



ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES
ADDIS ABABA INSTITUTE OF TECHNOLOGY

Assessment of Stormwater Drainage Systems in Kemise Town

A thesis submitted and presented to the school of graduate studies of Addis Ababa University in partial fulfillment of the degree of Masters of Science in Civil Engineering (Major in Hydraulic Engineering)

By

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Addis Ababa University
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Addis Ababa Institute of Technology

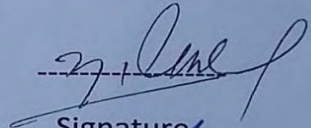
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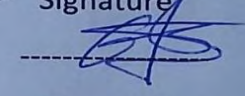
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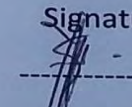
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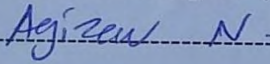


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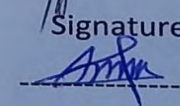
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Declaration

I, the undersigned, certify that I have read and here by recommended for acceptance by the Addis Ababa University a Thesis entitled: **Assessment of Stormwater Drainage Systems in Kemise Town** and here by recommend for acceptance by the Addis Ababa University in partial fulfillment of the requirements of the degree of Masters of Science in Hydraulic Engineering.

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Dedication

This Thesis is dedicated to my late father Asfaw G/medhine

Acknowledgement

First, I would like to thank the almighty God for his unspeakable gift, help and protection during my work.

I would like to express my honest thankfulness and appreciation to Dr. Yilma Seleshi whose encouragement, guidance and support me from the initial to the final level of this thesis. He enabled me to develop and understand the subject matter as well as the way of writing this research.

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Acronyms and Abbreviations

ASTM- American Society for Testing and Materials
AMC – Antecedent moisture condition
C1, C2...Cn and CM – Catchments
CSA Central Statistical Agency
CN - Curve Number
DEM - Digital Elevation Model
DMS - Degrees Minutes Seconds
ERA - Ethiopian Roads Authority
EMA- Ethiopian Mapping Agency (Current nomenclature)
FDRE- Federal Democratic Republic of Ethiopia
GIS - Geographic Information System
GPS – Geographical positioning system
HSG- Hydrological Soil Group
IDF- Intensity-Duration-Frequency
LVRs - Low Volume Roads
M.A.S.L – mean above sea level
NMA- National Meteorological Agency
QC1, QC2...QC9 – Discharge from each catchment
RC, Reinforced Concrete
SCS - Soil Conservation Service
SUDS - sustainable urban drainage systems
USACE- United States Army Corps of Engineers
USAID- United States Agency for International Development
USGS- United States Geological Survey
US NRCS- United States Natural Resources Conservation Service
US SCS- United States Soil Conservation Service
USWD -urban Storm water drainage
UTM- Universal Transfer Mercator
WS – Water shed

Abstract

Storm water drainage problem is one of the major challenges facing many countries in Ethiopia in general and kemise also faced the problem. So, the study focuses on the assessment of storm water drainage system in Kemise town using statistical analysis, GIS and other material. A statistical analysis was applied to analyze the existing condition system and to analyze the peak discharge for the town and to fix the diameter of the pipe .GIS software was used to delineate water shed by combining DEM and aerial photograph. Based on primary and secondary data collected, the problems in the areas are categorized as construction, management and design problem. The method used to investigate management problem is direct field data collection and site visit but the construction problem is analyzed using field survey as well as comparison of design with what is implemented in the ground. Design of the study area is evaluated by redesigning of the system using the computation method used in ERA drainage design manual. Design of storm drainage system evaluated in this thesis includes hydraulic performance for existing situation by using manning's and fixing new size of reinforced concrete pipe for existing and proposed area. Based on the result of this thesis the drainage system is insufficient at different area and Most of the stations of the catchments were investigated that required construction of additional storm drainage system and its existing coverage and its proposed based on assessing and hydrologic and hydraulic calculation of this study the existing drainage conveying capacity is only 19% at different catchment and the other 81% of new drainage system is required for different catchment at existing and proposed. Finally, appropriate mitigation measures were proposed for required 81% of new drainage system in order to serve the area from different negative effect and drainage structures for the future purposes sustainably. Additionally managing and improper construction alignment problem for the existing system were investigated and to avoid this problem periodic cleaning and modification of slope is recommended.

Key words; Major problems, Assessment of storm drainage system, statistical analysis, Hydraulic performance, Mitigation measures, Kemise.

1. Introduction

1.1 General Background

Urban storm water drainage facilities are part of the urban infrastructure elements and design of these facilities require due attention. In Ethiopian context, where watersheds of many urban centers receive significant amount of annual rainfall and where rainfall intensity is generally high, control of runoff at source, flood protection, and safe disposal of excess water/runoff through proper drainage facilities becomes essential. (AASHTO, 1991 et al)

Drainage problems in urban areas introduce flooding, deterioration of roads, land degradation, sedimentation, water logging and etc. With urbanization, impermeability of land increases with the increase in impervious surfaces (i.e. residential houses, commercial buildings, paved roads, parking lots, etc.) with this drainage pattern changes, overland flow gets faster, flooding and environmental problems such as land degradation increases. And it becomes a crucial problem facing the existing and future road infrastructure. . (AASHTO, 1991 et al)

In spite of these problems, drainage facilities in most urban centers of the country are nearly absent or at a lower coverage. Planning and design rarely guide construction or provision of such facilities and management.

Storm water discharges are produced when the capacity of the land to retain precipitation is exceeded and run-off occurs. Run-off will be influenced by rain fall and intensity (millimeter of rain fall per hour)) and duration, antecedent storms and a number of watersheds, and land use characteristics such as slope, soil type, and impervious surfaces. . (AASHTO, 1991 et al)

Before the 1960s the availability of both hydrologic data and computers was limited. So, flood frequency analysis and traditional hydrologic computations was the main inputs for the hydrologic modelers. The increase in urbanization in the last 50 years and development of impervious surfaces along with it changed the watershed and increase flood in the cities and this in turn made necessary the developments of new drainage models. (Hydrological Modeling Statistical Method). (AASHTO, 1991 et al)

Water has a key role when discussing the mechanical performance and lifetime of any traffic infrastructure. Increased water content reduces the bearing capacity of soil, which will increase the rate of deterioration and shorten the lifetime of the road and flooding. In such cases, the road will need rehabilitation more often than a well- drained road structures. (AASHTO, 1991 et al)

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. In addition to increased densification and impermeability of the urban landscape, the planning as well as implementation of storm water protecting structures is insufficient. (AASHTO, 1991 et al)

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. (AASHTO, 1991 et al)

1.2 Background of Drainage Structures of the Study Area

Cobble stone road and gravel road that is found in kemise town was provided for access road purpose for the community and the road 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 are 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 rectangular drainage system. At some stations, the drainage structures are lacking even if they are required for the drainage purpose.

1.3 Statement of the problem

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.

In addition to increased densification and impermeability of the urban landscape, the planning as well as implementation of storm water protecting structures is insufficient.

In the last decades, a little number of hydraulic structures has been constructed in Kemise to be used for drainage system. It is known that most of them were designed under inadequate hydrological data condition and hydraulic analysis. The urban drainage of these structures failed to deliver the design yields due to sediments, less capability and failure-stability problems. The problem is by now even become

worse due to unexpected flooding that arise from climate change in the town. Presently some of the problems that have been observed in the Kemise town are such as, base failure, Depressions, Shoving, Edge crack, Shoulder erosion, Abetment damage, Silted drainage ditches, flooding etc.

1.4 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 Drainage schemes that contribute for better service coverage.

To understand problems of damage and preserve the structures by avoiding further deterioration for taking correct measures as well as to reduce any inconvenience and disruption to travel due to over flow of water in the main road due to flooding.

The result of this study also may help in filling the gaps by identifying problems to Sustainability, taking proper designing of Stormwater drainage system and proper functioning of Drainage schemes in the town. In general, Kemise is the part of which facing the Drainage problems, so further investigation were contribute the solution for stormwater drainage problem and sustainable drainage system for future use in the area.

1.5 Objectives

1.5.1 General objective

- To contribute to efforts that aim at improving the storm water drainage problem of Kemise Town.

1.5.2 Specific objective

- To assess the existing condition and problems related to storm water drainage.
- To assess the hydraulic performance of existing drainage systems of the town.
- To recommend appropriate measure

1.6 Research questions

- What is the existing condition and problems related to storm water drainage?
- What is the hydraulic performance of existing drainage systems of the town?
- What is the appropriate recommendation?

1.7 Scope and limitation of this study

This study specifically focused on assessing, designing of drainage system (sizing) and involvement of concerned body of government in the planning, implementation, operation and maintenance of the storm water drainage systems to solve the problem. And also for sustainability in Drainage schemes that contribute for better service. The study is limited to the some parts of kemise town (Three Kebele; Kebele-2, Kebele-3 and Kebele-4) which are not including whole parts of the town because those kebeles are exposed to stormwater drainage problems. This research also does not include structural design of all types of drainage structures except proposing the type and size of required drainage structures.

1.8 Outline of the Research

The thesis is organized into five chapters from introduction part to the conclusion and recommendation. The first and the second chapter's deals with introduction, the objective of the research, research question, statement of the problem, significance of the study, scope of the study, and the second chapter contain literature review and state of ability related to Storm Water Drainage Systems.

The third chapter is about description of the study area, materials, methodology and procedure to be applied in the research from data collection to the result of the analysis. The forth chapter includes assessing parts, result and discussion and the last chapter focused on the conclusion and recommendation parts which concludes the study and recommends the possible solutions.

2. Literature Review

2.1 General concept

2.1.1 Historical Perspectives of Urban Drainage

Historically, urban drainage systems have been viewed with various perspectives. During different time periods and in different locations, urban drainage has been considered a vital natural resource, a convenient cleansing mechanism, an efficient waste transport medium, a flooding concern, a nuisance wastewater, and a transmitter of disease. In general, climate, topography, geology, scientific knowledge, engineering and construction capabilities, societal values, religious beliefs, and other factors have influenced the local perspective of urban drainage. For as long as humans have been constructing cities these factors have guided and constrained the development of urban drainage solutions. Historical accounts provide sights of many interesting and unique urban drainage techniques. (Burian, S. and Edwards, F. 2002)

2.1.2 Related Studies on Drainage issues at Addis Ababa

In order to establish the fact that drainage problem exists in the city and to understand the works that are done, literatures are reviewed. The literatures showed no doubt on the existence of drainage problem in the city. The presentation of the problems in the literature are presented either in the form of malfunctioning of specific component of the urban transportation or broader problems on urban drainage systems themselves. Two studies on drainage problems that exist in Addis Ababa city are reviewed in this sub section. Both studies are performed by post graduate students of Addis Ababa University. The first thesis is titled, “Study of the Urban Drainage System in Addis Ababa, Yeka Sub city” (Dagnachew, 2009). The second thesis is titled, “Investigation on Storm Drainage Problem of Addis Ababa - Case Study at Gotera – Wollo Sefer, Saris - Gotera and Ring Road” (Desalegn, 2011). The thesis third title is Integrated Urban Drainage System; the Case of Ayat to Megenagna LRT Route (Anteneh 2015) and the fourth title Performance Assessment of Road Drainage Structures and Proposed Mitigation Measures: The case of Daleti-Odagodere Gravel Road in Benishangul-Gumuz Region (Yifred, 2015).

2.1.3 Storm

A storm drain is that portion of the roadway drainage system that receives runoff from inlets and conveys the runoff to some point where it can be discharged into a ditch, channel, stream, pond, lake, or pipe. This section contains the criteria and procedures for the design of roadway drainage systems

Criteria for Storm Drains (ASTM, et al 2008)

Storm drains shall be designed using the following criteria where applicable:

- A. Minimum pipe size is 375 mm.
- B. Storm sewer pipe materials for proposed systems typically include concrete, corrugated metal, aluminum alloy. Manning's roughness coefficients for materials occasionally encountered.
- D. Design to flow full, based on uniform flow.
- E. Minimum self-cleaning velocity of 0.76 m/s should be maintained wherever possible.
- F. Maximum grade on which concrete pipe should be placed is 10%.
- G. Flared end-sections should be used whenever and wherever possible, for concrete and metal pipe.
- H. Pipe sizes should not decrease in the downstream direction even though an increase in slope would allow a smaller size.
- I. Pipe slopes should conform to the original ground slope as far as possible to minimize excavation.
- J. For durability, the minimum thickness for steel pipe is 14 gauges and for aluminum alloy pipe is 16 gauges. In extremely corrosive areas and where high abrasion can be expected the designer shall determine whether heavier gauges should be used.
- K. The drainage layout should attempt to avoid conflicts with existing underground utilities and such items as utility poles, water supply lines, telephone cables etc. Implementation of the following design approaches may be necessary.
 - a. Use of pipe material with the lowest friction factor to minimize pipe size
 - b. Use of elliptical or arch pipe to minimize vertical dimension of pipe.
 - c. Test pits should be obtained early in the design process to obtain horizontal and vertical information for existing utilities. If the suggested design approaches do not avoid conflict, use of special drainage structures may be used to avoid the utility.
- L. Precast manholes or inlets shall not be used for pipes 1350 mm or larger diameter or when three or more pipes tie in and at least two of them are connected at some angles. When these conditions exist, cast-in-place inlets or manholes are more practical.

M. Cleaning existing drainage pipes and structures shall be incorporated on all projects when the existing drainage system has substantial accumulation of sediments.

N. On projects where contaminated areas have been identified, the drainage system should be designed to avoid these locations, if possible. If avoidance is not feasible, a completely watertight conveyance system, including structures such as manholes, inlets, and junction chambers, should be designed to prevent contaminated groundwater or other pollutants from entering the system. Possible methods to accomplish this include joining pipe sections with a watertight sealant and/or gaskets, or the use of welded steel pipe. Retrofitting existing pipes to make them watertight may require installation of an appropriate internal liner. The designer shall provide recommendations prior to proceeding with the final design.

O. Existing drainage facilities that are not to be incorporated into the proposed drainage system are to be completely removed if they are in conflict with any element of the proposed construction. Existing drainage facilities that are not to be incorporated into the proposed drainage system that do not conflict with any element of the proposed construction are to be abandoned. Abandonment of existing drainage facilities requires plugging the ends of the concrete pipes to remain. Metal pipes shall be either removed or filled. (ASTM 2008)

2.1.4 Stormwater

Water is a finite and vulnerable resource, essential to sustain life, development and environment. Stormwater is the water draining off a site from the rain that falls on the roof and land, and everything it carries with it. The soil, organic matter, litter, fertilizers from gardens and oil residues from driveways it carries can pollute downstream waterways. Rainwater refers only to the rain that falls on the roof, which is usually cleaner. However, stormwater can be a valuable resource. Reusing stormwater can save potable water and reduce downstream environmental impacts. In urban areas stormwater is generated by rain runoff from roofs, roads, driveways, footpaths and other impervious or hard surfaces. In Australia the stormwater system is separate from the sewer system. Unlike Sewage, stormwater is generally not treated before being discharged to waterways and the sea. Poorly managed stormwater can cause problems on and off site through erosion and the transportation of nutrients, chemical pollutants, litter and sediments to waterways. Well-managed Stormwater can replace imported water for uses where high quality water is not required, such as garden watering. (Hatt, B, et al 2004).

2.1.5 Runoff

New developments have a direct impact on existing drainage infrastructure and the surrounding environment (Butler and Maksimovic, 2001). They increase the area of paved surfaces, thus reducing infiltration, while causing surface runoff to exhibit higher peak flows, larger volumes, and shorter times to peak and accelerated transport of pollutants and sediment from urban areas. This results in pollution of the receiving watercourses and increased flood risk within the development. Controlling surface runoff thus becomes a key element in working towards urban sustainability. (Markopoulos et al., 1999).

2.2. Flooding

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)

2.2.1. Flood Types

Flash Floods: According to H.M.Ranghunath (2006) on Melese Chanie’s 2011 thesis, a flash flood is a very direct response to rainfall with a very high intensity or sudden massive melting of snow. The area covered by water in a flash flood is relatively small compared to other types of floods. The amount of water that covers the land is usually not very large, but is so concentrated on a small area that it can raise very high. Because of the sudden onset and the high travelling speed of the water, flash floods can be very dangerous. The water can transport large objects like rocks, trees and cars. When a dyke breaks along the sea or along a river, the water may flow in so suddenly and with such speed that you could compare it with a flash flood .

Coastal Floods: According to H.M.Ranghunath (2006) on Melese Chanie’s 2011 thesis, a coastal flood is when the coast is flooded by the sea. The cause of such a surge is a severe storm. The storm wind pushes the water up and creates high waves. A storm is formed in al low pressure area. An interesting fact is that beneath a low pressure area the sea level is higher. This contributes to the high sea level, but the wind can have a larger effect. A flood starts when waves move inland on an undefended coast or overtop or breach the coastal defense works like dunes and dikes. The waves attack the shore time and again. When it is a sandy coast, each wave in a storm will take sand away. Eventually a dune may collapse that way. Very characteristic of a coastal flood is that the water level drops and rises with the tide. At high tide the water may flow in and at low tide it may recede again. When a sea defense is

breached, low tide is the time to repair the breach. In the animation you see the build-up of force by the sea and how the sea floods the coast.

Urban Floods: According to H.M.Ranghunath (2006) on Melese Chanie's 2011 thesis, urban flooding is specific in the fact that the cause is a lack of drainage in an urban area. As there is little open soil that can be used for water storage nearly all the precipitation needs to be transported to surface water or the sewage system. High intensity rainfall can cause flooding when the city sewage system and draining canals do not have the necessary capacity to drain away the amounts of rain that are falling. Water may even enter the sewage system in one place and then get deposited somewhere else in the city on the streets. Urban floods are a great disturbance of daily life in the city. Roads can be blocked, people can't go to work or to schools. The economic damages are high but the number of casualties is usually very limited, because of the nature of the flood. The water slowly rises on the city streets. When the city is on flat terrain the flow speed is low and you can still see people driving through it. The water rises relatively slow and the water level usually does not reach life endangering heights.

River floods: According to H.M.Ranghunath (2006) on Melese Chanie's 2011 thesis, Rainfall over an extended period and an extended area can cause major rivers to overflow their banks. The water can cover enormous areas. Downstream areas may be affected, even when they didn't receive much rain themselves. With large rivers the process is relatively slow. The rain water enters the river in many ways. Some rain will fall into the river directly, but that alone doesn't make the river rise high. A lot of rain water will run off the surface when the soil is saturated or hard. It will flow to small rivers that flow to larger rivers and these rivers flow into even larger rivers. In this way all the rain that fell in a large area (catchment area) comes together in this one very large river. When there is a lot of rain over a long period, it takes time for all the rainwater to reach the river. While the water level slowly rises, officials can decide to evacuate people before the river overflows. The area that is flooded can be huge. Villages surrounded by large stretches of water where cattle would normally graze. Whole communities can become isolated from the rest of the world as roads are blocked and communications are down. When a dike or a dam breaks and a lot of water is released suddenly, the speed of the water at the breach can be compared with the speed of a flash flood. As a larger area gets covered the speed will be reduced. The water spreads out as much as possible flowing to the lower lying areas before slowly rising. A breach is very dangerous for the people living close to it. The strength of the water may carry cars, trees and even houses away and cause loss of life.

Pluvial Floods: According to H.M.Ranghunath (2006) on Melese Chanie's 2011 thesis, Pluvial is a type of flooding that can happen in relatively flat areas. Rain water falling in an area is normally stored in the ground, in canals or lakes, or is drained away, or pumped out. When more rainwater enters a water system than can be stored, or can leave the system, flooding occurs. In this case, rain is the source of the flood: not water coming from a river, but water on its way to the river. That's why it is also called "pluvial flood". Puddles and ponds develop on the land, canals are filled to brim and spill over; gradually a layer of water covers the land. It is like urban flooding, but without the sewage systems and in more rural areas. Because of the gradual character people have time to go indoors or leave the area. The layer of water is no more than centimeters or perhaps decimeters high and causes no immediate threat to people's life. Depending on the economic activity and size of the area that is covered it may cause immense economic damage.

2.2.2 Causes of flooding

Floods are natural phenomena which cannot be prevented (EC 23 October 2007). „Flooding“ is described as a „condition where wastewater and/or surface water escapes from or cannot enter a drain or sewer system and either remains on the surface or enters buildings“. Flooding is often thought of as a result of heavy rainfall, but floods can arise in a number of ways that are not directly related to ongoing weather events. Thus, a complete description of flooding must include processes that may have little or nothing to do with meteorological events. Flooding, by its very nature, is usually a result of both meteorological and hydrologic processes; the character of a flood is determined both by the detailed behavior of the precipitation and by the nature of situation in which the event is likely to occur (soil conditions, amount of antecedent rainfall, and so on). It is not likely that precisely detailed forecasts of flooding events will ever be possible, although it is certainly well within our capability to anticipate the possibility of most flood events. (C A Doswell III, 2003)

2.2.3. Effects of Flooding

Floodwater can seriously disrupt public and personal transport by cutting off roads and railway lines, as well as communication links when telephone lines are damaged. Floods disrupt normal drainage systems in cities, and sewage spills are common, which represents a serious health hazard, along with standing water and wet materials in the home. Bacteria mould and viruses, cause disease, trigger allergic reactions, and continue to damage materials long after a flood. Floods can distribute large amounts of water and suspended sediment over vast areas, restocking valuable soil nutrients to agricultural lands. In contrast, soil can be eroded by large amounts of fast flowing water, ruining crops, destroying agricultural land /

buildings and drowning farm animals. Severe floods not only ruin homes / businesses and destroy personal property, but the water left behind cause's further damage to property and contents. The environment and wildlife is also at risk when damage when damage to businesses causes the accidental release of toxic materials like paints, pesticides, gasoline etc. Floodwater can severely disrupt public and personal transport by cutting off roads and railway lines, as well as communication links when telephone lines are damaged. Unfortunately, flooding not only disrupts many people's lives each year, but it frequently creates personal tragedies when people are swept away and drowned.(www.environment-agency.gov.uk/fun)

2.3 Sustainable Urban Drainage Systems

2.3.1 The concept of sustainable urban drainage systems

The idea with SUDS is that in the best possible way regenerate the natural system of stormwater handling, in order to reduce peak flows and provide treatment for the stormwater on its way to the recipients. With the urbanization follows an increase in hard surfaces, where the water is unable to penetrate. This means that stormwater runs on the hardened surfaces without any retardation. The consequences are high peak flows, which arrive quickly after the storm commences. Since the traditional pipe-systems in the cities normally aren't designed to handle these occasional peak flows, flooding is often the results. With the introduction of Sustainable Urban Drainage Systems, the water will be delayed on its way downstream, in resemblance with nature's way of handling stormwater runoff (Environment Agency, 2003).

A **sustainable drainage system** is designed to reduce the potential impact of new and existing developments with respect to surface water drainage discharges. The term sustainable urban drainage system is not the accepted name, the 'Urban' reference having been removed so as to accommodate rural sustainable water management practices. Increasing urbanization has caused problems with increased flash flooding after sudden rain. As areas of vegetation are replaced by concrete, asphalt, or roofed structures, the area loses its ability to absorb rainwater. This rain is instead directed into surface water drainage systems, often overloading them and causing floods. (Sharma, D., et al, 2008)

The idea behind SUDS is to try to replicate natural systems that use cost effective solutions with low environmental impact to drain away dirty and surface water run-off through collection, storage, and cleaning before allowing it to be released slowly back into the environment, such as into water courses. This is to counter the effects of conventional drainage systems that often allow for flooding, pollution of

the environment – with the resultant harm to wildlife – and contamination of groundwater sources used to provide drinking water. The paradigm of SUDS solutions should be that of a system that is easy to manage, requiring little or no energy input (except from environmental sources such as sunlight, etc.), resilient to use, and being environmentally as well as aesthetically attractive. Examples of this type of system are basins (shallow landscape depressions that are dry most of the time when it's not raining), rain-gardens (shallow landscape depressions with shrub or herbaceous planting), swales (shallow normally-dry, wide-based ditches), filter drains (gravel filled trench drain), bio retention basins (shallow depressions with gravel and/or sand filtration layers beneath the growing medium), reed beds and other wetland habitats that collect, store, and filter dirty water along with providing a habitat for wildlife (Sharma, D., et al, 2008)

A common misconception of SUDS systems is that they reduce flooding on the development site. In fact the SUDS system is designed to reduce the impact that the surface water drainage system of one site has on other sites. For instance, sewer flooding is a problem in many places. Paving or building over land can result in flash flooding. This happens when flows entering a sewer exceed its capacity and it overflows. The SUDS system aims to minimize or eliminate discharges from the site, thus reducing the impact, the idea being that if all development sites incorporated SUDS then urban sewer flooding would be less of a problem. Unlike traditional urban stormwater drainage systems, SUDS can also help to protect and enhance ground water quality. (Sharma, D., et al, 2008)

2.4 Functions of Stormwater 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. Another main objective of storm sewer is to protect;

- Public health and safety;
- Environmental protection;
- Sustainable development;
- Occupational health and safety.

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.(AASHTO, 1991)

2.5 Types of Stormwater drainage system

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 stormwater drainage pipes,
- Channels and culverts. (AASHTO, 1991 et al)

2.5.1 Stormwater Pipe Materials

Following the guidelines from the International Standards Organization for Testing and Materials (ASTM), in order to reduce flooding, promote adequate drainage, and reduce maintenance, certain pipe materials depending upon usage and location of the pipes. Material for pipe used for conveyance of stormwater shall be in accordance with the following (ASTM 2008):

- Reinforced concrete pipe, double-walled high-density polyethylene pipe, corrugated double-walled polyvinyl chloride pipe, corrugated metal pipe and non-reinforced concrete pipe may be used to convey onsite stormwater runoff (stormwater runoff generated by the subject property including areas such as buildings, parking lots, etc.).
- Reinforced concrete pipe is required for all detention basin outlet structures and outlet pipes.
- Reinforced concrete pipe is required when stormwater systems carry water from offsite areas (stormwater runoff generated by areas other than the subject property, where the runoff drains across the subject property) or when there is a potential to cause flooding or damage to adjacent properties. Reinforced concrete pipe is also required for all stormwater systems located within new residential developments (includes residential condominium developments).
- Stormwater pipes installed under city streets shall be reinforced concrete pipe. Stormwater pipes within the roadway prism of city streets and JPEs, but not under the pavement, shall also be of reinforced concrete pipe
- Any stormwater pipe/system that is installed with intent of dedicating to the City, whether inside or outside of the right-of-way, shall be constructed with reinforced concrete pipe.
- Existing residential subdivisions that have a roadside ditch and driveway pipe stormwater system may use reinforced concrete pipe , double-walled high density polyethylene pipe, corrugated

double-walled polyvinyl chloride pipe ,corrugated metal pipe , and non-reinforced concrete pipe as desired by the responsible agency, corporation, or individual. Reinforced concrete pipe is required underneath any driveways or entrances that are heavily traveled.

- For the situations referred to above where reinforced concrete pipe is required, ductile iron pipe might be allowed to be substituted for reinforced concrete pipe when site limitations exist. (ASTM, 2008)

2.6 Alignment of Drainage Structures

Culverts that have internal diameter less than or equal to 1.22m are minor drainage structures. The vertical alignment of a culvert with respect to the stream channel is important to its hydraulic performance, to stream stability, to construction and maintenance costs, and to the safety & integrity of the roadway. Proper alignment is also particular importance to prevent outlet scour or excessive sediment buildup in the culvert barrels. A culvert placed too low in relation to the channel bottom may lose hydraulic performance if the channel aggrades. In addition, a culvert placed at a slope different from the natural channel slope may have problems related to both sediment deposition and bed scour, and this affects hydraulic performance.

A culvert invert slope should match the streambed slope. Placing the culvert on a flatter or steeper gradient from the natural streambed can cause sediment deposition in the barrel. It can also cause scour that removes sediment from the barrel. (AASHTO, 1991 et al)

2.7 Storm Sewer Design

Population growth and urban development can create potentially severe problems In urban water management. One of the most important facilities in preserving and improving the urban water environment is an adequate and properly functioning storm water drainage system. Construction of houses, commercial buildings, parking lots, paved roads, and streets increases the impervious cover in a watershed, and reduces infiltration. Also, with urbanization, the spatial pattern of flow in the watershed is altered and there is an increase in the hydraulic efficiency of flow through artificial channels, curbing, gutters, and storm drainage and collection systems. These factors increase the volume and velocity of runoff and produce larger peak flood discharges from urbanized watersheds than occurred in the pre urbanized condition. Many urban drainage systems constructed under one level of urbanization are now operating under a higher level of urbanization and have in adequate capacity. (Vent cow, 1988)

The following constraints and assumptions are commonly used in storm sewer design practice:

1. Free-surface flow exists for the design discharges; that is, the sewer system is designed for "gravity flow"; pumping stations and pressurized sewers are not considered.
2. The sewers are of commercially available circular sizes no smaller than 8 inches in diameter.
3. The design diameter is the smallest commercially available pipe having flow Capacity equal to or greater than the design discharge and satisfying all the appropriate constraints.
4. Storm sewers must be placed at a depth such that they will not be susceptible to frost, will be able to drain basements, and will have sufficient cushioning to prevent breakage due to ground surface loading. To these ends, minimum cover depths must be specified.
5. The sewers are joined at junctions such that the crown elevation of the upstream sewer is no lower than that of the downstream sewer.
6. To prevent or reduce excessive deposition of solid material in the sewers, a minimum permissible flow velocity at design discharge or at barely full-pipe gravity flow is specified(e.g.,2.5ft/s).
7. To prevent scour and other undesirable effects of high-velocity flow, a maximum permissible flow velocity is also specified.
8. At any junction or man hole the downstream sewer cannot be smaller than any of the upstream sewers at that junction.
9. The sewer system is a dendritic, or branching, network converging in the downstream direction without closed loops. (Vent cow, 1988)

2.8 Factors Affecting Flood Runoff

For all hydrologic analyses, the following factors shall be evaluated and included if they have a significant effect on the final results:

- Drainage basin characteristics including size, shape, slope, land use, geology, soil type, surface infiltration, and storage;
- Stream channel characteristics including geometry and configuration, natural and artificial controls, channel modification, aggradation/degradation, and debris;
- Flood plain characteristics; and
- Meteorological characteristics including precipitation amounts and type (rain, hail, or combinations thereof), storm cell size and distribution characteristics, storm direction, and time rate of precipitation(ERA,2001)

2.9 Hydraulics of Storm Drainage Systems

2.9.1 Flow Type Assumptions

The design procedures presented here assume that flow within each storm drain segment is steady and uniform. This means that the discharge and flow depth in each segment are assumed to be constant with respect to time and distance. Also, since storm drain conduits are typically prismatic, the average velocity throughout a segment is considered to be constant. In actual storm drainage systems, the flow at each inlet is variable, and flow conditions are not truly steady or uniform. However, since the usual hydrologic methods employed in storm drain design are;

- **Rational Method** - only for drainage areas less than 50 hectares (0.5 kilometer²);
- **SCS and other Unit Hydrograph Methods** - for drainage areas greater than 50 hectares;
- **Watershed Regression Equations** - for all routine designs at sites where applicable;
- **Log Pearson III Analyses** - preferable for all routine designs provided there is at least 10 years of continuous or synthesized record for 10-year discharge estimates and 25 years for 100-year discharge estimates; and
- **Suitable Computer Programs** - such as HYDRAIN's HYDRO, HEC 1, and TR-20 will be used to facilitate tedious hydrologic calculations. (ERA, 2001)

2.9.2 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, 2001)

2.9.3 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. (NRCS and ERA, 2001)

2.9.4 Hydraulic Design Elements

General principles relating to channels, culverts, bridges, and other storm drainage elements are listed below.

- 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.
- A stable channel is an important aspect for a proper functioning of highway drainage structures.
- 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.
- Coordination with Ministry of Water Resources shall have high priority in the planning of highway facilities. (NRCS and ERA, 2001)

3. Research Methodology

3.1 Description of the Study Area

3.1.1 General

Location: Kemise is the capital town of Oromiya nationality zone of Amhara national Regional State, which is found 327km north of Addis Ababa and 75Km away from Dessie in southern direction. The town is located at an elevation of 1,437 meters above sea level.

Area: It has an area extension of 1 335 [sqkm].

Population and Coordinate: According to CSA (2007), its population, amounts is to be 23,861 and Its coordinates are 10°43'0" N and 39°52'0" E in DMS (Degrees Minutes Seconds) or 10.7167 and 39.8667 (in decimal degrees). Its UTM position is EM98 and its Joint Operation Graphics reference is NC37-07.

Service center:

- providing grain milling facilities;
- Personal services such as cooked food, lodgings and entertainment;
- Vehicle repair facilities and fuel (diesel and petrol) distribution outlets.

Communications center:

- Providing bus, telephone and postal services.

Health center:

- Government-provided clinics and health centers;
- Privately provided chemists and clinics.

Education Centre:

- The education system is structured into three levels in Ethiopia: elementary Schools (grades 1–6) and junior secondary schools (grades 7–8), where instruction is in the main vernacular of the Oromiya zone (Oromo); and secondary schools (grades 9–12), where instruction is in English.

Religious facilities:

- Catering for the religious needs (churches and mosques) of Coptic Christian and Muslim adherents from both rural and urban areas.

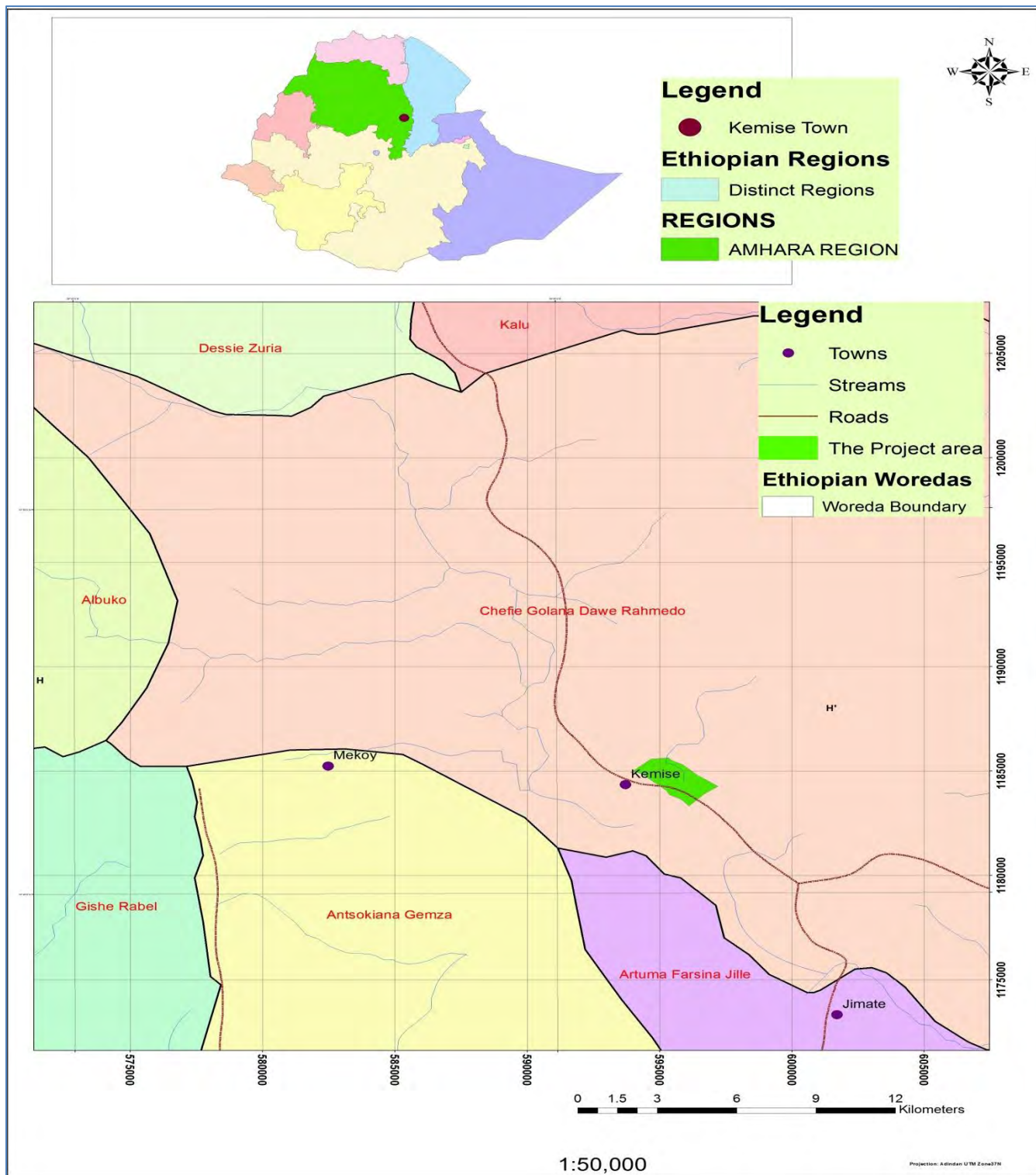


Figure 3.1 Location Map of the Study Area

Topography and Climate: The climatic condition of the area is generally classified as Temperate (Kola) with an altitude that ranges 1000_1564m from m.a.s.l, and a mean annual temperature that varies from 8_24 °C.

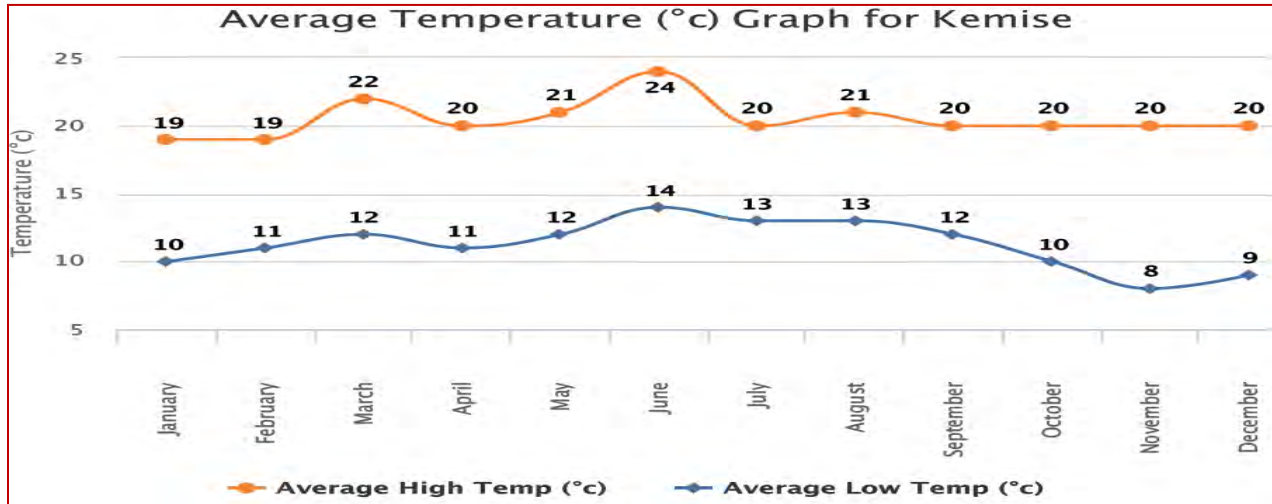


Figure 3.2 Average Temperatures

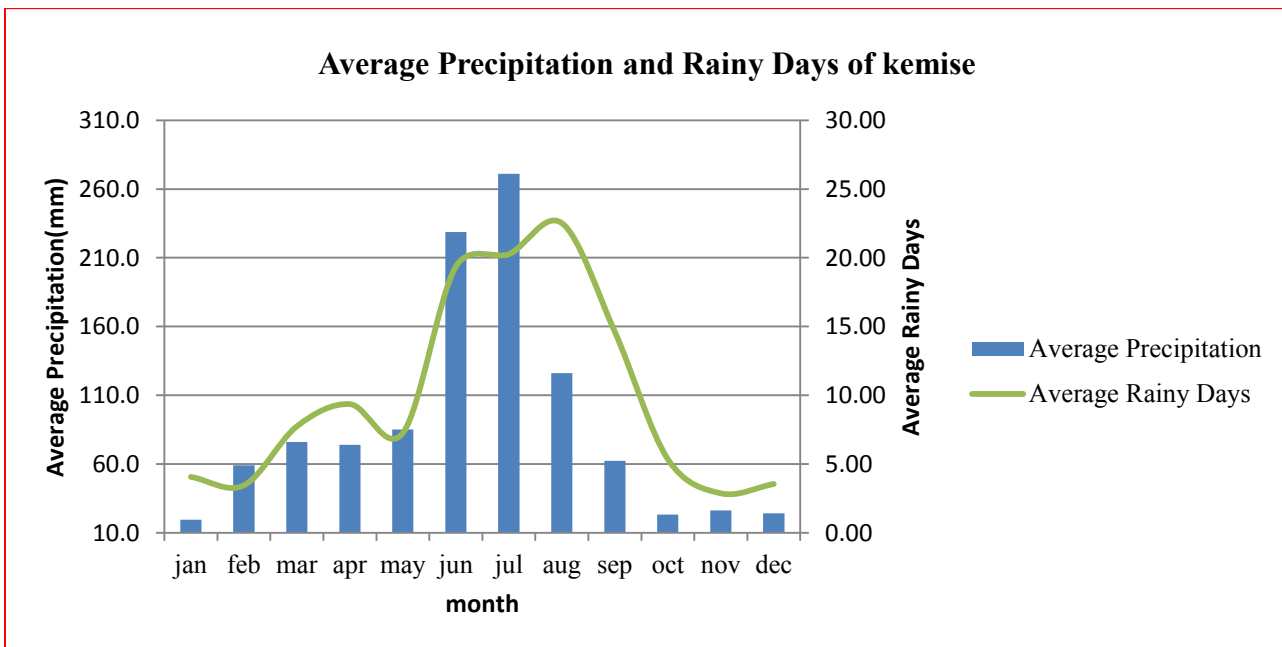


Figure 3.3 Average Daily Rainfalls and Precipitation

The data taken for the above graph is (1985-2015)

Land Use and Land Cover: The land use condition in kemise catchments on the right side of the main road(Addis Ababa-Dessie) includes mainly of agricultural land, , forest land (at the edge of the town), rural and urban settlements. In the upper most part where there is high rainfall, land use is complete in May with sorgom and teff. On the left side is also the same from right side except forest land. The most common soil types are clay, sand, clay-loam, silt-clay-loam, sand-clay, and silt-clay. Land use and soil type have a direct impact on the flood amount, speed and potential to create damage that is why the study gives notice for land use and land cover of the catchment.

3.2 Analysis Method of the study

The thesis will focus on performance assessment of drainage systems on kemise town. Therefore the data's gathered in the above method will be analyzed critically on the responses found from the record data, interviews, officially recognized reviews, etc.

The final data analyzed will be presented with suitable forms such as tabular, diagrammatic, graphical etc, approach with the summary and recommendation to each problems rise in the research.

The main steps that are used to address the specific objectives of this study are;

- Check the quality of data
- Design rainfall analysis (frequency analysis by using different statically parameter then check the coefficient R^2 for each formula and select better value).
- Estimate Design rainfall of shorter duration by using the formula provided by ERA drainage design manual(2013)
- Develop IDF curve for 2, 5,10,25,50 and 100 year return period.
- Delineate water shed using DEM and ArcGIS
 - i. Applying hydrological or mathematical equation to determine peak runoff Estimation of Time of concentration
 - ii. Estimation of Rainfall Intensity by using Tc and IDF curve
 - iii. Estimation of weighted runoff coefficient based on land use composition, and its percent of area coverage, soil type and permeability.
 - iv. Finally Estimation of peak discharge by using rational method.(ERA,2001)
- Estimation of hydraulic parameter by using Manning's equation for existing and proposed and then fixing the size of drainage structures.

- The final step is to solve the problem of town based on the analysis and by recommended possible mitigation measure.

3.3. Materials

The materials used for this research are:-

- ARC-GIS to obtain hydrological and physical parameters and spatial information of the catchments of the study area.
- DEM data is used as an input data for ARC-GIS software for catchment delineation and estimation of catchment characteristic.
- Google Earth Software to verify water shed and divides of catchments of the study area.
- Digital camera, GPS device, and measuring tape.
- Hydrological and meteorological data and etc.

3.4. Research design

This research relies on both descriptive and explanatory approaches when the examining of the current trend and basic principles in the storm water drainage construction carried out.

- Photographing the road sections suffering from inadequate storm water drainages
- Making a basic diagnosis of the storm water drainage problem
- Defining the solution for the problem

3.5. Data Sources

For this study, information and data collection were obtained via two sources which include: Primary and secondary sources.

A. Primary sources

i) Administering of interview

Interview for concerning body like head of municipality and water bureau etc. Concerning the most likely causes, effect of water drainage challenges as well as environmental challenges relating to the improper utilization of the drainage systems in the chosen locations.

ii) Study Area Observations:

For this study, pictures of different scenario as it relates to different drainage issues were taken to show the true state of things in the study area. Observations and discussion with residents were also made a measuring the size of the existing drainage structures, measuring the elevation difference

of the topography and gathering information is carried out about the overall performance of drainage structures during the rainy season and recorded. This also gave an insight to the major challenges encountered within these areas of study.

B. Secondary Sources

This was important to harvesting information on the over-looked causes of poor drainage challenges and unconstructed stormwater drainage. Other secondary sources of information that was used include books, journals, manuals, conference proceedings etc. Both quantitative and qualitative techniques in data collection and analysis were utilized as main instruments.

3.6 Hydro climatic Data Availability and Its Quality

3.6.1 Rainfall Data

Rainfall data sets in one rain gage station, but due to incompleteness of the data additional four rain gauge station data are used for thesis to fill unrecorded rainfall data. But due to the rain gage station in the study area is one the outlier test is appropriate to check the quality of the data.

3.6.2 Estimating missing rainfall data

Due to the absence of observer or instrumental failure rainfall data record occasionally are incomplete. In such a case one can estimate the missing data by using the nearest station rainfall data. There are different approaches for estimating missing rainfall data varying with and based on the effect of orography on rainfall, distance between the rainfall stations and the variation of rainfall amount recorded on the stations (Dingman, 2002).

Among different method Normal ratio method is one them which is recommended to estimate missing rainfall data in regions where annual rainfall between stations differ by more than 10%.

If for example rainfall data at day 1 is missed from station X having mean annual rainfall of N_x and there are three surrounding stations with mean annual rainfall of N_1 , N_2 , and N_3 then the missing data P_x can be estimated(yilma,2011)

$$P_x = \frac{1}{3} \left(P_1 \frac{N_x}{N_1} + P_2 \frac{N_x}{N_2} + P_3 \frac{N_x}{N_3} \right) \quad (3.1)$$

Where: P_x . missing rainfall data(daily, monthly or yearly)

P_1 , P_2 and P_3 – rainfall data at nearest different station (daily, monthly or yearly)

N_x - mean annual rainfall at missed station

N_1 , N_2 , and N_3 - mean annual rainfall at different nearest station

Check the quality of data

An outlier is an observation that deviates significantly from the bulk of the data, which may be due to errors in data collection, or recording, or due to natural causes. The presence of outliers in the data causes difficulties when fitting a distribution to the data. Low and high outliers are both possible and have different effects on the analysis.

3.7 Design rain fall analysis

3.7.1 Estimation of average depth of rainfall over a catchment

Several methods are commonly used for estimating average rainfall over a watershed. Choice of method requires judgment in consideration of quality and nature of the data, and the importance, use, and required precision of the result. Here in three methods of estimating areal average depth of rainfall are discussed. The first is the arithmetic mean method, the second is the Thiessen polygon method, and the third is the isohyetal method. Before we discuss these methods, first elaborate techniques for changing point rainfall to area rainfall in case of only one rain gauge is available in and around the area the arithmetic mean method is appropriate for the study area and were used for this thesis .

3.7.2 Frequency Analysis

Extreme rainfall events and the resulting floods can take thousands of lives and cause billions of dollars in damage. Flood plain management and design of flood control works, reservoirs, bridges, and other investigations need to reflect the likelihood or probability of such events. Hydrological studies also need to address the impact of unusually low rainfalls causing low stream flows which affects for example water quality and water supply. The term frequency analysis refers to the techniques whose objective is to analyze the occurrence of hydrologic variable within statistical framework, by using measured data and basin predictions on statistical laws. The historical rainfall data available is a 24hr duration rainfall hence appropriate IDF reduction methods need to be used to obtain rainfall intensities of shorter duration.

Any probability distribution can be used as the model but the reliability of the distribution is checked by the goodness of fit tests. Among many method, Gumbel and Log Pearson Type III methods are used for these research based on as suggested by Ethiopian Drainage Design Manual (ERA, 2013).

3.7.3 Design rainfall of shorter duration

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

$$R_{Rt} = \frac{t(b+24)^n}{24(b+t)^n} \quad (3.2)$$

Where:

R_{Rt} = Rainfall depth ratio R_t : R_{24}

R_t = Rainfall depth in a given duration t

R_{24} = 24hr rainfall depth

Coefficients $b = 0.3$ and $n = 0.78 - 1.09$

The methods employed to develop IDF curve for the shorter duration events using the above equations are as follows. Among many frequency analysis Log Pearson type III better R^2 value, so for this thesis Log Pearson type III distribution are selected

Using the trend line equation obtained from Log Pearson type III distribution method of frequency analysis, i.e. $y = 0.5456.x + 68.84$ where y is 24-hour rainfall depth (R_{24}) of a return period x under consideration, R_{24} is calculated for 2, 5, 10, 25, 50 and 100 year return period.

Rearranging the above equation gives

$$R_t = \frac{t(b+24)^n}{24(b+t)^n} * R_{24} \quad (3.21)$$

Substituting Intensity (mm/hr) in the above equation

$$I_t = \frac{R_t}{t} \quad (3.22)$$

$$I_t = \frac{R_{24}(b+24)^n}{24(b+t)^n} \quad (3.3)$$

Using $b = 0.3$ and $n = 0.92$ as suggested by ERA manual results are tabulated for rainfall durations 10, 20, 30 ... 180 minutes.

The resulting table is graphed for each return period.

In order to combine into software and systematize the estimation of runoff representation of intensity duration frequency relationship with an equation is required.

For instance, the coefficient of determination, R^2 , of a 10 and 25 year return period for duration less than 180 minutes is approximated to one(0.99) . Hence the polynomial equation can be used to represent the IDF curve of 10 and 25 year return period to obtain rainfall intensity for a given duration, provided that the duration is below 3 hours. Equations obtained representing IDF curves for different return period are presented in the results section.

3.8 Hydrological Modeling Using ArcGIS and DEM

3.8.1. Water shed delineation

Hydrological modeling starts from delineating streams and watersheds, and obtaining watershed properties such as area, slope, flow length, flow direction, stream network, etc. Using digital elevation models (DEM) and computer programs have made it possible to perform automated analysis.

3.9 Hydrological Equations for Determining Peak runoff

3.9.1 The SCS method

Soil Conservation Service (SCS) Curve Number (CN) model estimates precipitation excess as a function of cumulative precipitation, soil cover, land use, and antecedent moisture. SCS developed the method for small and mid-sized watersheds (< 1000 sq. km) (NRCS, 2004) the method also considered the initial rainfall losses to interception, depression storage and infiltration rate that decreases during the course of storm and converts basin storage into something simpler and more manageable (a “curve number” CN). The SCS runoff equation is a method of estimating direct runoff from 24-hour or 1-day storm rainfall.

It is therefore, potentially more accurate than the rational method and is applicable when the catchment area is larger than 50 hectares (ERA, 2011)

The main input variables for the SCS methods:

- drainage area,
- weighted runoff curve number (CN),
- time of concentration (T_c) and
- design point rainfall (P)

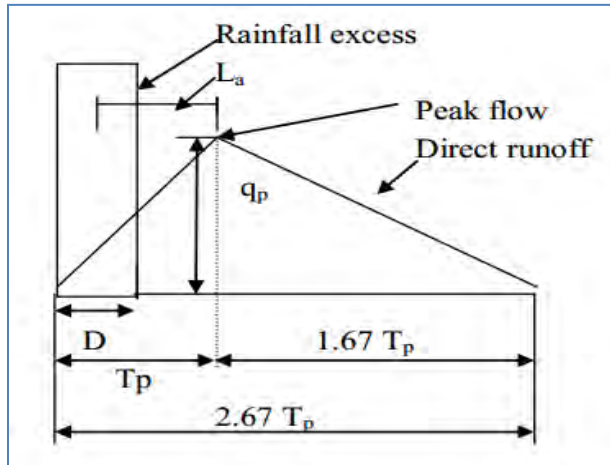


Figure 3.1 SCS triangular hydrograph

$$q_p = \frac{0.208Ard}{0.5D+0.6tc} \quad (3.4)$$

Where: q_p = peak discharge (m³/s)
 r_d = the excess rainfall depth (mm)
 A = watershed area (km²)
 D = excess rainfall depth (runoff depth)
 tc = time of concentration

The depth of runoff resulting from a required return period rainfall depth of duration corresponding to the time of concentration tc is estimated by

$$r_d = \frac{(P-0.2S)^2}{(P+0.8S)} \quad \text{For } P > 0.2S \quad (3.41)$$

$$r_d = 0 \quad \text{For } P \leq 0.2S \quad (3.42)$$

Where:

I_a = Initial Abstraction ($I_a = 0.2S$)
 r_d = Excess Rainfall
 P = Total Rainfall
 S = Potential Maximum Storage

The potential maximum soil water retention, S , is related to hydrologic soil properties, land cover and management hydrologic soil properties, land cover and management conditions as well as the soil moisture status of the catchment prior to rainfall event and there relation with curve number(CN) is

$$S = \frac{25400}{CN} - 254 \quad (3.43)$$

The CN number is selected according to soil type, moisture condition and land cover/use of the watershed. Three basic antecedent rainfall conditions are present for both dependent variables(S and CN)

- Normal conditions, AMC(II)
- Dry conditions, AMC(I)
- Wet conditions, AMC(III)

$$CN_{III} = \frac{CN_{II}}{0.43 + 0.0057CN_{II}} \quad (3.43a)$$

$$CN = \frac{75}{120} CN1 + \frac{45}{120} CN2 \quad (3.43b)$$

3.9.1.1 Time of Concentration

The time of concentration t_c is estimated by; (Kirpich formula)

$$t_c = 0.0078L^{0.77}S^{-0.385} \quad (\text{For bare soil or flow in road side ditch}) \quad (3.44a)$$

$$t_c = 0.0028L^{0.77}S^{-0.385} \quad (\text{For overland flow on concrete or asphalt}) \quad (3.44b)$$

$$t_c = 0.0014L^{0.77}S^{-0.385} \quad (\text{For concrete channels}) \quad (3.44c)$$

Where:

L = L=length of channel/ditch (km)

S = average watershed slope, (m/m)/land slope

t_c = time of concentration (min)

3.9.1.2 Catchment Area

Like rational method, the catchment area can be determined from topographic maps and field surveys. However, for large catchment areas, it is necessary to divide the area into sub-catchment areas to account for major land use changes. In general the drainage area contributing to the system being designed and the drainage sub area contributing to each inlet point must be measured.

3.9.1.3 Hydrological Soil Group

Soil properties influence the relationship between runoff and rainfall since soils have differing rates of infiltration. Permeability and infiltration are the principal data required to classify soils into Hydrologic Soils Groups (HSG). Based on infiltration rates, the Soil Conservation Service (SCS) has divided soils into four hydrologic soil groups as follows:

Group A: Sand, loamy sand or sandy loam. Soils having a low runoff potential due to high infiltration rates. These soils primarily consist of deep, well-drained sands and gravels and have a high rate of water

transmission (greater than 7.62mm/hr). Group A soils typically have less than 10 percent clay and more than 90 percent sand or gravel and have gravel or sand textures. Some soils having loamy sand, sandy loam or silt-loam textures can be placed in this group if they are well aggregated, of low bulk density, or contain greater than 35 percent rock fragments (NRCS, 2007)

Group B: Silt loam, or loam. Soils having a moderately low runoff potential due to moderate infiltration rates. These soils primarily consist of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission (3.81mm/hr-7.62mm/hr). Group B soils typically have between 10percent and 20 percent clay and 50 percent to 90 percent sand and have loamy sand or sandy loam textures. Some soils having loam, silt loam, silt, or sandy clay loam texture can be placed in this group if they are well aggregated, of low bulk density, or contain greater than 35 percent rock fragments (NRCS, 2007).

Group C: Sandy clay loam. Soils having a moderately high runoff potential due to slow infiltration rates. These soils primarily consist of soils in which a layer exists near the surface that impedes the downward movement of water or soils with moderately fine to fine texture. These soils have a low rate of water transmission (1.27mm/hr to3.81mm/hr). Group C soils typically have between 20 percent and 40 percent clay and less than 50 percent sand and have loam, silt loam, sandy clay loam, clay loam, and silty clay loam textures. Some soils having clay, silty clay, or sandy clay textures placed in this group if they are well aggregated, of low bulk density, or contain greater than 35 percent rock fragments (NRCS, 2007).

Group D: Clay loam, silty clay loam, sandy clay, silty clay or clay. Soils having a high runoff potential due to very slow infiltration rates. These soils primarily consist of clays with high swelling potential, soils with permanently-high water tables, soils with a clay pan or clay layer at or near the surface, and shallow soils over nearly impervious parent material. These soils have a very low rate of water transmission (0-1.27mm/hr). Water movement through the soil is restricted or very restricted. Group D soils typically have greater than 40 percent clay, less than 50 percent sand, and have clayey textures. In some areas, they also have high shrink-swell potential (NRCS, 2007).

3.9.2 Rational Method

In the design of stormwater drainage system, the main purpose of hydrologic analysis is to determine the maximum amount of run-off (peak discharge) that can be accumulated at certain storm drainage outlet (usually a ditch) along a highway/access road alignment section.

The Rational method, 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. For this thesis Rational Method is appropriate because of area for each catchment is less than 50 hectare (0.5sqkm). The peak runoff is given by the following expression:

$$Q = 0.00278 * C * I * A * F.S \quad (3.5)$$

Where

- Q – Discharge at outlet (m³/s)
- C – Rainfall-Runoff Coefficient
- I – Maximum probable rainfall Intensity (mm/hr)
- A– Catchment Area (hectares)
- F.S – Factor of Safety (factor of ignorance) (Optional)

The stormwater drainage outlet usually has a delineated tributary catchment area/ watershed, designated with variable A(Area) in the above equation, that contributes runoff to it, the size of which can be easily determined from photogrammetric, and less accurately, by the help of maps (1:50,000 and above if available) or topographic survey.

The main input variable to use rational method is;

- Rainfall intensity
- Rainfall duration
- Rainfall frequency
- Catchment area
- Hydrologic abstractions
- Runoff Concentration
- Run-off Diffusion

But, the peak discharge is the product of;

- Runoff coefficient
- Rainfall intensity
- Catchment area

Although simplistic, the rational method, especially coupled with rainfall frequency analysis and a judicious fine-tuning of runoff coefficients, is generally considered to serve justice to the determination of runoff quantity for storm drainage purposes with reasonable dependability.

The procedures in rational method to determine peak discharge are:

1. Collect the necessary information for each sub area or catchment such as;
 - i. Drainage area
 - ii. Land use
 - iii. Soil types (its permeability/highly permeable or impermeable)
 - iv. Distance from the farthest point of the drainage area to the point of discharge
 - v. Difference in elevation from the farthest point of the drainage area to the point of discharge
2. Determine the time of concentration for the selected recurrence interval with duration equal to the time of concentration
3. Determine the rainfall intensity for the selected recurrence intervals/return period
4. Select the appropriate runoff coefficient(C).
5. Compute the design flow ($Q = 0.00278CIA$)

3.9.2.1 Runoff Coefficient C Determination

The runoff coefficient C is the least precise variable of the rational method. Its Use in the formula implies a fixed ratio of peak runoff rate to rain fall rate for the drainage basin, which in reality is not the case. Proper selection of the runoff coefficient requires judgment and experience on the part of the hydrologist. The Proportion of the total rainfall that will reach the storm drains depends on the percent imperviousness, slope, and ponding character of the surface. The runoff coefficient accounts for the effects of infiltration, detention storage, surface retention, evapotranspiration, flow routing and interception.

$$C_w = (A_1C_1 + A_2C_2 + \dots + A_nC_n) / (A_1 + A_2 + \dots + A_n) \quad (3.5a)$$

C_w = Weighted Runoff Coefficient

C_1, C_2, \dots, C_n = coefficient of runoff for parts of the drainage area.

A_1, A_2, \dots, A_n = parts of drainage areas with different runoff coefficients.

Table 4.1 Runoff Coefficient for Use in Rational Method

Type of Drainage Area	Runoff Coefficient C
Business: Downtown areas	0.7-0.95
Neighborhood areas	0.5-0.7
Residential: Single-family	0.3-0.5
Multi units, detached	0.4-0.6
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

Source; (ERA, 2013)

3.9.2.2 Rainfall Intensity

The rainfall intensity i is the average rainfall rate in mm per hour for a particular Drainage basin or sub basin the design duration is equal to the time of concentration for the drainage area under consideration. The intensity is selected on the basis of the design Rainfall duration and return period. The return period is established by design standards or chosen by the hydrologist as a design parameter.

3.9.2.3 Time of Concentration

Runoff is assumed to reach a peak at the time of concentration t_c When the Entire watershed is contributing to flow at the outlet. The time of concentration Is the time for a drop of water to flow from the remotest point in the watershed to the point of interest. A trial and error procedure can be used to determine the critical time of concentration where there are several possible flow paths to consider. The

time of concentration to any point in a storm drainage system is the sum of the inlet time to (the time it takes for flow from the remotest point to reach the sewer inlet), and the flow time tf in the upstream sewers connected to the outer point: The velocity of flow depends on the catchment characteristics and slope of the water course.

3.9.2.4 Drainage Area

The size and shape of the catchment or subcatchment under consideration must be determined. The area may be determined by plan metering topographic maps or by field surveys where topographic data has changed or where the mapped contour interval is too great to distinguish the direction of flow. 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.

3.9.2.5 Time of Concentration T_c Determination

Many empirical equations are available for calculating time of concentration for a watershed. Among many that three equations are used for this thesis. The Manning Kinematic equation will be discussed for overland sheet flow. The NRCS Method will be used for shallow concentrated flow. Finally, the Manning Equation will be discussed for flow in a channel.

The time of concentration is the time for a drop of water to flow from the remotest point in the watershed to the point of interest.

- i. Sheet flow
- ii. Shallow concentrated flow, and
- iii. Open channel flow

Travel Time

Water moves through a catchment area as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. The type that occurs is a function of the conveyance system and is best determined by field inspection.

Travel time is the ratio of flow length to flow velocity:

$$T_t = L / (3600V) \quad (3.51)$$

Where:

T_t = travel time, hr

L = flow length, m

V = average velocity, m/s

3600 = conversion factor from seconds to hours.

i. Sheet Flow

Sheet flow is flow over plane surfaces. It usually occurs in the headwater of streams and its determined from the friction value (Manning's n) is an effective roughness coefficient that includes the effect of raindrop impact; drag over the plane surface; obstacles such as litter, crop ridges, and rocks; and erosion and transportation of sediment.

$$T_t = [0.091 (nL)^{0.8} / (P_2)^{0.5} S^{0.4}] \quad (3.51a)$$

Where:

T_t = travel time, hr

n = Manning's roughness coefficient

L = flow length, m

P_2 = 2-year, 24-hour rainfall, mm

S = slope of hydraulic grade line (land slope), m/m

Manning's kinematic solution is based on the following:

1. Shallow steady uniform flow,
2. Constant intensity of rainfall excess (rain available for runoff),
3. Rainfall duration of 24 hours, and
4. Minor effect of infiltration on travel time

ii. Shallow Concentrated Flow

After a maximum of 100 meters, sheet flow usually becomes shallow concentrated flow. And the average velocity for this flow can be determined by the following formula, in which average velocity is a function of watercourse slope and type of channel. (ERA, 2002)

$$\text{For Unpaved} \quad V = 4.9178 (s)^{0.5} \quad (3.51b)$$

$$\text{For Paved} \quad V = 6.1961 (s)^{0.5} \quad (3.51c)$$

Where:

V = average velocity, m/s

S = slope of hydraulic grade line (watercourse slope), m/m

According to ERA, 2002 the above two equations are based on the solution of Manning's equation with different assumptions for n (Manning's roughness coefficient) and r (hydraulic radius, meters). For unpaved areas, n is 0.05 and r is 0.12; for paved areas, n is 0.025 and r is 0.06.

After determining average velocity estimate travel time for the shallow concentrated flow segment using equation (4.51)

iii. Open Channels Flow

Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, or where blue lines (indicating streams) appear on Ethiopian Mapping Authority (EMA) topographic maps (1:50,000). Average flow velocity is usually determined for bank-full elevation. Manning's equation or water surface profile information can be used to estimate average flow velocity. When the channel section and roughness coefficient (Manning's n) are available, then the velocity can be computed using the Manning Equation

$$V = (R^{2/3} S^{1/2})/n \quad (3.51d)$$

Where: V = average velocity, m/s

R = hydraulic radius, m (equal to a/pw)

A = cross sectional flow area, m^2

Pw = wetted perimeter, m

S = slope of the hydraulic grade line, m/m

n = Manning's roughness coefficient

The time of concentration is the sum of T_t values for the various consecutive flow segments:

$$T_c = T_{t1} + T_{t2} + \dots + T_{tm} \quad (3.52)$$

Where:

T_c = time of concentration, hr

m = number of flow segments

3.10 Hydraulic Equations

3.10.1 Manning's Equation

Discharge is determined for a known opening size of the drainage structure and bottom slope and/or the size of the drainage structure is determined for a known discharge and bottom slope by trial and error method. The Manning's equation can be used for uniform flow in a pipe, and stream channel, but the Manning's roughness coefficient needs to be considered variable, dependent upon the depth of flow. The Manning's equation is used for calculating the cross-sectional area, wetted perimeter, and hydraulic radius for flow of a specified depth in a pipe of known diameter and/or stream channel cross-section. Manning's equation is applicable for a constant flow rate of water through a channel with constant slope, size & shape, and roughness.

$$Q = \frac{AR^{2/3}S^{1/2}}{n} \quad (3.6)$$

Where, Q = the volumetric flow rate passing through the channel reach in m³/sec.

A = the cross-sectional area of flow normal to the flow direction in m²

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

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

R = the hydraulic radius = A/P.

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

4. Result and Discussion

4.1 Existing Condition

4.1 .1 Existing Drainage Facilities

Existing storm drainage facilities are generally classified into closed and open drainage lines. Closed drainage lines are found in some areas especially along main roads. Open drainage channels, constructed by masonry are found along sub-mains and local roads. In many localities, access roads serve as wide open channels with severe erosion and flooding problems. Within the two broad classifications, there are three types of storm drainage facilities in the city, namely; access roads which serve as open drainage channels (gravel, earth surfaces and stone paved surfaces), concrete pipe conduits and stone lined open ditches. Regarding dimensions of existing drainage facilities, cement pipes vary from 60cm up to 120cm and that of masonry open channels vary depending on the need and design of each channel. The shape is: Rectangular.

Except in some places, the topography of kemise is generally suitable for both natural and artificial drainage facilities. However, drainage service in the city can be considered inadequate in terms of quality and coverage. Drainage facilities in the whole part of the town require construction of large number of drainage facility for existing and new or for unconstructed areas.

Currently, the storm water drainage management of the city of kemise is not efficient; as a result design construction and managing problem. Therefore, careful design of storm water drainage master plan is crucial.

4.1.2 Current Situation

The main challenges of the current drainage system are:

- Drainage systems are not well connected;
- Drainage systems are do not have the capacity to carry large amounts of water, hence resulting in overflowing;
- Ponds or other spaces are not properly allocated to accommodate overflow of flooding;
- Most ditches do not have proper slope to let water pass through them;
- In some areas there are no drainage systems provided at all
- Some of existing drainage ditches have been silted by sand and other rubbish materials



Figure 4.1 Drainage facility filled by rubbish materials and Small size of ditch along main road



Figure 4.2 Crossing of access-road along with drainage and filled by sediment



Figure 4.3 Open drainage system with no outlet and improper slope



Figure 4.4 Half open and half close drainage system silted and grassed

During the assessment the following effects are visited;

Existing drainage network Condition: The inspection work which consists of review of the available documents such as reports, as-built drawings, etc. will further help to justify and predict possible solution for the study area

Major dominant problems drainage problems that could be encountered in the area are:

- flooding or inundation,
- sedimentation,
- traffic disturbance,
- flood overtopping,
- deterioration of roads,
- pollution and unsanitary conditions,
- swamp formation at lower reaches of the catchment's, and
- Damage to properties like existing drainage facilities, buildings or houses, fences and some others.
- deposition of refuse,
- stagnation of water,

4.1.3 Design and Construction Practices

Among many problem the main cause for the over topping, Sedimentation, deterioration of roads, flood overtopping, was the lack of detail flood information during rainy season and inadequate hydraulic design. The construction of the drainage ditch design was carried out without some rational or statistical assessment of the expected flow which means that ignoring hydrological analysis and calculating

hydraulic parameters during the design stage. The hydrological analysis is used to now peak flood of surrounding catchment. In general, drainage crossings must be designed to pass the appropriate storm flows and debris or to survive overtopping.

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. If proper hydrological analysis and hydraulic calculation were not practiced, either overdesign or under design would occur that both involve excessive costs on a long-term basis.

A drainage structure designed to carry allowable recurrence interval 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. Design of the drainage structure for existing in study area is under estimated to carry out peak runoff and not properly managed by municipality even to reduced pollution and health related problem. Therefore appropriate measure is carried out in the next chapter even for existing drainage facility with proper estimation of hydrological and hydraulic analysis. The sectional views of existing drainage system along with road section are;

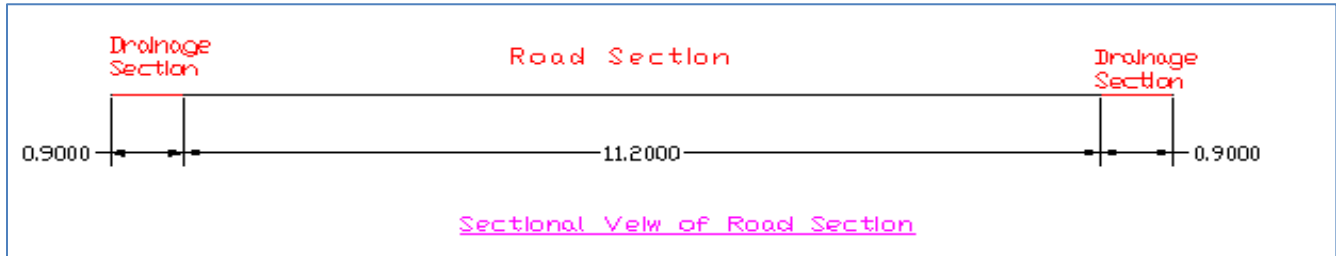


Figure 4.5 Road cross section corresponding to drainage at catchment two

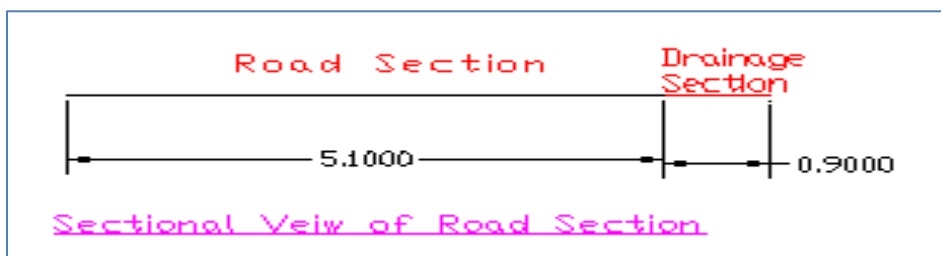


Figure 4.6 Road cross section corresponding to drainage at catchment five

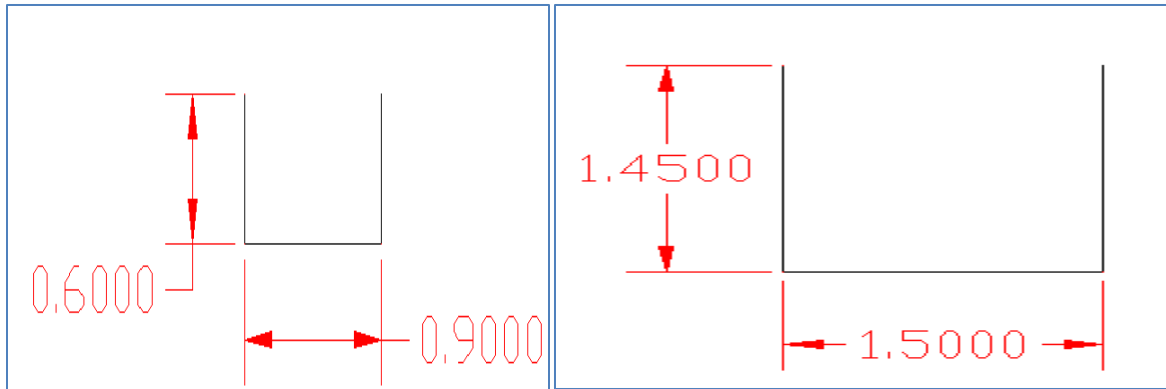


Figure 4.7 Representative existing drainage sizes at different catchment

Note: The dimension in the above figures is represented in meter (m)

4.1.4 Preliminary Engineering

For site investigations collecting topographical maps, infrared photography, remote sensing images, GIS coverage, and aerial photographs are required. Topographic maps can help when we are locating a drainage facility.

4.1.5 Hydrology and Hydraulics

Hydrological parameters calculation should be completed familiar with the local conditions and stream flows. The calculation of hydrology are carried out using rainfall data relate with empirical equation such as Gumbel, log-Pearson III and rational and SCS method for frequency analysis and peak discharge determination respectively

4.1.6 Road drainage Alignment

A good drainage alignment with respect to slope is important to avoid silting, purpose less and other related problem caused by improper design and construction.

4.1.7 Construction Practice

The proper construction practice is important after proper design for drainage structures to function properly. Only proper design by itself does not make the drainage structure to serve properly up to its design life but also proper construction practice must be carried out by appropriate personnel according to the design. For example the drainage system constructed in catcment2 or at osis improper alignment, which means that by missing its design even not properly designed therefore it require some modification like maintaining its slope and proper alignment in addition to appropriate measure presented in the next chapter.

In this chapter the results obtained from the analysis methodologies detailed in chapter three will be presented in order of their appearance. Furthermore, short discussion as to what the results mean will be presented together with the results.

4.2 Intensity–Duration–Frequency Curves (IDF)

The resulting IDF curve from steps shown in section 3.7.3 is as follows. The IDF curve is developed from 24-hour rainfall data of 31 years i.e. 1985 to 2015, obtained from Ethiopian Meteorological Agency rainfall gauge located in kemise town, Ethiopia. Appropriate reduction equation as described in the methodology Section has been applied.

For this thesis calculated IDF curve for specific study area (Kemise) is applicable, but for comparison of the result found from this study and IDF curve developed by ERA for region B, C and D are presented. Kemise also found in the region C (wollo), but the data ranges are different to develop IDF curves for the study area because of the data for this study is currently compared with ERA was used to develop IDF curve for the station. Even though the data ranges used for the study and Ethiopian Road Authorities are not the same, the IDF curve developed by ERA is presented here for comparison.

The comparison results are too difference this is because the rainfall data used for developing IDF curve for kemise town is from 1985 to 2015 and ERA used to develop IDF curve was seven year less than from this study data (up to 2001) and the other reason is ERA develop IDF curve for different station (B, C and D), and also the data was compiled data due to those reason the intensity values are too much different.

Basic Description of IDF Curve

Defining the rainfall intensity is the basic difference Stormwater Drainage system .Rainfall intensity for the design storm is needed to calculate peak runoff rate from a drainage area for the design of storm water structures using the rational method. For this study IDF curve from hydrological section of Kemise Town is applicable, but to correlate the calculated or developed IDF curve it is necessary to correlated calculated IDF curve with ERA drainage design manual even the values of rainfall intensity is much difference. In ERA drainage design manual Ethiopia is divided into several hydrological regions which display similar rainfall patterns. Kemise Town falls in region C of this division as mentioned in the above section.

According to the result of the rainfall intensity for this study and ERA developed for the station(c) which means that for wollo the maximum and the minimum rainfall intensity for the design period (10 years for design and 25 years for check), the rainfall intensity is for 5 minute duration 323mm/hr and for 120 minute duration 18.4mm/hr and for 120 minute duration 190mm/hr.

The values of rain fall intensity is much different because of the data used to develop the IDF curve for this study and the data used to develop for IDF curves for ERA as mentioned on the previous section of this thesis. And the detailed table is presented on Table 4.1 and Appendix 1 Table 1.3.

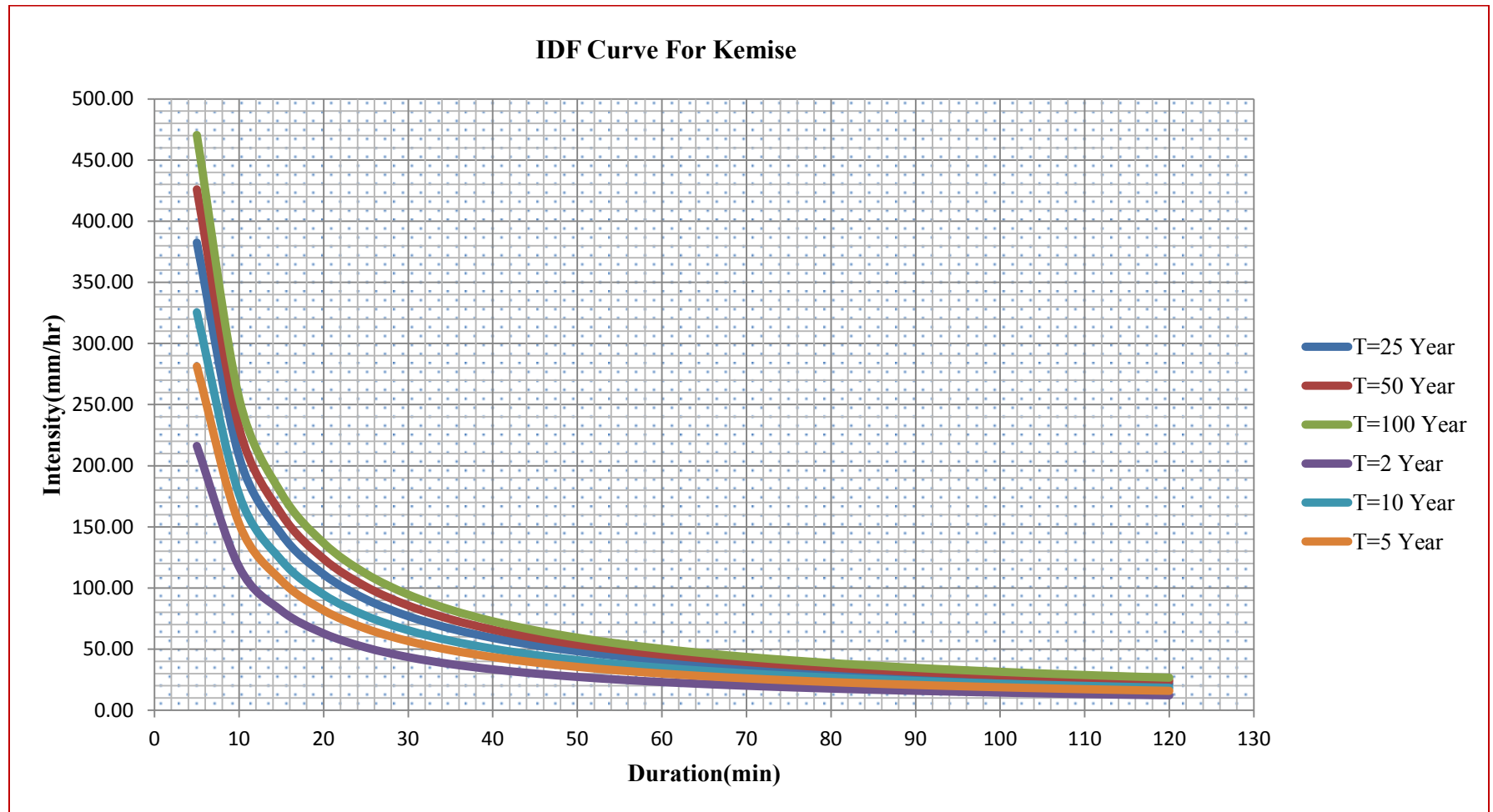


Figure 4.8 IDF Curve for kemise town

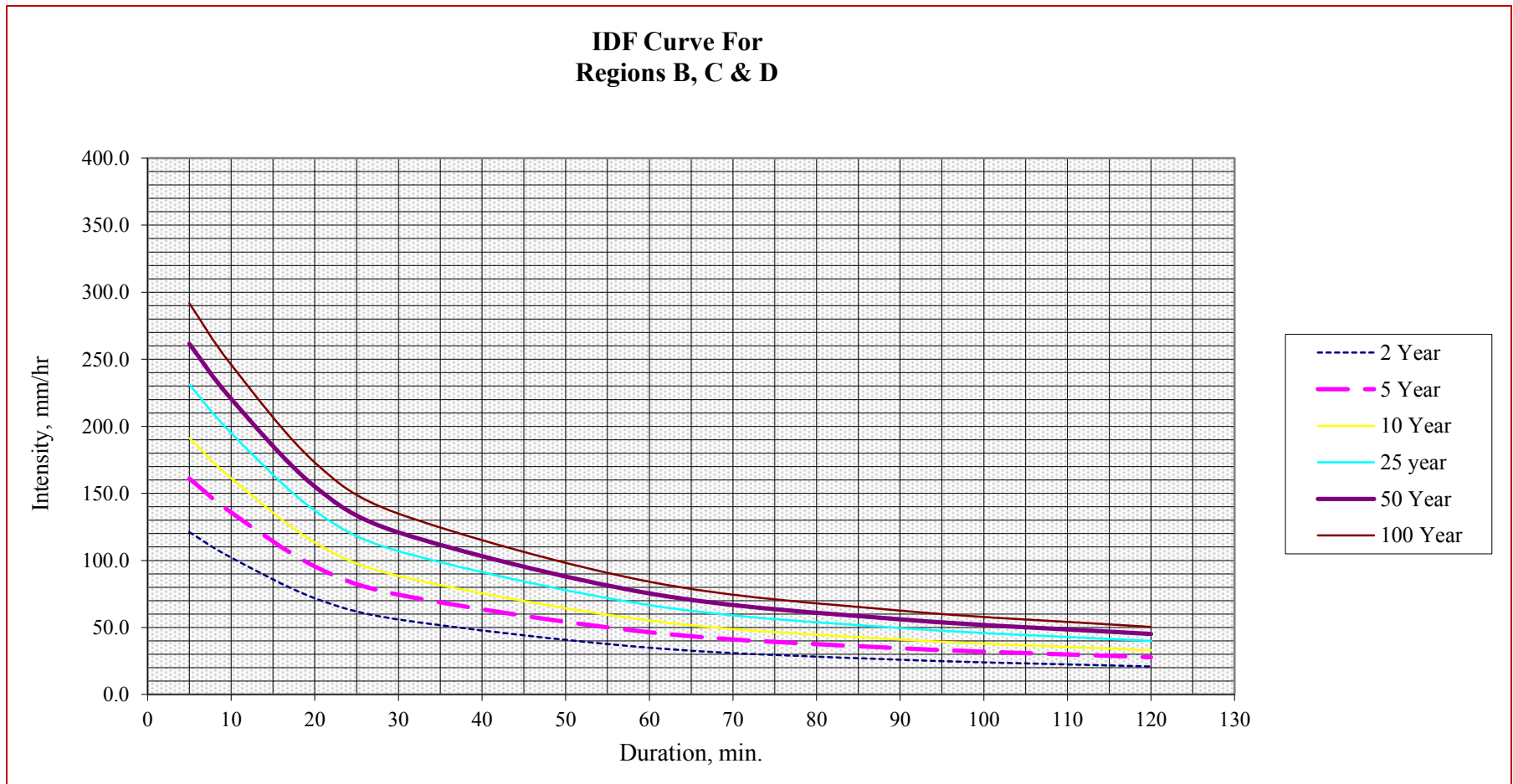


Figure 4.9 Intensity-Duration-Frequency curves for the region developed by ERA

Table 4.1 Comparison of IDF curve results with ERA IDF develop for station

	Self	ERA	Self	ERA	Self	ERA	Self	ERA	Self	ERA	Self	ERA
Duration	T-2	T-2	T-5	T-5	T-10	T-10	T-25	T-25	T-50	T-50	T-100	T-100
15	81.5	102.2	106.1	148.6	122.7	185.2	144.2	213.5	170.3	241.4	177.4	269.1
30	43.5	90.4	56.6	131.5	65.4	178.0	76.9	188.7	90.8	213.5	94.6	237.8
45	30.0	78.7	39.1	114.4	45.2	170.9	53.1	163.9	62.7	185.6	65.4	206.5
60	23.1	67.0	30.0	97.3	34.7	163.7	40.8	139.1	48.2	157.7	50.2	175.2
75	18.8	55.2	24.5	80.3	28.3	156.5	33.3	114.3	39.3	129.7	40.9	143.9
90	15.9	43.5	20.7	63.2	24.0	149.3	28.2	89.6	33.3	101.8	34.6	112.6
105	13.8	31.7	18.0	46.1	20.8	142.2	24.5	64.8	28.9	73.9	30.1	81.3
120	12.2	20.0	15.9	29.0	18.4	135.0	21.6	40.0	25.5	46.0	26.6	50.0

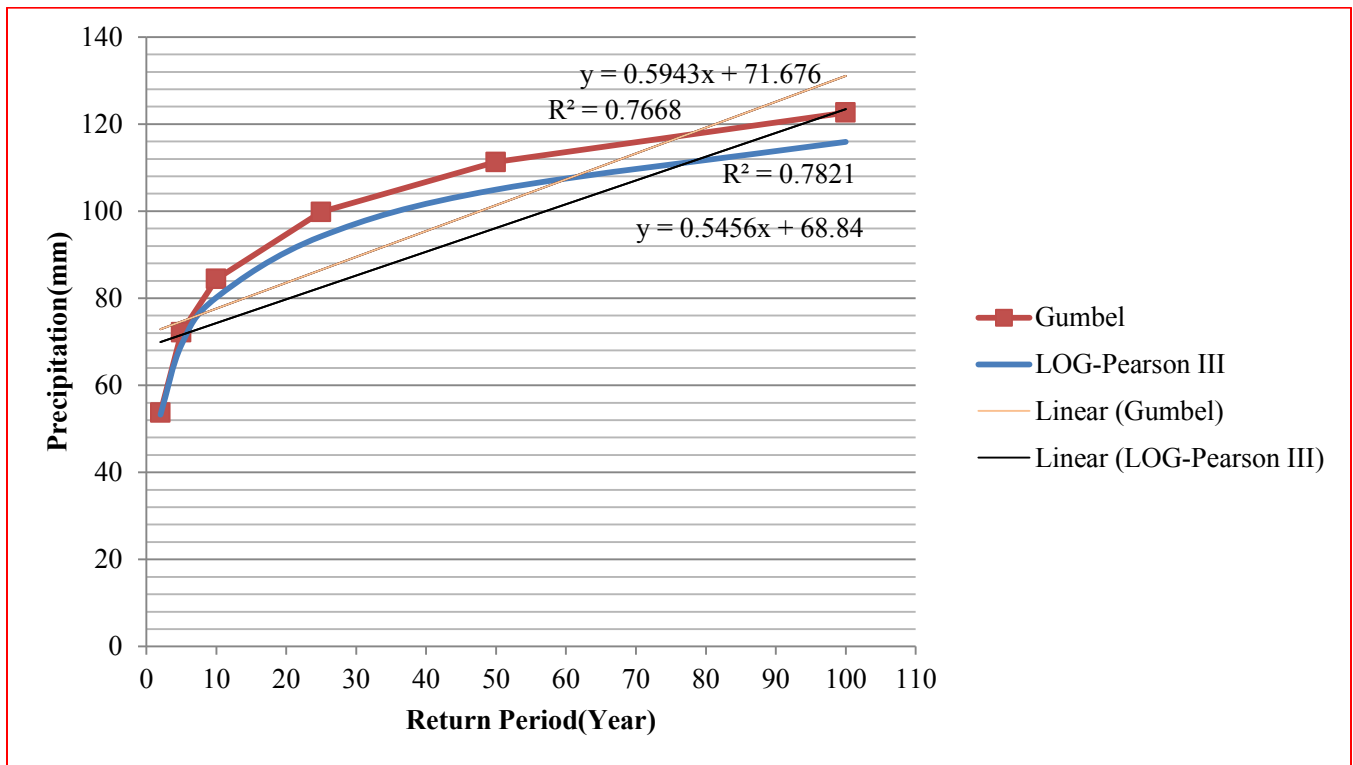


Figure 4.10 Plots of Frequency Analysis Results

From frequency analysis log-Pearson type III have better R^2 values and also used for this study for the analysis of design rain fall for the required return period or 10 years for design and 25 year for check. Because of the flood hazards happened per 10 years in once we need to check the max hazard flood for longest return period which means that for 25 year based on ERA drainage design manual.

4.3. Catchment Delineation and Stream Network

4.3.1 Delineation of Watershed Area

Terrain processing have been resulted in flow direction and flow accumulation grids. And stream networks are obtained based area to define stream parameter that specifies how fine the network should be. Those flow streams cross one to another catchment which means that from catchment nine (Hospital) to catchment one (Dragados) route at certain locations based on topographic map of the study area. The point is taken and catchment is delineated for it. Catchment delineation can be taken in to account based on it's the class of study area (urban) and elevation data used for modeling .All runoff generated from the catchment will not flow towards the outlet because of the above consideration. So the proposed and existing drainage facility resulted based on the possible out flow direction. The result is shown in figure 4.11 and fig 4.12.

Retaining wall used to avoid outside contribution of flood for the study area (L = 400m)

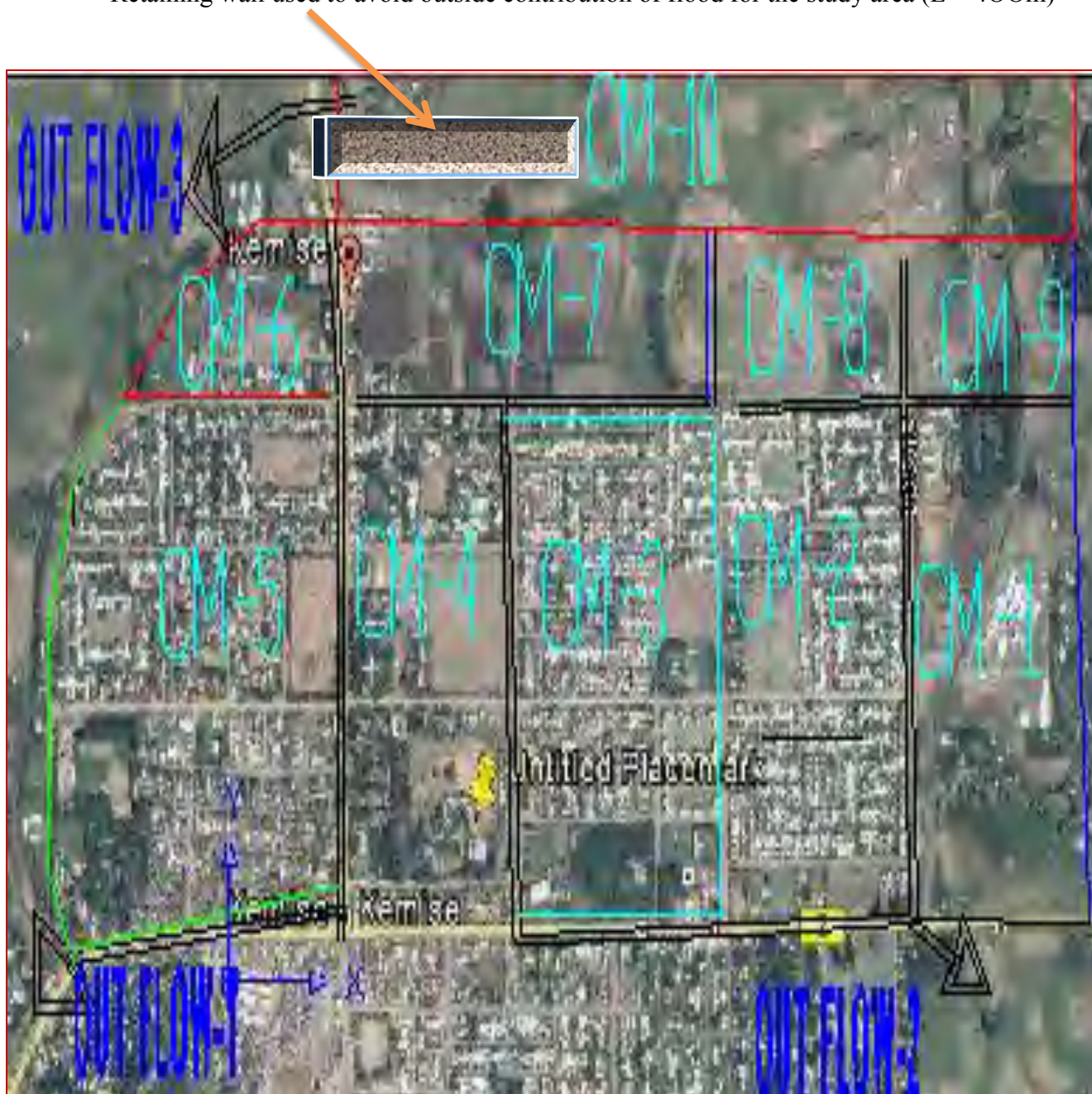


Figure 4.11.Land use composition of the study area and catchment out flow

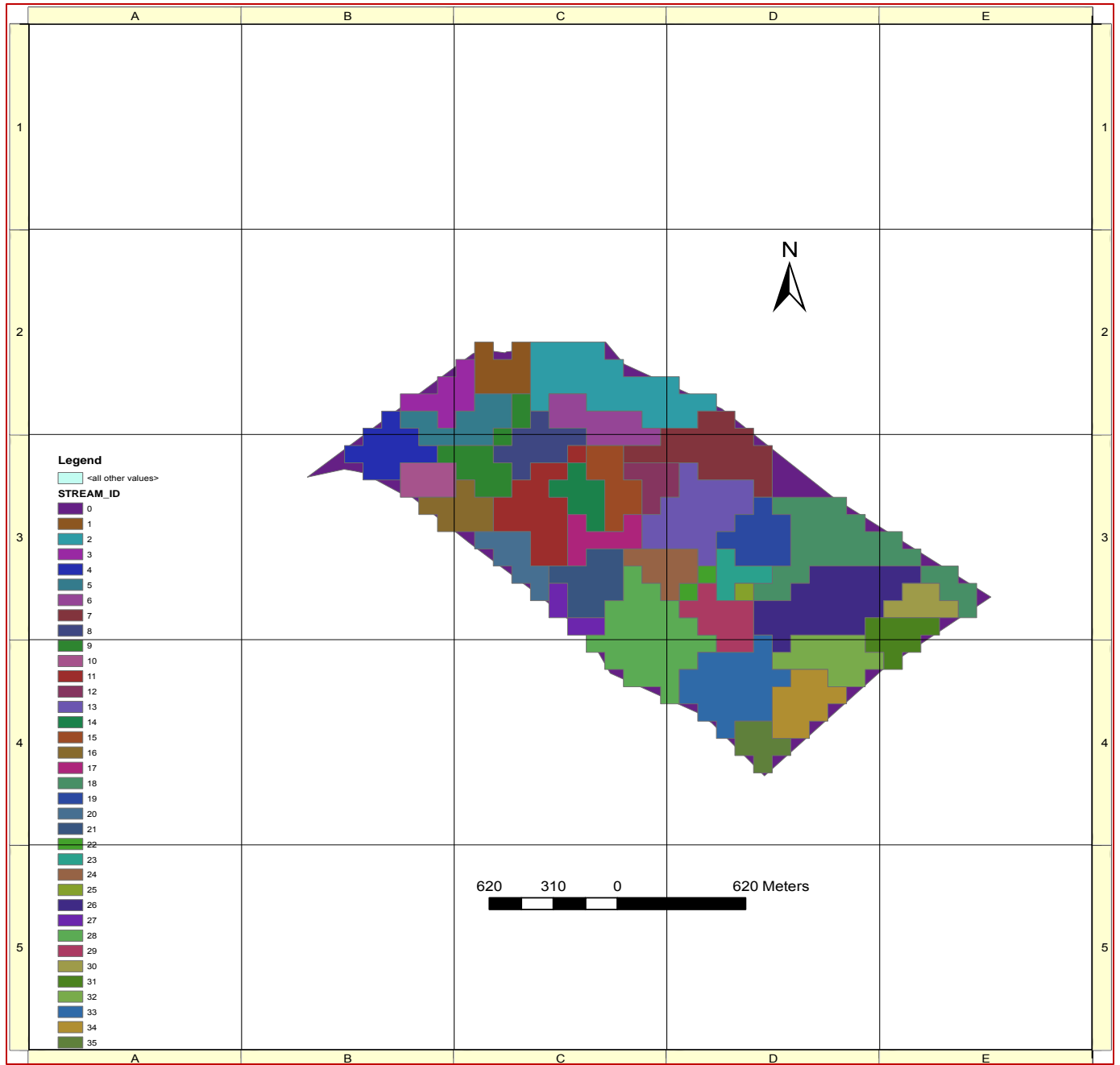


Figure 4.12 water shed area and stream network map

Before calculating the design flood for each catchment provide the retaining structure at catchment 10 and longest upstream flood contribution for the remaining 9 catchments are avoided. But the retaining structure provided based on the design flood at catchment 10, out flow condition and its topography.

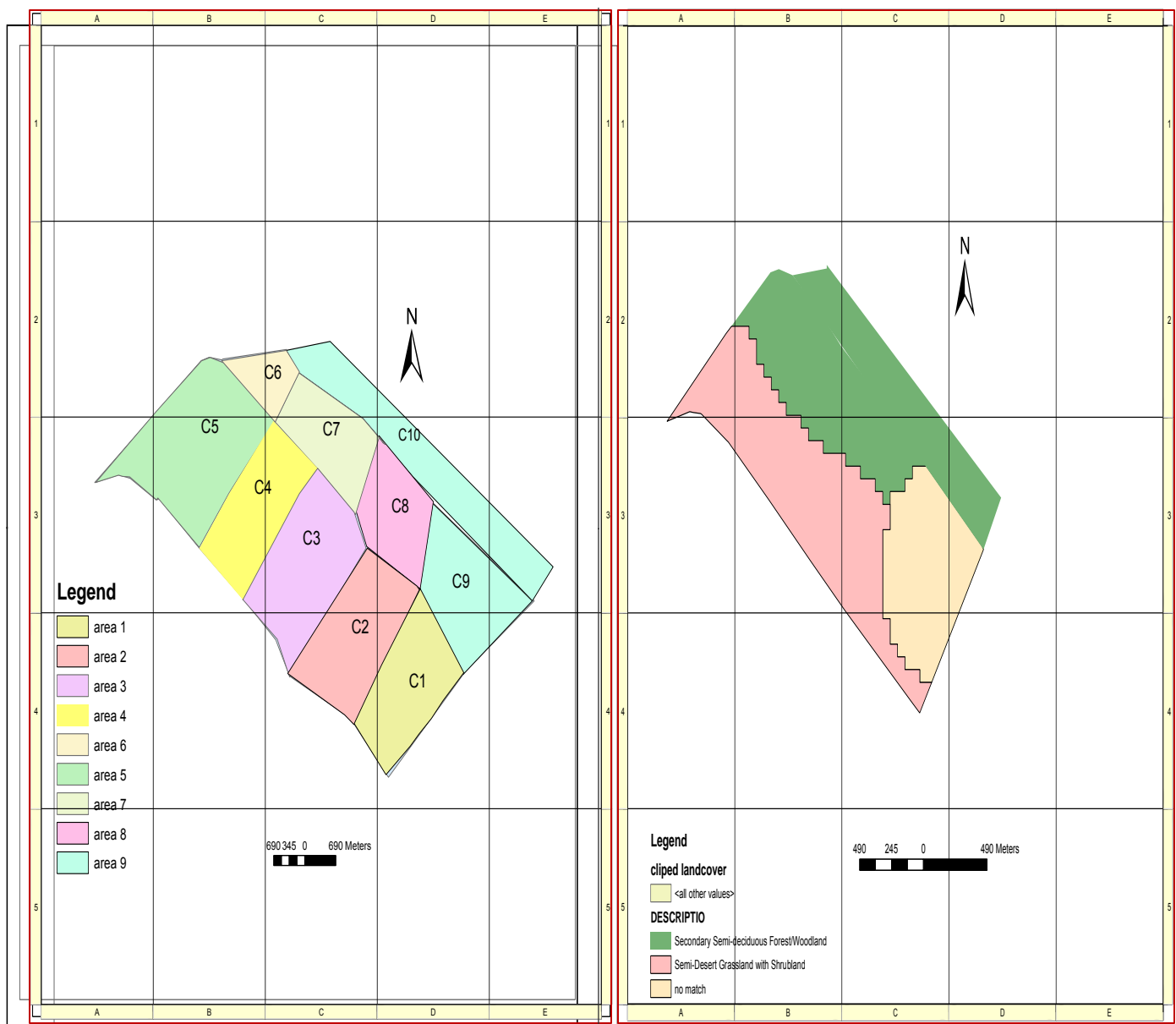


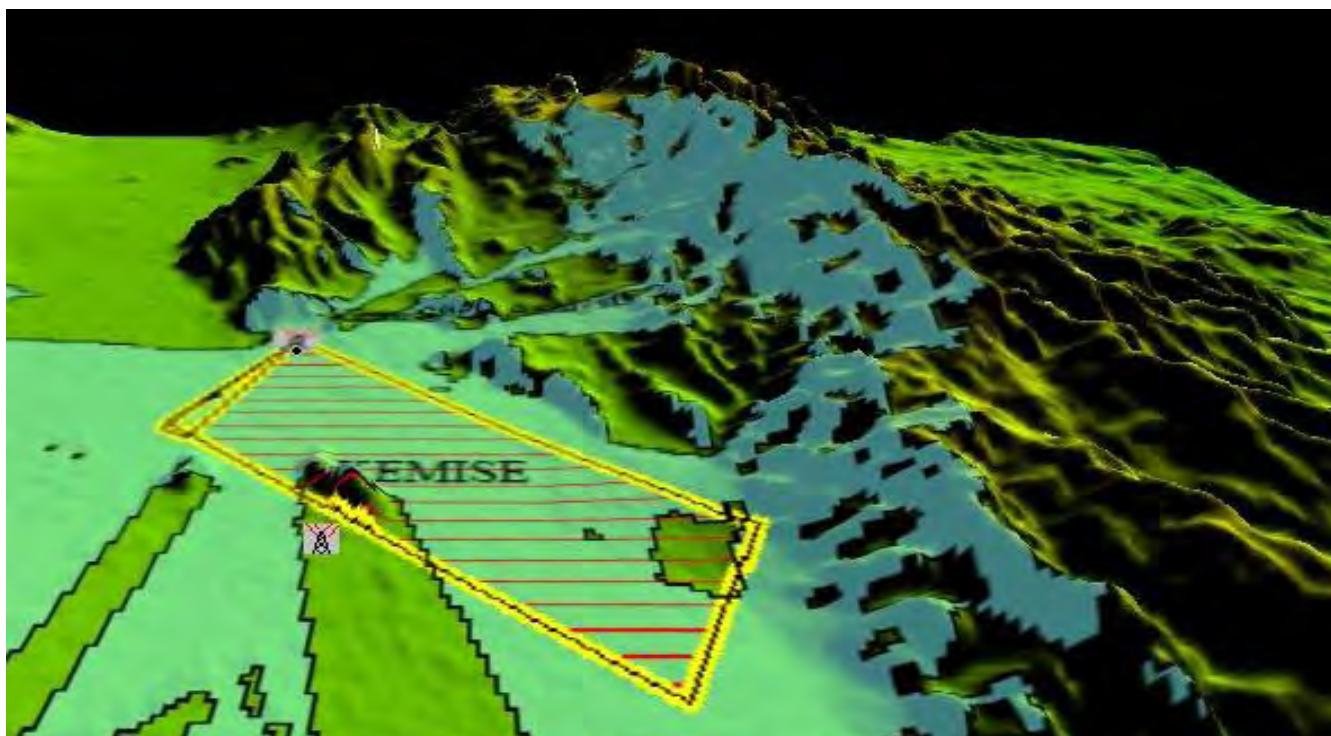
Figure 4.13 Study area catchment divides and Soil map of Study area

Table 4.2a Land use composition of the study area (urban)

Land Use	Area(m ²)	Percentage	Runoff Coefficient(C)
Mixed Residence	2,932,330.7	82.5	0.68
School	155,935.5	4.4	0.65
Health center	147,448.7	4.1	0.60
High Density Mixed	119,602.4	3.4	0.75
Government Compound	112,533.8	3.2	0.50
Cobble Stone road	77,088.1	2.2	0.75
Gravel filled road	11479	0.3	0.70
Total	3,556,418.2	100.0	
Representative C=			0.67

Table 4.2b Land use composition of the study area (rural)

Land Use	Area(m ²)	Percentage	Runoff Coefficient(C)
Undeveloped	494000	100	0.3
Representative C=			0.3

**Figure 4.14 General View shed layout of kemise town**

4.3.4 Hydrologic Analysis

The maximum peak flood is computed taking into consideration the road standard and codes specially ERA drainage design manual. By used frequency analysis equation as described in methodology part of this study (Log-Pearson Type III), for design and review or check which means that for the recommended design from standards and the maximum flood hazards that can happen from the longest recurrence intervals.

Table 4.3 Design and Checking of 24-hour Rainfall Depths

Design Rainfall Depth(mm) (T-10 Year)	Checking Rainfall Depth(mm) (T-25 Year)
80.16	94.22

Table 4.4 Delineation of catchment areas for existing and proposed drainage structures

S.N	Existing Drainage Structures	Area (hectare)	Proposed Drainage Structures	
	Station		Station	Area (hectare)
1	C1+880.8	40.8	C3+885	48.23
2	C2+883	39.9	C4+885	37.51
3	C5+400	50	C6+502	29.35
4	C9+283	40.0	C7+483	37.7
5			C8+483	29.9
			C10+400	49.4

4.4 Estimation of Catchment Parameters

4.4.1 Catchment Parameters at Station C2+883 Drainage Structure

After the watershed areas delineation, watershed properties like the land use coverage, soil type and other parameters that are used for thesis estimated.

i. Slope of the watershed

The average slope of the overland flow is approximated 4% (0.04) by referring from topography and field reconnaissance. And hence, the slope that obtained from field reconnaissance or topographic data.

ii. Time of concentration**a. sheet flow**

The sheet flow occurs up to 100 meters, but from topographic map obtained 74.5m Sheet flow, natural range and Short grass slope of 0.04 m/m, and length of 74.5m and from Appendix 1, Table 3.

For range (natural) Manning's roughness coefficient is 0.14. The 2-year, 24-hour rainfall depth is determined from APPENDIX 1, Table 1.6 to be 53.3mm. Hence, from Equation (3.51a), travel time for sheet flow is determined as:

$$\begin{aligned} T_t &= [0.091 (nL)^{0.8} / (P_2)^{0.5} S^{0.4}] \\ &= [0.091 (0.14 \cdot 74.5)^{0.8} / (53.32)^{0.5} \cdot 0.04^{0.4}] \\ &= \mathbf{0.14hr} \end{aligned}$$

b. Shallow Concentrated Flow

For shallow concentrated flow, unpaved watershed slope is approximated 0.04m/m and length from topography map is 880.8m. Using equation (3.51b), $V = 4.9178(S)^{0.5}$ for unpaved watershed. $V = 4.9178(0.04)^{0.5} = 0.95$ m/sec and from equation (3.51c), paved watershed $V = 6.1961 (s)^{0.5}$. Finally from equation (3.51), travel-time is determined as:

$$\begin{aligned} T_t &= L / (3600V) \\ &= 880.8 / (3600 \cdot 0.95) \\ &= \mathbf{0.26 hr} \end{aligned}$$

c. Channel Flow

For channel flow, natural stream channel, winding with weeds and pools, slope is 0.02m/m, and length is 880.8m. From hydrology of open channel flow (ERA-2002) for Ditch in clay and sandy loam; bottom, and cross section; grass on slopes. Manning's roughness coefficient for channels = 0.040. APPENDIX 1 in Table 1.9. By direct measuring the average bottom width of the stream channel is 1.5m, side slopes are 1V:1H, 10-year storm depth is observed from flood mark and measured to be 1.45m.

$$\begin{aligned} \text{Velocity through the channel (V)} &= (R^{2/3} s^{1/2}) / n \\ &= (0.49^{2/3} \cdot 0.02^{1/2}) / 0.04 \\ &= \mathbf{2.21m/s} \end{aligned}$$

Time of concentration at segment

$$\begin{aligned} T_t &= L / (3600V) \\ &= 880.8 / (3600 * 2.21) \\ &= \mathbf{0.11 \text{ hr}} \end{aligned}$$

$$\begin{aligned} \text{Total Time of Concentration (Tc)} &= T_{t1} + T_{t2} + T_{t3} \quad (\text{from 3.52}) \\ &= (0.14 + 0.26 + 0.11) \text{ hr} \\ &= \mathbf{0.51 \text{ hr}} \end{aligned}$$

By the same steps, catchment parameters at stations C2+883, C5+400 and C9+283 are determined and presented in APPENDIX 1 Table 1.10

4.4.2 Results of rainfall intensity

Rainfall intensity is found by used time of concentration for the design frequency .Hence, 10 year for design and 25 year for review or check. The results are presented in Table 4.6 for every catchment.

4.4.3 Peak Discharge Computation

To estimate the amount of runoff generated from the catchment area shown in Figure 4.11 above rational methods is used. As mentioned in the methodology section an alternative procedure is used to obtain the optimum contributing area that will result in small time of concentration so that the rainfall intensity is at maximum.

4.4.4 Peak Discharge at Station C1+880.8 Drainage Structure

Peak discharge is calculated using equation (3.5). The peak discharges on other stations are computed with the same procedure for design and checks, for the other catchment and presented in Table 4.6.

$$\begin{aligned} T=10 \text{ Year} \quad Q &= 0.00278 * C * I * A * 1.0 \\ T=25 \text{ Year} \quad Q &= 0.00278 * C * I * A * 1.1 \end{aligned}$$

From Table 4.2, Table 4.4 and Table 4.6 respectively, then peak discharge;

$$\begin{aligned} Q &= 0.00278 * 0.67 * 68.3 * 0 * 40.8 * 1 (T = 10 \text{ Year}) \\ &= \mathbf{5.18 \text{ m}^3/\text{s}} \\ Q &= 0.00278 * 0.67 * 80.8 * 40.8 * 1.1 (T = 25 \text{ Year}) \\ &= \mathbf{6.75 \text{ m}^3/\text{s}} \end{aligned}$$

For the other catchments by used the same procedure the results are presented on Table 4.6

Table 4.5 Catchment Parameters for Design and Check

Parameters of Design and Review	Design	Review(Check)
Return Periods(years)	10	25
Time of Concentration (hours)	0.51	0.51
Runoff coefficient(C)	0.67	0.67
Intensity(mm/hr)	68.3	80.8
Area(ha)	40.82	40.82
Peak discharge(m ³ /s)	5.18	6.75

Based on the analysis the result of summation of discharge from all contributors, diameter and its velocity are presented on APPENDEX 2 Table 2.6.

Table 4.6 Intensity and Peak runoff results

Catchment	Intensity From IDF	Intensity From IDF	Peak Runoff	Peak Runoff
	T-10 (mm/hr)	T-25 (mm/hr)	T-10(m ³ /s)	T-25(m ³ /s)
1	68.3	80.8	5.18	6.75
2	48.3	57.1	4.23	4.65
3	79.4	93.9	7.12	9.26
4	77.6	91.7	5.41	7.04
5	63.7	75.3	6.70	8.72
6	135.9	160.8	7.42	9.65
7	133.7	158.1	9.36	12.18
8	107.9	127.6	6.00	7.80
9	98.0	115.9	7.30	9.50
10	49.3	55.1	2.03	2.50

4.4 Hydraulic Analysis

4.4.1 Adequacy of Existing Drainage Structures and its proposed

Hydraulic calculations are carried out for drainage structures at stations C1+880.8, C2+883, C5+400 and C9+283, using the peak discharges that are tabulated in Tables 4.7.

i. Hydraulic Calculation for Drainage Structure at Station C1+880.8

The existing hydraulic calculation are performed by measured the existing structure accordingly. The following are measured data based on the geometry of the channel, its roughness and condition of occurrence. The other dates are measured by the same procedure and presented in Table 4.6.

$Z = 1.45\text{m}$, $b = 1.5\text{m}$, slope = 0.04 side slop: 1v:1h, and manning's roughness coefficient $n = 0.013$

Table 4.7 Existing Drainage hydraulic parameter result

Description	C1	C2	C5	C9
	side slop :1v:1h	side slop :1v:1h	side slop :1v:1h	side slop :1v:1h
Type of drainages	Rectangular	Rectangular	Rectangular	Rectangular
Dimension(m)	B=1.5,y=1.45	B=0.6,y=0.9	B=0.6,y=0.9	B=1.5,y=1.45
A(m²)	2.175	0.54	0.54	2.175
P(m)	4.4	2.1	2.1	4.4
R(m)	0.49	0.26	0.26	0.49
n	0.04	0.013	0.013	0.04
Velocity(m/s)	2.21	1.04	1.67	2.21
Discharge(m³/s)	4.81	0.56	0.90	4.81

Based on the hydraulic calculation of the result drainage capacity of existing system were checked and presented in Table 4.7 to compare with proposed discharge this process also done first by determining the peak discharge for each existing catchment by used empirical equations (Rational method) as described in the methodology part of this study and then subtracted existing discharge this step is important know over flow peak (excess discharge) for each catchments and the result also presented in the following table(Table 4.8).

Table 4.8 Existing, Proposed and Excess Discharge result

S.No	Existing Discharge capacity	Proposed Discharge capacity		Excess Discharge	
	Q(m3/s) = V*A	For design Q(m3/s)	For Check Q(m3/s)	For design Q(m3/s)	For Check Q(m3/s)
c1	4.81	5.18	6.75	0.38	1.94
c2	0.56	4.23	4.65	3.67	4.09
c5	0.90	6.20	8.07	5.80	7.82
c9	4.81	7.30	9.50	2.49	4.69

ii. Drainage Size determination for existing and proposed

By used manning equation determined the size of the sewer pipe based on the parameter

a. Size determination for existing

For Design

$$Q = \frac{AR^{2/3}S^{1/2}}{n} \text{ (Manning equation)}$$

$$A_{\text{circle}} = \frac{\pi D^2}{4} \quad P = \pi D \quad \text{and} \quad R_{\text{circle}} = \frac{D}{4} \quad (\text{i.e. } \pi = 3.14)$$

$$Q = \frac{\pi D^2 * 1}{4 * n} * \frac{D^{2/3}}{4} * S^{0.5}$$

$$0.38 = \frac{\pi D^2 * 1}{4 * 0.013} * \frac{D^{2/3}}{4} * 0.04^{0.5}$$

$$0.38 = 10.079 \frac{1}{n} * D^{8/3} * S^{0.5}$$

$D = 0.106\text{m}$ or 106mm (additional sewer pipe are required)

For check

$$Q = \frac{AR^{2/3}S^{1/2}}{n} \text{ (Manning equation)}$$

$$A_{\text{circle}} = \frac{\pi D^2}{4} \quad P = \pi D \quad \text{and} \quad R_{\text{circle}} = \frac{D}{4} \quad (\text{i.e. } \pi = 3.14)$$

$$Q = \frac{\pi D^2 * 1}{4 * n} * \frac{D^{2/3}}{4} * S^{0.5}$$

$$1.94 = \frac{\pi D^2 * 1}{4 * 0.013} * \frac{D^{2/3}}{4} * 0.01^{0.5}$$

$$1.94 = 10.079 \frac{1}{n} * D^{8/3} * S^{0.5}$$

$$D = 0.20\text{m or } \approx 200\text{mm}$$

The other drainage size for each catchment is estimated by the same procedure and presented in APPENDIX1, Table 1.14 .But storm sewer analysis result based total sum flow from each contributory and its alignment with respect to length; slope and velocity are presented on APPENDIX 2 Table 2.6 by related with to market availability.

b. Drainage Size determination for proposed

By used manning equation determined the size of the sewer pipe based on the parameter

For Design

$$Q = \frac{AR^{2/3}S^{1/2}}{n} \text{ (Manning equation)}$$

$$A_{\text{circle}} = \frac{\pi D^2}{4} \quad P = \pi D \text{ and } R_{\text{circle}} = \frac{D}{4} \quad (\text{i.e. } \pi = 3.14)$$

$$Q = \frac{\pi D^2 * 1}{4 * n} * \frac{D^{2/3}}{4} * S^{0.5}$$

$$7.12 = \frac{\pi D^2 * 1}{4 * 0.013} * \frac{D^{2/3}}{4} * 0.04^{0.5}$$

$$7.12 = 10.079 \frac{1}{n} * D^{8/3} * 0.04^{0.5}$$

$$D = 0.319\text{m or } \approx 320\text{mm} \quad (\text{additional sewer pipe are required})$$

For check

$$Q = \frac{AR^{2/3}S^{1/2}}{n} \text{ (Manning equation)}$$

$$A_{\text{circle}} = \frac{\pi D^2}{4} \quad P = \pi D \text{ and } R_{\text{circle}} = \frac{D}{4} \quad (\text{i.e. } \pi = 3.14)$$

$$Q = \frac{\pi D^2 * 1}{4 * n} * \frac{D^{2/3}}{4} * S^{0.5}$$

$$9.26 = \frac{\pi D^2 * 1}{4 * 0.013} * \frac{D^{2/3}}{4} * 0.04^{0.5}$$

$$9.26 = 10.079 \frac{1}{n} * D^{8/3} * 0.04^{0.5}$$

$$D = 0.42\text{m or } \approx 420\text{mm} \quad (\text{additional sewer pipe are required})$$

The other drainage size for each catchment is estimated by the same procedure and presented in APPENDIX1, Table 1.14 .But storm sewer analysis result based total sum flow from each contributory and its alignment with respect to length; slope and velocity are presented on APPENDIX 2, Table 2.6 by related with to market availability.

From the result of storm sewer design calculation a number of diameter of pipe are found, among different size of pipe the minimum and maximum section of pipe are drawn to define hydraulic type of pipe that are aligned for the study area in order to convey the peak runoff safely are presented in Figure 4.15 and the type of pipe is Reinforced Concrete pipe because based on the appropriate ness of the pipe for the study area and its adoptability in the hole of country in Ethiopia And also the sectional view of circular manholes that are provided for this study based on the diameter of storm sewer and standards are presented on Figure 4.16 and Figure 4.17.

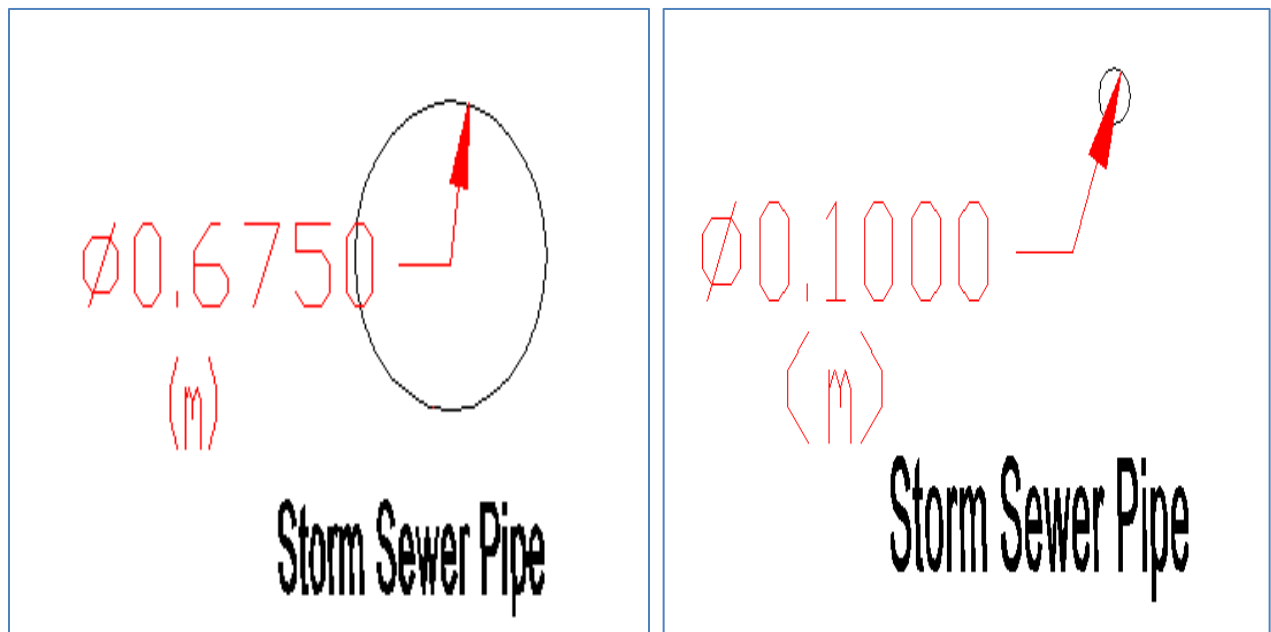


Figure 4.15 Calculated maximum and minimum storm sewer diameter

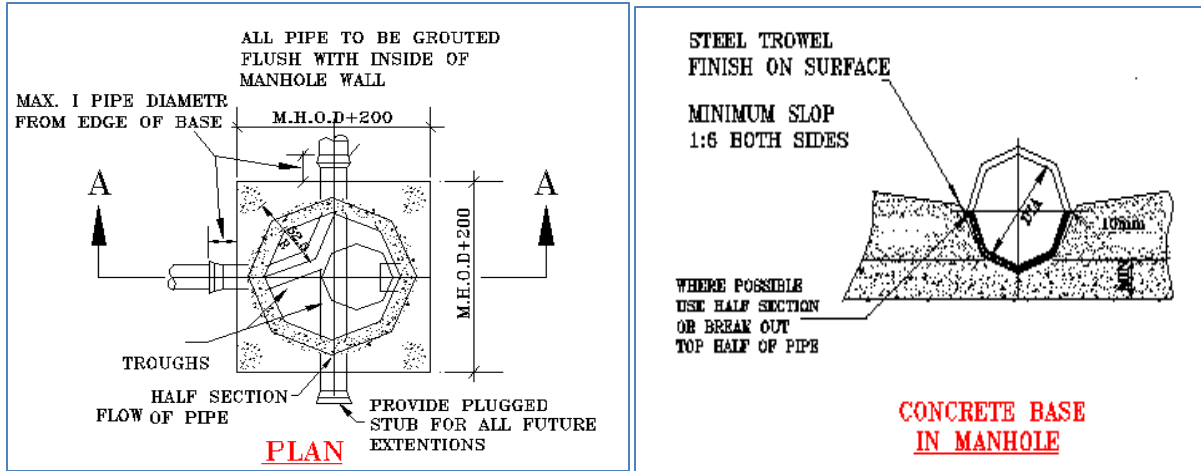


Figure 4.16 Proposed Plan and Concrete bases in circular manhole

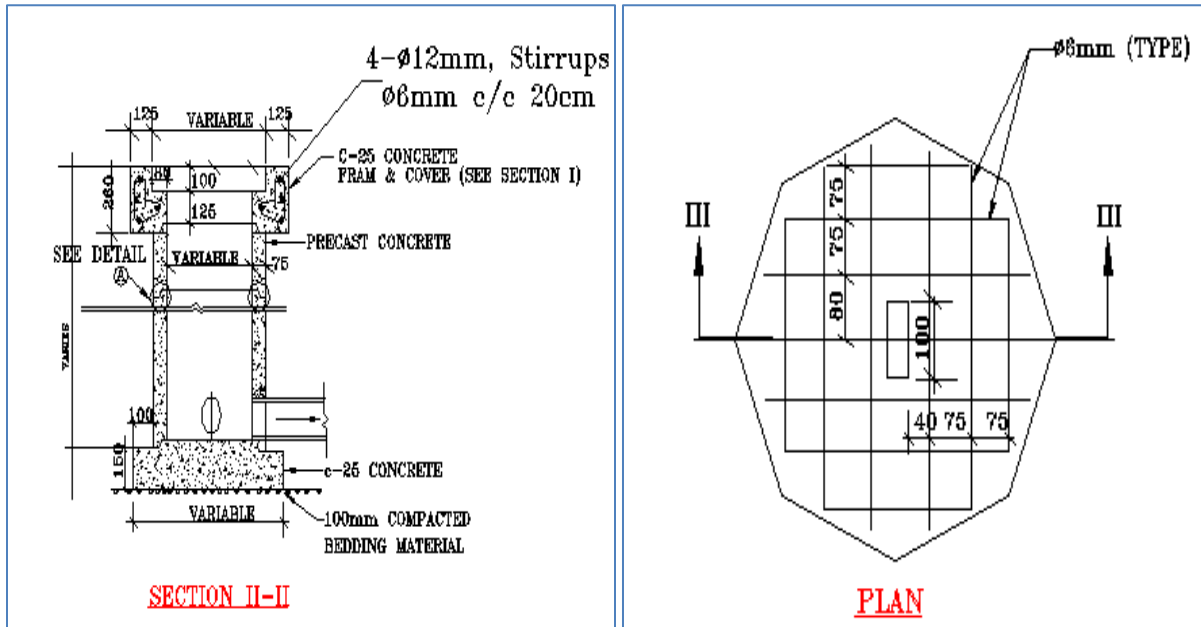


Figure 4.17 Proposed Sectional view of circular manhole

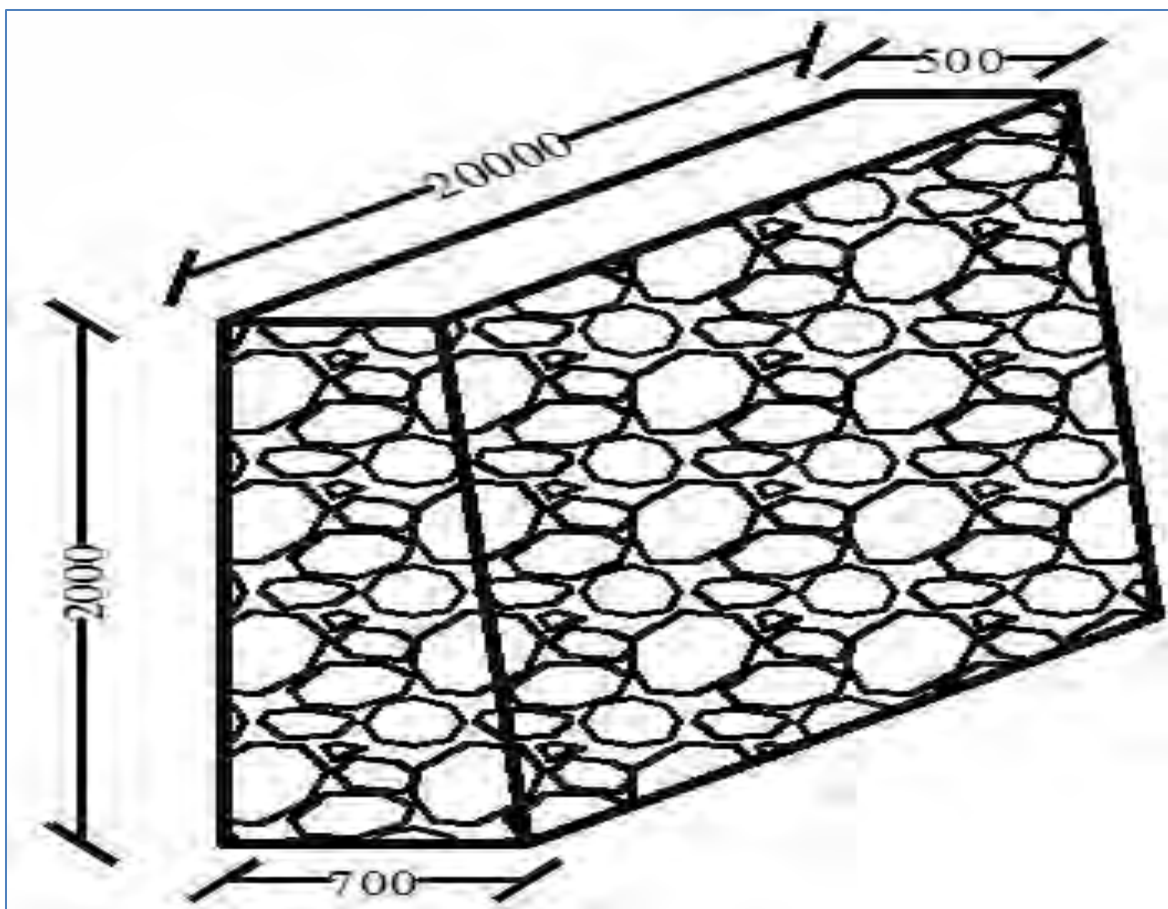


Figure 4.18 Masonry Retaining wall

Note: the dimension on masonry wall is in (mm)

The masonry wall provided for the study is based on the design discharge of contributed catchment, which means from catchment 10, and based on the topographic condition of the mountain the runoff flows to wards to out flow three or Worke River. The total length of the masonry wall is 400m or twice of the above drawing (Figure 4.18) and it connects with joints, because of safety purpose during maintenance of the wall and for the unforeseen condition. The provided masonry wall is starting from the top of catchment six (360m) up to (40m) of catchment seven.

Retaining wall used to avoid outside contribution of flood for the study area (L = 400m)

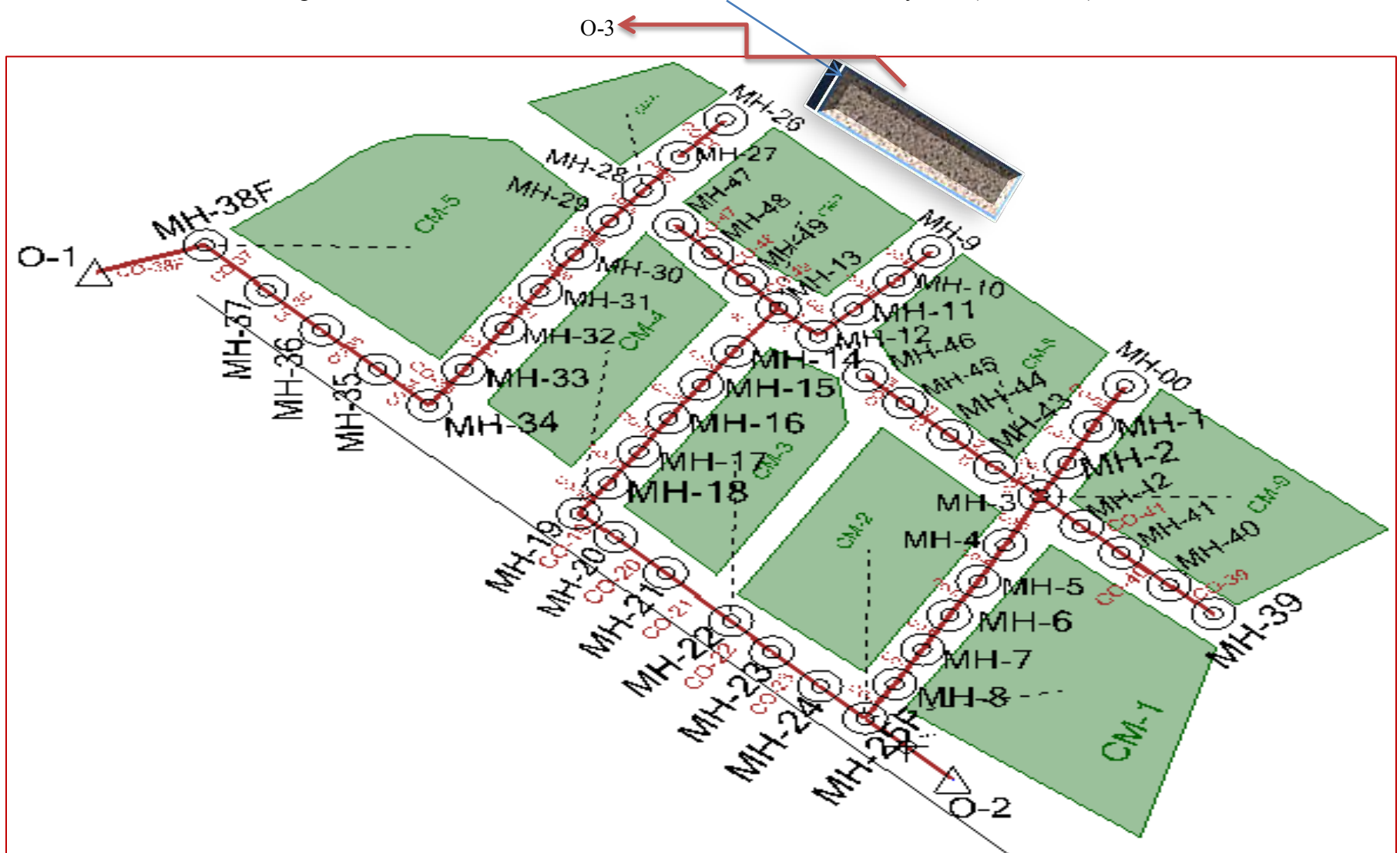


Figure 4.19 Final Storm Sewer alignment based on length, ground elevation and size of pipe

5. Conclusion and Recommendation

5.1 Conclusion

According to the result of this thesis the storm drainage facility are inadequate to convey the peak discharge for required design period and the drainage system filled by sediment and other rubbish materials, based on the result those problems are; due to the drainage design and construction practice adopted by ignoring of hydrology and hydraulic analysis, type of drainage system provided and managing problems form concerned body and un awareness of the community. Based on the result of this study open drainage ditch type is practiced, because of the drainage system are opened it is simply filled by solid materials and storm water for improper aligned drainage system this is caused for different negative impacts for the community during rainy season like malaria and other water born disease. In addition the managing problems form concerned body (municipality) and also the community is disposed their waste into drainage, because of according to study area observation there is lack of awareness on the concerning the impact of disposing solid material on drainage system. The existing drainage alignment is not properly aligned, based on the result this is due to improper design and construction practice. Due to nature of location of Kemise the runoff is contributed from top of mountain area.

5.2 Recommendations

Based on the engineering analysis of this thesis appropriate mitigation measures are recommended. On kemise town inadequacy of storm drainage structures have had serious negative impact on the community at home and road. In order to avoid these problems, the following appropriate mitigation measures are recommended;

- At station C1+880.8 due to inadequacy of drainage structure and improper alignment, over flowing and sedimentation occurred. Therefore, to avoid this problem and based on their connectivity, based on the slope of the catchment and their own contributor, the diameter were i recommend for the study area to use an incremental size of pipe diameter among catchments why due to the increasing of discharge which means that at C1, both C8 and C9 are contribute their flow so the corresponding recommended diameter (D) of pipe is 375mm and Reinforced Concrete Pipe type.
- At MH-25F (in Figure 4.19) the flow is contributed form C1, C2, C3, C4, C7, C-8 and C9 corresponding recommended diameter (D) of pipe is 675mm and reinforced concrete type. For all other recommended values (for existing and proposed) are presented on APPENDIX 2, Table 2.6 and

the alignment are reference for all values of recommended result because numbered and named given based on the alignment.

- At MH-38F (in Figure 4.19) the flow is contributed form both C5 and C6 corresponding recommended diameter (D) of pipe is 450mm and Reinforced Concrete type.
- At C10(outside of town) or mountain there is runoff contribution for the town, so therefore to avoid this problem retaining wall recommended from edge of catchment 6 up to some part of edge of catchment 7and its recommended length ,depth and thickness are 400m and 2m ,50cm at the top and 70cm at the bottom respectively.
- All other recommended values are presented in APPENDEX 2, Table 2.6, based on provided aliment presented on result section in Figure 4.19.
- For drainage filled and alignment problem, periodic cleaning and adjustment of the slope were recommended.
- For municipality, periodic cleaning and workmanship for drainage operation were recommended
- Finally for community creates awareness concerned the effects of disposing solid materials in to drainage facility by the municipality and other concerned body.

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APPENDIX 1 Design rainfall, frequency, IDF curve and its comparison, 24-hr Rainfall depth and related tables
 Table 1.1 Max rainfall data from 1985-2015 and its statistical calculation

S.No.	Year	Max. RF	Descending Order	Rank	Logarithmic Value/Yo/	(Yo-Ym) ²	(Yo-Ym) ³
1	1985	50.4	114.0	1	2.0569049	0.1053417	0.0341901
2	1986	43.3	84.2	2	1.9253121	0.0372378	0.0071858
3	1987	60.5	83.8	3	1.9232440	0.0364440	0.0069573
4	1988	114.0	81.0	4	1.9084850	0.0310267	0.0054652
5	1989	62.2	78.2	5	1.8932068	0.0258778	0.0041628
6	1990	38.0	69.8	6	1.8438554	0.0124355	0.0013867
7	1991	47.0	66.0	7	1.8195439	0.0076044	0.0006631
8	1992	42.0	66	8	1.8195439	0.0076044	0.0006631
9	1993	49.7	62.2	9	1.7936904	0.0037637	0.0002309
10	1994	47.6	60.5	10	1.7817554	0.0024418	0.0001207
11	1995	66.0	60.4	11	1.7810369	0.0023713	0.0001155
12	1996	84.2	59.7	12	1.7759743	0.0019039	0.0000831
13	1997	59.5	59.5	13	1.7745170	0.0017788	0.0000750
14	1998	69.8	58.0	14	1.7634280	0.0009664	0.0000300
15	1999	59.7	53.9	15	1.7315888	0.0000006	0.0000000
16	2000	81.0	51.6	16	1.7126497	0.0003877	-0.0000076
17	2001	78.2	50.9	17	1.7064333	0.0006712	-0.0000174
18	2002	83.8	50.4	18	1.7024305	0.0008946	-0.0000268
19	2003	58.0	50	19	1.6989700	0.0011136	-0.0000372
20	2004	51.6	49.7	20	1.6963564	0.0012949	-0.0000466
21	2005	41.2	47.6	21	1.6773027	0.0030292	-0.0001667
22	2006	53.9	47.0	22	1.6725172	0.0035789	-0.0002141
23	2007	50.9	44.7	23	1.6503075	0.0067295	-0.0005520
24	2008	31.4	43.3	24	1.6364879	0.0091878	-0.0008807
25	2009	50	42.0	25	1.6232493	0.0119010	-0.0012983
26	2010	60.4	41.2	26	1.6145457	0.0138757	-0.0016345
27	2011	40.9	40.9	27	1.6117233	0.0145486	-0.0017548
28	2012	31.7	38.0	28	1.5797836	0.0232738	-0.0035506
29	2013	44.7	33.9	29	1.5301997	0.0408611	-0.0082597
30	2014	66	31.7	30	1.5010593	0.0534912	-0.0123715
31	2015	33.9	31.4	31	1.4964684	0.0556359	-0.0131230
SUM			1751.40		53.7026	0.5172735	0.0173878
MEAN			56.50		1.7323	0.0166862	0.0005609
STANDARD DEVIATION			18.07		0.1313		

SKEWNESS COEFICIENT	1.179	0.2736
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Table 1.2 Design Point Rainfall and return period

Return Period (Year)	Design Point Rainfall	
	Log Person- III	Gumbel
2	53.3	53.8
5	69.3	72.2
10	80.2	84.4
25	94.2	99.8
50	105.0	111.3
100	115.9	122.6

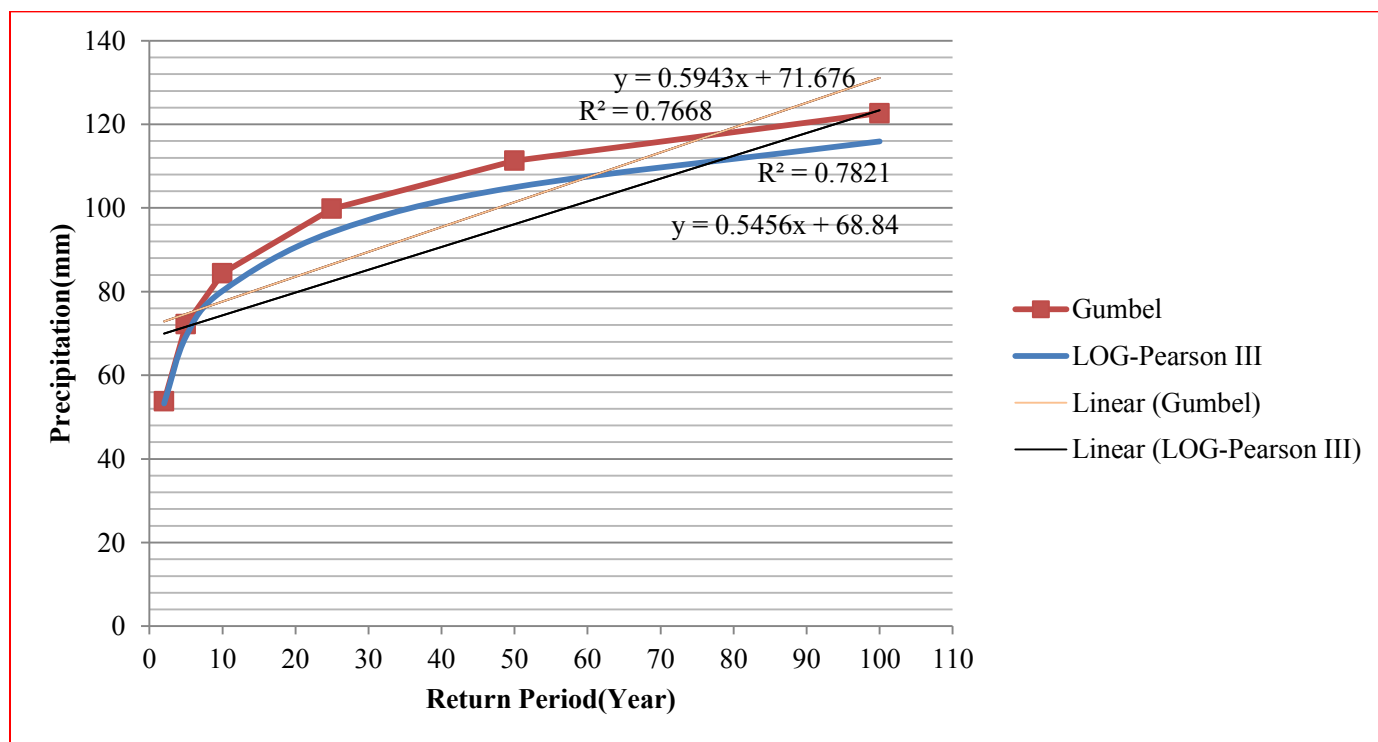


Figure 1.1 Plot of Frequency Analysis Results

Table 1.3 Rainfall of shorter duration using log-Pearson III

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

$$n = 0.92 \quad b = 0.3$$

Duration (min)	T-100 Yr	T-50 Yr	T-25 Yr	T-10Yr	T-5 Yr	T-2 Yr
	R24 =					
5	497.8	451.7	405.3	342.6	293.1	218.2
10	270.2	245.1	219.9	185.9	159.0	118.4
15	187.7	170.3	152.8	129.2	110.5	82.3
20	144.7	131.3	117.8	99.6	85.2	63.4
25	118.2	107.2	96.2	81.3	69.6	51.8
30	100.1	90.8	81.5	68.9	58.9	43.9
35	87.0	78.9	70.8	59.9	51.2	38.1
40	77.0	69.9	62.7	53.0	45.3	33.8
45	69.2	62.7	56.3	47.6	40.7	30.3
50	62.8	57.0	51.1	43.2	37.0	27.5
55	57.6	52.2	46.9	39.6	33.9	25.2
60	53.2	48.2	43.3	36.6	31.3	23.3
65	49.4	44.8	40.2	34.0	29.1	21.7
70	46.2	41.9	37.6	31.8	27.2	20.2
75	43.3	39.3	35.3	29.8	25.5	19.0
80	40.8	37.1	33.2	28.1	24.0	17.9
85	38.6	35.1	31.4	26.6	22.7	16.9
90	36.7	33.3	29.8	25.2	21.6	16.1
95	34.9	31.7	28.4	24.0	20.5	15.3
100	33.3	30.2	27.1	22.9	19.6	14.6
105	31.8	28.9	25.9	21.9	18.7	13.9
110	30.5	27.7	24.8	21.0	18.0	13.4
115	29.3	26.6	23.8	20.2	17.2	12.8
120	28.2	25.5	22.9	19.4	16.6	12.3

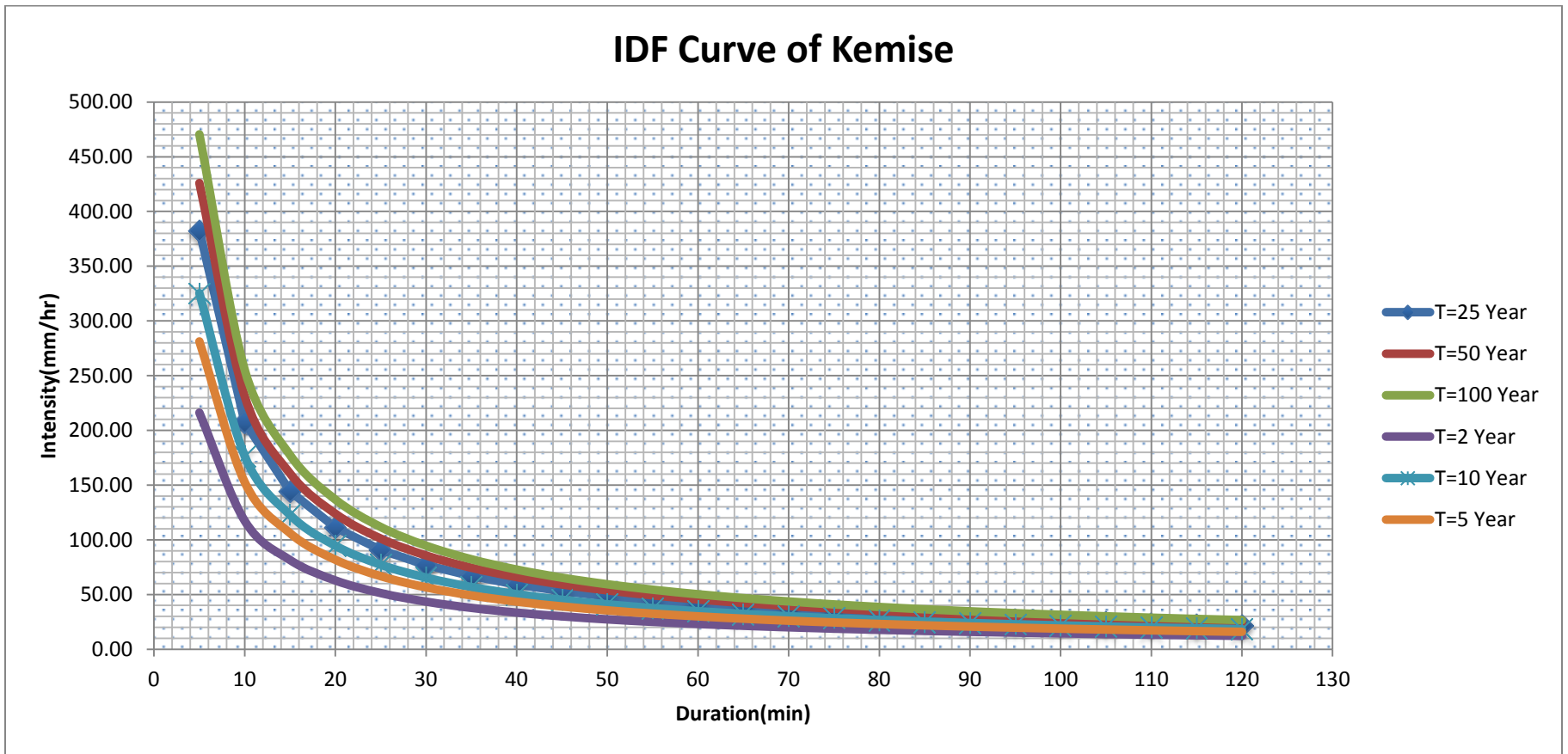


Figure 1.2 Intensity-Duration-Frequency curve of kemise

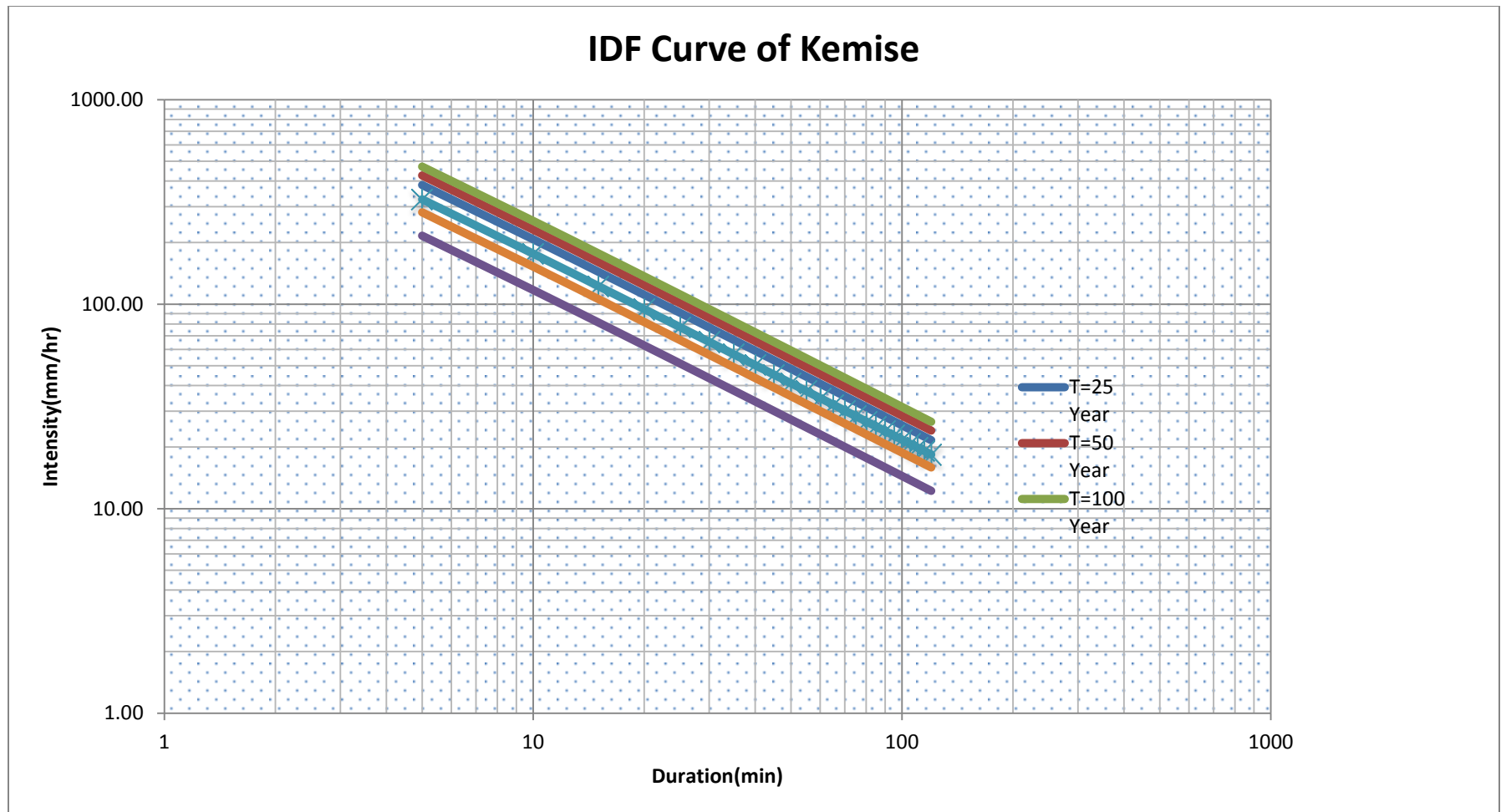


Figure 1.3 Intensity-Duration-Frequency curve of kemise (Log Scale)

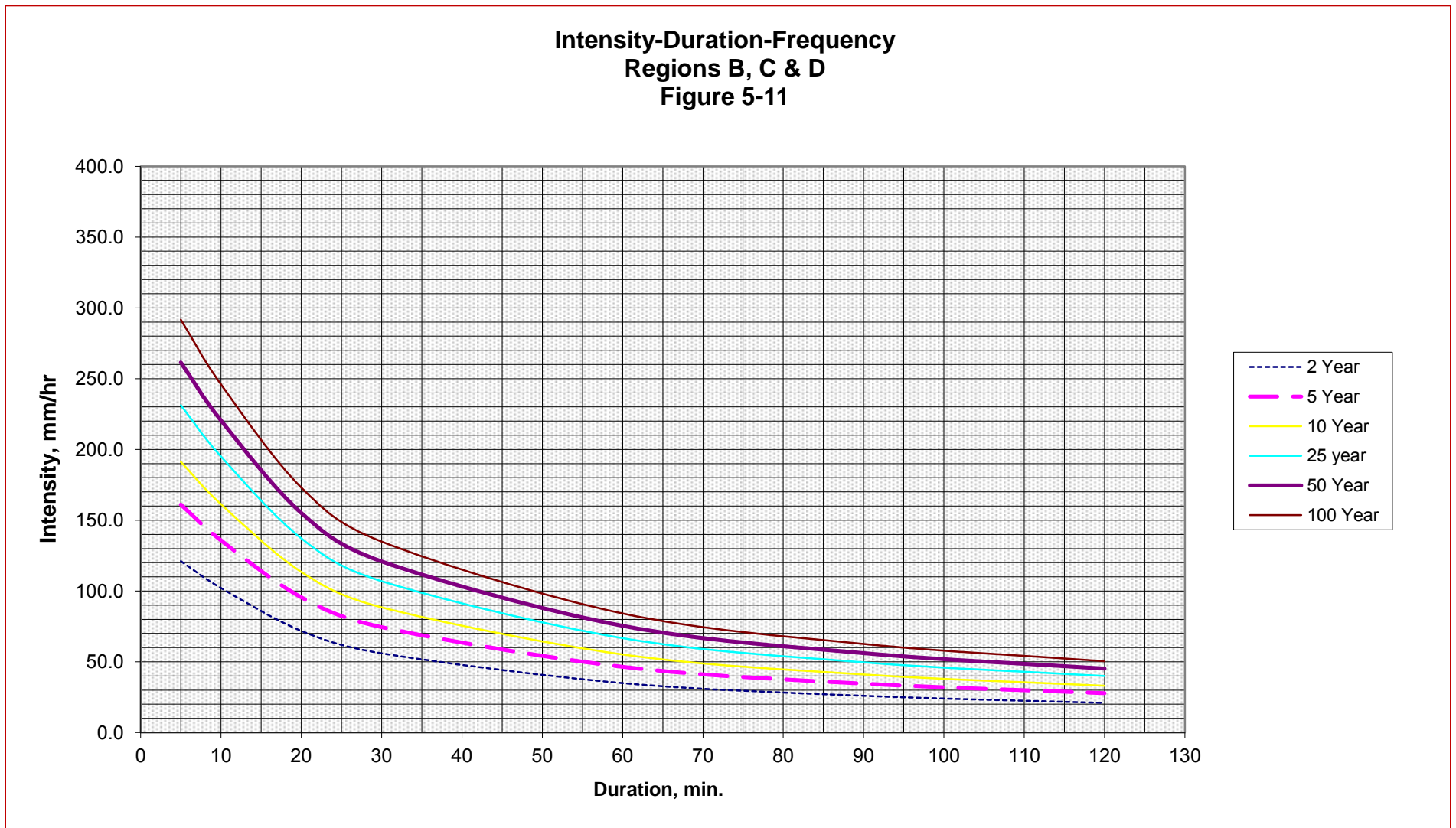


Figure 1.4 Intensity-Duration-Frequency curves for the region developed by ERA

Table 1.4 Comparison of IDF curve results with IDF curve from ERA for station(B,C and D)

	Self	ERA	Self	ERA	Self	ERA	Self	ERA	Self	ERA	Self	ERA
Duration	T-2	T-2	T-5	T-5	T-10	T-10	T-25	T-25	T-50	T-50	T-100	T-100
5	216.2	110.0	281.3	160.0	325.4	190.0	382.5	230.0	451.7	260.0	470.5	290.0
10	117.3	106.1	152.6	154.3	176.6	187.6	207.5	221.7	245.1	250.7	255.3	279.6
15	81.5	102.2	106.1	148.6	122.7	185.2	144.2	213.5	170.3	241.4	177.4	269.1
20	62.8	98.3	81.8	142.9	94.6	182.8	111.2	205.2	131.3	232.1	136.8	258.7
25	51.3	94.3	66.8	137.2	77.2	180.4	90.8	197.0	107.2	222.8	111.7	248.3
30	43.5	90.4	56.6	131.5	65.4	178.0	76.9	188.7	90.8	213.5	94.6	237.8
35	37.8	86.5	49.1	125.8	56.9	175.7	66.8	180.4	78.9	204.2	82.2	227.4
40	33.4	82.6	43.5	120.1	50.3	173.3	59.2	172.2	69.9	194.9	72.8	217.0
45	30.0	78.7	39.1	114.4	45.2	170.9	53.1	163.9	62.7	185.6	65.4	206.5
50	27.3	74.8	35.5	108.7	41.0	168.5	48.2	155.7	57.0	176.3	59.4	196.1
55	25.0	70.9	32.5	103.0	37.6	166.1	44.2	147.4	52.2	167.0	54.4	185.7
60	23.1	67.0	30.0	97.3	34.7	163.7	40.8	139.1	48.2	157.7	50.2	175.2
65	21.4	63.0	27.9	91.7	32.3	161.3	37.9	130.9	44.8	148.3	46.7	164.8
70	20.0	59.1	26.1	86.0	30.2	158.9	35.5	122.6	41.9	139.0	43.6	154.3
75	18.8	55.2	24.5	80.3	28.3	156.5	33.3	114.3	39.3	129.7	40.9	143.9
80	17.7	51.3	23.1	74.6	26.7	154.1	31.4	106.1	37.1	120.4	38.6	133.5
85	16.8	47.4	21.8	68.9	25.3	151.7	29.7	97.8	35.1	111.1	36.5	123.0
90	15.9	43.5	20.7	63.2	24.0	149.3	28.2	89.6	33.3	101.8	34.6	112.6
95	15.1	39.6	19.7	57.5	22.8	147.0	26.8	81.3	31.7	92.5	33.0	102.2
100	14.5	35.7	18.8	51.8	21.8	144.6	25.6	73.0	30.2	83.2	31.5	91.7
105	13.8	31.7	18.0	46.1	20.8	142.2	24.5	64.8	28.9	73.9	30.1	81.3
110	13.2	27.8	17.2	40.4	19.9	139.8	23.4	56.5	27.7	64.6	28.8	70.9
115	12.7	23.9	16.5	34.7	19.1	137.4	22.5	48.3	26.6	55.3	27.7	60.4
120	12.2	20.0	15.9	29.0	18.4	135.0	21.6	40.0	25.5	46.0	26.6	50.0

Surface Description	n ¹
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover ≤ 20%	0.06
Residue cover > 20%	0.17
Grasses:	
Short grass	0.15
Dense Grasses	0.24
Range (natural)	0.13
Woods: ²	
Light underbrush	0.40
Dense underbrush	0.80

¹ The n values are a composite of information compiled by Engman (1986).

² When selecting n, consider cover to a height of about 0.03 m. This is the only part of the plant cover that will obstruct sheet flow.

Table 1.6 24-hours Rainfall Depth (Using Log-Pearson Type III)

Return period T (year)	Design Point Rainfall (mm)
2	53.3
5	69.3
10	80.2
25	94.2
50	105.0
100	115.9

Table 1.7 Hydrological Characteristics of Soil Groups (ERA DDM, 2011)

Soil Group	Soil Group General Description	
A	Well drained, sandy	High infiltration, low runoff
B	B Sandy loam, low plasticity	
C	C Clayey loam, medium plasticity	
D	D High plastic clay	Low infiltration, high runoff

Table 1.8 Estimated Manning's roughness coefficient for sheet flow

Station	n
c1	0.14
c2	0.01
c3	0.01
c4	0.04
c5	0.01
c6	0.14
c7	0.14
c8	0.14
c9	0.14

Table 1.9 Manning's Coefficient for channels and Pipes

Conduit Material	Manning's n*
Closed Conduits	
Asbestos-cement pipe	0.011 - 0.015
Brick	0.013 - 0.017
Cast iron pipe	
Cement-lined and seal coated	0.011 - 0.015
Concrete (monolithic)	0.012 - 0.014
Concrete pipe	0.011 - 0.015
Corrugated-metal pipe - 13 mm by 64 mm (½ inch by 2 ½ inch) corrugations	
Plain	0.022 - 0.026
Paved invert	0.018 - 0.022
Spun asphalt lines	0.011 - 0.015
Plastic pipe (smooth)	0.011 - 0.015
Vitrified clay	
Pipes	0.011 - 0.015
Liner plates	0.013 - 0.017
Open Channels	
Lined channels	
Asphalt	0.013 - 0.017
Brick	0.012 - 0.018
Concrete	0.011 - 0.020
Rubble or riprap	0.020 - 0.035
Vegetal	0.030 - 0.400
Excavated or dredged	
Earth, straight and uniform	0.020 - 0.030
Earth, winding, fairly uniform	0.025 - 0.040
Rock	0.030 - 0.045
Unmaintained	0.050 - 0.140
Natural channels (minor streams, top width at flood stage <30 m (100 ft))	
Fairly regular section	0.030 - 0.070
Irregular section with pools	0.040 - 0.100
*Lower values are usually for well-constructed and maintained (smoother) pipes and channels.	

Table 1.10 Time of concentration result

i.	Sheet Flow	ii. Shallow Concentrated Flow		iii. Open Channels Flow		Total Tc	Total Tc
	(hr)	(m/s)	(hr)	(m/s)	(hr)	(hr)	(min)
	$T_t = [0.091 (nL) 0.8 / (P2)0.5s0.4]$	$V = 4.9178 (s) 0.5$	$T_t = L/(3600V)$	$V = (R 2/3 s^{1/2})/n$	$T_t = L/(3600V)$	ΣT_c	
	0.14	0.95	0.26	2.21	0.11	0.51	30.33
	0.01	0.50	0.49	1.04	0.24	0.74	44.38
	0.01	0.58	0.42	0.00	0.00	0.43	25.78
	0.02	0.59	0.42	0.00	0.00	0.44	26.52
	0.01	0.66	0.39	1.67	0.15	0.55	32.89
	0.07	0.81	0.17	0.00	0.00	0.24	14.41
	0.08	0.81	0.17	0.00	0.00	0.24	14.60
	0.17	0.98	0.14	0.00	0.00	0.31	18.60
	0.16	0.92	0.15	2.21	0.04	0.34	20.43
	0.01	0.50	0.49	1.04	0.24	0.72	43.4

Table 1.11 Peak discharge result for each station

T=10 Year

$$Q = 0.00278 * C * I * A * 1.0$$

T=25 Year

$$Q = 0.00278 * C * I * A * 1.1$$

Tc (min)	Intensity From IDF T-10 (mm/hr)	Intensity From IDF T-25 (mm/hr)	Peak Runoff T-10(m3/s)	Peak Runoff T-25(m3/s)
	30.3	68.3	80.8	5.18
44.4	57.1	57.1	4.23	4.65
25.8	79.4	93.9	7.12	9.26
26.5	77.6	91.7	5.41	7.04
28.3	72.1	85.2	6.70	8.72
14.4	135.9	160.8	7.42	9.65
14.6	133.7	158.1	9.36	12.18
18.6	107.9	127.6	6.00	7.80
20.4	98.0	115.9	7.30	9.50
43.4	49.3	55.1	2.03	2.50

Table 1.12 existing, proposed and excess discharge calculations for the catchments

S.No	Hydraulic Calculation $V = (R^{2/3} s^{1/2})/n$ (m/s)	Existing Discharge	Proposed Discharge		Excess Discharge	
		$Q(m^3/s) = V * A$	(For design) $Q(m^3/s)$	For Check $Q(m^3/s)$	(For design) $Q(m^3/s)$	Design $Q(m^3/s)$
c1	2.21	4.81	5.18	6.75	0.38	1.94
c2	1.04	0.56	4.23	4.65	3.67	4.09
c5	1.67	0.90	6.70	8.72	5.80	7.82
c9	2.21	4.81	7.30	9.50	2.49	4.69

Table 1.13 New proposed peak discharge calculation

S.No	Peak Runoff(m ³ /s)	
	T-10year	T-25year
c3	7.12	9.26
c4	5.41	7.04
c6	7.42	9.65
c7	9.36	12.18
c8	6.00	7.80

Table 1.14 output new diameter and its recommended values for each catchment

Station	Output New Diameter		Market available			
	For design	For Check	Design	check	Market available	
	(m)	(m)	D(mm)	D(mm)	D(mm) Design	D(mm) check
c1	0.11	0.20	106	196	150	225
c2	0.32	0.33	317	330	375	375
c3	0.32	0.42	319	423	375	450
c4	0.37	0.38	366	380	375	450
c5	0.34	0.39	344	385	375	450
c6	0.3	0.2	344	213	375	225
c7	0.2	0.4	209	415	225	450
c8	0.1	0.3	143	327	150	375
c9	0.22	0.11	218	110	225	150

APPENDIX 2 Rainfall Regions and metrological stations, Storm sewer analysis result

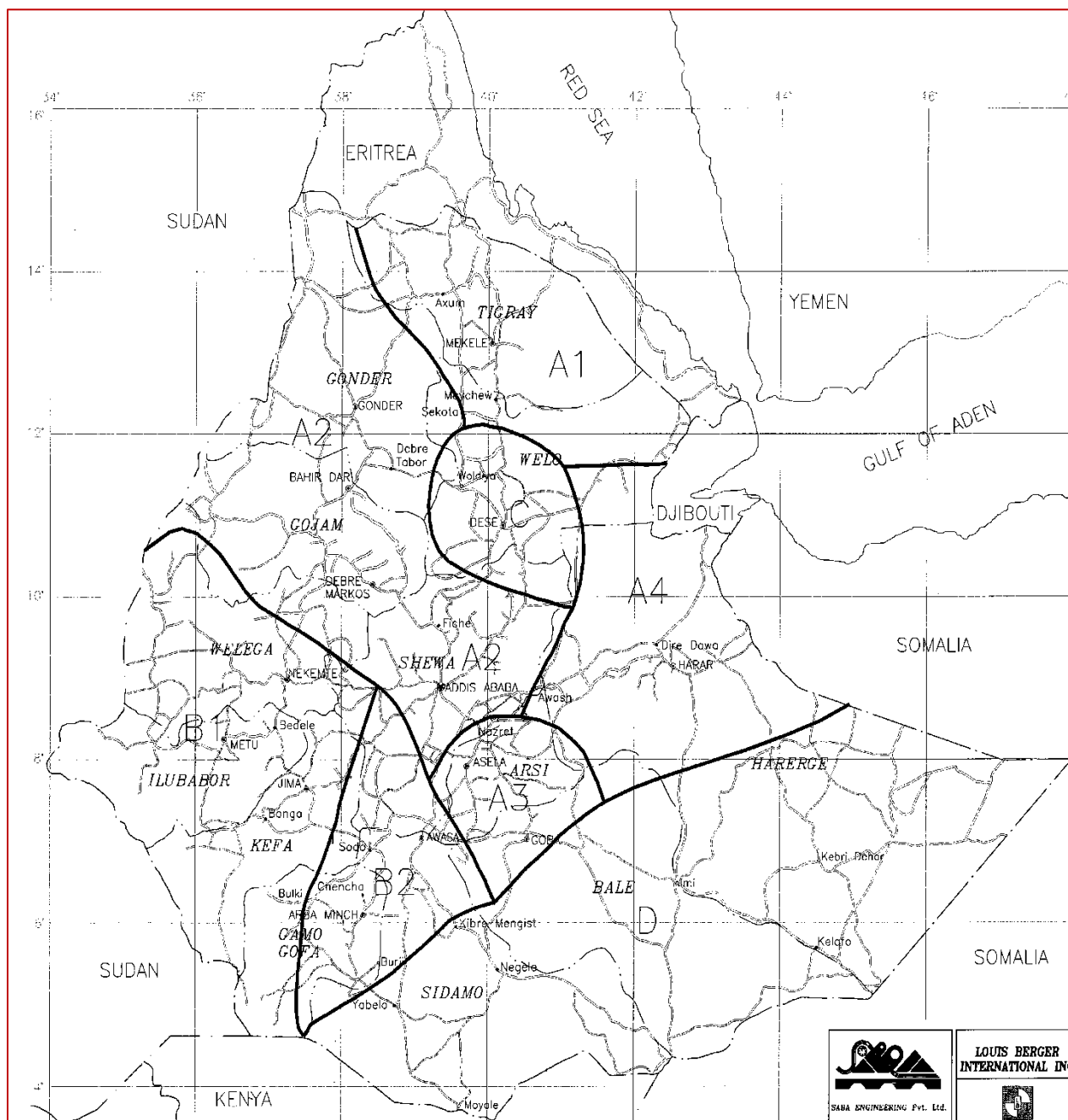


Figure 2.1 Rainfall Regions

Table 2.1 Meteorology Stations in 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
Addis Ababa	33	Woldiya	23		
A3	Nazareth	40	D1	Sirinka	17
	Kulumsa	31		Gode	29*
	Robe/Bale	19		Kebri Dihar	38
A4	Metehara	28	D2	Kibre Mengist	24
	Dire Dawa	46		Negele	45
	Mieso	35		Moyale	18
* max 24 hour rainfall not given				Yabelo	34

Years of record through 1997

Table 2.2 Frequency Factors for Rational Formula

Recurrence Interval (years)	C_f
5	1.0
10	1.0
25	1.1
50	1.2
100	1.25

Table 2.3 Access Hole Sizing

Manhole Dia. (mm)	K (mm/°)	Maximum Pipe Size (mm)
675	6.10	375
1050	9.40	675
1200	10.67	750
1350	11.94	900
1500	13.21	1050
1650	14.73	1200
1800	16.00	1350
1950	17.27	1500
2100	18.54	1650
2250	20.07	1800
2400	21.34	1800
2550	22.61	1950
2700	23.88	2100

Table 2.4 Spacing of Access Holes

<u>Size of Pipe (mm)</u>	<u>Maximum Spacing (m)</u>
300-600	100
675-900	125
1050-1350	150
1500-up	300

Table 2.5 Storm sewer Design analysis result based on out flow condition(contributor)

DISCRIPTION	Qtotal(design)	Qtotal(check)	D(mm) Design	D(mm) check	Market available	
					D(mm) Design	D(mm) check
QC8+QC9	8.5	12.5	337	390	375	450
QC7+QC8+QC1	14.3	21.9	414	486	450	525
QC7+QC8+QC1+QC2	18.0	26.0	495	569	525	600
QC7+QC4	14.77	19.22	424	518	525	525
QC7+QC4+QC3	21.89	28.49	514	568	525	600
QC7+QC4+QC3+QC2	25.56	32.58	537	600	600	600
QC8+QC9+QC1+QC2	12.54	18.52	441	452	450	525
MH-25F(OF-2)	34.43	47.01	610	685	675	750
QC6+QC5(OF-1)	13.22	17.47	450	500	450	525
QC7+QC4+QC3+QC2	25.56	32.58	521	600	525	600

C- Catchment

Q- Discharge

QC1 – Discharge from catchment one

QC9 - Discharge from catchment nine

QC8+QC9 = Discharge from catchment eight + Discharge from catchment nine

Table 2.6 Storm sewer alignment analysis result based on out flow condition and topographic map

Label	Ground Elevation (m)	ΔH	Length between MH	Slope(m/m)	Diameter(m)	Type of pipe	Manning Roughness(n)	Velocity(m/s)
MH-00	1455.75	4.25	100	0.043	0.15	RC	0.013	1.8
MH-1	1451.5	4.25	100	0.043	0.15	RC	0.013	1.8
MH-2	1447.25	4.25	100	0.043	0.15	RC	0.013	1.8
MH-3	1443	4.25	100	0.043	0.225	RC	0.013	2.3
MH-4	1437.42	5.58	100	0.056	0.225	RC	0.013	2.7
MH-5	1436.49	0.93	100	0.009	0.45	RC	0.013	1.7
MH-6	1435.56	0.93	100	0.009	0.475	RC	0.013	1.8
MH-7	1434.63	0.93	100	0.009	0.5	RC	0.013	1.9
MH-8	1433.71	0.93	100	0.009	0.525	RC	0.013	1.9
MH-9	1457.97	2.03	100	0.020	0.1	RC	0.013	0.9
MH-10	1455.94	2.03	100	0.020	0.1	RC	0.013	0.9
MH-11	1453.91	2.03	100	0.020	0.1	RC	0.013	0.9
MH-12	1451.88	2.03	100	0.020	0.1	RC	0.013	0.9

MH-13	1444.77	7.11	100	0.071	0.225	RC	0.013	3.0
MH-14	1444.07	0.70	100	0.007	0.4	RC	0.013	1.4
MH-15	1443.38	0.70	100	0.007	0.425	RC	0.013	1.4
MH-16	1442.68	0.70	100	0.007	0.45	RC	0.013	1.5
MH-17	1441.99	0.70	100	0.007	0.475	RC	0.013	1.6
MH-18	1441.29	0.70	100	0.007	0.5	RC	0.013	1.6
MH-19	1440.59	0.70	100	0.007	0.6	RC	0.013	1.8
MH-20	1432.56	0.44	100	0.004	0.6	RC	0.013	1.4
MH-21	1432.11	0.44	100	0.004	0.6	RC	0.013	1.4
MH-22	1431.67	0.44	100	0.004	0.6	RC	0.013	1.4
MH-23	1430.70	0.97	100	0.010	0.6	RC	0.013	2.1
MH-24	1430.39	0.30	100	0.003	0.6	RC	0.013	1.2
MH-25F	1430.09	0.30	125	0.003	0.675	RC	0.013	0.9
MH-26	1448.86	1.14	100	0.011	0.150	RC	0.013	0.9
MH-27	1447.72	1.14	100	0.011	0.150	RC	0.013	0.9

MH-28	1446.58	0.51	100	0.005	0.375	RC	0.013	1.1
MH-29	1446.07	0.51	100	0.005	0.450	RC	0.013	1.3
MH-30	1444.13	1.93	100	0.019	0.450	RC	0.013	2.5
MH-31	1442.20	1.93	100	0.019	0.450	RC	0.013	2.5
MH-32	1440.27	1.93	100	0.019	0.450	RC	0.013	2.5
MH-33	1438.33	1.93	100	0.019	0.450	RC	0.013	2.5
MH-34	1436.40	1.93	100	0.019	0.450	RC	0.013	2.5
MH-35	1430.67	5.73	100	0.057	0.450	RC	0.013	4.3
MH-36	1430.33	0.33	100	0.003	0.450	RC	0.013	1.0
MH-37	1430.00	0.33	100	0.003	0.450	RC	0.013	1.0
MH-38F	1429.67	0.33	100	0.003	0.450	RC	0.013	1.0
MH-39	1458.69	1.31	100	0.013	0.150	RC	0.013	1.0
MH-40	1457.38	1.31	100	0.013	0.150	RC	0.013	1.0
MH-41	1456.07	1.31	100	0.013	0.225	RC	0.013	1.3
MH-42	1454.76	5.24	100	0.052	0.225	RC	0.013	2.6

MH-43	1437.42	0.42	100	0.004	0.375	RC	0.013	1.0
MH-44	1437.83	0.42	100	0.004	0.375	RC	0.013	1.0
MH-45	1438.25	2.75	100	0.028	0.150	RC	0.013	1.4
MH-46	1441.00	2.75	100	0.028	0.100	RC	0.013	1.1
MH-47	1441.68	0.68	100	0.007	0.225	RC	0.013	0.9
MH-48	1442.35	0.68	100	0.007	0.225	RC	0.013	0.9
MH-49	1443.03	4.97	100	0.050	0.15	RC	0.013	1.9

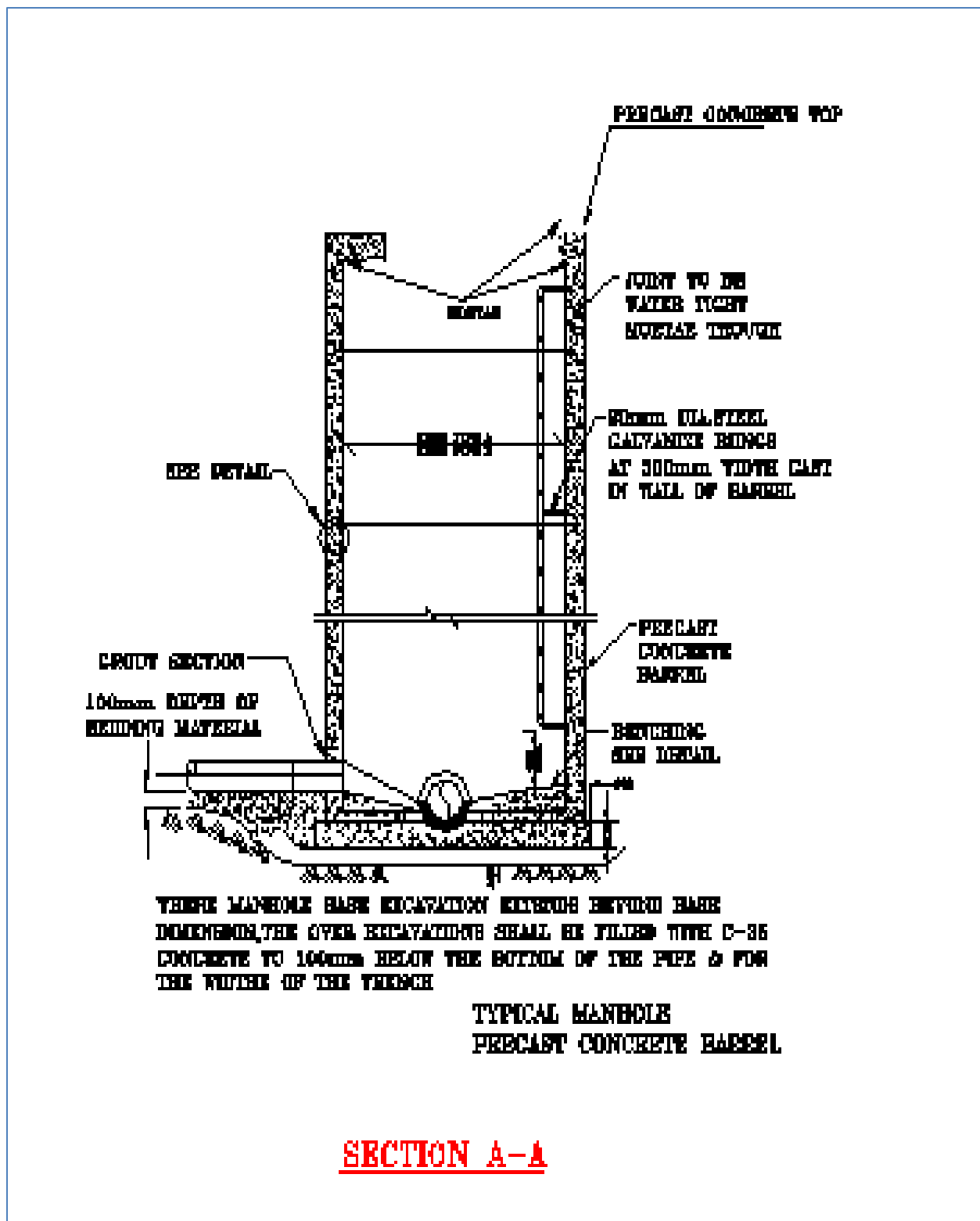


Figure 2.2 Plan and Concrete bases in circular manhole

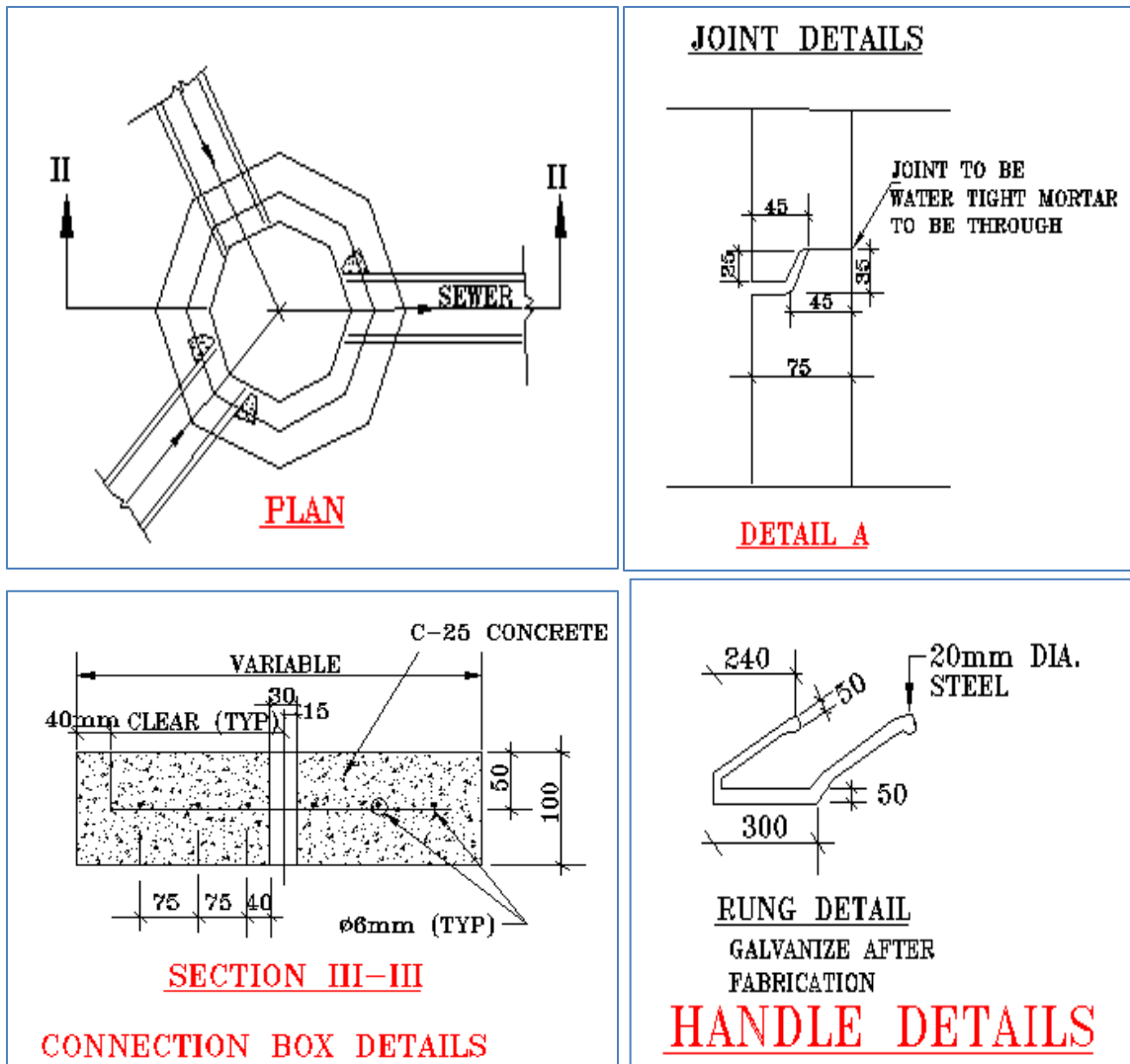


Figure 2.3 Proposed circular manhole Sectional view and details

