



HYDRAULIC PERFORMANCE ASSESSMENT OF STORM WATER DRAINAGE  
SYSTEMS OF DEJEN TOWN USING STORM WATER MANAGEMENT MODEL

MASTERS OF SCIENCE THESIS

BEKALU ZELALEM TILAHUN

HAWASSA UNIVERSITY, HAWASSA, ETHIOPIA

APRIL, 2021

HYDRAULIC PERFORMANCE ASSESSMENT OF STORM WATER DRAINAGE  
SYSTEMS OF DEJEN TOWN USING STORM WATER MANAGEMENT MODEL

BEKALU ZELALEM TILAHUN

A THESIS SUBMITTED TO THE

FACULTY OF BIO-SYSTEMS AND WATER RESOURCE ENGINEERING  
DEPARTMENT OF WATER RESOURCE AND IRRIGATION ENGINEERING

HAWASSA UNIVERSITY INSTITUTE OF TECHNOLOGY

SCHOOL OF GRADUATE STUDIES

HAWASSA UNIVERSITY

HAWASSA, ETHIOPIA

IN PARTIAL FULFILLMENT OF THE

REQUIREMENTS FOR THE

DEGREE OF

MASTERS OF SCIENCE IN IRRIGATION AND DRAINAGE ENGINEERING

(SPECIALIZATION: IRRIGATION AND DRAINAGE ENGINEERING)

APRIL, 2021

**SCHOOL OF GRADUATE STUDIES**

**HAWASSA UNIVERSITY**

**ADVISOR'S APPROVAL SHEET**

This is to certify the thesis entitled ‘‘Hydraulic performance assessment of storm water drainage systems of Dejen Town using Storm Water Management Model’’ submitted in partial fulfillment of the requirements for the Degree of Master’s with specialization in Irrigation and Drainage Engineering, the Graduate program of Faculty of Bio-systems and Water resources Engineering Department of Water resources and Irrigation Engineering and it has been carried out by Bekalu Zelalem Tilahun (Id: GPIrDrR0004/11) under our supervision. Therefore, we recommend that the student has fulfilled the requirements and hence hereby can submit the thesis to the department.

Tewodros Asefa (PhD)

\_\_\_\_\_

\_\_\_\_\_

Major Advisor

Signature

Date

Gonse Amalo (M.Sc.)

\_\_\_\_\_

\_\_\_\_\_

Co-Advisor

Signature

Date

**SCHOOL OF GRADUATE STUDIES**

**HAWASSA UNIVERSITY**

**EXAMINERS' APPROVAL SHEET**

As members of Examining board of the final M.Sc. open defense, we certify that we have read and evaluated the thesis prepared by Bekalu Zelalem entitled as ‘Hydraulic performance assessment of storm water drainage systems of Dejen Town using Storm Water Management Model’ and we recommend that it can be accepted as fulfilling the thesis requirement for the degree of masters of science in Irrigation and Drainage Engineering.

Tewodros Assefa (PhD)

\_\_\_\_\_

\_\_\_\_\_

Name of Major Advisor

Signature

Date

\_\_\_\_\_

\_\_\_\_\_

\_\_\_\_\_

Name of Internal Examiner-I

Signature

Date

\_\_\_\_\_

\_\_\_\_\_

\_\_\_\_\_

Name of Internal Examiner-II

Signature

Date

\_\_\_\_\_

\_\_\_\_\_

\_\_\_\_\_

Name of External Examiner

Signature

Date

\_\_\_\_\_

\_\_\_\_\_

\_\_\_\_\_

SGS Approval

Signature

Date

Final approval and acceptance of the thesis is contingent upon the submission of the final copy of the thesis to the School of Graduate Studies (SGS) through the Department/School Graduate Committee (DGC/SGC) of the candidate's department.

**Stamp of SGS Date**

**Remark**

(Use this form to submit the thesis with minor correction suggested by the examining board).

## **ACKNOWLEDGEMENTS**

It is important to acknowledge who had contributed to this study directly and indirectly. First of all I would like to express appreciation to my Advisors, Dr.Tewodros Assefa and Mr. Gonse Amalo. Their guidance, encouragement, and support during my work enable me to enhance and develop my knowledge. I would also like to thank Assosa University for giving sponsorship for my Master's study at Hawassa University. In addition, I would like to thank organizations who gave me necessary data for my study, like Ethiopian Meteorological Service Agency (EMSA) and Dejen Town municipality. Finally, I would like to express my genuine gratefulness to my father, mother, brothers, sisters, and friends for their motivation and support.

## Table of Contents

ACKNOWLEDGEMENTS .....	I
DECLARATION .....	V
LIST OF TABLES .....	VI
LIST OF ABBRIVATION AND ACRONOMYS .....	VII
LIST OF FIGURES .....	VIII
LIST OF TABLES IN APPINDICES .....	IX
ABSTRACT.....	X
1. INTRODUCTION .....	1
1.1. Background .....	1
1.2. Statement of the Problem .....	3
1.3. Objectives of the Study .....	4
1.3.1. General Objective of the Study .....	4
1.3.2. Specific Objectives of the Study .....	4
1.4. Research Questions .....	4
1.5. Significance of the Study .....	4
1.6. Scope and Limitation of the Study.....	5
2. LITERATURE REVIEW .....	7
2.1. History of Urban Storm water Drainage .....	7
2.2. Current Urban Storm water Drainage Practices.....	8
2.3. Storm water Drainage Practices in Ethiopia .....	8
2.4. Causes and Effects of Urban Storm water Drainage Systems .....	9
2.4.1. Causes of urban flooding.....	9
2.4.2. Effects of urban runoff .....	9
2.4.3. Urban drainage and its importance.....	10
2.5. Hydraulics of Storm Drainage Systems .....	10
2.5.1. Assumptions of flow type.....	10

2.6. Hydraulic and Hydrological Modeling Using SWMM.....	10
2.6.1. EPA’s Storm Water Management Model (SWMM).....	10
2.7. Land Use Types and Classification.....	15
2.8. Hydrological Soil Groups in Ethiopia.....	16
3. MATERIALS AND METHODS.....	17
3.1. Description of the Study Area.....	17
3.1.1. Population.....	18
3.1.2. Topography.....	18
3.1.3. Climate .....	18
3.2. Data Collection and Analyses .....	19
3.2.1. Assessing current situation of existing drainage systems.....	19
3.2.2. Development of Intensity-Duration-Frequency (IDF) curve of Dejen Town ....	19
3.2.3. Simulation of SWMM and predicting water level at links.....	35
3.3. Research Framework and Procedures .....	38
4. RESULTS AND DISCUSSIONS.....	39
4.1. Assessment of Current Situation of Existing Drainage System in Study Area.....	39
4.1.1. Existing drainage conditions of study Area.....	39
4.1.2. Lack of community awareness .....	43
4.1.3. Problem of outlet ditches sanitation .....	46
4.2. Developing IDF curve to Estimating Peak Runoff and Existing drainage capacity .	46
4.2.1. Intensity-Duration-Frequency curve of Dejen Town .....	47
4.2.2. Selection of distribution method and design rainfall of shorter duration.....	47
4.2.3. Gumbel’s extreme-value distribution method.....	47
4.2.4. Log-Pearson III distribution method .....	48
4.2.5. General Pareto distribution.....	49
4.2.6. Design rainfall of shorter duration .....	50
4.2.7. Land use Type of Study Area.....	53

4.2.8. Rational method and estimating peak runoff.....	54
4.2.9. Determining hydraulic capacity of existing drainage structures .....	56
4.3. Simulation of Storm Water Management Model (SWMM) .....	58
4.3.1. Water depth and flow in the Links .....	58
5. SUMMARY AND CONCLUSION .....	66
5.1. Summary and Conclusion .....	66
5.2. Recommendations .....	68
REFERENCES .....	69
APPENDICES .....	72

## **DECLARATION**

I declared that this thesis is my effort work and it has not been presented in whole and part by me to any other university for similar or any other degree award.

Additionally, I ensure that the work is original and to the best of my knowledge except that had been taken from the sources cited.

Name: Bekalu Zelalem

Signature: \_\_\_\_\_

Date: \_\_\_\_\_

## LIST OF TABLES

Table 3.1: Average temperature of study area in ( $C^0$ ).....	19
Table 3.2: Goodness of fit test by Hydrognomon.....	28
Table 3.3: Rank of probability distributions resulted from Easy fit 5.6.....	31
Table 3.4: Input parameters for the SWMM model.....	37
Table 3.5: Description of SWMM input parameters.....	37
Table 4.1: 24-hr RF depth Vs frequency of Gumbel's Extreme-value Distribution.....	47
Table 4.2: 24-hr RF depth Vs frequency of Log- Pearson distribution.....	48
Table 4.3: 24- hour rainfall depth Vs Frequency Dejen and ERA.....	49
Table 4.4: R24 rainfall depth Vs frequency of each distribution.....	49
Table 4.5: Calculated Intensity Vs Duration of Dejen Town.....	51
Table 4.6: Comparison of IDF curve of Dejen Town with IDF curve of ERA.....	53
Table 4.7: Peak runoff calculated by rational method.....	56
Table 4.8: Comparing estimated and existing peak discharge.....	57
Table 4.9: Comparing runoff resulted from rational method, SWMM and Manning's Formula.....	63

## LIST OF ABBRIVATION AND ACRONOMYS

CBE	Commercial Bank of Ethiopia
CMS	Cubic Meters per Second
CSA	Statistical Agency of Ethiopia
DEM	Digital Elevation Model
DL	Drainage Line
EDDM	ERA Drainage Design Manual
EPA	Environmental Protection Agency
ERA	Ethiopian Roads Authority
ESRI	Environmental System Research Institute
FAO	Food and Agriculture Organization
GIS	Geographic Information System
GPD	Generalized Pareto Distribution
GPS	Global Position System
GTZ	German Technical Cooperation
HSG	Hydrologic Soil Group
IDF	Intensity-Duration-Frequency
IDSM	Inverse Distance Square Method
LID	Low Impact Development
MPI	Model Performance Indicator
MRL	Mean Residual Life
NMA	National Meteorological Agency
NRCS	Natural Resources Conservation Services
POT	Peak-Over-Threshold
RR	Rainfall Region
SCS	Soil Conservation Services
SWMM	Storm Water Management Model
T <sub>c</sub>	Time of concentration
T <sub>ci</sub>	Time of concentration inlet
T <sub>t</sub>	Travel time
USGS	United States Geological Survey
USWD	Urban Storm Water Drainage

## LIST OF FIGURES

Figure 2.1: Urban waste and stormwater drainage techniques of ancient Greece.....	7
Figure 2.2: Elements of a typical urban drainage system.....	11
Figure 3.1: Location and map of the study area by ArcGIS.....	17
Figure 3.2: Border of Dejen Town and the surrounding by Google Earth.....	18
Figure 3.3: Rainfall Regions of Ethiopia.....	21
Figure 3.4: IDF Curve of Rainfall Region A-2.....	21
Figure 3.5: Thiessen polygon by ArcGIS.....	23
Figure 3.6: Maximum monthly rainfall/year of Dejen from (1987-2018).....	25
Figure 3.7: Double mass curve to check consistency of data of study area.....	26
Figure 3.8: Plot of Mean Residual Life.....	30
Figure 4.1: Drainage condition from CBE Abaysheleko branch to Fasiledes outfall.....	40
Figure 4.2: Drainage condition from Demis Hotel to Dejen 02 Primary school.....	40
Figure 4.3: Drainage condition around Seble Hotel.....	41
Figure 4.4: Drainage condition from CBE Dejen branch to new bus station.....	42
Figure 4.5: Drainage condition from Andinet College to Mulugojam hall.....	42
Figure 4.6: Drainage condition from St. George church to Litete gorge outfall.....	43
Figure 4.7: Clogged structures with electric poles, construction materials, and wastes...	44
Figure 4.8: Deposition plastic materials in to drainage ditches.....	45
Figure 4.9: Broken road side ditches.....	45
Figure 4.10: Clogged and broken outlet ditches.....	46
Figure 4.11: Intensity-Duration-Frequency curve of Dejen Town.....	52
Figure 4.12: Land use of Dejen Town and surrounding by ArcGIS.....	54
Figure 4.13: Water elevation profiles towards Fasileds bridge outfall.....	59
Figure 4.14: Water elevation profiles towards Aduadem gorge outfall.....	61
Figure 4.15: Water elevation profiles towards Litete gorge outfall.....	62
Figure 4.16: Discharge resulted from rational method, SWMM and Manning's Formula.	64
Figure 4.17: Map of Sub-catchments Modeled by SWMM.....	65

## LIST OF TABLES IN APPINDICES

APPENDICES.....	72
Appendix 1: Mean monthly rainfall, cumulative rainfall of stations, rainfall of shorter duration, 24-hr rainfall depth, and comparison of IDF Curve.....	72
Table 1a: Dejen Town mean monthly rainfall from (1987-2018) in (mm).....	72
Table 1b: Cumulative rainfall of each station with average station in (mm).....	73
Table 1c: Dejen Town design rainfall of shorter duration.....	74
Table 1d: IDF curve developed for Dejen Town and IDFC of ERA for RR-A2.....	76
Table 1e: Return period vs 24-hr rainfall depth for General Pareto, Gumbel and Log person-III for study Area.....	77
Appendix 2: Estimated drainage capacity and existing drainage capacity.....	78
Table 2a. Estimated drainage capacity of study Area by Rational method.....	78
Table 2b: Frequency factor for rational formula.....	79
Table 2c: Capacity of existing drainage canal by Manning’s Formula.....	79
Table 2d: Drainage condition of sub-catchments.....	85
Appendix 3: Distribution methods and Goodness of fit test.....	88
Table 3a: Goodness of Fit test results and rank of distribution methods.....	88
Appendix 4: SWMM input parameters and SWMM output.....	89
Appendix 5: Different standards and constants used for study.....	94

## **ABSTRACT**

*Stormwater drainage problem is a major challenge facing most of the Cities and Towns of the world including Cities and Towns in Ethiopia. As Dejen Town has stormwater drainage problems, this study was conducted to assess the current performance of the Town, to estimate and predict flood amount, and to design drainage structure for a design period of 10-years to solve such problems. To do this, first meteorological data such as rainfall data of the study area from (1987-2018), drainage line, and plan of the Town was collected from concerning bodies, and field survey of the study area was done to measure the drainage canal dimensions, and Global Position System of the Junctions. Then missing rainfall data was filled; data consistency was checked and analyzed using Microsoft-excel. Assessment of the current drainage condition of the study area was performed and the current drainage capacity of existing structures was calculated using measured canal dimensions by Manning's formula. Nevertheless, most of the existing drainage structures were inadequate to dispose runoff to the outfall area and most drainage structures in study area were poor. The goodness of fit test indicates General Pareto distribution fits for flood frequency analysis and Intensity-Duration- Frequency curve was developed by using this distribution method. By using rainfall intensity calculated from the developed Intensity-Duration-Frequency curve, the amount of peak discharges were estimated by the rational formula for each sub-catchments for a 10-years design period. Then, Storm Water Management Model was applied for the prediction of discharge in each sub-catchment. The Storm Water Management Model output indicated that the discharge resulted from the sub-catchments was greater than the existing capacity in most sub-catchment. This implies in most of the canals, Junctions, and outfalls the flood level was greater than the designed water level, and over-flooding occurs at drainage canals and most of the junctions were over flooded. To solve these problems the following mitigations were recommended; construction of additional drainage structures with proper dimension especially for secondary roadsides with no drainage structures, design and construction of well-connected structures, adopting the culture of clearing sediment and periodic repairing of drainage structures before total failure.*

**Keywords:** *Dejen Town, Performance Assessment, Runoff, Stormwater & SWMM*

# 1. INTRODUCTION

## 1.1. Background

Water is very essential for all life on the earth; it can also cause devastation through erosion and flooding if not managed properly. Due to the development of infrastructures as a result of urbanization, the surface runoff water greatly increased in the Town damaging the roads. The contributed runoff water thus needs to be safely disposed to the rivers or outlet channels so that the utility of the road infrastructure is maintained and thereby avoids the damages which otherwise occurred to the road and property (Warati and Getachew, 2015).

In many cases the so called floods are not caused by rivers overflowing but are caused by the inadequate drainage facilities. In urban areas this phenomenon occurs due to haphazard construction with poor planning which does not allow sufficient retention and percolation areas. In some cases people encroach drainage areas, even obstructing drainage paths and disrupting natural drainage patterns (Kapila, 2008).

Flooding belongs to the most threatening natural hazards causing immense economic and social losses throughout the world. Flood hazard is expected to increase in frequency and severity through the impacts of global change on climate, severe weather in the form of heavy rain, and river discharge conditions (Dihn *et al.*, 2012). An urban flood occurs due to a complex interplay of factors including intensity and duration of rainfall, the characteristics of the urban land surface, and engineering design of surface drainage and sewer systems (Jenkins, 2016). The increment of urbanization also increases impermeability because of the increase in impervious surfaces. This increment of impervious surface changes the drainage pattern, increase overland flow resulting in flooding and related environmental problems. This impact is severe on road structures. Because, flooding and its related environmental problems like sheet flow and gulley erosion, surface inundation tends to affect road services and its life span (Dagnachew, 2011).

Flood is inundation of land due to river overflow, which is caused by heavy rain or torrents of water from other areas in higher places. Floods are one of the disasters feared by people in society, because floods come with a high water discharge, inundate at a high level for a

long time, and carry waste materials that interfere with activities in society (Sholihah *et al.*, 2020).

Stormwater is any precipitation such as rain, and snow that falls on the surface of the earth. It is rainfall fallen from the built-up area. Stormwater drainage is the process of draining excess water from streets, roofs, sidewalks, buildings and other areas. The storm drain is the system used to drain stormwater (Sundara *et al.*, 2015). Stormwater drainage networks in cities are designed to collect effectively and convey excess runoff to prevent urban flooding (Gouri and Srinivas, 2015).

The provision of sufficient stormwater drainage structures is important during the design and construction of urban roads. Drainage facilities on the road should adequately be provided for the flow of water away from the surface of the pavement to properly designed channels. A properly designed and implemented drainage system should effectively intercept all surface and watershed runoff and direct this runoff into adequately designed channels for eventual discharge into proper outfall or the natural waterway (Kokeb and Addisu, 2016). Adequate drainage is very essential in the design of roads since it affects the road's serviceability and usable life. Drainage design involves providing facilities that collect, transport and remove stormwater from the highway.

Stormwater on residential sites can be dealt with in a number of ways. The following techniques, which can be integrated into new construction and existing residential settings, help to manage stormwater: Increasing permeability, Directing water to more permeable areas, Detaining water to allow infiltration, Intercepting and holding rainwater, and Utilizing water on-site as it is needed (John, 2015). Storm water management is the control and use of storm water runoff. It indicates planning for runoff, maintaining storm water systems, and regulating the collection, storage, and movement of storm water. Storm water management also considers drainage in the design of cities and housing developments. The goal of storm water management includes protecting our environment; reducing flooding to protect people and property; reducing demand on public storm water drainage systems; supporting healthy streams and rivers. Effective storm water management provides environmental, social, and economic benefits to local communities. When storm water management is done well streams, rivers, and lakes are cleaner, food risks are reduced, costs due to flood damage decrease, and community quality of life increases (Bobbi and T., 2014).

## **1.2. Statement of the Problem**

The pattern of urbanization and modernization in Ethiopia has meant increase densification along with urban infrastructure development and led to deforestation, use of corrugated roofs and paved surfaces. The combined effect of this results in higher raindrop intensity and consequently accelerated and concentrated runoff. Due to inadequate integration between road and urban stormwater drainage infrastructure provision and poor management, significant proportion of the area is exposed to flooding hazards (Kalantari, 2011). Inadequate stormwater drainage and lack of urban storm drainage management are the most common problems of the citizens in many Towns of Ethiopia and this problem is getting worse and worse with the ongoing high rate of urbanization (GTZ, 2006).

Currently, Dejen Town is the one facing stormwater drainage problems due to expansion of urbanization, and change in land use that results in a change in runoff characteristics in the Town and increases susceptibility to flooding. During intensive rainfall flooding occurs in the study area due to inadequate drainage structures to convey flood to the outlet point, increasing of urbanization and impervious surface, improper management of waste such as using drainage structures as solid waste removal, lack of periodic maintenance of manhole, lack of well-connected drainage networks, and lack of drainage structures in secondary roads.

Those problems create the structures to malfunction during the rainy season every year and leads to flooding over asphalts, walkways, and near the residences in Town. These drainage problems were observed near CBE Abaysheleko branch, around Demis Hotel, around Dejen 02 primary school, around Dejen Town 02 kinder gardens, around CBE Dejen branch, around Mulugojam hall, around Andinet business Technology College, around Serkalem Hotel, and around Kurar Amba Hotel.

To prevent the flooding problems in the study area it is essential to create public awareness not to dump solid waste material into drainage structures, periodic maintenance and cleaning of drainage structures before total failure, additional design and construction of drainage structures in secondary roads, adopting closed drainage structures to reduce sedimentation and breaking, periodic maintenance and cleaning of outlet ditches, and redesign must be done in flood-prone areas according to estimated discharge and flood discharge resulted from the rational method and predicted by SWMM.

According to Rossman (2016), Storm Water Management Model (SWMM) is essential software to design stormwater drainage structures because the routing portion of the Storm Water Management Model transports runoff through a system of pipes, channels, treatment devices, pumps, and regulators. Storm Water Management Model is used throughout the world for planning, analysis, and design related to stormwater runoff, combined and sanitary sewers, and other drainage systems in urban areas. It is used for a single event or long-term simulation of runoff quantity and quality from urban areas.

### **1.3. Objectives of the Study**

#### **1.3.1. General Objective of the Study**

The general objective of the study was to assess the hydraulic performance of the storm water drainage systems of Dejen Town by using Storm Water Management Model (SWMM).

#### **1.3.2. Specific Objectives of the Study**

The specific objectives of this study were:

- To assess the current condition of the urban drainage system in the study area.
- To develop the Intensity-Duration-Frequency (IDF) curve of Dejen Town for different return periods to determine the peak runoff.
- To simulate Storm Water Management Model (SWMM) for prediction of flooding problem at links and nodes in the drainage networks.

### **1.4. Research Questions**

1. What is the current condition of the existing drainage system of Dejen Town?
2. What is the Intensity-Duration-Frequency (IDF) curve of Dejen Town for various return periods, and what will be the estimated peak runoff?
3. What will be the flooding problem at links and nodes resulting from the SWMM simulation?

### **1.5. Significance of the Study**

The road that has no proper drainage systems causes failure of pavements, flooding of the environment, affecting traffic interruption, reproduction of aquatic animals like malaria,

and bad smell that causes health problems. It fails to serve its function and it requires a high maintenance cost, skilled people to redesign, working force, and time.

According to the study of Federal Democratic Republic of Ethiopia National Disaster Risk Management Commission, Early Warning and Emergency Response Directorate in 2018, Dejen is one of the flood alert Towns in Amhara Regional State with a high drainage problem. Therefore, the significance of this research is to solve the possible damage of roads caused due to flooding, and social problems through the provision of proper drainage structures that reduces the environmental and health safety problems of the Town. To give awareness to the community on the problems of damage and preserve the structures by avoiding further deterioration for taking corrective measures, and to decrease the overflow of flood in the road of the Town. Also, the research helps to fill the gaps by identifying problems for sustainable and proper designing of a stormwater drainage system. It also helps to create awareness for people not to dump the sewages and waste materials to drainage ditches, manholes, and sewerage systems.

In general, this research contributes the following importance; to identify the existing drainage condition to know the real problems of the drainage systems, to find possible solutions based on the recommendations given in this research, and to minimize the possible damage of pavement by providing proper and well-connected drainage structure.

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

The thesis focused on performance assessment of existing drainage systems of Dejen Town from Sg. George church to Andinet Business Technology College, and from CBE Abaysheleko branch to MAFLAB International up to flood outlet of Aduadem gorge to Fasiledes Bridge. The area modeled was covered about 136 hectares of the Town. It covers the areas above and below the main asphalt road. This study specifically focused on the analysis of stormwater drainage systems and to enable sustainability in drainage systems that contribute to better service. Based on the available data, the analysis of the existing drainage systems in Dejen Town was done and possible mitigation measures were recommended.

#### **○ Limitation of the Study**

Measuring the water level in the junctions and outfalls is not common in our country like developed countries. For instance, there is no recorded data of runoff at different junctions

and outfalls to calibrate the simulated result of the model at junctions and outfalls. As Storm Water Management Model (SWMM) requires a real record of runoff at different junctions and outfalls to calibrate the results of the simulation, the results of the model were simulations without calibration. But, the results of peak runoff resulted from SWMM were compared with estimated runoff calculated by Rational method, because results runoff obtained from SWMM approximates those obtained from other well-known methods such as Rational method, SCS Curve Number Method, and Unit Hydrograph Method.

## 2. LITERATURE REVIEW

### 2.1. History of Urban Storm water Drainage

Historical books of ancient civilization such as Indus and Minoan suggests urban drainage system were constructed with great care and the objective of systems were to collect rainwater, prevent flooding and convey wastes. The systems that ultimately met their objectives likely did after trial-and-error modifications. Planning and design in ancient civilizations were limited. Few numerical standards existed for urban drainage and engineering calculations were not used during design. Despite of lake of optimization and the use of trial-and error construction methods, several ancient urban drainage systems can be rated very successfully (Mumford, 1961).

The beginning of modern urban drainage practices was initiated in European cities during the 19<sup>th</sup> century. The Paris sewer system was improved by initial efforts of Engineer Pierre- Emmanuel Bruneseau. Bruneseau documented the deterioration of sewers by leading voyages into underground labyrinth. Also, Bruneseau suggested that unhealthiness was among the major causes of depopulation. Despite Bruneseau's efforts, his mission of reforming the Paris sewers was far from complete when he died in 1819. H.C. Emery, head of the Paris sewer system from 1832 to 1839, furthered the development of the Paris sewers by replacing the open channels running along the center of streets with gutters constructed under sidewalks. Inlets were provided from the gutters to the sewer system (Reid, 1991).



Figure 2.1: Urban waste and storm water drainage techniques of ancient Greece

## **2.2. Current Urban Storm water Drainage Practices**

Urban drainage in the early parts of the twentieth century was the first established as a vital public work system. During the second half of the twentieth-century regulatory elements were broadcasted in the U.S.A, Europe, and other locations addressing urban drainage issues. Extensive monitoring efforts vastly improved the understanding of urban drainage quality and quantity characteristics. Computer modeling tools advanced the methods used to design and analyze urban drainage systems. Methods to design and construct sustainable urban drainage systems are currently being researched and tested. Another development concept is influencing development practices to minimize the impacts of development on stormwater drainage. Urban drainage has certainly expanded significantly during the past few decades beyond a technical challenge to drain urban areas including consideration of social, economic, political, environmental, and regulatory factors (Steven *et al.*, 2002).

## **2.3. Storm water Drainage Practices in Ethiopia**

In Ethiopia, the pattern of urbanization and modernization has intended to increase densification along with urban infrastructure development. This has led to deforestation, the use of corrugated roofs and paved surfaces. The combined effect of this results in higher rainfall intensity and consequently accelerated and concentrated runoff in the urban areas (Dagnachew, 2011). Urban drainage systems used to handle wastewater originating from human uses and stormwater that comes from rainwater that has fallen from built-up areas intending to minimize the problems caused to human life and the environment (Kokeb and Addisu, 2016).

Good and effective stormwater management is seriously required at the moment all over the globe especially in developing countries like Ethiopia. The related studies on drainage issues in Ethiopia include studies on the following titles. Study of urban drainage system in Addis Ababa, yeka sub-city by (Dagnachew 2009), Hydraulic Analysis of Road Drainage Structures and Proposed Mitigation Measures (A Case study of Bulehora to Yirgacheffe Asphalt Road) by (Mitiku Chachu, 2018), Performance assessment of road drainage structures and proposed mitigation measures in case of Daleti-odagodere Gravel road in Benishangul gumuz region (Yefired, 2015), Assessment of stormwater drainage in kemise Town (Biniyam Asefa, 2016), Investigation on storm drainage problem of Addis Ababa (A

case study at Gotera – Wollo sefer, saris - gotera and ring road) by (Desalegn Getachew, 2011) and Performance assessment of stormwater drainage systems (Case study of Debere Berehan town) by (Eyosias Birhanu,2018).

## **2.4. Causes and Effects of Urban Storm water Drainage Systems**

### **2.4.1. Causes of urban flooding**

Urban runoff is surface runoff of precipitation created by urbanization. This runoff is a major source of flooding and water pollution in urban communities in the world. Impervious surfaces such as roads, parking lots, rooftops, and sidewalks are constructed during land development. During rainstorms and other precipitation events, these surfaces carry polluted stormwater to storm drains instead of allowing the water to percolate through soil (Reston, 2016).

The causes of urban flooding may include physical factors and susceptibility factors. Physical factors include the topography of the city such as slope and elevation and susceptibility factors like insufficient urban drainage, insufficient planned areas responsible for waterlogging of streets, etc. Among such factors, Dejen Town drainage problem is caused due to vulnerability factors such as insufficient urban drainage that hinders the flow of flood into outfall area.

**Topography (Slope and Elevation):** The influence of topography can be evidenced in many incidents. Even if the slope difference in the town is efficient insufficiency and quality of drainage structures in the town causes water accumulation and breaking of runoff into roads. Urbanization has also resulted in more built-up areas and reduced permeable surfaces and open spaces that lead to greater runoff (Bansal *et al.*, 2015). Urban runoff is a major cause of urban flooding, the inundation of land or property in a built up environment caused by rainfall overwhelming the capacity of drainage systems such as storm sewers (USGS, 2016).

### **2.4.2. Effects of urban runoff**

Urbanization constantly modifies land use within a city, leading to several effects in the hydrologic cycle such as decreasing infiltration and increasing flood peak and runoff. Urban flooding is one of the most dangerous impacts on human societies, leading not only huge economic damage and loss of properties but also the most sensitive issue in loss of

human lives (Diego *et al.*, 2014). Urban runoff is also the source of water quality problems, harming fish and other aquatic organisms (USGS, 2009).

### **2.4.3. Urban drainage and its importance**

Adequate drainage systems are needed in urban areas because of the interaction between human activity and the natural water cycle. There are two types of water that require drainage. The first type is wastewater that is water after human beings supply it to support their life, to maintain standards of living, and water removed after satisfying the needs of industries. If not drained properly after use it may cause pollution and health risks. The second type of water requiring drainage is stormwater that is rainwater fallen on a built-up area. If stormwater were not drained properly, it would cause inconvenience, damage, and flooding and health risks. It contains some pollutants originating from rain, air or catchment surface. Urban drainage systems handle these two types of water to minimize the problems caused to human life and the environment (Kokeb and Addisu, 2016).

## **2.5. Hydraulics of Storm Drainage Systems**

### **2.5.1. Assumptions of flow type**

The total runoff volume and peak rate of runoff is estimated by many methodologies developed earlier. These methods include a rational method, Modified rational method, and SCS method.

**Rational method:** The idea behind the Rational method is if rainfall intensity begins instantaneously and continues indefinitely, the rate of runoff will increase until the time of concentration  $t_c$  when the entire watershed is contributing to flow at the outlet. The product of rainfall intensity and the watershed area is the inflow rate of the system,  $i^*A$ , and the ratio of this rate to the rate of peak discharge  $Q$  is termed the runoff coefficient  $C$ . The rational method is applicable when the catchment area is less than 50 hectares (ERA, 2011).

## **2.6. Hydraulic and Hydrological Modeling Using SWMM**

### **2.6.1. EPA's Storm Water Management Model (SWMM)**

Storm Water Management Model (SWMM) is a dynamic rainfall-runoff simulation model used for short or long term simulation of runoff quantity and quality primarily from urban areas. The runoff component of SWMM operates on a collection of sub-catchment areas

that receive precipitation and generate runoff and pollutant loads. The routing portion of SWMM transports this runoff through a system of pipes, channels, pumps, treatment devices, and regulators. SWMM tracks the quantity and quality of runoff generated within each sub-catchment and the flow rate, flow depth, and quality of water in each pipe and channel during a simulation period comprised of multiple time steps (Lewis A. Rossman, 2017)

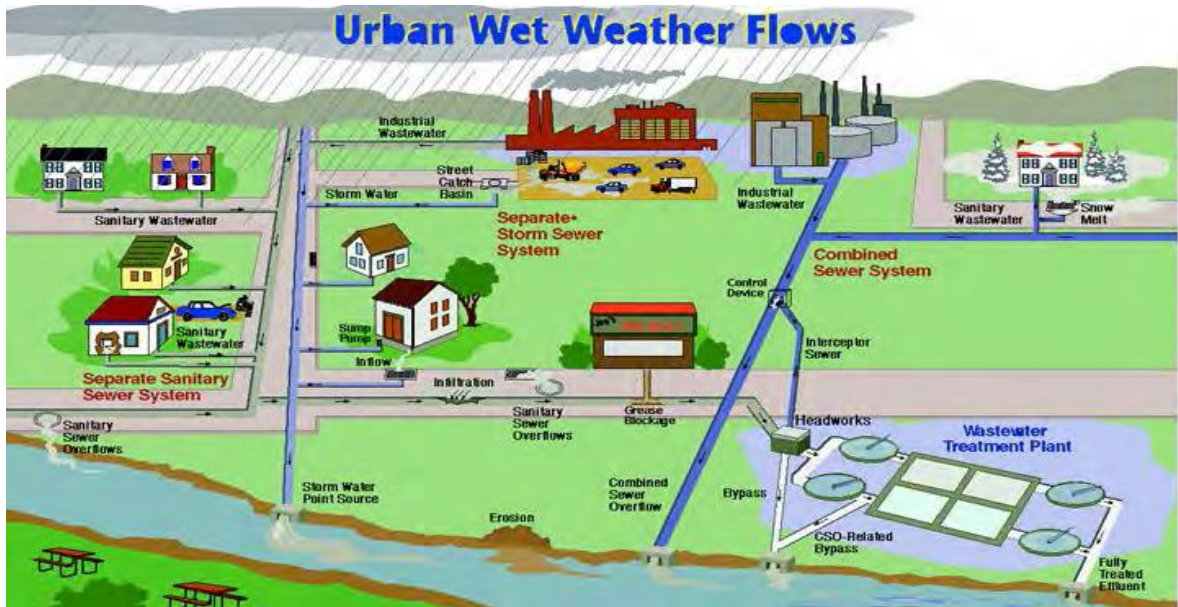


Figure 2.2: Elements of a typical urban drainage system (A.Rossman, Lewis, 2017)

### 2.6.1.1. Modeling capability of SWMM

Storm Water Management Model accounts for various hydrologic processes that produce runoff from urban areas. The hydrologic processes that SWMM accounts are time-varying rainfall, evaporation of standing surface water, snow accumulation and melting, rainfall interception from depression storage, infiltration of rainfall into unsaturated soil layers, percolation of infiltrated water into groundwater layers, interflow between groundwater and drainage system, and nonlinear reservoir routing of overland flow. Spatial variability in all of the processes is achieved by dividing a study area into a collection of smaller, homogeneous sub-catchment areas, each containing its own fraction of pervious and impervious sub areas. SWMM also contains a flexible set of hydraulic modeling capabilities used to route runoff and external inflows through a drainage system network of pipes, channels, treatment units, and diversion structures (Lewis A. Rossman, 2015).

**Hydraulic Modeling:** SWMM contains a flexible set of hydraulic modeling capabilities used to route runoff and external inflows through the drainage system network of pipes,

channels, storage/treatment units and diversion structures. These include the ability to do the following:

- ❖ Handle drainage networks of unlimited size.
- ❖ Use a wide variety of standard closed and open conduit shapes as well as natural channels.
- ❖ Model special elements, such as storage/treatment units, flow dividers, pumps, weirs, and orifices.
- ❖ Apply external flows and water quality inputs from surface runoff, groundwater interflow, rainfall-dependent infiltration/inflow, dry weather sanitary flow, and user-defined inflows.
- ❖ Utilize either kinematic wave or full dynamic wave flow routing methods.
- ❖ Model various flow regimes, such as backwater, surcharging, reverse flow, and surface ponding. Apply user-defined dynamic control rules to simulate the operation of pumps, orifice openings, and weir crest levels.
- ❖ Percolation of infiltrated water into groundwater layers.
- ❖ Interflow between groundwater and the drainage system.
- ❖ Nonlinear reservoir routing of overland flow.

**Accounting for Hydrologic Processes:** SWMM accounts for various hydrologic processes that produce runoff from urban areas, which include the following:

- Runoff reduction via green infrastructure practices.
- Time-varying rainfall (precipitation) and evaporation of standing surface water.
- Snow accumulation and melting.
- Rainfall interception from depression storage.
- Infiltration of rainfall into unsaturated soil layers.
- Percolation of infiltrated water into groundwater layers Interflow between groundwater and the drainage system.
- Nonlinear reservoir routing of overland flow.

Spatial variability in all of these processes is achieved by dividing a study area into a collection of smaller, homogeneous sub-catchment areas. Each of the areas contains its own fraction of pervious and impervious sub-areas. Overland flow can be routed between sub-areas, between sub-catchments, or between entry points of a drainage system.

**Pollutant Load Estimation:** SWMM can estimate the production of pollutant loads associated with stormwater runoff. The following processes can be modeled for any number of user-defined water quality constituents:

- Dry-weather pollutant buildup over different land uses.
- Pollutant wash-off from specific land uses during storm events.
- Direct contribution of rainfall deposition. Reduction in dry-weather buildup due to street cleaning.
- Reduction in wash-off load due to best management practices.
- Entry of dry weather sanitary flows and user-specified external inflows at any point in the drainage system.
- Routing of water quality constituents through the drainage system.
- Reduction in constituent concentration through treatment in storage units or by natural processes in pipes and channels (EPA, 2017).

#### **2.6.1.2. Application of Storm Water Management Model (SWMM)**

The typical application of SWMM includes

- Design and sizing of drainage system components for flood control,
- Sizing of detention facilities and their appurtenances for flood control and water quality protection,
- Flood plain mapping of natural channel systems,
- Designing control strategies for minimizing combined sewer overflows,
- Evaluating the impact of inflow and infiltration on sanitary sewer overflows (Lewis A. Rossman, 2015).

In this study, the rational method and Storm Water Management Model (SWMM) was used for the design of flood computation and its analysis. The rational method was compared with the existing carrying capacity of the main drainage canal system for a certain return period of rainfall intensity. Storm Water Management model is one dimensional model that allow the flow properties to vary along or within the channel only rather than to account for the changes across the channel (Lewis A.Rossman, 2004).

#### **2.6.1.3. Computational methods of SWMM**

Storm Water Management Model is a physically-based, discrete-time simulation model that employs principles of conservation of mass, energy, and momentum wherever appropriate. The methods SWMM uses to model stormwater runoff quantity and quality are described through the following processes.

**A. Surface runoff:** There are several outflows including infiltration, evaporation, and surface runoff. Surface runoff per unit area  $Q$  occurs only when the depth of water in the reservoir exceeds the maximum depression storage. The depth of water over the sub-catchment is continuously updated with time by solving numerically a water balance equation over the sub-catchment.

**B. Infiltration:** is the process of rainfall penetrating the ground surface into the unsaturated soil zone of pervious sub-catchment areas. SWMM offers four methods for modeling infiltration: these are Horton's method, Modified Horton Method, Green-Ampt method, and curve number method.

**HORTON'S METHOD:** is empirical in nature and the best known of the infiltration equations. It is applicable only to events for which the rainfall intensity always exceeds the infiltration capacity.

**MODIFIED HORTON'S METHOD:** the method uses the same parameters as the original Horton method but instead of tracking the time along Horton decay curve it uses the cumulative infiltration volume in excess of minimum infiltration rate as its state variable. It assumes the part of infiltrating water will percolate deeper into the soil at the minimum infiltration rate. This method gives more accurate infiltration estimates when low rainfall intensities occur.

**GREEN-AMPT METHOD:** has received considerable attention in recent years. The original equation was for infiltration with excess water at the surface at all times.

**C. Flow Routing:** Flow routing within a conduit link in SWMM is governed by the conservation of mass and momentum equations for gradual varied and unsteady flow.

Routing methods include;

- ✚ **Steady flow routing**
- ✚ **Kinematic wave routing**
- ✚ **Dynamic wave routing**

**Steady Flow Routing:** represents the simplest type of routing possible by assuming that within each computational time step flow is uniform and steady. Thus it simply translates inflow hydrographs at the upstream end of the conduit to the downstream end with no delay or change in shape. This type of routing cannot account for channel storage, backwater effects, entrance or exit losses, flow reversal or pressurized flow.

**Kinematic wave Routing:** this routing method solves the continuity equation along with a simplified form of momentum equation in each conduit. Kinematic wave routing allows

flow and area to vary both spatially and temporally within a conduit. Kinematic wave routing cannot account for channel storage, backwater effects, entrance or exit losses, flow reversal or pressurized flow and is also restricted to dendritic/tree shape network layouts.

**Dynamic Wave Routing:** solves the complete one-dimensional Saint Venant flow equations and therefore produces the most theoretically accurate results. This equation consists of continuity and momentum equations for conduits and a volume continuity equation at nodes. It is possible to represent pressurized flow with this routing when a closed conduit becomes full. Dynamic wave routing can account for channel storage, backwater, entrance and exit losses, flow reversal, and pressurized flow (GISpipe, 2018).

## **2.7. Land Use Types and Classification**

Land use and soil type have a direct impact on the flood amount, speed and potential to create damage on the environment. Due to these the study must give notice for land use and land cover of the catchments in the study area. This classification can be done with supervised or unsupervised classification.

**Supervised classification-** The main purpose of satellite remote sensing is to interpret the observed data and classify features. The broad types of classification procedures are supervised classification and unsupervised classification. Supervised classification is recommended to use when the classifier have enough knowledge about the features or ground truth data are available for the study area. In supervised classification the training areas are usually collected from the field, from high spatial and temporal resolution of aerial photographs, and satellite images. Supervised classification is used to classify areas perfectly known by the classifier whereas unsupervised classification type of classification used to classify areas which area not known by the classifier using symbols from Landsat image.

**Unsupervised Classification-** Unsupervised classification is recommended to use when no enough knowledge about the features or ground truth data are not available for the study area. It is often used to provide general outlook of participated classes in study area. Whereas supervised classification is used to produce specific classes for all features existed in the image (Fadi, 2015).

## 2.8. Hydrological Soil Groups in Ethiopia

Permeability and infiltration are the principal data required to classify soil in to hydrological soil groups (HSG). Soil conservation service (SCS) has divided soils in to four hydrologic soil groups based on infiltration rates.

**Group A:** includes soils such as sand, loamy sand or sandy loam. These soils have low runoff potential due to high infiltration rates and consist of deep, well-drained sands and gravels.

**Group B:** includes silt loam or loam soils. These soils have a moderately low runoff potential due to moderate infiltration rates and consists soils with moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures.

**Group C:** include sandy clay loam soil type. This soil have moderately high runoff potential due to slow infiltration rates and consists of soils in which a layer exists near the surface that impedes the downward movement of water with moderately fine to fine texture.

**Group D:** includes soils such as clay loam, silty clay loam, sandy clay, silty clay or clay. These soils have high runoff potential due to very slow infiltration rates and consist of clay with high swelling potential, soils with permanently high water tables, soils with clay layer at or near the surface, and shallow soils over nearly impervious parent material (ERA, 2002).

### 3. MATERIALS AND METHODS

#### 3.1. Description of the Study Area

This study was conducted at Dejen Town Amhara National Regional State. Dejen is located in the East Gojam Zone of the Amhara National Regional State on the edge of the canyon of the Abay Basin. It is located in west-central Ethiopia about 241 km from Addis Ababa and located at the latitude and longitude ranging from 10° 9'30" to 10°11'0"N and 38° 7'30" to 38° 10'0"E and its elevation range between 2421 and 2490 meters above sea level. Dejen Town is the administrative center of Dejen Woreda and the checkpoint of traffic crossing regional boundaries. Locating the study area is the first task to conduct certain research and indicates where the study was performed. To locate the study area the location and map of Dejen Town was done with ArcGIS as shown below from Figure 3.1.

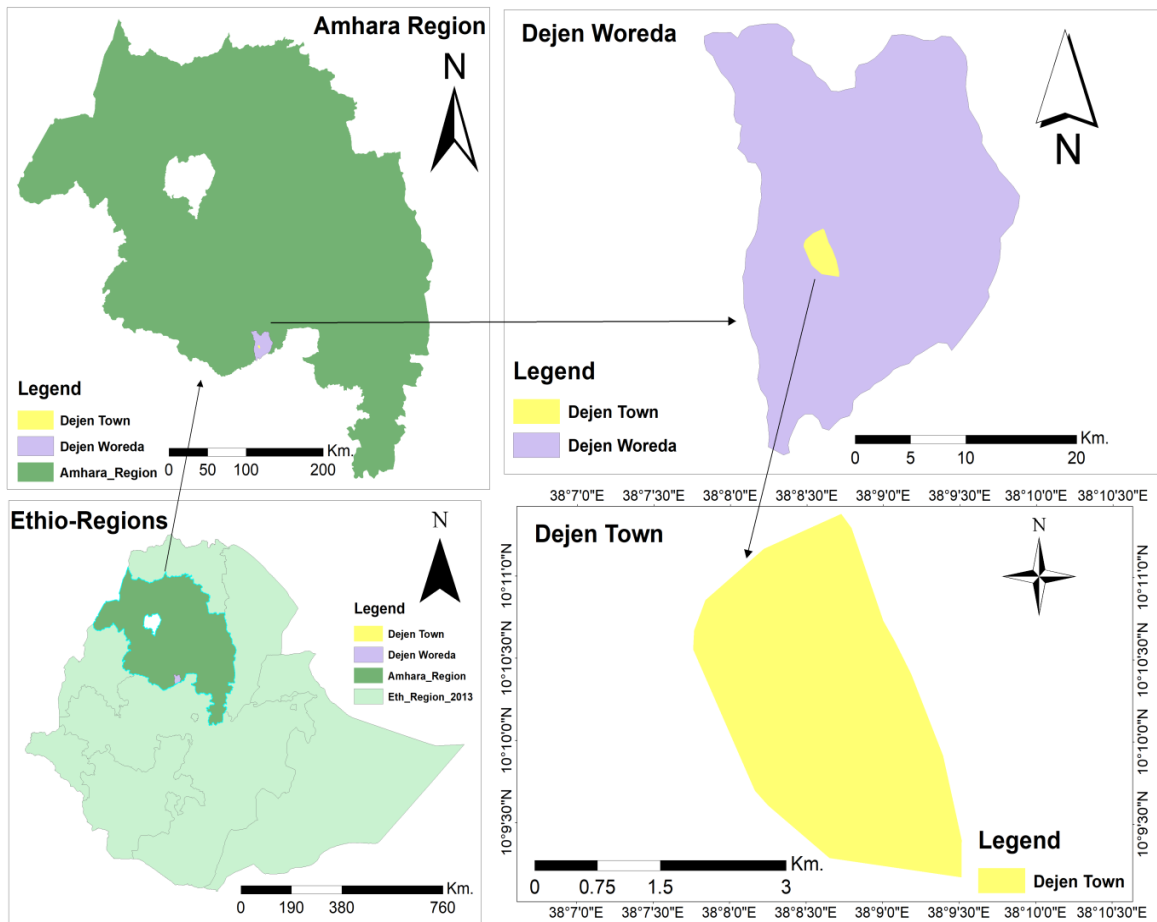


Figure 3.1: Location and map of the study area by ArcGIS

### 3.1.1. Population

The 1994 census reported as Dejen Town had a total population of 8,930 of whom 4,045 were men and 4,885 were women. But, according to the central statistical Agency in 2005, Dejen has an estimated total population of 15,483 of whom 7,688 were men and 7,795 were women.

### 3.1.2. Topography

The topography of Dejen and the surroundings consists of plains, trough, and gorges. The drainage of the project area is governed by the catchments of the Abay River basin.



Figure 3.2: Border of Dejen Town and the surrounding by (Google Earth)

### 3.1.3. Climate

#### ❖ Temperature

Dejen lies on 2421 to 2490m above sea level and its climate is classified as warm and temperate. The summers have a good deal of rainfall, while the winters have very little. The rainfall averages 1138 mm. The least amount of rainfall occurs in January. The average in this month is 4 mm. Most of the precipitation falls in July, averaging 308 mm.

Table 3.1: Average Temperature of study area (C<sup>0</sup>)

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Max.	28	28.1	27.9	28.1	28.0	27.7	26.9	27.6	27.8	28	27.5	29
Min.	8.7	11.7	9.2	10.5	11.7	11.2	10.6	11.0	8.7	9.1	10.5	7
Mean	18	19.9	18.5	19.8	19.8	19.4	18.8	19.3	18.2	18.5	19.0	18

## 3.2. Data Collection and Analyses

### 3.2.1. Assessing current situation of existing drainage systems

To assess the current situation of the existing drainage system of the study area primary data and different information was collected from concerning bodies.

#### Sources of Primary Data

##### **I. Interview of the community and administration**

To obtain the primary data interview was done for concerning bodies such as staff members and head of the municipality, a community in the Town. The primary data were also taken from the municipality and indicates improper design, aging, and improper utilization of drainage systems in the study area.

##### **II. Study Area Observation**

For this study, site visit and capturing different pictures that shows drainage condition at the study area were taken to show the true status of the study area. The dimensions of the drainage structures were measured to know their capacity. Primary data was obtained by a field survey of the study area, by interview to the community and municipality, and other concerning bodies for assessment and determination of existing performance conditions of drainage structures in the study area. Also, the size of existing drainage structures was measured and information was gathering about the overall performance of drainage structures.

Other necessary data that was important for this thesis to gather information about the overall causes and effects of poor drainage and effects of unconstructed stormwater drainage systems were journals, manuals, books, conferences, etc.

### 3.2.2. Development of Intensity-Duration-Frequency (IDF) curve of Dejen Town

To design structures influenced by runoff we must able to assess rainfall. The determination of the probable frequency of extreme rainfall events of different intensities and durations is the basic step in designing flood control structures with economical size.

Assessment of rainfall is usually done using Intensity-Duration-Frequency curves for various water resource-related schemes. Intensity-Duration-Frequency curve is a graphical representation of the probability that rainfall with a particular intensity and duration will occur and the probable time interval between storms with similar characteristics. Intensity-Duration-Frequency curves are a potent representation of the extreme rainfall that is expected in a region of interest which reflects the average intensity of rainfall at every return period for all duration of rainfall. Intensity-Duration-Frequency curves give useful information about when there will be flooding in an area and when a particular rainfall rate or a certain volume of flow will occur in the future. Intensity-Duration-Frequency (IDF) curves are useful in determining the peak runoff in a catchment using a rational method and for designing channels and other waterways (Violina and Beriti, 2017).

### **3.2.2.1. Required data and data source to develop IDF curve**

#### **➤ Rainfall Data**

The data required for the development of IDF curve includes annual maximum rainfall depth over a specific duration to determine maximum rainfall intensity for specific durations, duration of rainfall, depth of rainfall, amount of rainfall. The climate data such as rainfall of Dejen Town was obtained from the National Meteorological Service Agency (NMSA) of Ethiopia to develop the Intensity-Duration-Frequency curve and IDF curve developed by ERA for rainfall region A2 from ERA manual to compare with developed IDF curve for specific study area of this thesis.

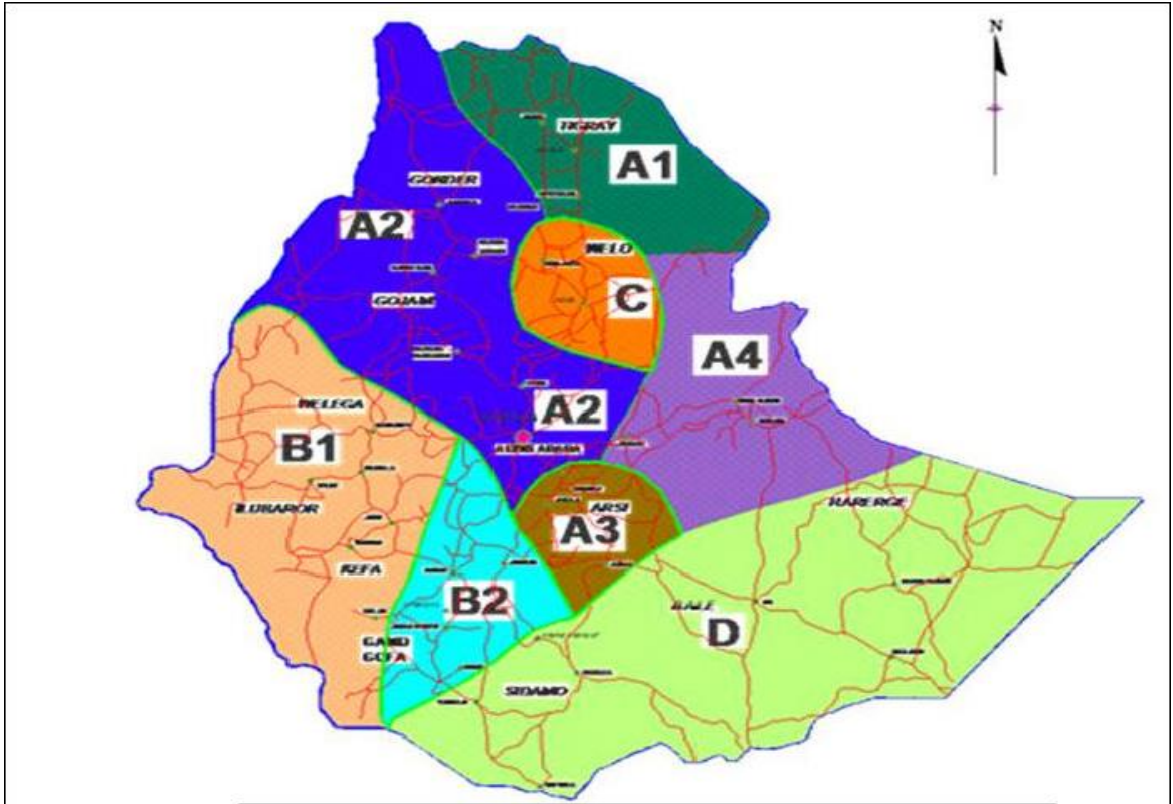


Figure 3.3: Rainfall Regions of Ethiopia (ERA, 2013)

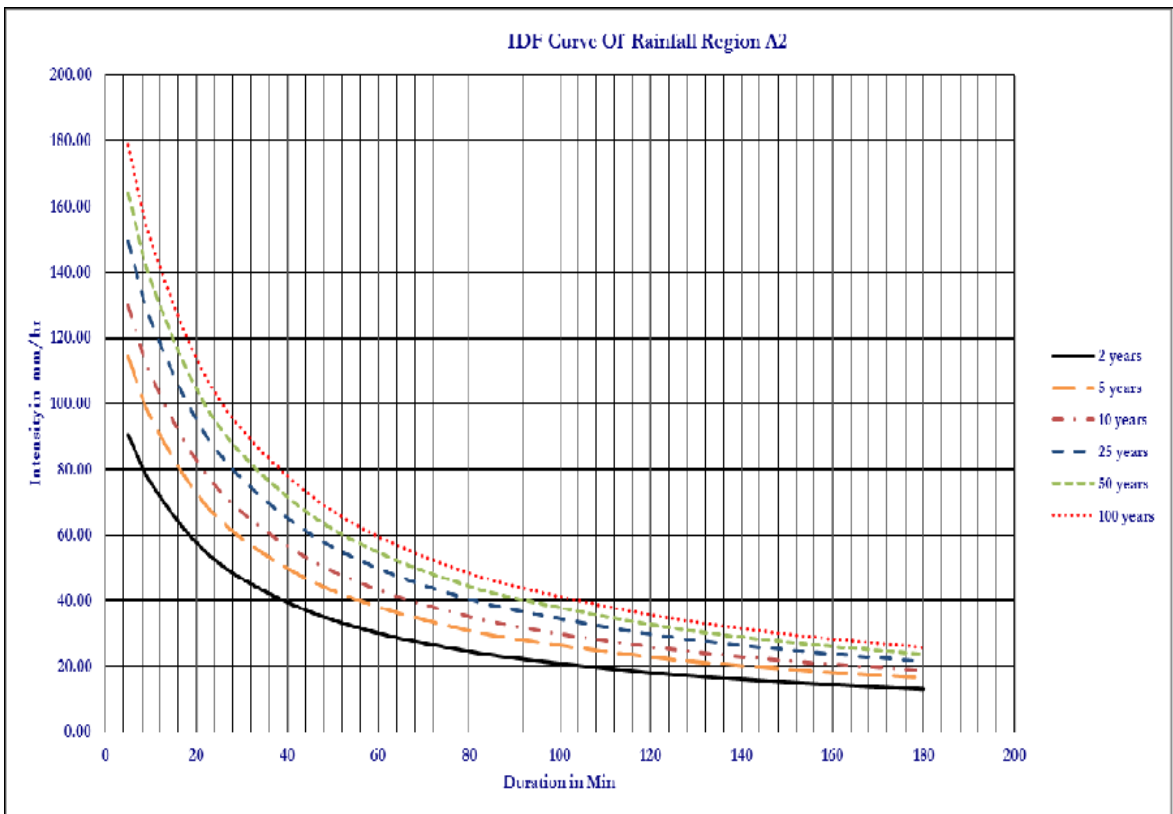


Figure 3.4: IDF Curve of Rainfall Region A2 (ERA, 2013)

### **3.2.2.2. Data processing**

Data processing is a method of collecting raw data to translate into usable information. It is a process of collecting raw data, filtering, sorting, analyzing, storing, and presenting in a readable format. There are different data processing types such as manual data processing, mechanical data processing, and electronic data processing. The data processing carried out for this thesis was mechanical data processing. That means the data was processed using devices and machines such as calculators and computers. Stages of data processing carried out for this research were:

**Data collection:** Data collection is the first step in the data processing. The data was collected from concerning bodies and by field surveying.

**Data preparation:** in this stage raw data was cleaned and organized i.e. data redundant is eliminated, incomplete data was filed and incorrect data was removed to create high-quality data. Then clean data was entered into its destination and translated that it can understand and begins to take the form of usable information. Then data is interpreted and begins to analysis to use for the research (Nikita, 2020).

### **3.2.2.3. Thiessen polygon and missing rainfall data**

The rainfall data is the basic input for many hydrological models because it affects the resulting peak runoff. It plays an important role in the hydrologic cycle which controls our water supplies and water disasters. By knowing the nature and characteristics of rainfall, we can conceptualize and predict its effects on runoff, infiltration, and water yield. Since rainfall is never evenly distributed over the area of study due to the topographic variability of the catchment areas, it is preferable to have as many rainfall stations as possible to estimate the areal rainfalls which represent the actual rainfalls over the basin (Wisuwat, 2009). For the study area, rainfall data were obtained from the National Meteorological Agency (NMA) of Ethiopia which has records of 24-hours daily rainfall for different durations of records from (1987-2018). But incompleteness of rainfall data was one problem that hinders the quality of data. To solve this problem additional three rain gauge stations were used for this thesis to fill unrecorded rainfall data of the study area. These additional stations were stations near to the study area which includes D/markos, Yetnora, and Abaysheleko stations.

### 3.2.2.4. Areal rainfall and Thiessen polygon

In the absence of information on the true probability distribution of aerial precipitation, point precipitation estimates area usually extended to develop an average precipitation depth over an area (Chow, 1988). It is necessary to have as many rainfall stations as possible to estimate the areal rainfall which represents the actual rainfall over the basin. Several areal rainfall estimation techniques are currently used for averaging rainfall depths collected at stations. These include Isohyet, Arithmetic mean, and Thiessen polygon techniques are conventional techniques that are usually applied to estimate the areal rainfall over the entire basin. For this thesis, Thiessen polygon method was used to relate the four stations and to calculate missing rainfall data in the study area.

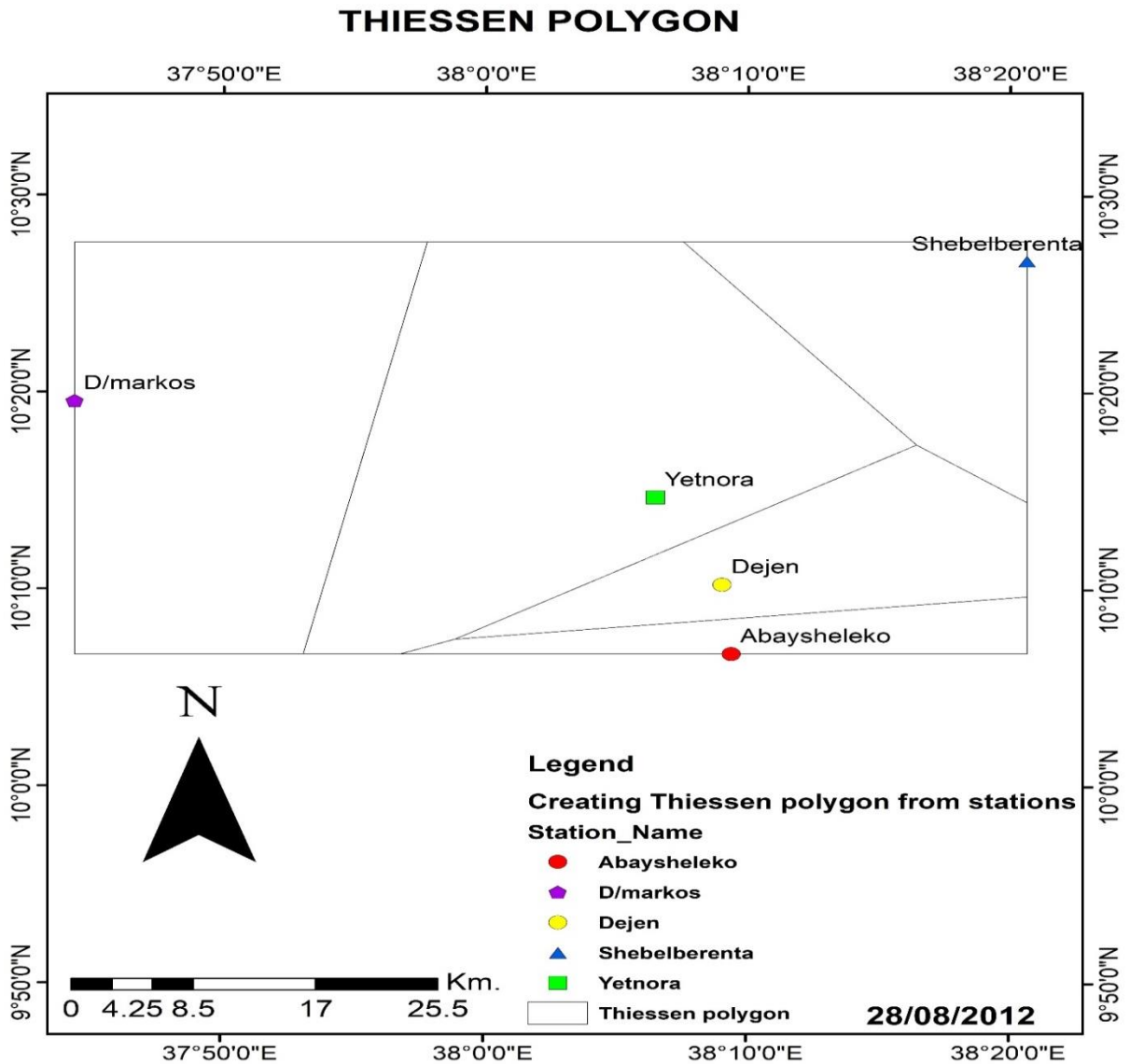


Figure 3.5: Thiessen polygon by ArcGIS

**3.2.2.5. Estimation of missing rainfall data**

Point precipitation observation from precipitation gage may have a short break in the record because of instrument failure, and absence of observer due to these reasons the rainfall data in the stations are mostly missing. To estimate missing records of rainfall data by using neighboring stations the following methods are commonly used techniques. These include the Simple Arithmetic method, Normal Ratio method, and Inverse distance method, linear programming method, and Modified normal ratio method. For this study normal ratio method was selected to fill missing rainfall data. This method is used if any surrounding gauges have normal annual precipitation exceeding 10% of the considered gauge (Muluken, 2020).

$$P_x = \frac{1}{m} \left( P_1 \frac{N_x}{N_1} + P_2 \frac{N_x}{N_2} + P_3 \frac{N_x}{N_3} + \dots + P_m \frac{N_x}{N_m} \right) \dots \dots \dots 3.1$$

Where:

P<sub>x</sub> - missing rainfall data (daily, monthly, or yearly).

P<sub>1</sub>, P<sub>2</sub>, P<sub>3</sub>- area rainfall data at nearest stations (daily, monthly, or yearly)

N<sub>x</sub> - mean annual rainfall at the missed station.

N<sub>1</sub>, N<sub>2</sub>, N<sub>3</sub> - mean annual rainfall at the different nearest station.

m- Number of stations.

As shown from Figure 3.6 below rainfall of Dejen Town and surrounding varies season to season, year to year, and the rainfall never distributed evenly due to many factors such as deforestation, topographic variability of catchment areas, and desertification.

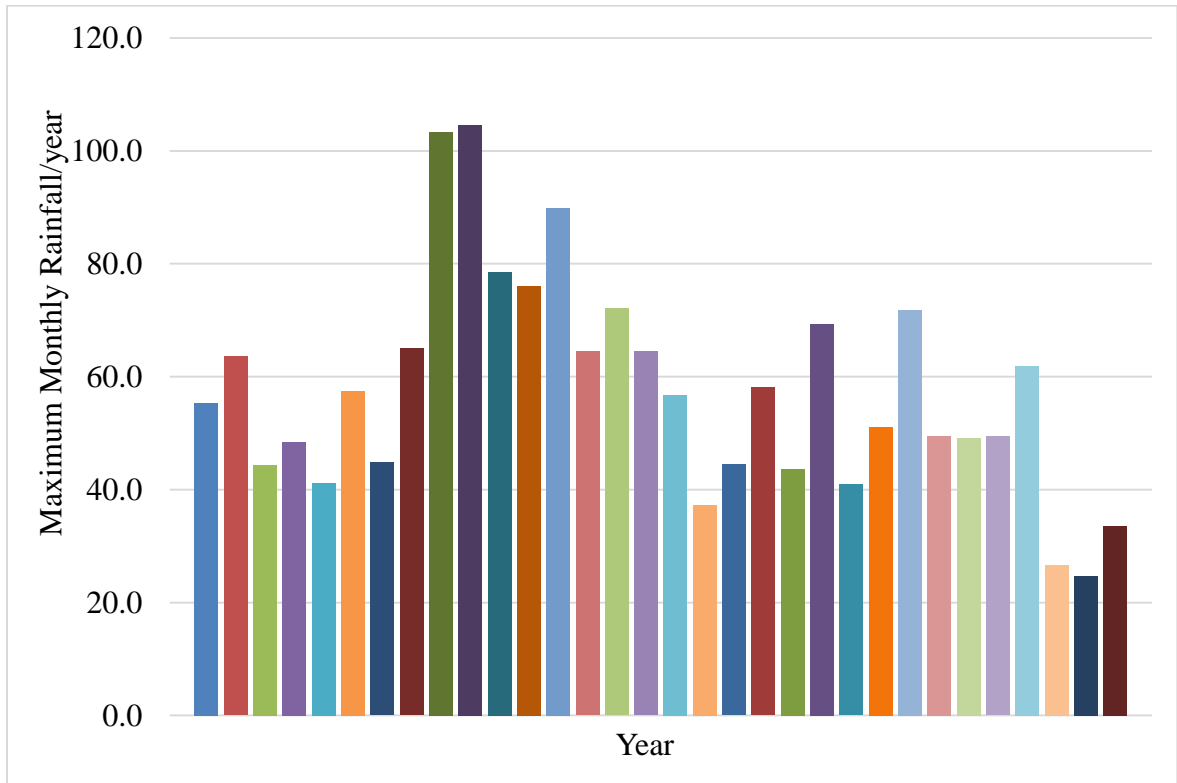


Figure 3.6: Maximum monthly rainfall/year of Dejen from 1987-2018

### 3.2.2.6. Checking consistency of data

Checking the consistency of meteorological records is an essential tool before taking data for analysis purposes. Double mass curve analysis is the method that is used to check the inconsistency of gaging records. It is determined by plotting the cumulative values of the observed time series of the stations for which consistency needs to be checked on the y-axis versus the cumulative values of the observed time series of average neighboring stations on the x-axis. For this thesis double mass curve analyses were used to adjust the inconsistency of gaged data. A breaking in the slope of the curve would indicate the conditions have changed that location was inconsistent and should be adjusted to consistent values as shown from Appendix 1, Table 1b cumulative rainfall of average station versus cumulative rainfall of each station.

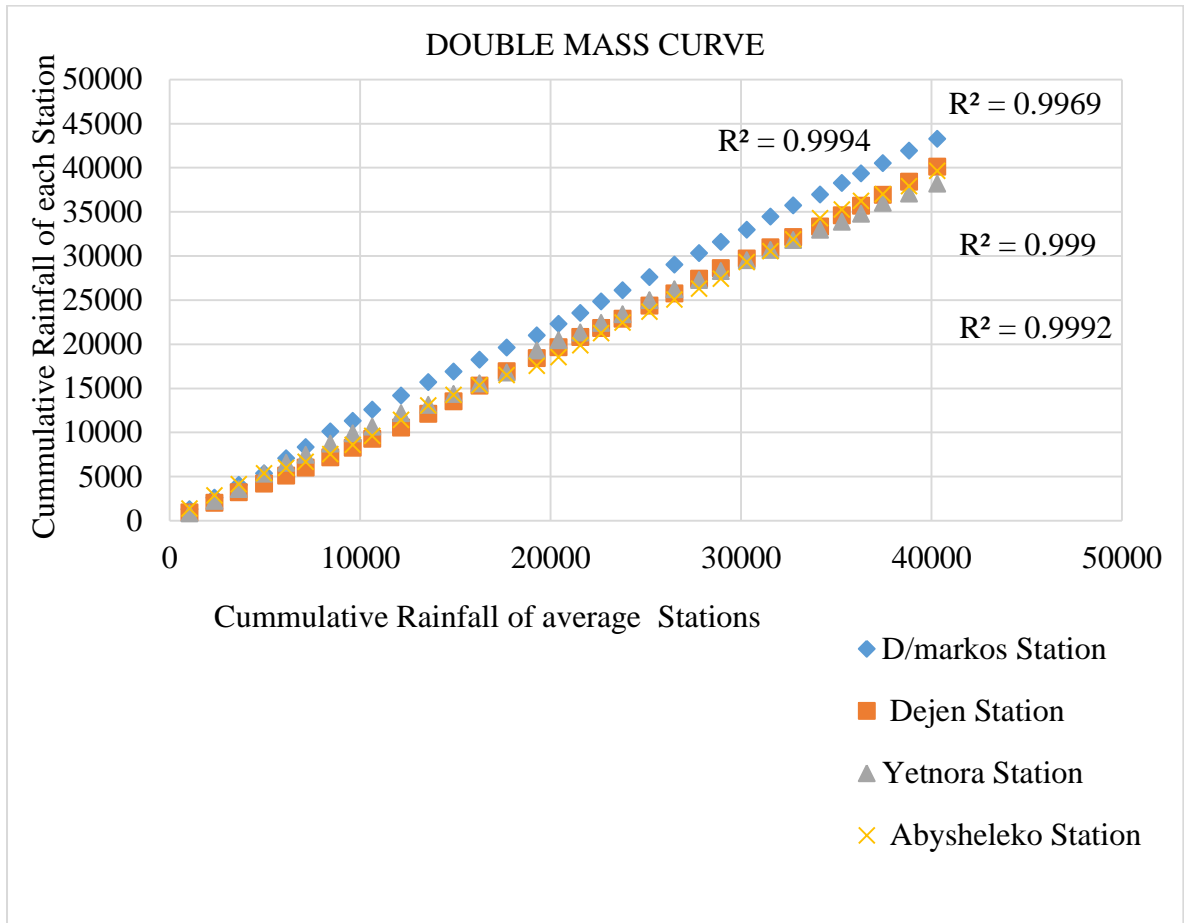


Figure 3.7: Double mass curve to check consistency of data of study area

### 3.2.2.7. Analyses of rainfall data

Data analysis is the process of systematically applying statistical and logical techniques to describe and illustrate, condense and recap, and evaluate data. The purpose of data analysis is to extract useful information from data and taking the decision based on data analysis. The rainfall data is analyzed by different methods such as Log Pearson-III, Gumbel, General Pareto and Log normal distributions.

#### 3.2.2.7.1. Methods of a probability distribution

Flood frequency analysis is an analytical technique that involves in the estimation of how often a specified event will occur. The analysis involves using observed annual peak flow data to calculate statistical information such as mean, standard deviation, and Skewness and recurrence interval. Reliable flood frequency estimates are vital for flood plain management for protection of public infrastructure; minimize flood related cost to government and private enterprises, for designing and locating hydraulic structures and

assessing hazards related to the development of flood plains and epidemic control (Raihan *et al.*, 2014).

The Empirical formula and unit hydrograph methods are the prediction methods of flood flows. Some of the commonly used frequency distribution functions of extreme flood values are; Gumbel’s extreme-value distribution, General Pareto distribution (peak over Threshold method), Log-Pearson Type III distribution, and Log-Norma distribution method (Subramanya, 2008).

**3.2.2.7.2. Gumbel’s extreme-value distribution**

In planning and designing water resources projects, engineers and planners are often interested to determine the magnitude and frequency of floods that will occur in the project areas. Besides, the rational method, unit hydrograph method, and rainfall-runoff models method, frequency analysis is one of the main techniques used to define the relationship between the magnitude of an event and the frequency with which that event is exceeded. Flood frequency analysis is the estimation of how often a specified event will occur. It is the most commonly used by engineers and hydrologists worldwide and consists of estimating flood peak quantities for a set of non-exceedance probabilities.

The equation for Gumbel’s distribution with return period T is given as

$$X_T = \bar{X} + K * \sigma_x \dots\dots\dots 3.2$$

$X_T$  = annual maximum mean flow of T years return period

$\sigma_x$  = standard deviation of the sample size

k = frequency factor,  $K = \frac{Y_t - \bar{y}_n}{S_n}$

$Y_t$  = reduced variant,  $Y_t = -\text{Ln}(\text{Ln}(\frac{T}{T-1})) \dots\dots\dots 3.3$

The value of  $\bar{Y}_n$  and  $S_n$  is selected from Gumbel’s Extreme value distribution table considered depending on the sample size (Nirman, 2017).

**3.2.2.7.3. Log-Pearson Type III distribution**

$$Z = \log X \dots\dots\dots 3.4$$

$$Z_T = \bar{Z} + K_z * \delta_z \dots\dots\dots 3.5$$

Where:  $K_z$  = frequency factor which is a function of T and the coefficient of Skewness,  $C_s$

$K_z = f(C_s, T)$  is given in the table.

$$\delta z = \text{Standard deviation of the Z variety sample, } \delta z = \sqrt{\frac{\sum (Z_i - \bar{z})^2}{N-1}}$$

Cs=coefficient of skew of variate z.

$$C_s = \frac{N \sum (Y_o - \bar{Y})^3}{(N-1)(N-2)(\delta z)^3}$$

$$X_T = 10^{z_T} \dots \dots \dots 3.6$$

### 3.2.2.8. Goodness of fit tests

The best fit probability distributions were identified based on the results obtained from different tests such as the Hydrognomon 4 software test and the Goodness of fit tests.

#### ➤ Hydrognomon 4 software

Hydrognomon is a software tool for the processing of hydrological data. Data are imported through standard text files, spreadsheets, or by typing. This program includes common hydrological applications such as evapotranspiration modeling, stage-discharge analysis, homogeneity tests, areal integration of point data series, processing of hydrological data, as well as lumped hydrological modeling with automatic calibration facilities (EGU, 2010).

Table 3.2: Goodness of Fit test Distribution Method for Maximum Rainfall of Dejen town by Hydro gnomon

Kolmogorov-Smirnov test for:All data	a=1%	a=5%	a=10%	Attained a	DMax
Normal	ACCEPT	ACCEPT	ACCEPT	93.5341%	0.08855
Normal (L-Moments)	ACCEPT	ACCEPT	ACCEPT	92.7979%	0.08994
LogNormal	ACCEPT	ACCEPT	ACCEPT	99.9619%	0.05626
Galton	ACCEPT	ACCEPT	ACCEPT	99.9304%	0.05839
Exponential	ACCEPT	ACCEPT	ACCEPT	75.6501%	0.11251
Exponential (L-Moments)	ACCEPT	ACCEPT	ACCEPT	76.9026%	0.11113
Gamma	ACCEPT	ACCEPT	ACCEPT	99.9612%	0.05632
Pearson III	ACCEPT	ACCEPT	ACCEPT	99.9739%	0.05503
Log Pearson III	ACCEPT	ACCEPT	ACCEPT	98.8854%	0.07233
EV1-Max (Gumbel)	ACCEPT	ACCEPT	ACCEPT	99.6526%	0.06542
EV2-Max	ACCEPT	ACCEPT	ACCEPT	62.4017%	0.12655
EV1-Min (Gumbel)	ACCEPT	ACCEPT	ACCEPT	44.6520%	0.14611
EV3-Min (Weibull)	ACCEPT	ACCEPT	ACCEPT	95.8881%	0.08337
GEV-Max	ACCEPT	ACCEPT	ACCEPT	99.9714%	0.05532
GEV-Min	ACCEPT	ACCEPT	ACCEPT	99.9593%	0.05649
Pareto	REJECT	REJECT	REJECT	0.03941%	0.35876
GEV-Max (L-Moments)	ACCEPT	ACCEPT	ACCEPT	99.9969%	0.04923
GEV-Min (L-Moments)	ACCEPT	ACCEPT	ACCEPT	99.9634%	0.05613
EV1-Max (Gumbel, L-Moments)	ACCEPT	ACCEPT	ACCEPT	99.8160%	0.06236
EV2-Max (L-Moments)	ACCEPT	ACCEPT	ACCEPT	67.3909%	0.12132
EV1-Min (Gumbel, L-Moments)	ACCEPT	ACCEPT	ACCEPT	48.3127%	0.14184
EV3-Min (Weibull, L-Moments)	ACCEPT	ACCEPT	ACCEPT	94.7856%	0.08597
Pareto (L-Moments)	REJECT	REJECT	REJECT	0.66405%	0.29227
GEV-Max (kappa specified)	ACCEPT	ACCEPT	ACCEPT	78.4721%	0.10937
GEV-Min (kappa specified)	ACCEPT	ACCEPT	ACCEPT	77.0127%	0.11100
GEV-Max (kappa specified, L-Moments)	ACCEPT	ACCEPT	ACCEPT	86.8880%	0.09908
GEV-Min (kappa specified, L-Moments)	ACCEPT	ACCEPT	ACCEPT	76.4437%	0.11163

As shown above, checking goodness of fit test by Hydrognomon 4 for identification of probability distribution General Pareto distribution is rejected. This is due to the lack of arrangement of data as the Pareto distribution fits. So, below the data is arranged as Pareto distribution fits and goodness of fit test is done by Easy fit 5.6.

➤ **General Pareto distribution**

The regional analysis of the Peak-Over-Threshold method series can be achieved through different means. Among these generalized least square regression model that explicitly accounts for interring site correlation to describe the variability of POT parameters using physiographic and climatic characteristics. The advantage of POT approach is more than one extreme value can be considered each year while only the most extreme value is kept using the annual maximum approach. The POT approach improves the sampling of extreme events (Alain *et al.*, 2013).

❖ **Threshold estimation and construction of POT series**

Various methods can be found to guide the choice of threshold. These include the Hill plot, mean excess plot or mean residual life plot, and bootstrap method on mean square error. The mean excess plot aims at estimating the threshold value based on a linear relationship between the threshold value and the mean excess of the threshold. This method identifies clusters within the series and only keeps the maximum value within each cluster (Alain *et al.*, 2013).

The plot of mean residual life (MRL) is widely used method of determining the threshold value from a time series. The MRL plot displays the mean excess against a range of different threshold values (Tewodros and A., 2018). This is done by arranging the data as the Pareto distribution fits and the goodness of fit test was done by Easy fit 5.6. From n observed data points of a variable X given as X1, X2....Xn new time series can be generated as;

$$Y=f(x/k, \sigma, \mu) = \left(\frac{1}{\sigma}\right) * \left(1 + k \left(\frac{x-\mu}{\sigma}\right)^{-1-1/k}\right) \dots\dots\dots 3.7$$



Figure 3.8: Plot of Mean Residual Life

There is a sort linear relationship between mean residual and threshold starting from 38mm. The selected data is minimum data of 38mm and selected at the point where mean residual vs threshold graph is linear.

➤ **Goodness of fit test by Easy fit 5.6**

For the selection of a particular distribution representing a rainfall series, it is essential to find the best fit method of frequency analysis. The three different most widely used Goodness of fit tests such as Kolmogorov-Smirnov, Anderson-Darling, and Chi-Square tests, are implemented to select the most appropriate model.

**Kolmogorov-Smirnov Test Statistic**-the Kolmogorov-Smirnov (k-s) goodness of fit test compares a hypothetical or fitted cumulative distribution function with empirical cumulative distribution function to assess fit. It measures the greatest inconsistency between the observed and hypothesized distribution.

**Anderson-Darling Test**-Anderson-Darling Test emphasizes inconsistencies in both tails of the distribution, and that is often prime importance in hydrologic frequency analysis. Lower statistics means a better fit when comparing different distributions.

**Chi-Squared Test**-Chi-Square goodness of fit test is a non-parametric test that is used to find out how the observed value of a given phenomenon is significantly different from the

expected value. In the Chi-Square goodness of fit test, the term goodness of fit is used to compare the observed sample distribution with the expected probability distribution. It determines how well theoretical distribution fits the empirical distribution.

Easy fit software employs the method of moments for the Gumbel, Log-Pearson-III distributions, General Pareto Distribution, and the maximum likelihood for the Log-Normal distribution for estimation of parameters. The estimated parameters for different parametric distributions were fitted to 24-hr series at all the stations. The test statistics were computed and probability distributions were ranked based on the lowest values of the test statistics (Sundar Sil *et al.*, 2015).

Table 3.3: Rank of probability distributions resulted from Easy fit 5.6

No.	Distribution	Kolmogorov Smirnov		Anderson Darling		Chi-Squared	
		Statistic	Rank	Statistic	Rank	Statistic	Rank
1	Gen. Pareto	0.05592	1	0.64848	1	8.6285	11
2	Log-Pearson 3	0.09554	18	1.8365	14	3.1057	15
3	Gumbel Max	0.14394	30	3.4854	17	10.694	12
4	Lognormal	0.13975	27	4.6105	24	15.434	24

### Testing for Outliers

When the probabilities of hydrologic data are plotted, commonly one or two events appear to be from different populations since they lie far from the line that represents other points. Outlier values can arise from three different causes.

1. An error in measurement or recording
2. An observation from a different population from that of most of the data, such as a flood triggered by a dam break rather than by rainfall
3. An occasional event from a single population that is rather skewed.

A common application is to eliminate outliers by throwing them away before constructing the database.

There are different methods of outlier tests. These include the z-Score method, Box Plot Method, Quality control method, Grupps-Beck Test, and Stedinger-Test.

**Stedinger-Test:** with this method higher and lower thresholds are identified by the following formulas,

$$X_H = \exp(Y_{\text{mean}} + KN * Sy) \dots \dots \dots 3.8$$

$$X_L = \exp(Y_{\text{mean}} - KN * Sy) \dots \dots \dots 3.9$$

Where: Y-is logarithm of data(Y=lnX)

Y<sub>mean</sub>=is the mean

Sy=is standard deviation of the logarithms

Critical value KN accurate approximation related to the sample size N (5≤N≤150)

$$KN = -0.9043 + 3.345\sqrt{(\log N)} - 0.4046 \log N \text{ (Omer L., 2017).}$$

N=137, Y<sub>m</sub> = mean of logarithms=3.89, S<sub>y</sub> =standard deviation of logarithm=0.23

$$\log N = \log 137 = 2.14, \sqrt{(\log N)} = 1.462$$

$$KN = -0.9043 + 3.345 * 1.462 - 0.4046 * 2.136721$$

$$KN = 3.5815$$

$$Y_{\text{mean}} + KN * Sy = 3.89 + 3.5815 * 0.23 = 4.714$$

$$X_H = \exp(Y_{\text{mean}} + KN * Sy)$$

$$X_H = \exp(4.714) = 111.5 = \text{Higher threshold}$$

$$Y_{\text{mean}} - KN * Sy = 3.89 - 3.5815 * 0.23 = 3.07$$

$$X_L = \exp(Y_{\text{mean}} - KN * Sy) = \exp(3.07)$$

$$X_L = 21.54 = \text{lower threshold}$$

Therefore, the smallest datum is 21.54mm and the highest datum is 111.5mm. Since there is no data that is lower than 21.54mm and greater than 111.5mm. The smallest data used is 38mm and the highest data is 104.6mm this means the data used satisfies the given condition.

### 3.2.2.9. Design rainfall of shorter duration

The rainfall depths obtained from the gauging station were 24-hour duration depth. However, when short time duration rainfall data were not available, the intensity of a short time rainfall of long time rainfall would be calculated using a reduction formula. According to the Ethiopian Roads Authority (ERA, 2013), the following equations were used to calculate the shorter duration of rainfall from 24-hour duration.

$$RR_t = \frac{t}{24} \frac{(b+24)^n}{(b+t)^n} \dots \dots \dots 3.10$$

Where:

RR<sub>t</sub> = Rainfall depth ratio R<sub>t</sub>: R<sub>24</sub>

R<sub>t</sub> = Rainfall depth in a given duration t

R<sub>24</sub> = 24-hr rainfall depth

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

Rearranging the above equation:

$$Rt = \frac{t(b+24)^n}{24(b+t)^n} * R24 \dots\dots\dots 3.11$$

Rainfall intensity in (mm/hr) is given by

$$I = \frac{Rt}{t} \dots\dots\dots 3.12$$

$$I = \frac{R24*(b+24)^n}{24(b+t)^n} \dots\dots\dots 3.13$$

As ERA manual suggestion  $b = 0.3$  and  $n = 0.92$  for tabulating results of rainfall durations 10, 20, 30 .....180 minutes.

### 3.2.2.10. Estimation of peak runoff and existing hydraulic capacity

#### ➤ Estimation of peak runoff using Rational method

The Rational formula estimates the peak rate of runoff at any location in the catchment area as a function of the catchment area, runoff coefficient, and rainfall intensity for the duration equal to the time of concentration. This method estimates the peak runoff rate for small urban and rural watersheds of less than 50 hectares (ERA, 2013). The main input variables of the rational method are rainfall intensity, weighted runoff coefficient of the catchment, time of concentration, and catchment area.

$$Q_p = \frac{CIA}{360} = 0.00278CIA \dots\dots\dots 3.14$$

Where:  $Q_p$  = catchment peak runoff ( $m^3/se$ ),  $C$  = runoff coefficient of the catchment

$I$  = rainfall intensity in (mm/hr),  $A$  = catchment area in (ha)

The following procedures summarize the rational method for estimating peak discharge

1. Determine catchment area
2. Determine the time of concentration
3. Assure consistency with the assumptions and limitations for application of the rational method
4. Select the appropriate runoff coefficient and
5. Compute the peak discharge for the watershed for a desired frequency

**Runoff coefficient (C):** Runoff coefficient is a dimensionless factor used to convert amount of rainfall to runoff. It is the function of the land use land cover of the study area and used as input for peak discharge estimation. The more the surface is impervious the higher the runoff would be the infiltration decreases. The rainfall intensity directly affects

the runoff coefficient. Vegetation cover reduces the impact of a raindrop on the ground and intercepts some of the rain on its leaves and branches letting them evaporate and decrease the runoff coefficient. A weighting method is employed to obtain the representative runoff coefficient (Ven Te Chow *et al.*, 2012).

$$C_w = \frac{A_1 \cdot C_1 + A_2 \cdot C_2 + \dots + A_n \cdot C_n}{A_1 + A_2 + \dots + A_n} \dots\dots\dots 3.15$$

Where:  $C_w$ =weighted runoff coefficient,  $C_1, C_2 \dots C_n$ =coefficient of runoff for parts of the drainage area.

$A_1, A_2, A_n$ = drainage area with different runoff coefficient

**Time of concentration ( $T_c$ ):** is the time required for runoff to flow from the most remote point of the basin to the point of interest. In a Rational method, it was used to determine the rainfall duration which would result in maximum runoff. The required design rainfall intensity was established from the IDF curve for the required recurrence interval (ERA, 2002).

$$T_c = T_{c \text{ inlet}} + T_t \dots\dots\dots 3.16$$

Where:  $T_c$ =time of concentration in (minutes)

$T_t$ = travel time (hr)

Inlet time for time of concentration is estimated using Federal Aviation Administration (1970) methods.

$$T_{ci} = \frac{3.64(1.1 - C_w) \left(\frac{L}{1000}\right)^{0.83}}{H^{0.33}} \dots\dots\dots 3.17$$

$T_{ci}$ =time of concentration inlet (hrs.)

$C_w$ =weighted runoff coefficient

$L$ =flow length from the remote point to the point of interest (km).

$H$ = elevation difference between the upstream and downstream of the basin (m)

➤ **Travel time:** water moves through the catchment area as sheet flow, shallow concentrated flow, and open channel flow.

✚ **Open channel flow time:** can be estimated from the hydraulic properties of the conduit or channel.

$$T_t = \frac{L}{3600V} \dots\dots\dots 3.18$$

Where:  $T_t$ =travel time in hr

$L$ =length of the main drainage canal (m)

$V$ =flow velocity of open channel (m/se)

$$V = \frac{1}{n} * R^{\frac{2}{3}} * S^{1/2} \dots\dots\dots 3.19$$

Where: V = the flow velocity (m/s),

R = the hydraulic radius (m),

S = slope the sub-catchment (m/m) and

n = the Manning roughness coefficient for open channels.

**Rainfall intensity (I):** is the average rainfall rate in mm/hr or mm/min for particular drainage basin drainage duration is equals to the time of concentration for the drainage area under consideration. Rainfall intensity is the average rainfall rate for a specific rainfall duration and selected frequency.

➤ **Manning’s formula**

The existing drainage systems and hydraulic capacity of existing drainage structures were checked and calculated from data obtained from Dejen Town municipality and data collected from municipality. Manning’s formula was used for calculating the existing discharge capacity of each drainage canal and the canal’s cross-sectional area, wetted perimeter, and hydraulic radius for the flow of specific depth.

$$Q = \frac{1}{n} * A * R^{2/3} * S^{1/2} \dots\dots\dots 3.20$$

Where: Q=flow rate passing through the canal in m<sup>3</sup>/s

A= Cross-sectional area of flow in m<sup>2</sup>, S=bottom slope of the canal m/m

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

n = manning’s roughness coefficient, R=hydraulic radius.

**3.2.3. Simulation of SWMM and predicting water level at links**

The Storm Water Management Model is a distributed discrete-time simulation model. It computes new values of its state variables over a sequence of time steps, where at each time step the system is subjected to a new set of external inputs. As its state variables are updated, other output variables of interest are computed and reported. Regarding the constant parameter vector P, the stormwater management model contains over 150 different user-supplied constants and coefficients within its collection of process models. Most of these are either physical dimension such as land area, pipe diameters, inverted elevation, or quantities that can be obtained from field observations like percent impervious cover, laboratory testing like various soil properties, or previously published data tables like pipe roughness based on pipe material (A.Rossman, 2016).

### 3.2.3.1. Storm Water Management Model (SWMM) modeling objects

#### ➤ Hydrology

**Rain Gage:** is source of precipitation data to one or more sub-catchments.

**Sub-catchment:** is a land parcel that receives precipitation associated with a rain gage and generates runoff that flows into a drainage system node or another sub-catchment.

#### ➤ Hydraulics

**Junction:** is a point in the conveyance system where conduits connect to one another with a negligible storage volume e.g. manholes, pipe fittings, or stream junctions.

**Outfall:** is an end point of the conveyance system where water is discharged to a receptor with known water surface elevation.

**Conduit:** is a channel or pipe that conveys water from one conveyance system to another.

**Time Series:** is a tabular function that describes how a quantity varies with time (A.Rossman, 2016).

### 3.2.3.2. Project setup and input parameters for simulation of SWMM

The first task to simulate the SWMM model is preparing sub-catchments of the project. The prepared sub-catchments were then opened in SWMM software and the default values were adjusted. After adjusting default values Junctions, outfall, Nodes, conduits, and Rain gage were added. Then the input parameters listed above were prepared and inserted into SWMM model. Some of the parameters required for simulation of SWMM were Manning's n, Per-DS, Imp-DS, N-Perv, N-Imperv, and soil infiltration. The other inputs of the SWMM model were width, slope, inverted elevation, depth of the canal, and Time series.

Table 3.4: Input parameters for the SWMM model

Parameter	Type	Symbol	Value
Manning's n	Overland flow	Imp-n	0.005-0.05
	Conduit flow	Per-n	0.05-0.5
	Open channel flow	Con-n	0.4
Depression storage	Per-DS		2.5-7.6(mm)
	Imp-DS		1.3-2.5(mm)
		Max.infi.rate	76.2(mm/hr)
		Min.infil.rate	3.18(mm/hr)
Soil infiltration	Horton infiltration	Decay constant	3.12 hr

Source: (Kong et al, 2017)

Table 3.5: Description of SWMM input parameters

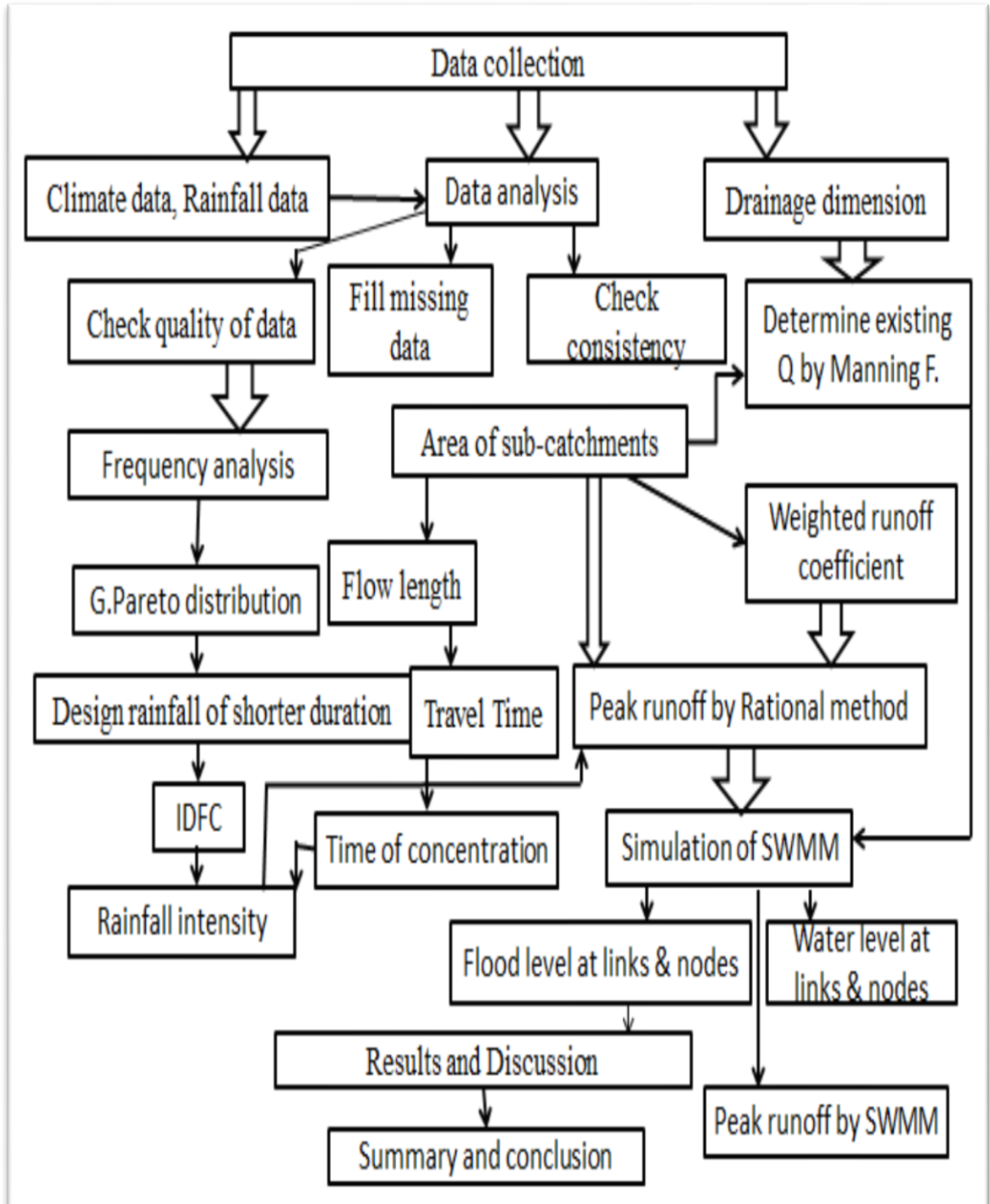
Parameter	Description
N-Imperv	Manning's roughness coefficient for impervious areas
N-Perv	Manning's roughness coefficient for pervious areas
Dstore-Imperv	Depth of surface depression storage in impervious areas (mm)
Dstore-Perv	Depth of surface depression storage in pervious areas (mm)

#### ❖ Soil type of Study Area

Dejen Town consists of three types of soils. These soils include chromic Vertisols (98.73 %), Eutric cambisols (1.26%), and Eutric Nitisols (0.01%) of typical hydrologic soil groups. These soil types were identified by using ArcGIS 10.3 and FAO soil classification of Ethiopia 2007. According to the hydrological soil group of Ethiopia, a soil to resist the occurrence of flooding must have low runoff potential and high infiltration rates even when thoroughly wetted, they must consist chiefly of deep, well to excessively drained sand or gravel and must have a high rate of water transmission (greater than 0.30 in/hr). Hydrologic soil group of Ethiopia groups chromic Vertisols under group D and Nitisols and Leptosols under group B. Dejen Town is mostly covered with Chromic Vertisols that are grouped under hydraulic soil group D that have high runoff potential due to its low infiltration rate with shallow soils over nearly impervious material. Due to this, there is the occurrence of high runoff potential in the Town that requires control of runoff and flooding

in the Town by the maintenance of existing ditches and designing drainage structures according to the peak flood that will occur in the Town.

### 3.3. Research Framework and Procedures



## **4. RESULTS AND DISCUSSIONS**

### **4.1. Assessment of Current Situation of Existing Drainage System in Study Area**

The existing drainage system of Dejen Town has a lot of problems that hinder the functionality of the structures. The drainage systems were not well-connected each other to pass flood to proper outfall, the drainage structures cannot carry a large amount of flood due to its improper dimensions and causes over flooding, most ditches do not have a proper slope to pass water through them, also in some areas there were no drainage systems provided at all, and some of the existing drainage ditches and manholes have been sediment and blocked by sand, rubbish materials, waste disposals, and plastic materials. The dominant problems that exist in the area were sedimentation of ditches and deposition of refuse, deterioration of roads, overtopping of flood due to sedimentation of ditches, swamp formation at lower reaches of the catchments due to inappropriate outlets, and stagnation of wastes. Also, as shown during the site visit, and from drainage data taken from Dejen Town Municipality, existing drainage systems of the study area was insufficient, drainage nodes were not well connected to convey floods to the next drainage system, and storm runoff and storm drains were not well-connected to convey and discharge stormwater into ditches and sewer systems. The other problem is most of the drainage structures were open ditches that were exposed to different failures. The topography of Dejen Town was generally suitable for the installation and construction of drainage facilities. However, drainage services in the Town were considered inadequate in quality and quantity or coverage.

#### **4.1.1. Existing drainage conditions of study Area**

During performance assessment of stormwater drainage system, first current condition of drainage structures and their coverage or amount of drainage structures must be studied. According to the data collected during a site visit and the data taken from Dejen Town municipality the major problem in drainage structures of the study area include joint damage, the collapse of the wall, floor damage due to aging, sedimentation and deposition, and overall deterioration of the structures. The drainage condition of the existing drainage system of the study area was checked as shown from Figures and appendix 2, Table 2d.

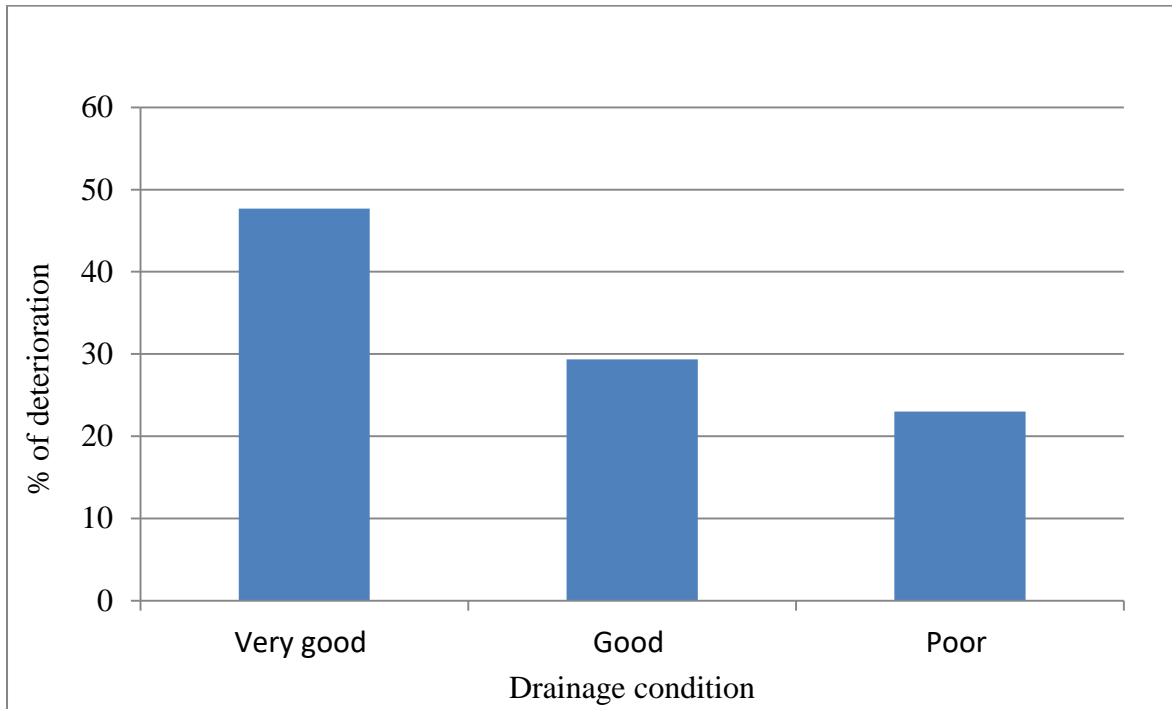


Figure 4.1: Drainage condition from CBE Abaysheleko branch to Fasiledes bridge outfall  
 As presented from Figure 4.1 above, 48% of drainage around CBE Abaysheleko branch to Fasiledes Bridge is very good, 29% is good and 23% is poor. That shows the flood around this area cannot discharge nearby outlets safely in all drainage canals. Even if the percent of poor drainage is less in number it may cause damage on the road pavements during over flooding due to its poor drainage system.

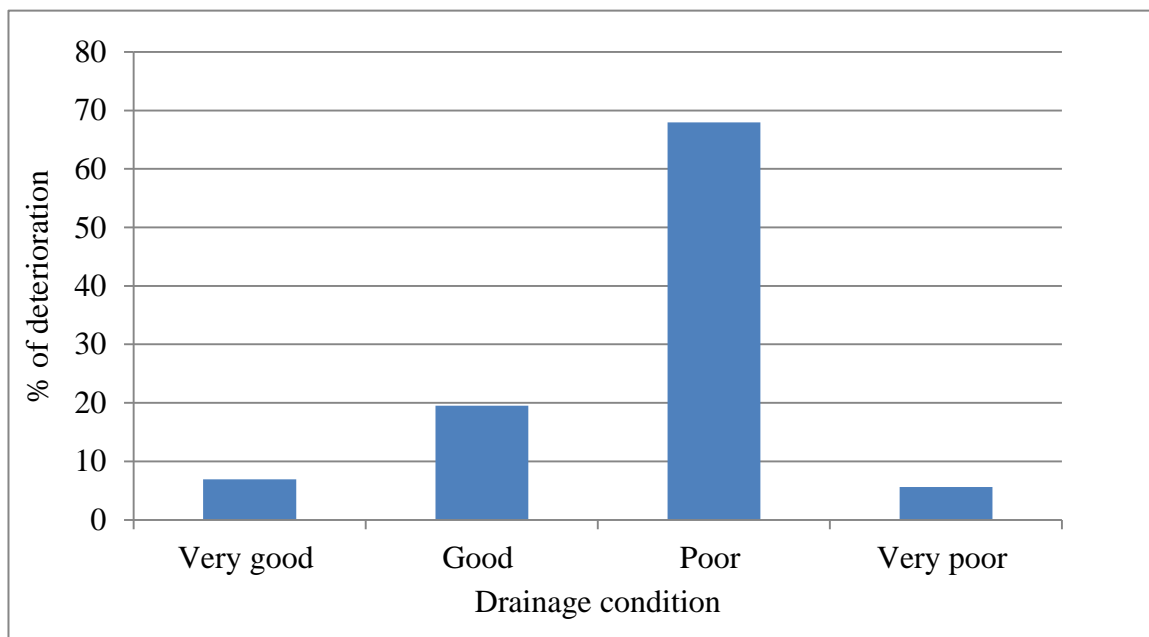


Figure 4.2: Drainage condition from Demis Hotel to Dejen 02 primary school

As shown from Figure 4.2 above, 68% of the drainage system is poor, 7% is very good, 20% is good and 6% is very poor. That means most of the drainage system is poor and cannot discharge a flood nearby outlets. Due to these poor drainage structures, the drainage canal cannot carry flood and over flooding occurred. These damages were due to collapse of wall, sedimentation and deposition, clogging of drainage structures due to disposal of plastic materials.

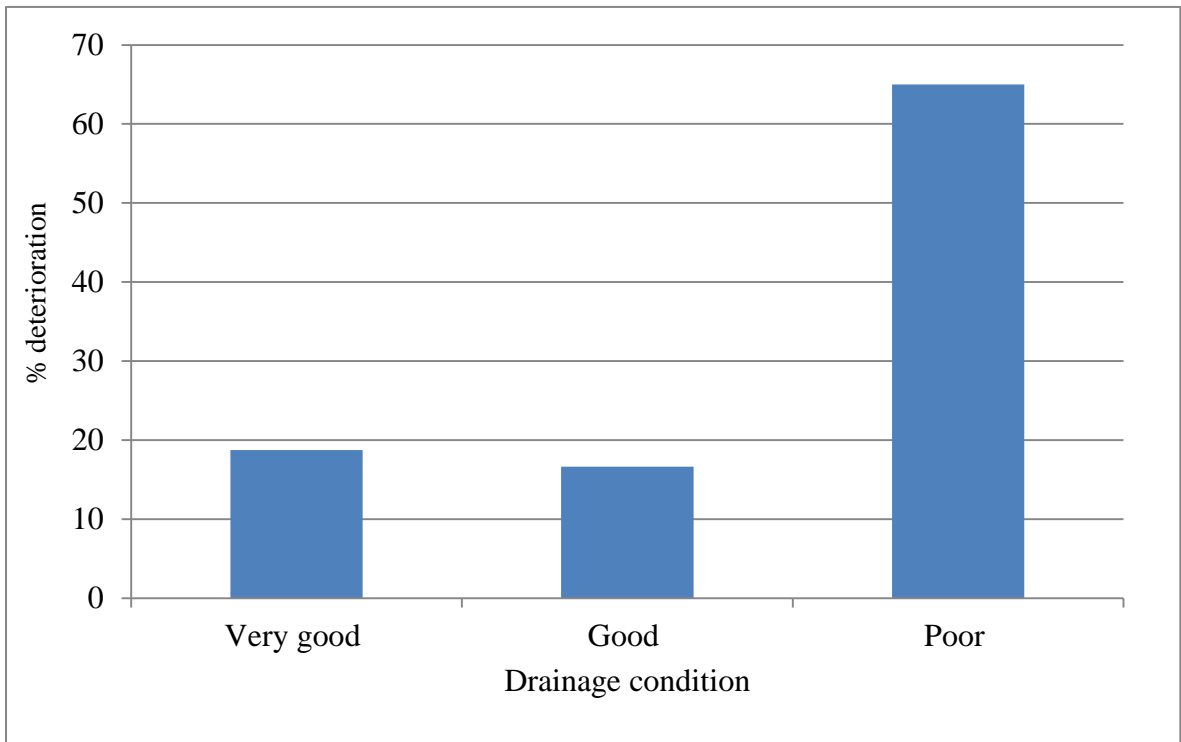


Figure 4.3: Drainage condition around Seble Hotel

According to Figure 4.3 above, from Seble hotel to Dejen kindergarten 19% of drainage structures are very good, 17% is good, 6% are moderate and 59% are poor. This shows flooding is more in this area because the drainage structure cannot carry flood and discharge into outlets. So the drainage system in this area cannot serve its function we and flooding will occur.



Figure 4.4: Drainage condition from CBE Dejen branch to new bus station

According to Figure 4.4 above, of most the performance of drainage systems in this area is poor. That is 29% of the drainage structure is very good, 12% of drainage structure is good, 4% is moderate, 46% poor and 8% is very poor or severe. On this, we can understand that most of the drainage system cannot perform its function well and cannot discharge flood into the outlet.

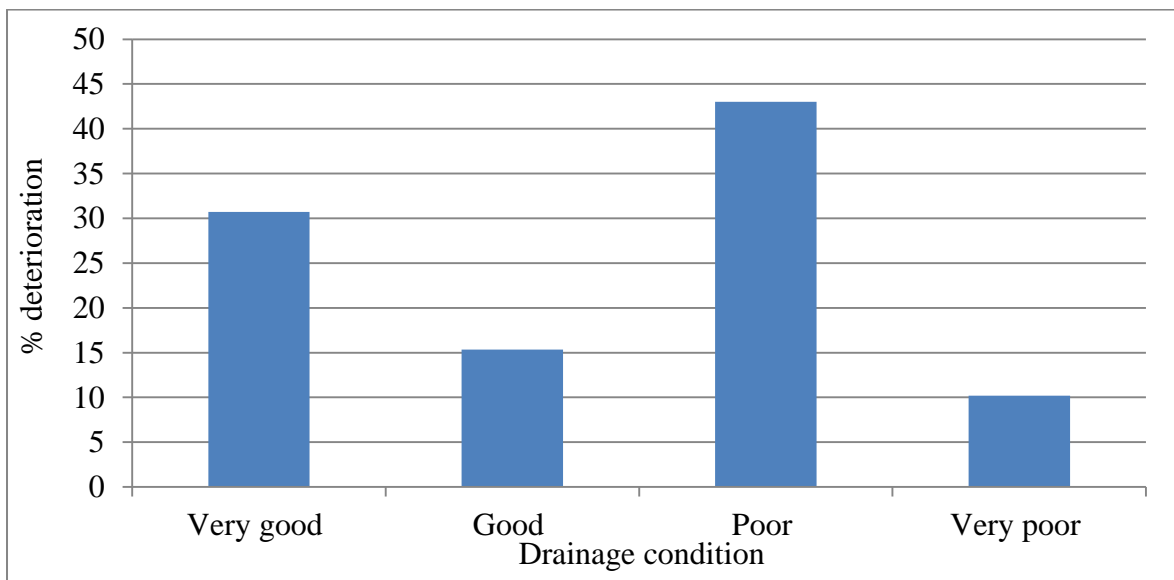


Figure 4.5: Drainage condition from Andinet College to Mulugojam hall

As shown from Figure 4.5 above, 32% is very good, 15% good, 15% is moderate, 28% is poor and 10% is very poor. In this area also the performance is poor and needs maintenance and redesign of the drainage system to serve its function properly.

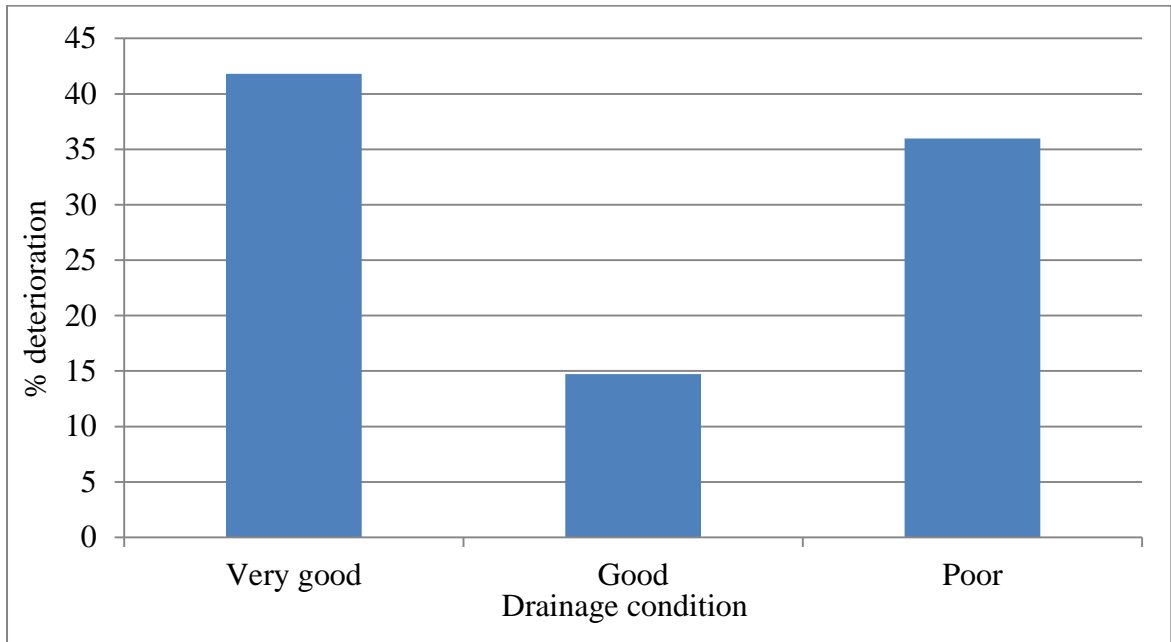


Figure 4.6: Drainage condition from St. George church to Litete gorge outfall

Also as shown from Figure 4.6 above, 42% is very good, 15% is good and 36% is poor. The major part of the drainage system in this area is very good but most of the drainage systems must be functional to reduce over flooding.

#### 4.1.2. Lack of community awareness

Creating awareness of households about drainage is the main solution to increase performance and age of existing drainage structures. Lack of community awareness about sanitation and the problems of flooding in the Town hinder the safety and functionality of drainage structures. In Dejen Town the public awareness about sanitation is very low and they have used the drainage structures for waste removal, to dump plastic materials, grass, rubbish materials, electric poles, construction materials, and many unwanted materials into manholes and stormwater drainage structures. Due to these social problems, the performance of drainage structures in the Town is reduced and became the challenge of stormwater drainage system.



Figure 4.7: Clogged structures with electric poles, construction materials, and disposed wastes (site visit)



Figure 4.8: Deposition plastic materials in to drainage ditches (site visit)



Figure 4.9: Broken road side ditches (site visit)

### 4.1.3. Problem of outlet ditches sanitation

Limited cleaning and maintenance of outlet ditches was the other problem in the study area. In Dejen Town the outlet ditches were broken and clogged by sediments, plastic materials, stone, and rubbish materials as shown below. This leads to backflow of flood, breaking of flood towards roads and residence. Due to this clogging, the outlet ditches in the study area cannot perform its function well which means it cannot discharge the coming flood towards the outfall.



Figure 4.10: Clogged and broken outlet ditches

### 4.2. Developing IDF curve to Estimating Peak Runoff and Existing drainage capacity

Manning's formula was used to calculate the hydraulic capacity of existing drainage structures of the study area. Existing stormwater drainage facilities in the study area include open masonry, earthen drain, and concrete-lined drain. To know the amount of discharge conveyed through existing drainage structures drainage dimensions were recorded by measuring tape during field survey and by taking data from Dejen Town Municipality. Land use type, the slope of the study area, area and length of the catchment, and the runoff coefficient were the main factors to estimate peak discharge of the study area. Existing capacity of drainage structures of each sub-catchment were calculated from Appendix-2, Table 2c and estimated peak runoff was as shown from Appendix 2, Table 2a.

#### 4.2.1. Intensity-Duration-Frequency curve of Dejen Town

For this thesis, Intensity-Duration-Frequency curve was developed from 24-hr rainfall data of a study area of 32 years obtained from the Meteorological Agency of Ethiopia. This Developed IDF curve was applicable specifically for the study area to interpolate rainfall intensity of the study area. Dejen Town was included under rainfall Region (RR-A2) and to increase accuracy IDF curve was developed for a specific area of Dejen Town. Recorded daily rainfall data from 1987-2018 for 32 years was used for developing IDF curve of Dejen Town, but ERA developed IDF curve of Rainfall Region A2 was not for specific area and include wider area under one rainfall region. The developed IDF curve was presented below from Table 4.5, and Figure 4.11.

#### 4.2.2. Selection of distribution method and design rainfall of shorter duration

#### 4.2.3. Gumbel's extreme-value distribution method

According to k.subramanya 2008 the value of  $\bar{Y}_n$  and  $S_n$  are selected from Gumbel's extreme value distribution table considered depending on the sample size but, if the sample size is greater than a hundred or  $\infty$  the value of  $\bar{Y}_n$  and  $S_n$  are 0.577 and 1.2825 respectively,  $\bar{X}$  and  $\sigma_x$  are 50.6 and 13.4 respectively.

$$Y_T = -\ln \left( \ln \left( \frac{T}{T-1} \right) \right)$$

$$K_T = Y_T - \frac{\bar{Y}_n}{S_n}, X_T = \bar{X} + K_T * \sigma_x$$

Table 4.1: 24-hr RF depth vs frequency of Gumbel's extreme-value Distribution method

Gumbel's extreme-value Distribution method				KT	XT= $\bar{X}$ +KT* $\sigma_x$			
T	T-1	T/T-1	Ln(T/T-1)	YT	YT- $\bar{Y}_n$	YT- $\bar{Y}_n/S_n$	KT* $\sigma_x$	XT
2	1	2.0	0.693	0.367	-0.210	-0.16412	-2.19924	48.4
5	4	1.3	0.223	1.500	0.923	0.719641	9.643194	60.2
10	9	1.1	0.105	2.250	1.673	1.30477	17.48392	68.1
25	24	1.0	0.041	3.199	2.622	2.044081	27.39069	78.0
50	49	1.0	0.020	3.902	3.325	2.592545	34.7401	85.3
100	99	1.0	0.010	4.600	4.023	3.136958	42.03524	92.6

**4.2.4. Log-Pearson III distribution method**

$Z = \log X \dots\dots\dots 4.1$

$Z_T = \bar{Z} + K_z * \delta z \dots\dots\dots 4.2$

Where:  $K_z$  = frequency factor which is a function of T and the coefficient of Skewness,  $C_s$

$K_z = f(C_s, T)$  is given in the table.

$\delta z$  = Standard deviation of the Z variety sample,  $\delta z = \sqrt{\frac{\sum (Z_i - \bar{z})^2}{N-1}}$

$C_s$  = coefficient of skew of variate z.

By using calculated  $C_s$  value,  $K_z$  was interpolated from Appendix 5, Table 5E.

$C_s = \frac{N * \sum (Y_o - \bar{Y})^3}{(N-1) * (N-2) * (\delta z)^3} = \frac{16.03}{18.36} = 0.87$

$Y_o = \log(X)$ ,  $\bar{y} = \bar{z}$  = mean of  $Y_o$ .

$X_T = 10^{Z_T}$

	If the Coefficient of Skewness $C_s$ is equals to 0.87					
T	2	5	10	25	50	100
$K_z$	-0.143	0.5975	1.3381	2.1486	2.4845	2.9372

Table 4.2: 24-hr RF depth vs frequency of Log- Pearson III distribution

T	Mean( $\bar{Z}$ )	STDEV	$K_z$	STDEV* $K_z$	$Z_T = \bar{Z} + (STDEV * K_z)$	$X_T = 10^{Z_T}$
2	1.69	0.1	-0.14	-0.014	1.676	47.4
5	1.69	0.1	0.5975	0.05975	1.74975	56.2
10	1.69	0.1	1.3381	0.13381	1.82381	66.7
25	1.69	0.1	2.1486	0.21486	1.90486	80.3
50	1.69	0.1	2.4845	0.24845	1.93845	86.8
100	1.69	0.1	2.9372	0.29372	1.98372	96.3

**4.2.5. General Pareto distribution**

Design point rainfall or 24-hr rainfall depth of General Pareto Distribution was calculated using Easy fit 5.6 by using maximum daily rainfall of the study area as an input to the software that was selected by mean residual life plot. From a given series of independent and identically distributed data, the values that exceed a sufficiently high threshold follow a generalized Pareto distribution (GPD). This implies that from n observed data points of a variable X given as X1, X2....Xn new time series can be generated as;

$$Y=f(x/k, \sigma, \mu) = \left(\frac{1}{\sigma}\right) * \left(1 + k \left(\frac{x-\mu}{\sigma}\right)^{-1-1/k}\right) \dots\dots\dots 4.3$$

Then by using selected data from mean residual life plot and simulating Easy fit 5.6 gives  $\mu$ ,  $\sigma$ , and k of 37.314, 12.698, and 0.01245 respectively.

Table 4.3: 24- hour rainfall depth vs Frequency of Dejen and ERA

R24	T-2	T-5	T-10	T-25	T-50	T-100
RR-A2	51.92	65.52	74.45	85.7	94.07	102.45
Dejen	46.15	57.96	66.98	79.02	88.22	97.50

Table 4.4: R24 Rainfall depth vs frequency of each distribution

Return period(T)	G.Pareto distribution	Log-Pearson type III	Gumbel
2	46	47	48
5	58	56	60
10	67	67	68
25	79	80	78
50	88	87	85
100	97	96	93

From Table 4.4, R24 rainfall depth vs frequency of each distribution methods General Pareto Distribution have optimum values and selected to calculate rainfall of shorter duration to develop IDF curve. For the rainfall data, General Pareto Distribution provided the best fit than Log-Pearson type III and Gumbel distribution based on Kolmogorov-Smirnov and Anderson Darling test of goodness of fit from Table 3.3. Then frequency analysis was performed using the best fit distribution method that is General Pareto distribution method for different return periods and frequency values used to develop IDF

curve that was used to find the design storm intensity which helps to estimate peak runoff from each sub-catchments.

#### 4.2.6. Design rainfall of shorter duration

The rainfall depths obtained from the gauging station were 24-hour duration depth. However, when short time duration rainfall data were not available, the intensity of a short time rainfall of long time rainfall would be calculated using a reduction formula. According to the Ethiopian Roads Authority (ERA, 2013), the following equations were used to calculate the shorter duration of rainfall from 24-hour duration.

$$RR_t = \frac{t}{24} \frac{(b+24)^n}{(b+t)^n} \dots\dots\dots 4.4$$

Where:

RR<sub>t</sub> = Rainfall depth ratio R<sub>t</sub>: R<sub>24</sub>

R<sub>t</sub> = Rainfall depth in a given duration t

R<sub>24</sub> = 24-hr rainfall depth

Coefficients b = 0.3 and n = 0.78 to 1.09

$$R_t = \frac{t(b+24)^n}{24(b+t)^n} * R_{24} \dots\dots\dots 4.5$$

Rainfall Intensity (mm/hr) is given as

$$I = \frac{R_t}{t} \dots\dots\dots 4.6$$

$$I = \frac{R_{24} * (b+24)^n}{24(b+t)^n} \dots\dots\dots 4.7$$

As ERA manual suggestion b = 0.3 and n = 0.92 for tabulating results of rainfall durations 10, 20, 30 ..... 180 minutes.

Sample Calculation of rainfall of shorter duration

Duration, t=0.083333 hr, for T-2, R<sub>24</sub>=46.15

$$R_t = \frac{t(0.3+24)^{0.92}}{24(0.3+t)^{0.92}} * R_{24} = \frac{0.083333(0.3+24)^{0.92}}{24(0.3+0.083333)^{0.92}} * 46.15 = 7.29$$

The design rainfall of shorter duration of Dejen Town is presented from Appendix 1, Table 1c and calculated Intensity vs Duration to develop IDF Curve of Dejen Town is shown from Table 4.5.

Sample calculation of Rainfall intensity of study area

R<sub>t</sub> =7.29, from Appendix 1, Table 1c and duration t=5 min=0.083333 hr and 2 years return period.

$$I = \frac{Rt}{t} = \frac{7.29}{0.083333} = 87 \text{ mm/hr}$$

Table 4.5: Calculated Intensity vs Duration to develop IDF Curve of Dejen Town

Duration (min)	T-2	T-5	T-10	T-25	T-50	T-100
5	87	109.838	126.931	149.751	167.189	184.779
10	73	91.656	105.919	124.961	139.512	154.190
15	63	78.797	91.060	107.431	119.940	132.559
20	55	69.206	79.976	94.354	105.341	116.423
25	49	61.767	71.379	84.211	94.017	103.908
30	44	55.822	64.508	76.106	84.968	93.907
35	41	50.958	58.888	69.475	77.565	85.725
40	37	46.902	54.201	63.945	71.391	78.902
45	35	43.466	50.230	59.261	66.161	73.122
50	32	40.517	46.822	55.240	61.672	68.161
55	30	37.957	43.863	51.749	57.775	63.853
60	28	35.712	41.270	48.689	54.359	60.078
65	27	33.728	38.977	45.984	51.339	56.740
70	25	31.961	36.935	43.575	48.649	53.767
75	24	30.377	35.104	41.415	46.237	51.102
80	23	28.948	33.453	39.467	44.063	48.698
85	22	27.652	31.956	37.701	42.091	46.519
90	21	26.472	30.592	36.092	40.295	44.534
95	20	25.393	29.344	34.620	38.651	42.718
100	19	24.401	28.199	33.268	37.142	41.050
105	19	23.487	27.142	32.022	35.751	39.512
110	18	22.642	26.165	30.869	34.464	38.090
115	17	21.858	25.259	29.800	33.270	36.770
120	17	21.128	24.416	28.805	32.159	35.543
125	16	20.447	23.629	27.877	31.124	34.398
130	16	19.811	22.894	27.010	30.155	33.327
135	15	19.214	22.205	26.197	29.247	32.324

140	15	18.654	21.557	25.433	28.394	31.382
145	14	18.127	20.948	24.714	27.592	30.495
150	14	17.630	20.374	24.037	26.836	29.659
155	14	17.161	19.832	23.397	26.121	28.869
160	13	16.717	19.318	22.792	25.446	28.123
165	13	16.296	18.832	22.218	24.805	27.415
170	13	15.897	18.371	21.674	24.198	26.743
175	12	15.518	17.933	21.157	23.620	26.105
180	12	15.157	17.516	20.665	23.071	25.498

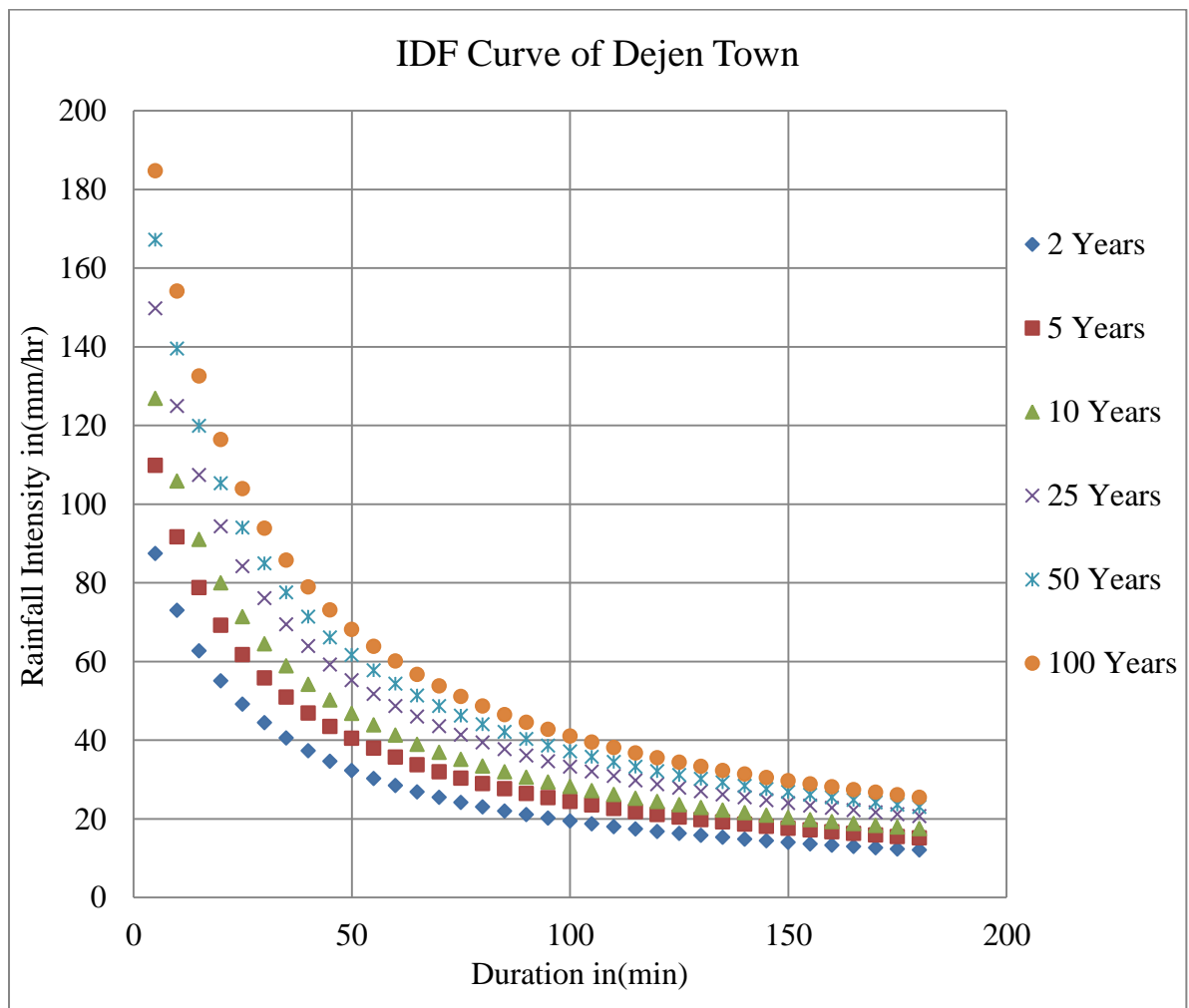


Figure 4.11: Intensity-Duration-Frequency curve of Dejen Town

Developed IDF curve from meteorological station of this study area shows less value of rainfall intensity than IDF curve developed by ERA for rainfall Region A-2 for each return

periods 2, 5, 10, 25, 50 and 100 years with corresponding rainfall duration from 5min to 180 minutes as described from Table 4.6 below.

Table: 4.6. Comparison of IDF curve of Dejen Town with IDF curve of ERA

	Mine	ERA	Mine	ERA	Mine	ERA	Mine	ERA	Mine	ERA	Mine	ERA
Duration(min)	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	87	98	110	124	127	141	150	162	167	178	185	194
10	73	82	92	104	106	118	125	136	140	149	154	162
15	63	71	79	89	91	101	107	117	120	128	133	139
30	44	50	56	63	65	72	76	83	85	91	94	99
45	35	39	43	49	50	56	59	64	66	71	73	77
60	28	32	36	40	41	46	49	53	54	58	60	63
75	24	27	30	34	35	39	41	45	46	49	51	54
90	21	24	26	30	31	34	36	39	40	43	45	47
105	19	21	23	27	27	30	32	35	36	38	40	42
120	17	19	21	24	24	27	29	31	32	34	36	37
135	15	17	19	22	22	25	26	28	29	31	32	34
150	14	16	18	20	20	23	24	26	27	29	30	31
165	13	15	16	18	19	21	22	24	25	26	27	29
180	12	14	15	17	18	19	21	22	23	25	25	27

The change in rainfall characteristics, and changes in the hydrological cycle due to increase in greenhouse gases are projected to cause variations in Intensity-Duration-Frequency curve of a certain area. Percentage change of rainfall across different reference periods also causes significant changes on IDFC. For this thesis, IDF curve was developed from 32 years rainfall data of the study area and that increases accuracy of the intensity of Dejen Town. But, IDF curve of ERA for rainfall region A-2 was developed from 27 years rainfall data obtained from the stations far from each other. This causes the difference of IDF curve developed for this thesis and developed by ERA rainfall region A-2.

#### 4.2.7. Land use Type of Study Area

Land use land cover of the study area was another required data for the study. The land use land cover data would be an essential input for selecting runoff coefficient for determining

peak rate of runoff using rational method. The land use condition in Dejen Town and surrounding is covered with urban settlements or residential areas, manufacturing industries, green areas, social services, open spaces, and places reserved for the special purpose, different administration offices, and agricultural lands. Area covered with each land use type from total area of Dejen Town and surrounding is, Residential Areas covers about 25.2%, Mixed Residential 16.3%, commercial 15.9%, social services, and administration offices 5%, reserved area and Neighborhood playground 10.6%, colleges and schools 3.7%, Green area and forest 8.5%, Agricultural area is 10.1%, and Manufacturing and small industries 4.7%.

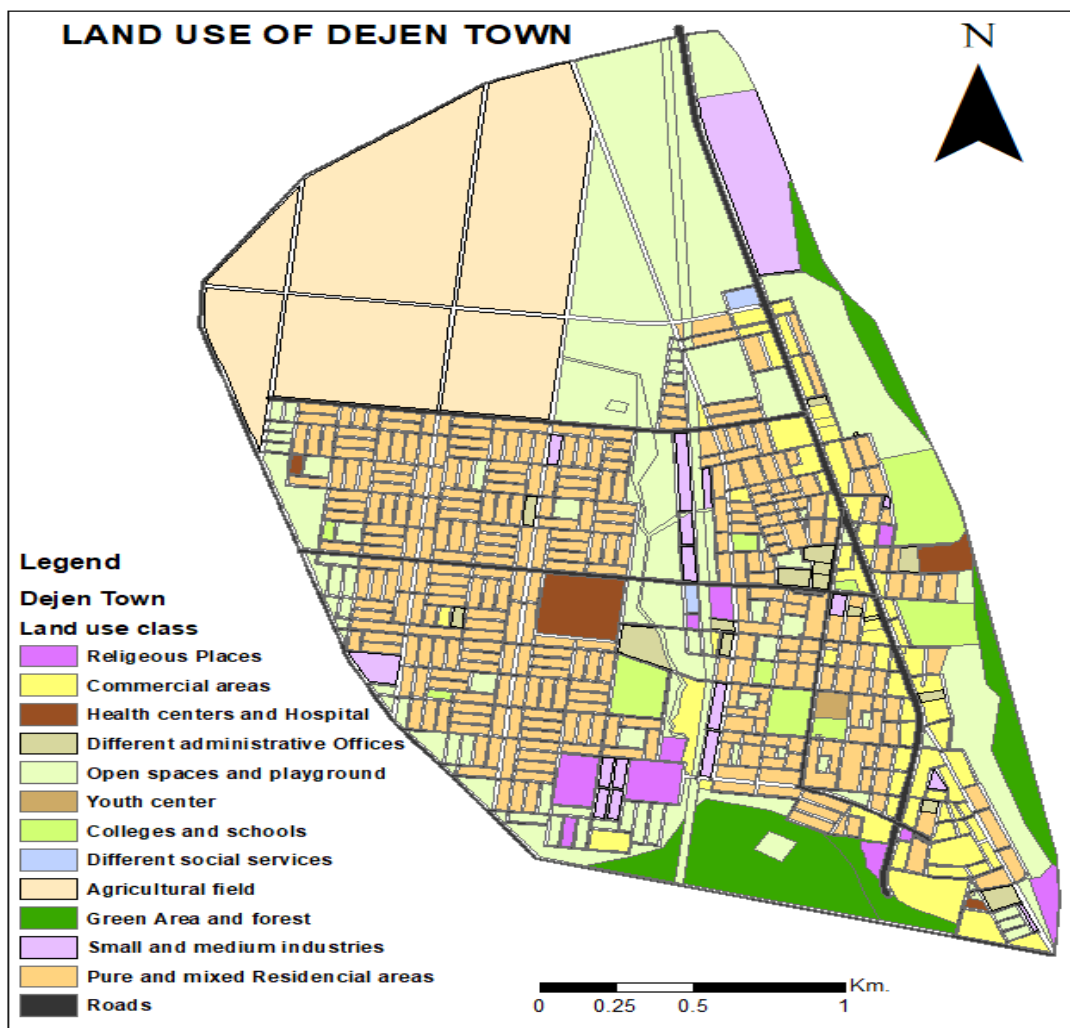


Figure 4.12: Land use of Dejen Town and surrounding by ArcGIS

#### 4.2.8. Rational method and estimating peak runoff

Estimating the peak drainage discharge of the study area is important to understand how much discharge is above the capacity of existing drainage structure and to design structures that can carry overflowing discharge. To calculate peak discharge in each sub-catchment

properly, we have to select proper parameters, knowing the land use type, and the area of sub-catchments. Also, proper selection of runoff coefficient and selecting frequency factor is important.

Calculating the amount of discharge by using rational formula is done by

$$Q=0.00278*C_w*C_f*I*A.....4.8$$

0.00278=1/360=conversion factor

Q=peak rate of runoff

C<sub>w</sub>= weighted runoff coefficient

A= area of sub-catchment in ha

C<sub>f</sub>=frequency factor

I=rainfall intensity selected by considering duration equals to time of concentration.

Sample calculation of Time of concentration (T<sub>c</sub>) for sub-catchment-1

$$T_c=T_t+T_{ci}.....4.9$$

$$\text{Travel or channel flow time (T}_t) = \frac{L}{3600V}.....4.10$$

L = length of the main drainage canal (m)

$$V=1/n*R^{2/3}*S^{1/2}=2.1\text{m/se, } T_t=\frac{1245}{3600*2.1} = 0.1647\text{hr}$$

$$\text{Inlet Time, } T_{ci} = \frac{3.64(1.1-C_w)*(L/1000)^{0.83}}{H^{0.33}}$$

L = flow length from the remotest point to the point t of interest (km).

C<sub>w</sub>=weighted runoff coefficient=A<sub>i</sub>C<sub>i</sub>/A<sub>t</sub> = 0.64 for sc-1

$$T_{ci} = (3.64*(1.1-0.64)*(1.245/1000))^{0.83}/(5)^{0.33} = 0.003821\text{hr}$$

$$T_c=T_t+T_{ci}=0.1647+0.003821=0.16852\text{hr} = 10.12\text{min}$$

Based on calculated T<sub>c</sub> rainfall intensity was determined from the IDF curve developed for study area corresponding with the return period by considering duration equals to time of concentration, rainfall intensity was interpolated from IDF curve i.e., I-10=105.563, I-25=124.24, I-50 = 139.04, I-100= 153.67 mm/hr

Then, Runoff Q for sub-catchment 1 for design period 10 years, 25 years, 50 years, and 100 years was calculated by Rational formula  $Q = 0.00278*C_w*C_f*I*A$ .

$$Q-10 = 1.4815 \text{ m}^3/\text{se}$$

$$Q-25 = 1.92 \text{ m}^3/\text{se}$$

$$Q-50 = 2.342 \text{ m}^3/\text{se}$$

$$Q-100 = 2.696\text{m}^3/\text{se}$$

Table 4.7: Peak runoff calculated by rational method for each sub-catchment

SC-ID	Area (ha)	TC- min	Cw	Intensity- 10 yr	Intensity- 25 yr	Intensity- 50 yr	Intensity- 100 yr	Q- 10 Yr	Q- 25 Yr
SC-1	7.92	10.12	0.64	105.56	124.24	139.04	153.67	1.48	1.92
SC-2	9.29	7.50	0.58	116.43	137.36	153.35	169.49	1.75	2.27
SC-3	7.71	8.43	0.67	112.64	132.89	148.37	163.98	1.62	2.10
SC-4	9.91	5.81	0.65	123.53	145.74	162.71	179.82	2.20	2.85
SC-5	7.15	10.00	0.66	105.92	124.96	139.51	154.19	1.39	1.80
SC-6	10.5	7.50	0.60	116.43	137.36	153.35	169.49	2.04	2.65
SC-7	4.17	6.16	0.71	122.06	144.00	160.77	177.68	1.00	1.30
SC-8	10.9	8.26	0.58	113.23	133.59	149.14	164.83	1.99	2.58
SC-9	34.1	11.30	0.65	102.05	120.40	134.42	148.57	6.30	8.18
SC-10	10.1	13.85	0.69	94.48	111.46	124.44	137.53	1.83	2.37
SC-11	3.09	9.34	0.82	108.69	128.23	143.17	158.23	0.77	1.00
SC-12	8.98	6.59	0.74	120.25	141.87	158.39	175.05	2.21	2.87
SC-13	1.63	5.42	0.70	125.17	147.61	164.87	182.21	0.40	0.52
SC-14	7.92	10.93	0.79	103.16	121.58	135.87	150.17	1.79	2.32

The peak discharge of this Thesis was estimated by using the rational method from the currently estimated amount of runoff for a design period of 10-years and for checking 25-years by considering the amount of runoff that will increase for each sub-catchment outfall. After calculating peak discharge of each sub-catchment with rational method, the existing hydraulic capacity in each sub-catchment was calculated by Manning's formula and compared with peak estimated runoff as shown from Table 4.8.

#### 4.2.9. Determining hydraulic capacity of existing drainage structures

##### ➤ Manning's formula

Existing hydraulic capacity is the capacity of existing drainage structures to carry discharge resulted from a certain sub-catchment. Manning's formula was used to calculate existing drainage capacity of each sub-catchment to compare the amount of existing drainage discharge the structures can carry with estimated runoff. Existing hydraulic capacity of each sub-catchment is calculated from Appendix 2, Table 2c.

Table 4.8: Comparing estimated and existing peak discharge of Sub-catchments using a rational method and Manning's Formula

SC-ID	Area (ha)	TC- (min)	Cw	Intensity- 10 yr	Intensity- 25 yr	Q (R)-10 Yrs	Q (R)-25 Yrs	Manning's (m <sup>3</sup> /se)
SC-1	7.92	10.12	0.64	105.56	124.24	1.48	1.92	1.09
SC-2	9.29	7.50	0.58	116.43	137.36	1.75	2.27	1.33
SC-3	7.71	8.43	0.67	112.64	132.89	1.62	2.10	1.56
SC-4	9.91	5.81	0.65	123.53	145.74	2.20	2.85	1.24
SC-5	7.15	10.00	0.66	105.92	124.96	1.39	1.80	2.24
SC-6	10.49	7.50	0.60	116.43	137.36	2.04	2.65	1.77
SC-7	4.17	6.16	0.71	122.06	144.00	1.00	1.30	0.87
SC-8	10.92	8.26	0.58	113.23	133.59	1.99	2.58	2.75
SC-9	34.11	11.30	0.65	102.05	120.40	6.30	8.18	3.86
SC-10	10.07	13.85	0.69	94.48	111.46	1.83	2.37	1.50
SC-11	3.09	9.34	0.82	108.69	128.23	0.77	1.00	1.67
SC-12	8.98	6.59	0.74	120.25	141.87	2.21	2.87	1.64
SC-13	1.63	5.42	0.70	125.17	147.61	0.40	0.52	1.03
SC-14	7.92	10.93	0.79	103.16	121.58	1.79	2.32	1.49

As shown from Appendix 2, Table 2a and 2c calculated Existing and Estimated capacity results obtained from the 10-year design period and the above Table 4.8 indicates that existing drainage capacity near CBE Abaysheleko branch (SC-1), drainage around 02 playgrounds (SC-2), drainage around Demis Hotel (SC-3), drainage around Dejen 02 primary school (SC-4), around Dejen town kinder garden (SC-6), around CBE Dejen branch (SC-7), around Mulugojam hall (SC-9), around Andinet business technology college (SC-10), around Serkalem Hotel (SC-12) and around Kurar Amba Hotel (SC-14) were inadequate and estimated capacity was greater than Existing Capacity and existing drainage structure can't carry estimated discharge and that shows overflow had happened in these sub-catchments and redesign additional drainage structures are required. Whereas, around Seble restaurant (SC-5), around new bus station (SC-8), around Ambasel restaurant (SC-11), and around St. Gorge church (SC-13) existing capacity was greater than estimated capacity and that shows no overflow of runoff.

### **4.3. Simulation of Storm Water Management Model (SWMM)**

Storm Water Management Model (SWMM) is well-known software to analyze the capacity of existing drainage system for the selected study area. The sub-catchments of the study area were divided using the ArcGIS tool and the rainfall intensity is obtained from IDF curves.

The input values used for this thesis were N-Imperv of 0.005, N-Perv of 0.05, Dstore-Imperv of 1.3, Dstore-Perv of 2.5, and manning's roughness of 0.4. Percent slope, %Imperv, are different for each sub-catchments and also length and depth of the conduit are different for each conduit. The flow routing type selected for this thesis was dynamic wave routing. This flow routing type was selected because it is possible to represent pressurized flow when a closed conduit becomes full with this routing type and it can account for channel storage, backwater effect, entrance and exit losses, flow reversal, and pressurized flow. The method selected for modeling infiltration was Modified Horton's method this is because this method gives more accurate infiltration estimation when low rainfall intensities occur but, Horton's method applies only to events for which the rainfall intensity always exceeds the infiltration capacity.

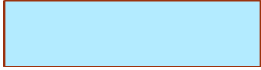
After the input parameters of the model were inserted and run simulation, the Storm Water Management Model was successful and the simulation results were obtained.

#### **4.3.1. Water depth and flow in the Links**

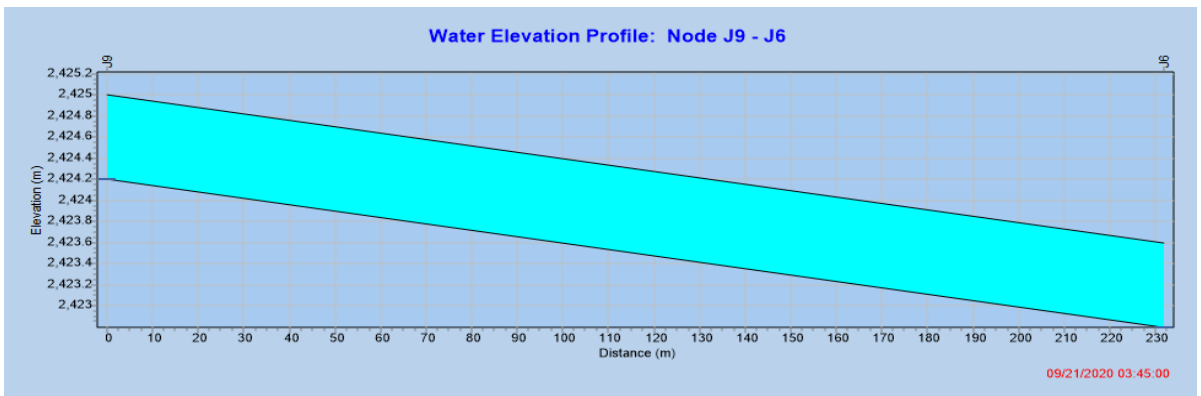
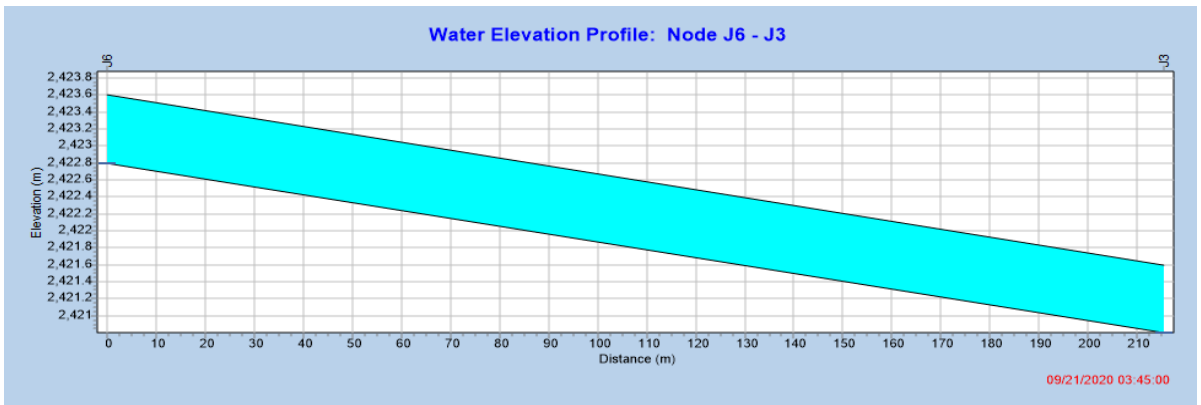
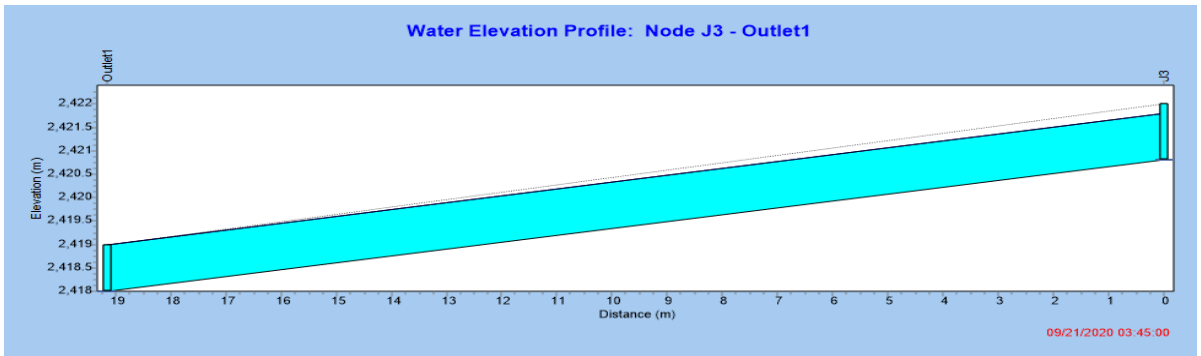
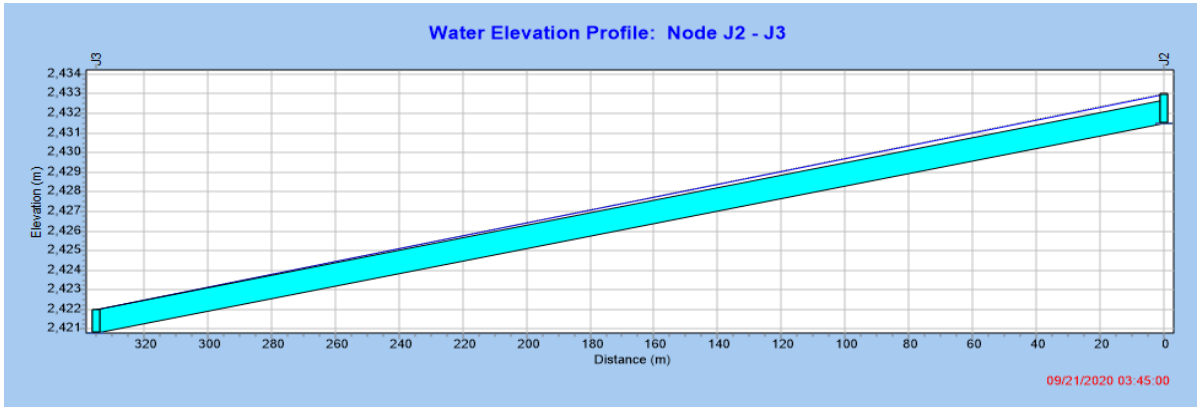
##### **Outfall towards Fasileds Bridge**

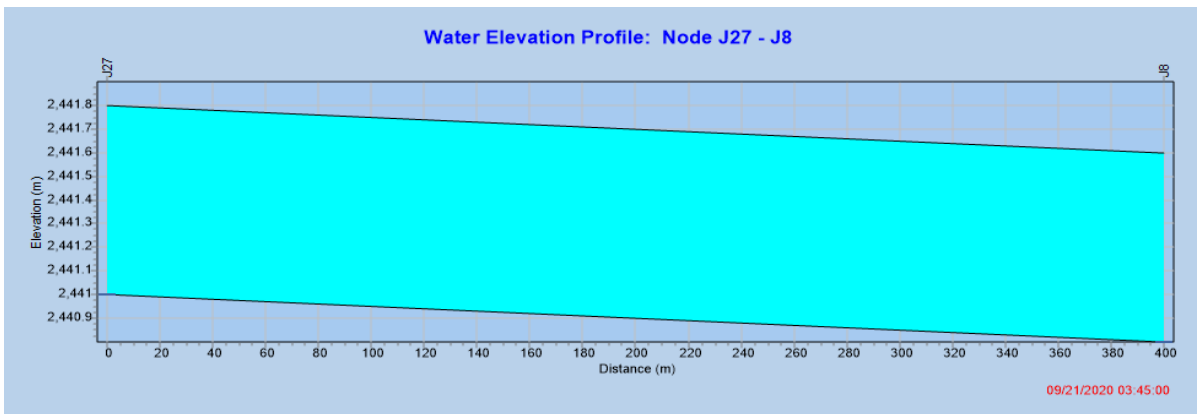
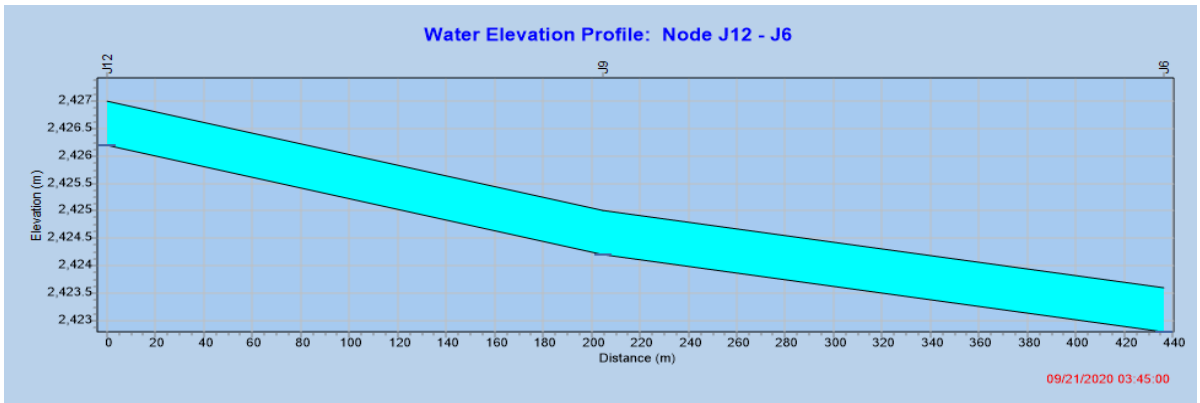
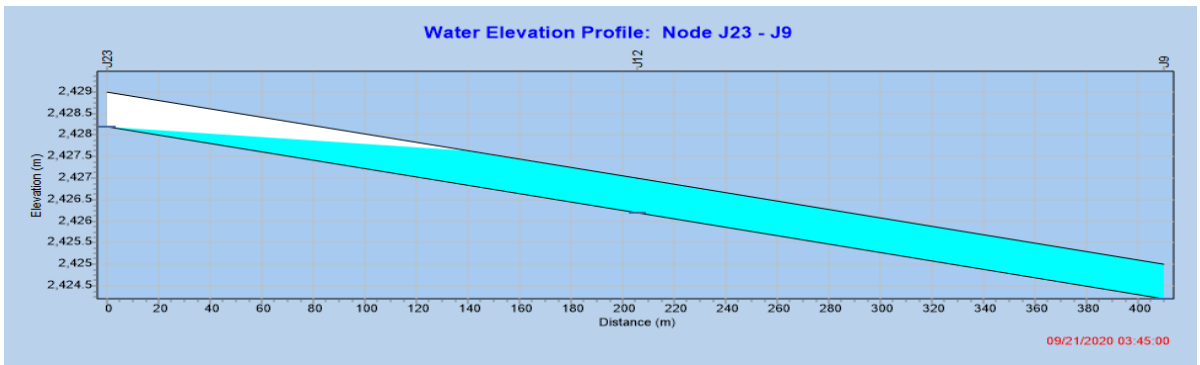
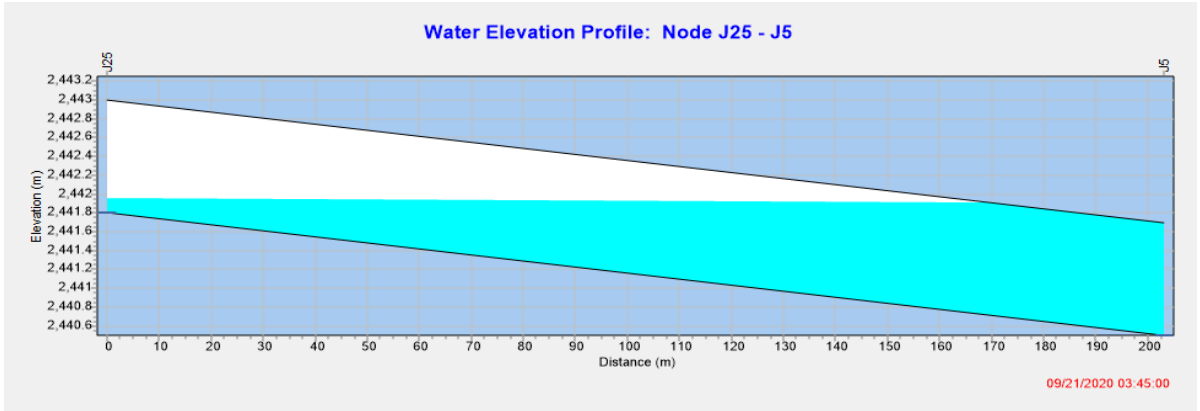
The figures blow shows the flow of flood through the conduit, amount of flood in the junction, and length of the conduit. It indicates how much the flood rises in the conduit and the amount of flood hazard in the study area.

As shown in Figures 4.13 below from the simulation results Junction J2, J3, J5, J6, J8, J9, J12, J27, and outlet 1 are over flooded as a result of the insufficient design of canal depth and width and outfall 1(conduit 3) is busy to remove flood coming from sub-catchments. But the drainage canals at Link 1, link 4, link 8, and link 12 can carry the generated runoff from sub-catchments.



Flow of water through the conduit





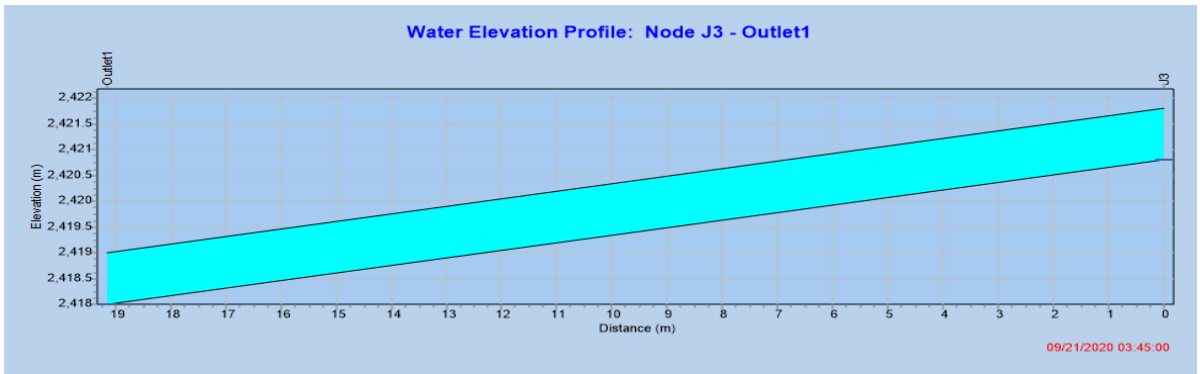


Figure 4.13: Water elevation profiles towards Fasileds bridge outfall

### Outfall towards Abuadem gorge

As shown from Figures 4.14 below simulation results of SWMM Junctions 15, 16, 17 that is Links C 16, 17, 18, and Outfall 2 are over flooded and insufficient capacity to carry runoff generated from sub-catchments. But Junction 26 (Link C<sub>26</sub>) can carry the generated flood.

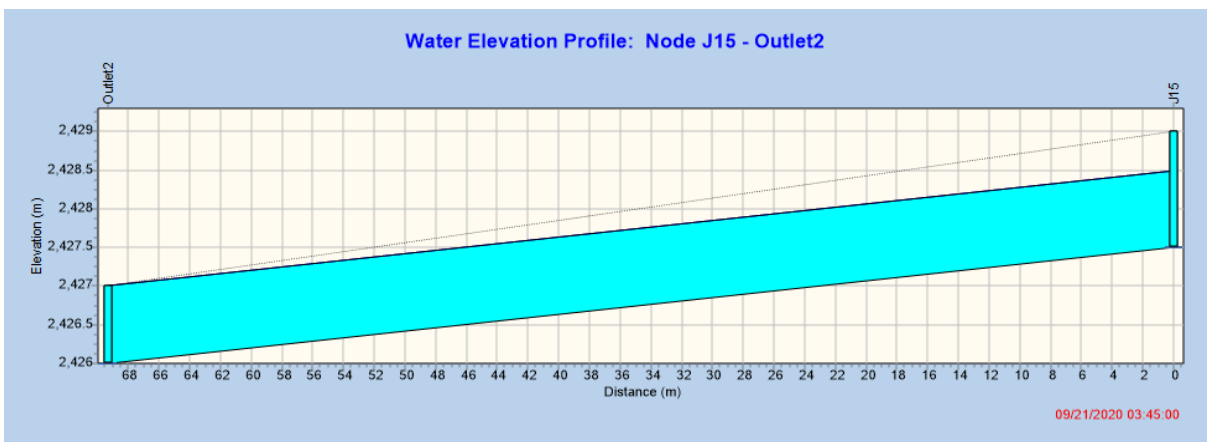
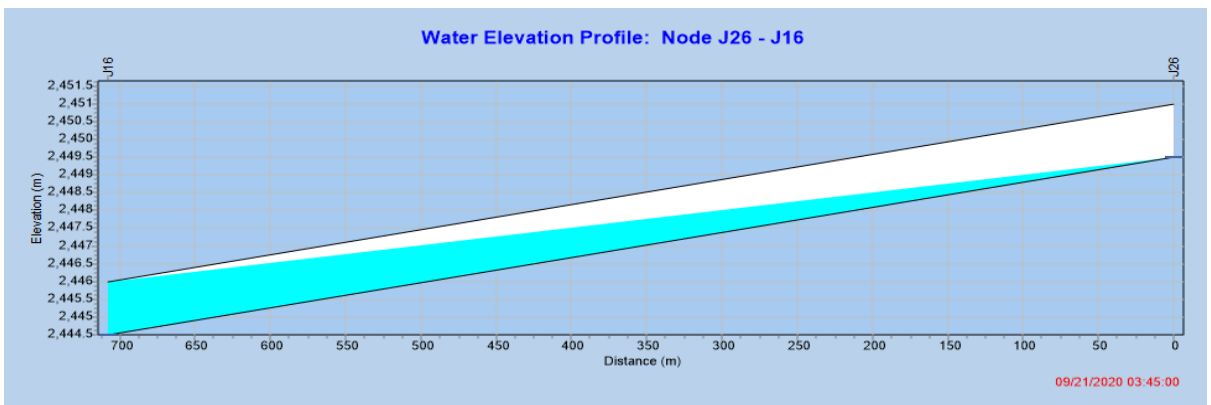
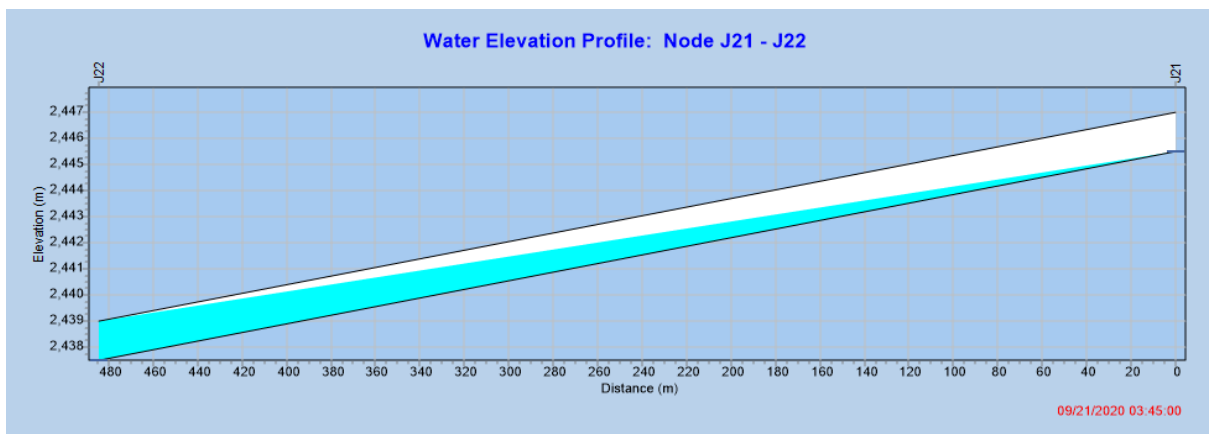
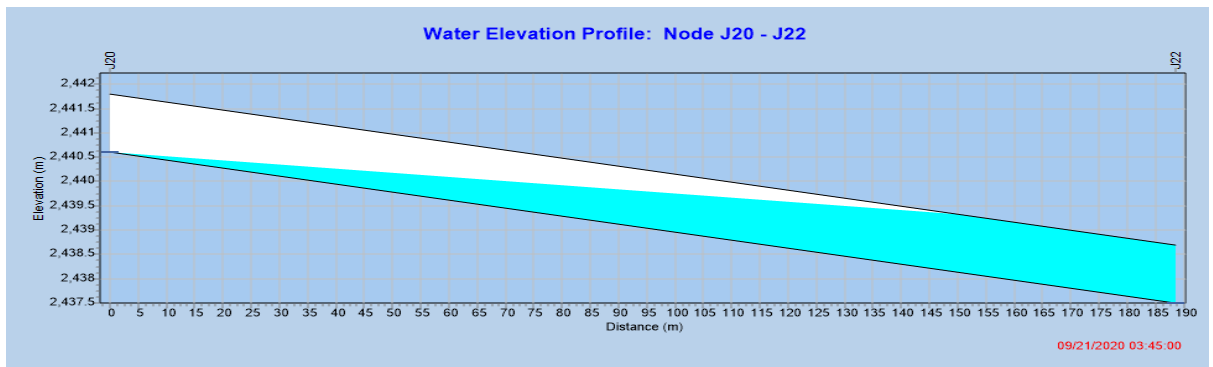
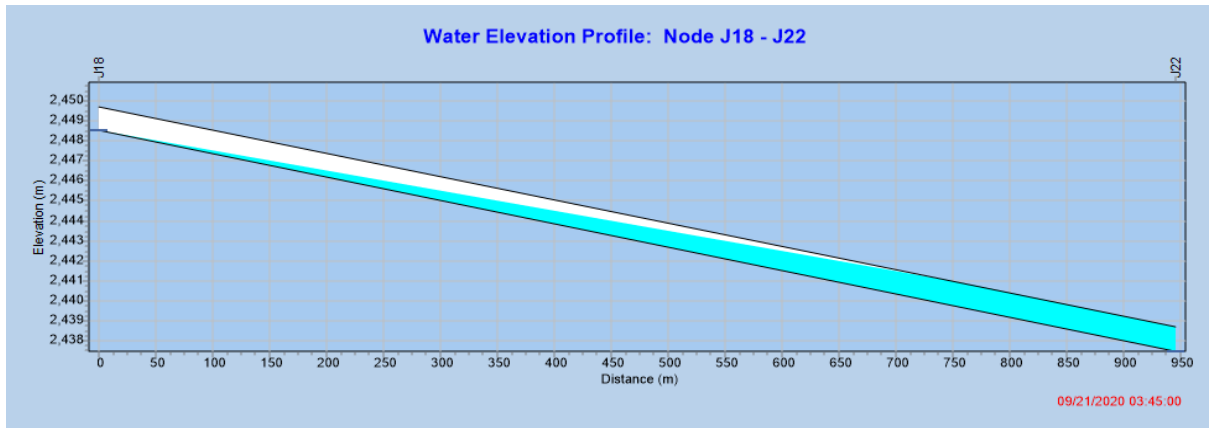


Figure 4.14: Water elevation profiles towards Aduadem gorge outfall

## Outfall towards Litete gorge

As show from Figures 4.15 below from simulation result of Storm Water Management Model Junction 22 is over-flooded and has no the capacity to carry flood coming from sub-catchments, but outfall 3 have the capacity to carry flood coming from sub catchments and no over flooding in this outfall.



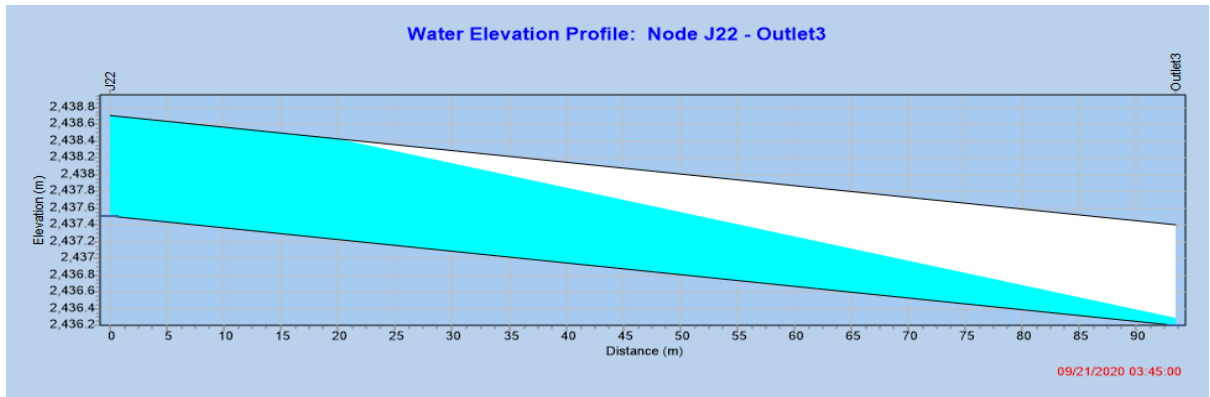


Figure 4.15: Water elevation profiles towards Litete gorge outfall

Table 4.9: Comparison of runoff resulted from a rational method, SWMM, and Manning

SC-ID	Rational Q- 10 Yrs (m <sup>3</sup> /se)	SWMM Q-10 Yrs (m <sup>3</sup> /se)	Rational Q- 25Yrs (m <sup>3</sup> /se)	SWMM Q-25Yrs (m <sup>3</sup> /se)	Manning's Q-(m <sup>3</sup> /se)	Velocity (m/s)
SC-1	1.48	1.34	1.92	1.63	1.09	2.1
SC-2	1.75	1.70	2.27	2.13	1.33	3.1
SC-3	1.62	1.4	2.10	1.7	1.56	2.12
SC-4	2.20	2.02	2.85	2.45	1.24	3.61
SC-5	1.39	1.21	1.80	1.47	2.24	2.17
SC-6	2.04	2.14	2.65	2.6	1.77	3.25
SC-7	1.00	0.79	1.30	0.96	0.87	3.21
SC-8	1.99	2.21	2.58	2.69	2.75	3.21
SC-9	6.30	5.21	8.18	6.34	3.86	3.23
SC-10	1.83	1.71	2.37	2.08	1.50	1.79
SC-11	0.77	0.54	1.00	0.66	1.67	1.48
SC-12	2.21	1.86	2.87	2.26	1.64	2.71
SC-13	0.40	0.34	0.52	0.42	1.03	2.39
SC-14	1.79	1.5	2.32	1.83	1.49	2.35

Figure 4.16 below shows discharge resulted from SWMM 5.1, rational method, and Manning's Formula for a design period of 10 years and for checking 25 years design period. The result and the Figure shows the discharge result of SWMM model and Rational formula are closer but discharge resulted from Manning is smaller in most of sub-catchments. That indicates the current performance of drainage structures is low. Due to

this, it is possible to conclude the design must be done according to the discharges resulted from Rational Formula and SWMM model. The results obtained from SWMM model shows flood occurs, from sub-catchments 1, 2, 4, 6, 9, 10, 12, and 14, but sub-catchment 3, 5, 7, 8, and 11 the existing structure can carry the incoming flood.

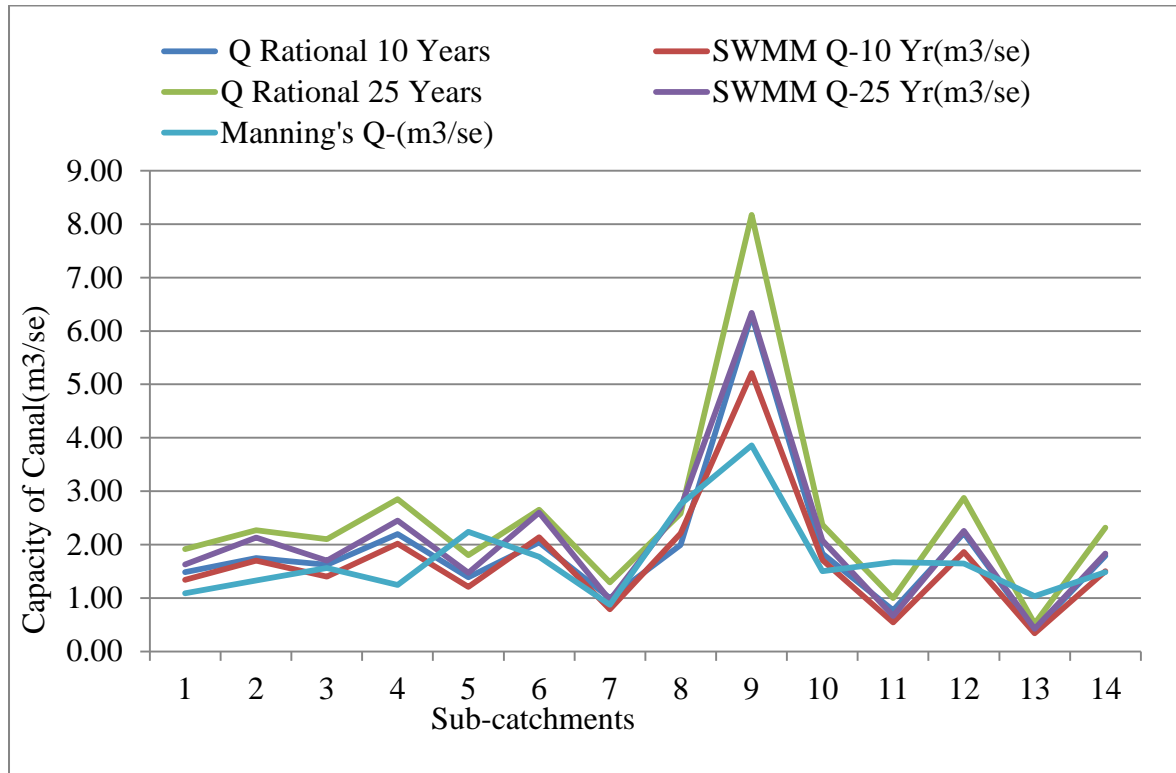


Figure 4.16: Discharge resulted from rational method, SWMM and Manning’s Formula  
 The sub-catchments of the study area to simulate by SWMM were first prepared by using ArcGIS and prepared for the SWMM model as shown from Figure 4.17. Then opening the prepared sub-catchments and filling the default values and constant parameters and simulating the software gave Figure 4.17 and different results.

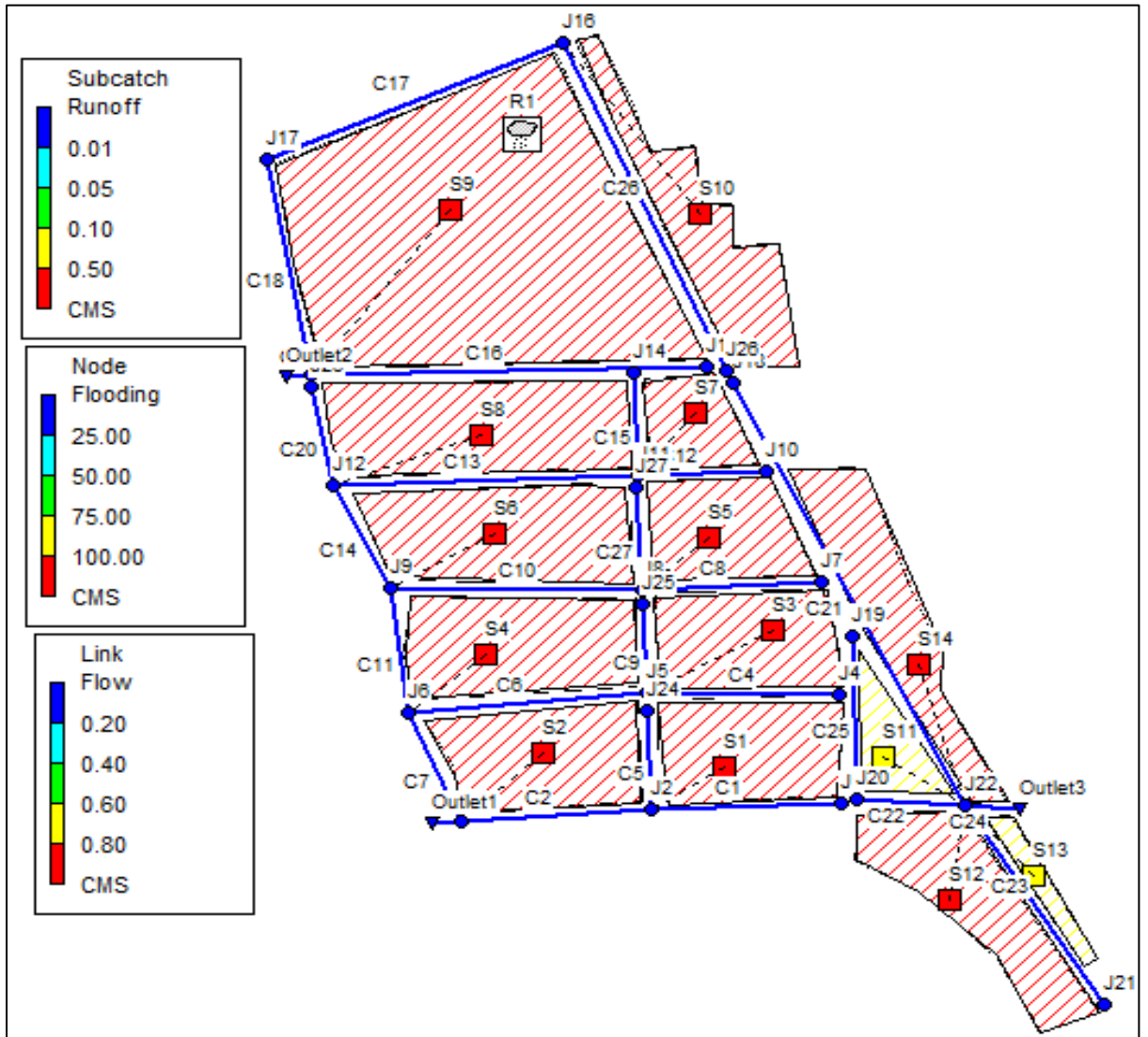


Figure 4.17: Map of Sub-catchments Modeled by SWMM

## 5. SUMMARY AND CONCLUSION

### 5.1. Summary and Conclusion

The study was aimed at performance assessment of stormwater drainage system of Dejen Town using Storm Water Management Model. To perform this task the necessary data such as primary and secondary data were collected from concerning bodies.

Rainfall data of study area was obtained from the National Meteorological Service Agency of Ethiopia for thirty-two years recorded data. The best-fit probability distribution selected was General Pareto distribution with the help of Hydrognomon 4 software and Easy-fit 5.6 Software. The existing urban drainage system performance was assessed and peak runoff volume was estimated with rational formula by using rainfall intensity obtained from the developed Dejen Town Intensity-Duration-Frequency curve for 10-years design period of storm capacity. In this case, most of the estimated drainage capacity calculated was greater than existing capacity of drainage structures and shows existing drainage capacity was inadequate to carry peak runoff. The performance assessment of urban storm drainage systems of Dejen Town was also evaluated with Storm Water Management Model 5.1. Most drainage discharges resulted from SWMM were greater than the existing drainage capacity of structures.

The results obtained from this thesis show the performance assessment of existing storm drainage systems of the Town was poor and the sewage systems were not proper to collect wastes originated from each household and most percent of the drainage systems were severe. This is because of lack of well-connected drainage networks, failure and breaking of drainage ditches, aging of drainage structures, and high traffic effect, unavailability of sufficient drainage structures in the proper place, most of the drainage structures were open drainage systems and exposed to failure as solid and liquid wastes were disposed into storm drainage systems. Another factor that leads Dejen Town to a drainage problem was the lack of waste management techniques to dispose of wastes extracted from each household.

According to the results obtained from engineering analysis of drainage structures of Dejen Town using rational formula and Storm Water Management Model as shown from Table 4.9, most of the storm drainage structures were inadequate and cannot carry existing runoff and have direct impact on road structures. The results obtained from rational method and

Storm Water Management Model shows that maximum flood occurs near CBE Abaysheleko branch, around 02 playgrounds, around Demis Hotel, around Dejen 02 primary school, around Dejen town kinder garden, around CBE Dejen branch, around Mulugojam hall, around Andinet business technology college, around Serkalem Hotel, and around Kurar Amba Hotel, and sub-catchments around Seble restaurant, around the new bus station, around Ambasel restaurant, and around St. George church the results indicated no over flooding.

## 5.2. Recommendations

According to the results obtained from engineering analysis of drainage structures of Dejen Town using rational formula and Storm Water Management Model, the storm drainage structures were inadequate and have no capacity to carry existing flood and have a direct impact on road structures. To solve the above flooding problems the following mitigation measures were recommended:

- Addition of drainage structures for secondary roadsides that have no drainage structures with proper dimension especially the roadsides below CBE Abaysheleko branch, around 02 playgrounds, around Demis Hotel, around Dejen 02 primary school, around Dejen town kinder garden, around CBE Dejen branch, around Mulugojam hall, around Andinet business Technology College, around Serkalem Hotel, and around Kurar Amba Hotel.
- Design and construction of well-connected structures to safely drain wastewater extracted from households and generated runoff in the sub-catchments into proper outfall.
- Using drainage structures for its purpose only by giving training and awareness to the community not to dump plastic materials into drainage structures, about the effect of poor drainage, and its health effect on the community.
- Adopting the culture of clearing silted and blocked structures, periodic repairing of drainage structures before total failure.
- Closed drainage structures were recommended to reduce sedimentation, growth of small plants, and collapse of drainage structures.
- Earthen canals must be lined with concrete or masonry to reduce erosion and sedimentation of canal.
- Generally, the developed Intensity-Duration-Frequency curve of Dejen Town must be used as an input for Dejen Town municipality and ERA to design drainage canals and other hydraulic structures rather than using ERA developed IDF curve for rainfall region A-2 because the developed IDF curve for Dejen Town was for specific area.

## REFERENCES

- Alain, Mailhot., Simon, Lachance., Guillaume, Talbot., Anne-Catherine Favre., 2013. Regional estimates of intense rainfall based on the Peak-Over-Threshold (POT) approach page-2
- Bansal, Neha., 2015. Causes and Impact of Urban Flooding in Dehradun. International Journal of Current Research Vol. 7.
- Bobbi and Thomas, 2014. Storm water management. what storm water management is and why it's important.
- Dagnachew, Adugna, Belete., 2011. "Road and urban storm drainage network integration in Addis Ababa." Journal of Engineering and technology research,.
- Diego, P., Arlex, S., Zoran,V., Solomon, S., and Wilmer, B., 2014. Multi-Objective-Rehabilitation of Urban Drainage Systems within Flood Risks Framework, 11<sup>th</sup> International Conference on Hydro informatics, New York City, USA: City University of New York (CUNY).
- Dihn, Balica., Popescu, and Jonoski., 2012. Climate change impact of flood hazard, vulnerability and risk of the Long Xuyen quadrangle in the Mekong delta.Int.J, River basin management, 10, 103-120.
- Engineering Toolbox-Resources, Tools and Basic information for Engineering and Design of Technical Applications ([www.Engineering Toolbox.com](http://www.EngineeringToolbox.com)).
- Environmental Protection Agency, EPA (2017). Storm Water Management Model (SWMM).
- Ethiopian Roads Authority (ERA), 2013. Hydrology, Ethiopian Roads Authority 5-17.
- Ethiopian Roads Authority (ERA), 2011. Drainage Design Manual, Addis Ababa.
- Ethiopian Roads Authority (ERA), 2002. Drainage design manual, Addis Ababa.
- European Geosciences Union (EGU), General Assembly Vienna Austria, May 2010. Open source software for the analysis of Hydrological data.
- Fadi, Ali., 2015. Urban classification by pixel and object-based approaches for very high resolution imagery. Page-9.
- Federal Democratic Republic of Ethiopia National Disaster Risk Management Commission, Early Warning and Emergency Response Directorate May, 2018.
- GISpipe, (2018, October 5th Friday). Retrieved from GIS based EPANET and SWMM integration,software.:<https://en.gispipe.com/index.php/2018/10/05/swmm-computational-methods>.

- German Technical Cooperation, (GTZ), 2006. Urban Drainage manual series on infrastructure Addis Ababa, Ethiopia, pp.17-54 Hydrologic Engineering Center (HEC), 2000. "HEC-RAS user's Manual".
- Gouri, R.L., and Srinivas, V.V, V.V., 2015. Reliability Assessment of a Stormwater Drain.
- Jenkins, K., 2016. Assessing surface water flood risk and management strategies under future climate change: An Agent-Based Model approach, Working Paper, 1-30.
- John, Freeborn., 2015. Decreasing Runoff and Increasing Stormwater Infiltration. Virginia Tech.
- Kalantari, K., 2011. Adaptation of Road Drainage Structures to Climate Change. TRITAL WRLIL 2061.
- Kapila Dahanayake., 2008. Natural Hazards and Disasters. Ministry of Education National Institute of Education for Social Cohesion, Disaster Risk Management and Psycho-social Care Programme. Page-4.
- Kokeb, Zena, Bessa., and Addisu, Adamu, Alemayehu., 2016. Assessment of Road and Surface Water Drainage Condition in Urban Ethiopia, with Special Reference of Assosa Town. 69-70. EUROPEAN ACADEMIC RESEARCH - Vol. IV, Issue 2 / May 2016.
- Kong, F., Ban, Y., James, P., and Dronova, I., 2017. Modeling stormwater management at the city district level in response to changes in land use and low impact development 14.
- Lewis, A. Rossman., 2017. Storm Water Management Model Reference Manual Volume II – Hydraulics 18.
- Lewis, A. Rossman., 2016. Storm Water Management Model Reference Manual Volume I . Hydrology (Revised), 86-118.
- Lewis, A. Rossman., 2015. Storm Water Management Model User's Manual Version 5.1.
- Lewis, A. Rossman., 2004. Storm Water Management Model user's manual (version 5.1).
- Muluken, Lebay., 2020. Techniques of filling missing values of daily and monthly rainfall data: A review article.
- Mumford, L., 1961. The city in history: its origins, its transformations, and its prospects.
- Nikita, Duggal., 2020. What is data processing Types, methods, steps, and examples for data processing cycle?
- Nirman, Bhagat., 2017. Flood Frequency Analysis Using Gumbel's Distribution method: A case study of Lower Mahi Basin. Volume 6. Page 52-53.

- Omer, Levend., 2017. Outlier Detection in Extreme Value Series. Vol. 4 Issues 5, May - 2017
- Qomariyatus, Sholihah., Wahyudi, Kuncoro., Sri,Wahyuni., Sisilia, Puni, Suwandi., Elisa, Dwi, Feditasari., 2020. The analysis of the causes of flood disasters and their impacts in the perspective of environmental law. IOP Conf. Series: Earth and Environmental Science 437. page-2
- Raihan, K., 2014. Flood Frequency Analysis by Probability and Stochastic Method for Padma River, Bangladesh, American Journal of Civil Engineering. Vol. 2.
- Reid, D., 1991. Paris sewers and sewer men, Harvard University Press, Cambridge, MA.
- Reston, V. U., 2016. "Runoff (surface water runoff)", USGS Water Science School.
- Steven, J., Burian, and Findlay, G.Edwards., 2002. Historical Perspectives of Urban Drainage.
- Subramanya, D. K.-H., 2008. Engineering Hydrology, Tata Mc Graw-Hill publishing company.
- Sundara, Kumar., T Santhi, P Manoj, M Anjaneya, and Praveen., 2015. Stormwater Drainage Design (Case Study Vijayawada).
- Tewodros, Assefa., and Abdusselam, Altunkaynak., 2018. Impact of climate change on the trends of extreme rainfall indices and values of maximum precipitation at Olimpiyate station Istambul, Turkey.
- United States Geological Survey. Atlanta, GA, 2009. "The effects of urbanization on water quality.
- United States Geological Survey. Atlanta, GA, 2016. "Surface Runoff - The Water Cycle".
- Ven Te, Chow., David, R., Maidment, Larry, W., May, 2012. Applied Hydrology. Fourth edition Tata McGraw Hill 497-498.
- Ven Te, Chow., 1988. Applied Hydrology. McGraw Hill Book Co. New York.
- Violina, Basumatary., and Briti, Sundar, Sil., 2017. Generation of Rainfall Intensity Duration Frequency curves for the Barak River Basin.
- Warati, Tamene., and Getachew, Kebede., 2015. Assessment of the effect of urban road surface drainage: A case study at Ginjo Guduru Kebele of Jimma Town. Vol-3, No.4, pp 164-173.
- Wisuwat, Taesombat, A. N., 2009. Areal rainfall estimation using spatial interpolation techniques 268-275.

## APPENDICES

Appendix 1: Mean monthly rainfall, cumulative rainfall of stations, 24-hr rainfall depth, and comparison of IDF Curve

Table 1a: Dejen Town mean monthly rainfall from (1987-2018) in (mm)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Sum
1987	0	16.5	27.2	9.5	23.2	25	55.3	41.9	25.7	32.5	0	0.9	257.7
1988	8.7	8.9	0	9.8	3.9	13.1	44.7	63.6	40.4	30.6	0	0.3	224.0
1989	0	13.7	28.5	44.3	14.7	32.8	28.2	37.5	38.1	34.3	0	17	289.1
1990	0.8	8.1	24.6	36.7	9.9	22.4	45.3	48.5	31.2	20	0	0	247.5
1991	1.7	2.4	3.6	6.6	24.1	31.9	38.7	41.1	34.6	11.1	0	10.1	205.9
1992	35.2	16.8	7.8	24.6	16.4	57.5	33.2	39.8	33.1	17.8	9.3	0	291.5
1993	8.8	5.3	14.5	35	44.8	27.2	25.9	30.7	42.4	22.5	0	0	257.1
1994	0	0	19.9	4.4	21.5	18.4	65	44.4	34.5	0	1.1	0	209.2
1995	0	0	13.6	14.5	17.1	29.7	78.3	95.5	103.4	3.3	4.7	17	377.1
1996	3.3	2.3	17.9	33.6	50.8	61.3	104.6	35.8	23.4	14	15	7.2	369.2
1997	24.4	0	48.8	40.2	18.4	48.2	59.2	36.3	12.1	78.5	62.5	13.3	441.9
1998	0.4	10.6	20.9	2.1	44.2	35.5	70.1	76.1	62.4	47.7	0	0	370.0
1999	2.8	0	0	16.6	30.4	29.1	72.7	89.8	34.5	48.8	2.1	3.9	330.7
2000	0	0	0	35.6	28.9	24.8	56.1	64.5	45	35.8	18	2.1	310.8
2001	0	7.2	42.9	20.7	23.2	33.1	72.1	41.4	59.3	33.3	2.2	2.5	337.9
2002	20.5	12.3	34.1	11.1	9.1	38.2	64.5	46.5	48.1	0	0	4.1	288.5
2003	0.8	28.2	24.8	39.6	1.9	29.1	32.1	56.7	24.9	0	15.3	9.9	263.3
2004	5.1	7.1	13.9	14.6	4.6	22.2	37.2	28.1	17.1	29.1	9.8	0	188.8
2005	3.1	0	20.1	19.4	24.7	34.9	44.5	29.4	32.6	38.1	5.4	0	252.2
2006	7.1	7.1	29.7	46.7	39.1	34.3	25.1	58.1	56.4	26.9	4.4	21.1	356.0
2007	8.7	34.6	14.5	23.9	43.7	26.3	40.1	28.7	32.5	7.2	0	0	260.2
2008	0	0	0	31.2	26.9	57.4	60.5	41.4	69.3	35.5	42.2	0	364.4
2009	0	1.6	38.1	31.8	13.5	20.6	31.8	36.8	40.9	33.5	9.9	7.1	265.6
2010	0	2.6	46.7	26.1	34.4	16.2	33.3	51.1	15.3	13.7	16.5	0	255.9
2011	0	0	24.4	13.1	13.13	24.2	35.3	71.8	31.5	0	31.9	0	245.3

2012	0	0	24.4	13.1	17.5	27.9	33.5	30.2	49.5	1.4	11.8	0	209.3
2013	0	0	8	4.4	26.2	33.3	37.9	26.9	26.2	49.2	8.1	0	220.2
2014	0	30.1	6.6	26.3	38.6	34.2	27.2	36.2	49.4	36.4	28.6	0	313.6
2015	0	2.7	4.1	0	37.2	44.9	27.7	61.9	41.9	15.5	41.4	20.9	298.2
2016	16.1	6.2	28.4	40.5	16.1	18.0	72.0	34.5	41.5	49.3	41.8	44.2	408.6
2017	0.4	4.2	6.7	9.5	13.8	15.4	14.3	16.1	12.9	22.0	1.7	0	117.0
2018	0	4.8	6.8	11	12.1	22	33.2	22.3	20.3	7.2	13.1	0	152.8

Table 1b: Cumulative Rainfall of each station with average station in (mm)

Cummu.of 5 stations	D/markos Station	Dejen Station	Yetnora Station	Abysheleko
1040	1273	930	819	1349
2356	2596	2021	2236	2862
3626	4040	3195	3589	4112
4952	5353	4197	5330	5355
6078	7053	5096	6562	5976
7104	8310	6022	7458	6644
8411	10119	7182	8742	7518
9573	11320	8256	9877	8583
10600	12569	9268	10640	9599
12127	14159	10554	12165	11424
13545	15677	12112	13103	13036
14870	16880	13531	14361	14252
16230	18224	15297	15549	15316
17661	19625	16917	16777	16484
19239	20999	18427	19285	17556
20396	22305	19658	20429	18504
21522	23515	20800	21302	19873
22628	24834	21809	22397	21247
23755	26086	22891	23365	22430
25165	27607	24379	24979	23688
26478	29008	25737	26199	25052

27764	30323	27418	27268	26300
28916	31588	28600	28291	27414
30268	32951	29708	29503	29300
31508	34435	30953	30686	30532
32700	35707	32114	31814	31856
34122	36934	33339	32976	34268
35258	38272	34609	33874	35239
36262	39331	35678	34770	36225
37340	40498	36531	35993	36976
38625	41935	37544	37059	37897
40003	43265	38716	38176	39620

Sample calculation for comparison of Intensity of self and ERA

Self-rainfall Intensity

Rt =7.29, and duration t=5 min=0.083333 hr

$$I = \frac{Rt}{t} = \frac{7.29}{0.083333} = \underline{87 \text{ mm/hr}}$$

ERA Rainfall intensity

R24 for T-2=51.92

$$Rt = \frac{t(0.3+24)^{0.92}}{24(0.3+t)^{0.92}} * R24 = \frac{0.083333(0.3+24)^{0.92}}{24(0.3+0.083333)^{0.92}} * R24 = \frac{1.5688}{9.9335} * 51.92 = \underline{8.2}$$

$$I = \frac{Rt}{t} = \frac{8.2}{0.08333} = \underline{98}$$

Table 1c: Dejen Town Design Rainfall of shorter duration

Duration(min)	T-2	T-5	T-10	T-25	T-50	T-100
0.083333	7.29	9.15	10.58	12.48	13.93	15.40
0.166667	12.17	15.28	17.65	20.83	23.25	25.70
0.25	15.69	19.70	22.76	26.86	29.99	33.14
0.333333	18.37	23.07	26.66	31.45	35.11	38.81
0.416667	20.50	25.74	29.74	35.09	39.17	43.30
0.5	22.23	27.91	32.25	38.05	42.48	46.95
0.583333	23.67	29.73	34.35	40.53	45.25	50.01
0.666667	24.90	31.27	36.13	42.63	47.59	52.60

---

0.75	25.96	32.60	37.67	44.45	49.62	54.84
0.833333	26.89	33.76	39.02	46.03	51.39	56.80
0.916667	27.71	34.79	40.21	47.44	52.96	58.53
1	28.44	35.71	41.27	48.69	54.36	60.08
1.083333	29.10	36.54	42.22	49.82	55.62	61.47
1.166667	29.69	37.29	43.09	50.84	56.76	62.73
1.25	30.24	37.97	43.88	51.77	57.80	63.88
1.333333	30.74	38.60	44.60	52.62	58.75	64.93
1.416667	31.20	39.17	45.27	53.41	59.63	65.90
1.5	31.62	39.71	45.89	54.14	60.44	66.80
1.583333	32.02	40.21	46.46	54.82	61.20	67.64
1.666667	32.39	40.67	47.00	55.45	61.90	68.42
1.75	32.73	41.10	47.50	56.04	62.56	69.15
1.833333	33.06	41.51	47.97	56.59	63.18	69.83
1.916667	33.36	41.89	48.41	57.12	63.77	70.48
2	33.65	42.26	48.83	57.61	64.32	71.09
2.083333	33.92	42.60	49.23	58.08	64.84	71.66
2.166667	34.18	42.92	49.60	58.52	65.34	72.21
2.25	34.43	43.23	49.96	58.94	65.81	72.73
2.333333	34.66	43.53	50.30	59.34	66.25	73.22
2.416667	34.89	43.81	50.62	59.73	66.68	73.70
2.5	35.10	44.08	50.93	60.09	67.09	74.15
2.583333	35.30	44.33	51.23	60.44	67.48	74.58
2.666667	35.50	44.58	51.52	60.78	67.85	74.99
2.75	35.69	44.81	51.79	61.10	68.21	75.39
2.833333	35.87	45.04	52.05	61.41	68.56	75.77
2.916667	36.04	45.26	52.30	61.71	68.89	76.14
3	36.21	45.47	52.55	61.99	69.21	76.49

---

Table 1d: Comparison of IDF curve developed for Dejen Town and IDF curve developed by ERA for RR-A2

	Mine	ERA	Mine	ERA	Mine	ERA	Mine	ERA	Mine	ERA	Mine	ERA
Duration (min)	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	87	98	110	124	127	141	150	162	167	178	185	194
10	73	82	92	104	106	118	125	136	140	149	154	162
15	63	71	79	89	91	101	107	117	120	128	133	139
20	55	62	69	78	80	89	94	102	105	112	116	122
25	49	55	62	70	71	79	84	91	94	100	104	109
30	44	50	56	63	65	72	76	83	85	91	94	99
35	41	46	51	58	59	65	69	75	78	83	86	90
40	37	42	47	53	54	60	64	69	71	76	79	83
45	35	39	43	49	50	56	59	64	66	71	73	77
50	32	36	41	46	47	52	55	60	62	66	68	72
55	30	34	38	43	44	49	52	56	58	62	64	67
60	28	32	36	40	41	46	49	53	54	58	60	63
65	27	30	34	38	39	43	46	50	51	55	57	60
70	25	29	32	36	37	41	44	47	49	52	54	56
75	24	27	30	34	35	39	41	45	46	49	51	54
80	23	26	29	33	33	37	39	43	44	47	49	51
85	22	25	28	31	32	36	38	41	42	45	47	49
90	21	24	26	30	31	34	36	39	40	43	45	47
95	20	23	25	29	29	33	35	38	39	41	43	45
100	19	22	24	28	28	31	33	36	37	40	41	43
105	19	21	23	27	27	30	32	35	36	38	40	42
110	18	20	23	26	26	29	31	33	34	37	38	40
115	17	20	22	25	25	28	30	32	33	35	37	39
120	17	19	21	24	24	27	29	31	32	34	36	37
125	16	18	20	23	24	26	28	30	31	33	34	36
130	16	18	20	22	23	25	27	29	30	32	33	35

135	15	17	19	22	22	25	26	28	29	31	32	34
140	15	17	19	21	22	24	25	28	28	30	31	33
145	14	16	18	20	21	23	25	27	28	29	30	32
150	14	16	18	20	20	23	24	26	27	29	30	31
155	14	15	17	19	20	22	23	25	26	28	29	30
160	13	15	17	19	19	21	23	25	25	27	28	30
165	13	15	16	18	19	21	22	24	25	26	27	29
170	13	14	16	18	18	20	22	24	24	26	27	28
175	12	14	16	18	18	20	21	23	24	25	26	27
180	12	14	15	17	18	19	21	22	23	25	25	27

Table 1e: Return period vs 24-hr rainfall depth for General Pareto, Gumbel and Log-person-III for study Area

Return period ( T)	GPD	Gumbel	person-III
2	46.154	48.102	47.024
5	57.956	59.237	57.742
10	66.975	66.608	65.832
25	79.016	75.923	77.166
50	88.217	82.833	86.432
100	97.498	89.692	96.422

Sample calculation of estimated drainage capacity of SC-1

Area = 7.92 ha, Cw=0.64, H1=2433, H2= 2438, H2-H1= 5, n= 0.04, L= 1245

Slope= 5/1245=0.004,

$R = 1.523$ ,  $V = 1/n * S^{1/2} * R^{2/3} = 25 * 0.004^{0.5} * 1.523^{0.667}$ ,  $V = \underline{2.1m/se}$

$Tt = \frac{L}{3600V} = \frac{1245}{3600 * 2.1} = \underline{0.165 \text{ hr}}$

Inlet Time,  $Tci = \frac{3.64(1.1 - Cw) * (L/1000)^{0.83}}{(H)^{0.33}}$

L = flow length from the remotest point to the point t of interest (km).

Cw=weighted runoff coefficient= $AiCi/At = 0.64$  for sc-1

$$T_{ci} = (3.64 * (1.1 - 0.64) * (1.245 / 1000))^{0.83} / (5)^{0.33} = \underline{0.003821 \text{ hr}}$$

$$T_c = T_t + T_{ci} = 0.1647 + 0.003821 = 0.16852 \text{ hr} = \underline{10.12 \text{ min}}$$

By considering duration equals to time of concentration, Rainfall intensity was interpolated from IDF curve i.e. I-10=105.563, I-25= 124.24, I-50 = 139.04, I-100= 153.67 mm/hr

Then, Runoff Q for sub catchment 1 for design period 10 year, 25 year, 50 year and 100 year was calculated by Rational formula  $Q = 0.00278C * C_f * I * A$

$$Q-10 = 1.4815 \text{ m}^3/\text{se}$$

$$Q-25 = 1.92 \text{ m}^3/\text{se}$$

$$Q-50 = 2.342 \text{ m}^3/\text{se}$$

$$Q-100 = 2.696 \text{ m}^3/\text{se}$$

#### Appendix 2: Estimated drainage capacity and existing drainage capacity

Table2a. Estimated drainage capacity of study Area by Rational method

SC-ID	Area (ha)	TC-min	Cw	Intensity 10-yr	Intensity 25-yr	Intensity 50-yr	Intensity 100-yr	Q 10-yr	Q 25-yr	Q 50-yr	Q 100-yr	V-m/s
SC-1	7.92	10.12	0.64	105.56	124.24	139.04	153.67	1.48	1.92	2.34	2.70	2.1
SC-2	9.29	7.50	0.58	116.43	137.36	153.35	169.49	1.75	2.27	2.76	3.18	3.1
SC-3	7.71	8.43	0.67	112.64	132.89	148.37	163.98	1.62	2.10	2.56	2.95	2.12
SC-4	9.91	5.81	0.65	123.53	145.74	162.71	179.82	2.20	2.85	3.47	4.00	3.61
SC-5	7.15	10.00	0.66	105.92	124.96	139.51	154.19	1.39	1.80	2.19	2.53	2.17
SC-6	10.49	7.50	0.60	116.43	137.36	153.35	169.49	2.04	2.65	3.23	3.72	3.25
SC-7	4.17	6.16	0.71	122.06	144.00	160.77	177.68	1.00	1.30	1.58	1.82	3.21
SC-8	10.92	8.26	0.58	113.23	133.59	149.14	164.83	1.99	2.58	3.15	3.62	3.21
SC-9	34.11	11.30	0.65	102.05	120.40	134.42	148.57	6.30	8.18	9.96	11.46	3.23
SC-10	10.07	13.85	0.69	94.48	111.46	124.44	137.53	1.83	2.37	2.89	3.33	1.79

SC-11	3.09	9.34	0.82	108.69	128.23	143.17	158.23	0.77	1.00	1.21	1.40	1.48
SC-12	8.98	6.59	0.74	120.25	141.87	158.39	175.05	2.21	2.87	3.50	4.03	2.71
SC-13	1.63	5.42	0.70	125.17	147.61	164.87	182.21	0.40	0.52	0.63	0.72	2.39
SC-14	7.92	10.93	0.79	103.16	121.58	135.87	150.17	1.79	2.32	2.83	3.25	2.35

Table 2b: Frequency factor for Rational Formula

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

(Source: ERA 2002)

Table 2c: Capacity of existing drainage canal by Manning's Formula

Drainage near CBE Abaysheleko branch

SC-1	Geometry	Material	Y (m)	B (m)	A (m <sup>2</sup> )	n	Slope (%)	P (m)	R (m)	V (m/se)	Q (m <sup>3</sup> /sec)
DL-1	Rectangular	masonry	1.5	0.6	0.9	0.025	0.001	3.6	0.250	0.502	0.452
DL-2	Rectangular	Earth drain	0.7	0.6	0.42	0.035	0.002	2	0.210	0.451	0.190
DL-3	Rectangular	Masonry drain	1.2	0.7	0.84	0.025	0.001	3.1	0.271	0.529	0.445
										1.482	1.086

Drainage around Dejen 02 playground

SC-2	Geometry	Material	Y (m)	b (m)	A (m <sup>2</sup> )	n	Slope (%)	P (m)	R (m)	V (m/se)	Q (m <sup>3</sup> /sec)
DL-1	Rectangular	masonry	1.5	0.6	0.9	0.025	0.002	3.6	0.250	0.710	0.639
DL2	Rectangular	Earth drain	0.8	0.4	0.32	0.035	0.002	2	0.160	0.376	0.120
DL-3	Rectangular	masonry	1.5	0.6	0.9	0.025	0.001	3.6	0.250	0.502	0.452
DL-4	Rectangular	Earth drain	0.8	0.5	0.4	0.035	0.001	2.1	0.190	0.299	0.120
										1.887	1.330

Drainage around Demis Hotel

SC-3	Geometry	Material	Y (m)	b (m)	A (m <sup>2</sup> )	n	Slope (%)	P (m)	R (m)	V (m/se)	Q (m <sup>3</sup> /sec)
DL-1	Rectangular	masonry	1.4	0.6	0.84	0.025	0.001	3.4	0.247	0.498	0.418
DL-2	Rectangular	masonry	1.2	0.7	0.84	0.025	0.001	3.1	0.271	0.529	0.445
DL-3	Rectangular	masonry	1.4	0.7	0.98	0.025	0.001	3.5	0.280	0.541	0.530
DL-4	Rectangular	masonry	0.8	0.5	0.4	0.025	0.001	2.1	0.190	0.419	0.167
										1.987	1.561

Drainage around Dejen 02 primary school

SC-4	Geometry	Material	Y (m)	B (m)	A (m <sup>2</sup> )	n	Slope (%)	P (m)	R (m)	V (m/se)	Q (m <sup>3</sup> /sec)
DL-1	Rectangular	masonry	0.8	0.5	0.4	0.025	0.002	2.1	0.190	0.592	0.237
DL-2	Rectangular	masonry	1.2	0.55	0.66	0.025	0.002	2.95	0.224	0.659	0.435
DL-3	Rectangular	masonry	0.9	0.65	0.585	0.025	0.002	2.45	0.239	0.688	0.403
DL-4	Rectangular	masonry	0.8	0.5	0.4	0.025	0.001	2.1	0.190	0.419	0.167

											1.939	1.242
--	--	--	--	--	--	--	--	--	--	--	-------	-------

Drainage around Seble restaurant

SC-5	Geometry	Material	Y (m)	b (m)	A (m <sup>2</sup> )	n	Slope (%)	P (m)	R (m)	V (m/se)	Q (m <sup>3</sup> /sec)
DL-1	Rectangular	masonry	1.2	0.7	0.84	0.025	0.002	3.1	0.27	0.749	0.629
DL-2	Rectangular	Earth drain	0.8	0.4	0.32	0.035	0.001	2	0.16	0.266	0.085
DL-3	Rectangular	masonry	1	0.4	0.4	0.025	0.001	2.4	0.17	0.383	0.153
DL-4	Rectangular	masonry	1.5	0.7	1.05	0.025	0.002	3.7	0.28	0.772	0.811
DL-5	Rectangular	masonry	0.7	0.6	0.42	0.025	0.001	2	0.21	0.447	0.188
DL-6	Rectangular	masonry	0.7	0.6	0.42	0.025	0.001	2	0.21	0.447	0.188
DL-7	Rectangular	masonry	0.7	0.6	0.42	0.025	0.001	2	0.21	0.447	0.188
										3.510	2.241

Drainage around Dejen town kindergarten

SC-6	Geometry	Material	Y (m)	b (m)	A (m <sup>2</sup> )	n	Slope (%)	P (m)	R (m)	V (m/se)	Discharge (m <sup>3</sup> /sec)
DL-1	Rectangular	masonry	1.2	0.7	0.84	0.025	0.001	3.1	0.271	0.529	0.445
DL-2	Rectangular	Earth drain	0.7	0.6	0.42	0.035	0.001	2	0.210	0.319	0.134
DL-3	Rectangular	masonry	1.4	0.8	1.12	0.025	0.001	3.6	0.311	0.581	0.650
DL-4	Rectangular	masonry	1.2	0.8	0.96	0.025	0.001	3.2	0.300	0.567	0.544
										1.996	1.773

Drainage around CBE Dejen branch/Meaza hotel

SC-7	Geometry	Material	Y (m)	b (m)	A (m <sup>2</sup> )	n	Slope (%)	P (m)	R (m)	V (m/se)	Q (m <sup>3</sup> /sec)
DL-1	Rectangular	open masonry	1.2	0.7	0.84	0.025	0.001	3.1	0.271	0.529	0.445
DL-2	Rectangular	Earth drain	0.7	0.6	0.42	0.022	0.001	2	0.210	0.508	0.213
DL-3	Rectangular	Earth drain	0.7	0.6	0.42	0.022	0.001	2	0.210	0.508	0.213
										1.545	0.871

Drainage around new bus station

SC-8	Geometry	Material	Y (m)	b (m)	A (m <sup>2</sup> )	n	Slope (%)	P (m)	R (m)	V (m/se)	Q (m <sup>3</sup> /sec)
DL-1	Rectangular	Open masonry	1.2	0.7	0.84	0.025	0.001	3.1	0.271	0.529	0.445
DL-2	Rectangular	Earth drain	1.2	0.7	0.84	0.035	0.001	3.1	0.271	0.378	0.318
DL-3	Rectangular	Earth drain	0.7	0.6	0.42	0.035	0.001	2	0.210	0.319	0.134
DL-4	Rectangular	Earth drain	0.7	0.6	0.42	0.035	0.001	2	0.210	0.319	0.134
DL-5	Rectangular	Earth drain	0.7	0.6	0.42	0.035	0.001	2	0.210	0.319	0.134
DL-6	Rectangular	Earth drain	0.7	0.6	0.42	0.035	0.001	2	0.210	0.319	0.134
DL-7	Rectangular	Earth drain	0.7	0.6	0.42	0.035	0.001	2	0.210	0.319	0.134
DL-8	Rectangular	Earth drain	0.7	0.6	0.42	0.035	0.001	2	0.210	0.319	0.134
DL-9	Rectangular	Earth drain	0.7	0.6	0.42	0.035	0.001	2	0.210	0.319	0.134
DL-10	Rectangular	Earth drain	0.7	0.6	0.42	0.035	0.001	2	0.210	0.319	0.134
DL-11	Rectangular	open masonry	1.4	0.8	1.12	0.025	0.002	3.6	0.311	0.821	0.920

										4.281	2.754
--	--	--	--	--	--	--	--	--	--	-------	-------

Drainage around Mulugojam hall

SC-9	Geometry	Material	Y (m)	b (m)	A (m <sup>2</sup> )	n	Slope (%)	P (m)	R (m)	V (m/se)	Q (m <sup>3</sup> /sec)
DL-1	Rectangular	masonry	1.2	0.6	0.72	0.025	0.001	3	0.24	0.488	0.352
DL2	Rectangular	Earth drain	0.8	0.5	0.4	0.03	0.001	2.1	0.19	0.349	0.140
DL-3	Rectangular	Earth drain	0.8	0.6	0.48	0.022	0.001	2.2	0.22	0.521	0.250
DL-4	Rectangular	Earth drain	0.8	0.6	0.48	0.03	0.001	2.2	0.22	0.382	0.183
DL-5	Rectangular	Earth drain	1	0.6	0.6	0.03	0.002	2.6	0.23	0.561	0.336
DL-6	Rectangular	masonry	1.2	0.5	0.6	0.025	0.002	2.9	0.21	0.625	0.375
DL-7	Rectangular	Earth drain	0.8	0.5	0.4	0.03	0.001	2.1	0.19	0.349	0.140
DL-8	Rectangular	Earth drain	0.8	0.6	0.48	0.022	0.001	2.2	0.22	0.521	0.250
DL-9	Rectangular	Earth drain	1	0.5	0.5	0.022	0.002	2.5	0.20	0.695	0.347
DL-10	Rectangular	Earth drain	0.8	0.6	0.48	0.03	0.001	2.2	0.22	0.382	0.183
DL-11	Rectangular	Earth drain	0.8	0.6	0.48	0.03	0.001	2.2	0.22	0.382	0.183
DL-12	Rectangular	open masonry	0.8	0.6	0.48	0.025	0.002	2.2	0.22	0.648	0.311
DL-13	Rectangular	open masonry	0.8	0.6	0.48	0.025	0.002	2.2	0.22	0.648	0.311
DL-14	Rectangular	open masonry	0.8	0.6	0.48	0.025	0.002	2.2	0.22	0.648	0.311
DL-15	Rectangular	Earth drain	0.8	0.6	0.48	0.03	0.001	2.2	0.22	0.382	0.183
										7.579	3.856

Drainage around Andinet business Technology College

SC-10	Geometry	Material	Y (m)	b (m)	A (m <sup>2</sup> )	n	Slope (%)	P (m)	R (m)	V (m/se)	Q (m <sup>3</sup> /sec)
DL-1	Rectangular	masonry	1.2	0.6	0.72	0.025	0.001	3	0.240	0.488	0.352
DL-2	Rectangular	masonry	1.2	0.7	0.84	0.025	0.001	3.1	0.271	0.529	0.445
DL-3	Rectangular	masonry	1.5	0.8	1.2	0.025	0.001	3.8	0.316	0.586	0.704
										1.604	1.500

Drainage around Ambasel restaurant

SC-11	Geometry	Material	Y (m)	b (m)	A (m <sup>2</sup> )	n	Slope (%)	P (m)	R (m)	V (m/se)	Q (m <sup>3</sup> /sec)
DL-1	Rectangular	Open masonry	1.4	0.7	0.98	0.025	0.001	3.5	0.280	0.541	0.530
DL-2	Rectangular	Open masonry	1.5	0.8	1.2	0.025	0.001	3.8	0.316	0.586	0.704
DL-3	Rectangular	Open masonry	1.3	0.65	0.845	0.025	0.001	3.25	0.260	0.515	0.435
										1.643	1.669

Drainage near Serkalem Hotel

SC-12	Geometry	Material	Y (m)	b (m)	A (m <sup>2</sup> )	n	Slope (%)	P (m)	R (m)	V (m/se)	Q (m <sup>3</sup> /sec)
DL-1	Rectangular	Open masonry	1.5	0.6	0.9	0.025	0.001	3.6	0.250	0.502	0.452
DL-2	Rectangular	Open masonry	1.5	0.7	1.05	0.025	0.001	3.7	0.284	0.546	0.573
DL-3	Rectangular	Concrete (cement)	0.8	0.4	0.32	0.012	0.002	2	0.160	1.098	0.351

DL-4	Rectangular	Earth drain	0.7	0.6	0.42	0.035	0.001	2	0.210	0.319	0.134
DL-5	Rectangular	Earth drain	0.7	0.6	0.42	0.035	0.001	2	0.210	0.319	0.134
										2.784	1.644

Drainage around St. Gorge church

SC-13	Geometry	Material	Y (m)	b (m)	A (m <sup>2</sup> )	n	Slope (%)	P (m)	R (m)	V (m/se)	Q (m <sup>3</sup> /sec)
DL-1	Rectangular	masonry	1.6	0.7	1.12	0.025	0.002	3.9	0.287	0.778	0.872
DL-2	Rectangular	Earth drain	0.7	0.5	0.35	0.022	0.001	1.9	0.184	0.465	0.163
										1.243	1.035

Drainage around kurar amba hotel

SC-14	Geometry	Material	Y (m)	b (m)	A (m <sup>2</sup> )	n	slope (%)	P (m)	R (m)	V (m/se)	Q (m <sup>3</sup> /sec)
DL-1	Rectangular	masonry	1.5	0.8	1.2	0.025	0.002	3.8	0.316	0.826	0.992
DL2	Rectangular	masonry	1.2	0.6	0.72	0.025	0.002	3	0.240	0.688	0.495
										1.514	1.487

Table 2d: Drainage condition around selected sub-catchments

Table 2d1: Drainage condition around CBE Abayshaleko branch to Fasileds Bridge

Drainage ID	Geometry	Drainage & pavement	Y (m)	B (m)	Length (m)	Existing condition	% deterioration
DL-1	Rectangular	Open Masonry	1.5	0.6	331.4	V. Good	25.02
DL-2	Rectangular	Earth drain	0.7	0.6	206.9	Poor	15.62
DL-3	Rectangular	Open Masonry	1.2	0.7	55.1	Good	4.16
DL-4	Rectangular	Open masonry	1.5	0.6	300.0	V. Good	22.65

DL-5	Rectangular	Earth drain	0.8	0.4	215.5	Good	16.27
DL-6	Rectangular	Open masonry	1.5	0.6	118.1	Good	8.91
DL-7	Rectangular	Earth drain	0.8	0.5	97.7	poor	7.38

Sample calculation

V.Good=25.02+22.65=48%, good=4.16+16.27+8.91=29%, and poor=15.62+7.38=23%.

Table 2d2: Drainage condition from Demis Hotel to 02 Primary school

Drainage code	Geometry	Drainage & pavement	Y (m)	B (m)	Length (m)	Existing condition	% Deterioration
DL-1	Rectangular	open masonry	1.4	0.6	345.7	poor	18.50
DL-2	Rectangular	open masonry	1.2	0.7	129.4	V. Good	6.93
DL-3	Rectangular	open masonry	1.4	0.7	132.7	good	7.10
DL-4	Rectangular	open masonry	0.8	0.5	203.2	poor	10.87
DL-5	Rectangular	open masonry	0.8	0.5	320.3	poor	17.14
DL-6	Rectangular	open masonry	1.2	0.55	400.3	poor	21.42
DL-7	Rectangular	open masonry	0.9	0.65	105.3	Very Poor	5.63
DL-8	Rectangular	open masonry	0.8	0.5	231.9	Good	12.41

Table 2d3: Drainage condition from Seble hotel to Dejen kindergarten

Drainage ID	Geometry	Drainage and pavement	Y (m)	B (m)	Length (m)	Existing condition	% deterioration
DL-1	Rectangular	open masonry	1.2	0.7	310.6	Very Good	12.92
DL-2	Rectangular	Earth drain	0.8	0.4	202.6	Poor	8.43
DL-3	Rectangular	open masonry	1	0.4	105.6	Poor	4.39
DL-4	Rectangular	open masonry	1.5	0.7	400.0	Good	16.64
DL-5	Rectangular	open masonry	0.7	0.6	265.8	Poor	11.06
DL-6	Rectangular	open masonry	0.7	0.6	99.8	Poor	4.15
DL-7	Rectangular	open masonry	0.7	0.6	100.8	Poor	4.19
DL-8	Rectangular	open masonry	1.2	0.7	139.7	Very Good	5.81

DL-9	Rectangular	Earth drain	0.8	0.6	204.5	Poor	8.51
DL-10	Rectangular	open masonry	1.4	0.8	440.3	poor	18.32
DL-11	Rectangular	open masonry	1.2	0.8	133.7	Moderate	5.56

Table 2d4: Drainage condition from CBE Dejen branch to new bus station

Drainage ID	Geometry	Drainage & pavement	Y (m)	B (m)	Length (m)	Existing condition	% deterioration
DL-1	Rectangular	open masonry	1.2	0.7	219.6	V. Good	8.82
DL-2	Rectangular	Earth drain	0.7	0.6	101.1	Moderate	4.06
DL-3	Rectangular	Earth drain	0.7	0.6	201.3	Poor	8.08
DL-4	Rectangular	open masonry	1.2	0.7	514.2	V. Good	20.66
DL-5	Rectangular	Earth drain	1.2	0.7	205.6	Good	8.26
DL-6	Rectangular	Earth drain	0.7	0.6	103.1	Good	4.14
DL-7	Rectangular	Earth drain	0.7	0.6	102.7	Poor	4.13
DL-8	Rectangular	Earth drain	0.7	0.6	203.0	Poor	8.16
DL-9	Rectangular	Earth drain	0.7	0.6	100.2	Poor	4.03
DL-10	Rectangular	Earth drain	0.7	0.6	183.7	Poor	7.38
DL-11	Rectangular	Earth drain	0.7	0.6	202.5	Poor	8.13
DL-12	Rectangular	Earth drain	0.7	0.6	195.7	V. Poor	7.86
DL-13	Rectangular	Earth drain	0.7	0.6	70.1	Poor	2.82
DL-14	Rectangular	open masonry	1.4	0.8	86.6	poor	3.48

Appendix 3: Distribution methods and Goodness of fit Test

Table 3a: Goodness of Fit – Test Summary by Easy fit 5.6 results and rank of distribution methods

#	Distribution	Kolmogorov Smirnov		Anderson Darling		Chi-Squared	
		Statistic	Rank	Statistic	Rank	Statistic	Rank
1	Gen. Pareto	0.05592	1	0.64848	1	8.6285	11
2	Weibull (3P)	0.06342	2	4.3425	20	N/A	
3	Log-Logistic (3P)	0.06413	3	1.1261	6	2.5695	3
4	Exponential (2P)	0.06512	4	1.2916	10	11.45	16
5	Erlang (3P)	0.06512	5	1.2168	9	11.45	15
6	Gen. Gamma (4P)	0.06994	6	4.435	23	N/A	
7	Johnson SB	0.07176	7	0.86534	4	14.506	21
8	Pareto	0.0744	8	2.606	16	1.513	1
9	Fatigue Life (3P)	0.0747	9	0.66686	2	2.8995	4
10	Dagum (4P)	0.07769	10	5.5131	30	N/A	
11	Gen. Extreme Value	0.07807	11	1.6843	13	12.083	17
12	Lognormal (3P)	0.07855	12	0.84113	3	2.4101	2
13	Kumaraswamy	0.08395	13	5.3719	29	N/A	
14	Inv. Gaussian (3P)	0.0851	14	0.87365	5	3.7297	6
15	Frechet (3P)	0.08708	15	1.167	8	4.1611	8
16	Gamma (3P)	0.08928	16	4.8473	26	N/A	
17	Pearson 5 (3P)	0.08931	17	1.1464	7	3.9632	7
18	Log-Pearson 3	0.09554	18	1.8365	14	3.1057	5
19	Pearson 6 (4P)	0.09756	19	1.3376	11	5.3259	9

20	Frechet	0.10128	20	1.9702	15	11.345	14
21	Burr (4P)	0.10519	21	1.6211	12	5.7111	10
22	Beta	0.12461	22	7.4116	41	N/A	
23	Pert	0.12661	23	4.4072	22	14.138	19
24	Pearson 5	0.12919	24	4.0665	18	18.131	28
25	Log-Gamma	0.1323	25	4.2041	19	14.241	20
26	Log-Logistic	0.13414	26	4.3865	21	18.186	29
27	Lognormal	0.13975	27	4.6105	24	15.434	24
28	Burr	0.1406	28	51.686	54	N/A	
29	Fatigue Life	0.14254	29	4.7015	25	15.048	22
30	Gumbel Max	0.14394	30	3.4854	17	10.694	12
31	Chi-Squared (2P)	0.1484	31	5.5874	31	18.648	30

Appendix 4: SWMM INPUT PARAMETERS and SWMM Output

Table 4a: sub-catchment input parameters

SC-ID	Area (ha)	Length (m)	Width (m) =A/L	Avg. slope (%)	N-Imperv	N-Perv	Dstore- Imperv	Dstor- Perv
SC-1	7.92	889.2	89.07	0.82	0.005	0.05	1.3	2.5
SC-2	9.29	1060.0	87.64	2	0.005	0.05	1.3	2.5
SC-3	7.71	937.3	82.26	1.1	0.005	0.05	1.3	2.5
SC-4	9.91	812.1	122.03	2.4	0.005	0.05	1.3	2.5
SC-5	7.15	1058.3	67.56	1.1	0.005	0.05	1.3	2.5
SC-6	10.49	787.5	133.2	2.3	0.005	0.05	1.3	2.5
SC-7	4.17	874.8	47.67	1.3	0.005	0.05	1.3	2.5
SC-8	10.92	807.1	135.3	2.3	0.005	0.05	1.3	2.5
SC-9	34.11	1799.1	189.6	1.4	0.005	0.05	1.3	2.5
SC-10	10.07	1039.8	96.85	0.7	0.005	0.05	1.3	2.5

SC-11	3.09	641.1	48.2	0.6	0.005	0.05	1.3	2.5
SC-12	8.98	625.7	143.53	1.7	0.005	0.05	1.3	2.5
SC-13	1.63	474.5	34.35	2	0.005	0.05	1.3	2.5
SC-14	7.92	894.1	88.58	1.5	0.005	0.05	1.3	2.5

Table 4 b: Sub-catchment Output Properties

Sub Catchment	Total precipi (mm )	Total Infi (mm)	Total Runoff (mm)	Total Runoff (10 <sup>6</sup> ltr)	Peak Runoff 10-yr (m <sup>3</sup> /s )	Runof f Coeff.	Peak Runof f 25-yr (m <sup>3</sup> /s)
S1	120.5	5.47	113.16	8.96	1.34	0.94	1.63
S2	120.5	4.86	114.49	10.65	1.7	0.95	2.13
S3	120.5	4.86	114.22	8.79	1.4	0.95	1.7
S4	120.5	4.86	115	11.4	2.02	0.95	2.45
S5	120.5	5.47	113.09	8.14	1.21	0.94	1.47
S6	120.5	4.86	115.01	12.08	2.14	0.95	2.6
S7	120.5	4.86	114.45	4.81	0.79	0.95	0.96
S8	120.5	4.86	114.98	12.53	2.21	0.95	2.69
S9	120.5	5.47	112.03	38.2	5.21	0.93	6.34
S10	120.5	4.86	113.58	11.47	1.71	0.94	2.08
S11	120.5	5.47	113.49	3.52	0.54	0.94	0.66
S12	120.5	4.86	115.11	10.35	1.86	0.96	2.26
S13	120.5	5.47	114.83	1.88	0.34	0.95	0.42
S14	120.5	4.86	114.53	9.07	1.5	0.95	1.83

Table 4 c: Conduit input parameters

Conduit code	Inlet node	Outlet Node	Geometry	Conduit length (m)	Width b(m)	depth y(m)	Roughness (n)
C1	J1	J2	Rectangular	131	0.8	1.5	0.4
C2	J2	J3	Rectangular	135	0.5	1.2	0.4

C3	J3	OUTLET-1	Rectangular	19	0.5	1	0.4
C4	J4	J5	Rectangular	347	0.5	1.5	0.4
C5	J24	J2	Rectangular	206	0.5	1.2	0.4
C6	J5	J6	Rectangular	400	0.5	1.2	0.4
C7	J6	J3	Rectangular	215	0.6	0.8	0.4
C8	J7	J8	Rectangular	311	0.7	1.2	0.4
C9	J25	J5	Rectangular	203	0.5	1.2	0.4
C10	J8	J9	Rectangular	440	0.7	1.2	0.4
C11	J9	J6	Rectangular	232	0.5	0.8	0.4
C12	J10	J11	Rectangular	220	0.5	1.2	0.4
C13	J11	J12	Rectangular	514	0.6	0.7	0.4
C14	J12	J9	Rectangular	205	0.5	0.8	0.4
C15	J14	J11	Rectangular	202	0.5	1.2	0.4
C16	J13	J15	Rectangular	701	0.8	1.5	0.4
C17	J16	J17	Rectangular	546	0.5	0.7	0.4
C18	J17	J15	Rectangular	415	0.5	0.8	0.4
C19	J15	OUTLET- 2	Rectangular	69	0.4	1	0.4
C20	J23	J12	Rectangular	206	0.5	0.8	0.4
C21	J18	J22	Rectangular	946	0.8	1.2	0.4
C22	J20	J22	Rectangular	189	0.6	1.2	0.4
C23	J21	J22	Rectangular	484	0.5	1.5	0.4
C24	J22	OUTLET3	Rectangular	93	0.5	1.2	0.4
C25	J19	J20	Rectangular	307	0.6	1.2	0.4
C26	J26	J16	Rectangular	708	0.5	1.5	0.4
C27	J27	J8	Rectangular	400	0.7	0.8	0.4

Table 4 d: Junction properties

Junction	X (m)	Y (m)	Max. depth (m)	Inverted depth (m)	Elevation (m)
J1	407005	1123369	1.5	2436.5	2438
J2	406779	1123363	1.5	2433	2434.5
J3	406427	1123353	1.2	2422	2423.2
J4	407029	1123585	1.5	2444.5	2446
J5	406674	1123574	1.5	2442	2443.5
J6	406388	1123564	1.2	2424	2425.2
J7	406976	1123805	1.5	2446.5	2448
J8	406661	1123785	1.2	2442	2443.2
J9	406370	1123779	1.4	2425.6	2427
J10	406884	1124018	1.5	2448.5	2450
J11	406654	1124008	1.2	2442	2443.2
J12	406366	1124002	1.2	2427.4	2428.6
J13	406786	1124222	1.5	2448.5	2450
J14	406646	1124209	1.2	2444.8	2446
J15	406276	1124204	1.5	2429	2430.5
J16	406513	1124856	1.5	2446	2447.5
J17	406181	1124714	0.8	2434.2	2435

J18	406812	1124206	1.5	2448.5	2450
J19	407026	1123697	1.5	2446.5	2448
J20	407027	1123367	1.2	2440.6	2441.8
J21	407078	1123956	1.5	2445.5	2447
J22	407236	1123364	1.5	2439	2440.5
J23	406276	1124184	0.8	2428.2	2429
J24	406671	1123565	1.2	2438.8	2440
J25	406659	1123779	1.2	2442	2443.2
J26	406808	1124219	1.5	2449.5	2451
J27	406652	1124004	0.8	2442.1	2442.9

Table 4 e: Outfall properties

Outfall Node	X (m)	Y (m)	Inverted depth(m)	Max.depth (m)	Elevation (m)	Location of Outfall
Outlet1	406391	1123351	2418	1	2419	Fasiledes bridge
Outlet2	406237	1124200	2426	1	2427	Aduadem gorge
Outlet3	407306	1123361	2436.8	1.2	2438	Litet gorge

Appendix 5: Different standards and constants used for study

Table 5a: Typical Hydrologic soil groups in Ethiopia

Soil-ID	Soil Types	Hydrologic Soil Groups
Ao	Orthic Acrisols	B
Bc	Chromic cambisols	B
Bd	Dystric cambisols	B
Be	Eutric cambisols	B
Bh	Humic cambisols	C
Bk	Calcic cambisols	B
Bv	Vertic cambisols	B
Ck	Calcic chernozems	B
E	Rendzins	D
Hh	Haplic phaeozems	C
HI	Luvic phaezems	C
I	Lithosols	D
Jc	Calcaric Fluvisols	B
Je	Eutric Fluvisols	B
Lc	Chromic Luvisols	B
Lo	Orthic Luvisols	B
Lv	Vertic Luvisols	C
Nd	Dystric Nitosols	B
Ne	Eutric Nitosols	B
Od	Dystric Histosols	D
Oe	Eutric Histosols	D
Qc	Cambric Arenosols	A
Rc	Calcaric Regosols	A
Re	Eutric Regosols	A
Th	Humic Andosols	B
Tm	Mollic Andosols	B
Tv	Vitric Andosols	B
Vc	Chromic Vertisols	D
Vp	Pellic Vertisols	D

Xh	Haplic Xerosols	B
Xk	Caloic Xerosols	B
XI	Luvic Xerosols	C
Yy	Gypsic Yermosols	B
Zg	Gleyic Solonchaks	D
Zo	Orthic Solonchaks	B

(Source: ERA, 2013)

Table 5 b: Recommended runoff coefficient C for various selected land uses

<u>Description of Area</u>	<u>Runoff coefficient</u>
Business: Downtown areas	0.7-0.95
Neighborhood areas	0.5-0.7
Residential: Single family areas	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 hectare 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
Street: Asphalt	0.7-0.95
Concrete	0.8-0.95
Drives and walks	0.75-0.85
Roofs	0.75-0.95

(Source: ERA, 2002)

Table 5c: Manning's roughness coefficient for some commonly used surface materials to calculate flow in open channels

Surface material	Manning's Roughness coefficient (n)
Asbestos cement	0.011
Asphalt	0.016
Brass	0.011
Brick and cement mortar sewer	0.015
Clay tile	0.011
Concrete- steel forms	0.012
Cement finished	0.015
Concrete-wooden forms	0.013
Concrete centrifugally spun	0.011
Corrugated metal	0.018
Earth, smooth	0.022
Earth channel clean	0.025
Earth channel-gravelly	0.03
Earth channel-weedy	0.035
Earth channel-stony, cobbles	0.035
Floodplains-pasture, farmland	0.05
Floodplains-light brush	0.075
Floodplains-trees	0.15
Galvanized iron	0.016
Grass	0.01
Gravel firm	0.023

Lead	0.011
Masonry	0.025
Natural channel very poor condition	0.06

Source: ([www.Engineering Toolbox.com](http://www.EngineeringToolbox.com))

Table 5d: GTZ Standards of drainage condition

Classification	Surface Condition
Very good	Shapes of USWD lines as still in original design condition
Good	No significant depression, undulation and deformation
Light/moderate	Shape of the USWD lines deteriorate, but still sheds water
Sever	Total collapse of the USWD lines structure and barely passable

Source (GTZ, 2006)

Table 5E:  $K_z = F(C_s, T)$  for use in Log-Pearson Type III Distribution

Coefficient of skew, $C_s$	Recurrence interval $T$ in years						
	2	10	25	50	100	200	1000
3.0	-0.396	1.180	2.278	3.152	4.051	4.970	7.250
2.5	-0.360	1.250	2.262	3.048	3.845	4.652	6.600
2.2	-0.330	1.284	2.240	2.970	3.705	4.444	6.200
2.0	-0.307	1.302	2.219	2.912	3.605	4.298	5.910
1.8	-0.282	1.318	2.193	2.848	3.499	4.147	5.660
1.6	-0.254	1.329	2.163	2.780	3.388	3.990	5.390
1.4	-0.225	1.337	2.128	2.706	3.271	3.828	5.110
1.2	-0.195	1.340	2.087	2.626	3.149	3.661	4.820
1.0	-0.164	1.340	2.043	2.542	3.022	3.489	4.540
0.9	-0.148	1.339	2.018	2.498	2.957	3.401	4.395
0.8	-0.132	1.336	1.998	2.453	2.891	3.312	4.250
0.7	-0.116	1.333	1.967	2.407	2.824	3.223	4.105
0.6	-0.099	1.328	1.939	2.359	2.755	3.132	3.960
0.5	-0.083	1.323	1.910	2.311	2.686	3.041	3.815
0.4	-0.066	1.317	1.880	2.261	2.615	2.949	3.670
0.3	-0.050	1.309	1.849	2.211	2.544	2.856	3.525
0.2	-0.033	1.301	1.818	2.159	2.472	2.763	3.380
0.1	-0.017	1.292	1.785	2.107	2.400	2.670	3.235
0.0	0.000	1.282	1.751	2.054	2.326	2.576	3.090
-0.1	0.017	1.270	1.716	2.000	2.252	2.482	2.950
-0.2	0.033	1.258	1.680	1.945	2.178	2.388	2.810
-0.3	0.050	1.245	1.643	1.890	2.104	2.294	2.675
-0.4	0.066	1.231	1.606	1.834	2.029	2.201	2.540
-0.5	0.083	1.216	1.567	1.777	1.955	2.108	2.400
-0.6	0.099	1.200	1.528	1.720	1.880	2.016	2.275
-0.7	0.116	1.183	1.488	1.663	1.806	1.926	2.150
-0.8	0.132	1.166	1.448	1.606	1.733	1.837	2.035
-0.9	0.148	1.147	1.407	1.549	1.660	1.749	1.910
-1.0	0.164	1.128	1.366	1.492	1.588	1.664	1.880
-1.4	0.225	1.041	1.198	1.270	1.318	1.351	1.465
-1.8	0.282	0.945	1.035	1.069	1.087	1.097	1.130
-2.2	0.330	0.844	0.888	0.900	0.905	0.907	0.910
-3.0	0.396	0.660	0.666	0.666	0.667	0.667	0.668

[Note:  $C_s = 0$  corresponds to log-normal distribution]

Source (K.subramanya, 2008)