STORMWATER ANALYZING URBAN FLOOD IN CASE OF DUKEM TOWN USING THE STORMWATER MANAGEMENT MODEL (SWMM)

MASTER OF SCIENCE THESIS

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HAWASSA UNVERSITY, HAWASSA, ETHIOPIA

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A THESIS SUBMITTED TO

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ADVISORS' APPROVAL SHEET

This is to certify that the thesis entitled "Stormwater analyzing urban flood in case of Dukem Town using the Stormwater Management Model (SWMM)'' is submitted in partial fulfillment of the requirements for the Degree of Master of Science with a specialization in Water Resources Engineering and Management, the Graduate Program of Department of Water Resource and Irrigation Engineering, and it has been carried out by Miliyon Dida Feye (Id No PGWRER/ 0010/11) under our supervision. Therefore, we recommend that the student has fulfilled the requirements and hence hereby can submit the thesis to the department.

Major Advisor: Tewodros Assefa (PhD) ________________ ______________

Signature Date

Co-Advisor: Mihret Dananto (PhD) ________________ ______________

Signature Date

DECLARATION

I, Miliyon Dida, declare that this research report entitled “Stormwater analyzing the urban flood in case of Dukem Town using the Stormwater Management Model (SWMM)’’ is my original work and all sources of materials were used for this thesis have been duly acknowledged. I declare that this thesis is not submitted to any other University for an award of any academic Degree, Diploma, and Certificate.

Name: Miliyon Dida Feye

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Date: __________________________________

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LISTS OF ABBREVIATIONS AND ACRONYMS

BM	Block Maxima
CAD	Computer-Aided Design
CN	Curve Number
CS	Collection System
DEM	Digital Elevation Model
DL	Drainage Line
ERA	Ethiopia Road Authority
EPA	Environmental Protection Agency
EV	Exeme Value
FAO	Food Agriculture Organization
FWR	Foundation for Water Research
GOF	Goodness of Fit
GPD	General Pareto Distribution
GPS	Geographical Position System
GTZ	German Technical Cooperation Agency
IDF	Intensity Duration Frequency
ICM	Integrated Catchment Model
JP	Junction Point
LID	Low Impact Development
MRL	Mean Residual Life
MOUSE	Model of Urban Sewer
MPI	Model Performance Indicator
NMSA	National Meteorological Service Agency
POT	The Peak-Over –Threshold
RF	Rainfall
RS	River System
SC	Sub-catchment
SCS	Soil Conservation Service
SWMM	Stormwater Management Model
TC	Time of Concentration
Tt	Time Travel
TVC	Tecnic and Vocation College
UDS	Urban Drainage System
USWD	Urban Stormwater Drainage

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ABSTRACT

Urban floods are caused due to increase population density in urban areas and the development of urban infrastructure without consideration to drainage aspects. Due to inadequate provision of conveying the runoff to the outlet properly runoff through the town can destruct the structures. The well-connected drainage network and maintenance of the existing stormwater drainage facilities are the mitigating measures of the prevailing drainage problems. This study is concerned stormwater drainage system analysis of Dukem Town focusing on the Southern part around Tedecha area. Recorded rainfall data from 1987 to 2017 years were obtained from NMSA which is used for the hydrological analysis. The catchment area and upper catchment were delineated by using Arc GIS 10.4.1 tool. The rainfall frequency analysis was conducted by identifying the best-fit probability distribution with different return periods. This enables to develop the Intensity-Duration-Frequency (IDF) curve. The intensity duration frequency curve was developed by using the General Pareto Distribution method. Estimation of peak-rate discharge was carried out by using Rational and SCS methods. Hydraulic parameters were determined after measuring the dimension of existing canals by using a tape meter. The total simulated runoff flow from the design period was found to occur 32m^3/s from the twenty sub-catchments. The adequacy of the existing drainage systems was checked by comparing the estimated runoff with the existing drainage capacity. However, it was observed that most of the drainage lines were inadequate to safely dispose runoff. The problem of existing drainage systems was lack of a well-connected drainage lines net-work, inappropriate maintenance of drainage facilities, and unavailability of drainage line at the proper place. The major portion of the drainage system performs poorly; e.g. at SC 19, 48.8%, Oda Nabe Secondary School 12.8%, Kera 28.8%, SC11, 30.3%, and SC13, 46% drainage system were severely degraded. This result shows the majority of the drainage system is not adequate to carry the design flood. To overcome the prevailing issues, the major portion of the drainage system is required to be re-constructed and maintain properly to safely pass the generated runoff, and also giving awareness to the community about such an issue is essential.

Keywords: *Stormwater Drainage System, Goodness of Fit, and Runoff*

1. INTRODUCTION

1.1. Background

Urban stormwater drainage facilities are part of the urban infrastructure elements and the design of these facilities require attention. Developing urban areas need appropriate drainage infrastructure to work efficiently in extreme events of rainfall and for rehabilitation works to keep environmental safety. Infrastructure is one of the crucial elements in the process of urbanization, and continuity of urban growth. However, in developing countries like Ethiopia, numerous problems have made the supply of physical infrastructure and services continually lag behind the urban population growth rate (Habtamu, 2020).

Cities are growing fast and large amounts of impervious surfaces replace the natural landscape as a result of urban development. Impervious surfaces can affect local streams and flooding characteristics. With urbanization, the impermeability of land increases with the increase in impervious surfaces (i.e. residential houses, commercial buildings, paved roads, parking lots, etc.) with this drainage pattern changed, the overland flow gets faster and causes environmental problems such as land degradation increases (Barreto, 2012).

The pattern of urbanization and modernization in Ethiopia has meant to increase densification along with urban infrastructure expansion. This has led to deforestation, the use of corrugated roofs, and paved surfaces. The collective effect of this result in higher rain drop intensity and consequently speeded and concentrated runoff (Belete, 2011).

Recent increases in the intensities of precipitation events due to global climate changes in various geographic locations further aggravate the impact of urbanization on the natural water system (Hanak and Lund, 2012). In Ethiopia context, where watersheds of many urban centers receive significant amounts of annual rainfall and where rainfall intensity is generally high, control of runoff at the source, flood protection, and safe disposal of excess water/ runoff through proper drainage facilities becomes essential (Asfaw, 2016).

In the design of highway/access roads, stormwater drainage structures are an extremely important component. The provision of adequate drainage is an important factor in the location and geometric design of highways. An adequate level of service can be acquired by properly designing them. Initial cost, design life, and the risk of loss of use of the roadway for a time due to runoff exceeding the capacity of the drainage structure need to be considered in the design (Adisu and Hailemikael, 2017).

In general stormwater drain structure construct without adequate provision conveying runoff is a major cause of flooding. Many of our city drain structures are made of mostly concrete and impermeable material. When we have an urban drainage basin that is made of concrete, there is no groundwater recharge. So when a drain is fill up it is going to flooding over surface areas. Flooding occurs when the depth at the node exceeds the maximum available depth (Mulugeta, 2016).

1.2. Statement of the Problem

Most cities are facing drainage problems due to a change in land-use/land management because of rapid urbanization. As a result changes in the runoff characteristics within the town increase runoff and greater susceptibility to flooding (Belete, 2011).

Currently, Dukem Town is facing over toping and flooding problems area during intensive rainfall. This is due to inadequate provision of conveying the runoff to the outlet properly. Runoff through the town can change the features of the surface land, destroying the structures (road, culvert, and ditches), create land degradation, and affect the asphalt pavement by eroding. The major drainage system problems that are observed in the town include lack of well-connected drainage networks, unavailability of drainage systems at the proper place upstream tedecha market, and around Amanuel church the existing drainage system was overflow; also Lack of waste management techniques and awareness of the community to use drainage system issues. Additionally the absence of appropriate maintenance of the existing drainage facilities either partially or fully filled with silts and causing the stormwater to flow over the road surface and insufficient capacity of ditches. Those problems created the road to mal-functional during the rainy season every year. Flooding over asphalts, walkways, and near the residences has been a big problem in town.

In Dukem Town nonetheless, research was done concerning the stormwater drainage analysis. The upper catchment contributes significant runoff and consequently causes flooding in the study area. Therefore, an attempt is initiated to investigate and minimize the destruction of the stormwater drainage structure due to runoff at the southern part Tedecha area.

1.3. Objectives of the Study

1.3.1. General Objective

The general objective of this study was to evaluate the performance of the stormwater drainage system of Dukem Town by using Stormwater Management Model (SWMM).

1.3.2. Specific Objectives

The specific objectives of this research were;

- To assess the current condition and evaluate the hydraulic capacity of the existing drainage system of the study area, and
- To estimate peak-rate discharge and assess the performance of the drainage system in the study area by taking into consideration extreme precipitation of various return periods.

1.4. Research Questions

- What are the current conditions and hydraulic capacity of the existing drainage system of the study area?
- What are the peak-rate discharge and the condition of the performance drainage system?

1.5. Significance of the Study

The benefit that would be drawn from this study may contribute to current efforts by governments and other concerning bodies to solve the problem of drainage schemes that contribute to better service coverage. To understand problems of damage and preserve the structures by avoiding further deterioration for taking corrective measures as well as to reduce any inconvenience and disruption to travel due to the overflow of water in the main road. Therefore, this study will be contributing the following significances to Ethiopia Road Authority (ERA), Dukem Town Administration, Academicians, Researchers, and other stakeholders who will conduct similar researches on other road drainage structures. Some of them include:-

- Minimize the possible damage of pavement through proper drainage structure provisions.
- Identify the current situation to know the real problems of the drainage systems.
- To reduce the environmental and health safety problems.

1.6. Scope of the Study

The study specifically focused on the assessment of the stormwater drainage system, current conditions of the storm drainage structures, and their net-work condition in drainage schemes that contribute to better service. This study is geographically limited to Dukem Town focused on the Southern part around Tedecha area.

- **Limitation of the study**

Measuring the water level in the junction's points and outfalls were not common in our country like developed countries; especially the USA and Canada have the installed automatically recording sensor. For instance, there is no recorded data of runoff at different junction points and outfalls to calibrate the simulated result of the model at the junctions as well as outfalls. So the model results were simulated without calibrated.

2. LITERATURE REVIEW

2.1. Urban Storm Drainage System in History

Historically, urban drainage systems have been regarded with numerous perspectives. During different periods and in different locations, it has been considered a vital natural resource, a convenient washing mechanism, an efficient waste transport medium, flooding concern, and a transmitter of disease. In general climate, topography, geology, scientific knowledge, engineering and construction capabilities, societal values, religious beliefs, and other factors have influenced the local perspective of urban drainage (Burian and Edwards, 2002).

Although the roots of the idea of draining populated areas can be traced back to the ancient Mesopotamian Empire. Modern urban drainage concepts have begun in the 18th and 19th centuries as the large cities in Europe were threatened by diseases due to insufficient waste and wastewater disposal. They were able to find the system that met their objectives after trial and error modifications. At the time, planning and design were limited. Few statistical standards existed for urban drainage and engineering calculations were not used during design. Despite the lack of optimization and the use of trial and error construction methods, numerous ancient urban drainage systems can be rated very successfully (Saara and Saarenketo, 2006).

2.2. Current Urban Storm Drainage Systems Perspectives

Urban drainage in the early parts of the twentieth century was firmly established as a vital public work system. Engineers continued to improve design concepts and methods. During the second half of the 20th century, regulatory elements were promulgated in the United States, Europe, and other locations addressing urban drainage issues. Extensive monitoring efforts vastly improved the understanding of urban drainage quantity and quality characteristics. Computer modeling tools advanced the methods used to design and analyze urban drainage systems. Regulations, monitoring, computer modeling, and environmental concerns have altered the perspective of urban drainage from public health and nuisance flooding concerns during the first half of the 20th century; public health and nuisance flooding with additional concerns for ecosystem protection and urban sustainability. Methods to design and construct sustainable stormwater drainage systems were currently being researched and tested. Alternative development concepts(e.g., low-impact development) are influencing development practices to minimize the impacts of development on stormwater drainage (Saara and Saarenketo, 2006).

Besides, alternative on-site wastewater management strategies are being touted as more sustainable than centralized wastewater management for some situations. Societies are searching for innovative techniques to detect, delay, and use rainwater within the watershed instead of constructing massive drainage structures. Many societies are developing watershed-wide stormwater quality management plans to meet the dual objectives of flood prevention and water quality control. Urban drainage has indeed expanded significantly during the past few decades beyond a technical challenge to drain the urban area expeditiously to include the consideration of social, economic, political, environmental, and regulatory factors (Burian and Edwards, 2002).

2.3. Stormwater Drainage Practice in Africa

From the earliest time, people in cities and towns devised drainage systems to remove surface water from their streets and house. These systems normally took the form of an open drain in the street into which flow run-off from the surrounding surfaces. There was a strict demarcation between surface water and wastewater and it was a punishable offense to contaminate surface water sewers with waste or manure; though in the day's animal-drawn transport, dung deposit on the roads would have contaminated street run-off (FWR, 2012). In Dakar and Banjul town, poorly drained area urban runoff mixes with sewage from overflowing latrines and sewers causing pollution and wide range of problems associated with the increased risk of waterborne disease (Armitage and Rooseboom, 2000). Recently Khartoum is facing extensive water logging during the rainy season, as a result of a serious problem of poor drainage. Poor existing drains and their improper operation and management mainly cause severe flooding which creates damages and problems to the road pavement (Rokade et al, 2012).

Different types of structures are employed in the drainage system open channels whether artificial or natural convey the flows of water surface and sub-surface drainage system culverts and bridges convey flows under road cross-section (ERA, 2002).

2.4. Stormwater Drainage Practice in Ethiopia

The contributed run-off water needs to be safely disposed to the rivers/outlet channels so that the functional utility of the stormwater drainage infrastructure is maintained and damages to the infrastructures may avoid.

According to Belete (2011), in Ginjo Guduru kebele of Jimma Town, Asfaw (2016), in Kemise Town, Bekele (2018), in Burayu Town, Birhanu (2018), in Debre Berehan Town, the major causes of flooding are the blockage of urban stormwater drainage lines along with inadequate integration between the urban stormwater drainages. Besides urbanization, deforestation, impermeability increases with the increase in impervious surfaces (i.e. residential houses, commercial buildings, paved roads, parking lots, etc.), drainage pattern changes, the overland flow gets faster flooding, and environmental problems such as land degradation increases. It is a crucial problem facing the existing and future stormwater drainage infrastructure.

2.5. Storm Drainage System Problems

Due to the development of infrastructure as a result of urbanization, the surface runoff water greatly increased in the town damaging stormwater drainage structures. Lack of urban stormwater drainage (USWD) management represents one of the most common sources of compliance from the residents in many urban centers of Ethiopia and this problem gets worsen with the rate of urbanization. The problems in urban stormwater drainage net-works are also challenging in urban areas because the runoff produced within a particular urban area could not safely be discharged into the final receiving system. In addition to increased densification and impermeability of the urban landscape, planning, as well as the implementation of stormwater protecting structure is insufficient (Belete, 2011).

The storm drainage system problem is that any soil has its water infiltration limit. After that point, the soil cannot accept any more water and the water begins to flow on top of the soil. This runoff water can create flooding, transport pollutant particles in natural streams, and create erosion either to the land or to the nearby natural stream. The increase in water content also reduces the bearing capacity of the soil; which will increase the rate of deterioration and shorten the lifetime of the road. In such cases, the road will need rehabilitation more often than a well-drained road structure. Mainly the problem was observed on poorly working structures, such as storm drains, culverts, and ditches (Saara and Saarenketo, 2006).

According to Wahren *et al* (2009), urbanization also adversely impacts receiving water through change it causes to local hydrology. Increases in population density and imperviousness stemming from urbanization result in changes to stream hydrology. This includes when peak discharge increased compare to pre-development levels, it increases the volume of stormwater runoff. As well as it increases the frequency and severity of floods, decreases infiltration, and diminishes groundwater recharge.

The problems posed by inadequate drainage channels have attracted a varying dimension of environmental problems. However, the most prominent among such problems the absence of drainage channels and easily translates into environmental deterioration (Jimoh, 2003). Generally, drainage problems in urban areas include flooding, deterioration of roads, land degradation, sedimentation, blockage of drainage facilities, waterlogging, etc. The most common reason for the occurrence of damage is blockage of the ditch inlet with materials such as fine soil, gravel, stone, and tree branches. Since that is normally the result of high water flows, generally there is no enough time to remove the barrier before the water flows over the road and the channels are destroyed (Wahren *et al*.2009).

Lateral Drainage Structures:

The Lateral channels were channels that collect and dispose of runoff from the road surface and side ditches to the main drainage. Due to the absence of lateral drainage structures on the right side of Mosques, around the cattle market, sub-urban area, on the left and right side of Tedecha primary school, the stormwater was retained on the surface which may cause environmental pollution and decreasing the aesthetical values of the town.

2.6. Urban Stormwater Drainage Management and System

2.6.1. Stormwater Management

Stormwater is a term used to describe water that originates throughout precipitation events. Stormwater that does not penetrate the ground becomes surface runoff, which either flows into surface waterways or is channeled into storm drains. Stormwater drainage is the process of draining excess water from streets, sidewalks, roofs, buildings, and other areas. The system used to drain stormwater is often referred to as storm drains and is also called storm sewers or drainage wells (Harshil and Gajjar, 2014). The impact of drain systems on the receiving waters shall meet the requirements of any local regulations and the relevant authority.

2.6.2. Stormwater Drainage System

2.6.2.1. Functions of Stormwater Drainage System

One of the drainage system functions is to collect surface water and/or groundwater direct it away, thereby keeping the ballast bed drained. The drainage system must also protect the sub-structure from erosion, from becoming sodden, and losing its load-bearing capacity and stability. The main objective of storm drains to protect public health and safety, environmental protection, and sustainable development. A drain system is provided to prevent the spread of disease by contact with fecal and other waterborne waste; to protect drinking water sources from contamination waste, carry runoff surface water away while minimizing hazards to the public (Hammer *et al*, 2009).

2.6.2.2. Types of Stormwater Drainage System

A drainage system will include all the components needed to ensure that the sub-structure is properly drained, and may be formed of components such as open ditches, closed ditches with pipe drains, the drainage through stormwater drainage pipes, channel, and culvert. Provision shall be made to remove runoff from streets into drainage channels, watercourses, and pipe systems at low points and at intervals that will assure the ponding of stormwater on streets does not occur for long durations (AASHTO, 1991)

2.7. Hydraulic Analysis of Stormwater Drainage System

A. Flow Type Assumptions

The design procedures presented here assume that flow within each storm drain segment is steady and uniform. This means that the discharge and flow depth in each segment is assumed to be constant concerning time and distance. Also, since storm drain conduits are typically prismatic, the average velocity throughout a segment is considered to be constant. In actual storm drainage systems, the flow at each inlet is variable, and flow conditions are not truly steady or uniform (Debo and Reese, 2003).

B. Hydraulic Capacity

Hydraulic capacity refers to the wastewater facility enable to maintain a given flow rate and the amount of liquid that can be discharged. The hydraulic capacity of a storm drain is controlled by its size, shape, slope, and friction resistance. Several flow friction formulas have been advanced which define the relationship between flow capacity and these parameters. The most widely used formula for gravity and pressure flow in storm drains is Manning's equation (ERA, 2002).

2.8. Estimating Peak-Rate Discharge

According to ERA Drainage Design Manual, 2002, 2013, the following methods are used to estimate peak flow.

A. Rational Method

The Rational method provides an estimation of peak runoff rates of small urban and rural watersheds of less than 50 hectares (0.5 sq km) and in which natural and man-made storage is small. It is best suited to the design of urban storm drain systems, small side ditches, and median ditches, and driveway pipes (ERA, 2013). The Rational method is one of the most commonly used simplified models for road storm drainage and primarily based on the concept that the peak discharge from a watershed will always occur when the rain lasts long enough at its maximum intensity to enable all portions of the basin to contribute to the flow (ERA, 2013).

B. Soil Conservation Services (SCS) Method

This method was developed by the U.S. Soil Conservation Service for calculating rates of runoff and requires the same basic data as the rational method; catchment area, time of concentration, and rainfall. The SCS approach is more sophisticated in that it considers also the time distribution of rainfall, the initial rainfall losses to interception, depression storage, and an infiltration rate that decreases during a storm. With the SCS method, the direct runoff can be calculated for any storm, either real or fabricated, by deducting infiltration and other losses from the rainfall to obtain the precipitation excess. It relates rainfall intensities to catchment parameters and uses a standard unit hydrograph to calculate the time of distribution of the runoff. It is potentially more accurate than the rational method and is applicable when the catchment area is greater than 50 hectares (ERA, 2013).

C. Statistical Analysis of Stream Gauge Data

When stream gauge data are available, stream gauge data can be used to estimate peak discharges. The Ministry of Water and Energy keeps annual stream gauge data and this method commonly used for estimating the peak discharge is usually log- Pearson type III (ERA, 2013).

D. Regional Regression Equation

The regional regression equation provides an estimate of peak discharge for watersheds in a specific geographic region. This method is based on the available data, it should be noted that at present time. Only rational and SCS methods are applied to the whole country (ERA, 2013).

2.8.1. Delineation of the Study Area

For urban stormwater drainage systems, the canal was designed and constructed for the removal of runoff from the town. Hence, the total molded area was divided into twenty sub-catchment unit elements to know the size of the canal which could be carrying the maximum discharge that was contributed into or from the sub-catchment(s).

Upper catchment delineation that influence the study area:

Upper catchment 164 ha was delineated by using Arc GIS which contributes runoff into this study area. It was delineated by using the process of fill (to remove local depression), flow direction, and flow accumulation tool (to calculate the flow into each cell that flows into the downslope cell).

Drainage of net-work and flow direction of the study area:

The well-connected drainage lines were very important in the urban drainage system (UDS) to safely pass flow to the outlet. All of the generated runoff from the catchment would not flow towards the outlet. The urban runoff flows not only on the surface, but it flows through man-made drainage a structure such as roadside ditches. The proposed and existing drainage facility resulted based on the possible outflow direction. The catchment was delineated based on direction flow through the drainage canal and verify the topographic elevation from Google earth pro. As a result, the whole catchment of this study area has three outfalls; outfall at Tedecha boundary, outfall Gabriel Church, and outfall at Parrot Hotel.

2.8.2. Intensity - Duration -Frequency (IDF) Curve

The rainfall Intensity–Duration–Frequency (IDF) relationship is one of the most important tools in water resources engineering for assessing the vulnerability of water resource structures as well as planning, design, and operation. The IDF relationship can be developed through rainfall frequency analysis, which is used to estimate rainfall depth at a point for a specified exceedance probability and duration. Rainfall intensities of various frequencies and durations are important parameters for the hydrologic design of storm sewers, culverts, and other hydraulic structures. This can be achieved by the rainfall Intensity–Duration–Frequency (IDF) relationship, which is determined through rainfall frequency analysis (Vivekanandan, 2013).

The design of water resource projects with inadequate data remains a continual engineering challenge, despite modern and widespread observational techniques. In particular, Intensity-Duration-Frequency (IDF) curves for precipitation are urgently needed for water resources projects in tropical Africa. IDF curves for precipitation determine the relationship between the rainfall intensity, the duration, or more precisely the aggregation time, and the frequency of an event. This information is required for water resources projects, sewer system design, or water quality management projects in large urban areas (Vandevyver and Demaree, 2010).

2.8.3. Frequency Analysis of Rainfall

The primary objective of frequency analysis is to relate the magnitude of extreme events to their frequency of occurrence through the use of probability distributions. Data observed over an extended period in a river system are analyzed in frequency analysis. For the design of urban drainage systems, reservoirs, hydraulic works in river valleys, irrigation schemes, etc; knowledge of the frequency occurrence rainfall data is often essential. The types of data required depend on the purpose of urban drainage system rainfall intensity (Chow.V, 1998). Some of the commonly used frequency distribution functions for the prediction of extreme maximum values are normal distribution method, gumbel distribution method, log-Pearson type III distribution method distribution periods (ERA, 2013), and peak-over threshold (Goda, 2010). Concerning the extreme value determination of various frequency analyses, two methods were widely used: Block Maxima (BM) and Peak-Over-Threshold (POT). BM is the most commonly used method due to its simplicity. However, it has its demerits of using only one data point from each block, which may cause loss of some important information and the accuracy of the parameter estimates. While the POT method is more time consuming than BM, but was found to yield better results in many studies. Further, the POT approach advantageously can produce larger sample sizes (data points), particularly for short rainfall records. The POT method enables one to use observed data more efficiently by considering more than one sample per year **(**Caires, 2009)**.** The best fit probability distribution was identified based on the result of GOF obtained from Easy Fit 5.5 professional software.

A. Peak-Over-Threshold

The POT model was introduced by Goda (1988). The model can be developed with a long length data set to obtain the results in a smaller range of confidence intervals (Goda, 2010). According to Solari and Losada (2012), the peak-over-threshold method enables one to use observed data more efficiently by considering more than one sample per year. However, the number of extreme data points to be used for analysis depends on the selected threshold value. The threshold value can be selected from the linear domain where there exists linearity between mean excess and threshold, on average 1.65–3.0 extreme events per year are considered for further analysis.

Threshold value:

Selecting an appropriate threshold value is crucial as the parameter controls the sample size and affects the assumption of independence of arrival times and the adequacy of General Pareto distribution. First, the mean residual life (MRL) plot is applied as exploratory techniques, which are carried out before the model estimation.

2.8.4. Rainfall Data

Rainfall is an important part of the hydrological cycle. In the hydrologic cycle, moisture comes from the atmosphere to the surface as precipitation. The rainfall pattern and intensity greatly influence the runoff (Fetter, 2001). If the rainfall intensity is lower than the equilibrium capacity, then all the water reaching the land surface will infiltrate. If the rainfall intensity is greater than the equilibrium infiltration capacity, but less than the initial infiltration capacity, at the beginning all the water will infiltrate, but when the infiltration capacity drops below the rainfall intensity, some of the water will remain on the ground surface. Finally, if the rainfall intensity is greater than the initial infiltration capacity, some water wills immediate remains on the land surface.

Therefore nature of the rainfall pattern is great importance in dealing with the runoff process. Rainfall is extremely variable both in time and space. Rainfall data sets in one rain gauge station, but due to the incompleteness of the rainfall data, additional rain gauge station data are needed to fill unrecorded rainfall data. The variation is brought about by differences in the type and scale of development precipitation producing process and is also strongly influenced by local regional factors, such as topography and wind direction at the rainfall (Chow, V.1988).

2.8.5. Estimating Missing Rainfall Data

Continuous and error-free precipitation data are important to many problems in hydrologic analysis and design. But there may be missing values due to many reasons such as the error of observer or instrumental failure. In such cases, one can estimate the missing data by using the nearest station rainfall data. There are different approaches for estimating missing rainfall data. In such cases, one can estimate the missing data by using the nearest station rainfall data. There are different approaches for estimating missing rainfall data.

Arithmetic method – this method is conceptually simple, but may not accurate when the total annual catch at any of the specified area gages differs from the annual catch at the point of interest by more than 10%.

Normal ratio method - when annual catches differ by more than 10%, this method is preferable.
Inverse weight distance method - In this method, the amount of rainfall for the ungauged station is estimated with respect to rainfall measured at the nearest station where location, topography, and distance between the rainfall stations are considered. Among the above method, the best one will be recommended depending on the availability of rainfall data (Garg, 2005).

2.8.6. Overview of the Stormwater Model

There are various numbers of techniques for the assessment of stormwater runoff based on water balance equations, empirical equations, and viable models.
Storm CAD- is a tool used for designing and analyzing stormwater sewer systems. The modeling software program is suitable for land developers, stormwater master planners, transportation designers as well as plan reviewers. Storm CAD uses the rational method to calculate peak flows and does not account for detention structures and flow changes over time (Bentley, 2014b).

MOUSE - MOUSE (Danish Hydraulic Institute, 1988) stands for modeling of urban sewers and is a hydrological and hydraulic model applicable only for modeling of urban catchments. This model is used extensively for sewerage design in Australia compared to the design of stormwater drainage net-works (Lindberg and Car, 1992).

Info-works - It consists of Info works ICM, (Integrated Catchment Model), Info-works CS (Collection System), Info-works SD (Stormwater Drainage), and Info-works RS (River Systems). The tools in this product group can act as stand-alone tools or together**.** Info-works SD is a comprehensive tool specialized for stormwater modeling and can be used for both single event and long-term continuous simulations. Info-works RS is a river and channel modeling solution (Innovyze, 2014a).

CIVIL Storm - is a dynamic model that accounts for storage, detention, and flows over time and is a more advanced modeling tool than Storm CAD. It is used for master planning, modeling the effect of LID structures as well as studying the water quality (Bentley, 2014a).

SWMM - It is primarily used for urban catchments but can also be applied to non-urban areas. The model includes stormwater and wastewater systems, combined systems, and natural drainage systems. Storm Water Management Model (SWMM) is chosen for this study to simulate various hydrologic/hydraulic processes in urban areas and is suitable for both continuous long-term predictions as well as single events (Rossman, 2010; 2004). The Stormwater Management Model (SWMM 5.1) was selected due to its simplicity and availability (Getachew and Tamene, 2015).

2.8.7. Performance of the Drainage System

The hydraulic performance was the capacity of regulated drainage structures to contain or safely pass flowable substances based on the design criteria specified for the related category in the manual for assessing consequence categories and hydraulic performance. The wastewater collection system performance could be determined as the system's ability to convey stormwater without hydraulic overload, besides minimizing the environmental impacts and retaining good structural integrity.

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3. MATERIAL AND METHODS

3.1. Description of the Study Area

3.1.1. Location and Topography

This study was conducted in Dukem Town, which is located in the Oromia Special Zone surrounding Addis Ababa of Oromia Region at about 37 kilometers south-east of Addis Ababa and 10 kilometers northwest of Bushoftu. Its astronomical location is at 08^{o}46'30''- 08 ^{o}48'30'' North latitude, 38^{o}54'00''- 38^{o}55'30''East longitude, and at a mean elevation of 1950 meters above sea level in Figure 3.1. The area of the town was 15.955 sq. kilometers. Most of the slope that exists in the catchment area is flat. The existing landscape of the area was formed by Great East African Rift Valley formation and subsequent volcanism, erosion, and deposition. The cracking of the soil during the dry season facilitates soil erosion. As a result, there are deep gorges in different parts of the town, especially along with the courses of seasonal streams and Dukem River Vally because of the nature of the soil (DPDO, 2007).

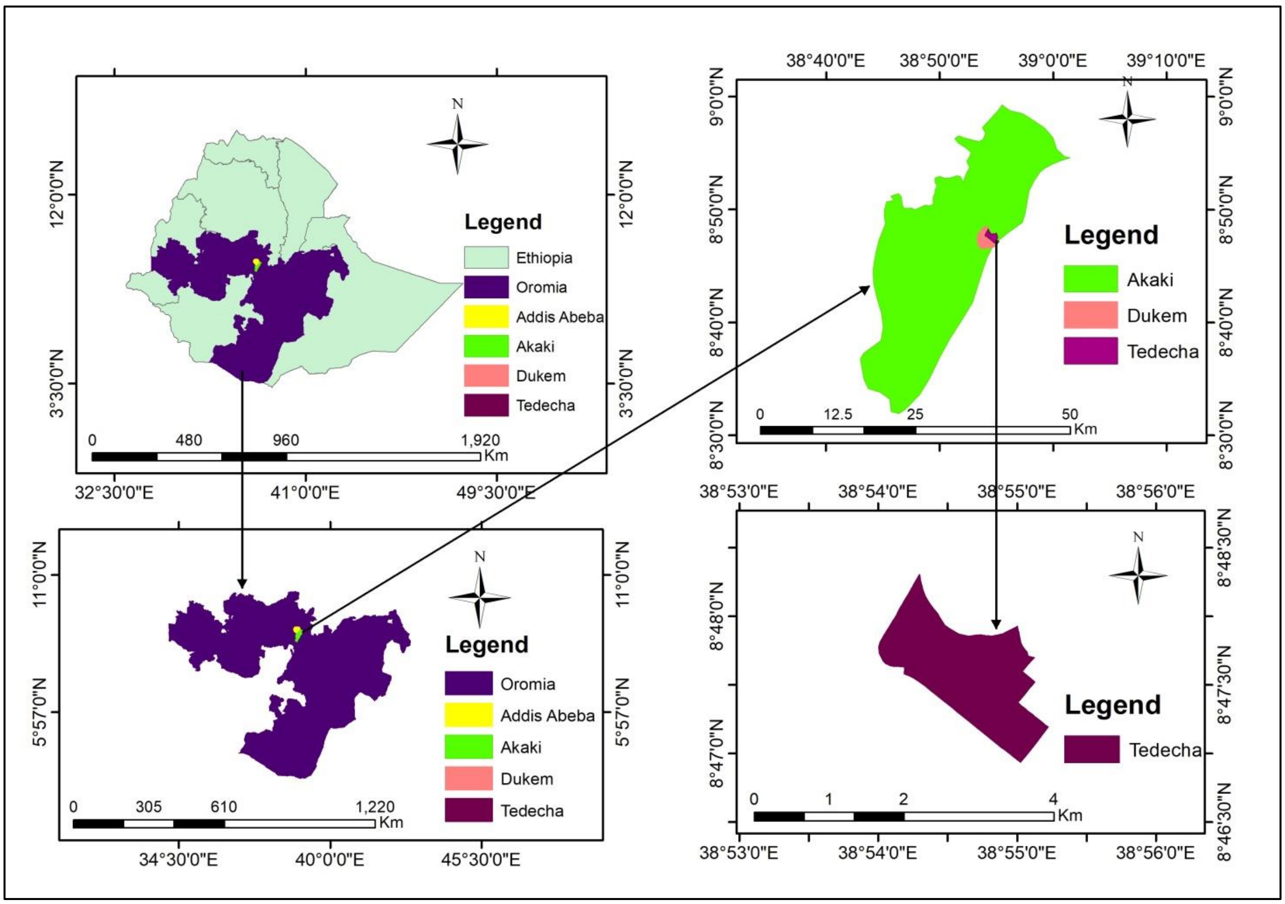

Figure 3.1: Location Map of the Study area

3.1.2. Climate

Temperature

The climatic condition of the study area is generally classified as a Weinadega zone. The monthly average temperature ranges between 28.2^{o}c and 11.6 ^{o}c as present in Table 3.1.

Table 3.1: Mean Monthly Temperature

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Avg.Tmax	26.6	27.8	28.2	28	28.2	26.6	24.4	24	25.2	26.3	26.2	25.9
Avg.Tmin	11.9	12.7	13.5	14.5	15.1	14.2	13.9	13.9	13.7	12.8	12.3	11.6

Precipitation

The annual rainfall of the study area ranges between 590-1090 mm, with an intensity of rainfall, which is highest from July-August and lowest in November-December (Figure 3.2).

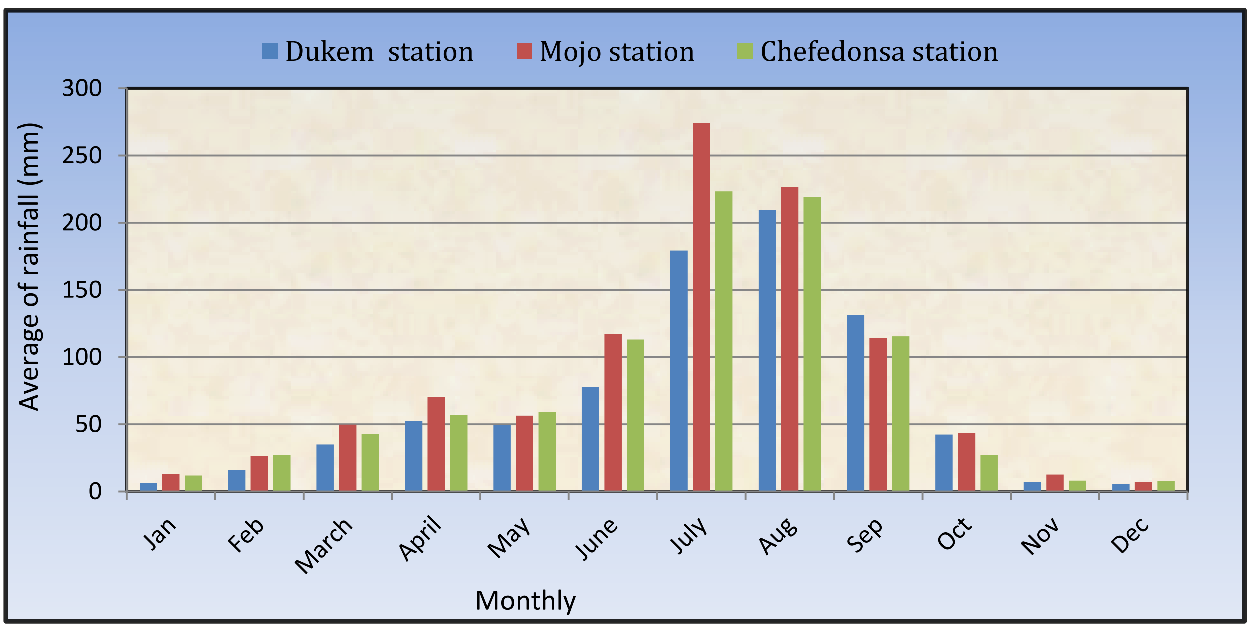

Figure 3.2: Average Monthly Rainfall over the period (1987-2017)

3.1.3. Demography

The population of Dukem Town has estimated in 2007 E.C, according to the central statistics Authority it was 66,678. The total population in Dukem Town is projected to be 67,659 in 2013. The population of the town become increasing from time to time with the town development in investment, trade, and other activities.

3.1.4. Soil of the Study Area

Soil data was an important parameter for the determination of curve number to determine peak flow. According to the FAO Soil Map of Ethiopia classification (FAO, 1998) and the Ministry of Agriculture (Appendix B, Table B2), the soil type of study area is classified as pellic vertisols with D hydrological soil group.

Pellic vertisols: Heavy clay soils with a high proportion of swelling clays. These soils form deep wide cracks from the surface downward when they dry out are moderate.

Hydrological soil group D: Soils in these groups are clay loam, silty clay loam, sandy clay, and silty clay, or clay soils having a high runoff potential due to very slow infiltration rates. Water movement through the soil is restricted or very restricted (ERA, 2013).

3.1.5. Land-use

The land-use data was an essential input for the calculation of the runoff coefficient in the determination of runoff rate using the rational and estimation of curve number in the SCS method (ERA, 2011). The classification of different land use was done by the use of Google Earth pro verified. To do this, google earth pro was opened and then searching the study area, and after searching the study area specified the molded area then digitize all the land-use until the molded area was demarcated. Then saved the digitized polygons as kml, and then convert the saved polygons from kml to layer in Arc GIS 10.4.1. The process was continued until the molded area was a full presentation of land use type as obtained in Figure 3.3.

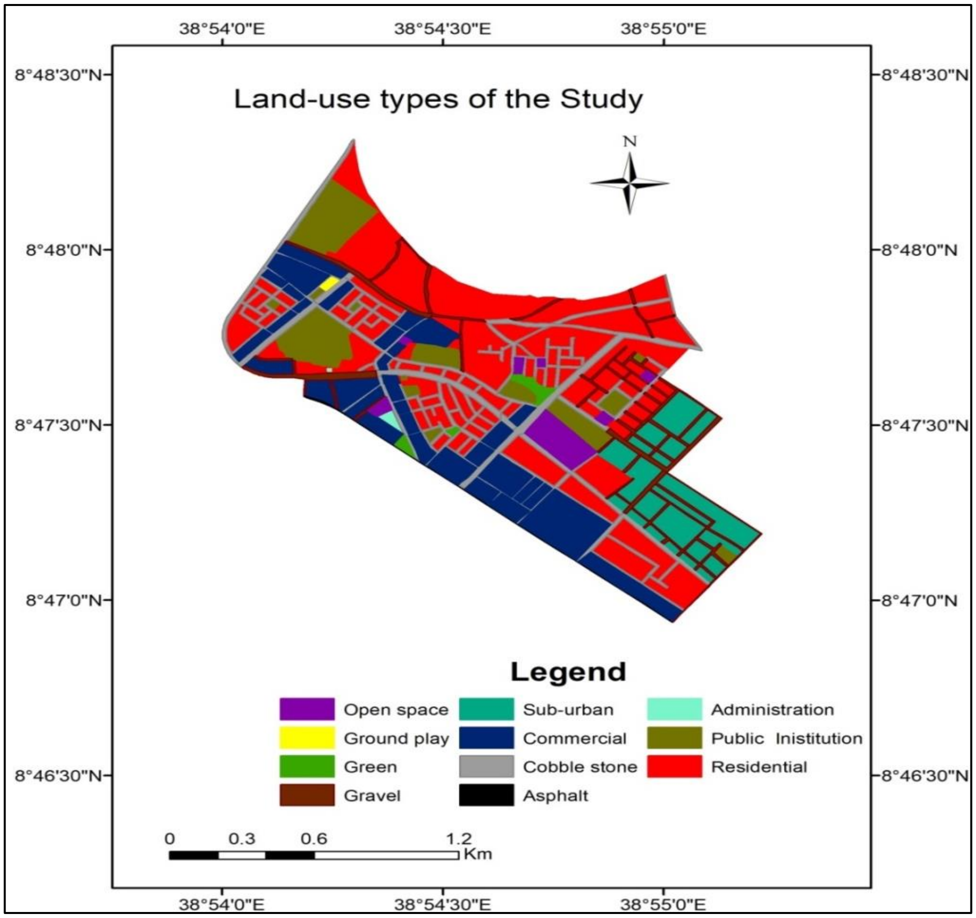

Figure 3.3: Land-use types of the study area

3.2. Data Collection and Analyses

3.2.1. Assessing the Current Condition and Hydraulic Capacity of Existing Drainage System

3.2.1.1. Assessing the Current Condition of Stormwater Drainage

For this study, field observation was carried out and pictures of different drainage status were taken to show the real scenarios of the drainage infrastructure of the study area. The field observation was made to identify the status of drainage lines that are properly functioning, partially, and fully blocked, collapsed, deteriorated, the significance of depression, undulation, and deformation of surface drainage condition.

3.2.2.2. Assessing Hydraulic Capacity of the Existing Drainage System

Hydraulic capacity:

Hydraulic capacity refers to the wastewater facility enable to maintain a given flow rate and the amount of liquid that can be discharged.

$$Q = \frac{AR^{2/3}}{n} * S^{1/2} \quad \text{……………………………………………………………………...} \quad (3.1)$$

Where: - Q = volumetric flow rate passing through the channel reach (m^3/sec).

A = cross-sectional area of flow normal to the flow direction (m^2)

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

n = dimensionless empirical constant called the Manning roughness coefficient.

R = hydraulic radius (m)

Roughness coefficient (n) varies considerably, depending on the characteristics of a channel or the smoothness of a canal and pipe. The roughness coefficient represents the resistance to flood flows in channels.

The total study area was divided into twenty sub-catchments with 234.254 ha area. The field survey was employed to measure the dimensions of drainage lines located in the study area to gather information about the current condition of the canal drainage system with the help of a tape meter, GPS, and digital cameras as per the objective of this study.

The dimensions (width, length, depth) of the existing canal in each sub-catchment were measured by using the tape meter. From these hydraulic parameters such as area, perimeter, and hydraulic radius were determined. A GPS device was used to measure the elevation and take the coordinates of different road drainage stations. The slope of each canal segment was estimated from the differences between the highest and the lowest elevation divided by the measured canal length.

Hydraulic Analysis:

The tasks of hydraulic analysis were used to determine the size of open drainage structures' capacity. Under this topic, it computes the hydraulic characteristics of the channel influencing the maximum discharge, such as the velocity of flow, the slope of the channel, a cross-sectional area of the channel, and shape and roughness coefficient of the channels.

Manning Formula:

The flow in a conduit operating as an open channel can be evaluated numerically using the uniform flow equation. Manning's equation is one of the commonly used formulas for uniform flow in an open channel and this formula was applicable for a constant flow rate through the channel with a constant slope, size, and roughness coefficient. Uniform flow is when the average velocity successive cross-section of the channels are the same and this equation is also used to determine the cross-section area, hydraulic radius, and velocity of channel flow. The hydraulic capacity of the existing drainage system was estimated by using the Manning formula as in equation 3.1.

3.2.3. Estimation of Peak-Rate Discharge and Performance of Drainage System

The peak discharge estimation is used to know the capacity of conveyance systems such as storm drains and open channels. Among different methods of estimate peak-rate discharge, the SCS and rational method are used to estimate peak-rate in this thesis.

3.2.3.1. The Soil Conservation Service (SCS) Method

There were upper catchments that are contributing runoff into this study area. The soil conservation service method is used for the whole upper catchment except upper catchment class C-3. It is potentially more accurate than the rational method and is applicable when the catchment area is greater than 50 hectares. Curve number (CN) estimates runoff excess as a function of cumulative precipitation, soil cover, land-use, and antecedent moisture. The SCS runoff present in Equation 3.2 is a method of estimating direct runoff from 1-day storm rainfall (ERA, 2011).

$$Q = \frac{(P-0.2*S)^2}{P+0.8*S} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.2)$$

for P > 0.2S

Where: - Ia = Initial Abstraction (Ia = 0.2S)

Q = Excess runoff

P = Total rainfall

S = Potential Maximum Storage

The potential maximum soil water retention (S), present in Equation 3.3 is related to hydrologic soil properties, land-use, and management conditions, as well as the soil moisture status of the catchment, rainfall event, and their relation with curve number (CN).

$S = \frac{25400}{CN} - 254$ ……………………………………..……………………….. (3.3)

Peak-rate discharge:

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

$q_p = q_u * Q * A$ ……………………………………………………………....(3.4)

Where: - q_p = is peak discharge, m^3/s

q_u = unit peak discharge, $m^3/s/km^2/mm$

A = drainage area, km^2

Q = depth of runoff, mm

The unit peak discharge was obtained from Equation (3.5), which requires the time of concentration (tc) in hours, and the initial abstraction rainfall (Ia/p) ration as input.

qu= $\propto 10^{Co+C1\ logtc+C2(logtc)^2}$ ………………………………………………..(3.5)

Where: - C_0, C_1, and C_2 = regression coefficients given in Table 3.2 for various Ia/p ratios

$\propto$ = unit conversion factor equal to 0.000431 in SI unit (ERA, 2013).

Table 3.2: SCS Peak discharge method regression coefficient

Rainfall type	Ia/P	Co	C1	C2	Rainfall type	Ia/P	Co	C1	C2
I	0.1	2.306	-0.514	-0.117	II	0.1.	2.553	-0.615	-0.164
	0.2	2.235	-0.504	-0.090		0.3	2.465	-0.623	-0.117
	0.3	2.106	0.457	-0.028		0.35	2.419	-0.616	-0.088
	0.4	1.877	-0.322	-0.058		0.4	2.364	-0.599	-0.056
	0.5	1.679	-0.069	0.000		0.5	2.203	-0.516	-0.013
IA	0.1	2.033	-0.316	-0.138	III	0.1	2.473	-0.518	-0.171
	0.2	1.919	-0.282	-0.07		0.3	2.396	-0.512	-0.133
	0.25	1.838	-0.256	-0.026		0.35	2.355	-0.497	-0.120
	0.3	1.727	-0.198	-0.026		0.4	2.307	-0.413	-0.111
	0.5	1.634	-0.091	0.000		0.5	2.178	-0.368	-0.095

Source: Drainage Design Manual (ERA,2013*)*

3.2.3.2. Rational Method

The Rational method was used for the whole study area catchments and the upper catchment class C-3 and this method provides to estimate peak runoff rates for small urban and rural watersheds less than 50 hectares. It is best suited to the design of urban storm drain systems, small side ditches, and driveway pipes (ERA, 2002).

$$Q = 0.00278 * Cf * C * I * A \quad \text{.....} \quad (3.6)$$

Where: - Q = Discharge at outlet (m^3/s)

C = Rainfall-runoff coefficient

I = Maximum probable rainfall Intensity (mm/hr)

A = Catchment area (ha)

Cf = Frequency factors for rational formula (Appendix A, Table A4)

The procedure outlines the rational method to estimate peak discharge

- ✓ Determine the watershed area
- ✓ Determine the time concentration
- ✓ Determine rainfall intensity
- ✓ Select the appropriate runoff coefficient
- ✓ Compute the peak discharges

Runoff Coefficient (C):

The runoff coefficient (C) which is expressed in Equation 3.7 is a dimensionless value representing characteristics of water that contributes to area affects how much of the rainfall becomes runoff and is a function of the various land-uses.

$$Cw = \sum\left(\frac{Ai*Ci}{AT}\right) \quad \text{.....} \quad (3.7)$$

Where: - Ci = Runoff coefficient for each drainage area (-)

Ai = Area under each of the sub-catchment area (ha)

A_T = Total catchments area

Cw = weight runoff coefficient (-)

Rainfall Intensity:

The rainfall intensity (I) is the average rainfall rate in mm/hr for the duration of time equal to the time of concentration for a selected return period. Once a particular return period has been selected for design and time of concentration calculated for the catchments area, the rainfall intensity can be determined from the Rainfall-Intensity-Duration curves (Subramanya, 1994). Rainfall Intensity is a function of geographic location, design exceedance frequency (return period), and storm duration. The magnitude was carried out by using return period maps and Intensity-Duration-Frequency (IDF) curves. The IDF relationship is a mathematical relationship between the rainfall intensity, the duration, and the return period (ERA, 2011). For this thesis, the IDF curve was developed to obtain the rainfall intensity which is used to estimate peak discharge.

Time of Concentration (Tc):

According to Thomas *et al*. (2000), the time of concentration was defined as the period required for water to travel from a distance point of watershed contributing to the drainage system under consideration. The time of concentration (Tc) is used to determine the critical rainfall duration, which can be combined with an appropriate rainfall-intensity-duration frequency (IDF) curve relations to establishing the required design rainfall intensity for corresponding return periods and time of concentration. For a specific drainage basin, the time of concentration consists of an inlet time plus the time of flow in an open channel to the design point (ERA, 2013).

Inlet time (Tci):

Is the time required for runoff to flow over the surface to the nearest inlet and is primarily a function of the length of overland flow, the slope of the drainage basin, and surface cover. The inlet time for the time of concentration is estimated using the Airport or Federal Aviation Administration (1970) method (AASHTO, 1991; 1997).

$$\mathrm{Tci} = \frac{3.64(1.1-\mathrm{C})\left(\frac{\mathrm{Li}}{1000}\right)^{0.83}}{\mathrm{H}^{0.33}} \quad \ldots\ldots (3.8)$$

Where: - Tci = Time of concentration inlet (hrs)

C = Runoff coefficient

Li = Flow length from the remotest point to the point of interest (km).

H = Elevation difference between the upstream and downstream of the canal (m).

Open or pipe channel flow time:

It can be estimated from the hydraulic properties of the conduit or channel. For each catchment area, the distance was determined from the inlet to the most remote point in the tributary area. From a topographic map, the pipe or open channel flow time must be calculated and added to the inlet time. After determining the average flow velocity in the pipe or channel, the travel time (Tt) is obtained by dividing velocity into the pipe or channel length presented in Equation3.9. Manning's equation can be used to determine velocity.

$$Tt = \frac{L}{3600V} \quad (3.9)$$

Where: - Tt = travel time in, hr

L = length of the main drainage canal (m)

V = flow velocity in the channel (m/s)

3600 = conversion factor from seconds to hours.

$$V = \frac{R^{2/3}S^{1/2}}{n} \quad (3.10)$$

Where: - R = hydraulic radius (m)

S = channel slope (m/m)

n = manning roughness coefficient

$$Tc = Tc\ inlet + Tt \quad (3.11)$$

3.2.3.3. Rainfall Data

Rainfall is an important part of the hydrological cycle (Fetter, 2001). One of the first steps in any hydrological and meteorological study is accessing reliable data. However, precipitation data is frequently incomplete. The incompleteness of precipitation data may be due to damaged measuring instruments, absence of an observer, measurement errors, errors due to uncertainty in converting point measurement data into estimates of regional precipitation, and changes to instrumentation over time (Subramanya, 1994). In countries where it has not been possible to accurately and consistently record precipitation data in a particular time section, it is necessary to use methods to estimate the missing precipitation data and apply it in hydrological models. Daily rainfall data (precipitation) of the study area were obtained from the Ethiopian National

Meteorological Service Agency for 31 (1987-2017) year's record data. But, due to the incompleteness of rainfall data of the study area, rainfall data of two neighboring rainfall stations were selected to fill up the missing data using normal ratio methods.

Presently, there is no active rainfall station in Dukem Town. Hence daily rainfall data from the nearest neighbor station, namely Bushoftu located around 10 km east of Dukem Town are collected and used in the present study. The land-use and land features of both Towns are almost the same. These two areas have more similar features which are presented in Table 3.3.

Table 3.3. Similar properties of Dukem and Bushoftu Town

Description	Dukem Town	Bushoftu Town
Soil type considered as pellic-vertisols with hydrological D soil group	✓	✓
The feature land characteristics of the Town was flat	✓	✓
Rainfall Region A2	✓	✓
Awash river basin	✓	✓
Upper sub-basin of Awash river basin	✓	✓

Table 3.4: Summary of missing data in percent or station and neighbor stations

Station	Latitude	Longitude	Altitude (m)	Missing in (%)	Average Annual Precipitation (mm)
Dukem	8.7333	38.95	1900	2.86%	809.846
Mojo	8.60533	39.1081	1763	1.79%	952.568
Chefedonsa	8.97	39.1232	2392	10.51%	820.61

3.2.3.4. Estimation of Missing Rainfall Data

There are different approaches for estimating the missing rainfall data varying with the effect of orography on rainfall, the distance between the rainfall stations, and the variation of rainfall amount recorded on the stations (Singh, 1994, Subramanya, 1994).

Among different methods, the normal ratio method was recommended for estimating missing rainfall data in the station, where the annual rainfall values between the stations differ by more than 10%. This method approach enables to estimate of missing rainfall data by weighting the observation at Nx gauges by their respective with mean annual rainfall values of N_1, N_2, N_3, and N_4 then the missing data Px can be estimated (Garg, 2005) as expressed by the Equation(3.12).

$$Px = \frac{Nx}{m}\left[\frac{P1}{N1} + \frac{P2}{N2} + \frac{P3}{N3} + - - \frac{Pn}{Nn}\right] \ldots\ldots (3.12)$$

Where:-Px – Missing rainfall data (daily, monthly or yearly)

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

Nx- Mean annual rainfall at missed station

N_1, N_2, N_3, and Nn –Mean annual rainfall at different nearest stations corresponding to

P_1, P2, P_3, and Pn.

m - Number of neighboring stations

From Table 3.4, there were missing rainfall data for all stations. But, complete and reliable data is required for water resources planning and management. The average annual precipitation in Table 3.4 of all the stations was obtained from NMSA. The ten percent of the average annual rainfall of the station under the study area was calculated and became 80.98 mm. Add and subtracts 80.98 from the average annual precipitation of the study area ranges between 728.86 mm and 890.83 mm. The average annual precipitation of Mojo station is greater than the range and Chefedonsa is in the range, hence the normal ration method is accepted.

3.2.3.5. Checking Quality of Data

A. Outlier Test

An outlier test is an observation that deviates significantly from the greater magnitude of the data, helps to avoid those data lie out of the range between the lowest outlier and the highest outlier, which may be happening due to errors in data collection, recording, and due to natural causes (Haka *et al*. 2003). The presence of outliers in the data causes difficulties when fitting a distribution to data. Low and high outliers are both possible to have different effects on the analysis. The lowest and highest outlier threshold values are expressed by Grubbs- beck test in Equation (3.16, 3.17).

$$Y = \log^{x} \quad (3.13)$$

Lowest outlier

$$L = 10^{Y} \quad (3.14)$$

Highest outlier

$$RH = 10^{YH} \quad (3.15)$$

Where

$$\text{Lowest test}(YL) = \bar{Y} - Kn * S_y \quad (3.16)$$

$$\text{Highest test}(YH) = \bar{Y} + Kn * S_y \quad (3.17)$$

Kn- factor from (Appendix A, Table A1) corresponding to the number of year data.

Kn is 2.8304, for N is 59

$\bar{Y}$ – Mean of data, $\bar{Y}$ = 1.7

S_y = The standard deviation of data =0.082

$$YL = \bar{Y} - Kn * S_y = 1.7 - 2.8304 * 0.082 = 1.468$$

$$YH = \bar{Y} + Kn * S_y = 1.65 + 2.8304 * 0.082 = 1.932$$

$RL=10^{YL} = 10^{1.468}$ = 29.37 mm

$RH=10^{YH} = 10^{1.932}$ = 85.5 mm

The highest and lowest outlier test was calculated 29.37 mm and 85.5 mm respectively. Hence, there was no data that is lower than (29.37) mm and higher than (85.5) mm. That means all the available data were satisfied with the given condition (shown in Appendix A, Table A2).

B. Consistency Test

Water resources development and management are heavily dependent on hydrological and meteorological data. To make sure that the results obtained from these data are reliable for practical applications, such data should be, homogeneous and consistent either to carry out frequency analysis or to simulate a hydrological system. The consistency of time series was analyzed based on the theory that a plot of two cumulative quantities that measured for the same period should be a straight line and their proportionality unchanged, which is represented by the slope. Sometimes a significant change may occur in and around a particular rain gauge station. Such a change occurring in a particular year will start affecting the rain gauge data, being reported from that particular station. After several years, it may be felt that the data of that station is not giving consistent rainfall values. Therefore, the inconsistency of the record data was corrected by a double mass curve (Garg, 2005). This technique can be used to adjust inconsistent precipitation data values. Therefore, if a significant change has occurred on the plot, it should be corrected by the formula which is expressed in Equation (3.18).

$$Px' = Px\frac{M'}{M} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.18)$$

Where: - Px' = Corrected precipitation at station x

Px = Original recorded precipitation at station x

M' = Corrected slope of the double mass curve

M = Original slope of the double mass curve

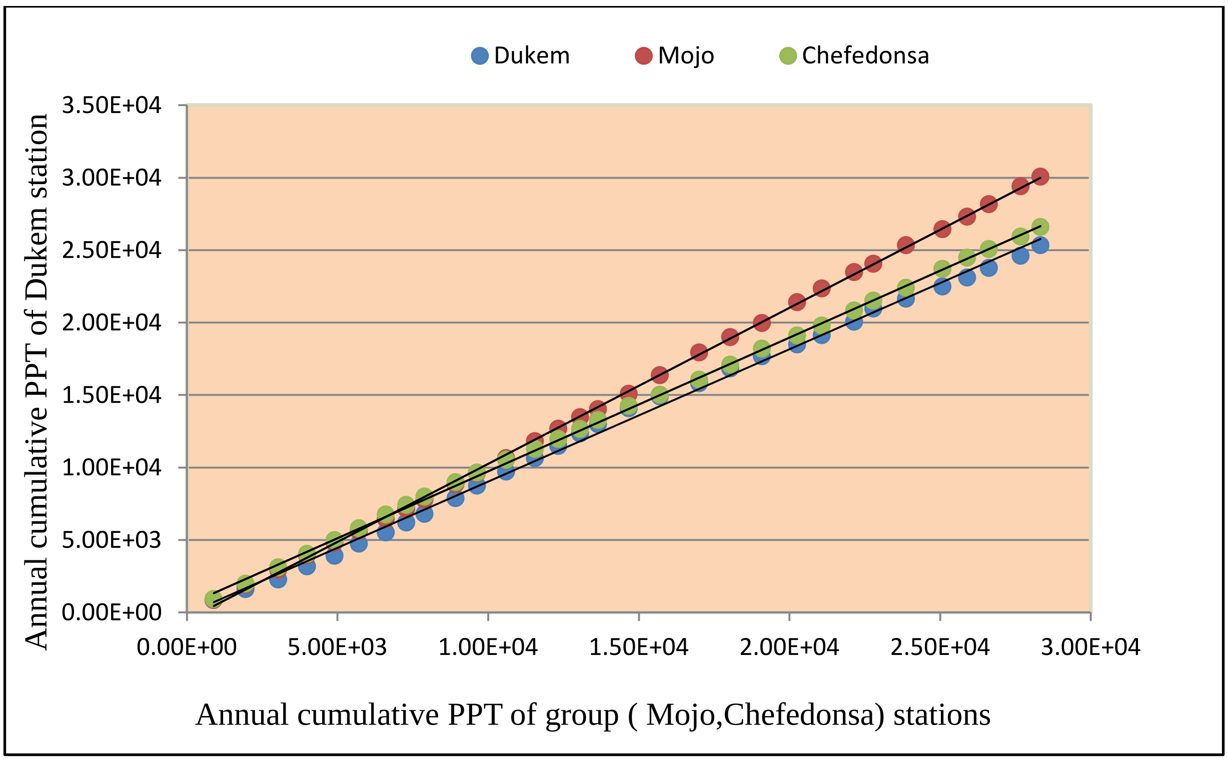

Figure 3.4: Consistency of Dukem station with other cumulative stations

The above graphs showed all points set on or form almost straight lines, which was plotted for checking of consistency of rainfall.

C. Homogeneity test

The selected stations were plotted for comparison with each other; the homogeneity test was plotted from monthly average rainfall (mm) to check the similarity of selected group station as shown in Figre 3.5.

Table 3.5: Monthly Average rainfall of study area and neighboring stations (mm)

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Avg. RF of Dukem(mm)	7.6	18.0	43.8	53.4	52.2	93.3	217.1	202.3	96.5	23.1	4.5	5.7
Avg. RF of Mojo(mm)	12.0	20.7	51.1	69.8	61.5	108.4	274.9	218.3	108.9	27.1	9.5	7.6
Avg. RF of Chefedonsa	10.6	22.5	42.8	57.6	59.7	98.7	214.8	224.3	100.8	13.6	5.5	6.9

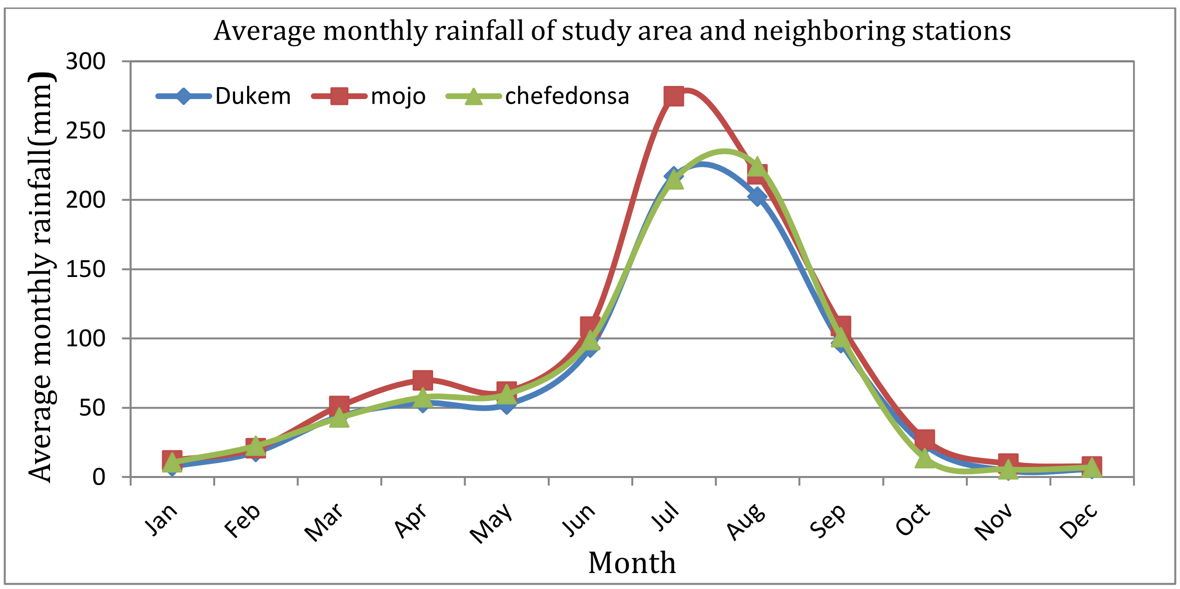

Figure 3.5: Homogeneity of each station

3.2.3.6. Rainfall Frequency Analysis Methods

Hydrologic processes such as floods are complex natural events. To analyze the maximum discharge expected in T years we can use the frequency distribution functions peak-over-threshold value.

❖ Peak-Over-Threshold value method:

The POT is also known as partial duration series and is one of the methodologies widely used in studies involving extreme value (EV) analysis. In this method, as a first step, a high threshold value a time series of extremes (involving data points of the original time series that exceed the predetermined threshold value) is developed. The probability density function for GPD with a shape parameter k ≠ 0, a scale parameter σ, and a threshold (location) parameter μ, is given as Equation (3.19).

$$Y = f((x)k, \delta, \mu) = \frac{1}{\delta}\left(1 + K\,\frac{(X-\mu)}{\delta}\right)^{-1-1/K} \quad \text{……………………………………….….. (3.19)}$$

Where: - x- flood peak flow, μ - Continuous Threshold value (location parameter)

δ- Continuous scale parameter ($\delta > 0$).

K- Continuous shape parameter. Mean excess and threshold values are in mm unit.

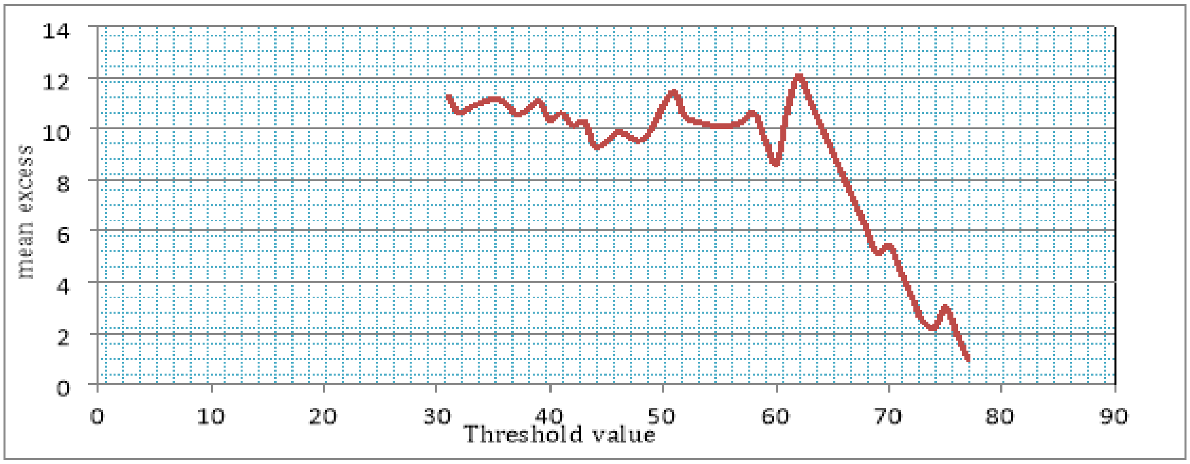

Figure 3.6: Mean residual life plot

There is a sort linear relationship between mean residual and threshold value started from 37mm as it was found to result in the smallest variability and there were 59 sample size values. However, by taking into consideration the recommended mean number of observations per year that need to be considered for analysis as (1.65-3) and after testing various threshold values in the linear range, the threshold of the reference period daily rainfall time series was decided to be 37mm. The continuous shape parameter (k), the continuous scale parameter (σ), and the continuous location parameter (μ) of the generalized Pareto probability distribution function were found to be -0.06077, 11.33, and 36.53 respectively.

3.2.3.7. The Goodness of Fit Tests (GOF) of the Probability Distributions

The best probability distributions were identified based on the result of GoF obtained from different tests**.** There are several tests of goodness of fit tests that exist in determining which distribution is the best model for this study.

A. Easy Fit 5.5 professional software:

This software is used to identify which distribution fits the theoretical probability distribution. Kolmogorov-Smirnov test, Chi-Squared test, and Anderson-Darling were used in this study for the GoF test using Easy Fit 5.5 professional software.

- **Kolmogorov-Smirnov Test:**

This test is used to decide if a sample comes from a hypothesized continuous distribution. It is based on the empirical cumulative distribution function. When comparing different distribution, lower statistics mean is a better fit.

- **Anderson-Darling Test:**

The Anderson Darling procedure is a general test to compare the fit of an observed cumulative distribution function to an expected cumulative distribution function. When comparing different distribution, lower statistics mean a better fit.

- **Chi-Squared Test:**

The chi-squared test was used to determine if a sample comes from a population with a specific distribution. This test is applied to binned data, so the value of the test statistic depends on how the data is binned. When comparing different distribution, lower statistics mean a better fit.

Table 3.6: Goodness of fit tests with Easy fit 5.5 software

Distribution	Kolmogorov Smirnov		Anderson Darling		Chi-Squared	
	Statistic	Rank	Statistic	Rank	Statistic	Rank
Gumbel Max	0.12799	4	1.6017	3	3.4471	3
Log-Pearson 3	0.08559	2	0.86277	2	2.3798	1
Lognormal	0.11889	3	2.2639	4	7.0188	5
Normal	0.15578	5	3.8623	5	6.7853	4
Gen. Pareto	0.05798	1	0.35729	1	2.5121	2

As explained in Table 3.6, the best fit distribution ranking system was ordered with rank one is the best fit, rank two is the second best, the three is the third best, and so on. Therefore, the General Pareto distribution method (GPD) was provided for the best fit at the rainfall gauging station analyzed in this study.

3.2.3.8. Intensity- Duration- Frequency (IDF) Curves

An intensity-duration-frequency curve is a mathematical function that relates the rainfall intensity with its duration and frequency of occurrence. These curves are commonly used for the hydrologic design of storm sewers, urban drainage design, culverts, and other hydraulic structures (ERA, 2013).

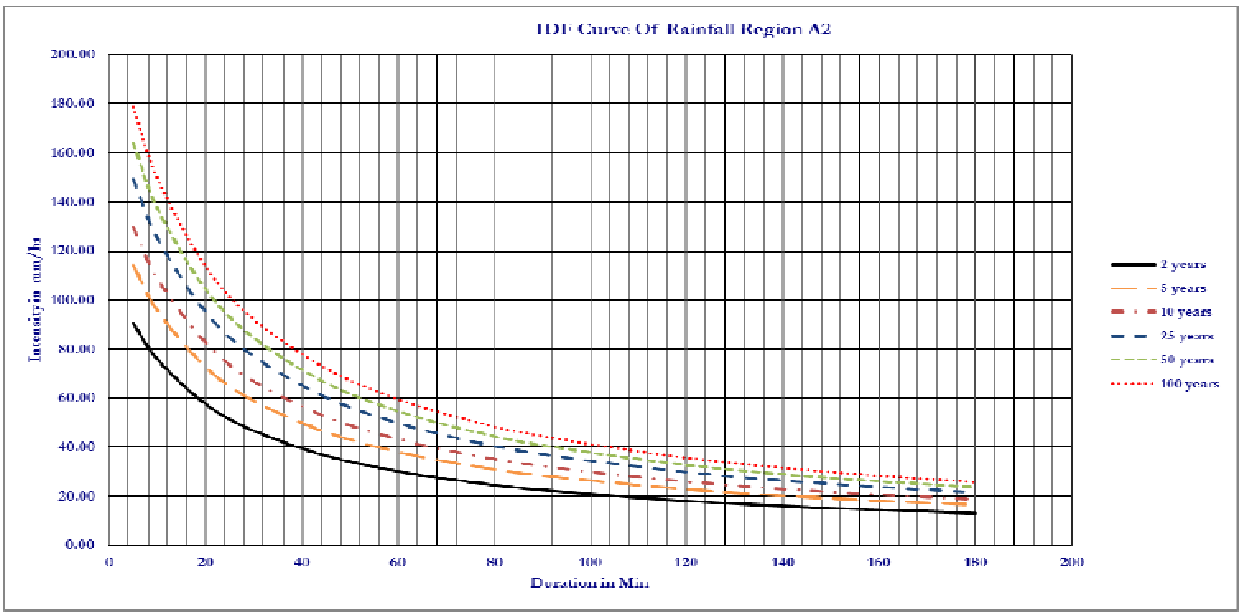

Figure 3.7: IDF curve developed by ERA

3.2.3.9. Design Rainfall of Shorter Duration

The rainfall depth obtained from the gauging station is a twenty-four-hour duration depth. Design and analysis of drainage structures require rainfall intensity-duration- frequency relationship of shorter duration. Because rainfall data of shorter duration is unavailable, to appropriate IDF derivation for the shorter duration presented in Equation 3.20 (ERA, 2013).

$$R_{Rt} = \frac{t\ (b+24)^{n}}{24\ (b+t)^{n}} \qquad (3.20)$$

Where: - R_{Rt} =Rainfall depth ratio Rt: R_{24}

Rt = Rainfall depth in a given duration t

R_{24} =24 hr rainfall depth

Coefficients b = 0.3 and n = 0.78 - 1.09.

The above formula helps to develop the IDF curve for the shorter duration events. Among from frequency analysis method the best fit probability is recommended. R_{24} is calculated for 2, 5, 10, 25, 50, and 100 year return periods.

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

Substitute Intensity (It) in Equation (3.21)

$$\mathrm{It} = \frac{\mathrm{Rt}}{\mathrm{t}}$$

$$\mathrm{It} = \frac{(\mathrm{b}+24)^{\mathrm{n}}}{24\ (\mathrm{b}+\mathrm{t})^{\mathrm{n}}} * R_{24} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.22)$$

3.3. Stormwater Management Model (SWMM)

3.3.1. Modeling Capacities

SWMM provides an integrated environment for editing study area input, running hydrologic, hydraulic, and viewing the results in a variety of formats. SWMM accounts for various hydrologic processes that produce runoff from urban areas. Those 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 the drainage system, nonlinear reservoir routing of overland flow and capture and retention of rainfall/ runoff with various types of low impact development practice. The runoff component operates on a collection of sub-catchment areas that receive precipitation, generate runoff and pollutant loads. SWMM tracks the quantity and quality of runoff made within each sub-catchment (Rossman, 2004).

3.3.2. Application of SWMM

SWMM has been used in stormwater studies throughout the world. Typical applications include design and size of drainage system components for flood control, sizing of detention facilities and their appurtenances for flood, water quality protection, designing control strategies for minimizing combined sewer overflows, evaluating the impact of inflow and infiltration on sanitary sewer overflows, generating non-point source pollutant loading for waste load allocation studies (Rossman, 2004).

3.3.3. Computational Methods of SWMM

SWMM is physically based on a discrete-time simulation model. It employs principles of conservation of mass, energy, and momentum where appropriate. The methods SWMM uses to model stormwater runoff quantity and quality through the following physical process:

Surface Runoff:

Inflow comes from precipitation and any designated upstream sub-catchments. Each sub-catchment surface is treated as a nonlinear reservoir. There are several outflows, including infiltration, evaporation, and surface runoff. The capacity of this reservoir is the maximum depression storage, which is the maximum surface storage provided by ponding, surface wetting, and an interception.

Infiltration:

Infiltration is the process of rainfall penetrating the ground surface into the unsaturated soil zone of pervious sub-catchment areas. For modeling infiltration SWMM offers:-

A. Horton's Method

This method is based on empirical observations showing that infiltration decreases exponentially from an initial maximum rate to some minimum rate throughout a long-rainfall event. The maximum and minimum infiltration rates, a decay coefficient that describes how fast the rate decreases over time, and the time it takes a fully saturated soil to completely dry are some of the input parameters required by this method (Rossman, 2004).

B. Modified Horton Method

This is a modified version of the classical Horton Method that uses the cumulative infiltration above the minimum rate as its state variable (instead of time along the Horton curve), providing a more accurate infiltration estimate when low rainfall intensities occur. It uses the same input parameters as the traditional Horton Method (Rossman, 2004).

C. **Green-Ampt Method**

This method for modeling infiltration assumes that a sharp wetting front exists in the soil column, separating soil with some initial moisture content below from saturated soil above. The initial moisture deficit of the soil, the soil's hydraulic conductivity, and the suction head at the wetting front are input parameters required in this method (Rossman, 2004).

D. Modified Green-Ampt Method

This method modifies the origin Green-Ampt procedure by not depleting moisture deficit in the top surface layer of soil during initial periods of low rainfall was done in the original method. This change can produce more realistic infiltration behavior for storms with long initial periods where the rainfall intensity is below the soil's saturated hydraulic conductivity (Rossman, 2004).

E. Curve Number Method

This approach is adopted from the Natural Resources Conservation Service (NRCS) curve number method for estimating runoff. It assumes that the total infiltration capacity of a soil can be found from the soil's tabulated curve number. During a rain event, this capacity is depleted as a function of cumulative rainfall and remaining capacity. The input parameters for this method are the curve number and the time it takes a fully saturated soil to completely dry (NRCS, 2004).

Flow Routing:

Flow routing within a conduit in SWMM is governed by the conservation of mass and momentum equations for gradually varied, unsteady flow (i.e. the Saint- Venant flow equations). The SWMM user has a choice on the level of sophistication used to solve these equations. Those are steady flow routing, kinematic wave routing, and dynamic wave routing. Each of these routing methods employs the Manning equation to relate the flow rate to flow depth and bed (friction) slope. For user-designated force main conduits, either the Hazen-Williams or Darcy-Weisbach equation can be used when pressurized flow occurs (Rossman, 2010).

A. Steady Flow 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. The normal flow equation is used to relate the flow rate to the flow area or depth. This type of routing cannot account for channel storage, backwater effects, entrance/ exit losses, flow reversal, or pressurized flow. It can only be with dendritic conveyance net-works, where each node has only a single outflow link.

B. Kinematic Wave Routing

This routing method solves the continuity and momentum equation in each conduit. The maximum flow that can be conveyed through a conduit is the full normal flow value. Allow flow and area to vary both spatially and temporally within a conduit. However, this form of routing cannot account for backwater effects, entrance/exit losses, flow reversal, or pressurized flow and is also restricted to dendritic net-work layouts (Rossman, 2004).

C. Dynamic Wave Routing

Dynamic wave routing solves the complete one-dimensional Saint-Venant flow equation and produces the most theoretically accurate results. These equations consist of the continuity and momentum equations for conduit and a volume continuity equation at nodes. With this form of routing it is possible to represent pressurized flow when a closed conduit becomes full, such that flow can exceed the full normal flow value. Flooding occurs when the water depth at a node exceeds the maximum available depth and the excess flow is either lost from the system or can pond a top of the node and re-enter the drainage system. Dynamic wave routing can account for channel storage, backwater, entrance/ exit losses, flow reversal, and pressurized flow. Because it couples together the solution for both water levels at nodes and flows in conduits. It can be applied to any general net-work layout with much smaller time steps on the order of a thirty-second or less (Rossman, 2010; 2004). Generally, flood waves for one- dimension situations can be described by the saint-venant equations.

Continuity equation: $\frac{\delta Q}{\delta x} + \frac{\delta A}{\delta t} = 0$ ………………………………………………….. (3.23)

Momentum equation: $\frac{\delta Q}{\delta t} + \frac{\delta (Q^2)}{\delta x(A)} + gA\frac{\delta H}{\delta x} - gASo + gASf = 0$ ……………………. (3.24)

Where x is the distance along the conduit, t is time, A is the cross-sectional area (m^2), and Q is the flow rate ($m^3.s^{-1}$), H *is* flow depth (m), *So* is channel bottom slope (head loss per unit length), Sf friction slope ($m.m^{-1}$), g is the acceleration of gravity (m/s^2).

The friction slope Sf can in terms of being expressed the Manning equation as

$$Sf = \frac{n^2 v^2}{R^{2/3}} \quad (3.25)$$

Where: n is the Manning roughness coefficient, V is velocity (m/s) and R is hydraulic radius (m)

Time-series:

Computed results at each reporting time step for the variables were available for viewing on the map, can be plotted, tabulated, and statically analyzed. Those variables sub-catchments, nodes, and links were selected to have detailed time series results.

Model setup:

The simulation software package selected for this study was the EPA Stormwater Management Model (SWMM5.1). The first step in the model setup was building a conceptual model divided into sub-catchments that represent the study area. Sub-catchment information such as area and slope were obtained, while typical values from the literature were taken for manning impervious and pervious areas. To obtain the width parameter, area of the sub-catchment is divided by the average length of that sub-catchment (Rossman, 2010).

$$W = \frac{A}{L} \quad (3.26)$$

Where: - W = Width of each sub-catchments (m)

A = Area of each sub-catchments (m^2)

L = Average maximum length of sub-catchments (m)

Model input parameters:

Parameters included in sub-catchments, junctions, links, and outfalls were used for the simulation of the model. Some key parameters used for sensitive analysis was described in Appendix C, Table 5C. The parameters were fixed based on the flow rate estimated with the manning equation and flow rate simulated by the model. Hence, the flow rate obtained for the same sub-catchment was used to compare the estimated flow rate with the simulated.

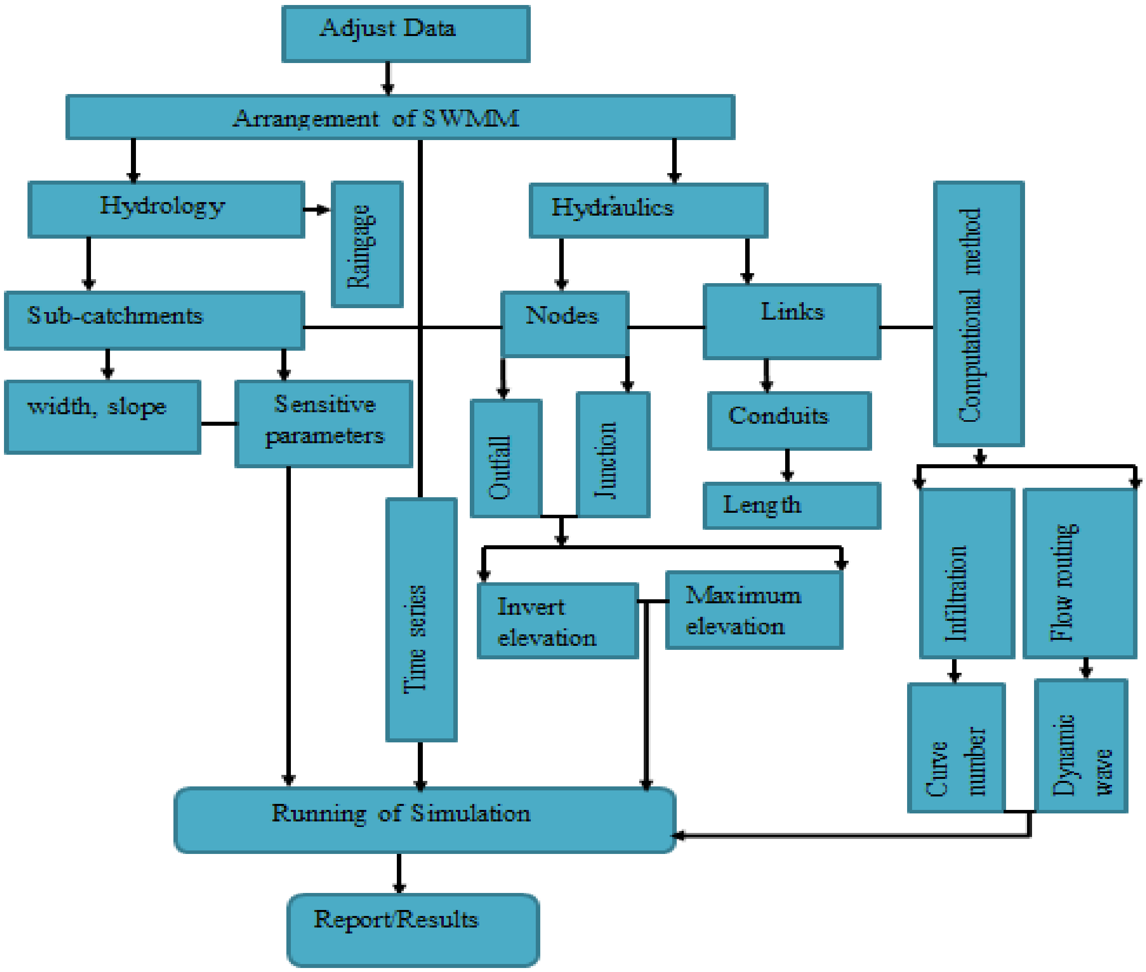

Figure 3.8: Schematic of SWMM

3.3.4. Performance of Drainage System

The hydraulic performance of the drainage system has to be analyzed, based on the assessment of the current condition of the drainage system, the hydraulic capacity of the existing drainage system, and estimated peak-rate discharge results. The performance of the current drainage system was evaluated by using the indicator classification of drainage conditions include the shape of the urban stormwater drainage line and the significance of depression, undulation, and deformation of surface drainage condition.

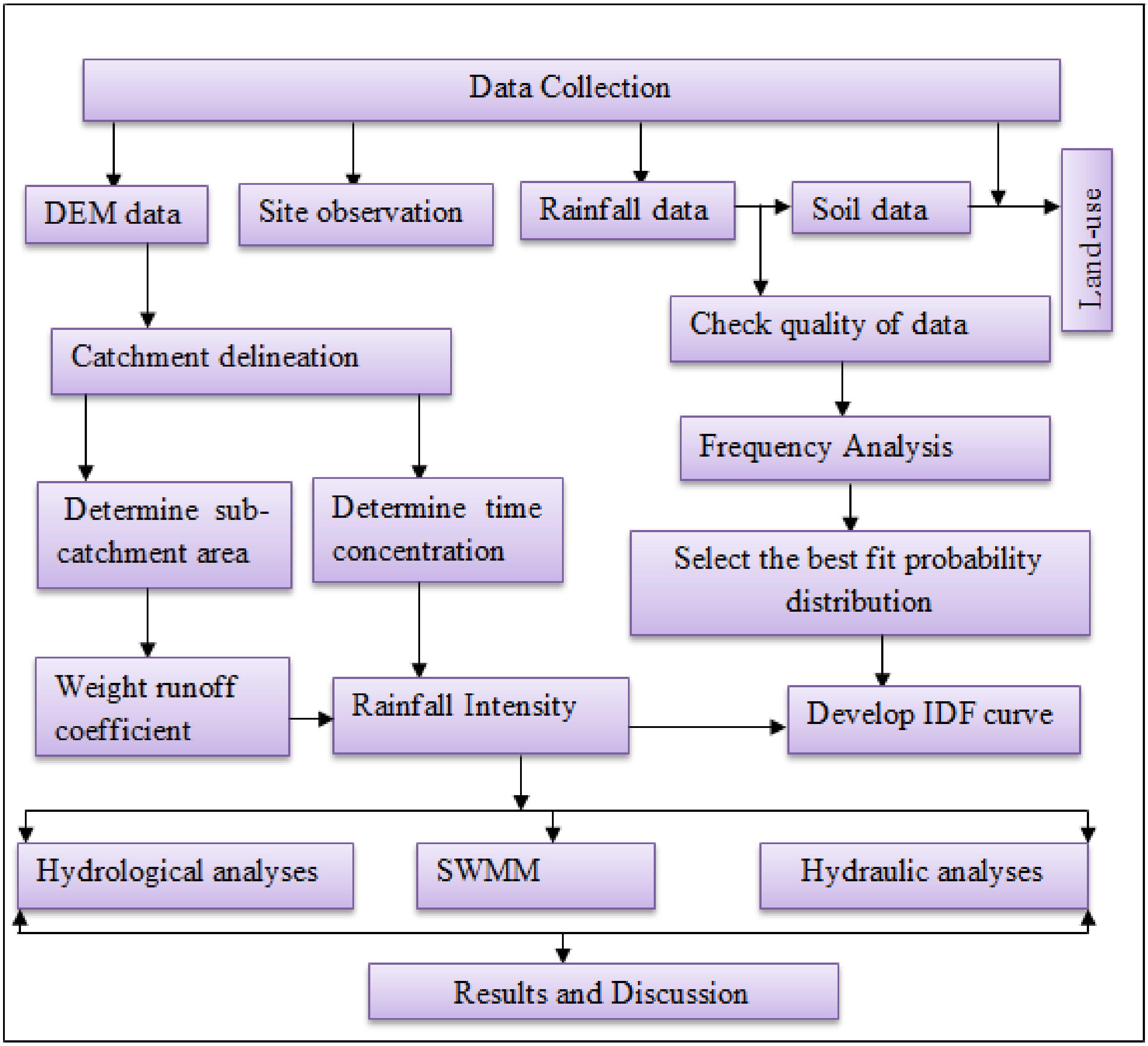

Figure 3.9: The General Framework of this study

4. RESULTS AND DISCUSSION

4.1. Current Condition and Hydraulic Capacity of Existing Drainage

4.1.1. Current Condition Drainage Systems in the Study Area

From the field observation, the current storm drainage system facilities are generally classified into closed and open drainage lines. Most of the drainage lines are open drainage channels constructed by masonry and rectangular geometric shape. The drainage service in the town was found inadequate in terms of quality and substantiate. From field observation at various, it was found that the major canals were deteriorated, undulated, collapsed, and deformed of surface condition. Due to this, surface water generated from upstream Tedecha market, the Sub-urban area, on the left side of TVC, and around Amanuel church within the town was not significantly as expected. Currently, the stormwater drainage management of the town was not efficient because of local management issues. At various places, the size of canals was not sufficient to safely dispose of the runoff, and drainage line services were not well-connected; also lateral drainage line services were neglected. The major drainage line problems that were observed in this study include lack of appropriate maintenance of the drainage facilities, drainage channels were either partially or fully filled with debris and silts, soil erosion and poor waste management system, the drainage channels were filled and blocked by silt and garbage.

A. Awareness and Understanding of the Community

During field observation, it was found that there was illegal dumping of household solid and liquid wastes which blocks the drainage systems. The creation of awareness is one of the best methods of active measure for environmental protection and managing urban drainage structures. Once the drainage structure was constructed by the responsible organizations, and the maintenance of the deteriorated, destructed and collapsed structures were weak. No immediate solution has been taken and they wait until the structures were destroyed as shown in Figure 4.1. The gap between the community and the government bodies has created a lack of awareness of environmental protection, and decrease the responsibility of the community to keep their aesthetical environment.

Figure 4.1: Lack of drainage system

Dumping solid and liquid waste materials into the drainage system was the challenge of the stormwater drainage system in Dukem Town. Normally, it consists of manufactured materials such as bottles, cans, plastic and paper wrappings, newspapers, shopping bags, cigarette packets, and remains of chat. When dumping materials such as solid wastes into drains, the drainage systems have been clogged and cause flooding over streets which lead to drainage failures. The liquid released directly into the drainage systems was gathering up pollutants from motor oils and gasoline, waste from sewer lines, and anything that would float or dissolve in water. Most of the drainage lines in town accommodated with waste disposal and blocked by liquid and solid wastes.

Figure 4.2: Blockage of the drainage system by waste materials

4.1.2. Hydraulic Capacity of Existing Drainage

In the specified study area, the most existing storm drainage facilities were generally classified as open channels. Channel types, measurement of dimensions canal, and sub-divide catchments were the key point to obtained area, hydraulic radius, Manning roughness coefficient, elevation, length, and slope of catchments to determine hydraulic capacity. Consequently, a Manning equation was used to determine the peak discharge convey into the existing canal. Roughness coefficient (n) varies considerably, depending on the characteristics of a channel smoothness and roughness of a canal. The slope of each canal segment was estimated from the differences between the highest and the lowest elevation divided by the measured canal length.

Table 4.1: Hydraulic capacity by Manning equation method

SC	Elevation		L	Slope		Canal cross-section						Manning
	H1	H2			N	B (m)	Y(m)	A(m^2)	P (m)	R (m)	V(m/s	Q(m^3/s
SC1	1955	1951	154	0.012987	0.035	0.65	1	0.65	3.25	0.20	1.1	0.72
SC2	1953	1937	626	0.025559	0.035	0.67	1.25	0.837	3.67	0.228	1.71	1.43
SC3	1960	1951	542	0.016605	0.035	0.65	0.9	0.585	2.45	0.239	1.42	0.83
SC4	1969	1960	1011	0.008902	0.025	0.65	0.95	0.617	2.33	0.265	1.56	0.96
	1961	1959	205	0.009756	0.025	0.63	0.9	0.567	2.43	0.233	1.50	0.85
	Avg. 0.009329											1.81
SC5	1967	1957	692	0.014451	0.035	0.75	1.25	0.937	3.95	0.237	1.32	1.23
SC6	1967	1963	231	0.017316	0.035	0.73	1	0.73	3.23	0.226	1.40	1.02
SC7	1957	1955	611	0.003273	0.025	0.64	1	0.64	2.64	0.242	0.89	0.57
	1957	1955	115	0.017391	0.035	0.75	1	0.75	2.75	0.273	1.73	1.30
	Avg. 0.020665											1.86
SC8	1956	1955	560	0.019643	0.035	0.65	0.8	0.52	2.15	0.242	1.55	0.81
	1945	1945	485	0.020619	0.035	0.6	0.87	0.522	2.34	0.223	1.51	0.79
	Avg. 0.040261											1.60
SC9	1957	1945	522	0.022989	0.025	0.75	1	0.75	2.75	0.273	2.55	1.91
SC10	1955	1942	679	0.019146	0.035	0.75	1	0.75	2.75	0.273	1.66	1.25

SC11	1955	1941	688	0.020115	0.035	0.75	0.85	0.637	2.4	0.266	1.67	1.07
	1942	1941	370	0.002703	0.035	0.72	0.85	0.612	2.42	0.253	0.59	0.36
	Avg. 0.022818											1.43
SC12	1945	1940	515	0.009709	0.035	0.82	0.86	0.705	2.64	0.267	1.17	0.82
SC13	1938	1935	482	0.006224	0.025	0.64	0.85	0.544	2.1	0.259	1.28	0.70
SC14	1937	1934	265	0.011321	0.025	0.45	0.67	0.301	1.79	0.168	1.30	0.39
SC15	1935	1934	106	0.009434	0.025	0.5	0.75	0.375	2	0.188	1.27	0.48
SC16	1934	1930	247	0.016194	0.035	0.67	0.9	0.603	2.45	0.246	1.43	0.86
SC17	1942	1932	608	0.016447	0.035	0.75	0.95	0.712	2.25	0.317	1.70	1.21
SC18	1930	1925	510	0.009804	0.035	0.78	1.12	0.873	3.2	0.273	1.19	1.04
SC19	1941	1932	600	0.015000	0.035	0.82	1	0.82	2.45	0.335	1.69	1.38
SC20	1932	1924	500	0.016	0.035	0.82	1	0.82	2.82	0.291	1.59	1.30

4.3. Peak Discharge of Study Area

4.3.1. Selection the Best- fit Probability Distribution

The best fit probability distribution of daily maximum rainfall based on a twenty-four-hour rainfall sample in Dukem Town was selected. General Pareto distribution method (GPD) was provided for the best fit at the rainfall gauging station analyzed in this study. The maximum expected rainfall (mm) values for return periods of 2, 5,10,25,50, and 100 years at the rainfall gauging station of the study area were calculated and the results were included in Table 4.2.

Table 4.2: Maximum rainfall values for General Pareto distribution with return periods

Probability distribution	Return period (T)					
	T-2	T-5	T-10	T-25	T-50	T-100
GPD (mm)	44.22	53.907	60.882	69.663	75.989	82.054

4.3.2. Intensity- Duration- Frequency (IDF) Curve

For this study, Intensity- Duration -Frequency (IDF) curve was developed from storm events of 31 years from 1987 to 2017 for different durations. The appropriate reduction formula which was described in the methodology section has been applied to estimate rainfall for a shorter duration (Rt) and the corresponding rainfall intensities. The intensity of rainfall in a shorter duration is higher than that of a longer duration within the same return period.

For this thesis, the developed IDF curve was applicable for the specific study area whereas the IDF curve developed by ERA 2013 was suitable for multiple regions. The IDF curve developed by ERA could not represent Dukem Town. Because there was a long-distance of kilometers between rainfall regions (RR-A2) to a specific study area and ERA IDF curve developed for thousand kilometers large scale whereas a Dukem Town IDF curve for small scale.

Therefore, Dukem Town IDF curve was developed separately to investigate the drainage infrastructure of the study area. The recorded rainfall data length for this study is 31 years from 1987 to 2017 and the ERA was 24 years only, which means ERA is seven years less data than recorded data. The IDF curve developed by recorded data of the study area is expected to provide a better result. the rainfall intensities developed by self has less numerical value than ERA. Rainfall data analysis for the determination of design point rainfall has been done and compared with the A2 rainfall region of Ethiopia. Even if the numerical rainfall intensity value estimated by ERA was greater, because of the above reason the rainfall intensities developed by Dukem Town were used for computing peak discharge.

Therefore the IDF curve developed by using Dukem Town and the rainfall intensity information for various return periods are as shown in Figure 4.4. Rainfall intensity is a key point parameter in the design of stormwater to calculate the peak-rate flow of the channel. For this study IDF curve developed from the meteorological station of the study area produces rainfall intensities for 2, 5,10,25,50, and 100 years return period with the corresponding rainfall duration of 5 minutes to 180 minutes (AppedixA, TableA11).

Table 4.3: Comparison of rainfall intensities computed by study area (self) and ERA developed

duration	T-10		T-25		T-50		T-100	
(min)	Self	ERA	Self	ERA	Self	ERA	Self	ERA
5	115.4	141.1	132.0	162.4	144.0	178.3	155.5	194.2
20	72.7	88.9	83.2	102.3	90.7	112.3	98.0	122.3
35	53.5	65.5	61.2	75.4	66.8	82.7	72.1	90.1
50	42.6	52.0	48.7	59.9	53.1	65.8	57.4	71.6
65	35.4	43.3	40.5	49.9	44.2	54.7	47.8	59.6
80	30.4	37.2	34.8	42.8	38.0	47.0	41.0	51.2
95	26.7	32.6	30.5	37.5	33.3	41.2	36.0	44.9

110	23.8	29.1	27.2	33.5	29.7	36.7	32.1	40.0
125	21.5	26.3	24.6	30.2	26.8	33.2	28.9	36.1
140	19.6	24.0	22.4	27.6	24.5	30.3	26.4	33.0
155	18.0	22.0	20.6	25.4	22.5	27.9	24.3	30.3
170	16.7	20.4	19.1	23.5	20.8	25.8	22.5	28.1
180	15.9	19.5	18.2	22.4	19.9	24.6	21.5	26.8

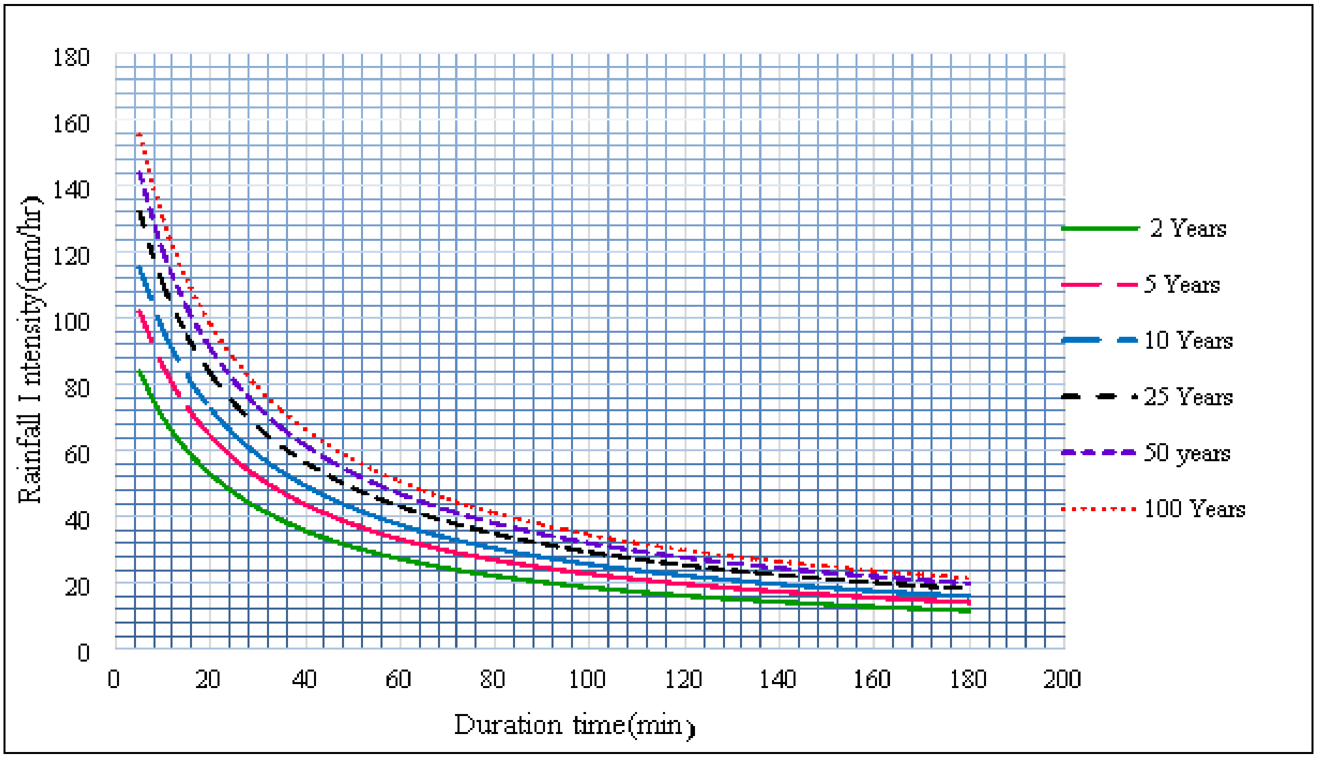

Figure 4.3: IDF curve of Dukem Town

4.3.3. Land-use

The Land-use of the study area was classified as Commercial and Business, Administration office, Residential, Public Institutions (school, health centers, churches, mosques, etc), Open spaces, Road (asphalt, cobblestone, and gravel), Green areas, Sub-urban area, and Playground. From the total area, land-use was covered by the residential area. The percentage land-use of Residential 38.99%, Commercial and Business 16.7%, Asphalt Road 0.42%, Cobblestone 13.53%, Playground 0.093%, Gravel 5.3%, Green area 0.45%, Sub-urban 9.6%, Open space 2.3%, Administration Office 0.21% and Public Institutions 12.8%

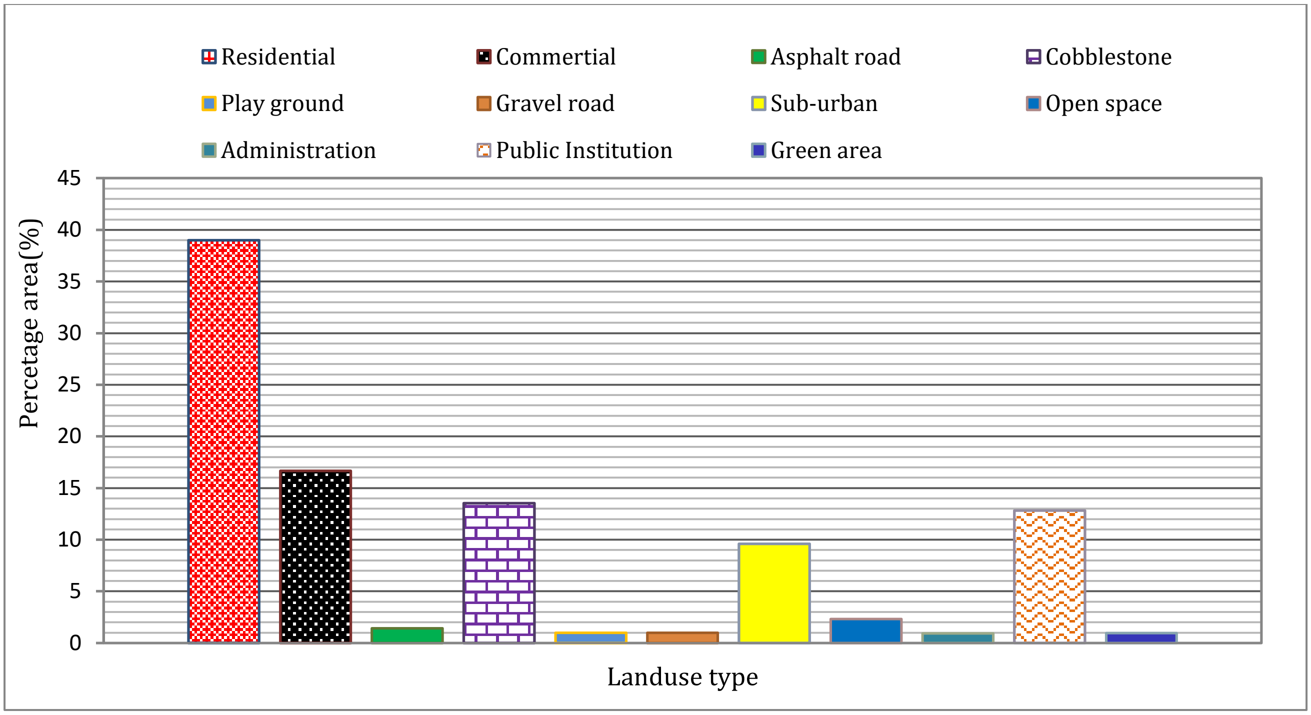

Figure 4.4: Percentage area of land-use

4.3.4. Delineation of the Study Area

A sub-catchment was an area of land containing a mix of pervious and impervious surfaces that runoff drains to a common outlet point or either anode of the drainage network. Arc GIS was used to digitize the contributing land-use area based on the direction of flow, and elevation. The verified result from the sub-catchment delineation was determining the weight runoff coefficient from land-use that contributes to the catchment of the study area as shown in Appendix B, Table 6B.

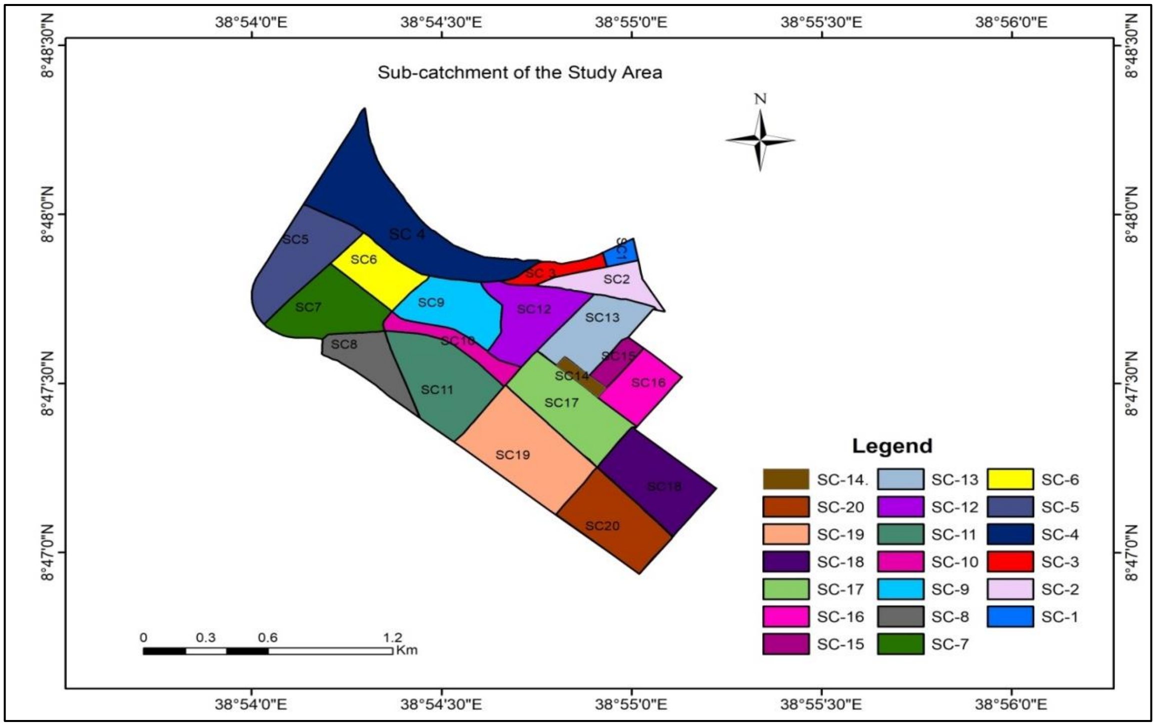

Figure 4.5: Sub-catchment of the Study Area

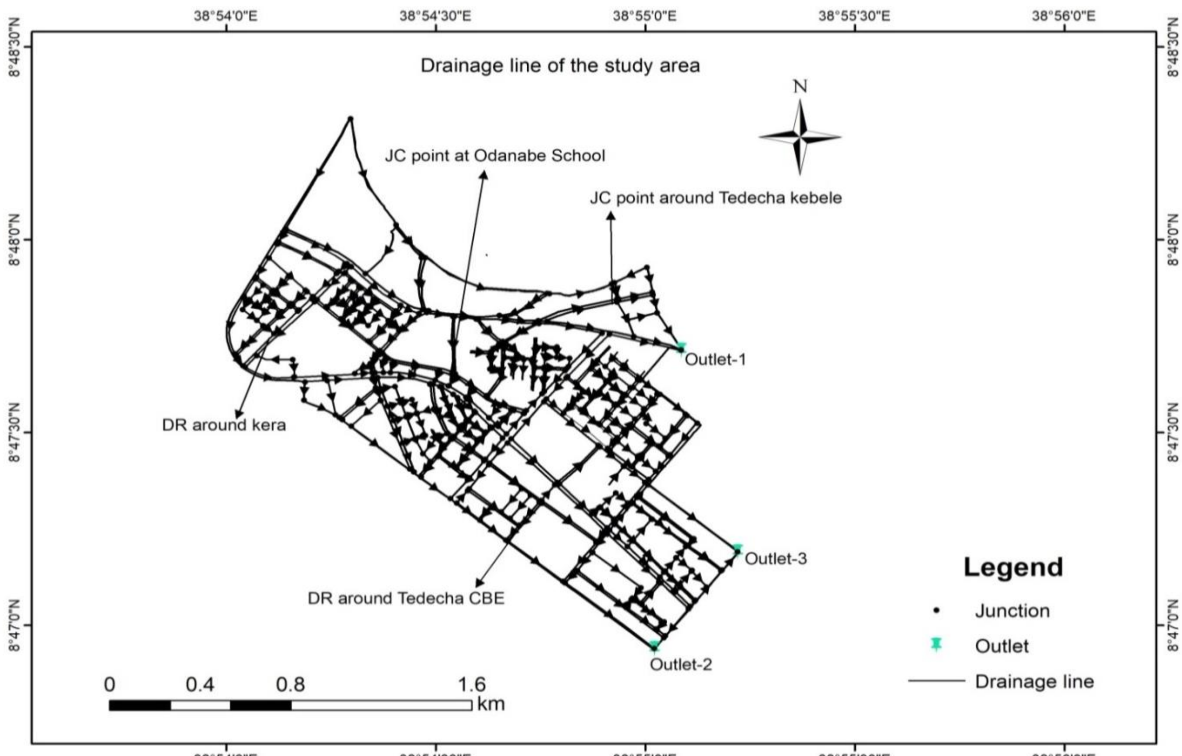

Figure 4.6: Drainage Net-work of Study Area

4.3.5. Upper Catchment Delineation that Influence the Study Area

This upper catchment was divided into three classification element units as shown in figure 4.7, the class one (C-1) was flowing in the direction of the sub-catchment four study area, class two (C-2) was flowing in the direction of the sub-catchment three study area, and class three (C-3) was flowing in the direction of sub-catchment one of this study area. The upper catchment was a natural drainage channel, and it was divided based on the principles of elevation and flow direction.

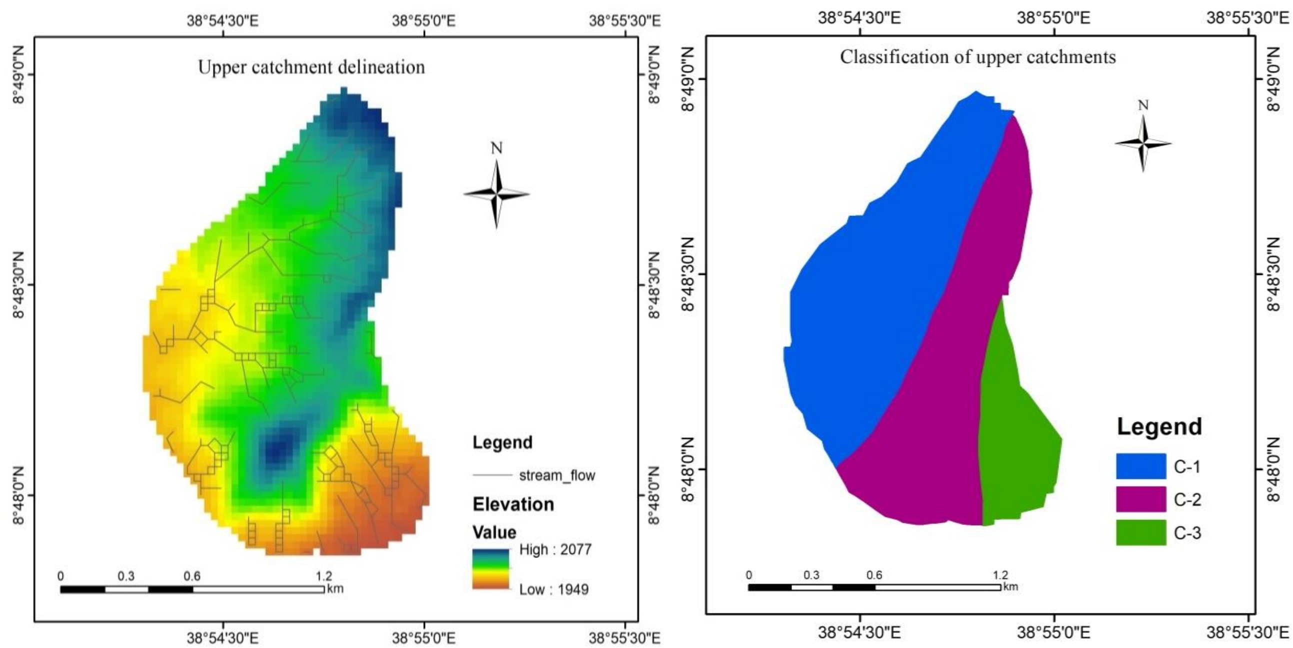

Figure 4.7: Upper catchment delineation and classification to sub-catchments

Note: *C-1*: class one, *C-2*: class two, *C-3*: class three

Table 4.4: Land-use area (ha) compositions in each upper catchment classification

land-use	C-1	C-2	C-3	Total
residential	2.560203	17.699832	11.892832	32.15287
railway	7.201296	2.872102	2.916213	12.98961
drive and walk	0.689408	2.248986		2.938394
open space	7.22778	2.742831		9.970611
cemetery	23.83314	36.736249	7.690018	68.25941
forest	27.93226		9.70445	37.63671
Total	69.4441 ha	62.3ha	32.203513ha	163.948 ha

4.3.6. Runoff Estimated for Upper Catchment

The upper catchment which contributes runoff to the study area was made significant and divides into three classes (C-1, C-2, and C-3). SCS method was used to estimate peak runoff for C-1 and C-2 which has an area greater than 50 hectares and the rational method was used to estimate peak runoff for C-3 which has an area less than 50 hectares.

A. SCS Method

According to the classification of catchments, one classified sub-catchment contains different land-use. The area of each land-use in each upper catchment classification was done. To get the curve number, the area of land use was multiplied with the corresponding curve number (CN) then divided by the sum of area in each classification sub-catchment (Appendix Table B11).As discussed under the methodological section; Equation 3.2 was used to compute direct runoff (Q), potential maximum soil water retention, and Initial Abstraction (Ia) respectively in Table 4.5.

Table 4.5: Accumulative Direct runoff and Initial abstraction ratio

			Accumulative direct runoff, Q (mm)						Ia/ P					
C	S mm	Ia mm	Accumulated rainfall, P (mm) for return period 2,5,10,25,50,100 respectively											
			44.2	53.9	60.88	69.6	75.98	82.05	T2	T5	T10	T25	T50	T100
C1	81	16.2	7.20	11.9	15.89	21.2	25.39	29.53	0.37	0.30	0.27	0.23	0.21	0.20
C2	48	9.6	14.5	21.2	26.49	33.3	38.53	43.58	0.22	0.18	0.16	0.14	0.13	0.12

Unit peak discharge (qu) in $m^3/s/km^2/mm$

Unit peak discharge was obtained from Equation 3.4 explained under the methodological section, which requires the time of concentration (tc) in hrs and initial abstraction rainfall (Ia/p) ratio as an input parameter**.** The regression coefficient was obtained by interpolation.

Table 4.6: Unit Peak discharge ($m^3/s/km^2/mm$)

C	Tc (min)	log (tc/60)	qu ($m^3/s/km^2/mm$)					
			T-2	T-5	T-10	T-25	T-50	T-100
C-1	27	-0.347	0.200	0.098	0.118	0.105	0.122	0.121
C-2	26	-0.363	0.214	0.104	0.128	0.125	0.128	0.128

Table 4.7: SCS peak discharge (m^3/s)

C	Area (ha)	Area (km^2)	$q_p = q_u * Q * A$ (m^3/s)					
			T-2	T-5	T-10	T-25	T-50	T-100
C-1	69.4	0.694	1.001	0.812	1.304	1.546	2.159	2.482
C-2	62.3	0.623	1.934	1.379	2.115	2.594	3.080	3.465

B. Rational Method

Table 4.8: Rational peak discharge for upper catchment class C-3

C	Area(ha)	Tc(min)	Rainfall intensities (mm/hr)				Q (m^3/s)			
			T-10	T-25	T-50	T-100	T-10	T-25	T-50	T-100
C-3	32.2	23.78	66.62	76.23	83.15	89.79	1.58	1.81	1.97	2.13

4.3.8. Rational Method

The whole of these study area sub-catchments was estimated by rational methods, which have an area of less than 50 hectares. Land-use area, runoff coefficient, time concentration, and sub-divide catchments were the key point to obtain the weighted runoff coefficient, rainfall intensities, and catchment area were parameters required in this method. The weighted runoff coefficient was computed for each sub-catchment Appendix B, Table B5.

Table 4.9: Rational method peak discharge

			Rainfall intensities (mm/hr)				Rational peak discharge Q (m^3/s)			
SC	Area ha	Tc min	T-10	T-25	T-50	T-100	T-10	T-25	T-50	T-100
SC1	1.49	5.09	114.9	131.55	143.50	154.9	0.29	0.33	0.43	0.48
SC2	6.91	17.89	76.62	87.67	95.63	103.2	0.90	1.03	1.35	1.52
SC3	4.26	12.91	87.92	100.60	109.74	118.4	0.62	0.71	0.93	1.04
SC4	29.24	24.94	64.97	74.34	81.09	87.56	3.17	3.63	4.75	5.34
SC5	10.66	17.98	76.45	87.48	95.42	103.0	1.80	2.06	2.70	3.03
SC6	7.84	5.42	113.4	129.84	141.63	152.9	1.57	1.79	2.34	2.64
SC7	15.11	9.23	98.78	113.03	123.30	133.1	2.69	3.08	4.03	4.54
SC8	9.24	26.85	62.42	71.42	77.91	84.13	1.17	1.34	1.75	1.97

SC9	12.70	15.93	80.68	92.32	100.70	108.7	1.84	2.11	2.76	3.10
SC10	6.56	18.90	74.69	85.46	93.23	100.6	0.98	1.12	1.46	1.65
SC11	18.86	19.23	74.08	84.76	92.46	99.84	2.80	3.20	4.19	4.72
SC12	13.56	10.14	95.85	109.67	119.63	129.2	2.25	2.57	3.37	3.79
SC13	13.01	10.43	94.93	108.62	118.49	127.9	2.10	2.40	3.14	3.53
SC14	1.74	5.82	111.7	127.82	139.43	150.5	0.30	0.34	0.44	0.50
SC15	3.17	5.00	115.3	132.02	144.01	155.5	0.61	0.70	0.92	1.03
SC16	8.67	5.87	111.5	127.61	139.20	150.3	0.94	1.07	1.40	1.58
SC17	16.75	14.36	84.29	96.45	105.20	113.6	1.80	2.06	2.70	3.03
SC18	16.23	10.34	95.22	108.95	118.84	128.3	1.63	1.70	2.22	2.50
SC19	22.77	17.23	77.95	89.19	97.29	105.1	3.82	4.38	5.73	6.44
SC20	15.50	13.15	87.28	99.87	108.94	117.6	2.58	2.95	3.86	4.34

For this study the analysis of design rainfall for the required returns period, 10 years for design, and 25 years for the checked. Because the flood hazard happened 10 years at once we need to check the maximum hazard flood for the longest return period which means 25 years based on the ERA drainage design manual (ERA,2013).

4.4. Hydraulic Performance of Drainage System of the Study Area

A. Hydraulic performance of existing drains system against the potential estimated runoff

The result of existing hydraulic capacity which was calculated by Manning equation compared with the estimated runoff for 10 years returns period. This is to identify which drainage system is sufficient or insufficient to carry the generated runoff.

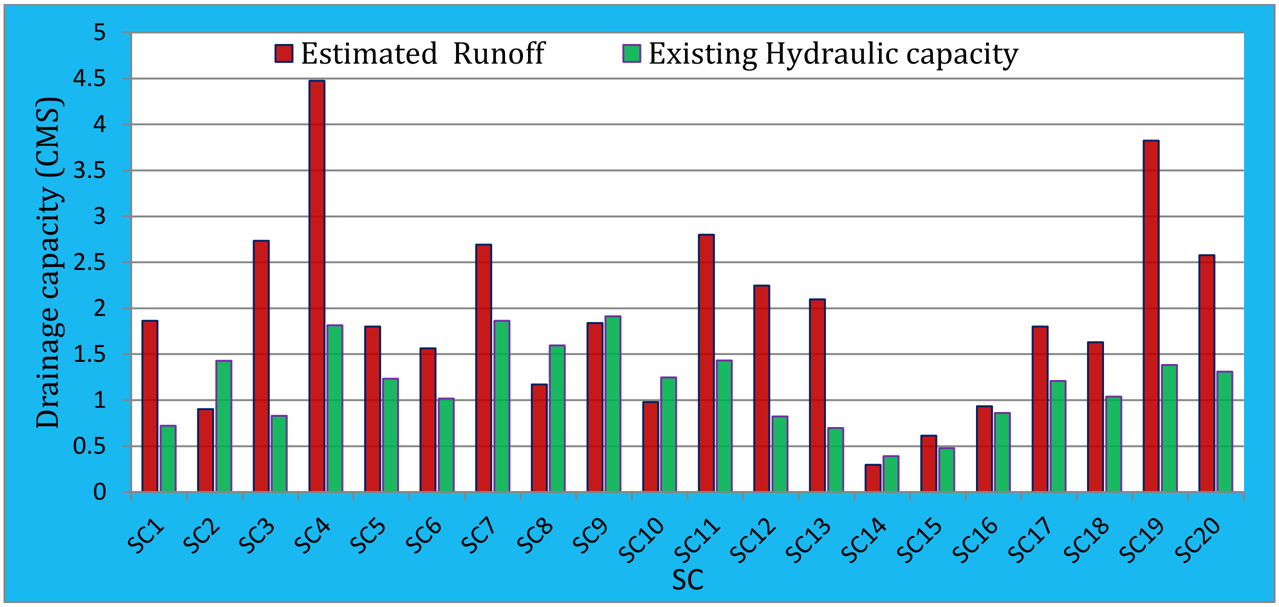

Figure 4.8: Estimated Runoff and Existing canal capacity

The estimated runoff and the existing hydraulic capacity were calculated and the result was obtained for 10 years design period as shown in Figure 4.8, the existing drainage capacity of SC-2, SC-8, SC-9, SC-10, and SC-14 were adequate and SC-1, SC-3, SC-4, SC-5, SC-6, SC-7, SC-11, SC-12, SC-13, SC-15, SC-16, SC-17, SC-18, SC-19 and SC-20 were in adequate to carry the generated runoff. The performances of the drainage system from the total study area, 25% were adequate and 75% were inadequate.

B. The performance of the existing drainage system evaluated by using the GTZ standard of drainage conditions

Table 4.10: GTZ Standard of drainage conditions

Indicators classification	Surface condition
Very good	Shapes of USWD lines as still in original design condition.
Good	No significant depressions, undulations, and deformation.
Light	The shape of the USWD lines deteriorates but still sheds water.
Severe	The total collapse of the USWD lines structure and barely passable.

Source: GTZ, 2006

Table 4.11: Performance drainage around at SC-19

Drainage type	drainage surface	coordinate at begin		coordinate at end		canal length (m)	drainage condition	Percent (%)
		X (m)	Y (m)	X (m)	Y (m)			
Open drainage	masonry	490801	971513	491037	971310	310	severe	9.40
Open drainage	masonry	490854	971577	491244	971254	507	good	15.3
Open drainage	masonry	490675	971358	949103	970988	514	severe	15.6
Open drainage	masonry	491035	970986	491244	971254	337	good	10.2
Open drainage	masonry	491053	971012	49069	971384	514	severe	15.6
Open drainage	masonry	490678	971355	490849	971570	275	severe	8.30
Open drainage	masonry	490757	971276	490863	971408	168	light	5.10
Open drainage	masonry	490781	971484	491169	971162	508	good	15.4
Open drainage	masonry	490969	971058	491075	971194	172	light	5.20
	Severe = 48.8%		Light = 10.29%				good = 40.91%	

As the above Table 4.11, from the total drains, about 48.8% of drainage lines were severely collapsed, 40.91% of drainage lines were no more significant depressions, and 10.29% of drainage lines have deteriorated. That means there was flood generated around this location that cannot be safely discharged into the near outlet. The collapsed stormwater drainage lines needed to reconstruct and maintain.

Table 4.12: Performance drainage system at SC-10, SC-9 (Oda Nabe Secondary School)

drainage type	drainage surface	coordinate at the begin		coordinate at the end		canal length (m)	drainage condition	Percent (%)
		X (m)	Y (m)	X (m)	Y (m)			
Open drainage	masonry	489879	972396	490097	972363	220	very good	10.2
Open drainage	masonry	489685	972203	489879	972396	275	good	12.7

Open drainage	masonry	489685	972203	489995	972102	321	severe	8.2
Open drainage	masonry	489994	972378	489995	972103	274	good	12.7
Open drainage	masonry	489994	972102	490142	971996	189	good	15.40
Open drainage	masonry	490067	972204	490158	972197	92	severe	4.30
Open drainage	masonry	490119	972240	490118	972209	32	very good	1.5
Open drainage	masonry	490160	972207	490118	972209	43	light	2.0
Open drainage	masonry	490167	972156	490212	972248	111	Very good	5.1
Open drainage	masonry	490096	972364	490212	972247	165	good	7.6
Open drainage	masonry	490139	97995	490212	972247	270	light	12.5
Open drainage	masonry	490002	972100	490140	971996	172	Very good	7.9
Severe = 12.8%			Good = 48%		Light= 15%		Very good =24.2%	

From the above Table 4.12, the performance drainage around Oda Nabe Secondary school about 12.8% drainage lines were severely degraded, 48% drainage lines were no significance depressions, 15% drainage lines have deteriorated but still sheds water and 24.2% drainage lines were original design condition. That shows some parts deteriorated and the major parts were able to safely dispose runoff. The severely degraded drainage lines needed to reconstruct and maintain to carry the generated runoff.

Table 4.13: Performance of existing drainage system at SC-5 (around Kera)

drainage type	drainage surface	coordinateat begin		coordinate at end		canal length (m)	drainage condition	percent (%)
		X (m)	Y (m)	X (m)	Y (m)			
Open drainage	masonry	489042	972200	489164	972331	179	good	14.7
Open drainage	masonry	489047	972191	489171	972325	183	severe	15.0

Open drainage	masonry	489073	972408	489164	972331	120	severe	9.80
Open drainage	masonry	489083	972408	489161	972296	175	light	14.3
Closed drainage	masonry	489075	972408	489110	970593	48	severe	3.90
Closed drainage	masonry	489084	972407	489103	972445	43	good	3.50
Open drainage	masonry	489076	972465	489170	972393	118	very good	9.70
Open drainage	masonry	489070	972143	489315	972403	355	light	29.1
severe = 28.8%			Light = 43%		Good = 18.5%		Very good = 9.7%	

From the above Table 4.13, 28.8% of drainage lines have collapsed, 43% of drainage lines have deteriorated, and 18.5% of drainage lines have no significant depressions and 9.7% drainage lines are still in original design condition (have performed very well). Therefore some parts were safely discharged (passes run-off) to the near outlet. The severe and light conditions needed to re-construct and maintain the deteriorate stormwater drainage lines.

Table 4.14: Performance of existing drainage system around SC-13

drainage type	drainage surface	coordinate at begin		coordinate at end		canal length	drainage condition	Percent (%)
		X (m)	Y (m)	X (m)	Y (m)			
Open drainage	masonry	490677	972289	490938	972226	269	severe	24.33
Open drainage	masonry	490701	972174	490809	972084	140	light	12.66
Open drainage	masonry	490631	972172	490776	972043	193	severe	17.45
Open drainage	masonry	490398	971985	490672	971745	359	good	32.47
Open drainage	masonry	490670	972058	490636	972018	52.4	severe	4.740
Open drainage	masonry	490636	972018	490705	971957	92.4	good	8.360
Severe = 46%				Good = 41%			Light = 13%	

As shown in Table 4.14, 46% of drainage lines were collapsed and barely passable, 41% were no more significant depressions of drainage lines and 13% of drainage lines have deteriorated. Major parts of the drainage lines did not pass safely runoff and it needs to re-construct and maintain.

Table 4.15: Performance of existing drainage around SC-11 (Michot hotel)

Drainage type	drainage surface	coordinate at begin		coordinate at end		Canal length (m)	drainage condition	percent (%)
		X (m)	Y (m)	X (m)	Y (m)			
open drainage	masonry	490215	971800	489982	971507	374	good	14.5
Open drainage	masonry	489663	972087	490215	971800	650	severe	25.2
Open drainage	masonry	490086	971926	489845	971616	395	good	15.3
closed drainage	masonry	489741	972041	489680	972032	66	severe	2.6
Open drainage	masonry	489743	972042	489896	971677	395	light	15.3
closed drainage	masonry	489773	971762	489826	971781	57	Very good	2.2
Open drainage	masonry	490159	971850	490003	971656	245	Very good	9.5
Open drainage	masonry	490006	971989	490130	971817	219	good	8.5
Open drainage	masonry	489814	971979	489706	971944	113	good	4.4
Open drainage	masonry	489680	972032	489741	972041	66	severe	2.6
Severe = 30.3%			Light = 15%		Good = 42.7%		Very good = 12%	

As shown above Table 4.15, from the total drains, about 30.3% of drainage lines were severely degraded, 42.7% of drainage lines have no significant depressions and deformation, 15% of drainage lines deteriorated but still sheds water, and 12% shapes of drainage lines were still in original condition. Therefore some parts were safely discharged (passes run-off) to the near outlet. The severe and light conditions needed to re-construct and maintain the deteriorate stormwater drainage lines.

4.4.1. SWMM Simulation

The Stormwater Management Model is a dynamic rainfall-runoff simulation model used for a single event or long-term (continuous) simulation of runoff quantity and quality in urban areas. In the stormwater management model, a given catchment can be developed as a set of physical components include sub-catchments, conduits, junctions, and outlets (Rossman, 2010). The total study area was divided into twenty sub-catchments with a 234.25 ha area.

Table 4.16: Maximum flow in links by SWMM (m3/s)

Drains	Link	From	To	SC	Max. flow (m^3/s)
drl 28	C28	C23,C24	C3	SC1	1.63
drl 2	C2	C1	outlet-1	SC2	1.397
drl 7	C7	C5	C9	SC7	1.406
drl 4	C4	JC5	C18	SC5	1.392
drl12	C12	C16	C14	SC9	1.375
drl 23	C23	JC26	C28	SC3	1.884
drl 1	C1	JC1	C28	SC4	1.208
drl 42	C42	C41	outlet-2	SC20	1.203
drl 3	C3	C25	outlet-1	SC2	1.295
drl 9	C9	C8,C7	C21	SC11	1.068
drl 14	C14	C12,C13	C15	SC12	1.056
drl 5	C5	JC7	C7	SC6	0.987
drl 15	C15	C14	C38	SC10	0.97
drl 21	C21	C11,C9	C41	SC11	0.936
drl 11	C11	C10	C21	SC8	0.894
drl 46	C46	C44,C36	outlet-3	SC18	0.893
drl 36	C36	C35,C31	C46	SC16	0.868
drl 16	C16	C7,C8	C17	SC10	0.851
drl 38	C38	C15	C39	SC17	0.842
drl 39	C39	C35,C31	C45	SC18	0.81
drl 41	C41	C21,C17	C42	SC19	0.792
drl 8	C8	C4	C9	SC7	0.675
drl 45	C45	C39	outlet-3	SC18	0.598
drl 40	C40	JC44	outlet-2	SC20	0.572
drl 35	C35	C37,C34	C36	SC14	0.468
drl 43	C43	JC43	C42	SC19	0.451
drl 34	C34	C33,C32	C35	SC15	0.433
drl 17	C17	C16	C41	SC11	0.366
drl 10	C10	JC13	C11	SC8	0.074

4.4.1.1. Surface Runoff Simulation

Runoff simulations for different land-use scenarios in the SWMM required a substantial number of input parameters. The sensitive parameters of model input parameters were used for the simulation of the model. The model hydrology parameters within the SWMM runoff are: sub-catchment area, sub-catchment width, maximum depth of the canal, inverted elevation, canal length, Manning's roughness of the canal, and slope of the canal were input data used for the simulations of the model and listed under Appendix C.

Table 4.17: Runoff simulated to 10 yrs and 25yrs

SC	SC1	SC2	SC3	SC4	SC5	SC6	SC7	SC8	SC9	SC10	SC11	SC12	SC13	SC14	SC15	SC16	SC17	SC18	SC19	SC20
SWMM10 yrs	0.25	1.02	0.61	3.43	1.45	1.24	2.2	1.22	1.83	0.9	2.73	2.21	1.86	0.25	0.47	1.08	1.91	1.83	3.21	2.32
SWMM25 yrs	0.29	1.19	0.71	4	1.7	1.43	2.57	1.4	2.12	1.06	3.19	2.56	1.8	0.29	0.55	1.27	2.22	2.13	3.77	2.7

The simulated runoff from SWMM 5.1 within the design and checked period was 32 m^3/s, and 36.95m^3/s respectively,

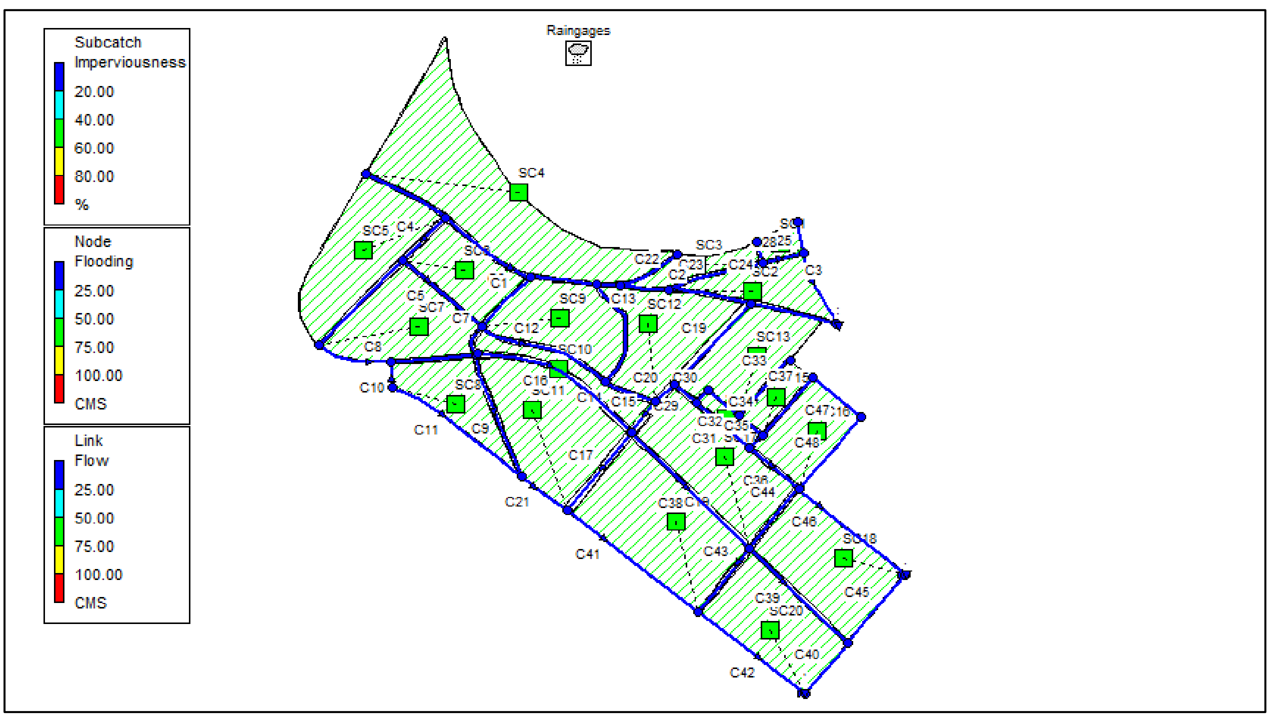

Figure 4.9: Map of Sub-catchment by SWMM

5. SUMMARY, CONCLUSION, AND RECOMMENDATIONS

5.1. Summary

Most cities are facing drainage problems due to a change in land-use/land management because of rapid urbanization. As a result changes in the runoff characteristics within the town increase runoff and greater susceptibility to flooding. The objective of this study was to analyze the performance stormwater drainage systems at Dukem Town. Rainfall data was a key point input hydrological models dealing with the runoff process. The daily rainfall data was obtained from the NMSA for thirty-one years of recorded data. The missing rainfall data of the rainfall stations were filled up using suitable techniques and data of the neighboring rainfall stations.

The current conditions of the drainage structure were assessed through field observation. The field observation revealed that the quality of storm drainage structure in the study area was severely underperforming e.g. at SC-19, 48.8%, Oda Nabe Secondary School 12.8%, Kera 28.84%, SC-13,46%, and SC-11,30.3% were also severely degraded. The major drainage problems were observed due to lack of appropriate maintenance of the existing drainage facilities; drainage channels were either partially or filled with debris, silts, and unavailable drainage line system at the proper place.

This study also evaluated the performance of the existing stormwater drainage systems by using NMSA data, and IDF curve developed by study area. The estimation of peak discharge done using the rational method, the SCS method while Manning equation was used to determine the capacity of existing drainage structures.

5.2. Conclusion

The existing drainage lines were inadequate to carry runoff. This is because of lack of a well-connected drainage line network, illegal dumping of solid and liquid waste materials were disposed into stormwater drainage, lack of waste management, and awareness of community and the natural location of Dukem Town contribute runoff from the upper catchment.

A field survey was conducted to measure the existing canal dimension system by using a tape meter. After hydraulic parameters were known, the Manning equation was used to estimate the discharge of existing drainage channels.

This study identified the best-fit probability distribution of annual or daily maximum rainfall based on the twenty-four-hour sample. Thus, based on the scores obtained from Easy fit 5.5 software professions, the General Pareto distribution method was found the best fit distribution at Dukem Town rainfall gauging station. The appropriate reduction equation developed by ERA was applied to estimate rainfall for shorten duration (Rt) and the corresponding rainfall intensities (It). Consequently, the IDF curve developed for the present study was used to estimate peak rate discharge instead of the ERA-developed IDF curve as the latter was not suitable for specific areas.

Performances of the drainage system from the total study area, 25% were adequate and 75% were inadequate. This is because the estimated runoff is greater than the existing canal capacity.

5.3. Recommendations

- ✓ Concerning body will establish well-connect drainage lines, waste management techniques to safely release wastewater, to suitably manage and use stormwater drainage lines as it will anticipate.
- ✓ Giving awareness to the community and develop a strong relationship between the communities and government bodies concerning environmental protection, and their participation to maintain an aesthetical environment.
- ✓ The major portion of the drainage system was found to perform poorly; it needs immediate attention to re-construct and maintain drainage structure to safely pass the generate runoff.
- ✓ Recurrently clear of the drainage canals and maintain it before rainfall seasons.
- ✓ At SC-3, SC-4 there is runoff contributed to the town from the upper catchment, so to avoid this problem retaining wall will be recommended from edge SC-4 to edge SC-1.

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APPENDICES

Appendix A: Hydrological Analysis and Constants

Table A1: Outlier test Kn values

Sample size (n)	Kn	Sample size (n)	kn	Sample size (n)	kn
10	2.036	29	2.549	48	2.753
11	2.088	30	2.563	49	2.76
12	2.134	31	2.577	50	2.768
13	2.175	32	2.591	55	2.804
14	2.213	33	2.604	60	2.837
15	2.247	34	2.616	65	2.866
16	2.279	35	2.628	70	2.893
17	2.309	36	2.639	75	2.917
18	2.335	37	2.65	80	2.94
19	2.361	38	2.661	85	2.961
20	2.385	39	2.671	90	2.981
21	2.408	40	2.682	95	3.00
22	2.429	41	2.692	100	3.017
23	2.448	42	2.7	110	3.049
24	2.467	43	2.71	120	3.078
25	2.486	44	2.719	130	3.104
26	2.502	45	2.727	140	3.129
27	2.519	46	2.736		
28	2.534	47	2.744		

Table A2: Outlier tests results

Date	Max RF(x)	Rank(m)	$Y=\log^{x}$	Probability % ,p=1/T	T=(N+1)/m
6/21/1999	78.0	1	1.892	1.7	60
10/29/2006	74.4	2	1.872	3.3	30
7/2/1999	74.0	3	1.869	5.0	20
8/24/1998	70.0	4	1.845	6.7	15
6/6/1996	62.0	5	1.792	8.3	12
3/16/2011	61.0	6	1.785	10.0	10

3/16/2012	61.0		1.785	10.0	10
6/12/1996	57.8	8	1.762	13.3	8
7/20/1997	57.0	9	1.756	15.0	7
2/21/2006	55.7	10	1.746	16.7	6
8/16/1996	54.8	11	1.739	18.3	5
8/10/1999	53.6	12	1.729	20.0	5
2/20/2006	52.4	13	1.719	21.7	5
6/19/2013	51.0	14	1.708	23.3	4
9/3/2011	50.7	15	1.705	25.0	4
7/19/2016	50.0	16	1.699	26.7	4
8/9/1997	49.6	17	1.695	28.3	4
10/19/1997	49.1	18	1.691	30.0	3
7/28/1993	49.0	19	1.690	31.7	3
5/14/2017	48.9	20	1.689	33.3	3
7/11/2006	48.1	21	1.682	35.0	3
8/11/2011	48.0	22	1.681	36.7	3
7/10/2016	48.0		1.681	36.7	3
7/17/2000	47.0	23	1.672	38.3	3
6/21/2004	46.2	24	1.665	40.0	3
9/26/2008	45.9	25	1.662	41.7	2
3/5/1987	45.8	26	1.661	43.3	2
3/23/2003	45.6	27	1.659	45.0	2
8/26/2011	45.2	28	1.655	46.7	2
2/20/1987	45.0	29	1.653	48.3	2
7/25/2002	44.7	30	1.650	50.0	2
8/7/2010	44.6	31	1.649	51.7	2
8/21/1996	44.2	32	1.645	53.3	2
8/4/2004	43.0	23	1.633	38.3	3
8/16/2011	42.8	34	1.631	56.7	2
7/24/2003	42.6	35	1.629	58.3	2
3/30/1990	42.3	36	1.626	60.0	2

7/18/1996	42.0	37	1.623	61.7	2
5/24/1987	41.6	38	1.619	63.3	2
4/13/2016	41.0	39	1.613	65.0	2
4/16/2010	40.5	40	1.607	66.7	2
8/5/2012	40.3	41	1.605	68.3	1
7/6/2017	40.2	42	1.604	70.0	1
8/20/1996	40.1	43	1.603	71.7	1
3/29/2001	39.8	44	1.600	73.3	1
8/6/1993	39.0	45	1.591	75.0	1
7/12/2010	38.8	46	1.589	76.7	1
7/6/1990	38.5	47	1.585	78.3	1
1/27/2003	38.3	48	1.583	80.0	1
4/8/1997	38.2	49	1.582	81.7	1
7/1/2007	38.1	50	1.581	83.3	1
7/7/2001	38.0	51	1.580	85.0	1
4/10/2008	38.0	52	1.580	86.7	1
7/18/1999	37.9	53	1.579	88.3	1
9/28/1989	37.8	54	1.577	90.0	1
8/17/2008	37.4	55	1.573	91.7	1
7/30/2011	37.1	56	1.569	93.3	1
4/29/2005	37.0	57	1.568	95.0	1
7/11/2016	37.0		1.568	95	1
Mean	***47.2***		***1.7***		
St.DEV	*9.875*		*0.082*		
Kn 2.8304					

Outlier Test

*Lowest Test, YL = Mean- Std. Deviation*kn = 1.468*

*Highest Test, YH= Mean+ Std. Deviation*kn = 1.932*

Lowest Test in X= antilog YL = 29.37 Minimum outlier in mm

Highest Test in X = antilog YH = 85.5 Maximum outlier in mm

Table A3: The goodness of fit Summary

#	Distribution	Kolmogorov Smirnov		Anderson Darling		Chi-Squared	
		Statistic	Rank	Statistic	Rank	Statistic	Rank
1	Beta	0.15117	34	6.7934	46	N/A	
2	Burr	0.11084	24	20.412	52	N/A	
3	Burr (4P)	0.19576	45	11.339	48	N/A	
4	Cauchy	0.2123	51	5.0505	42	11.701	35
5	Chi-Squared	0.12905	31	2.5623	24	6.9663	22
6	Chi-Squared (2P)	0.12367	28	2.4132	23	4.3138	6
7	Dagum	0.43279	55	39.192	57	81.233	45
8	Dagum (4P)	0.10799	21	5.4423	45	N/A	
9	Erlang	0.15339	36	2.6659	26	4.6073	8
10	Erlang (3P)	0.08037	10	0.69583	4	11.193	33
11	Error	0.18774	43	4.3636	37	12.215	37
12	Error Function	0.99907	59	764.12	60	2.6826E+5	50
13	Exponential	0.52984	56	26.909	53	123.61	46
14	Exponential (2P)	0.08037	9	0.81281	5	11.193	32
15	Fatigue Life	0.12099	27	2.3238	22	7.0625	24
16	Fatigue Life (3P)	0.09274	19	0.92666	9	6.8907	21
17	Frechet	0.08967	18	0.87817	8	3.9447	5
18	Frechet (3P)	0.08841	16	1.0988	13	7.4935	25
19	Gamma	0.15237	35	2.6905	27	4.6454	9
20	Gamma (3P)	0.07662	8	4.534	39	N/A	

21	Gen. Extreme Value	0.07455	6	0.87271	7	5.7096	14
22	Gen. Gamma	0.13114	32	2.7335	28	6.3244	16
23	Gen. Gamma (4P)	0.06956	4	4.3517	36	N/A	
24	Gen. Pareto	0.05798	1	0.35729	1	2.5121	3
25	Gumbel Max	0.12799	30	1.6017	15	3.4471	4
26	Gumbel Min	0.20756	48	12.586	49	15.23	40
27	Hypersecant	0.16936	41	3.8906	35	9.6101	30
28	Inv. Gaussian	0.14624	33	3.4642	29	9.1501	29
29	Inv. Gaussian (3P)	0.0868	15	0.99851	11	9.8897	31
30	Johnson SB	0.06213	3	0.37379	2	2.4943	2
31	Kumaraswamy	0.08132	12	5.1716	44	N/A	
32	Laplace	0.19769	46	4.7108	40	14.424	39
33	Levy	0.5371	57	37.916	56	242.07	48
34	Levy (2P)	0.20977	50	4.8251	41	29.71	41
35	Log-Gamma	0.11712	25	2.0143	20	4.6931	10
36	Log-Logistic	0.12403	29	1.9948	19	6.636	19
37	Log-Logistic (3P)	0.08269	13	1.2094	14	13.783	38
38	Log-Pearson 3	0.08559	14	0.86277	6	2.3798	1
39	Logistic	0.15689	39	3.6858	32	7.9126	27
40	Lognormal	0.11889	26	2.2639	21	7.0188	23
41	Lognormal (3P)	0.08069	11	0.99226	10	11.657	34
42	Nakagami	0.201	47	3.6769	31	6.5651	18
43	*Normal*	*0.15578*	*37*	*3.8623*	*34*	*6.7853*	*20*
44	Pareto	0.07542	7	2.5776	25	4.3886	7

45	Pareto 2	0.62863	58	39.789	58	145.97	47
46	Pearson 5	0.11075	23	1.917	18	5.181	13
47	Pearson 5 (3P)	0.0893	17	1.0694	12	7.7146	26
48	Pearson 6	0.11013	22	1.8671	17	4.9494	12
49	Pearson 6 (4P)	0.07408	5	4.4399	38	N/A	
50	Pert	0.10706	20	1.8474	16	6.3452	17
51	Power Function	0.17469	42	8.4268	47	N/A	
52	Rayleigh	0.36067	54	12.801	50	42.811	43
53	Rayleigh (2P)	0.16071	40	3.8358	33	12.016	36
54	Reciprocal	0.33606	53	27.148	54	53.661	44
55	Rice	0.19246	44	3.5637	30	6.2003	15
56	Student's t	0.99951	60	641.34	59	1.6969E+5	49
57	Triangular	0.23834	52	14.841	51	31.775	42
58	Uniform	0.20788	49	28.905	55	N/A	
59	Weibull	0.15634	38	5.1601	43	8.9404	28
60	Weibull (3P)	0.05834	2	0.48544	3	4.8843	11
61	Johnson SU	No fit					

Table A4: Frequency factors for the rational method

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

Table A5: Rainfall of shorter duration in mm using peak- over-threshold value

T (Minu)	T (hrs)	R_t for 2, 5, 10, 25,50, 100 Return periods and R_{24} respectively					
		R_{24} =					
		44.22	53.907	60.882	69.663	75.989	82.054
5	0.08	6.98	8.51	9.62	11.00	12.00	12.96
10	0.17	11.66	14.21	16.05	18.36	20.03	21.63
15	0.25	15.03	18.32	20.69	23.68	25.83	27.89
20	0.33	17.60	21.46	24.23	27.73	30.25	32.66
25	0.42	19.64	23.94	27.03	30.93	33.74	36.44
30	0.50	21.30	25.96	29.32	33.55	36.59	39.52
35	0.58	22.68	27.65	31.23	35.73	38.97	42.08
40	0.67	23.86	29.08	32.85	37.58	41.00	44.27
45	0.75	24.87	30.32	34.24	39.18	42.74	46.15
50	0.83	25.76	31.40	35.47	40.58	44.27	47.80
55	0.92	26.55	32.36	36.55	41.82	45.62	49.26
60	1.00	27.25	33.22	37.51	42.93	46.82	50.56
65	1.08	27.88	33.99	38.38	43.92	47.91	51.73
70	1.17	28.45	34.68	39.17	44.82	48.89	52.79
75	1.25	28.97	35.32	39.89	45.64	49.78	53.76
95	1.58	30.68	37.40	42.23	48.33	52.71	56.92
100	1.67	31.03	37.83	42.72	48.88	53.32	57.58
105	1.75	31.36	38.23	43.18	49.40	53.89	58.19
110	1.83	31.67	38.61	43.60	49.89	54.42	58.77
115	1.92	31.96	38.97	44.01	50.35	54.93	59.31
120	2.00	32.24	39.30	44.39	50.79	55.40	59.82
125	2.08	32.50	39.62	44.75	51.20	55.85	60.31
130	2.17	32.75	39.92	45.09	51.59	56.28	60.77
135	2.25	32.99	40.21	45.41	51.96	56.68	61.21
140	2.33	33.21	40.49	45.72	52.32	57.07	61.62
145	2.42	33.42	40.75	46.02	52.66	57.44	62.02

150	2.50	33.63	41.00	46.30	52.98	57.79	62.40
155	2.58	33.82	41.23	46.57	53.29	58.13	62.76
160	2.67	34.01	41.46	46.83	53.58	58.45	63.11
165	2.75	34.19	41.68	47.08	53.87	58.76	63.45
170	2.83	34.37	41.89	47.31	54.14	59.06	63.77
175	2.92	34.53	42.10	47.54	54.40	59.34	64.08
180	3	34.69	42.29	47.77	54.65	59.62	64.38

Table A6: Yearly precipitation and accumulative of the station and neighboring (mm)

Year	Dukem	Mojo	Chefedonsa	Average Neighbors	Acc. Dukem	Accu. Neighbors	Acc. Mojo	Accu. Chefedonsa
1987	860.0	865.0	918.0	891.5	860.0	891.5	865.0	918.0
1988	752.0	1061.5	1064.7	1063.1	1611.9	1954.6	1926.5	1982.7
1989	669.2	1033.6	1135.0	1084.3	2281.1	3038.9	2960.1	3117.7
1990	905.4	984.5	925.3	954.9	3186.5	3993.8	3944.6	4043.0
1991	712.0	878.6	939.0	908.8	3898.5	4902.6	4823.2	4982.0
1992	850.0	798.0	819.0	808.5	4748.5	5711.1	5621.2	5801.1
1993	780.0	829.4	959.6	894.5	5528.5	6605.7	6450.6	6760.7
1994	684.6	697.7	657.1	677.4	6213.1	7283.0	7148.3	7417.7
1995	592.2	648.3	576.8	612.5	6805.2	7895.6	7796.7	7994.5
1996	1077.4	1060.8	1002.9	1031.9	7882.6	8927.4	8857.5	8997.4
1997	854.4	760.3	663.2	711.8	8737.0	9639.2	9617.8	9660.6
1998	984.7	1033.9	881.2	957.6	9721.7	10596.7	10651.7	10541.8
1999	920.8	1172.8	738.8	955.8	10642.5	11552.5	11824.5	11280.6
2000	841.7	852.1	699.5	775.8	11484.2	12328.3	12676.6	11980.1
2001	864.5	788.1	675.7	731.9	12348.7	13060.2	13464.7	12655.8
2002	648.3	564.3	625.3	594.8	12997.0	13655.0	14029	13281.1
2003	1089.3	1060.1	972.1	1016.1	14086.3	14671.1	15089.1	14253.2
2004	826.3	1281.4	773.5	1027.4	14912.6	15698.5	16370.4	15026.7
2005	915.3	1579.0	1028.7	1303.9	15827.9	17002.4	17949.4	16055.4
2006	999.2	1042.0	1033.2	1037.6	16827.1	18040.0	18991.4	17088.6
2007	851.4	983.2	1113.0	1048.1	17678.5	19088.1	19974.6	18201.6

2008	821.0	1441.4	904.5	1173.0	18499.5	20261.0	21416.0	19106.1
2009	640.6	952.0	689.9	820.9	19140.1	21082.0	22368.0	19796.0
2010	919.1	1110.4	1030.1	1070.3	20059.2	22152.2	23478.4	20826.1
2011	911.3	578.6	690.4	634.5	20970.5	22786.8	24057.0	21516.5
2012	684.1	1288.6	880.4	1084.5	21654.6	23871.2	25345.6	22396.9
2013	834.2	1096.4	1320.2	1208.3	22488.8	25079.5	26442	23717.1
2014	615.0	875.1	757.3	816.2	23103.8	25895.7	27317.1	24474.4
2015	663.0	856.8	607.0	731.9	23766.8	26627.7	28173.9	25081.4
2016	851.0	1234.1	862.2	1048.2	24617.8	27675.8	29408.1	25943.6
2017	724.8	660.5	648.5	654.5	25342.6	28330.3	30068.6	26592.1

Table A7: Twenty-four-hour Rainfall depth (mm)

Return period	T-2	T-5	T-10	T-25	T-50	T-100
R- Dukem	44.2	53.907	60.882	69.663	75.989	82.054
RR-A2	51.92	65.52	74.45	85.7	94.7	102.45

Note: RR- Rainfall Region, R- Rainfall

Table A8: Comparison of rainfall intensities computed by study area (self) and ERA developed

duration (min)	Rainfall intensity(mm/hr)											
	T-2		T-5		T-10		T-25		T-50		T-100	
	Self	ERA	Self	ERA	Self	ERA	Self	ERA	Self	ERA	Self	ERA
5	83.8	98.4	102	124.2	115.4	141.1	132.0	162.4	144.0	178.3	155.5	194.2
10	69.9	82.1	85.3	103.6	96.3	117.7	110.2	135.5	120.2	148.8	129.8	162.0
15	60.1	70.6	73.3	89.1	82.8	101.2	94.7	116.5	103.3	127.9	111.6	139.3
20	52.8	62.0	64.4	78.2	72.7	88.9	83.2	102.3	90.7	112.3	98.0	122.3
25	47.1	55.3	57.5	69.8	64.9	79.3	74.2	91.3	81.0	100.3	87.4	109.2
30	42.6	50.0	51.9	63.1	58.6	71.7	67.1	82.5	73.2	90.6	79.0	98.7
35	38.9	45.6	47.4	57.6	53.5	65.5	61.2	75.4	66.8	82.7	72.1	90.1
40	35.8	42.0	43.6	53.0	49.3	60.2	56.4	69.4	61.5	76.1	66.4	82.9
45	33.2	38.9	40.4	49.1	45.7	55.8	52.2	64.3	57.0	70.5	61.5	76.8
50	30.9	36.3	37.7	45.8	42.6	52.0	48.7	59.9	53.1	65.8	57.4	71.6
55	29.0	34.0	35.3	42.9	39.9	48.8	45.6	56.1	49.8	61.6	53.7	67.1
70	24.4	28.6	29.7	36.1	33.6	41.1	38.4	47.3	41.9	51.9	45.2	56.5
75	23.2	27.2	28.3	34.3	31.9	39.0	36.5	44.9	39.8	49.3	43.0	53.7

85	21.1	24.8	25.7	31.3	29.0	35.5	33.2	40.9	36.3	44.9	39.1	48.9
90	20.2	23.7	24.6	29.9	27.8	34.0	31.8	39.1	34.7	43.0	37.5	46.8
95	19.4	22.7	23.6	28.7	26.7	32.6	30.5	37.5	33.3	41.2	36.0	44.9
100	18.6	21.9	22.7	27.6	25.6	31.3	29.3	36.1	32.0	39.6	34.5	43.1
110	17.3	20.3	21.1	25.6	23.8	29.1	27.2	33.5	29.7	36.7	32.1	40.0
120	16.1	18.9	19.7	23.9	22.2	27.1	25.4	31.2	27.7	34.3	29.9	37.3
135	14.7	17.1	17.9	21.6	20.2	24.5	23.1	28.2	25.2	31.0	27.2	33.7
140	14.2	16.7	17.4	21.1	19.6	24.0	22.4	27.6	24.5	30.3	26.4	33.0
150	13.5	15.8	16.4	19.9	18.5	22.6	21.2	26.1	23.1	28.6	25.0	31.2
155	13.1	15.4	16.0	19.4	18.0	22.0	20.6	25.4	22.5	27.9	24.3	30.3
160	12.8	15.0	15.5	18.9	17.6	21.5	20.1	24.7	21.9	27.1	23.7	29.6
165	12.4	14.6	15.2	18.4	17.1	20.9	19.6	24.1	21.4	26.5	23.1	28.8
170	12.1	14.2	14.8	18.0	16.7	20.4	19.1	23.5	20.8	25.8	22.5	28.1
175	11.8	13.9	14.4	17.5	16.3	19.9	18.7	22.9	20.3	25.2	22.0	27.4
180	11.6	13.6	14.1	17.1	15.9	19.5	18.2	22.4	19.9	24.6	21.5	26.8

Table A9: Rainfall intensity (mm/hr) of study area

Duration(min)	Rainfall intensity(mm/hr)					
	T-2	T-5	T-10	T-25	T-50	T-100
5	84	102.163	115.381	132.023	144.012	155.506
10	69.93	85.250	96.281	110.168	120.172	129.763
15	60.12	73.291	82.774	94.712	103.313	111.559
20	52.80	64.370	72.698	83.184	90.737	97.980
25	47.13	57.450	64.884	74.242	80.983	87.447
30	42.59	51.921	58.639	67.096	73.189	79.030
45	33.16	40.429	45.660	52.245	56.989	61.538
50	30.91	37.685	42.562	48.700	53.123	57.363
55	28.96	35.304	39.872	45.623	49.766	53.738
60	27.25	33.217	37.514	42.925	46.823	50.560
65	25.73	31.371	35.430	40.540	44.222	47.751
70	24.39	29.727	33.574	38.416	41.905	45.249
75	23.18	28.254	31.910	36.512	39.828	43.006
85	21.10	25.720	29.048	33.238	36.256	39.150
90	20.20	24.622	27.808	31.819	34.709	37.479

95	19.37	23.618	26.674	30.522	33.293	35.950
100	18.62	22.696	25.633	29.330	31.993	34.547
105	17.92	21.846	24.672	28.231	30.795	33.252
115	16.68	20.330	22.961	26.272	28.658	30.945
120	16.12	19.651	22.194	25.395	27.701	29.912
125	15.60	19.018	21.479	24.577	26.809	28.949
130	15.12	18.426	20.811	23.812	25.974	28.048
135	14.66	17.872	20.184	23.095	25.193	27.203
140	14.23	17.351	19.596	22.422	24.458	26.410
145	13.83	16.860	19.042	21.788	23.767	25.664
150	13.45	16.398	18.520	21.191	23.115	24.960
155	13.09	15.962	18.027	20.627	22.500	24.296
160	12.75	15.549	17.561	20.093	21.918	23.667
165	12.43	15.157	17.119	19.588	21.366	23.072
170	12.13	14.786	16.699	19.108	20.843	22.507
175	11.84	14.433	16.301	18.652	20.346	21.970
180	11.56	14.098	15.922	18.218	19.873	21.459

Appendix B: Rational, SCS Peak Discharge, Manning Equation, and their Calculations

Table B1: Meteorological station

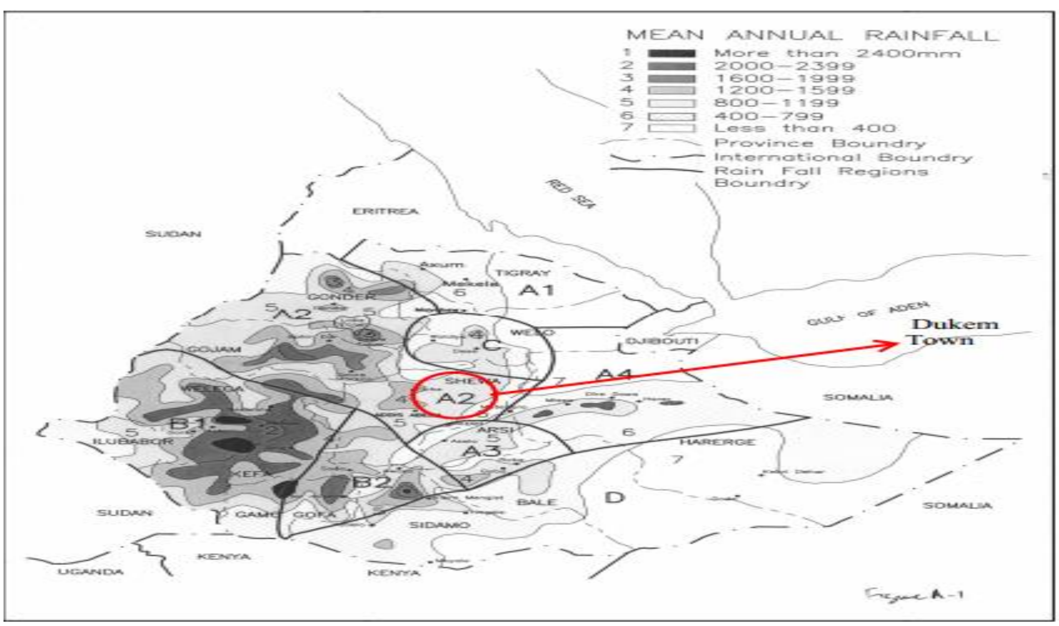

Figure 1: Mean Annual Rainfall for Ethiopia**,** *Source: ERA drainage design manuals 2013*

Table B2: Typical Hydrological soils groups for Ethiopia

	Soil Types	Hydrologic Soil Group
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	Rendzinas	D
Hh	Haplic Phaeozems	C
Hi	Luvic Phaeozems	C
I	Lithosols	D
Jc	Calcaric Fluvisols	B
Je	Eutric Fluvisols	B
Lc	Chromic Luvisols	B
Lo	Orthic Luvisols	B
Lv	Vetic Luvisols	C
Nd	Dystric Nitosols	B
Ne	Eutric Nitosols	B
Od	Dystric Histosils	D
Oe	Eutric Histosols	D
Qc	Cambric Arenosols	A
Rc	Calcaric Regosols	A
Re	Eutric Regosols	A
Th	Humic Andosols	B
Th	Mollic Andosols	B
Tv	Vitric Andosols	B

Vc	Chromic Vertisols	D
Vp	Pellic Vertisols	D
Xh	Haplic Xerosols	B
Xk	Caloic Xerosols	B
Xl	Luvic Xerosols	C
Yy	Gypsic Yermosols	B
Zg	Gleyic Solonchaks	D
Zo	Orthic Solonchaks	B

Source: Ministry of Agriculture

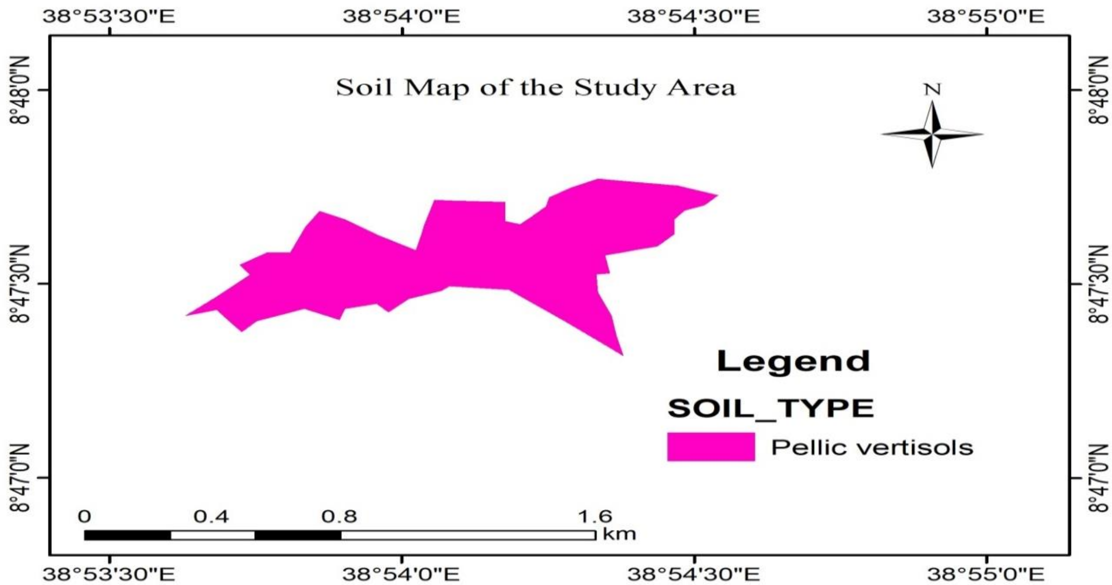

Figure 2: soil map

Table B3: Manning's n for open channels

Channel types	Manning's n	Channel types	Manning's n
Lined channels		Excavated or dredged	
Asphalt	0.013-0.017	Earth, Straight & uniform	0.02-0.03
Brick	0.012 0.018	Earth, winding, fairly uniform	0.025-0.04
Concrete	0.011-0.020	Rock	0.03-0.045
Rubble or Riprap	0.020-0.035	Unmaintained	0.05-0.14
Vegetal	0.030-0.40		

Natural channels (Lateral streams, top width at flood stage <100ft)	
Fairy regular section	0.03-0.07
Irregular section with pool	0.04-0.100

Source: ASCE (1982). Gravity Sanitary sewer Design and Construction, ASCE Manual of Practice No.60, New York, NY

Table B4: Manning's n for closed channels

Conduit material	Manning's n	Conduit material	Manning's n
Asbestos- cement pipe	0.011-0.015	Spun asphalt lined	0.011-0.015
Brick	0.013-0.017	Plastic pipe (smooth)	0.011-0.015
Cast iron pipe		Vitrified clay	
Cement-lined & seal coated	0.011-0.015	Pipes	0.011-0.015
Concrete (monolithic)		Liner plates	0.013-0.017
Smooth forms	0.012-0.014		
Rough forms	0.015-0.017	Corrugated-metal pipe (1/2-in.*2-2/3in.corrugations)	
Concrete pipe	0.011-0.015	Plain	0.022-0.026
		Paved invert	0.018-0.022

Table B5: Land-use composition for each sub-catchment in hectare and weight runoff coefficient

SC	Land use code											
	L001	L002	L003	L004	L005	L006	L007	L008	L009	L010	L011	Total area(ha)	Cw
SC1	1.489											1.489	0.600
SC2	5.908			0.744		0.261						6.912	0.612
SC3	4.027					0.231						4.258	0.595
SC4	20.269			0.567		0.742					7.658	29.237	0.600
SC5		6.442		3.572	0.228						0.416	10.658	0.795
SC6	4.650			2.241		0.794					0.158	7.842	0.633
SC7	3.794	1.981		2.083		0.713				0.024	6.519	15.114	0.649
SC8		5.471	0.460	0.629		1.318	0.360		0.503	0.498		9.240	0.729
SC9	7.088	2.298		0.658		0.506			0.123		2.023	12.696	0.646
SC10	2.575	1.790		2.193								6.557	0.718
SC11	7.066	6.052	0.108	4.881							0.750	18.858	0.721
SC12	8.135			2.825			0.962		0.424		1.219	13.565	0.622

SC13	7.885			2.265		1.446			0.209		1.205	13.010	0.610
SC14	0.984			0.171		0.288			0.294			1.737	0.547
SC15	2.152			0.430		0.589						3.171	0.602
SC16						2.090		6.576				8.665	0.348
SC17	2.861			1.642		1.058		4.067	4.272		2.848	16.749	0.459
SC18						2.611		12.915			0.704	16.230	0.345
SC19	5.188	12.993	0.325	4.268								22.774	0.775
SC20	8.932	3.286	0.344	2.737		0.198						15.497	0.685
Total												**234.2545**	

Table B6: Land use composition for each neighbor sub-catchment and weight runoff coefficient.

SC	Land-use code							
	1	2	3	4	5	6	Total Area(ha)	Cw
SC-1	2.560	7.201	0.689	7.228	23.833	27.932	69.4	0.21
SC-2	17.700	2.872	2.249	2.743	36.736		62.3	0.29
SC-3	11.893	2.916			7.690	9.704	32.2	0.26

Table B7: Rational method peak discharge

				Intensity rainfall (mm/hr)							
SC	Area(ha)	cw	TC (min)	T-2	T-5	T-50	T-100	Q (m^3/s)			
				1	1	1.2	1.25	t-2	t-5	t-50	t100
SC1	1.49	0.60	5.09	83.50	101.80	143.50	154.95	0.207	0.253	0.43	0.48
SC2	6.91	0.61	17.89	55.65	67.84	95.63	103.27	0.655	0.798	1.35	1.52
SC3	4.26	0.59	12.91	63.86	77.85	109.74	118.49	0.449	0.548	0.93	1.04
SC4	29.24	0.60	24.94	47.19	57.52	81.09	87.56	2.303	2.807	4.75	5.34
SC5	10.66	0.79	17.98	55.53	67.69	95.42	103.04	1.308	1.594	2.70	3.03
SC6	7.84	0.63	5.42	82.42	100.48	141.63	152.94	1.137	1.386	2.34	2.64
SC7	15.11	0.65	9.23	71.75	87.47	123.30	133.14	1.955	2.384	4.03	4.54
SC8	9.24	0.73	26.85	45.34	55.27	77.91	84.13	0.849	1.036	1.75	1.97
SC9	12.70	0.65	15.93	58.60	71.44	100.70	108.74	1.336	1.629	2.76	3.10
SC10	6.56	0.72	18.90	54.25	66.13	93.23	100.67	0.710	0.866	1.46	1.65
SC11	18.86	0.72	19.23	53.80	65.59	92.46	99.84	2.033	2.478	4.19	4.72
SC12	13.56	0.62	10.14	69.62	84.87	119.63	129.18	1.632	1.990	3.37	3.79
SC13	13.01	0.61	10.43	68.95	84.05	118.49	127.94	1.522	1.855	3.14	3.53

SC14	1.74	0.55	5.82	81.14	98.91	139.43	150.55	0.214	0.261	0.44	0.50
SC15	3.17	0.60	5.00	83.80	102.16	144.01	155.51	0.445	0.542	0.92	1.03
SC16	8.67	0.35	5.87	81.00	98.75	139.20	150.31	0.680	0.828	1.40	1.58
SC17	16.75	0.46	14.36	61.22	74.63	105.20	113.60	1.308	1.595	2.70	3.03
SC18	16.23	0.35	10.34	69.16	84.31	118.84	128.33	1.077	1.313	2.22	2.50
SC19	22.77	0.78	17.23	56.62	69.02	97.29	105.06	2.778	3.386	5.73	6.44
SC20	15.50	0.68	13.15	63.40	77.28	108.94	117.64	1.871	2.281	3.86	4.34

Table B8: Land use compositions of the study area and the upper catchment

Land-use type	Land-use code	Area(ha)	Percentage (%) area	Runoff coefficient.
Residential	L001	93.0134	38.99205485	0.6
Commercial	L002	40.384064	16.65076982	0.85
Asphalt Road	L003	1.236492	0.41955818	0.9
Cobblestone Road	L004	32.101444	13.53247535	0.75
Playground	L005	0.228094	0.092855469	0.3
Gravel Road	L006	12.898388	5.291552674	0.5
Green Area	L007	1.321846	0.452047051	0.4
Developing(Suburban)	L008	23.580692	9.599534481	0.3
Open Space	L009	5.674293	2.30996492	0.3
Administration Office	L010	0.521843	0.212438629	0.5
Public Institution	L011	23.351612	12.82435094	0.6
		234.31216	100	

Land-use type	Area(ha)	Percentage(%) area	Runoff coefficient
Residential(single)	32.15287	19.60541	0.4
Railway	12.98961	7.920495	0.3
Drive and walk	2.938394	1.791704	0.8
Open space	9.970611	6.079641	0.3
Cemetery	68.25941	41.62159	0.2
Forest	37.63671	22.94921	0.15
	163.9476		

Table B9: Design Storm Frequency (yrs.) by Geometric Design Criteria

Structure Type	Geometric Design Standard			
	DS1/DS2	DS3/DS4	DS5/DS6/DS7	DS8/DS9/DS10
Gutters and Inlets	10 or 5	2	2	
Side Ditches	10	10	5	5
Ford/low water bridge				5
Culvert Pipe(span<2m)	25	10	5	5
Culvert (2m<span<6m)	50	25	10	10
Short span bridges				
(6m<span<15m)	50	50	25	25
Medium span				
bridges (15m<span<50m)	100	50	50	50
Long span bridges (span>50m)	100	100	100	100
Check/review flood	200	100	100	100

Source: (ERA DDM, 2011)

Table B10: land-use area in each upper catchment classification

C	Land-use area in the catchment (ha)						sum
	residential	railway	drive &walk	open space	Cemetery	forest	
	CN= 84	CN= 86	CN= 86	CN= 84	CN= 84	CN= 63	
C-1	2.5602	7.201	0.689	7.23	23.83	27.93	69.4
C-2	17.6998	2.872	2.25	2.74	36.74	0	62.3

Table B11: Weight curve number

C	Land-use (ha)						sum	weigh CN
	residential	railway	drive& walk	open space	cemetery	Forest		
	CN=84	CN=86	CN=86	CN=84	CN=84	CN=63		
C-1	215.06	619.31	59.29	607.13	2001.9	1759.73	5262.5	76
C-2	1486.8	247.00	193.41	230.40	3085.8	0.00	5243.4	84

Table B12 : Regression Coefficient

C	Coefficient	T-2	T-5	T-10	T-25	T-50	T-100
C1	C0	2.465	2.485	2.500	2.461	2.520	2.527
	C1	-0.623	-0.621	-0.619	-0.619	-0.618	-0.617

	C2	-0.117	-0.129	-0.136	-0.143	-0.147	-0.147
C2	C0	2.491	2.5231	2.539	2.543	2.553	2.553
	C1	-0.618	-0.6226	-0.616	-0.6154	-0.615	-0.615
	C2	-0.145	-0.15228	-0.1569	-0.1616	-0.164	-0.164

Appendix C: **SWMM Input Parameters**

Table C1: Sub-catchment properties

SC	Area (m^2)	Area (ha)	Length(m)	width(m)	Slope (%)
SC1	14889.5	1.49	154	96.69	1.2987
SC2	69123.4	6.91	626	110.42	2.5559
SC3	42578.3	4.26	542	78.56	1.6605
SC4	292370.5	29.24	1011	289.19	0.9329
SC5	106578.2	10.66	692	154.01	1.4451
SC6	78420.7	7.84	231	339.48	1.6253
SC7	151143.1	15.11	611	247.37	2.0665
SC8	92400.3	9.24	560	165.00	4.0261
SC9	126955.8	12.70	522	243.21	2.2989
SC10	65570.8	6.56	679	96.57	1.9146
SC11	188575.2	18.86	688	274.09	2.2818
SC12	135647.3	13.56	515	263.39	0.9709
SC13	130103.6	13.01	482	269.92	0.6224
SC14	17365.2	1.74	265	65.53	1.1321
SC15	31709.1	3.17	106	299.14	0.9434
SC16	86654.8	8.67	247	350.83	1.6194
SC17	167490.0	16.75	608	275.48	1.6447
SC18	162297.7	16.23	510	318.23	0.9804
SC19	227738.2	22.77	610	373.34	1.5
SC20	154970.6	15.50	500	309.94	1.6

Table C2: Conduit properties

Link	Inlet node	Outlet node	Length (m)	Width (m)	Depth (m)
C1	JC1	JC25	1011	0.8	0.86
C2	JC25	JC31	744	0.8	0.86
C3	JC29	JC31	291	0.75	1.25
C4	JC5	JC6	635	0.75	1.25
C5	JC7	JC8	385	0.73	1
C6	JC9	JC10	275	0.75	1
C7	JC8	JC11	110	0.85	1
C8	JC6	JC11	580	0.65	1
C9	JC11	JC12	456	0.72	0.87
C10	JC13	JC14	86	0.65	0.87
C11	JC14	JC12	564	0.65	0.9
C12	JC10	JC15	516	0.75	1
C13	JC4	JC15	419	0.82	0.86
C14	JC15	JC16	188	0.82	0.86
C15	JC16	JC18	123	0.75	0.85
C16	JC17	JC20	669	0.72	0.85
C17	JC20	JC19	370	0.65	0.9
C19	JC21	JC22	416	0.82	0.86
C20	JC32	JC22	98	0.82	0.86
C21	JC12	JC19	229	0.72	0.85
C22	JC24	JC25	229	0.65	0.9
C23	JC26	JC27	556	0.67	1.25
C24	JC28	JC27	81	0.65	1.25
C25	JC30	JC29	120	0.65	1.25
C28	JC27	JC29	151	0.65	1
C29	JC22	JC23	95	0.65	0.75
C30	JC35	JC33	53	0.55	0.67
C31	JC33	JC36	258	0.55	0.67
C32	JC37	JC38	134	0.6	0.75
C33	JC39	JC38	279	0.55	0.75
C34	JC38	JC40	88	0.55	0.75

C35	JC40	JC36	51	0.55	0.67
C36	JC36	JC41	233	0.65	0.95
C37	JC45	JC40	271	0.55	0.75
C38	JC18	JC43	597	0.65	0.9
C39	JC43	JC44	522	0.65	1
C40	JC44	JC2	241	0.67	0.85
C41	JC19	JC46	599	0.9	1
C42	JC46	JC2	654	0.85	1
C43	JC47	JC46	292	0.65	0.95
C44	JC48	JC41	275	0.75	1
C45	JC43	JC3	230	0.75	1
C46	JC41	JC3	511	0.75	1
C47	JC42	JC50	232	0.65	0.9
C48	JC50	JC49	343	0.65	0.9

Table C3: Outfall properties

Outfall	coordinate		Elevation (m)	Depth (m)	Invert elevation (m)	Located around
	X (m)	Y (m)				
Outfall-1	490993	972215	1937.75	1.25	1935.5	Tedecha Boundary
Outfall-3	491033	970987	1924	1	1923.75	Gebriel Church
Outfall-2	490872	970782	1924	0.9	1923.1	Parot Hotel

Table C4: Junction properties

JC	coordinate		elevation(m)	max. depth(m)	Invert elevation(m)
	X (m)	Y (m)			
JC1	489251	972794	1969.5	0.95	1968.5
JC4	490103	972366	1958.1	1.1	1957
JC6	489076	972133	1957.0	1.25	1955.75
JC7	489381	972475	1962.0	1	1961
JC8	489677	972210	1957.0	1	1956
JC9	487862	972400	1964.0	1	1963
JC10	489677	972210	1957.0	1	1956
JC11	489658	972099	1955.2	1	1954.15
JC12	489815	971636	1945.1	0.95	1944.15

JC13	489342	972071	1956.0	0.87	1955.13
JC14	489340	971976	1956.0	0.9	1955.1
JC15	490142	971993	1945.0	1	1944
JC16	490317	971914	1942.2	0.95	1941.2
JC17	489658	972099	1955.0	0.85	1954.15
JC18	490229	971793	1942.2	1.1	1941.1
JC19	489998	971492	1941.0	0.9	1940.1
JC20	490229	971793	1942.2	1.1	1941.1
JC21	490665	972299	1944.0	0.86	1943.14
JC22	490399	971990	1938.0	0.86	1937.14
JC24	490409	972485	1960.2	1.1	1959.1
JC25	490172	972367	1958.2	1.1	1957.1
JC26	490381	972349	1951.0	1.25	1949.75
JC27	490746	972456	1950.0	1.3	1948.7
JC28	490696	972535	1953.0	1.3	1951.7
JC29	490867	972487	1949.0	1.25	1947.75
JC30	490842	972612	1955.0	1.25	1953.7
JC33	490470	971913	1937.0	0.75	1936.25
JC34	490470	971913	1937.0	0.75	1936.25
JC35	490514	971962	1938.3	0.95	1937.33
JC36	490674	971743	1934.3	0.95	1933.33
JC37	490514	971962	1938.3	0.95	1937.33
JC38	490635	971859	1934.0	0.75	1933.25
JC39	490812	972076	1935.0	0.75	1934.25
JC40	490712	971787	1935.0	0.75	1934.25
JC41	490848	971575	1930.1	1	1929.14
JC42	490897	972006	1936.3	1	1935.25
JC43	490663	971352	1933.1	1.1	1932
JC44	491033	970967	1926.0	1	1925
JC46	490481	971102	1932.1	1.1	1931
JC47	490663	971352	1933.1	1.1	1932
JC23	491033	970967	1926.0	1	1925
JC50	491072	971841	1929.0	0.9	1928.14

JC49	491033	970967	1925.0	0.9	1924.1
JC32	490317	971914	1942.0	0.9	1941.1
JC45	490897	972006	1936.0	0.75	1935.25
JC48	490663	971352	1933.1	1.1	1932
JC31	490665	972299	1936.5	1.25	1935.25
JC3	491254	971251	1925.1	1.12	1924
JC2	490879	970781	1925.0	1	1924
JC5	489548	972637	1966	1.25	1964.5

Table C5: Some key parameters used for sensitivity analysis in this study and their allowed range of change (Li et, al., 2014)

Parameter	Description	Allowed range	selected
N-Imperv	Manning's roughness coefficient for impervious areas	0.005-0.05	0.0056
N-Perv	Manning's roughness coefficient for pervious areas	0.05-0.5	0.1
Dstore-Imper	Depth of surface storage in impervious areas (mm)	1.3-2	1.5
Dstore-Perv	Depth of surface storage in pervious areas (mm)	2-7	2.5