.392	76.07096	8.121
2	96.423	0.3917	106.472	85.49323	8.977
3	118.258	0.3809	143.004	71.03577	8.895
4	112.275	0.4809	75.508	103.3123	**15.507**
5	87.314	0.4012	144.268	70.61165	6.876
6	64.223	0.3699	74.155	104.4194	6.896
7	116.762	0.3793	75.426	103.3794	12.728
8	54.356	0.5850	49.473	126.4367	11.177
9	99.0451	0.4252	113.049	82.48903	9.658
10	45.927	0.3383	43.877	133.2057	5.754

T=5 years

Catchment	Area (Hectare)	Runoff coefficient(C)	Time of concentration(Tc)	Intensity(mm/hr)	Peak discharge(m^3/s)
1	104.378	0.3679	128.392	85.54991	9.133
2	96.423	0.3917	106.472	96.14631	10.095
3	118.258	0.3809	143.004	79.88702	10.004
4	112.275	0.4809	75.508	116.1857	**17.440**
5	87.314	0.4012	144.268	79.41004	7.733
6	64.223	0.3699	74.155	117.4308	7.755
7	116.762	0.3793	75.426	116.2612	14.314
8	54.356	0.5850	49.473	142.1916	12.570
9	99.0451	0.4252	113.049	92.76776	10.861
10	45.927	0.3383	43.877	133.2057	6.471

T=10 year

Catchment	Area (Hectare)	Runoff coefficient(C)	Time of concentration(Tc)	Intensity(mm/hr)	Peak discharge(m^3/s)
1	104.378	0.3679	128.392	91.84058	9.804
2	96.423	0.3917	106.472	103.2162	10.837
3	118.258	0.3809	143.004	85.76112	10.739
4	112.275	0.4809	75.508	124.7292	**18.722**
5	87.314	0.4012	144.268	85.24906	8.302
6	64.223	0.3699	74.155	126.066	8.326
7	116.762	0.3793	75.426	124.810	15.367
8	54.356	0.5850	49.473	152.647	13.494
9	99.0451	0.4252	113.049	99.589	11.660
10	45.927	0.3383	43.877	160.820	6.946

T=50 year

T=100 year

Catchment	Area (Hectare)	Runoff coefficient(C)	Time of concentration(Tc)	Intensity(mm/hr)	Peak discharge(m^3/s)
1	104.378	0.3679	128.392	109.1775	11.655
2	96.423	0.3917	106.472	122.7006	12.883
3	118.258	0.3809	143.004	101.95	12.767
4	112.275	0.4809	75.508	148.2741	**22.256**
5	87.314	0.4012	144.268	101.3412	9.869
6	64.223	0.3699	74.155	149.8635	9.897
7	116.762	0.3793	75.426	148.371	18.267
8	54.356	0.5850	49.473	181.4629	16.041
9	99.0451	0.4252	113.049	118.3889	13.861
10	45.927	0.3383	43.877	174.7111	8.258

Catchment	Area (Hectare)	Runoff coefficient(C)	Time of concentration(Tc)	Intensity(mm/hr)	Peak discharge(m^3/s)
1	104.378	0.3679	128.392	115.4038	12.320
2	96.423	0.3917	106.472	129.6158	13.609
3	118.258	0.3809	143.004	107.7639	13.495
4	112.275	0.4809	75.508	156.7306	**23.525**
5	87.314	0.4012	144.268	107.1205	10.432
6	64.223	0.3699	74.155	158.4101	10.462
7	116.762	0.3793	75.426	156.8324	19.309
8	54.356	0.5850	49.473	191.8115	16.956
9	99.0451	0.4252	113.049	124.5579	14.583
10	45.927	0.3383	43.877	202.0806	8.728