HYDROLOGICAL AND HYDRAULIC PERFORMANCE EVALUATION OF STORMWATER DRAINAGE SYSTEMS, OF AWASHO-ALELU KEBELE (01 AND 10) SHASHEMENE TOWN, ETHIOPIA.

M.Sc. THESIS

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HAWASA UNIVERSITY, HAWASA, ETHIOPIA

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This thesis entitled "Hydrological and Hydraulic Performance Evaluation of Stormwater Drainage Systems, of Awasho-Alelu Kebele (01 and 10) Shashemene Town, Ethiopia." submitted in partial fulfillment of the requirements for the degree of Master's of Science with specialization in Water Resource Engineering and Management, the Graduate Program of the Faculty of Bio-systems and Water Resource Engineering, Department of Water Resource and Irrigation Engineering, and has been carried out by Usman Tusa Id.No.PGWREMR/0017/11. Therefore, we recommend that the student has fulfilled the requirements and hence hereby can submit the thesis to the department.

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Declaration

The author declares this thesis is not submitted to any other institution for the award of any academic degree or diploma. The source of all materials used in this work has been acknowledged. This thesis has been submitted in partial fulfillment of the requirements for the MSc. degree in Water Resource Engineering and Management at Hawassa University and deposited at the University Library to be made available to borrowers under the rules of the Library.

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Date: ___________________________

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Lists of Abbreviations and Acronyms

Adm.Office	Administration office
AGS	Average Group Station
As	Area of the cross-section canal
Av	Average
Av.S	Average Slope
BMPS	Best Management Practices
C	Conduit
CAD	Computer-Aided Design
CGS	Cumulative Group Station
CN	Curve Number
Con	Concrete
Cum	Cumulative
CW	Weighted Runoff Coefficient
DDM	Drainage Design Manual
DEFRA	Department for Environment Food and Rural Affairs
DEM	Digital Elevation Model
DL	Drain Line
EPA	Environmental Protection Agency
ERA	Ethiopian Road Authority
ERA	Ethiopian Road Authority
ES	Essa River
FHWA	Federal Highway Administration
FUPCOB	Federal Urban Planning Coordinating Bureau

G	Gumbel distribution
GCS	Geographic Coordinate System
GIS	Geographic Information System
GOF	Goodness of Fit
GPS	Geographic Positioning System
GTZ	German Technical Cooperation
HA	Hectare
IDF	Intensity-Duration-Frequency
IDFC	Intensity Duration Frequency Curve
J	Junction
Km^2	Square Kilometer
LN	Log-Normal distribution
LPIII	Log-Pearson III distribution
LU	Land-Use
M^3/sec	Cubic Meter
Mas.	Masonry
MOWUD	Ministry of Works and Urban Development
MOR	Malka Oda River
N	Normal distribution
n	Manning's Roughness Coefficient
NRCS	Natural Resources Conservation Service
P.Institution	Public Institution
Rec	Rectangular
RM	Rational Method

R24	24 hour rainfall depth
S	Side Slope
Sc	Sub-Catchment
SS	Side Slope
Stdev	Standard deviation
SUDS	Sustainable Urban Drainage System
SWMM	Storm Water Management Model
Trap.	Trapezoidal
TC	Time of Concentration
TCI	Time of Concentration Inlet
Tem. Max	Temperature maximum
Tem. min	Temperature minimum
U.S	United State
US SCS	United States Soil Conservation Service
USWD	Urban Storm Water Drainage
VDOT	Virginia Department of Transport

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Abstract

Shashemene town is one of the fastest-growing towns to urbanization and increases the amount of runoff generated from the urban catchment. This study intended to hydrological and hydraulic performance evaluation of stormwater drainage systems, of Awasho-Alelu kebele (01 and 10). Due to urbanization, lack of proper design and management, the transport infrastructure and drainage systems are caused by flooding. Data used for this study were meteorological (rainfall) data, drainage systems dimensions, digital elevation model (DEM). Determining the peak discharge by the rational method using land-use, rainfall intensity and the catchment area of internal all sub-catchments and also the upstream catchment inflow by SCS (CN). These results show, that 78.9% of the drainage systems are not adequate to carry the design flood based on intensity with a return period of 10 years. The other problems lack well-connected and sufficient drainage lines, solid wastes were directly disposed into the storm drainage system and insufficient slope provides which results in decreasing the efficiency of the system. Most drainage systems perform severely degraded are at Farma academy school 54.3%, at new bus station 44.181%, at Malka Oda Hospital 61.6%, at Alelu market 46.5% and Alelu primary School 56.2% were severely degraded.

Keywords Shashemene town, Storm drainage systems, SWMM, Runoff.

1. INTRODUCTION

1.1. Background

Stormwater is rainwater that runoff from impervious or saturated surfaces in the urban environment: roofs, roads, pavements, and green spaces. Urban expansion replaces large areas of vegetated ground replaced by impervious surfaces such as roofing and paving which leads the volume of stormwater runoff to increase. Drainage systems have been needed in developed urban areas because of the interaction between human activity and the natural water cycle (Butler and Devies, 2011).

Flooding in urbanized areas has become a very significant issue around the world. Cities are growing fast and large amounts of impervious surfaces replace the natural landscape because of urban development. Impervious surfaces can affect local streams and flooding characteristics (Barreto Cordero, 2012).

Stormwater drain networks in cities are usually designed to effectively collect and convey excess surface runoff to prevent urban flooding (Gouri and Srinivas, 2015). Urban stormwater drainage facilities are part of the urban infrastructure elements and the design of these facilities requires due attention. In the Ethiopian context, where watersheds of many urban centers receive an important amount 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 correct drainage facilities becomes essential (AASHTO, 1991 and MOWUD, 2008).

Urban stormwater affects the service life of urban infrastructures. The rainfall intensity and features of the catchment area are the major factors for designing urban stormwater drainage facilities. These facilities have a paramount benefit safely dispose of the generated floods to ultimate receiving systems. Stormwater management is concerned with the collection, conveyance, storage, treatment, and disposal of storm runoff in a way that minimizes accelerated channel erosion increased flood damage. Proper drainage is an important factor that should be given due consideration in the design of a highway since inadequate drainage facilities can lead to early deterioration of the highway and the development of adverse safety conditions (Mukherjee, 2014).

Urban drainage is a vital city infrastructure to Hydrological and hydrodynamic modeling plays a key role in the hydrologic, structural, and environmental assessment. Sustainable approaches, oriented to the control of runoff volumes from the beginning of the rainfall are preferable to methodologies based on conveyance. These sustainable approaches are also oriented to keep the environmental, social, and economic values in balance (Barreto, 2012). The EPA Storm Water Management Model (SWMM) is a dynamic rainfall-runoff simulation model that computes runoff quantity and quality from primarily urban areas. The runoff component of SWMM operates on a collection of sub-catchment areas that receive precipitation and generate runoff and pollutant loads (Rossman et al., 2015).

From these studies, I can conclude, that the performance evaluation hydrological and hydraulic of stormwater drainage systems estimate peak discharge and compare with existing hydraulic capacity. To identify the adequacy and performance of drainage systems in each sub-catchments of the study area.

1.2. Statement of the Problem

Shashemene Town, facing a drainage problem due to changes in land use and land management as a result of the rapid growth of urbanization now. As a result, a change in the runoff characteristics within the town increased runoff and greater susceptibility to flooding.

Dumping solid waste materials into the drainage system was the challenge of the stormwater drainage system around: Tesho and DH Green hotel, Rani Cafes, Malka oda hospital, etc. It consists of a large and small substance that becomes stick together in each deposited time and no passed the stormwater through drainage systems. The manufactured materials such as bottles, cans, plastic and paper wrappings, newspapers, shopping bags, cigarette packets, and remains of chat. Dumping such solid wastes into drains, the drainage systems have clogged and caused flooding over streets mud and solid material and drainage systems failures.

Most people in the town do not worry about drainage systems to protect property rather than how to depose of the waste dump and discharging material from domestic, compounds and commercial areas regardless of the effects that could cause to the drain structure. The reason for flooding of this area is blockage of the crossing between the asphalt and the cobblestone road and other infrastructures are without properly cleaning, managing solid waste, and maintenance. Due to improper maintenance, not a well operational function of the system and to extend its working life.

System defects are often the catalyst for blockage formation. These can range from poor design such as severe changes of sewer direction through to poor construction practice. Not clean periodically the solid waste and sediment which, make block the stormwater flow freely in the drain structure. They have implied that there is no proper sewage system and trash bin to collect wastes extracted from each household. As a result, blockage of drainage systems by solid waste material.

The drainage systems through the town were the inadequate provision of conveying the runoff to the outlet points, lack of well-connected drainage networks and unavailability of drainage systems of the existing drainage facilities around: near New B

us Station (kebele 01), Malka Oda Hospital (kebele 01), Shashemene Stadium and Alelu Market (kebele 010).

1.3. Objectives of the Study

1.3.1. General Objective

The general objective of the study was Hydrological and Hydraulic Performance Evaluation of Stormwater Drainage Systems, of Awasho-Alelu Kebele (01 and 10) Shashemene Town, Ethiopia.

1.3.2. Specific Objectives

The specific objectives of this research include:

1. To evaluate the hydraulic capacity of the existing drainage systems.
2. To estimate the water level at links and junctions in the drainage networks.

1.4. Research Questions

1. Is the existing drainage systems sufficient enough to pass the design flood of a 10 year return period without causing any town facility?
2. Is the water level at junctions and links in the drainage networks carrying design flood?

1.5. Significance of the Study

Overall, Shashemane town is the part of which facing drainage problems, so further investigation contributed to the solution for stormwater drainage problems and sustainable drainage systems for future use in the area. Also, the study contributes the following significances to Ethiopia Road Authority (ERA), Shashemane town administration, academicians, researchers, and other stakeholders who will conduct similar researches on other road drainage structures. Preserving the road network and save the asset value, identifying the existing situation to know the real problems of the drainage Systems, finding possible solutions based on the recommendations to avoid the problems.

1.6. Scope and Limitation of this Study

This study extends to hydrological and hydraulic performance evaluation of the existing stormwater drainage systems, to identify adequacy and performances by field observation and using Google Earth and GIS to classify the current land-use, drain line, sub-catchments e.t.c. This study was limited to Awasho-Alelu kebele (01 and 10) of Shashemene Town, for the absence of land-use data from the municipality. The classifying whole sub-catchment of the town difficult and tedious.

2. LITERATURE REVIEW

2.1. Historical Perspectives of Urban Storm Drainage Systems

Historically, artificial drainage systems have developed as soon as humans attempted to control their environment. Archaeological evidence reveals that drainage has provided to the buildings of many ancient civilizations such as the Mesopotamians, the Minoans (Crete) and the Greeks (Athens). The Romans are well known for their public health engineering feats, particularly the impressive aqueducts bringing water into the city; less remarkable, but equally vital, were the artificial drains they built, of which the most well-known is the cloaca maxima, built to drain the Roman Forum still in use today. The English word sewer derived from an old French word, sewer, meaning 'to drain off, related to the Latin ex- (out) and aqua (water). The Oxford English Dictionary gives the earliest meaning as 'an artificial watercourse for draining marshy land and carrying off surface water into a river or the sea (Butler and Devies, 2011).

Urban drainage in the early parts of the twentieth century has firmly established as a vital public works system. Engineers continued to improve design concepts and methods. During the second half of the twentieth century, regulatory elements have spread 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, capabilities, societal values, religious beliefs and other factors that have influenced the local view of urban drainage (Burian and Edwards, 2002).

2.2. Runoff Process in Urban Area

The development of urbanization has a direct impact on existing drainage infrastructure and the surrounding environment (Butler and Maksimovic, 2001). They increase the area of paved surfaces, thus reducing infiltration while causing surface runoff to exhibit higher peak flows, larger volumes, and shorter times to peak and accelerated transport of pollutants and sediment from urban areas. This results in pollution of the receiving watercourses and increased flood risk within the development (Markopoulos,1999 and Natsis, 2008).

Runoff generally develops over days, when there is too much rainwater to fit in the rivers and water spreads over the land. However, they can happen very quickly when lots of heavy rainfalls over a short period. These flash floods occur with little or no warning and cause the biggest loss of human life than any other type of flooding. (Vent cow, 1988) According to ERA Drainage

Design Manual, 2013, the following are some of the most widely used flow estimations of runoff methods:

2.2.1. Rational Method

Estimation of peak discharge and runoff values for use in designing certain hydraulic structures like urban storm drainage systems and other hydraulic structures is an important and challenging aspect of engineering hydrology (Viessman and Lewis, 2003). Various methods are available to estimate peak discharges and runoff volumes from urban watersheds (Chow, et al., 1988 and US. EPA, 1983). The rational method is commonly used by hydraulic and drainage engineers to estimate design discharges, which are used to size a variety of drainage structures for small urban (developed) and rural (undeveloped) watersheds (Viessman and Lewis, 2003). The rational method was developed in the United States by Emil Kuichling (1889) and introduced to Great Britain by Lloyd-Davies (1906).

The rational method provides an estimation of peak runoff rates of small urban and rural watersheds of less than 50 hectares (0.5 square km) and in which natural and artificial storage is small. It is best suited to the design of urban storm drain systems, small side ditches and median ditches, and driveway pipes. It is used with caution if the time of concentration exceeds 30minutes. Characteristics of the rational method that generally limit its use to 50 hectares. The rate of runoff resulting from any rainfall intensity is a maximum when the rainfall intensity lasts as long as or longer than the time of concentration. That is, the entire catchment area does not contribute to the peak discharge until the time of concentration has elapsed; the frequency of peak discharges is the same as that of the rainfall intensity for the given time of concentration and the fraction of rainfall that becomes runoff is independent of rainfall intensity. Rainfall intensity, runoff coefficient, and area of the catchment are necessary input for this method of flow estimation (ERA, 2013).

2.2.2. Soil Conservation Service (Curve Number)

Runoff is one of the most important hydrologic variables used in water resource applications. Apart from these rainfall characteristics, there are several catchment-specific factors, which have a direct effect on the occurrence and volume of runoff. This includes soil type, vegetation cover, slope, and catchment type. SCS-CN provides an empirical relationship for estimating initial abstraction and runoff as a function of soil type and land use. Curve Number (CN) is an index developed by the Natural Resource Conservation Service (NRCS), to represent the potential for stormwater runoff within a drainage area. The CN for a drainage basin is estimated using a combination of land use, soil, and antecedent soil moisture condition (AMC). There are four hydrologic soil groups: A, B, C, and D. Group A has high infiltration rates and group D has low infiltration rates. The Soil Conservation Service Curve Number (SCS-CN) method is widely used for predicting direct runoff volume for a given rainfall event (NRCS, 2002).

The Natural Resources Conservation Service formerly Soil Conservation Service developed the runoff curve number as a means of estimating the amount of rainfall appearing as runoff. The procedure allows the designer to estimate the effect of urbanization, channel storage, flood control storage, and multiple tributaries. The unit hydrograph used by the SCS method is based upon an analysis of a large number of natural unit hydrographs from a broad cross-section of geographic locations and hydrologic regions in the USA. However, the SCS Curve Number method applies to small catchments (maximum area 6,500 ha) with a time of concentration for any sub-area of 0.1 – 10 hours (NRCS, 2002).

The SCS method is used only when the CN is greater than 50, area above 50 hectors and the TC is greater than 0.1 hours and less than 10 hours. The computed value of Ia/P should be between 0.1 and 0.5 (ERA, 2013).

2.3. Impact Of Urbanization On Storm runoff

Urbanization increases the risk of flooding due to increased peak discharge and volume and decreased time to a peak that restricted the development of cities (Nirupama and Simonovic, 2007; Saghafian et al., 2008). In highly urbanized areas, most of the rainfall becomes surface runoff, and deep infiltration is only a small fraction of the natural situation (Chester et al., 1998).

The increases the danger of sudden flooding. It also has strong implications for water quality. The rapid runoff of stormwater is likely to cause pollutants and sediments to be washed off the surface or scoured. In an artificial environment, there are likely to be more pollutants on the catchment surface and in the air than there would be in a natural environment. Also, drainage systems in which there is the mixing of wastewater and stormwater may allow pollutants from the wastewater to enter the river (Butler and Devies, 2011).

2.4. Rainfall intensity and urban drainage systems

The frequency of rainfall of various intensities and duration can be used in the hydrologic design of structures that control storm runoff and floods, such as storm sewers, high way culverts. Rainfall frequency analysis typically provides rainfall accumulation values at a point for a specified exceedance probability and various durations. Rainfall frequency analysis has been used extensively for the design of engineering works that control storm runoff. These include municipal storm sewer systems, highway and railway culverts, and agricultural drainage systems. Precipitation frequency analysis also plays an important role in a diverse range of nonstructural problems involving natural hazards associated with extreme rainfall events (Maidment, 1993)

2.4.1. Intensity Duration Frequency (IDF) Curve Development

Many previous studies have been done on rainfall analysis in various regions of the world. The IDF relationship is a mathematical relationship between the rainfall intensity (I), the duration (t), and the return period (T) frequency from exceedance frequency (Koutsoyiannis, et al., 1998)

The existing IDF curves are updated utilizing newly developed Regional frequency analysis techniques. IDF relationships are based on regions and should be done separately for different regions (Al-Khalaf, 1997). Quantification of rainfall was generally done using isopluvial maps and intensity-duration-frequency (IDF) curves (Chow et al., 1988). The regional properties of IDF relationships have been studied in several countries, and in general, maps have been constructed to provide the rainfall intensities or depths for various return periods and durations (IHP-VII, 2008).

The intensity duration frequency (IDF) curve is necessary to calculate the intensity of rainfall used for the design. Depending on the location of the Shashemene Town area concerning the station where the IDF will produce. To predict the rainfall depth and intensity for data 1987 to 2017 spread in and around Shashemene town by using the various frequency distributions like Normal, Log-normal, Log-Pearson type III, and Gumbel distribution method (Subramanya, 1994)**.**

- Normal Distribution
- Gumbel Distribution
- Log- Pearson Type III Distribution
- Lognormal distribution

Finally, select the best distribution using Easy-Fit 5.6, Hydrognomon software and correlation (R^2). The best distribution system gives us the estimated results for short-duration IDF curves developed.

2.4.2. Urban Stormwater Drainage System

Storm drainage facilities consist of curbs, gutters, inlets, storm drains, ditches, and culverts. Different types of structures are employed in the drainage systems are open channels whether artificial or natural convey the flows of water; surface and sub-surface drainage systems; culverts and canal convey flows under road cross-section; energy dissipaters used to control the velocities of flows, canal outlets. A complete drainage system design includes consideration of both major and minor drainage systems. The minor system sometimes referred to as the conventional system, consists of the components that historically considered as part of the "storm drainage system. These components include curbs, gutters, ditches, inlets, access holes, pipes, and other conduits, open channels, detention basins, and water quality control facilities (Alderson, 2006). A drainage system will include all the components needed to ensure that the substructure is properly drained, and may be formed of components such as open ditches, closed ditches, the drainage through stormwater drainage systems, Channels, and culverts (AASHTO, 1991).

The flow of water in a conduit may be either open channel flow or pipe flow. An open channel is a conduit for a flow that has a free surface, i.e. a boundary exposed to the atmosphere. The two kinds of flows, i.e. open channel and pipe flow, are similar in many ways but differ in one important aspect. Open channel flow must have a free surface, whereas pipe flow has none since the water must fill the whole conduit. A free surface is subject to atmospheric pressure. Pipe flow being confined in closed conduit exerts no direct atmospheric pressure but hydraulic pressure. In the case of the flowing fluid in an open channel, gravity effects usually cause the motion, and the pressure distribution within the fluid is generally hydrostatic (Chow, 1988 and Froehlich et al., 1988).

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 assumed 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. However, since the usual hydrologic methods employed in storm drain design are based on computed peak discharges at the beginning of each run, it is a conservative practice to design using the steady uniform flow assumption (Reese, 2003).

2.4.2.1. Hydraulic Capacity

The hydraulic capacity of a storm drains is computed by size, shape, slope, and friction resistance. Several flow friction formulas 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) Manning's Formula has been used to compute the average flow velocity (V) in any urban drainage system with uniform flow. Manning's formula can readily solve for a given canal when the known or assumed depth of flow has assumed of flow is used. The introduction of drainage systems to convey the streamflow exceed allowable velocity drainage systems can cause erosion, sediment deposes and scouring downstream of the structure. The flow Velocities less than 0.5m/s should be avoided because siltation will take place and ultimately reduce the channel cross-section. The allowable maximum velocity should not be more than 4m/s where otherwise; there will be the possibility of erosion of particles from bed and bank of channels. (ERA, 2002).

2.5. Storm Water Management Model (SWMM)

The Environmental Protection Agency, Storm Water Management Model (EPA SWMM) is a dynamic rainfall-runoff simulation model used for a single event or long-term (continuous) simulation of runoff quantity and quality from primarily urban areas. The runoff component of SWMM operates on a collection of sub-catchment areas that receive precipitation and generate runoff and pollutant loads. The routing portion of SWMM transports this runoff through a system of pipes, canals, storage/treatment devices, pumps, and regulators. SWMM tracks the quantity and quality of runoff generated within each sub-catchment, and the flow rate, flow depth, and quality of water in each pipe and canal during a simulation period comprised of multiple time steps (Jorge, 2009 and Rossman, 2015).

SWMM had been first developed in 1971 in Washington DC by Metcalf & Eddy and has undergone several major upgrades by Huber and Dickinson. It continues to be widely used throughout the world for planning, analysis, and design related to stormwater runoff, combined sewers, sanitary sewers, and other drainage systems in urban areas, with many applications in non-urban areas as well. The current edition, Version 5, is a complete re-write of the previous release. Running under Windows, SWMM 5 provides an integrated environment for editing study area input data, running hydrologic, hydraulic, and water quality simulations, and viewing the results in a variety of formats. These include color-coded drainage area and conveyance system maps. Time-series graphs and tables, profile plots, and statistical frequency analyses. SWMM is a physically-based, discrete-time simulation model. It employs principles of conservation of mass, energy, and momentum wherever appropriate (Jorge, 2009 and Rossman, 2015).

Modeling Capabilities are accounts for various hydrologic processes that produce runoff from the urban area are;

- Time-varying rainfall
- Rainfall interception from depression storage
- Infiltration of rainfall into unsaturated soil layers
- Percolation of infiltrated water into groundwater layers
- Nonlinear reservoir routing of overland flow (Jorge, 2009 and Rossman, 2015).

2.5.1. Flow Routing

SWMM also contains a flexible set of hydraulic modeling capabilities used to routing are steady flow, kinematic wave, and dynamic wave external inflows through the drainage system network of pipes, canals, storage/treatment units, and diversion structures. SWMM uses the manning equation to express the relationship between flow rates (Q), Cross-sectional area (A), hydraulic radius (R), and slope (S) in the canals (conduits). Flow routing within a conduit link in SWMM is governed by the conservation of mass and momentum equations for gradually varied, unsteady flow (i.e., the Saint Venant flow equations) (Rossman, 2015 and Jorge, 2009).

- **Dynamic Wave Routing**

Dynamic Wave routing solves the complete one-dimensional Saint Venant flow equations and therefore produces the most theoretically accurate results. These equations consist of the continuity and momentum equations for conduits 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 flows can exceed the full-flow Manning equation 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 atop the node and re-enter the drainage system. Dynamic wave routing can account for canal storage, backwater, entrance/exit losses, flow reversal, and pressurized flow. Because it couples the solution for both water levels at nodes and flows in conduits it can be applied to any general network layout, even those containing multiple downstream diversions and loops. It is the method of choice for systems subjected to significant backwater effects due to downstream flow restrictions and with flow regulation via weirs and orifices (Jorge, 2009; Rossman, 2015).

2.5.2. Infiltration Rate.

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: Also, infiltration models are; Horton model, modified Horton model, green amp model, modified green ampt model, and curve number model (Rossman, 2015 and Jorge, 2009).

- **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 (Jorge, 2009 and Rossman, 2015).

3. MATERIAL AND METHODS

3.1. Location and Description of the Study Area

Shashemene town is located in the southern part of Ethiopia at a distance of about 250 km from the capital Addis Ababa in West Arsi Zone of Oromia Regional State. This town is located between 7° 10' 30” to 7º 13′ 30" N and 38° 34' 30" to 38° 37' 30" E in the main Ethiopian Rift Valley and the total area covered $22km^2$. The study area for this thesis covers 438 ha for this research has seen (Figure 3.1).

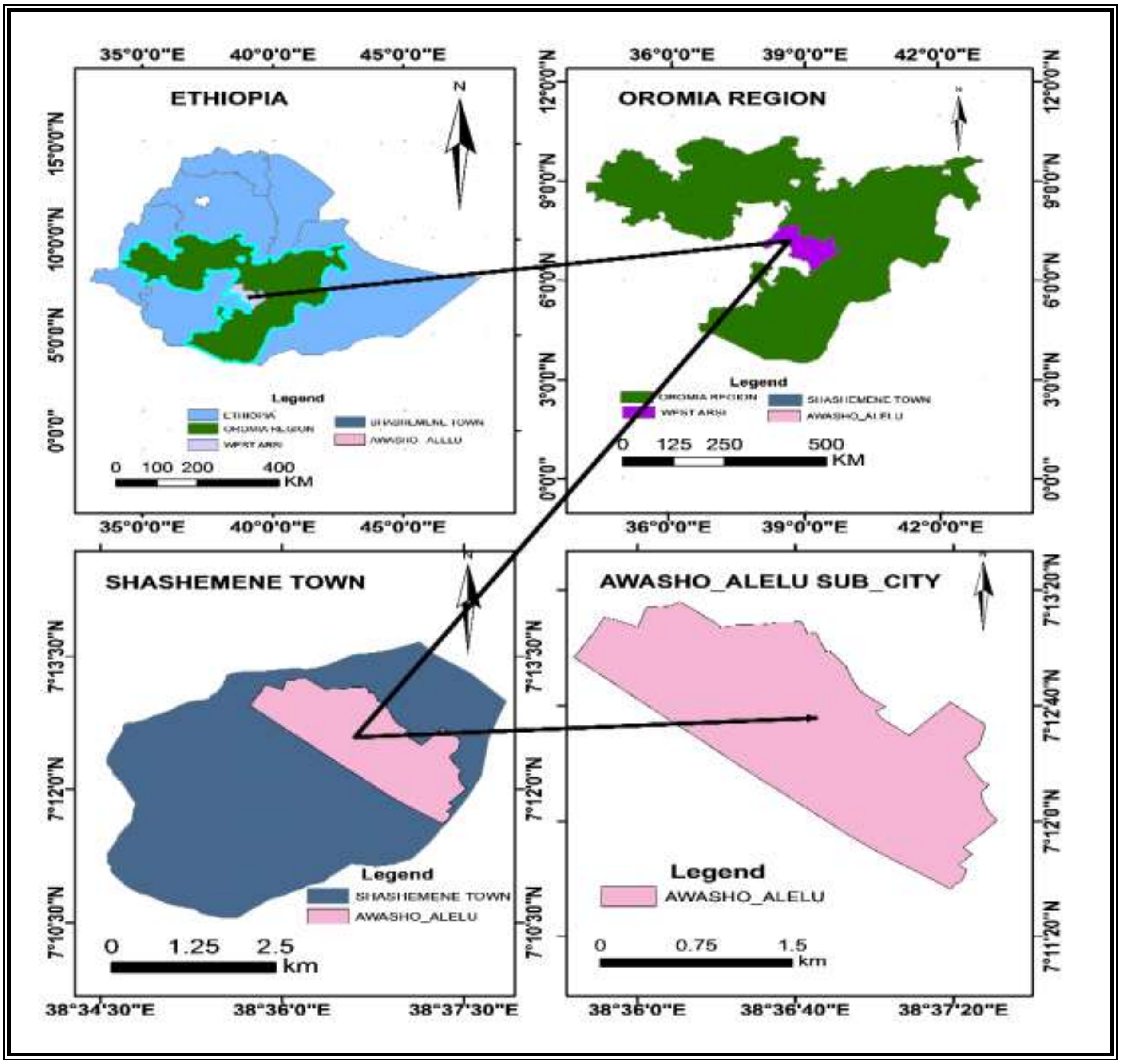


Figure 3.1:Lacation of study Area

3.1.1. Topography

Shashemene Town is characterized as flat with an average slope of 3 increasing particularly along with river courses and decreases from the southeast towards the northwest. The elevation of the town is ranging from 1840 to 2025 m.a.s.l.

3.1.2. Climate

3.1.2.1 Rainfall

The Climate of Shashemene town and its surroundings have moderate climatic conditions and experience two distinct wet and dry seasons. The rainfall in the region has a weak bi-modal pattern with the first peak in April-May and the second and main peak occurring in July – September. The annual rainfall range from (800-1199) mm. The dry season is from November to February with very few sporadic rains occur occasionally. The National Meteorological Services Agency (NMSA) operates a meteorological station at Shashemene town, which records only rainfall. The station's Rainfall data has collected from NMSA covering the 1987-2017 period (Figure 3.2).

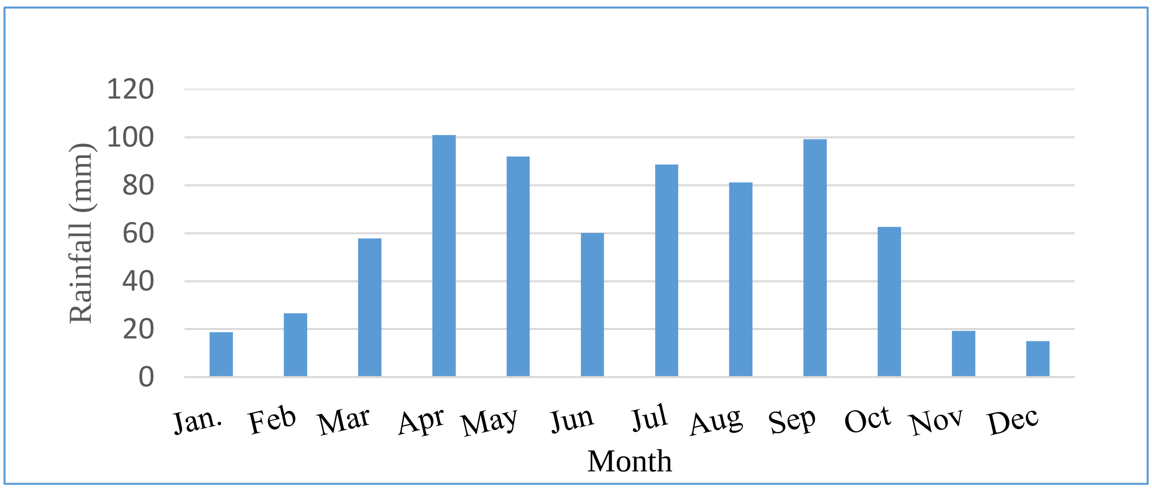


Figure 3. 1: Mean monthly rainfall over the period (1987- 2017)

3.1.2.2 Temperature

The average annual maximum and minimum temperatures are 29.1^{0}C and 12.1^{0}C respectively. March and April are the hottest months with a mean monthly temperature of 29.1^{0}c and January is the coldest month temperature of 12.1 ^{0}C see (Table 3.1).

Table 3. 1: Average temperature of the study area.

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Tem .max	24.8	28.5	29.1	29.1	27.6	27.3	26.8	26.3	27.2	27.7	28.0	27.0
Temp.min	12.1	12.9	13.6	14.1	13.6	14.3	13.2	13.6	13.6	13.2	12.4	13.0
Mean	18.5	20.7	21.3	21.6	20.6	20.8	20.0	19.9	20.4	20.4	20.2	20.0

3.1.3. Soil Data

Soil properties influence the relationship between runoff and rainfall since soils have differing rates of infiltration. The runoff coefficient of a surface varied with the soil type of the area. According to the FAO Soil Map of Ethiopia classification (FOA, 2007) and Ethiopia Minister of Agriculture (Appendix 27), the types of soil in this study area are covered chromic vertisols and eutric regosols. This soil classifies in Hydrologic soil group D and A respectively coverage study area see (Figure 3.3).

Group A: Sand, loamy sand, or sandy loam. Soils having a low runoff potential due to high infiltration rates.

Group B: Silt loam, or loam. Soils having a moderately low runoff potential due to moderate infiltration rates.

Group D: Clay loam, silty clay loam, sandy clay, silty clay, or clay. Soils having a high runoff potential due to very slow infiltration rates. Each of type soil group contributes a different infiltration rate. This value uses for rainfall run-off analysis to estimate peak discharge in the Rational method, SCN, and SWMM.

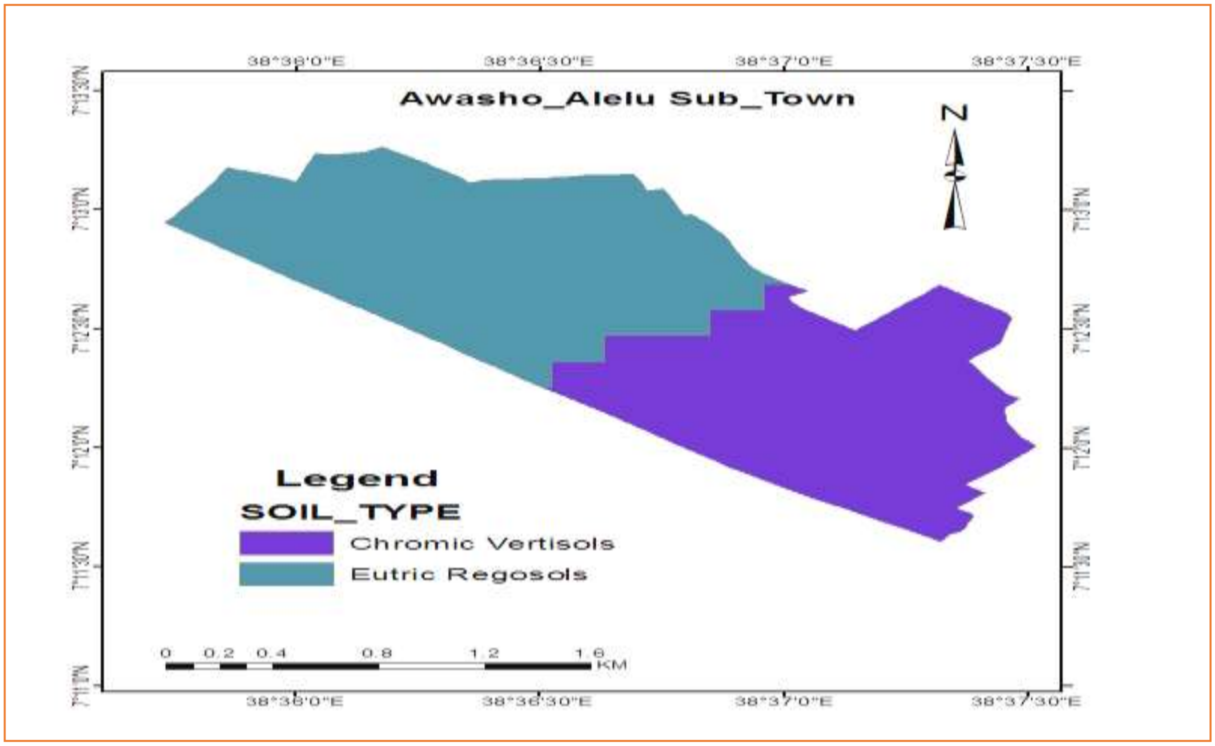


Figure 3. 2: Soil type of study area

3.1.4. Land-Use

Land use significantly affects the parameters of a runoff event. The human activity within most watersheds varies concerning time. For example, a rural watershed developed into a commercial area in a short period. Factors subject to change with general variations in land use such as a permeable change to impermeable areas, deforestation, topographic features change, and features of Drainage systems changes. All of these factors usually affect the rate and volume of runoff expected from a watershed (ERA, 2013).

The land use data would be an essential contribution to the calculation of runoff coefficient computing of runoff rate using the rational method, SCS, and SWMM. The categorization of different land-use done by field survey, shapefile, DEM, and with the help of Google earth and Arc GIS. These areas land use covers residential, commercial, road (asphalt, cobblestone, and gravel), industrial, green area, public Institution, Bare land, and Administration office as shown (Figure 3.4).

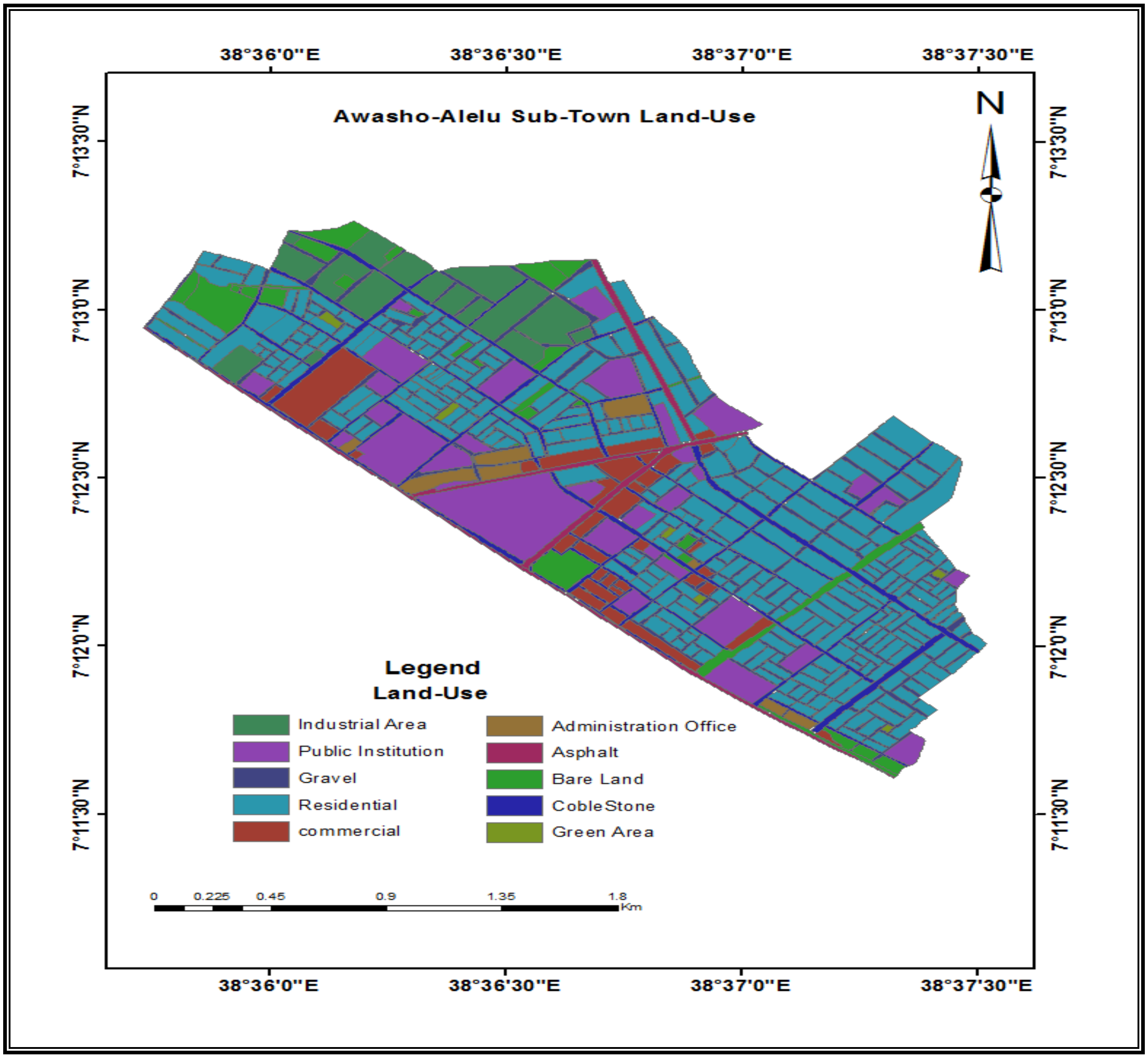


Figure 3. 3: Land use type of study area

3.2. Data Collection and Analyses

The data collection acquisition from two sources of data includes primary and secondary.

- **The Primary Source**

  Observations Study Area

The other source of primary data collected for this Thesis is captured pictures by the camera and measuring the cross-section of existing drainage structures by tape meter and elevation by GPS.

- **Secondary Sources**

Secondary data are collected for supplementary information that was used include metrological data, soil data, DEM, books, journals, manuals, conference proceedings, etc.

3.2.2. Hydraulic Capacity of the Existing Drainage systems

Hydraulic structure of Shashemene town the volume of drainage structure, which, collect runoff from each sub-catchments to common point of the outfall. From field, surveying collected data for computation hydraulic structure such as discharge, velocity, and hydraulic surface as shown (Appendix 21).

3.2.2.1 Manning's Equation

Manning's formula has used for calculating the cross-sectional area, wetted perimeter, and hydraulic radius for the flow of a specified depth in a canal. The formula was applicable for a constant flow rate through the canal with a constant slope, size, and shape, roughness geometry, and canal type. The hydraulic design of the existing drainage system is checked by using the Manning formula as an equation. This method has been used for major streams to compute the design flood levels at crossing sites after the discharges have been estimated by the hydrological methods of either Rational or SCS (CN) method. Manning's Formula has been used to compute the average flow velocity (V) in any canal with the uniform flow (equation 3.2). Manning's formula can readily solve for a given canal when the known depth of flow has assumed of flow is used. However, to determine the depth of flow that a computed discharge will produce in a canal obtained from the existing drainage system of Shashemane town. Canal discharge quantity (Q) is the product of the mean canal velocity (V) and the area (A) of the canal. To determine the discharge (Q) in natural drainages, canals, and non-erosive and scouring velocity, the following formula is used (3.1).

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

$$V = \frac{Q}{A} = \frac{1}{n} R^{\frac{2}{3}}\, S^{1/2} \quad \text{………………………………………………………… (3.2)}$$

Where, Q = the volumetric flow rate passing through the canal reach in m^3/sec.

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

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

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

R = the hydraulic radius, which is the cross-sectional area of flow normal to the flow direction in m^2 divided by the wetted perimeter of the cross-sectional area of flow in m.

n = manning roughness is a factor used to estimate resistance to the flow of water. Local conditions contributing to roughness include the degree of channel meandering in the area and the amount and type of obstructions in the channel. These channel characteristics are factors in computing flows by Manning's Equation (Arcement and Schneider, 1989; Barnes, 1967, Torres, 2006 and Chow et al., 2012).

The roughness coefficient reflects the amount of resistance that surface of drain structure and overland sub-catchment flow of the runoff generation. The values of roughness typically increase as canal vegetation and debris (ERA DDM, 2013). Roughness Coefficient (n) varies considerably, depending on the characteristics of a canal of smoothness of a canal, pipe, etc. see (Appendix 17). Tape meter and GPS: to measure the existing stormwater drainage lines: depth, width, length, and elevation which helps to evaluate the capacity of the drainage system.

3.2.2.2 Hydrologic Analysis

3.2.2.2.1 Runoff using Rational Method

The rational method is the most commonly used model for storm drainage systems, has primarily based on the concept that peak discharge from catchments would occur when the rain takes long enough at its maximum intensity to enable all portions of the basin to contribute to the flow. In the design of stormwater drainage systems, the main purpose of the hydrologic analysis is to determine the maximum amount of run-off (peak discharge) that accumulated at a certain storm drainage outlet (usually a ditch) along a highway. The rational method is one of the deterministic methods and is recommended for the size of the drainage area of less than 50 ha (ERA, 2023). As a result, all Sub-catchments of this study area from Sc-1 to Sc-19 for this thesis involved in this method. Therefore, the rational method to calculate the estimated peak runoff from each sub-catchment equation (3.3).

$$Q = 0.00278CIA \qquad (3.3)$$

Where:

Q =Peak run-off the catchment (m^3/sec), C=Runoff coefficient of the catchment,

I = rainfall intensity in (mm/hr.),

A = catchment area in (Ha).

A. Catchment Area

A catchment is defined as an area of land that captures precipitation and funnels it to the outlet. This catchment area will be is determined from topographic maps after delineation of the catchments from the Digital Elevation Model, and Shapefile of the regarding the study area.

The catchment of study area delineation based on polygons classification of land-use features on Google Earth image. Then the polygons with Kml were converted to layers by (ArcGIS 10.4) and continued to divide into much more cells according to the drainage systems. The cells are the sub-catchments for calculating rainfall-runoff. The runoff contribution area of existing drainage canals is delineated based on the flow direction map by indicating the outfall boundary of the study area. For a large catchment area, it is necessary to divide the area into sub-catchment accounts for artificial drainage systems, obtain analysis results at different points within the catchment area, and for locating stormwater drainage structures and assess their effects on the flood flows to the outlet point.

It is used to calculate and model-simulated peak runoff from Rational, SWMM, and SCS (CN) Method in each sub-catchments. The material used during area delineation is ArcGIS (10.4), Google Earth Pro (7.3), DEM, field observation, and shapefile. This parameter defines as flowing.

- **Geographical Information System (ArcGIS 10.4) Model**

The model was used to obtain the physical parameters and spatial information of the Catchment of the study area to show flow direction, land cover, flow length, catchment boundaries, drainage areas, and average slope of the catchment.

- **Google Earth Pro (7.3) version**

Google Earth uses to verify Catchment and divides of catchments of the study area to compare and identify the different land features. The land use of study area classify from Google Earth image and field survey are; residential, commercial, cobblestone, asphalt, gravel, green area, Administration office, Public Institution, Industrial and Bare Land. Within the different image interpretation. From images of the Google earth land use to obtain Land use area A (ha), it is also used to have the most recent data of the study area in the urban drainage system of the catchment as well as a sub-catchment.

B. Runoff Coefficient (C)

The runoff coefficient C is the function of the land use of the study area and used as input for peak discharge estimation. The more the surface is impervious the higher the runoff would be as the infiltration decreases. Vegetation cover reduces runoff the impact of a raindrop on the ground and intercepts some of the rain on its leaves, branches letting evaporate and lead to high infiltration rate consequently this directly decreases the runoff coefficient. Impervious surfaces increased in the urban area covered by pavement, asphalt, roof, and highly compacted soil profile which, increasing the runoff due to low infiltration rate (Chow, et al, 2012).

The runoff coefficient, C, is a dimensionless value representing characteristics of water contributing area that affects how much of the rainfall will become runoff and is a function of the land-use and a host of other hydrologic abstractions. Each land-use has its runoff coefficients. A weighted runoff coefficient is shown below (equation 3.4) method is employed to obtain the representative runoff coefficient i.e. the individual areas multiplied by their specific runoff coefficient and their values added together and divided by the cumulative area (Chow, et al, 2012) see (Appendix 1).

$$Cw = \frac{\Sigma CiAi}{\Sigma Ai} \qquad (3.4)$$

Ci = runoff coefficient for each drainage area of land-use

Ai = Area under each type of drainage area of land-use Cw= runoff coefficient for catchment

C. Intensity Duration Frequency (IDF) Curves

The IDF curve developed by (ERA 2013) was for several regions (RR-B2) see (Appendix 9). The rainfall region for the study area is (RR-B2) see (Figure 3.5)

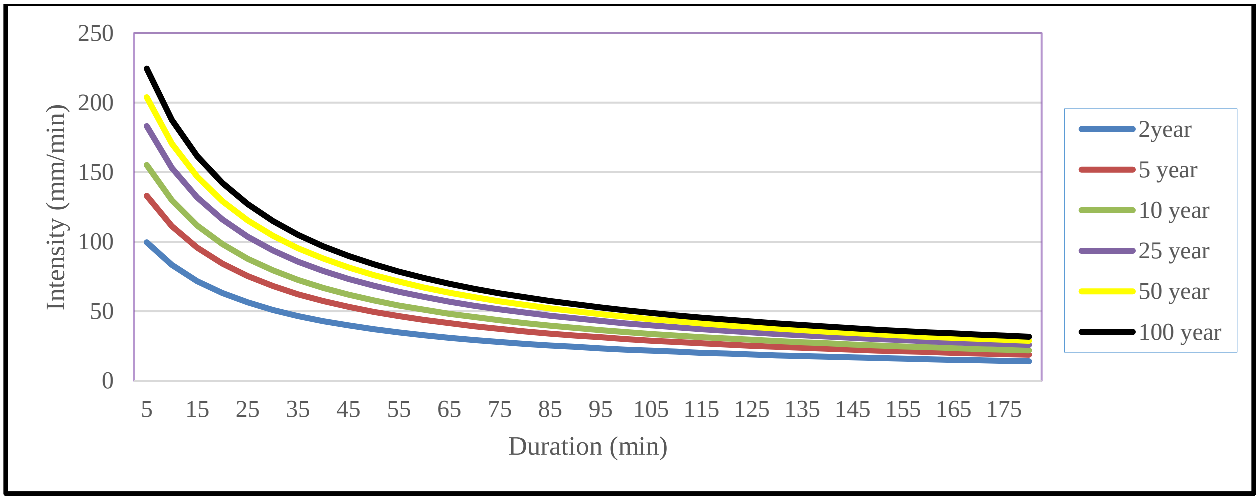


Figure 3. 4: IDFC of ERA Rainfall Region (RR-B2)

I. Rainfall Data

At present, there are several meteorological stations, which were installed by the National Meteorological Agency (NMA) of Ethiopia. For the study area, rainfall data were obtained from NMSA and from Shashemene-station, which has a record of 24 hours daily rainfall for different durations of record (1987-2017). The rainfall data is the basic input to many hydrological models as it affects the resulting runoff peak and volume as well as sets in one rain gauge station but due to the incompleteness of rainfall data additional three rain gauge stations are used for this thesis to fill unrecorded rainfall data. Point rainfall is the rainfall at a single station.

Estimating Missing Rainfall Data

Measured precipitation data were important to many problems in hydrologic analysis and design. For gages that require periodic observation, the failure of the observer to make the necessary visit the gage may result in missing data. Vandalism of recording gages was another problem that results in incomplete data records, and instrument failure because of mechanical or electric malfunctioning can result in missing data. Any such causes of instrument failure reduce the length and information content of the precipitation record (Dingmans, 2002).

Several methods have for estimating missing rainfall data were station average method, normal ratio method, and inverse-distance weighting method, and regression methods. From the methods, the normal ratio is used for this study. There were different approaches for estimating missing rainfall data varying with and based on the effect of orography on rainfall, the distance between the rainfall stations, and the variation of rainfall amount recorded on the stations. Normal ratio method, which is recommended to estimate missing data in regions where annual rainfall among stations differs by more than or not within 10% (Dingmans, 2002). This approach enables estimation of missing rainfall data by weighting the observation at Nx gauges by their respective with mean annual rainfall values of N1, N2 N3 then the missing data Px can be estimated by the see below equation (3.5).

$$Px = \frac{1}{3} * (P1 * \frac{Nx}{N1} + P2 * \frac{Nx}{N2} + P3 * \frac{Nx}{N3}) \quad \text{……….. (3.5)}$$

Where,

P_X - missing rainfall data (daily, monthly, or yearly).

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

N_x - mean annual rainfall at the missed station.

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

- ✓ **Check the Quality of Data**

Outlier test

An outlier test is a drive data that deviates significantly from the bulk of the data, helps to avoid those data lie out of the range in between the lowest datum and the highest datum, which may be due to errors in data collection, or recording, or due to natural causes. The presence of outliers in the data causes difficulties when fitting a distribution to the data. Low and high outliers are both possible and have different effects on the analysis (Chow, 1988). The lowest datum and the highest datum were calculated in equations 3.6 and 3.7.

Y=Log(x)

Lowest datum

$RL=10^{YL}$……………………………………………………………………………… (3.6)

Highest datum

$RH=10^{YH}$……………………………………………………………………………….. (3.7)

Where,

$YL = Yav - kn * \varepsilon_{n-1}$

$YH = Yav + kn * \varepsilon_{n-1}$

Yav=mean of data

Kn=factor form (appendix) corresponding to the no of year data

Kn=2.577 for 31 years (Appendix 1)

ε_{n-1}= standard deviation of the data computation of the equation as follows (Table 3.2)

X= maximum daily rainfall

Table 3. 2: Value of computation Outlier test

s.no	Yav	Kn	ε_{n-1}	YL	YH	RL	RH
1	1.6	2.577	0.13	1.26	1.95	18.11	89.25

Therefore, the smallest and highest of the given datum were 18.11 and 89.25 mm respectively. Since there is no data that is, lower than 18.11 mm and higher than 89.25 mm. That means all the available data satisfied the given condition.

Consistency of data

The consistent record was one where the characteristics of the record have not changed with time. Adjusting for gauge consistency involves the estimation of an effect rather than a missing value. An inconsistent record may result from any one of several events; specifically, an adjustment may be necessary due to changes in observation procedures, changes in exposure of the gage, change in land use that makes it unreasonable to maintain the gage at the old location, and where vandalism frequently occurs (Hardision, 1960).

Double mass curve (DMC) was used to check the consistency of rainfall for adjustment of inconsistent data. This technique is based on the principle that when each record data comes from the same parent sample, they are consistent. A group of four base stations in the neighborhood of the station was selected. A consistent record is one where the characteristics of the record have not changed with time. A double-mass curve is a graph of the cumulative catch of rain gage of interest versus the cumulative of one or more gage in the region that has been subjected to a similar meteorological occurrence and is known to be consistent. If a rainfall record is consistent with the estimator of the meteorological occurrences throughout the record, the double mass of the curve will have a constant slope. A change in the slope of the double mass curves would suggest that an external factor has caused changes in the character of the measured values. If a change in slop was evident, then either the record needs to be adjusted with the early or the later period of record adjusted (Hardision, 1960). It was determined by plotting the cumulative values of the observed time series of the station for which consistency needs to check on the y-axis versus the cumulative values of the observed time series of a group of the station on the x-axis (Appendix 6). A break in slope of the curve would indicate that conditions have changed that location that was inconsistent and adjusted to consistent values by multiplying by factored determine from the slope of the graph.

$$P`x = Px * \frac{M`}{M} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(3.8)$$

Where: $P`_x$ = Corrected precipitation at station x

P_x = original recorded Slope of the double mass curve

M` = corrected slope of the double mass curve

M = original slope of the double mass curve

According to the double mass curves analysis, all the rain gage stations around Shashemene town were consistent. For illustration, the double mass curves for some stations are presented see (Figure 3.6).

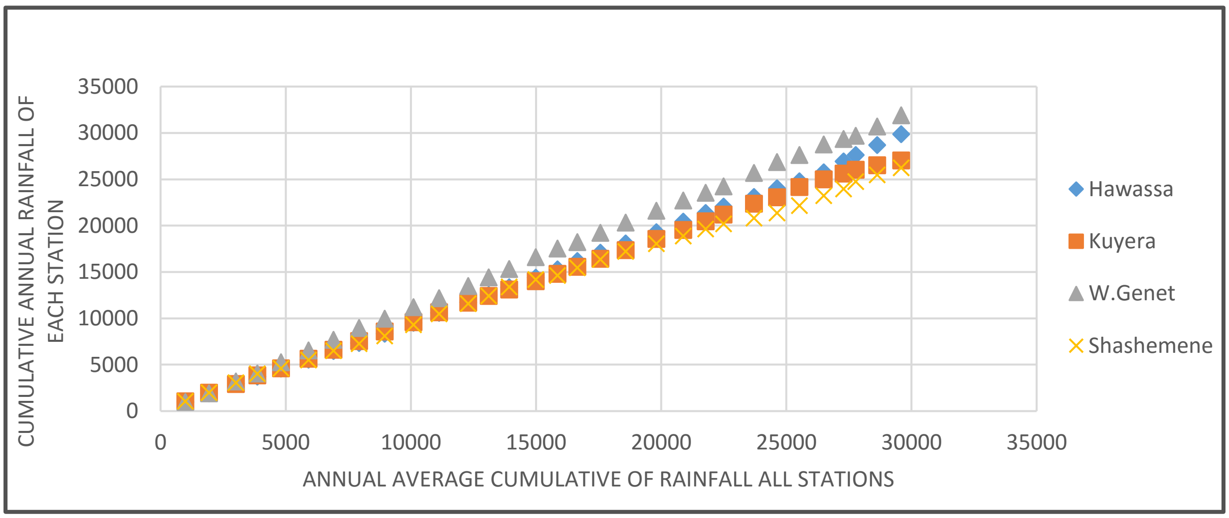


Figure 3. 5: Double mass curve of all station

II. Rainfall Frequency Analysis Methods

The term frequency analysis refers to the techniques to analyze the occurrence of the Metrological data variable within the static framework, by using measured data and statistical laws. The objective of frequency analysis of metrological data is to relate the magnitude of extreme events to their frequency of occurrence fit with probability distributions. The historical rainfall data available is a 24hr duration rainfall hence appropriate IDF reduction methods need to be used to obtain rainfall intensities of shorter durations. Rainfall during a year consists of several storms. The characteristics of a rainstorm are intensity (mm/hr), duration (min), frequency once in 2, 10, 20, 50, and 100 years. The frequency analysis method for these studies is annual maxima rainfall used (Appendix 7). This method of frequency analysis takes maximum daily rainfall event in each year See (Figure 3.7).

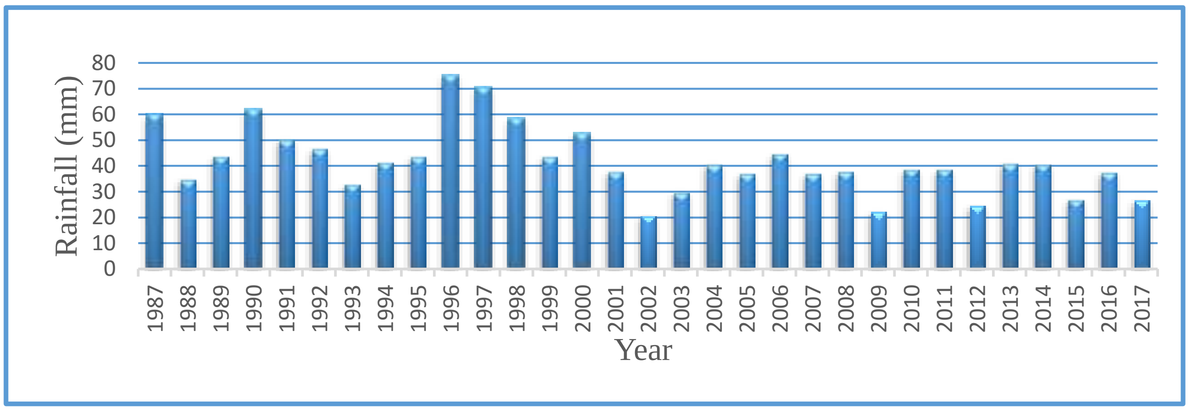


Figure 3. 6: Daily Maximum Rainfall Year (1987-2017)

III) The Goodness of Fit (GOF) Test of the Frequency Analysis.

The goodness of fit (GOF) tests measure the compatibility of a random sample with a theoretical probability distribution function. These tests show how well the selected distribution fits data. The IDF reduction formula suggested in the Ethiopian Road Authority drainage Design Manual 2013 used in this study. That is any probability distribution can be used as the model but the reliability of the distribution was checked by the goodness of fit tests. These tests were checked by Easy Fit 5.6 and Hydrognomon software.

- ✓ **Easy Fit 5.6 professional software**

The other method used to identify which distribution fits the theoretical probability distribution; GOF test conducted using Easy Fit 5.6 professional software. Kolmogorov-Smirnov test, Chi-Squared tests, and Anderson-Darling used in this Thesis. In all three tests, a parameter or statistic unique to each method was calculated for the required distribution types and these distributions ranked based on their parameter values (Appendix 8).

Kolmogorov-Smirnov Test

This test is used to decide if a sample comes from a hypothesized continuous distribution. It is based on the empirical cumulative distribution function (ECDF). When comparing different distribution, lower statistics mean a better fit see (equation 3.9).

$$\Delta = |P(Xi) - F(Xi)| \qquad (3.9)$$

Where;

Δ = Kolmogorov-Smirnov statistic

P(xi) =The cumulative probability for each of the observations
F(xi) = The theoretical cumulative probability for each of the observation

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 (equation 3.10).

$$AD = -\mathrm{n} - \sum_{i=1}^{n} \frac{(2i-1)\ln(p(1-p_{n-1+i}))}{n} \qquad (3.10)$$

Where

AD= Anderson-Darling statics

P=On the ordered probabilities

N= Number of sample size

Chi-Squared Test

The Chi-Squared test is used to determine if a sample comes from a population with a specific distribution. This test applied to binned data, so the value of the test statistic depends on how the data binned (equation 3.110.

$$(CS)^2 = \sum_{i=1}^{k} \frac{(Ni-Ei)^2}{Ei} \qquad (3.11)$$

Where; CS= Chi-Squared statics, Ei= Experimental data, Ni= observation data

When comparing different distribution, lower statistics mean a better fit. The result obtained from Easy Fit shows that the log-Pearson Type III distribution fits for the statistical value for all the three different test methods was lesser than that of the other values (table 3.3).

Table 3. 3: Goodness of Fit by Easy Fittest

S.no.	Distribution	Kolmogorov Smirnov		Anderson Darling		Chi-Squared	
		Statistic	Rank	Statistic	Rank	Statistic	Rank
25	Gumbel Max	0.13853	3	0.44559	3	0.20241	3
43	Normal	0.15423	4	0.67143	4	4.077	4
40	Lognormal	0.13491	2	0.41014	22	0.18739	2
38	Log-Pearson III	0.12985	1	0.4009	1	0.17573	1

✓ **Hydrognomon 4 software**

Rainfall frequency analysis of 24 hours duration daily maximum rainfall records of the study area-station carried out for different distributions. Comparison among the goodness of fit assessments for the different distributions indicates that station data is found to be more appropriate for fitting the empirical points for Shashemene Town meteorological and it is used for computation of magnitude of maximum. The goodness of fit (GOF) tests measure the compatibility of a random sample with a theoretical probability distribution function. Then from Hydrognomon 4 software result the log-Pearson type III attained the goodness of fit test @100% was 94.88% and the Dmax value 0.08677. Having Dmax value the least GOF test is highest and vice versa. Accordingly, the Log-Pearson III was selected for further analysis (Table 3.4).

Table 3. 4: Goodness of Fit by Hydrognomon test

Distribution	Kolmogorov-Smirnov test					
	a=1%	a=5%	a=10%	a=100%	Dmax	Rank
Normal	ACCEPT	ACCEPT	ACCEPT	48.92%	0.14305	4
Log-Normal	ACCEPT	ACCEPT	ACCEPT	91.07%	0.09404	2
Log Pearson III	ACCEPT	ACCEPT	ACCEPT	94.88%	0.08677	1
EV1-Max (Gumbel)	ACCEPT	ACCEPT	ACCEPT	85.92%	0.10162	3

Therefore, the result obtained from Hydrognomon and Easy-Fit 5.6 professional software the Log-Pearson III selected for the analysis of design rainfall for the required return period 10 years for design and 25 years for check-in this study. Because of the flood hazards that happened per 10 years at once, we need to check the max hazard flood for the longest return period, which means that for 25 years as suggested by Ethiopian Drainage Design Manual (ERA, 2013).

- **Frequency Distributions**

Normal Distribution

The normal distribution is the most widely known and used of all distributions. Because the normal distribution approximates many natural phenomena so well, it has developed into a standard of reference for many probability problems. The normal distribution is easy to work with mathematically (Subramanya, 1994). In many practical cases, the methods developed using normal theory work quite well even when the distribution is not normal (equation 3.12).

$$XT = \bar{X} + K_T * \delta_{n-1} \qquad (3.12)$$

Where:

XT=annual mean maximum flow of T-year return period.

$\bar{X}$= mean of annual maximum flow.

δ_{n-1}= standard deviation of the sample size.

KT = frequency factor expressed as,

$KT= W- ((2.51557+0.80285W+0.01033W^2)/ (1+1.143279\ W+0.1992W^2+0.00135W^3))$

$W = (\ln (1/Pr)^2)^{1/2}$

Pr=1/T

$$\delta_{n-1} = \sqrt{\frac{(Xi-\bar{X})^2}{n-1}}$$

$$\bar{X} = \frac{\sum Xi}{N}$$

N=Number of sample size. (Subramanya, 1994)

Gumbel Distribution

Distribution frequency analysis of flood first proposed by Gumbel in 1941. Gumbel distribution methodology was selected to perform the flood probability analysis. The Gumbel theory of distribution is the most widely used distribution for relationship intensity duration frequency (IDF) analysis and relatively simple and uses only extreme events (maximum values or peak rainfalls). The Gumbel method calculates the 2, 5, 10, 25, 50, and 100-year return intervals for each duration period and requires several calculations. In this method, from the raw data to compute the maximum precipitation (P) and the statistical variables (arithmetic average and standard deviation) for each duration (10, 20, 30, 60, 120, 180, 360, 720 minutes) computed. It is one of the widely used probability distribution functions for the estimation of peak or maximum rainfalls and is expressed by this equation (3.13) (Subramanya, 1994).

$XT = \bar{X} + K_T * \delta_{n-1}$ …………………………………………………….………. (3.13)

Where:

XT=annual mean maximum flow of T-year return period.

δ_{n-1}= standard deviation of the sample size.

KT=frequency factor and expressed as

$K_T = \frac{(Y_T - \overline{yn})}{Sn}$

Y_T=reduced variety, a function of T and given by,

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

y_n = reduced mean, it is a function of sample size.

S_n =reduced standard deviation which is also a function of the sample.

ȳn and Sn has to be obtained from the Gumbel table

- **Log- Pearson Type III Distribution**

The Log Pearson type III (LPIII) probability model is used to calculate the rainfall intensity at different rainfall durations and studies return periods to form the historical data IDF curves for each station. Log Pearson type III distribution involves logarithms of the measured values. The mean and the standard deviation (statistical variables) are determined using the logarithmically transformed data, using these frequency distribution functions, the maximum rainfall intensity for considered durations 2, 5, 10, 20, 50, and 100 years studies return periods. In this method, the flow data first transform into logarithmic form (base ten) and the transformed data then analyze If X is the variety of random flow series then the series of Z varieties where Z are obtained for this series for any recurrence interval T see the equation (3.14 and 3.15) (Subramanya, 1994).

$Z = \log X$

$Z = \bar{Z} + K_Z * \delta_Z$……………………………………………………………….. (3.14)

Where:

Kz = f (CS, T) is given in the table as a function of Cs and T see (Appendix...)

C_s =coefficient of sleekness,

δ_Z= standard deviation of Z variety sample.

$$\delta z_{n-1} = \sqrt{\frac{\sum(Zi-\bar{Z})^2}{N-1}}$$

$$C_s = \frac{N\sum(Z-\bar{Z})3}{\delta_Z(N-1)(N-2)}$$

$X_T = 10^{zt}$ ……………………………………………………………………… (3.15)

Lognormal distribution

The lognormal distribution is a special type of Pearson type III distribution with a Cs = 0 equation (3.16) (Subramanya, 1994).

$Z_t = \bar{Z} + K_Z * \delta_Z$

XT= 10^{Zt}…………………………………………………………………….... (3.16)

Table 3.5 the frequency analysis log-Pearson type III has better R^2 values with 0.767. Also as suggested by Ethiopian Drainage Design Manual (ERA, 2013), a distribution that has better R^2 values was chosen as the goodness of fit tests. Accordingly, the Log-Pearson III is chosen for further analysis.

Table 3. 5: Frequency Distribution

Distribution	Return year 10	Equation	Correlation R^2	Rank
R24N	57.3	y=0.3757+52.42	0.650	4
R24Gl	59.6	y=0.36x+50.22	0.739	3
R24LgPIII	61.8	y=0.43+52.63	0.767	1
R24LN	69.1	y=0.376x+57.42	0.755	2

Point rainfall depth developed Shashemane station less than ERA rainfall depth design for large coverage (RR B2) region as shown below (Figure 3.8)

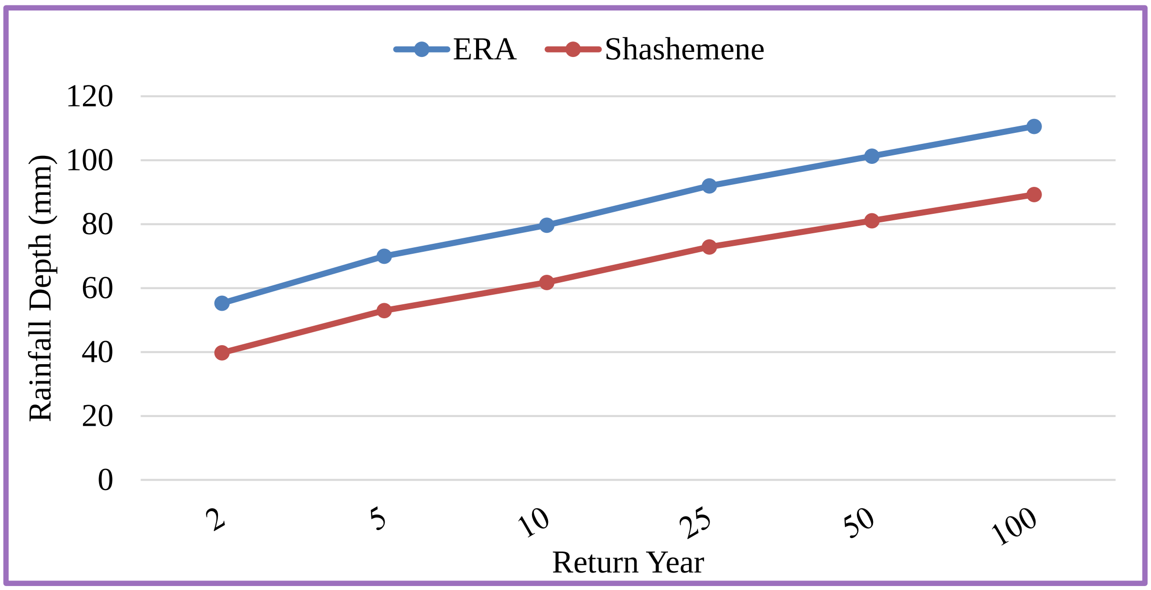


Figure 3. 7: Rainfall depth ERA VS Shashemene Town station

IV. Design rainfall of shorter duration

The rainfall depths obtained from the gauging station were as 24hr duration depth. However, when short time duration rainfall data were not available, the intensity of a short time rainfall of longtime rainfall would be calculated using a reduction formula. Design and analysis of drainage structures require rainfall intensity-duration relationship of shorter duration. Because rainfall data of shorter duration is unavailable, appropriate IDF derivation for the shorter duration is required. Ethiopian Road Authority (ERA) Drainage Design Manual of 2013 suggests the following (equation 3.4) for calculation of shorter duration rainfall from 24-hour duration rainfall (appendix 11).

$RRt = \frac{t(b+24)^n}{24(b+t)^n}$ …………………………………………………………………………… (3.17)

Where,

RRt = Rainfall depth ratio (Rt: R24)

Rt = Rainfall depth in a given duration t (mm)

R24 = 24hr rainfall depth

Coefficients b = 0.3 and n = 0.78 - 1.09

The methods employed to develop IDF curve for the shorter duration events using the above (equations 3.18) are as follows see (appendix 12).

$Rt = \frac{t(b+24)^n}{24(b+t)^n} * R_{24}$ ……………………………………………………………..…… (3.18)

The average rainfall intensity (I), (mm/hr), for a storm duration equal to the time of concentration for the Sub-catchments. Once the time of concentration has been calculated, the rainfall intensity can be read from the intensity-duration curve and then used in the Rational and SCN Method equation. The rainfall intensity (I) is the average rainfall rate in mm/hr for a duration equal to the time of concentration for a selected return period. The IDF relationship is a mathematical relationship between the rainfall intensity, the duration, and the return period (the annual frequency of exceedance). For this research, The IDF Curves developed by ERA or point rainfall were compared and used to quantify the rainfall. Substituting Intensity (mm/hr.) in the above (equation 3.19) (Appendix 13).

$I_t = \frac{R_t}{t} = \frac{(b+24)^n}{24(b+t)^n} * R_{24}$ ……………………………………………………………….. (3.19)

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

Using b = 0.3 and n = 0.91 selected depending on geographic and rainfall region ERA manual results are tabulated for rainfall durations 10, 20, 30 … 180 minutes. The maximum peak flood computed using the return period considering the road standard and the design life span of the structure,10 years for design, and 25-year check (review) return period was adopted (design storm frequency by Geometric Design Criteria (ERA DDM, 2011).

- ✓ **Time of Concentration (Tc)**

The time of concentration (Tc) is used in the Rational Method to determine the critical rainfall duration, which can then be combined with an appropriate rainfall intensity duration frequency (IDF) relation to establishing the required design rainfall intensity, which is intended to the maximum runoff. The Tc is the time required for water to flow from the most remote point of the basin to the location being analyzed (ERA, 2013; Fang et al., 2007; Pilgrim, 1975, USDA-SCS, 1986 and Chow, 1959). The time concentration total time of concentration inlet and travel time (equation 3.20).

$$Tc = Tci + Tt \quad (3.20)$$

Where:

Tc= time of the concentration (minute),

Tci= time of concentration inlet (hrs.)

Tt= the travel time (hr)

Time of concentration inlet (Tci)

The time of concentration of drainage area is the time required for runoff from the farthest part of the drainage area to reach the outlet used as the duration for the design storm. The inlet time for the time of concentration is estimated using Airport or Federal Aviation Administration (1970) methods. This method could desirably be used for inner areas especially for developed areas of urban centers as (equation 3.21).

$$Tci = \frac{3.64(1.1-C)(L/1000)^{0.83}}{H^{0.33}} \quad (3.21)$$

Where;

Tci = time of concentration inlet (min.),

C = runoff coefficient

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

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

Travel Time

Water moves through a catchment area as sheet flow, shallow concentrated flow, open canal flow, or some combination of these. The type that occurs is a function of the conveyance system and is determined by field inspection. Travel time is the ratio of flow length to flow velocity as (equation 3.22)

$$Tt = \frac{L}{3600V} \quad \ldots\ldots\ldots\ldots (3.22)$$

Where,

Tt = travel time, hr

L = length of the main drainage canal (m)

V = flow velocity in the canal (m/s) from above equation.

3600 = conversion factor from seconds to hours.

3.2.2.2.2 Runoff using SCS Curve Number (CN)

The U.S. Soil Conservation Service for calculating rates of runoff develops the method. Curve Number (CN) estimates excess runoff as a function of cumulative precipitation, soil cover, land use, and antecedent moisture. SCS developed the method for calculating watersheds above 50 ha (ERA, 2011), and only upstream of this study area include this method. The SCS runoff present below is a method of estimating direct runoff from 24-hour rainfall. This method is calculating rates of runoff requires catchment area, runoff curve number, time of concentration, and rainfall depth, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the storm, either real or fabricated, by subtracting infiltration and other losses from the rainfall to obtain effective precipitation calculated by equation (3.23).

$$Q = \frac{(P-0.2S)^2}{P+0.8S} \quad \ldots\ldots\ldots\ldots (3.23)$$

Where,

Ia = Initial Abstraction (Ia = 0.2S)

Q = Excess runoff (mm)

P = Total rainfall depth (mm)

S = Potential Maximum Storage (mm)

$$S = \frac{25400}{CN} - 254$$

CN= Curve number

The potential maximum soil water retention, present related to hydrologic soil properties, land-use/ cover, and management conditions as well as the soil moisture status of the catchment before the rainfall event and their relation with curve number (CN) in the equation above. The value of the curve number different in land-use covered of each sub-catchment see (Appendix 19 and 20)

A. Ia/P Parameter

Ia/P is a parameter that is necessary to estimate peak discharge rates. Ia denotes the initial abstraction and P is the 24-hour rainfall depth for a selected return period. The 24 hr rainfall depth taken from the frequency analysis result in the rainfall region rainfall depth recommendations (ERA DDM, 2013). For this particular study, the 24 hr rainfall depth be taken from the frequency analysis of the rainfall region (Appendix 18).

B. Time of concentration

Sheet flow is defined as flow over plane surfaces. Sheet flow usually occurs in the headwaters of a stream near the ridgeline that defines the watershed boundary (Merkel, 2011). A simplified version of Manning's kinematic solution was used to compute travel time for sheet flow (equation 3.24). For sheet flow, the roughness coefficient includes the effects of roughness and the effects of raindrop impact including drag over the surface; obstacles such as litter, crop ridges, and rocks; and erosion and transport of sediment.

$$Tc = \frac{0.007(Ln)^{0.8}}{(P)_2{}^{0.5}(S)^{0.4}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(3.24)}$$

Where:

T_c = travel time, hr

n = Manning's roughness coefficient (Appendix 16)

L = sheet flow length, m

P2 = 2-year, 24-hour rainfall (mm)

S = slope of land surface, m/m

C. Peak Discharge Estimation

The following equation is used for the estimation of the peak discharge in the SCS Method (equation 3.25).

$q_p = q_u * Q * A$……………………………………………………………… (3.25)

Where:

Q_p=: Peak discharge (m^3/s)

q_u = Unit peak discharge ($m^3/s/Km^2/mm$)

Q= depth of rainfall (mm)

A=drainage area (km^2)

$q_u = \alpha * 10^{C_0 + C_1 * Log(Tc) + C_2 * Log(Tc)}$

C0, C1 and C2 = regression coefficients given in the table of EDDM, 2013 for various Ia/P ratios.

α = is unit conversion factor equal to 0.000431 in SI unit.

The unit peak discharge got from the above equation, which requires the time of concentration (Tc) in hours and the initial abstraction rainfall (Ia/P) ratio as input see (Appendix 18)

3.2.3 Storm Water Management Model (SWMM)

The EPA Storm Water Management Model (SWMM) is a dynamic rainfall-runoff simulation model used for a single event or long-term (continuous) simulation of runoff quantity and quality from primarily urban areas. The runoff component of SWMM operates on a collection of sub-catchment areas that receive precipitation and generate runoff. The routing portion of SWMM transports this runoff through a system of pipes, canal SWMM tracks the quantity and quality of runoff generated within each sub-catchment, and the flow rate, flow depth, and quality of water in each pipe, and canal during a simulation period comprised of multiple time steps (Rossman, 2004).

Dynamic Wave routing solves the complete one-dimensional Saint Venant flow equations and therefore, produces the most theoretically accurate results. These equations consist of the continuity and momentum equations for conduits and a volume continuity equation at nodes. 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 atop the node and re-enter the drainage system. Dynamic wave routing can account for canal storage, backwater, entrance/exit losses, flow reversal, and pressurized flow. Because it couples the solution for both water levels at nodes and flows in conduits it can be applied to any general network layout, even those containing multiple downstream diversions and loops. It is the method of choice for systems subjected to significant backwater effects due to downstream flow restrictions and with flow regulation via weirs and orifices. Flow routing within a conduit link is governed by the conservation of mass and momentum equations for gradually varied, unsteady flow (i.e., the Saint Venant flow equations).

Continuity equation.

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0 \qquad (3.26)$$

Momentum equation

$$\frac{\partial Q}{\partial t} + \frac{\partial(\frac{Q^2}{A})}{\partial x} + gA\frac{\partial H}{\partial x} + gASf + gAhL = 0 \qquad (3.27)$$

Where x is the distance long the conduit, t is time, A is the cross-sectional area, and Q is flow rate, H is the hydraulic head of water in the conduit (elevation head plus any possible pressure head), Sf is the friction slope (head loss per unit length), hL is the local energy loss per unit length of conduit, and g is the acceleration of gravity. Note that for a known cross-sectional geometry, area A is a known function of flow depth y, which in turn can obtain from the head H. Thus the dependent variables in these equations are flow rate Q and head H, which are functions of distance x and time t. The friction slope Sf can in terms of being expressed the Manning equation as:

$$Sf = \frac{n^2 v^2}{K^2 R^{4/3}} \qquad (3.28)$$

Where *n* is the Manning roughness coefficient, *V* is the flow velocity (equal to the flow rate *Q* divided by the cross-sectional area *A*), *R* is the hydraulic radius of the flow's cross-section, and *k* = 1.49 for US units or 1.0 for metric units. The local loss term *hL* can be expressed as KV2 2gL where K is a local loss coefficient at location x and L is the conduit length. To solve equations (1) and (2) over a single conduit, one needs a set of initial conditions for *H* and *Q* at time 0 as well as boundary conditions at *x* = 0 and *x* = *L* for all times *t*. When analyzing a network of conduits, an additional continuity relationship has needed for the junction nodes that connect two or more conduits. In SWMM a continuous water surface is assumed to exist between the water elevation at the node and in the conduits that enter and leave the node (except for free fall drops should they occur). The change in hydraulic head *H* at the node concerning time can express as:

$$\frac{\partial H}{\partial t} = \frac{\sum Q}{Astore + \sum As} \qquad (3.29)$$

where A_{store} the surface area of the node itself, ΣA_s is the surface area contributed by the conduits connected to the node, and ΣQ is the net flow into the node (inflow –outflow) contributed by all conduits connected to the node as well as any externally imposed inflows. Note that the flow depth at the end of a conduit connected to a node can compute as the difference between the head at the node and the invert elevation of the conduit. Finally, select the best model type for runoff routing.

3.2.3.1 Input Parameters of SWMM

Table 3. 6: Hydrology Elements for SWMM input

Hydrology Elements			
S.no	Type	In Put Properties	Description
1	Rain gage	time interval	Successive time (hr.)
		rain fall format(type)	intensity(mm/hr)
		data source	Time series
2	Sub-catchment	rain gage	Associated with the sub-catchment.
		Outfall	Name of the node or sub-catchment which receives the sub-catchment runoff
		Area	Area of the sub-catchment in ha.
		Width	Characteristic width of the overland flow path for sheet flow runoff (m)(A/L)
		% Av.slope catchments	Average percent slope of the sub-catchment.
		% impervious	Percent of the land area, which is impervious.
		N-impervious	Manning’s n for overland flow over the impervious portion of the sub-catchment
		N-previous	Manning’s n for overland flow over the previous portion of the sub-catchment
		infiltration model	Horton

Table 3. 7: Hydraulic Elements for input SWMM

Hydraulic Elements				
S.no	Type		In Put Properties	properties description
1	Node	Junctions	inverted elevation	Invert elevation (at base) of the junction (m).
			Max. depth	Maximum depth of junction (m).
		Outfalls	inverted elevation	Invert elevation (at base) of the outfall (m).
			time-series	
2	Links	Conduits	inlet node	Name of the node on the inlet end of the conduit at a higher elevation.
			outlet node	Name of the node on the outlet end of the conduit at the lower elevation
			Shape/geometry)	Rectangular, trapezoidal, circular (conduit's cross-section)
			Max. (d, S, b, Ø)	conduit's properties (m)
			Length	Conduit length (m).
			Roughness	Manning's roughness coefficient for opened and closed conduit values.
			offset (in and out)let	The height of the conduit invert above the node inverts at u/s and d/s end of the conduit.
			Initial inflow	The initial flow of the conduit at the start of the simulation (flow units)

3.2.3.2 Run of Simulation

After a study area was suitably described, its runoff and routing behavior was simulated. This section describes how to specify options to use in the analysis, how to run the simulation. For the constant current land use condition with different rainfall intensities (especial heavy rainfall intensity of short durations), the flooding and other flow characteristics are high or cause flooding problems at the junctions and links. The maximum designed water level of the drainage canal is less than the flood level in the junctions and links of both upstream and downstream parts of the systems.

3.2.3.3 Flow Chart of SWMM Model

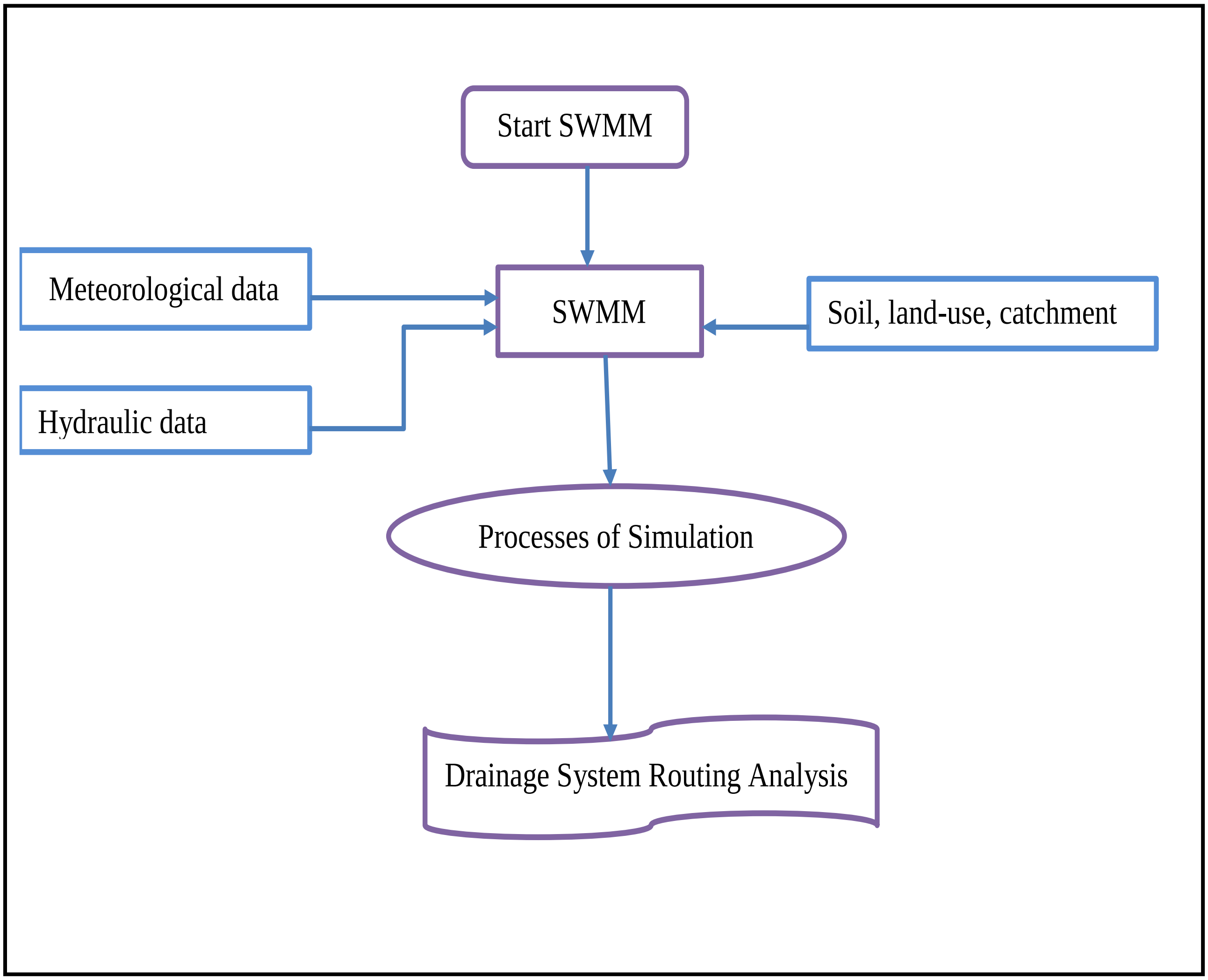


Figure 3. 8: Flow chart of SWMM Model

3.3. Research Methods

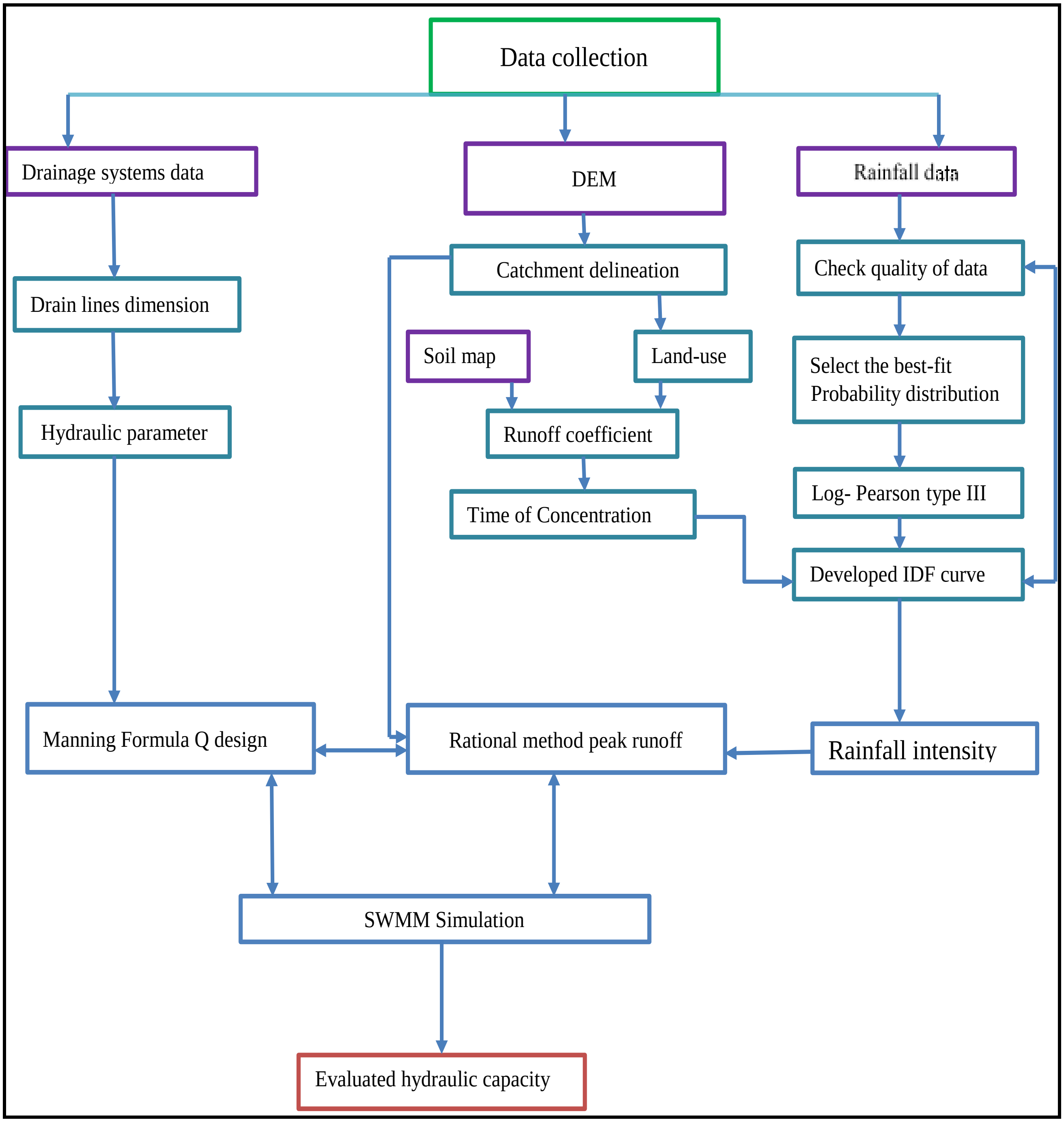


Figure 3. 9: General framework of Study

4. RESULTS AND DISCUSSIONS

4.1. Current Situation of the Existing Drainage System.

After the result of a field survey and visit the existing drainage system covers most of the developed areas not sufficient to divert and convey the excess run-off from other areas of the town. The drainage system consists of a considerable length of open masonry, gutters and concrete ditches, outfalls, and cross-drainage structure. The drainage service in the town was uncontrolled stormwater runoff has highly damaged roads, drainage system, private and public property, different infrastructure developments, life loss of human and animal and the pollutant discharged with stormwater in the town and downstream of around Shashemene town. Most open drainage ditches are inadequately carried capacity, not well connected.

4.1.1. Lack of carrying capacity of the drainage system during a storm.

Among many problems, the main cause for the overtopping was the lack of adequate hydraulic design. The rapidly increasing rate of urbanization leads to change land-use from the previous surface to impervious surface. The impervious surfaces are generated high runoff that creates overflow or flooding on infrastructure (Figure 4.1). The construction of the drainage ditch design was carried out without considering the ERA or Shashemene hydrological and hydraulic during the design stage. The hydrological analysis used to know peak discharge of the surrounding sub-catchment and hydraulic also analyze carry capacity peak discharge. Drainage crossing is designed to no pass the appropriate storm flows and debris or to survive overtopping.

Most of the drainage systems are not adequate to carry the design flood based on intensity with a return period of 10 years. The sub-catchments include in Awasho-Alelu kebeles existing storm drainage systems insufficient carry capacity of overflow are Sc1, Sc2, Sc3, Sc4, Sc5, Sc6, Sc7, Sc10, Sc11, Sc13, Sc14, Sc15, Sc17, and Sc19.

Figure 4. 1*:* Overtopping due to insufficient carrying capacity hydraulic structure

4.1.2. Siltation and ponding.

Sediment is defined as any settled particulate material from the surface of the sub-catchment that that which, under certain conditions, form bed deposits in sewers and associated hydraulic structures. Urban areas in developing countries like Shashemene Town have significant proportions of exposed soil liable to erosion, giving rise to large quantities of sediment. Building sites, whether in areas where the town is expanding or within the developed urban area and dumping solid waste do not normally have controls for erosion prevention or for retaining sediment so that it does not reach the streets, storm drains, etc.

The consequence of sediment is blockage and loss of hydraulic capacity of drain structure. The sediment deposited in drainage ditches because of inadequately slopes provided a bed of the canal, the accumulated of solid waste in and damaged canal. The inadequate slopes provide drainage ditches filled with solid waste, sedimentation, and sewerage resulting in the formation of sewage ponds in the channel as well as on the road see (Figure 4.2). Gravitational settling of suspended particles from the water column.

Figure 4. 2: Lack of inadequate slope cause siltation and ponding.

4.2. Catchment of the Study Area

A sub-catchment is an area of land containing a mix of different Land-Use types and surfaces whose runoff drains to a common outlet point, which could be either a node of the drainage network or another sub-catchment. Then the result from google earth was an area of land containing a mix of previous and impervious surfaces that dividing into nineteen sub-catchments see (Appendix 15). The sub-catchment of the study area (Figure 4.3).

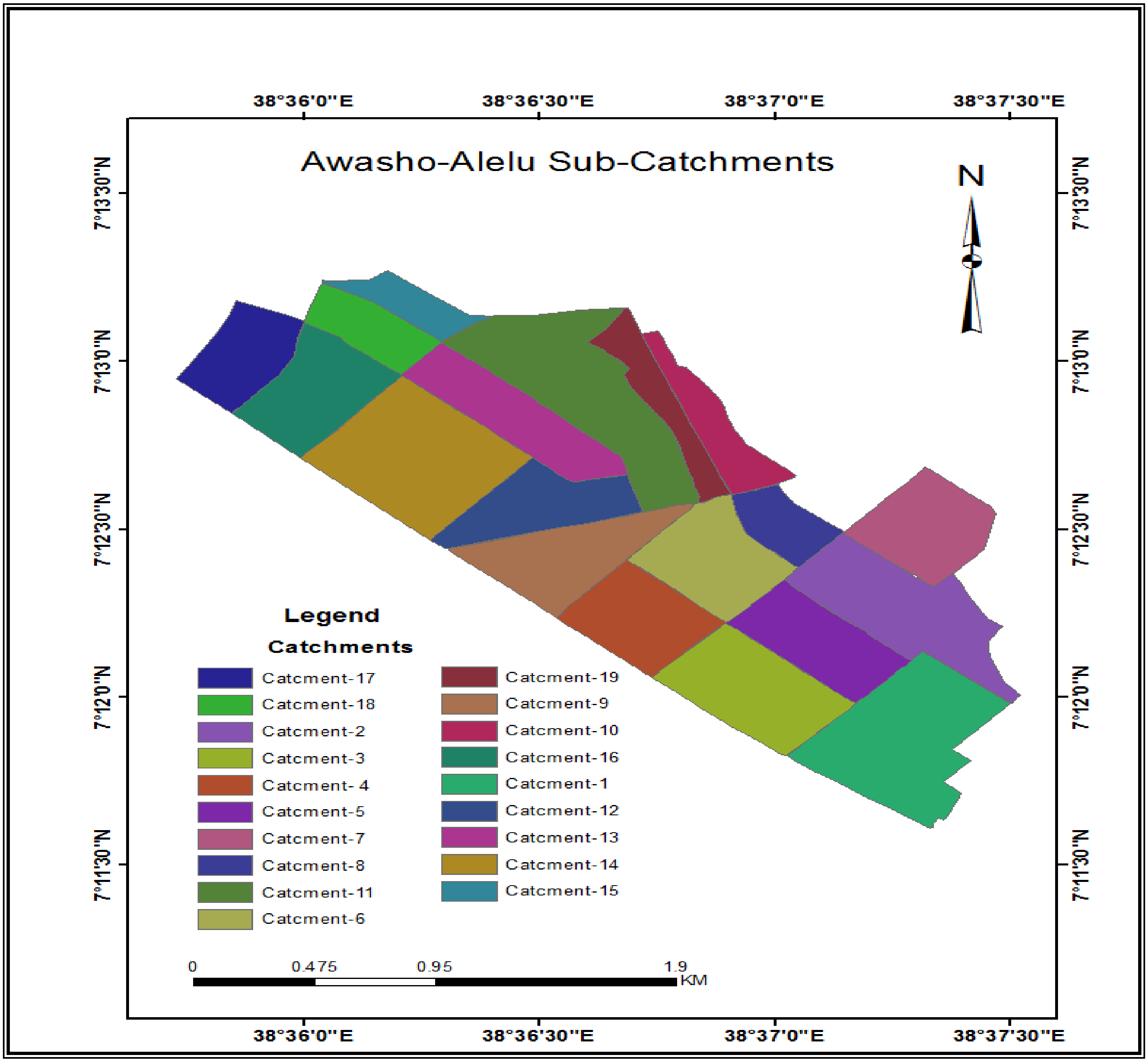


Figure 4. 3:Sub-catchment of the study area

4.2.1. Land-Use Type

Shashemene Town has experienced a rapid physical expansion in all directions. The town had different land-use maps, which developed at different times in history. Land-use is an activity human that land uses for a different purpose. The Land-Use of the study area are: residential, commercial, industrial, cobblestone, gravel, asphalt, green area, bare land, public institution, and administration office see (Figure 4.3). The peak runoff generates from different land-use types. It depends on surface coverage (i.e. previous and impervious) of each land-use type called runoff coefficients. A runoff coefficient is an empirical-constant value that represents the percentage of rainfall that becomes runoff. Using the runoff coefficient to represent the percent imperviousness of sub-catchment results estimate the runoff of both the impervious and pervious areas of the sub-catchment. The weighted runoff coefficient determines from each land-use and sub-catchments.

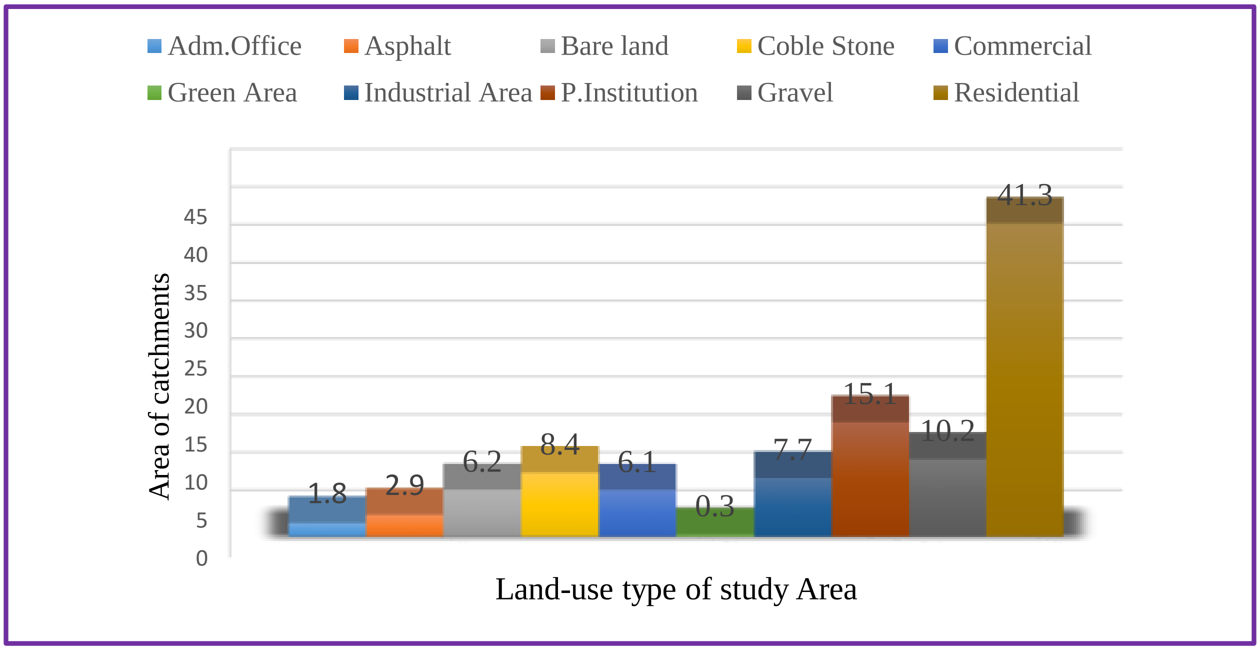


Figure 4. 4: Land-Use type of study area

4.2.2. Drainage network of the study area

The urban drainage system different from the natural drainage system all runoff generated from the catchment would not flow towards the natural outlet that runoff flows not only on the surface but also flows through man-made drainage structures such as roadside ditches, cancel, and gutter. However, urban catchment delineation not like natural catchment delineation, it depends on existing artificial drainage flow direction. Urban drainage systems layout based on the possible outflow direction which using elevation difference. As a result, the study area catchment of Shashemene town has three outfalls: Outfall 1 at Essa river at Alelu and 2 and 3 of outfalls into Malka Oda river at (Marana and Awasho) respectively as shown (Figure 4.5). Sub-catchments are delineated based on the flow direction of the drainage structures.

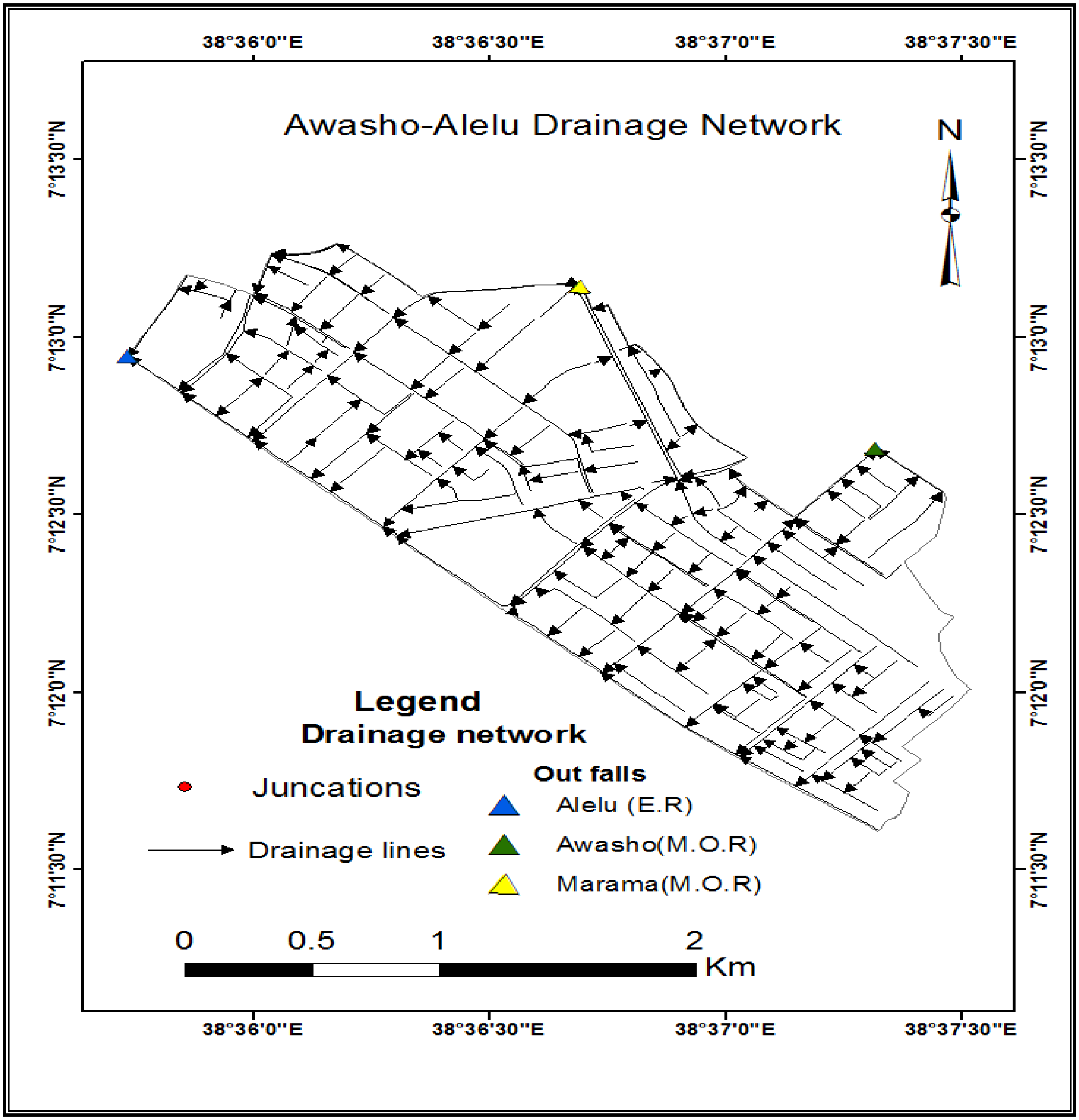


Figure 4. 5: Drainage network of the study area

4.2. Hydraulic Capacity

Hydraulic capacity is the process of combining all inflows that enter the upstream end of each conduit in a conveyance network and transporting these flows to the outlet. The land-use type and drainage network of the study area was the key point for the result of area, runoff coefficient, rainfall intensity, and length of the catchment to determine hydraulic capacity. The existing storm drainage facilities were most of the Study area covered by the open canal and constructed with masonry and concrete. Most of these canals' geometry shapes are rectangular. The discharge and velocity conveyed in the existing drainage system determined by the Manning equation as shown (Table 4.1) and (Appendix 21).

Table 4. 1: Existing canal capacity.

Sub-catchments	Q Manning (m^3/s)	Q Rational (m^3/s)
Cs1	2.05	4.37
Cs2	1.47	4.39
Cs3	3.08	3.89
Cs4	3.23	3.29
Cs5	2.38	2.84
Cs6	0.74	2.74
Cs7	2.14	2.97
CS8	2.17	1.85
Cs9	4.11	3.48
Cs10	0.95	3.57
Cs11	2.87	5.49
Cs12	3.35	2.75
Cs13	2.68	2.73
Cs14	2.51	6.02
cs15	0.89	1.16
cs16	4.19	3.45
cs17	0.79	1.83
Cs18	0.67	1.81
Cs19	0.84	1.46

4.3. Internal Peak Discharge

The rational method runoff used a proper selection of the runoff coefficient calculated depending on the different land cover that contributes to the sub-catchments of the drainage area. The portion of the total rainfall that would reach the storm drains depends on the percent imperviousness, slope, and ponding character of the surface. The frequency analysis of the extreme rainfall events analyzed using the Log-Pearson type III distribution and then peak flood discharge calculated using a rational method as presented in (Table 4.2) and (Appendix 22).

Table 4. 2: Peak Discharge using the Rational Method and hydraulic capacity of Manning.

Catchments	Area (Ha)	C	Tc(hr.)	Return year		Discharge Rational (m^3/s)		Discharge Manning(m^3/s)	Difference (m^3/s)
				I10	I25	Q10	Q25		
Cs1	41.22	0.56	0.47	68.16	80.39	4.37	5.66	2.05	+ 2.32
Cs2	33.04	0.55	0.30	86.27	101.75	4.39	5.70	1.47	+ 2.92
Cs3	29.09	0.60	0.35	80.79	95.29	3.89	5.04	3.08	+ 0.81
Cs4	22.32	0.65	0.28	88.97	104.94	3.29	4.26	3.23	+ 0.74
Cs5	21.20	0.55	0.30	87.07	102.69	2.84	3.69	2.38	+ 0.46
Cs6	21.70	0.61	0.41	74.08	87.37	2.74	3.55	0.74	+ 2.00
Cs7	20.76	0.56	0.26	92.04	108.56	2.97	3.86	2.14	+0.84
Cs8	11.45	0.58	0.22	99.72	117.62	1.85	2.40	2.17	- 0.32
Cs9	25.98	0.66	0.41	73.37	86.54	3.48	4.51	4.11	- 0.64
Cs10	13.05	0.61	0.33	82.30	97.07	3.57	4.64	0.95	+2.62
Cs11	41.06	0.62	0.41	73.76	87.00	5.49	7.12	2.87	+ 2.62
Cs12	19.68	0.62	0.34	81.62	96.27	2.75	3.56	3.35	-0.60
Cs13	23.21	0.56	0.39	76.01	89.65	2.73	3.54	2.68	+ 0.05
Cs14	40.42	0.65	0.33	82.98	97.87	6.02	7.82	2.51	+ 3.51
Cs15	8.01	0.60	0.30	86.25	101.72	1.16	1.51	0.89	+0.27
Cs16	23.15	0.58	0.26	92.59	109.21	3.45	4.48	4.19	- 0.74
Cs17	16.53	0.48	0.33	82.71	97.55	1.83	2.38	0.79	+1.04
Cs18	12.41	0.64	0.34	81.91	96.61	1.81	2.35	0.67	+ 1.14
Cs19	11.75	0.61	0.42	99.72	117.62	1.46	1.90	0.84	+ 0.62

The peak discharge of the study area estimated using a rational method from currently existed hydraulic capacity for design 10 years and check (review) 25 years considering the sub-catchments that contribute drainage area with the outfall of the study area. From above table indicate the difference discharge between estimated peak discharge Rational and existed hydraulic capacity Manning (+) indicates sign carrying capacity of canal insufficient that generates overflow and (-) sign indicates carrying capacity canal sufficient, no overflow exist. The existing and estimated drainage capacity the result obtained for a design period of 10 years as in (Figure 4.5), indications that 78.9% of the drainage systems of sub-catchments include in the existing storm drainage systems insufficient carry capacity of the storm and a maximum flood occurs. These drain lines insufficient to carry the capacity of the storm found in sub-catchments: Sc-1, Sc-2, Sc-3, Sc-4, Sc-5, Sc-6, Sc-7, Sc-10, Sc-11, Sc-13, Sc-14, Sc-15, Sc-17, and Sc-19 and have higher peak discharge than the existing canal capacity that cause an overflow. However, the existing capacity of drainage at Sc-9, Sc-8, Sc-12, and Sc-16 was adequate to carrying peak discharge.

4.4.1.Developing Intensity Duration Frequency (IDF) Curves

The IDF curve developed from 24-hour rainfall data of 31 years from 1987 to 2017 obtained from Ethiopian Meteorological Agency-rainfall gauge, and Shashemene town. Also, applying the appropriate reduction equation described in the methodology section with 24-hour rainfall depth obtained at different duration and return periods (T) (Table 4.3).

Table 4. 3: Log-Pearson Type III Distribution Method

Return period	X	$\varepsilon n - 1$	Kz	Z=x+kz*$\varepsilon n - 1$	R24LPIII=10^{Z}
2	1.600	0.134	0.005	1.599	39.753
5	1.600	0.134	0.928	1.724	53.006
10	1.600	0.134	1.425	1.791	61.780
25	1.600	0.134	1.960	1.863	72.867
50	1.600	0.134	2.307	1.909	81.091
100	1.600	0.134	2.618	1.951	89.255

The IDF curve developed compare to the specific area of Shashemene town with ERA 2013 for this thesis. From the result of the developed IDF Curve (ERA 2013) for regions (RR-B2) greater than the study area of Shashemene Town. Comparing the two developed IDF curves ERA and Shashemene Town by peak discharge the rational method with hydraulic existing drainage systems in this study area, the drainage systems Shashemene Town not sufficient to carrying capacity of peak discharge both of it. The study area of the IDF curve developed less than the ERA IDF curve but most of the drains are generate overflows. For this reason, I took IDF developed using the 31 years of rainfall data (1987-2017) of Shashemene town and the rainfall intensity information for various return periods and the duration 5 minutes to 180 minutes as shown (Figure 4.9).

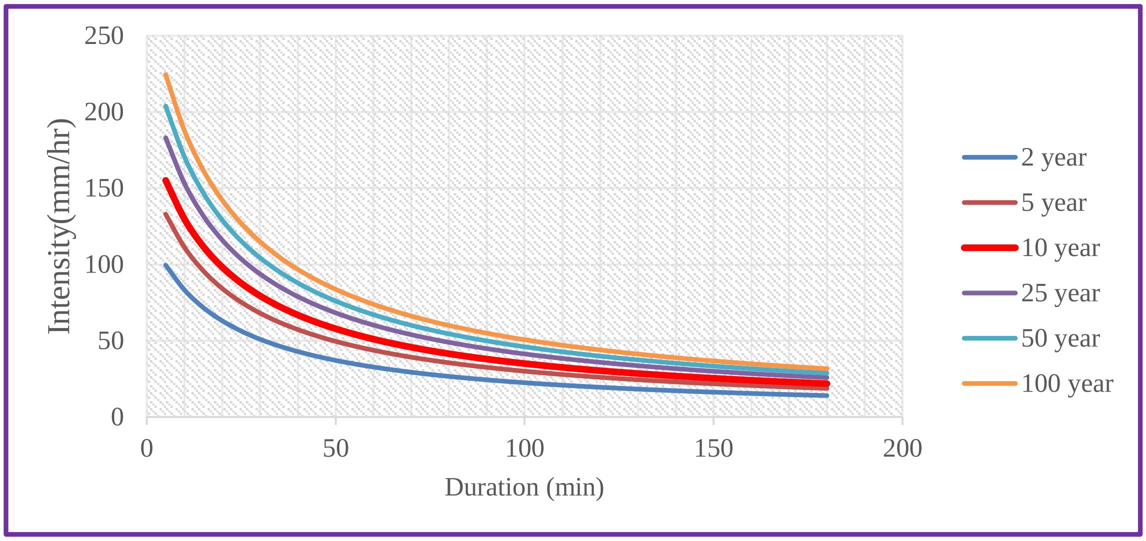


Figure 4. 6: IDF Curve of Shashemene Town.

4.4. External Peak Discharge

The external flood generates from upstream of the study area is covered 315 ha to the sub-catchment of the study area (Figure. 4.7). The flood frequency 10 year flows added 12.061 m^3/s to from around high land Abaro mountain in too low land study area mainly through sub-catchment one (Sc-1) and combined with the internal flood which, make the highest overflow in the study area see (Table 4.4). The carrying capacity of the existing canal Sc-1 not enough to carrying for internal flow of the study area but also external inflow generates from u/s of the study area.

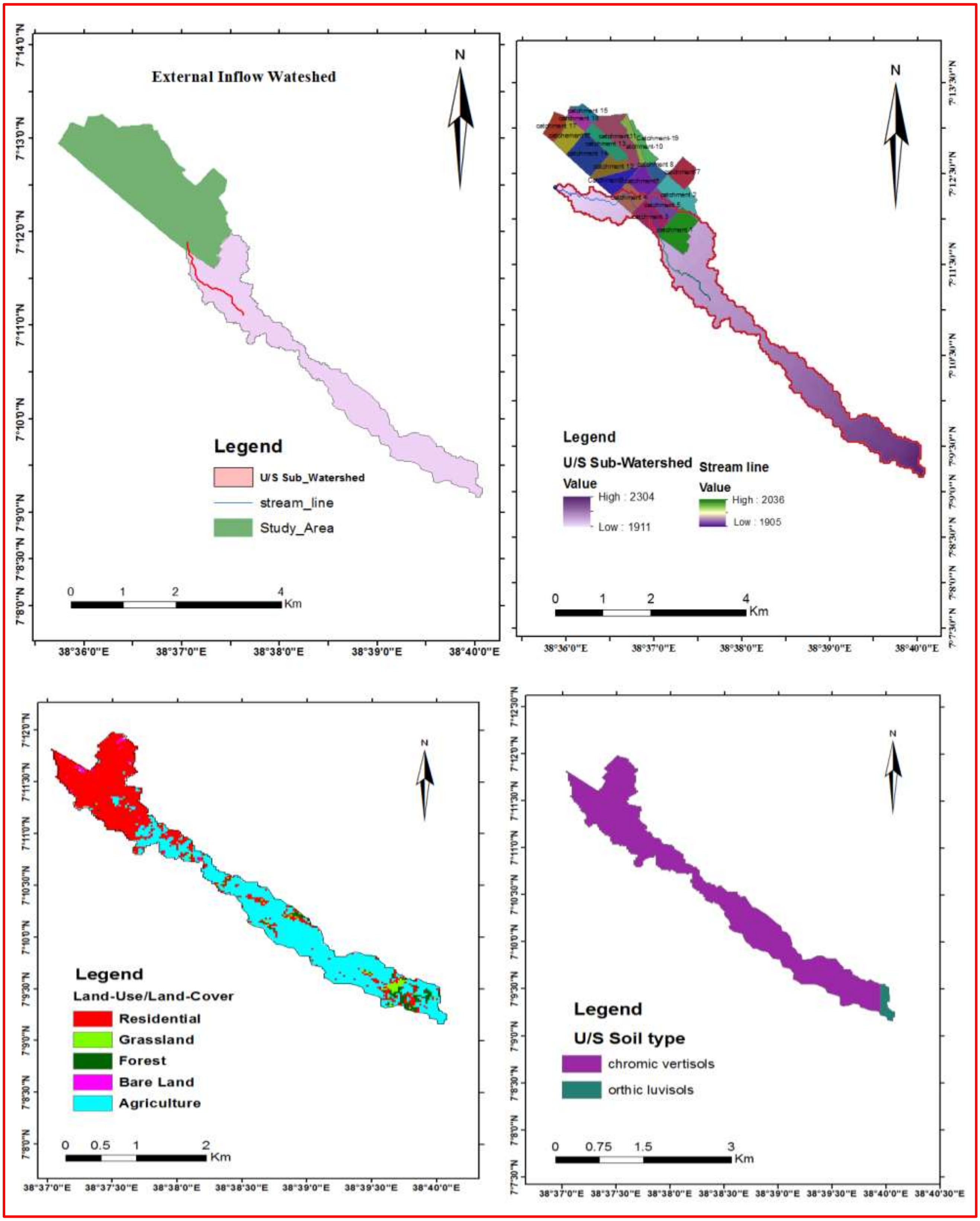


Figure 4. 7: Upstream Sub-Watershed, LU/LC, Soil type

Table 4. 4: External peak discharge

T (Year)	Ia	P(mm)	Ia/p	Q(mm)	Tc (min)	q_u(m^3/s/Km^2/mm)	q_p (m^3/s)
2	8.179	39.753	0.206	13.894	77.863	0.118	4.942
5	8.179	53.006	0.154	23.640	77.863	0.118	8.408
10	8.179	61.780	0.132	30.637	77.863	0.131	12.061
25	8.179	72.867	0.112	39.904	77.863	0.131	15.709
50	8.179	81.091	0.101	47.009	77.863	0.131	18.506
100	8.179	89.255	0.092	54.211	77.863	0.131	21.341

4.5. Performance of the Drainage System in the Study Area

Using GTZ Standards (GTZ, 2006) on stormwater drainage examining the performance of current drainage systems, coverage of the drainage system and evaluating the current conditions of the drainage system see below (Table 4.5).

Table 4. 5: GTZ standards for evaluation of drainage performance

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 deteriorate, but still sheds water
Severe	The total collapse of the USWD lines structure and barely passable

Table 4. 6: Urban drainage system condition around Dr.UrgesHospital

Code	Drainage Type/shape/Surface	Inlet coordinate		Exit coordinate		Canal Length(m)	drainage condition	%
		X	Y	X	Y			
DL1	Open/Rec/mas/Canal.	458261	796126	457723	795550	782	Light	16.9
DL2	Open/Rec/mas/Canal.	458469	795949	457935	795399	764	Sever	16.5
DL3	Open/Rec/mas/Canal.	458402	795489	458006	795831	524	Sever	11.3
	Open/Rec/mas/Canal.							
DL4	Open/Rec/mas/Canal.	458590	795842	458268	796119	425	Sever	9.2
DL5	Open/Rec/mas/Canal.	458269	796119	457734	795545	786	Good	16.9

DL6	Open/Trap/mas canal	458283	795161	457743	795537	662	V. good	14.3
DL7	Open/Trap/mas canal	457195	795981	457734	795543	695	Good	15.0

From the total drain lines, about 37 % severely degraded which, the total collapse of the drain lines structure and not pass the stormwater and 16.9% is the light, which, the configuration of geometry the drain lines deteriorate and store the stormwater. The drain lines around these sub-catchments cannot safely discharge into a nearby outlet. The drainage systems in this area blockage by the soil debris, solid waste, absence of drainage lines, and most of the drainage lines damaged. As a result, major coverage drainage systems around this location do not perform safely to dispose of the stormwater (Table 4.6).

Another 31.9% good which, no significant depressions, undulations, and deformation and 14.3% is very-good whish, Shapes of drain lines as still in original design condition. Some drain lines around this area pass the stormwater safely to the outlet junction.

Table 4. 7: Urban drainage system condition around Farma academy school

Code	Drainage Type/shape/Surface	Inlet coordinate		Exit coordinate		Canal Length (m)	drainage condition	%
		X	Y	X	Y			
DL8	Open/Rec/mas/Canal	457503	796285	457127	796631	515	Sever	13.2
DL9	Open/Rec/mas/Canal	457718	796285	457505	796282	313	Good	8.0
DL10	Open/Rec/mas/Canal	45797	795848	457501	796284	660	Sever	16.9
DL11	Open/Rec/mas/Canal	457504	796291	457197	795979	433	Sever	11.1
DL12	Open/Rec/mas/Canal	457492	796277	457194	795985	413	Good	10.6
DL13	Open/Rec/mas/Canal	457486	796271	457108	796619	512	Sever	13.1
DL14	Open/Rec/mas/Canal	457486	796580	457491	796299	390	Good	10.0
DL15	Open/Rec/mas/Canal	458000	795841	457502	796202	665	Light	17.0

From the total drain lines, about 54.3 % severely degraded which, the total collapse of the drain lines structure and not pass the stormwater and 17 % is the light, which, the configuration of geometry the drain lines deteriorate and store the stormwater (Table 4.7). The drain lines around these sub-catchments cannot safely discharge into a nearby outlet. The drainage systems in this area blockage by the soil debris, solid waste, absence of drainage lines, and most of the drainage lines damaged. Another 28.6 % good, which, no significant depressions, undulations, and deformation. As the result, most of the drainage systems around this location do not perform safely to dispose of the stormwater.

Table 4. 8: Urban drainage system condition around new Bus Station (kebele 01)

Code	Drainage Type/shape/Surface	Inlet coordinate		Exit coordinate		Canal Length(m)	drainage condition	%
		X	Y	X	Y			
DL16	Open/Rec/mas/Canal	456840	796331	456404	796692	596	V.Good	15.7
DL17	Open/Rec/mas/Canal	457118	796633	456851	796320	402	Light	10.6
DL18	Open/Rec/mas/Canal	457202	795986	456852	796321	499	V.Good	13.2
DL19	Open/Rec/mas/Canal	457335	796921	456841	796331	783	Sever	20.6
DL20	Open/Rec/mas/Canal	457323	796114	456928	796414	531	Sever	14.0
DL21	Open/Rec/mas/Canal	457275	796262	457006	796501	362	Sever	9.5
DL22	Open/Rec/mas/Canal	457286	796475	456986	796159	424	Good	11.2
DL23	Open/Rec/mas/Canal	457254	796197	457115	796048	197	Light	5.2

From (Table 4.8) the total drain lines, about 44.18 % severely degraded which, the total collapse of the drain lines structure and not pass the stormwater and 15.79 % is the light, which, the configuration of geometry the drain lines deteriorate and store the stormwater. The drain lines around these sub-catchments cannot safely discharge into a nearby outlet. The drainage systems in this area blockage by the soil debris, solid waste, absence of drainage lines, and most of the drainage lines damaged. As a result, major coverage drainage systems around this location do not perform safe dispose of the stormwater. Then other 26.8% good which, no significant depressions, undulations, and deformation and 28.86 % is very-good which, Shapes of drain lines as still in original design condition. Some drain lines around the area pass the stormwater safely to the outlet junction.

Table 4. 9: Urban drainage system condition around Malka Oda Hospital

Code	Drainage Type/shape/Surface	Inlet coordinate		Exit coordinate		Canal Length(m)	drainage condition	%
		X	Y	X	Y			
DL24	Open/Rec/mas/Canal	457496	796982	457095	798002	1085	V. Good	25.0
DL25	Open/Rec/mas/Canal	457504	796993	457114	797988	1064	V. Good	24.5
DL26	Open/Rec/mas/Canal	457762	797086	457506	796995	274	Light	6.3
DL27	Open/Rec/mas/Canal	454960	796979	457110	796624	523	Sever	12.0

DL28	Open/Rec/mas/Canal	457764	796575	457498	796977	496	Sever	11.4
DL29	Open/Rec/mas/Canal	457699	797034	457494	796975	208	Light	4.8
DL30	Open/Rec/mas/Canal	457946	796782	457696	797036	360	Good	8.3
DL31	Open/Rec/mas/Canal	457492	796988	457170	796910	333	Sever	7.7

From (Table 4.9) above the total drains, about 61.6% is severely degraded, 22% is light, and 16% good. That is the flood generated around this location cannot safely be discharged into the nearby outlet. On the other hand, run-over road surfaces cause flood hazards. The drainage system around this has been damaged by erosion, Siltation, ponding, and overflow. The major part of drainage systems around Malka oda hospital shows severely degraded.

Table 4. 10: Urban drainage system condition around Alelu primary school

Code	Drainage Type/ Shape/ Surface	Inlet coordinate		Exit coordinate		Canal Length(m)	drainage condition	%
		X	Y	X	Y			
DL32	Open/Rec/mas/Canal	456365	797833	455915	798148	566	Good	3.1
DL33	Open/Rec/mas/Canal	456222	797659	455845	797951	478	G00d	2.6
DL34	Open/Rec/mas/Canal	457165	796906	456387	797818	1237	Good	6.8
DL35	Open/Rec/mas/Canal	457086	797993	456390	797820	748	Light	4.1
DL36	Open/Rec/mas/Canal	456385	797812	456234	797641	225	Good	1.2
DL37	Open/Rec/mas/Canal	456900	797058	456225	797646	885	Good	4.9
DL38	Open/Rec/mas/Canal	456228	797628	455840	797178	596	Good	3.3
DL39	Open/Rec/mas/Canal	456223	797641	455830	797221	588	Light	3.3
DL40	Open/Rec/mas/Canal	456477	797430	456064	796988	602	Good	3.3
DL41	Open/Rec/mas/Canal	456718	797200	455811	797928	10154	Sever	56.2
DL42	Open/Rec/mas/Canal	457112	797111	455871	798140	1604	Light	8.9
DL43	Open/Rec/mas/Canal	456476	797946	456218	797651	390	Good	2.2

From (Table 4.10) total drain lines, about 56 % severely degraded which, the total collapse of the drain lines structure and not pass the stormwater and 16.3% is the light, which, the drain lines deteriorate and store the stormwater. The drain lines around these sub-catchments cannot safely discharge into a nearby outlet. The drainage systems in this area blockage by the soil debris, solid waste, absence of drainage lines, and most of the drainage lines damaged. Another 27 % good which, no significant depressions, undulations, and deformation. As a result, major coverage drainage systems around this location do not perform safe dispose of the stormwater.

Table 4. 11: Urban drainage system condition around Alelu market

Code	Drainage Type/ shape/ Surface	Inlet coordinate		Exit coordinate		Canal Length(m)	drainage condition	%
		X	Y	X	Y			
DL44	Open/Rect/Con/Canal	455564	797435	1911	455355	283	V.Good	9.3
DL45	Open/Rec/mas/Canal	455581	798041	1906	455356	478	Sever	15.7
DL46	Open/Rec/mas/Canal	455845	797939	1910	455563	585	Good	19.2
DL47	Open/Rect/Con/Canal	455850	797177	1921	455561	374	V.Good	12.3
DL48	Open/Rec/mas/Canal	455843	797945	1910	455560	587	Sever	19.3
DL49	Open/Rec/mas/Canal	455996	797395	1920	455737	351	Sever	11.5
DL50	Open/Rec/mas/Canal	456120	797530	1921	455798	389	Light	12.8

As (Table 4.11), from the total drains, about 46.5% is severely degraded, 12.8% is light, 19.2% good and 6% is very good. That is the flood generated around this location cannot safely discharge into the nearby outlet. On the other hand, run-over road surfaces cause a flood. The drainage system around this has been damaged by erosion and overflow.

Table 4. 12: Urban drainage system condition around Shashemene Stadium

Code	Drainage Type/shape/Surface	Initial coordinate		Terminal coordinate		Canal Length (m)	drainage condition	%
		X	Y	X	Y			
DL51	Open/Rectangular/con	456339	796747	455840	797185	673	V.Good	15.52
DL52	Open/Rectangular/mas	457157	796907	456731	797186	613	Light	14.14
DL53	Open/Rectangular/mas	456730	797180	456348	796730	592	Light	13.65
DL54	Open/Rectangular/mas	456403	796691	456344	796736	76	V. Good	1.75
DL55	Open/Rectangular/mas	457360	796922	456404	796691	998	Sever	23.02
DL56	Open/Rectangular/mas	457152	796904	456390	796708	788	Sever	18.17
DL57	Open/Rectangular/mas	456730	797196	456328	796751	596	Good	13.75

From (Table 4.12) the total drain lines, about 44.2 % severely degraded which, the total collapse of the drain lines structure and not pass the stormwater and 27.3 % is the light, which, the configuration of geometry the drain lines deteriorate and store the stormwater. The drain lines around these sub-catchments cannot safely discharge into a nearby outlet. The drainage systems in this area blockage by the soil debris, solid waste, absence of drainage lines, and most of the drainage lines damaged. Another 13.7 % good which, no significant depressions, undulations, and deformation and 17.3 % is very-good whish, Shapes of drain lines as still in original design condition. Some drain lines around this area pass the stormwater safely to the outlet junction. Most of the drainage systems around this location do not perform safely dispose of the stormwater to nearby outlet junction that, cause flooding and drainage system damaged by erosion and overflow.

4.6. Water Level at Links and Junctions in the Drainage Networks

4.6.1. Water Depth and Flow In The Links

- **Around Dr.Urgesa Hospital**

The link and nodes around Dr. Urges Hospital the simulation result indicates in junctions J2, J3 and J4 are flooded due to insufficient design water depth but only at junction J1 sufficient water depth. The link C2 (i.e. J3-J2) and C4 (i.e. J2-J4) were insufficient to carry capacity of the runoff from sub-catchment upstream and downstream whereas, C1 (i.e. J1-J2) sufficient and insufficient to carry capacity of the runoff from sub-catchment upstream and downstream of the canal respectively as shown below (Figure 4.8). Consequently, the runoff overflow to the roadway, drain lines, and the surface of the downstream sub-catchments.

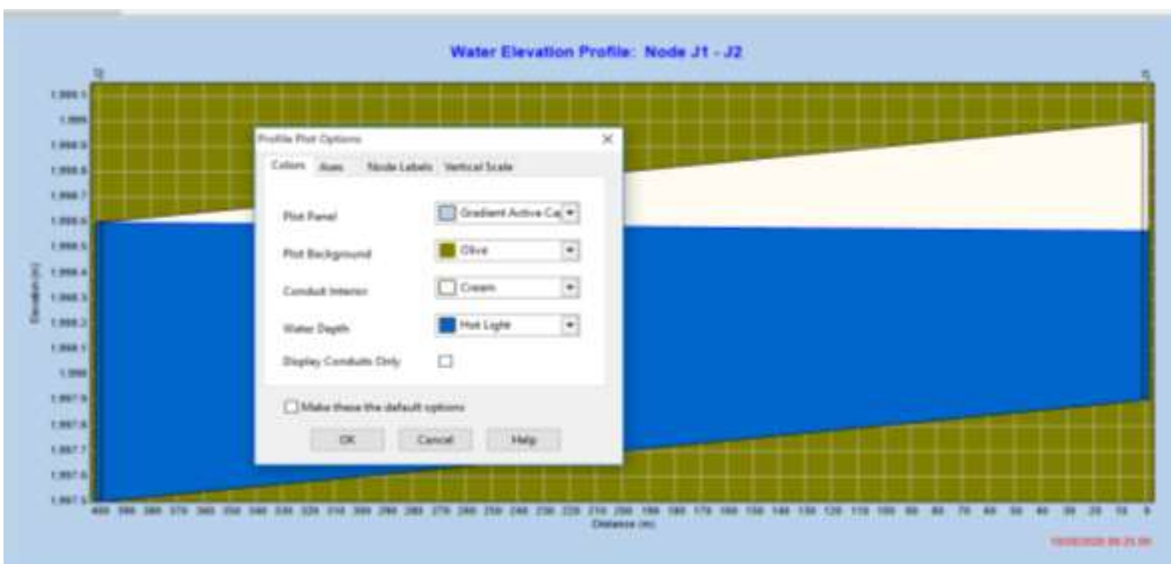


Figure 4. 8: Water Elevation Profile at Dr.Urgesa Hospital

- **Farm Academy School**

The drainage canal around Farm school the result simulation shows that except junction J8, the J6, J7, and J17 are flooded due to insufficient design water depth and holding capacity the generated runoff from upstream sub-catchment. The link C6 (i.e. J6-J7) and C14 (i.e. J7-J17) were insufficient to carry the capacity of the runoff from the sub-catchment upstream and downstream but, C5 (i.e. J6-J8) as shown below (Figure 4.9). Consequently, the runoff overflow to the roadway, deterioration drain lines, siltation, and surface flooding of the downstream sub-catchments.


Water Elevation Profile: Node J8 - J7
J7
J8
Elevation (m)
Distance (m)
10/20/2020 00:55:00


Water Elevation Profile: Node J7 - J17
J17
J7
Elevation (m)
Distance (m)
10/20/2020 03:00:00

Figure 4. 9: Water Elevation Profile at Farm Academy School

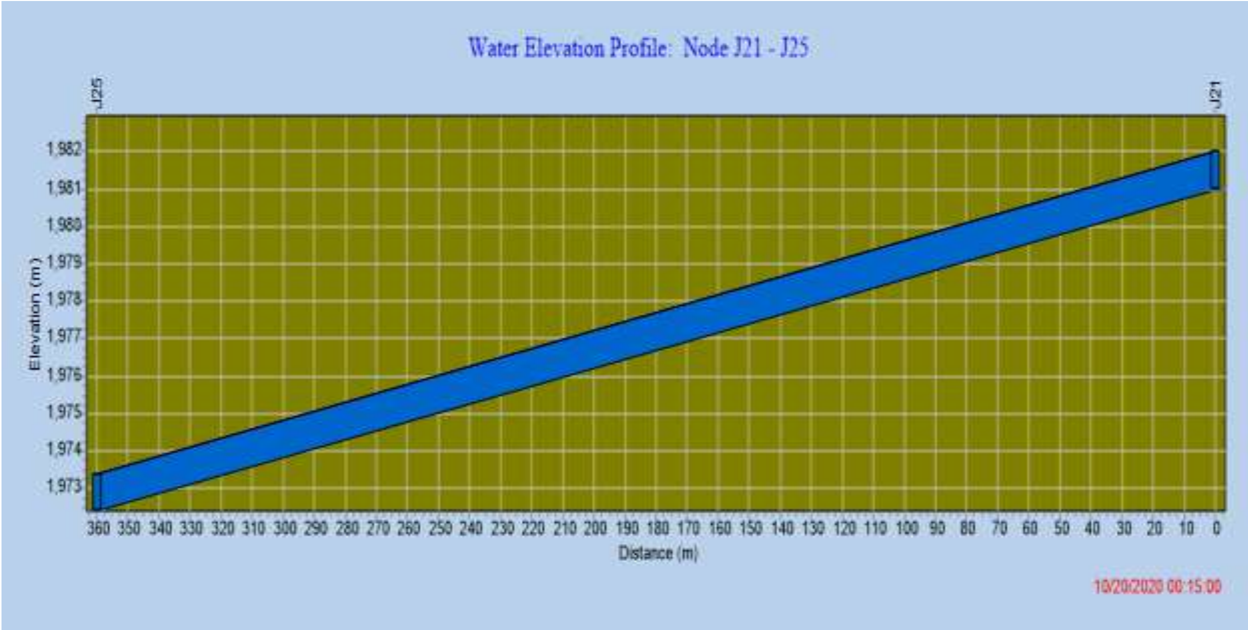
- **Around New Bus Station (Kebele 01)**

The drainage canal located new bus station as the simulation result indicates Junction J14, J34 and J35 are flooded due to insufficient design water depth and holding capacity the generated runoff from upstream sub-catchment. The link C12 (i.e. J4-J14) and C13 (i.e. J14-J17), C25 (i.e. J14-J34), and C26 (i.e. J34-J35) were insufficient to carry the capacity of the runoff from sub-catchment upstream and downstream as shown below (Figure 12). Consequently, the runoff overflow to the roadway, deterioration drain lines, siltation, and surface flooding of the downstream sub-catchments.

Figure 4. 10: Water Elevation Profile at Around New Bus Station

- **Around Malka oda Hospital**

Both of the links and junctions are C17 (J21-J25) and C18 (J25-J26) the simulation result indicates insufficient to carry the generated runoff but, C54 (J30-J62). This causes overflow, siltation deterioration and the collapse of drain lines, and surface flooding to downstream Sub-catchment has happened in this area as shown in (Figure 4.11).



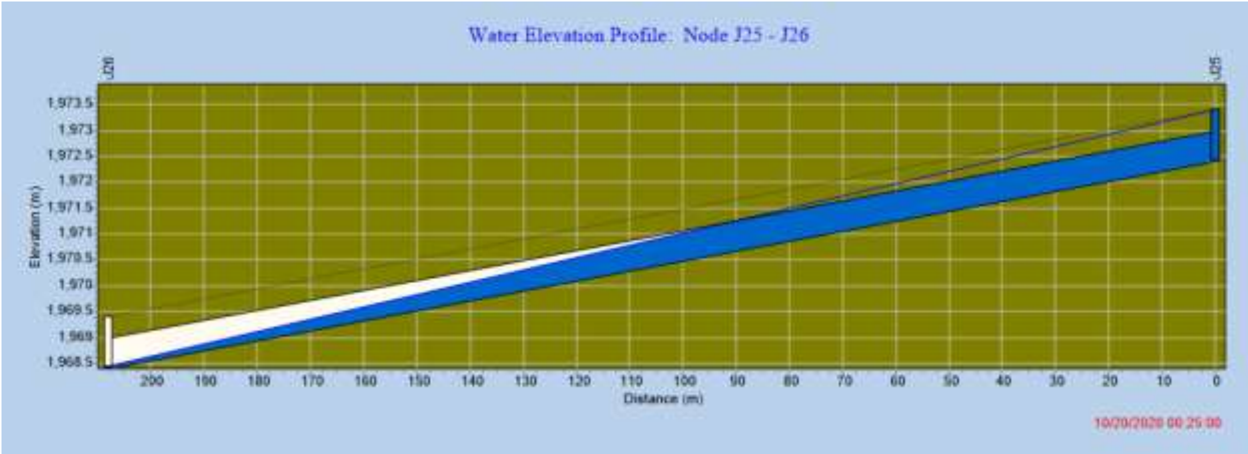


Figure 4. 11: Water Elevation Profile at Around Malka oda Hospital

Where, Water depth , Conduit interior

4.6.2. Water Level and Flood Level in the Junctions

- **Drainage Around new Bus Station**

The designed water level of the drainage canal of junctions 14, and 17 were 0.85 and 0.6 m respectively. Also, the maximum flooding level of each junction is greater than the designed water level of drainage canal depth. So that the designed canal depth of junction 14, and 17 around the new bus station is insufficient and the simulation result shows that there is the overflow of the water around the area (Figure 4.12). The cross-section of this canal narrows results overflows.

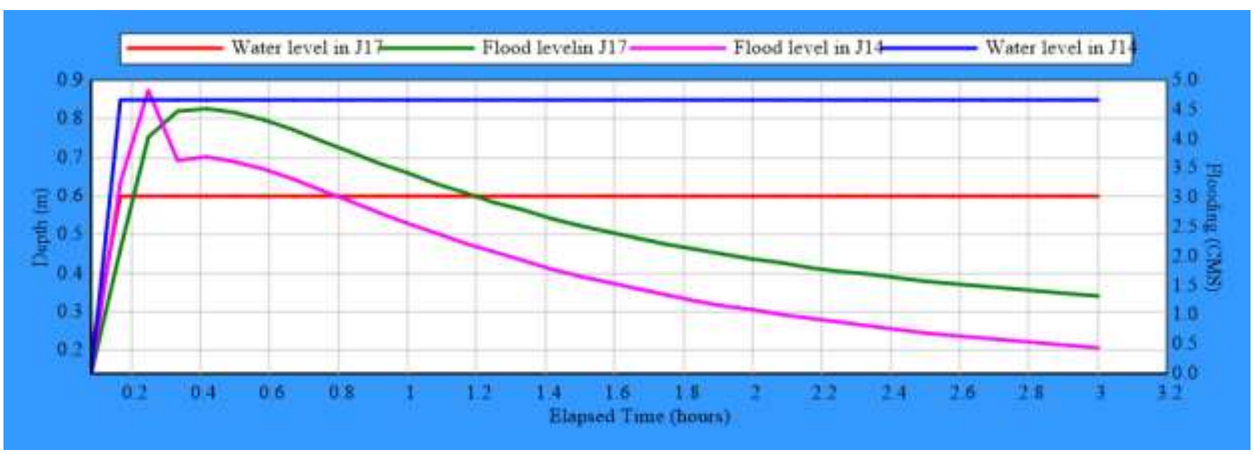


Figure 4. 12: Designed Water level and flood in the junction of the around the new bus station

- **Around Farma Academy School**

The drainage canal at J5 and J7 were 1 and 0.6m, and the designed canal depth J5 was insufficient to carry the capacity of the storm but, sufficient for J7. The simulation result shows that there was no flood occurrence around at the J7, the size of this junction sufficient to holding capacity of the flood but, links. (Figure 4.13).

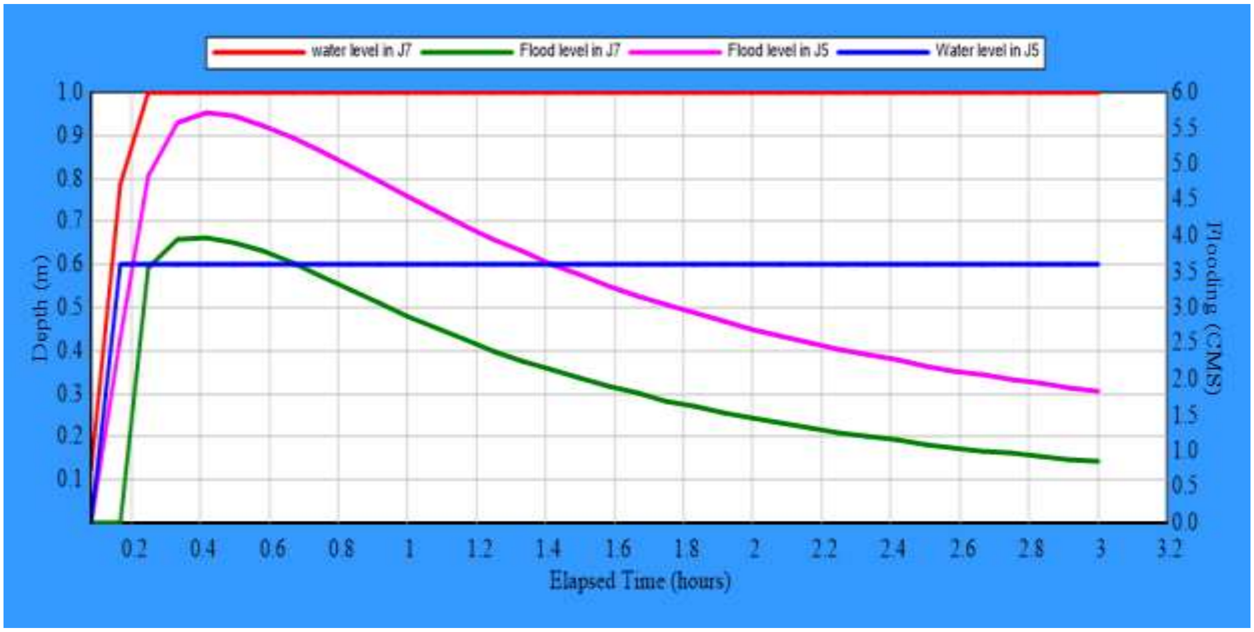
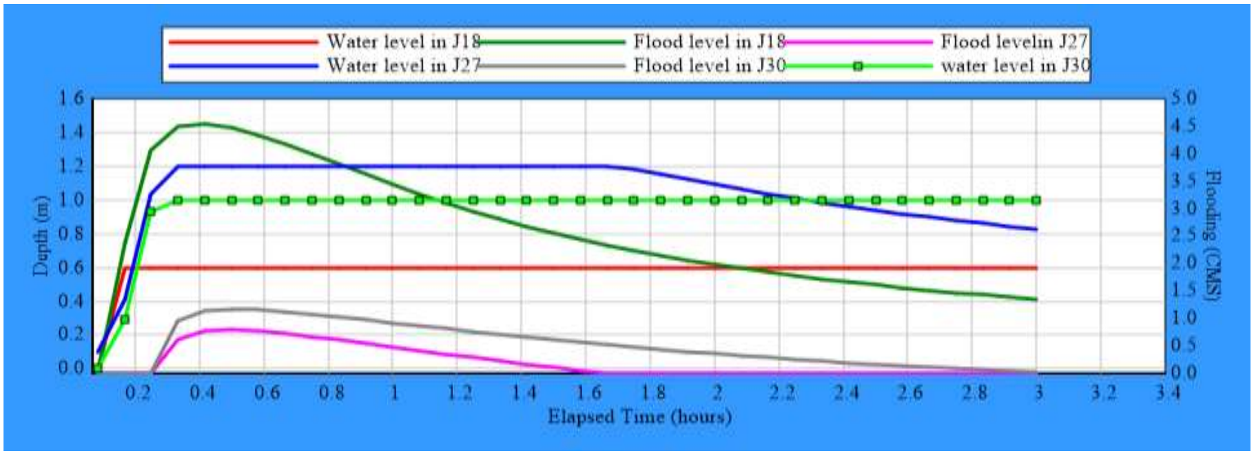


Figure 4. 13: Designed Water level and flood injunction of Farma academy and Hamilton Hotel

- **Drainage Malka Oda Hospital**

Drainage around Malka Oda Hospital, the maximum designed water level of the drainage canal of J21, J26, and J25 were 1m, 1m, and 1.2 m respectively and the flooding level attained with 3hrs rainfall intensity of each junction was less than the maximum designed water level. The result of the simulation indicates, the designed canal depth was insufficient and shows that there is a flood at this location (Figure 4.14).



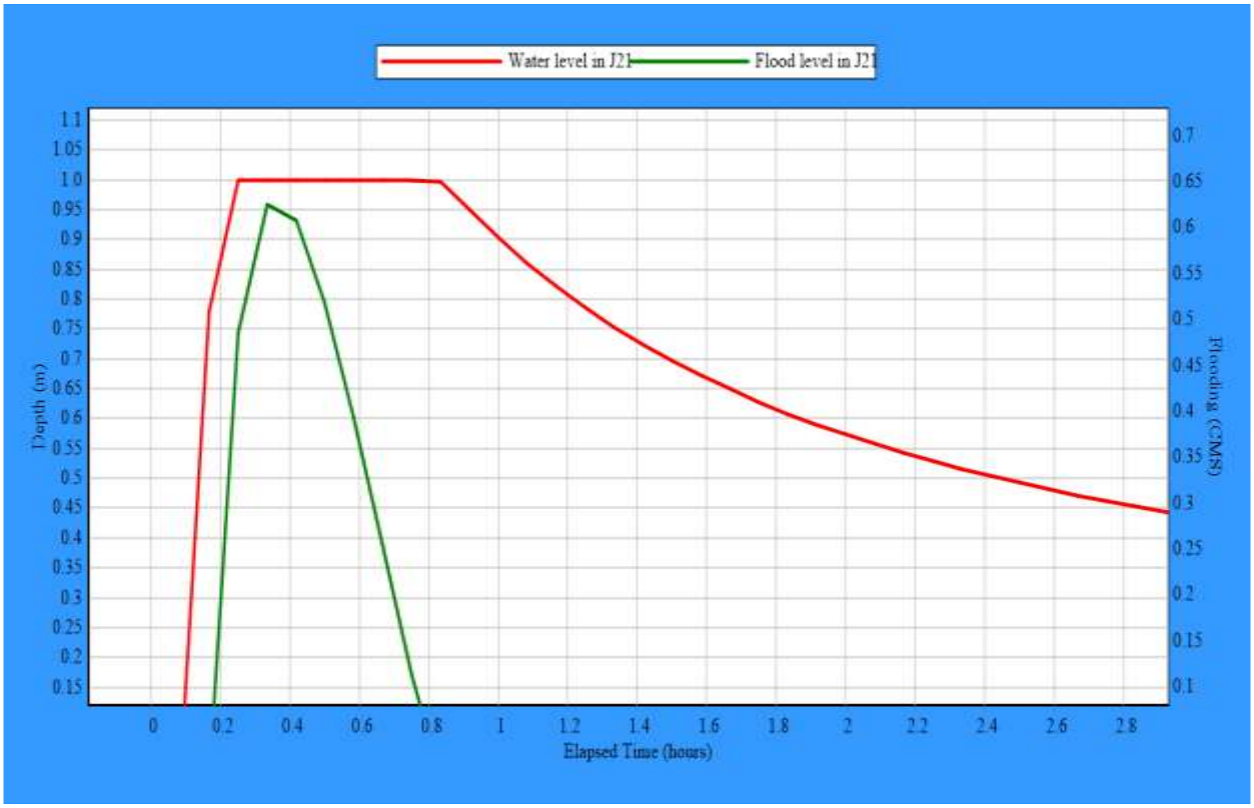


Figure 4. 14: Designed Water level and flood injunction of Drainage around Nock

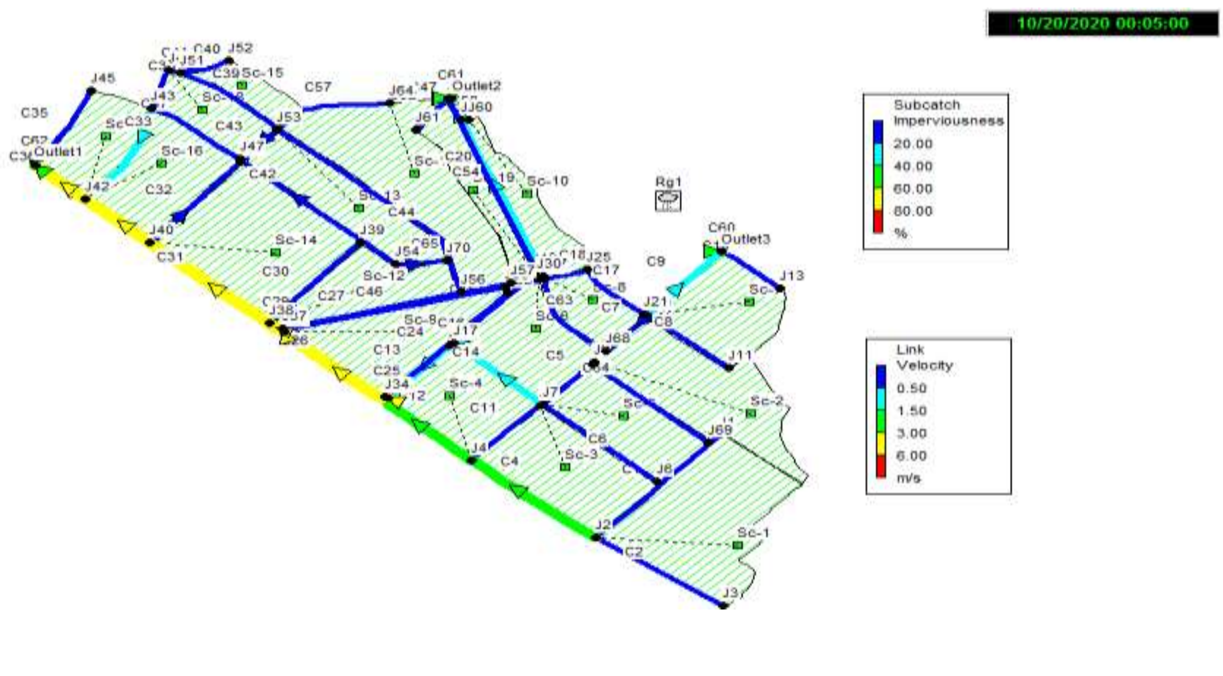


Figure 4.15: Catchment area of SWMM results

5. CONCLUSIONS AND RECOMMENDATIONS

5.1. Conclusions

This study analyzed storm drainage systems of Shashemene Town, by using SCS and rational method, hydraulic drain lines by Manning's equation of Sub-catchments of the study area. According to the result of this study, the most coverage of drain lines study area are severely degraded and inadequate to convey the peak discharge to the outlet due to improper connection of drainage systems, absence of drain lines, deposited solid waste, insufficient slope providing that leads siltation and ponding in drain lines, and increasing impervious surface area. For this reason, storm drainage weakens their facility to convey the runoff properly.

To evaluate the hydraulic capacity of the existing storm drainage system checked by comparing the estimated runoff with the existing drainage capacity. The peak discharge estimated from the rational method based on frequency analysis of the NMA daily maximum rainfall data and design flood with a return period of 10 years developed the IDF curve of the town, land-use, and catchment area. The main existing drainage system problem in the town is not constructed based on hydrological and hydraulic. Manning's equation was used to analysis of the adequacy of existing drainage structures in the sub-catchments. These result shows 78.9% of the drainage systems of sub-catchments include in the Awash-Alelu Sub-town existing storm drainage systems insufficient carry capacity of the storm and a maximum flood occurs are Sc-1, Sc-2, Sc-3, Sc-4, Sc-5, Sc-6, Sc-7, Sc-10, Sc-11, Sc-13, Sc-14, Sc-15, Sc-17, and Sc-19.

5.2. Recommendations

Based on the field visit, computation, and analysis of hydrological and hydraulic of this thesis appropriate that on Shashemene Town inadequacy of storm drainage structures have had a serious negative impact on the community and road structure. To avoid these problems, the following

- ✓ Addition of new drainage lines at the proper place of Dr.Urgesa hospital, Malka oda hospital and Farma primary school e.t.c to reduce peak runoff and flooding hazards happened in the area.
- ✓ Establish well-connected drainage lines and use stormwater drainage lines as it were their outfall.
- ✓ Constructed at upstream of study area retaining wall to mitigate flooding by making detention and diverting to downstream safely.
- ✓ Increasing the cross-section or depth of drain structure around New bus station, DH Hotel, Radiet Hotel, Kella, Alelu market and mainly the drain lines across the slope (i.e. southeast to the north west).

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Appendixes

Appendix 2: kn value for different sample size used for outlier

S/ size (n)	Kn	S/ size (n)	Kn	S/ size (n)	Kn	S/ size (n)	kn
10	2.036	24	2.467	38	2.661	60	2.837
11	2.088	25	2.486	39	2.671	65	2.866
12	2.134	26	2.502	40	2.682	70	2.893
13	2.175	27	2.519	41	2.692	75	2.917
14	2.213	28	2.534	42	2.7	80	2.94
15	2.247	29	2.549	43	2.71	85	2.961
16	2.279	30	2.563	44	2.719	90	2.981
17	2.309	31	2.577	45	2.727	95	3
18	2.335	32	2.591	46	2.736	100	3.017
19	2.361	33	2.604	47	2.744	110	3.049
20	2.385	34	2.616	48	2.753	120	3.078
21	2.408	35	2.628	49	2.76	130	3.104
22	2.429	36	2.639	50	2.768	140	3.129 ,
23	2.448	37	2.65	55	2.804		

Source: (Chow, 1988)

Appendix 3: Max rainfall and its statistical computation

Year	Max.RF.(x)	Z/y/=logx	$(Z-\bar{z})$	$(Z-\bar{z})^2$	$(Z-\bar{z})^3$
1987	60.2	1.8	0.1751	0.0307	0.0054
1988	35.0	1.5	-0.0604	0.0037	-0.0002
1989	43.9	1.6	0.0381	0.0015	0.0001
1990	62.2	1.8	0.1892	0.0358	0.0068
1991	50.4	1.7	0.0980	0.0096	0.0009
1992	47.1	1.7	0.0686	0.0047	0.0003
1993	33.0	1.5	-0.0857	0.0074	-0.0006
1994	41.6	1.6	0.0146	0.0002	0.0000
1995	44.0	1.6	0.0392	0.0015	0.0001
1996	75.4	1.9	0.2728	0.0744	0.0203
1997	70.6	1.8	0.2447	0.0599	0.0146
1998	58.6	1.8	0.1639	0.0268	0.0044
1999	44.0	1.6	0.0392	0.0015	0.0001
2000	52.8	1.7	0.1184	0.0140	0.0017
2001	38.1	1.6	-0.0238	0.0006	0.0000
2002	20.9	1.3	-0.2837	0.0805	-0.0228

2003	30.0	1.5	-0.1271	0.0162	-0.0021
2004	40.8	1.6	0.0064	0.0000	0.0000
2005	37.4	1.6	-0.0314	0.0010	0.0000
2006	44.9	1.7	0.0478	0.0023	0.0001
2007	37.4	1.6	-0.0314	0.0010	0.0000
2008	38.0	1.6	-0.0245	0.0006	0.0000
2009	22.4	1.4	-0.2532	0.0641	-0.0162
2010	38.9	1.6	-0.0144	0.0002	0.0000
2011	38.9	1.6	-0.0144	0.0002	0.0000
2012	25.0	1.4	-0.2060	0.0425	-0.0087
2013	41.1	1.6	0.0100	0.0001	0.0000
2014	40.7	1.6	0.0052	0.0000	0.0000
2015	27.0	1.4	-0.1729	0.0299	-0.0052
2016	37.7	1.6	-0.0279	0.0008	0.0000
2017	26.9	1.4	-0.1741	0.0303	-0.0053
Sum	1304.8	49.73215	0.00000	0.54189	-0.00660
Mean	z̄	1.6042628			
Stdev	ε_(n-1)	0.1322135			
Skew	Cs	-0.098203			

Appendix 4: Kz=F (Cs, T) for use in Log-Pearson Type III Distribution Used for R24

Coef. of skew, C_s	Return Period T in years						
	2	10	25	50	100	200	1000
3.0	-0.396	1.180	2.278	3.152	4.051	4.970	7.250
2.5	-0.360	1.250	2.262	3.048	3.845	4.652	6.600
2.2	-0.330	1.284	2.240	2.970	3.705	4.444	6.200
2.0	-0.307	1.302	2.219	2.912	3.605	4.298	5.910
1.8	-0.282	1.318	2.193	2.848	3.499	4.147	5.660
1.6	-0.254	1.329	2.163	2.780	3.388	3.990	5.390
1.4	-0.225	1.337	2.128	2.706	3.271	3.828	5.110
1.2	-0.195	1.340	2.087	2.626	3.149	3.661	4.820
1.0	-0.164	1.340	2.043	2.542	3.022	3.489	4.540
0.9	-0.148	1.339	2.018	2.498	2.957	3.401	4.395
0.8	-0.132	1.336	1.993	2.453	2.891	3.312	4.250
0.7	-0.116	1.333	1.967	2.407	2.824	3.223	4.105
0.6	-0.099	1.328	1.939	2.359	2.755	3.132	3.960
0.5	-0.083	1.323	1.910	2.311	2.686	3.041	3.815
0.4	-0.066	1.317	1.880	2.261	2.615	2.949	3.670
0.3	-0.050	1.309	1.849	2.211	2.544	2.856	3.525
0.2	-0.033	1.301	1.818	2.159	2.472	2.763	3.380
0.1	-0.017	1.292	1.785	2.107	2.400	2.670	3.235
0.0	**0.000**	**1.282**	**1.751**	**2.054**	**2.326**	**2.576**	**3.090**
-0.1	0.017	1.270	1.716	2.000	2.252	2.482	2.950
-0.2	0.033	1.258	1.680	1.945	2.178	2.388	2.810
-0.3	0.050	1.245	1.643	1.890	2.104	2.294	2.675
-0.4	0.066	1.231	1.606	1.834	2.029	2.201	2.540
-0.5	0.083	1.216	1.567	1.777	1.955	2.108	2.400
-0.6	0.099	1.200	1.528	1.720	1.880	2.016	2.275
-0.7	0.116	1.183	1.488	1.663	1.806	1.926	2.150
-0.8	0.132	1.166	1.448	1.606	1.733	1.837	2.035
-0.9	0.148	1.147	1.407	1.549	1.660	1.749	1.910
-1.0	0.164	1.128	1.366	1.492	1.588	1.664	1.800
-1.4	0.225	1.041	1.198	1.270	1.318	1.351	1.465
-1.8	0.282	0.945	1.035	1.069	1.087	1.097	1.130
-2.2	0.330	0.844	0.888	0.900	0.905	0.907	0.910
-3.0	0.396	0.660	0.666	0.666	0.667	0.667	0.668

Source: (Chow, 1988)

Appendix 5: Reduce mean $\overline{y_n}$ in Gumbel`s extreme value distribution=sample size

N	0	1	2	3	4	5	6	7	8	9
10	0.4952	0.4996	0.5035	0.5070	0.5100	0.5128	0.5157	0.5181	0.5202	0.5220
20	0.5236	0.5252	0.5268	0.5283	0.5296	0.5309	0.5320	0.5332	0.5343	0.5353
30	0.5362	0.5371	0.5380	0.5388	0.5396	0.5402	0.5410	0.5418	0.5424	0.5430
40	0.5436	0.5442	0.5448	0.5453	0.5458	0.5463	0.5468	0.5473	0.5477	0.5481
50	0.5485	0.5489	0.5493	0.5497	0.5501	0.5504	0.5508	0.5511	0.5515	0.5518
60	0.5521	0.5524	0.5527	0.5530	0.5533	0.5535	0.5538	0.5540	0.5543	0.5545
70	0.5548	0.5550	0.5552	0.5555	0.5557	0.5559	0.5561	0.5563	0.5565	0.5567
80	0.5569	0.5570	0.5572	0.5574	0.5576	0.5578	0.5580	0.5581	0.5583	0.5585
90	0.5586	0.5587	0.5589	0.5591	0.5592	0.5593	0.5595	0.5596	0.5598	0.5599
100	0.5600									

Appendix 6: Reduced Standard Deviation Sn in Gumbel`s extreme value distribution

N	0	1	2	3	4	5	6	7	8	9
10	0.9496	0.9676	0.9833	0.9971	1.0095	1.0206	1.0316	1.0411	1.0493	1.0565
20	1.0628	1.0696	1.0754	1.0811	1.0864	1.0915	1.0961	1.1004	1.1047	1.1086
30	1.1124	1.1159	1.1193	1.1226	1.1255	1.1285	1.1313	1.1339	1.1363	1.1388
40	1.1413	1.1436	1.1458	1.1480	1.1499	1.1519	1.1538	1.1557	1.1574	1.1590
50	1.1607	1.1623	1.1638	1.1658	1.1667	1.1681	1.1696	1.1708	1.1721	1.1734
60	1.1747	1.1759	1.1770	1.1782	1.1793	1.1803	1.1814	1.1824	1.1834	1.1844
70	1.1854	1.1863	1.1873	1.1881	1.1890	1.1898	1.1906	1.1915	1.1923	1.1930
80	1.1938	1.1945	1.1953	1.1959	1.1967	1.1973	1.1980	1.1987	1.1994	1.2001
90	1.2007	1.2013	1.2020	1.2026	1.2032	1.2038	1.2044	1.2049	1.2055	1.2060
100	1.2065									

Appendix 7: Annually average and Cumulative rainfall.

Year	Annual Average Station Rainfall					Cumulative Station Rainfall				
	Hawasa	Kuyera	W.genet	Shasheman	A.G.S	C.G.S	Hawassa	Kuyera	W.Genet	Shashemene
1987	959	1051	959	1051	990	990	959	1051	959	1051
1988	957	926	957	920	947	1936	1916	1978	1916	1971
1989	1031	944	1259	1076	1078	3014	2946	2922	3175	3047
1990	757	909	887	957	851	3865	3703	3831	4061	4003
1991	867	767	1192	594	942	4807	4570	4598	5254	4597
1992	962	1058	1283	907	1101	5908	5532	5656	6537	5505
1993	928	938	1152	992	1006	6914	6461	6593	7689	6496
1994	862	943	1266	755	1023	7938	7322	7536	8955	7251
1995	1004	1045	1015	853	1021	8959	8326	8581	9970	8104
1996	1189	1004	1226	1198	1140	10099	9516	9585	11196	9301
1997	1054	1065	975	1188	1031	11130	10570	10650	12171	10490
1998	1148	1042	1304	1104	1165	12295	11718	11692	13475	11593
1999	809	701	932	813	814	13109	12527	12393	14407	12406
2000	822	714	913	986	816	13925	13348	13106	15320	13392
2001	1022	902	1283	763	1069	14993	14370	14008	16603	14155
2002	919	776	906	352	867	15860	15289	14784	17508	14635
2003	889	782	698	600	789	16650	16178	15565	18206	15454
2004	898	839	1013	660	916	17566	17076	16404	19219	16354
2005	1003	930	1105	633	1012	18579	18079	17334	20323	17218
2006	1198	1238	1266	615	1234	19812	19276	18572	21589	18057
2007	1153	939	1117	589	1069	20882	20429	19511	22706	18860
2008	915	948	836	570	900	21781	21344	20459	23541	19637
2009	704	720	698	401	707	22489	22048	21179	24239	20184
2010	1032	1185	1420	455	1213	23701	23080	22363	25660	20805
2011	923	698	1175	396	932	24633	24003	23062	26835	21346
2012	785	1088	785	585	886	25519	24788	24150	27620	22144
2013	968	845	1120	801	978	26497	25757	24995	28739	23236
2014	1155	596	610	527	787	27284	26911	25591	29349	23955
2015	686	445	344	410	492	27775	27597	26036	29693	24514
2016	1092	468	1004	509	854	28630	28689	26504	30697	25208
2017	1183	518	1192	546	964	29594	29872	27021	31889	25953

Appendix 8: Annual and monthly maximum rainfall (1987-2017)

Year	January	February	March	April	May	June	July	August	September	October	November	December	Annual max
1987	6.8	32.8	24.6	28.8	60.2	16.8	22.3	36.7	31.2	32.6	0.2	2.9	60.2
1988	13.7	26.3	15.8	35.0	12.2	30.0	17.4	16.2	26.3	25.6	0.0	5.2	35.0
1989	31.6	24.9	22.5	43.9	10.6	15.3	8.9	28.8	28.0	13.5	6.4	25.6	43.9
1990	2.8	34.0	31.6	62.2	6.8	15.3	32.5	24.3	13.7	12.9	5.8	0.0	62.2
1991	2.8	15.8	23.2	6.4	13.5	8.6	50.4	15.9	16.6	5.9	0.0	5.9	50.4
1992	18.9	16.5	9.3	47.1	25.5	23.0	29.0	27.2	22.4	6.3	21.8	41.5	47.1
1993	33.0	20.5	0.0	22.5	15.7	18.2	22.3	15.0	18.2	17.0	0.0	0.0	33.0
1994	0.0	0.0	11.3	41.6	20.8	10.0	32.2	20.0	31.1	2.6	7.3	9.3	41.6
1995	0.0	5.7	34.8	31.3	17.2	22.0	44.0	20.6	27.5	17.8	5.2	12.1	44.0
1996	7.7	0.0	37.4	20.5	30.3	30.7	75.4	10.4	42.3	42.0	14.4	7.9	75.4
1997	22.0	0.0	39.6	48.9	39.3	60.6	24.0	22.2	11.5	55.3	70.6	2.2	70.6
1998	27.4	31.5	5.0	25.0	25.3	24.6	15.3	12.0	15.2	58.6	5.0	0.0	58.6
1999	10.0	0.0	11.5	8.4	44.0	28.9	28.4	2.4	35.6	22.3	2.3	4.2	44.0
2000	0.0	0.0	2.1	32.2	42.0	9.5	21.1	8.4	15.3	52.8	4.8	5.2	52.8
2001	0.0	3.4	28.2	8.2	22.0	38.1	10.6	13.2	18.4	12.6	0.7	2.7	38.1
2002	20.9	2.4	8.0	10.9	7.1	10.0	6.0	10.0	7.0	5.0	0.0	10.0	20.9
2003	4.3	10.0	10.0	10.0	8.0	30.0	10.5	30.0	30.0	3.0	9.3	10.5	30.0
2004	10.0	15.3	6.3	40.8	18.3	24.2	26.2	40.8	6.0	15.2	5.3	5.2	40.8
2005	18.3	3.2	8.3	37.4	8.3	4.3	15.2	14.0	8.3	9.0	9.3	23.0	37.4
2006	0.0	13.0	18.2	44.9	20.3	5.2	18.3	9.3	20.3	8.2	8.3	4.2	44.9
2007	2.0	4.2	15.3	37.4	15.0	18.3	8.2	8.2	18.3	5.3	1.2	0.0	37.4
2008	3.0	3.2	1.0	6.2	5.3	13.0	13.2	15.3	7.3	25.0	38.0	0.0	38.0
2009	9.3	4.0	12.3	10.2	13.0	9.0	22.4	8.3	5.2	8.0	6.2	8.2	22.4
2010	7.3	9.2	15.3	16.3	20.0	10.2	8.2	8.3	6.0	3.0	1.0	38.9	38.9
2011	2.2	2.0	5.3	15.3	23.0	13.0	38.9	12.0	5.0	1.0	18.0	6.2	38.9
2012	0.0	0.0	3.2	15.2	25.0	10.0	14.1	9.6	8.9	3.6	7.0	4.4	25.0
2013	13.7	1.1	13.9	9.5	13.8	13.6	25.5	41.1	17.5	4.0	18.4	3.7	41.1
2014	0.0	9.0	7.8	9.0	18.0	40.7	5.2	18.3	4.2	23.3	3.0	0.0	40.7
2015	1.0	1.3	13.2	27.0	27.0	8.0	27.0	16.0	4.0	2.2	3.2	3.7	27.0
2016	18.2	0.0	13.2	37.7	12.0	15.0	8.0	3.0	4.0	18.0	23.2	1.0	37.7
2017	0.0	14.6	13.0	10.0	26.9	6.5	26.9	10.6	16.3	9.5	2.6	1.8	26.9
Monthly max.	33.0	34.0	39.6	62.2	60.2	60.6	75.4	41.1	42.3	58.6	70.6	41.5	75.4

Appendix 9: Goodness fit easy fit software result

S.no.	Distribution	Kolmogorov Smirnov		Anderson Darling		Chi-Squared	
		Statistic	Rank	Statistic	Rank	Statistic	Rank
4	Cauchy	0.09316	1	0.38615	4	0.47417	26
7	Dagum	0.11397	2	0.33394	1	0.24848	23
2	Burr	0.11583	3	0.34112	3	0.2418	21
37	Log-Logistic (3P)	0.11612	4	0.33848	2	0.24766	22
22	Gen. Gamma	0.1173	5	0.41448	14	2.7156	35
6	Chi-Squared (2P)	0.1176	6	0.47304	27	2.7198	36
59	Weibull	0.12114	7	0.76658	39	2.8468	37
10	Erlang (3P)	0.12191	8	0.42547	20	0.17383	3
19	Gamma	0.12363	9	0.41491	15	2.4573	34
5	Chi-Squared	0.1262	10	1.5395	41	1.6343	31
53	Rayleigh (2P)	0.12716	11	0.46217	25	2.2781	33
47	Pearson 5 (3P)	0.12719	12	0.39334	6	0.19193	13
41	Lognormal (3P)	0.1285	13	0.39857	7	0.18762	12
29	Inv. Gaussian (3P)	0.12963	14	0.40415	10	0.18327	10
16	Fatigue Life (3P)	0.1298	15	0.40481	11	0.18283	9
38	Log-Pearson 3	0.12985	16	0.4009	9	0.17573	5
42	Nakagami	0.13027	17	0.53976	30	4.9184	44
8	Dagum (4P)	0.13036	18	0.38648	5	0.22046	18
49	Pearson 6 (4P)	0.13269	19	0.41187	13	0.17619	6
20	Gamma (3P)	0.13306	20	0.417	16	0.1744	4
40	Lognormal	0.13491	21	0.41014	12	0.18739	11
21	Gen. Extreme Value	0.13566	22	0.40069	8	0.20429	15
32	Laplace	0.13648	23	0.59756	36	1.3178	30
28	Inv. Gaussian	0.13653	24	0.42048	19	0.17804	7
15	Fatigue Life	0.13668	25	0.41836	18	0.18155	8
27	Hypersecant	0.13765	26	0.57602	33	3.0046	40
25	Gumbel Max	0.13853	27	0.44559	22	0.20241	14
30	Johnson SB	0.13867	28	0.43758	21	0.17313	2
18	Frechet (3P)	0.13991	29	0.41815	17	0.22474	19
60	Weibull (3P)	0.14017	30	0.47312	28	0.13565	1
55	Rice	0.14264	31	0.71045	38	3.061	41
39	Logistic	0.14417	32	0.59393	34	2.9682	39

11	Error	0.14424	33	0.59479	35	2.9583	38
50	Pert	0.14648	34	0.54212	31	0.54789	27
35	Log-Gamma	0.14668	35	0.4469	23	0.21377	17
48	Pearson 6	0.14813	36	0.46314	26	0.21295	16
43	Normal	0.15423	37	0.67143	37	4.077	42
46	Pearson 5	0.15492	38	0.49705	29	0.23159	20
36	Log-Logistic	0.15935	39	0.45323	24	0.38906	25
24	Gen. Pareto	0.16463	40	8.0622	54	N/A	
9	Erlang	0.16495	41	0.56157	32	0.29927	24
58	Uniform	0.17795	42	8.3589	56	N/A	
31	Kumaraswamy	0.18658	43	4.8209	49	N/A	
54	Reciprocal	0.18688	44	1.879	44	4.3226	43
57	Triangular	0.187	45	1.6185	42	0.74932	28
52	Rayleigh	0.19681	46	2.409	46	12.686	48
51	Power Function	0.19985	47	2.2615	45	19.437	49
1	Beta	0.20103	48	1.7879	43	0.84699	29
26	Gumbel Min	0.21808	49	2.4687	47	6.6259	45
17	Frechet	0.22088	50	1.1926	40	2.1156	32
23	Gen. Gamma (4P)	0.26325	51	5.9373	52	N/A	
14	Exponential (2P)	0.27352	52	3.1819	48	7.9001	47
44	Pareto	0.32392	53	5.8314	51	6.8299	46
34	Levy (2P)	0.35626	54	4.8565	50	21.044	50
3	Burr (4P)	0.38864	55	10.454	57	N/A	
13	Exponential	0.39277	56	7.2457	53	31.382	51
45	Pareto 2	0.42208	57	8.2851	55	32.399	52
33	Levy	0.52544	58	10.987	58	54.682	53
12	Error Function	0.94557	59	149.25	59	1822.1	54
56	Student's t	0.99887	60	209.05	60	40630	55
61	Johnson SU	No fit					

Appendix 10: RR- Rainfall Region 24 hr Rainfall Depth (mm) vs Frequency (yr)

RR- Rainfall Region 24 hr Rainfall Depth (mm) vs Frequency (yr)								
Return Period Years	2	5	10	25	50	100	200	500
RR-A1	50.3	66.02	76.28	89.13	98.63	108.06	117.48	130
RR-A2	51.92	65.52	74.45	85.7	94.07	102.45	110.91	122.27
RR-A3	47.54	59.61	67.66	77.92	85.62	93.34	101.13	111.58

RR-A4	50.39	63.83	72.28	82.55	89.97	97.2	104.32	113.63
RR-B1	58.87	71.26	79.29	89.35	96.84	104.37	112.02	122.41
RR-B2	55.26	69.95	79.68	92.03	101.29	110.61	120.07	132.87
RR-C	56.52	71.04	80.54	92.52	101.48	110.5	119.66	132.06
RR-D	56.23	76.84	90.37	107.46	120.23	133.05	146	163.44

Appendix 11: Rainfall depth (ERA 2013) RR B2 Vs Shashemene town.

Distribution	Return period					
	2	5	10	25	50	100
Shashemene	39.75	53.01	61.78	72.87	81.09	89.26
RR B2	55.26	69.95	79.68	92.03	101.29	110.61

Appendix 12: Rainfall depth ratio (Rt: R24)

	Duration (min)										
t(min)	5 min	10 min	15min	20min	25 min	30 min	60 min	90 min	120 min	150 min	180 min
t(hr)	0.083	0.167	0.250	0.333	0.417	0.500	1.000	1.500	2.000	2.500	3.000
t/24	0.003	0.007	0.010	0.014	0.017	0.021	0.042	0.063	0.083	0.104	0.125
(b+24)^n	18.826	18.826	18.826	18.826	18.826	18.826	18.826	18.826	18.826	18.826	18.826
(b+t)^n	0.414	0.496	0.577	0.657	0.736	0.814	1.273	1.717	2.152	2.579	2.999
RRt	0.158	0.264	0.340	0.398	0.444	0.482	0.616	0.685	0.729	0.761	0.785

Appendix 13: Rainfall depth in a given duration t.

t(min)	Return Period					
	2	5	10	25	50	100
5	8.297	11.083	12.928	15.259	16.988	18.705
10	13.874	18.533	21.618	25.516	28.408	31.278
15	17.921	23.939	27.924	32.959	36.694	40.402
20	21.015	28.073	32.746	38.651	43.031	47.379
25	23.474	31.358	36.578	43.173	48.066	52.923
30	25.486	34.045	39.713	46.873	52.185	57.458
35	27.170	36.295	42.336	49.970	55.633	61.254
40	28.605	38.213	44.574	52.611	58.573	64.491
45	29.848	39.873	46.510	54.897	61.118	67.293
50	30.938	41.329	48.209	56.901	63.349	69.750
55	31.904	42.619	49.714	58.677	65.327	71.928

60	32.768	43.774	51.060	60.267	67.097	73.876
65	33.548	44.815	52.274	61.700	68.692	75.633
70	34.255	45.760	53.377	63.002	70.141	77.229
75	34.902	46.624	54.385	64.191	71.466	78.687
80	35.496	47.418	55.311	65.284	72.683	80.026
85	36.045	48.151	56.166	66.293	73.806	81.264
90	36.554	48.831	56.959	67.229	74.848	82.411
95	37.028	49.464	57.698	68.101	75.819	83.480
100	37.471	50.056	58.388	68.916	76.726	84.479
105	37.886	50.611	59.035	69.680	77.577	85.415
110	38.277	51.133	59.644	70.399	78.377	86.296
115	38.646	51.625	60.219	71.076	79.131	87.127
120	38.994	52.091	60.761	71.717	79.845	87.913
125	39.324	52.532	61.276	72.325	80.521	88.657
130	39.638	52.951	61.765	72.902	81.164	89.364
135	39.937	53.350	62.230	73.451	81.775	90.038
140	40.221	53.730	62.674	73.975	82.358	90.680
145	40.494	54.093	63.098	74.475	82.915	91.293
150	40.754	54.441	63.503	74.954	83.448	91.880
155	41.003	54.774	63.892	75.412	83.959	92.442
160	41.243	55.094	64.265	75.853	84.449	92.982
165	41.473	55.401	64.624	76.276	84.920	93.500
170	41.694	55.697	64.968	76.683	85.373	93.999
175	41.907	55.982	65.301	77.075	85.810	94.480
180	42.113	56.257	65.621	77.453	86.231	94.944

Appendix 14: IDF table Developed for Shashemene Station

t (hr)	Return Period					
	2 year	5 year	10 year	25 year	50 year	100 year
5	99.56047468	132.99843	155.1373	183.1097	203.8613	224.45967
10	83.24257016	111.20006	129.7104	153.0982	170.4486	187.67086
15	71.68224592	95.757136	111.6968	131.8366	146.7775	161.60804
20	63.0458082	84.220101	98.23933	115.9527	129.0934	142.13714
25	56.33820357	75.259709	87.78739	103.6162	115.3588	127.01481
30	50.9717757	68.090936	79.42531	93.74632	104.3705	114.91616

35	46.5766499	62.219682	72.57673	85.66289	95.37095	105.00733
40	42.90815247	57.319099	66.8604	78.91586	87.85929	96.736678
45	39.7978297	53.164157	62.01383	73.19541	81.49055	89.724437
50	37.1258306	49.594752	57.85026	68.28112	76.01933	83.7004
55	34.80450701	46.493798	54.23313	64.01179	71.26616	78.466962
60	32.76824642	43.773648	51.06018	60.26674	67.09668	73.876201
65	30.9669352	41.367356	48.25334	56.9538	63.4083	69.815134
70	29.36162297	39.222891	45.75191	54.00134	60.12124	66.195949
75	27.92156817	37.299185	43.50798	51.35282	57.17256	62.949337
80	26.62217663	35.563386	41.48324	48.963	54.51191	60.019851
85	25.44353295	33.988888	39.64666	46.79526	52.09851	57.362592
90	24.36933465	32.553914	37.97282	44.81962	49.89896	54.940806
95	23.38610556	31.240462	36.44073	43.01128	47.88569	52.72411
100	22.48260667	30.033518	35.03288	41.34958	46.03568	50.687167
105	21.6493887	28.920459	33.73454	39.81714	44.32957	48.808672
110	20.87844823	27.890594	32.53324	38.39925	42.75098	47.070582
115	20.16296035	26.934805	31.41836	37.08333	41.28594	45.457511
120	19.49706864	26.04527	30.38075	35.85864	39.92245	43.956254
125	18.87571864	25.215236	29.41255	34.71586	38.65017	42.555417
130	18.2945246	24.438845	28.50692	33.64694	37.4601	41.245111
135	17.749662	23.710987	27.6579	32.64484	36.34444	40.016716
140	17.23778026	23.027187	26.86028	31.7034	35.2963	38.862675
145	16.75593123	22.383506	26.10945	30.81719	34.30966	37.776344
150	16.30151031	21.776465	25.40136	29.98143	33.37918	36.751849
155	15.87220761	21.202979	24.73241	29.19186	32.50014	35.783984
160	15.46596729	20.6603	24.0994	28.44471	31.66831	34.868113
165	15.08095339	20.145977	23.49946	27.7366	30.87995	34.000097
170	14.71552112	19.657813	22.93004	27.06451	30.13169	33.176227
175	14.36819251	19.193832	22.38882	26.42571	29.4205	32.393173
180	14.03763566	18.752255	21.87374	25.81775	28.74364	31.647931

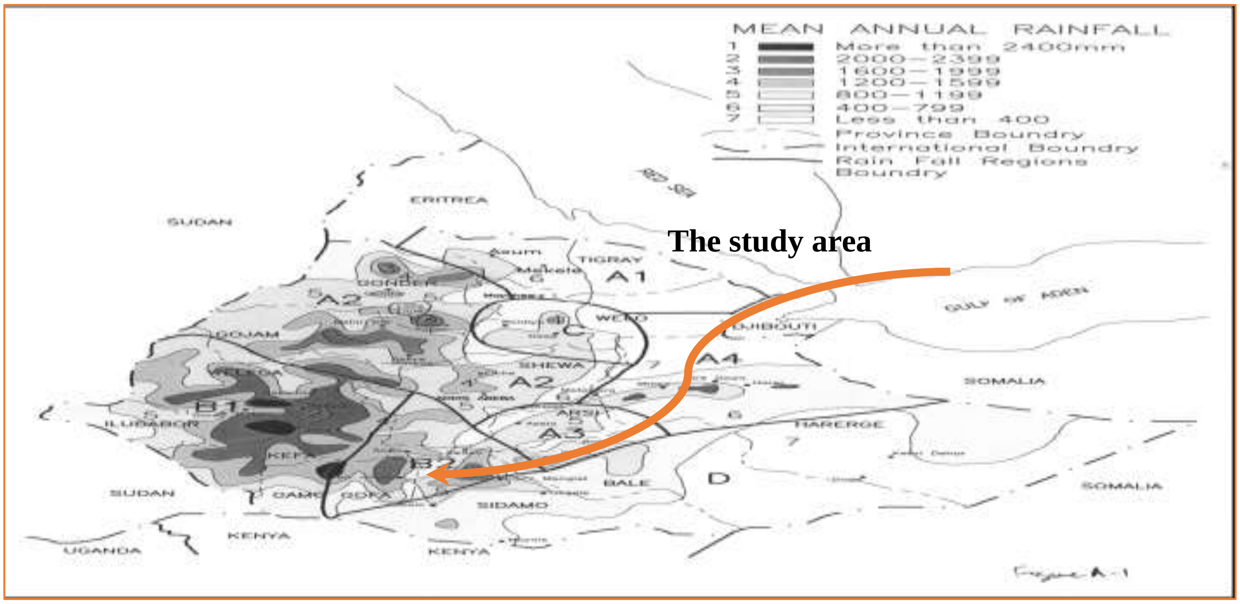

Figure 2: Rainfall region (RR B2)

Appendix 15: Design Storm Frequency (years.) 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 (15<span<50m)	100	50	50	50
Long span bridges (span>50m)	100	100	100	100
Check/review flood	200	100	100	10

Source: (ERA DDM, 2011)

Appendix 16: Area of sub-catchment, Land-Use, and weighted runoff coefficient

Catchment		Asphalt	Bare land	Coblestone	commerci	Admn. Of	P. Institut	Gravel	Residenti	Green Are	Industrial	Ai*Ci	At	Cw
Cs1	Ai	0.81	2.61	3.98	0.27	1.15	1.59	6.68	24.03	0.10			41.2	
	Ci	0.90	0.40	0.75	0.85	0.55	0.60	0.50	0.55	0.35				
	Ai*Ci	0.73	1.04	2.98	0.23	0.63	0.96	3.34	13.22	0.04		23.2		0.56
Cs2	Ai		1.44	3.40			0.53	5.20	22.30	0.17			33.0	
	Ci		0.40	0.75			0.60	0.50	0.55	0.35				
	Ai*Ci		0.58	2.55			0.32	2.60	12.27	0.06		18.4		0.56
Cs3	Ai	0.89	1.45		3.25		10.13	3.41	9.79	0.16			29.1	
	Ci	0.90	0.40		0.85		0.60	0.50	0.55	0.35				
	Ai*Ci	0.80	0.58		2.76		6.08	1.71	5.38	0.06		17.4		0.60
Cs4	Ai	1.14	3.56	3.89	5.09		4.43	1.13	3.06				22.3	
	Ci		0.40	0.75	0.85		0.60	0.50	0.55					
	Ai*Ci		1.42	2.92	4.33		2.66	0.57	1.68			13.6		0.61
Cs5	Ai		0.88	1.07	0.22			2.91	15.51				20.6	
	Ci		0.40	0.75	0.85			0.50	0.55					
	Ai*Ci		0.35	0.80	0.19			1.46	8.53			11.3		0.55
Cs6	Ai	0.45	0.56	2.22	2.94	0.16	2.71	2.34	10.15	0.17			21.7	
	Ci	0.90	0.40	0.75	0.85	0.55	0.60	0.50	0.55	0.35				
	Ai*Ci	0.40	0.22	1.67	2.50	0.09	1.63	1.17	5.58	0.06		13.3		0.61
Cs7	Ai		0.05	0.70			2.11	2.40	15.51				20.8	
	Ci		0.40	0.75			0.60	0.50	0.55					
	Ai*Ci		0.02	0.53			1.27	1.20	8.53			11.5		0.56
Cs8	Ai	0.15		1.56	0.28			1.30	8.17				11.5	
	Ci	0.90		0.75	0.85			0.50	0.55					
	Ai*Ci	0.13		1.17	0.24		0.00	0.65	4.49			6.7		0.58
Cs9	Ai	2.42		1.50	2.42		17.92	0.27	1.46				26.0	
	Ci	0.90		0.75	0.85		0.60	0.50	0.55					
	Ai*Ci	2.18		1.12	2.06		10.75	0.13	0.80			17.0		0.66
Cs10	Ai	1.70	0.07	0.33	0.16		2.98	0.78	7.03				13.1	
	Ci	0.90	0.40	0.75	0.85		0.60	0.50	0.55					
	Ai*Ci	1.53	0.03	0.25	0.14		1.79	0.39	3.87			8.0		0.61
Cs11	Ai	0.38	3.44	2.23	1.40	1.67	18.66	3.33	4.72	7.31			43.1	
	Ci	0.90	0.40	0.75	0.85	0.55	0.60	0.50	0.55	0.35				
	Ai*Ci	0.34	1.37	1.67	1.19	0.92	11.20	1.66	2.59	2.56		23.5		0.55
Cs12	Ai	1.02	0.03	1.41	1.96	4.62	3.09	1.86	5.69				19.7	
	Ci	0.90	0.40	0.75	0.85	0.55	0.60	0.50	0.55					
	Ai*Ci	0.92	0.01	1.06	1.67	2.54	1.85	0.93	3.13			12.1		0.62
Cs13	Ai		1.61	2.54			2.89	3.51	12.67				23.2	
	Ci		0.40	0.75			0.60	0.50	0.55					
	Ai*Ci		0.64	1.91			1.73	1.76	6.97			13.0		0.56
Cs14	Ai	0.33		3.79	8.49	0.37	12.97	2.80	11.36	0.31			40.4	
	Ci	0.90		0.75	0.85	0.55	0.60	0.50	0.55	0.35				
	Ai*Ci	0.30		2.84	7.22	0.20	7.78	1.40	6.25	0.11		26.1		0.65
Cs15	Ai		2.29	1.10				0.12			4.51		8.0	
	Ci		0.40	0.75				0.50			0.70			
	Ai*Ci		0.92	0.83				0.06			3.15	1.8		0.22
Cs16	Ai	0.17	1.13	2.56	0.34		0.94	2.43	12.74	0.39	2.45		23.2	
	Ci	0.90	0.40	0.75	0.85		0.60	0.50	0.55	0.35	0.70			
	Ai*Ci	0.15	0.45	1.92	0.29		0.56	1.22	7.01	0.14	1.72	11.7		0.51
Cs17	Ai	0.13	6.40	0.89				1.14	7.96				16.5	
	Ci	0.90	0.40	0.75				0.50	0.55					
	Ai*Ci	0.12	2.56	0.67				0.57	4.38			8.3		0.50
Cs18	Ai		1.55	1.15				1.14	0.06		8.50		12.4	
	Ci		0.40	0.75				0.50	0.55		0.70			
	Ai*Ci		0.62	0.86				0.57	0.03		5.95	2.1		0.17
Cs19	Ai	1.62	0.07	0.32	0.15		2.86	0.74	6.00				11.8	
	Ci	0.90	0.40	0.75	0.85		0.60	0.50	0.55					
	Ai*Ci	1.46	0.03	0.24	0.13		1.72	0.37	3.30			7.2		0.62

Appendix 17: Manning's n for Overland Flow

Surface	n	Surface	n
Smooth Asphalt	0.011	Vitrified clay	0.015

Smooth Concrete	0.012	Cast iron	0.015
Ordinary concrete lining	0.013	Corrugated metal pipes	0.024
Good wood	0.014	Cement rubble surface	0.024
Brick with cement mortar	0.014	Fallow soils (no residues)	0.05
Cultivated soils Residue cover < 20% Residue cover > 20%	0.06 0.17	Woods Light underbrush Dense underbrush	0.40 0.80
Grass Short, prairie Dense remuda grass	0.15 0.24 0.41		

Source: McCuen, R.et al. (1996). Hydrology, FHWA-SA- 96-067, Federal Highway Administration, Washington, DC.

Appendix 18: 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 (minor streams, top width at flood stage <100ft) - Fairy regular section -Irregular section with pools	0.03-0.07 0.04-0.100		

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

Appendix 19: Soil conservation service peak discharge coefficients

Rainfall type	Ia/P	C0	C1	C2	Rainfall type	Ia/P	C0	C1	C2
I	0.1	2.3055	-0.5143	-0.1175	II	0.1	2.55323	-0.6151	-0.164
	0.2	2.23537	-0.5039	-0.0893		0.3	2.46532	-0.6226	-0.1166
	0.25	2.18219	-0.4849	-0.0659		0.35	2.41896	-0.6159	-0.0882
	0.3	2.10624	-0.457	-0.0284		0.4	2.36409	-0.5986	-0.0562
	0.35	2.00303	-0.4077	0.01983		0.45	2.29238	-0.5701	-0.0228
	0.4	1.87733	-0.3227	0.05754		0.5	2.20282	-0.516	-0.0126
	0.45	1.76312	-0.1564	0.00453	III	0.1	2.47317	-0.5185	-0.1708
	0.5	1.67889	-0.0693	0		0.3	2.39628	-0.512	-0.1325
IA	0.1	2.0325	-0.3158	-0.1375		0.35	2.35477	-0.4974	-0.1199
	0.2	1.91978	-0.2822	-0.0702		0.4	2.30726	-0.4654	-0.1109
	0.25	1.83842	-0.2554	-0.026		0.45	2.24876	-0.4131	-0.1151
	0.3	1.72657	-0.1983	0.02633		0.5	2.17772	-0.368	-0.0953

Appendix 20: curve number of cultivated agricultural Land

Cover description	Curve numbers for hydrologic soil group	Curve number for hydrologic soil group				
Cover Type	Treatment2	Hydrologic condition	A	B	C	D
Fallow	Bare soil Crop residue cover (CR)	- Poor Good	77 76 74	86 85 83	91 90 88	94 93 90
Row Crops	Straight row (SR)	Good Poor	72 67	81 78	88 85	91 89
SR + CR	Poor Good	71 64	80 75	87 82	90 85	
Contoured (C)	Poor Good	70 65	79 75	84 82	88 86	
C + CR	Poor Good	69 64	78 74	83 81	87 85	
Contoured & terraced (C & T)	Poor Good	66 62	74 71	80 78	82 81	
C&T + CR	Poor Good	65 61	73 70	79 77	81 80	
Small grain SR	Poor Good	65 63	76 75	84 83	88 87	
SR + CR	Poor Good	64 60	75 72	83 80	86 84	

C	Poor	63	74	82	85	
	Good	61	73	81	84	
C + CR	Poor	62	73	81	84	
	Good	60	72	80	83	
C&T	Poor	61	72	79	82	
	Good	59	70	78	81	
C&T + CR	Poor	60	71	78	81	
	Good	58	69	77	80	
Close-seeded SR or broadcast	Poor	66	77	85	89	
	Good	58	72	81	85	
Legumes or C Rotation	Poor	64	75	83	85	
	Good	55	69	78	83	
Meadow C&T	Poor	63	73	80	83	
	Good	51	67	76	80	

Appendix 21**:** Runoff curve numbers- Urban areas

Cover description	Curve numbers for hydrologic soil groups				
Cover type and Hydrologic condition	Average % impervious area[2]	A	B	C	D
Open space (lawns, parks, cemeteries, etc.)[3]					
Poor condition (grass cover <50%)		68	79	86	89
Fair condition (grass cover 50 % to 75%)		49	69	79	84
Good condition (grass cover >75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm drains (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Desert urban areas:					
Natural desert cover		63	77	85	88
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
0.05 hectare or less	65	77	85	90	92
0.1 hectare	38	61	75	83	87
0.135 hectare	30	57	72	81	86
0.2 hectare	25	54	70	80	85
0.4 hectare	20	51	68	79	84
0.8 hectare	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation)		77	86	91	94

Appendix 22: Existing Drainage Capacity from the Manning Equation

Code	Catchment	Length	Geometry	Material	n	width (m)	depth	side slop (	As	P	R	R^2/3	Av.slope%	100*Av.slop	S^(0.5)	1/n	v(M/S)	Q(M^3/S)
DL1	Cs1	662	Trap.canal	masonry	0.025	0.4	0.5	1.9	0.67	2.65	0.252	0.399	0.017	1.73	0.131	40.0	2.10	1.40
DL2	Cs1	786	Rec.canal	masonry	0.025	0.95	1.1		1.05	3.15	0.332	0.479	0.010	1.73	0.102	40.0	1.96	2.05
DL3	Cs2	561	Rec.canal	masonery	0.025	0.8	1		0.80	2.80	0.286	0.434	0.011	1.13	0.106	40.0	1.84	1.47
DL4	Cs2	345	Rec.canal	masonery	0.025	0.55	0.6		0.33	1.75	0.189	0.329	0.011	1.13	0.106	40.0	1.40	0.46
DL5	Cs3	693	Rec.canal	Rcconcreat	0.011	0.8	0.85		0.69	2.51	0.274	0.422	0.014	1.36	0.116	90.9	4.47	3.08
DL6	Cs3	434	Rec.canal	masonry	0.025	0.55	0.6		0.33	1.75	0.189	0.329	0.014	1.36	0.116	40.0	1.53	0.51
DL7	Cs4	499	Rec.canal	Rcc	0.011	0.8	0.85		0.68	2.50	0.272	0.420	0.016	1.55	0.125	90.9	4.75	3.23
DL8	Cs4	402	Rec.canal	masonry	0.025	0.5	0.6		0.30	1.70	0.176	0.315	0.016	1.55	0.125	40.0	1.57	0.47
DL9	Cs5	313	Rec.canal	masonery	0.025	0.55	0.6		0.33	1.75	0.189	0.329	0.029	2.94	0.171	40.0	2.25	0.74
DL10	Cs5	660	Rec.canal	masonery	0.025	0.8	1		0.80	2.80	0.286	0.434	0.029	2.94	0.171	40.0	2.97	2.38
DL11	Cs6	515	Rec.canal	masonery	0.025	0.8	1		0.80	2.80	0.286	0.434	0.010	0.97	0.098	40.0	1.71	1.37
DL12	Cs6	523	Rec.canal	masonery	0.025	0.5	0.6		0.30	1.70	0.176	0.315	0.010	0.97	0.098	40.0	1.24	0.37
DL13	Cs7	336	Rec.canal	masonery	0.025	0.8	1		0.80	2.80	0.286	0.434	0.024	2.37	0.154	40.0	2.67	2.14
DL14	Cs7	476	Rec.canal	masonery	0.025	0.8	1		0.80	2.80	0.286	0.434	0.024	2.37	0.154	40.0	2.67	2.14
DL15	CS8	208	Rec.canal	masonery	0.025	0.5	0.6		0.30	1.70	0.176	0.315	0.025	2.45	0.157	40.0	1.97	0.59
DL16	CS8	360	Rec.canal	masonery	0.025	0.8	1		0.80	2.80	0.286	0.434	0.025	2.45	0.157	40.0	2.72	2.17
DL17	Cs9	596	Rec.canal	Rcc	0.011	0.8	0.85		0.68	2.50	0.272	0.420	0.025	2.51	0.158	90.9	6.05	4.11
DL18	Cs9	998	Rec.canal	masonry	0.025	0.56	0.6		0.34	1.76	0.191	0.332	0.025	2.51	0.158	40.0	2.10	0.71
DL19	Cs10	65	Rec.canal	masonery	0.025	0.5	0.6		0.30	1.70	0.176	0.315	0.014	1.37	0.117	40.0	1.47	0.44
DL20	Cs10	1085	Rec.canal	masonery	0.025	0.56	1		0.56	2.56	0.219	0.363	0.014	1.37	0.117	40.0	1.70	0.95
DL21	Cs11	1237	Rec.canal	masonery	0.025	0.6	0.7		0.42	2.00	0.210	0.353	0.037	3.70	0.192	40.0	2.72	1.14
DL22	Cs11	748	Rec.canal	masonery	0.025	0.9	1		0.90	2.90	0.310	0.458	0.030	3.03	0.174	40.0	3.19	2.87
DL23	Cs12	76.5	Rec.canal	Rcc	0.011	0.62	0.88		0.55	2.12	0.257	0.405	0.028	2.78	0.167	90.9	6.14	3.35
DL24	Cs12	592	Rec.canal	masonery	0.025	0.8	0.2		0.20	2.20	0.091	0.202	0.028	2.78	0.167	40.0	1.35	0.27
DL25	cs13	885	Rec.canal	masonery	0.025	0.6	0.7		0.42	2.00	0.210	0.353	0.026	2.64	0.162	40.0	2.30	0.96
DL26	Cs13	225	Rec.canal	masonery	0.025	0.9	1		0.90	2.90	0.310	0.458	0.026	2.64	0.162	40.0	2.98	2.68
DL27	Cs14	673	Rec.canal	Rcc	0.011	0.88	0.62		0.55	2.12	0.257	0.405	0.016	1.57	0.125	90.9	4.61	2.51
DL28	Cs14	596	Rec.canal	masonery	0.025	1	0.9		0.90	2.80	0.321	0.469	0.016	1.57	0.125	40.0	2.35	2.12
DL29	cs15	566	Rec.canal	masonery	0.025	0.6	0.7		0.42	2.00	0.210	0.353	0.023	2.27	0.151	40.0	2.13	0.89
DL30	cs15	265	Rec.canal	masonery	0.025	0.4	0.6		0.24	1.60	0.150	0.282	0.023	2.27	0.151	40.0	1.70	0.41
DL31	cs16	374	Rec.canal	Rcc	0.011	0.62	0.88		0.55	2.12	0.257	0.405	0.044	4.36	0.209	90.9	7.68	4.19
DL32	cs16	585	Rec.canal	masonery	0.025	0.4	0.6		0.24	1.60	0.150	0.282	0.044	4.36	0.209	40.0	2.36	0.57
DL33	cs17	283	Rec.canal	Rcc	0.025	0.62	0.88		0.42	2.00	0.210	0.353	0.018	1.77	0.133	40.0	1.88	0.79
DL34	cs17	476	Rec.canal	masonery	0.025	0.4	0.6		0.24	1.60	0.150	0.282	0.018	1.77	0.133	40.0	1.50	0.36
DL35	Cs18	478	Rec.canal	masonery	0.025	0.6	0.7		0.42	2.00	0.210	0.353	0.013	1.29	0.113	40.0	1.60	0.67
DL36	Cs18	208	Rec.canal	masonery	0.025	0.4	0.6		0.24	1.60	0.150	0.282	0.013	1.29	0.113	40.0	1.28	0.31
DL37	Cs19	1064	Rec.canal	masonery	0.025	0.5	1.2		0.60	2.90	0.207	0.350	0.010	1.01	0.100	40.0	1.41	0.84
DL38	Cs19	251	Rec.canal	masonery	0.025	0.4	0.5		0.20	1.40	0.143	0.273	0.010	1.01	0.100	40.0	1.10	0.22

Appendix 23: Peak discharge estimated Vs existing drainage systems capacity

Catchments	Area (ha)	Tc(min)	Return year (T)				Discharge Rational method (M^3/s)				
			10	25	50	100	Q10	Q25	Q50	Q100	Q Manning
Cs1	41	28.34	68	80	89	98	4.4	5.7	6.9	7.9	2.0
Cs2	33	18.16	86	102	113	125	4.4	5.7	6.9	7.9	1.5
Cs3	29	20.75	81	95	106	117	3.9	5.0	6.1	7.0	3.1
Cs4	22	17.01	89	105	117	129	3.3	4.3	5.2	5.9	3.2
Cs5	21	17.81	87	103	114	126	2.8	3.7	4.5	5.1	2.4
Cs6	22	24.45	74	87	97	107	2.7	3.6	4.3	4.9	0.7
Cs7	21	15.78	92	109	121	133	3.0	3.9	4.7	5.4	2.1
Cs8	11	13.05	100	118	131	144	1.9	2.4	2.9	3.3	2.2
Cs9	26	24.88	73	87	96	106	3.5	4.5	5.5	6.3	4.1
Cs10	13	20.00	82	97	108	119	3.6	4.6	5.6	6.5	1.0
Cs11	41	24.65	74	87	97	107	5.5	7.1	8.6	9.9	2.9
Cs12	20	20.33	82	96	107	118	2.7	3.6	4.3	5.0	3.3
Cs13	23	23.32	76	90	100	110	2.7	3.5	4.3	4.9	2.7
Cs14	40	1500.00	83	98	109	120	6.0	7.8	9.5	10.9	2.5
Cs15	8	18.17	86	102	113	125	1.2	1.5	1.8	2.1	0.9
Cs16	23	15.57	93	109	122	134	3.5	4.5	5.4	6.2	4.2
Cs17	17	1500.00	83	98	109	119	1.8	2.4	2.9	3.3	0.8
Cs18	12	20.19	82	97	108	118	1.8	2.4	2.9	3.3	0.7
Cs19	12	25.00	100	118	131	144	1.5	1.9	2.3	2.6	0.8

Appendix 24: Types of drainage area with runoff coefficient

Type of Drainage Area	Runoff coefficient (C)
Business:	
Commercial area	0.70-0.95
Neighborhood areas	0.50-0.70
Residential:	
Single-family areas	0.30-0.50
Multi-units, detached	0.40-0.60
Multi-units, attached	0.60-0.75
Suburban	0.25-0.40
Apartment dwelling areas	0.50-0.70
Industrial:	
Light areas	0.50-0.80
Heavy areas	0.60-0.90
Parks, cemeteries	0.10-0.25

Playgrounds	0.20-0.40
Railroad yard areas	0.20-0.40
Unimproved areas	0.10-0.30
Lawns:	
Sandy soil, flat, < 2%	0.05-0.10
Sandy soil, average, 2 to 7%	0.10-0.15
Sandy soil, steep, > 7%	0.15-0.20
Heavy soil, flat, < 2%	0.13-0.17
Heavy soil, average 2 to 7%	0.18-0.22
Heavy soil, steep, > 7%	0.25-0.35
Streets:	
Asphaltic	0.70-0.95
Concrete	0.80-0.95
Brick	0.70-0.85
Drives and walks	0.75-0.85
Roofs	0.75-0.95

Appendix 25: Peak discharge of the study area using the Rational Method.

Catchments	Area (ha)	Tc(hr)	Return year				Discharge Rational				Q manning
			10	25	50	100	Q10	Q25	Q50	Q100	
Cs1	41	28.34	68	80	89	98	4.4	5.7	6.9	7.9	2.0
Cs2	33	18.16	86	102	113	125	4.4	5.7	6.9	7.9	1.5
Cs3	29	20.75	81	95	106	117	3.9	5.0	6.1	7.0	3.1
Cs4	22	17.01	89	105	117	129	3.3	4.3	5.2	5.9	3.2
Cs5	21	17.81	87	103	114	126	2.8	3.7	4.5	5.1	2.4
Cs6	22	24.45	74	87	97	107	2.7	3.6	4.3	4.9	0.7
Cs7	21	15.78	92	109	121	133	3.0	3.9	4.7	5.4	2.1
Cs8	11	13.05	100	*118*	*131*	144	1.9	2.4	2.9	3.3	2.2
Cs9	26	24.88	73	*87*	*96*	106	3.5	4.5	5.5	6.3	4.1
Cs10	13	20.00	82	97	108	119	3.6	4.6	5.6	6.5	1.0
Cs11	41	24.65	74	87	97	107	5.5	7.1	8.6	9.9	2.9
Cs12	20	20.33	82	96	107	118	2.7	3.6	4.3	5.0	3.3
Cs13	23	23.32	76	90	100	110	2.7	3.5	4.3	4.9	2.7
Cs14	40	1500.00	83	98	109	120	6.0	7.8	9.5	10.9	2.5
Cs15	8	18.17	86	102	113	125	1.2	1.5	1.8	2.1	0.9
Cs16	23	15.57	93	109	122	134	3.5	4.5	5.4	6.2	4.2
Cs17	17	1500.00	83	98	109	119	1.8	2.4	2.9	3.3	0.8
Cs18	12	20.19	82	97	108	118	1.8	2.4	2.9	3.3	0.7
Cs19	12	25.00	100	118	131	144	1.5	1.9	2.3	2.6	0.8

Appendix 26: Depression Storage and infiltration both previous and impervious

Parameter	Description	Allowable Range	used
N-impervious	Manning's n for overland flow over the impervious portion of the sub-catchment	0.005-0.5	0.01
N-previous	Manning's n for overland flow over the previous portion of the sub-catchment	0.05-0.5	0.1
Dstore - impervious	Depth of surface storage in impervious areas (mm)	1.3-2.5	1.5
Dstore – previous	Depth of surface storage previous areas (mm)	2.5-7.6	3

Source: ASCE, Design, and Construction of Urban Storm Management Systems, New York,

Appendix 27: Hydrological soil group and infiltration rate

Soil group	Infiltration Rate (in/hr)	Hydrologic Soil Group
A	0.3 – 0.45	High infiltration rates. Deep, well-drained sands and gravels
B	0.15 – 0.30	Moderate infiltration rates. Moderately deep, moderately well-drained soils with moderately coarse textures (silt, silt loam)
C	0.05 – 0.15	Slow infiltration rates. Soils with layers, or soils with moderately fine textures (clay loams)
D	0.00 – 0.05	Very slow infiltration rates. Clayey soils, high water table, or shallow impervious layer

Source (NRCS, 2007)

Appendix 28: Typical Hydrologic Soils Groups for Ethiopia

Soil Types	Hydrologic Soil Group	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
Hl	Ludovic Phaeozems	C
I	Lithosols	D
Jc	Calcaric Fluvisols	B
Je	Eutric Fluvisols	B
Lc	Chromic Luvisols	B
Lo	Orthic Luvisols	B
Lv	Vertic Luvisols	C
Nd	Dystric Nitosols	B
Ne	Eutric Nitosols	B
Od	Dystric Histosols	D
Oe	Eutric Histosols	D
Qc	Cambric Arenosols	A
Rc	Calcaric Regosols	A
Re	Eutric Regosols	A
Th	Humic Andosols	B
Tm	Mollic Andosols	B
Tv	Vitric Andosols	B
Vc	Chromic Vertisols	D
Vp	Pellic Vertisols	D
Xh	Haplic Xerosols	B
Xk	Caloric Xerosols	B
Xl	Luvic Xerosols	C
Yy	Gypsic Yermosols	B
Zg	Gleyic Solonchaks	D
Zo	Orthic Solonchaks	B

(Source: Ministry of Agriculture)

Appendix 29: Junction properties

Junction	X	Y	Elevation	Max.depth	Inverted depth
J1	457700	796134	1999	1.1	1997.9
J2	457327	795733	1998.6	1.1	1997.5
J3	457709	795466	2020	0.5	2019.5
J4	456962	796034	1977.85	0.85	1977
J5	457164	796243	1978.6	0.6	1978
J6	457513	795948	1996	1	1995

J7	457172	796251	1979	1	1978
J8	457318	796405	1983	0.6	1982.4
J9	457326	796414	1963	1	1962
J10	457483	796597	1984	1	1983
J11	457723	796391	1994	1	1993
J13	457878	796703	1987	1	1986
J14	456712	796277	1964.95	0.8	1964.15
J16	456893	796479	1966.7	0.6	1966.1
J17	456905	796490	1967.2	0.6	1966.6
J18	457158	796735	1968	0.6	1967.4
J20	457158	796734	1968	0.6	1967.4
J21	457474	796602	1981.7	0.7	1981
J25	457301	796776	1973.1	0.7	1972.4
J26	457177	796739	1969.4	1	1968.4
J27	457171	796730	1943.2	1.2	1942
J28	456931	797357	1967	1	1966
J30	457155	796748	1967	1	1966
J33	457067	796686	1965.7	0.2	1965.5
J34	456704	796280	1964.5	1	1963.5
J35	456405	796534	1945.3	0.8	1944.5
J36	457061	796708	1966	0.6	1965.4
J37	456398	796545	1945	1	1944
J38	456358	796567	1941.88	0.5	1941.38
J39	456629	796878	1943.2	0.2	1943
J40	456002	796879	1921.38	1	1920.38
J41	456273	797191	1921	1	1920
J42	455813	797050	1910.26	0.88	1909.38
J43	456008	797401	1911.1	0.7	1910.4
J45	455829	797472	1907	0.6	1906.4
J46	456060	797475	1912.7	0.7	1912
J47	456271	797207	1920	0.7	1919.3
J48	456380	797329	1924	0.7	1923.3
J51	456094	797542	1909.1	0.7	1908.4
J52	456236	797587	1914	0.6	1913.4
J53	456381	797317	1925	1	1924
J54	456734	796796	1952	0.7	1951.3
J56	456928	796688	1943	0.7	1942.3
J57	457076	796722	1965.6	0.6	1965
J60	456950	797359	1946.6	0.6	1946
J61	456796	797319	1943.6	0.6	1943
J62	456888	797440	1943	1	1942

J12	457705	796846	1976	1	1975
J66	456716	797424	1953	1	1952
J58	456897	797441	1941.2	1.2	1940
J44	455665	797180	1906.1	0.8	1905.3

Appendix 30: Outfall properties

Outfall node	X	Y	Elevation	Max.depth	Inverted	Location	Outfall river
Outlet-1	455661.11	797183.64	2411	0.88	1905	Alelu	Essa
Outlet-2	456901.38	797442.33	2400	1	1938	Alelu	Malka Oda
0utlet-3	457704.54	796847.19	2395	1	1974.5	Awasho	Malka Oda

Appendix 31: Sub-catchments properties

Name	Area(ha)	Slope %	L(M)	b (m)	Around
Sc-1	41.16	1.73	786.00	523.70	Awasho
Sc-2	32.93	1.13	561.00	587.05	Nock
Sc-3	29.22	1.36	693.00	421.67	Awasho
Sc-4	22.13	1.55	499.00	443.44	New-bus station
Sc-5	21.05	2.94	660.00	319.01	Awasho
Sc-6	21.77	0.97	515.00	422.73	Nock
Sc-7	20.70	2.37	476.00	434.77	Malka oda
Sc-8	11.48	2.45	360.00	319.02	Nock
Sc-9	25.97	2.51	998.00	260.26	New bus station
Sc-10	13.05	1.37	1085.00	120.28	Marana
Sc-11	41.06	3.70	1237.00	331.94	Industrial area
Sc-12	19.22	2.78	76.50	2512.33	Batu college
Sc-13	23.30	2.64	885.00	263.29	Alelu school
Sc-14	40.54	1.57	673.00	602.31	Stadium
Sc-15	8.02	2.27	566.00	141.67	Alelu market
Sc-16	23.20	4.36	585.00	396.54	Alelu market
Sc-17	16.58	1.77	476.00	348.41	Alelu market
Sc-18	12.41	1.29	478.00	259.62	Alelu market

Sc-19	11.75	1.01	1064.00	110.43	Malka oda Hospital

Appendix 32: Simulation Result Discharge 10 year from SWWM

Sub-catchment	Total precip mm	Total infiltration mm	Total Runoff mm	Total Runoff 10^6 ltr	Peak runoff CMS	Runoff Coeff
Sc-1	148.85	0.01	132.72	54.63	8.07	0.892
Sc-2	148.85	0.01	133.97	44.12	6.65	0.9
Sc-3	148.85	0.01	133.99	39.15	5.9	0.9
Sc-4	148.85	0.01	136.43	30.19	4.75	0.917
Sc-5	148.85	0.01	136.75	28.79	4.56	0.919
Sc-6	148.85	0.01	136.48	29.71	4.68	0.917
Sc-7	148.85	0.01	138.37	28.64	4.69	0.93
Sc-8	148.85	0.01	140.23	16.1	2.77	0.942
Sc-9	148.85	0.47	131.79	34.23	5.02	0.885
Sc-10	148.85	0.47	127.01	16.58	2.3	0.853
Sc-11	148.85	0.47	131.55	54.02	7.9	0.884
Sc-12	148.85	0.47	145.45	27.95	6.23	0.977
Sc-13	148.85	0.47	133.29	31.06	4.67	0.895
Sc-14	148.85	0.47	133.42	54.09	8.14	0.896
Sc-15	148.85	0.47	148.41	11.9	3.45	0.997
Sc-16	148.85	0.47	138.61	32.16	5.35	0.931
Sc-17	148.85	0.47	136.91	22.71	3.64	0.92
Sc-18	148.85	0.47	135.62	16.83	2.63	0.911
Sc-19	148.85	0.47	125.1	14.7	2.01	0.84

Appendix 33: Simulation Result Discharge 25 year from SWWM

Sub-catchment	Total precip mm	Total infil mm	Total Runoff mm	Total Runoff 10^6 ltr	Peak runoff CMS	Runoff Coeff
Sc-1	175.68	0.01	158.33	65.17	9.83	0.901
Sc-2	175.68	0.01	159.7	52.6	8.09	0.909
Sc-3	175.68	0.01	159.73	46.67	7.18	0.909

Sc-4	175.68	0.01	162.41	35.94	5.77	0.924
Sc-5	175.68	0.01	162.76	34.26	5.53	0.926
Sc-6	175.68	0.01	162.46	35.37	5.68	0.925
Sc-7	175.68	0.01	164.54	34.05	5.69	0.937
Sc-8	175.68	0.01	166.57	19.13	3.37	0.948
Sc-9	175.68	0.47	157.35	40.86	6.12	0.896
Sc-10	175.68	0.47	152.1	19.85	2.8	0.866
Sc-11	175.68	0.47	157.09	64.5	9.63	0.894
Sc-12	175.68	0.47	172.27	33.11	7.52	0.981
Sc-13	175.68	0.47	158.99	37.05	5.68	0.905
Sc-14	175.68	0.47	159.14	64.52	9.91	0.906
Sc-15	175.68	0.47	175.25	14.05	4.07	0.998
Sc-16	175.68	0.47	164.84	38.24	6.51	0.938
Sc-17	175.68	0.47	162.96	27.03	4.42	0.928
Sc-18	175.68	0.47	161.55	20.05	3.2	0.92
Sc-19	175.68	0.47	149.98	17.62	2.45	0.854

Appendix 34: SWMM Simulation, Rational, and Manning Discharge.

Sub-Catchments	Q Rational		Q Manning	Simulation SWMM	
	Q10 Rational	Q25 Rational		Q10 SWMM	Q25 SWMM
Sc-1	4.37	5.66	**2.05**	8.07	9.83
Sc-2	4.39	5.70	**1.47**	6.65	8.09
Sc-3	3.89	5.04	**3.08**	5.90	7.18
Sc-4	3.29	4.26	**2.54**	4.75	5.77
Sc-5	2.84	3.69	**2.38**	4.56	5.53
Sc-6	2.74	3.55	**0.74**	4.68	5.68
Sc-7	2.97	3.86	**2.14**	4.69	5.69
Sc-8	**1.85**	**2.40**	**2.17**	2.77	3.37
Sc-9	**3.48**	**4.51**	**4.11**	5.02	6.12
Sc-10	3.57	4.64	**0.95**	2.30	2.80
Sc-11	5.49	7.12	**2.87**	7.90	9.63
Sc-12	**2.75**	**3.56**	**3.35**	6.23	7.52
Sc-13	2.73	3.54	**2.68**	4.67	5.68
Sc-14	6.02	7.82	**2.51**	8.14	9.91
Sc-15	1.16	1.51	**0.89**	3.45	4.07
Sc-16	**3.45**	**4.48**	**4.19**	5.35	6.51
Sc-17	1.83	2.38	**0.79**	3.64	4.42

Sc-18	1.81	2.35	**0.67**	2.63	3.20
Sc-19	1.46	1.90	**0.84**	2.01	2.45

Appendix 44: Maximum velocity of lined canal.

Paved	Velocity (m/s)
a. Concrete, w/all surfaces:	
1) Trowel finish	6
2) Float finish	6
3) Formed, no finish	6
b. Concrete bottom, float finished, w/sides of:	
1) Dressed stone in mortar	5.4 - 6.0
2) Ramdom stone in mortar	5.1 - 5.7
3) Dressed stone or smooth concrete rubble (riprap)	4.5
4) Rubble or random stone (riprap)	4.5
c. Gravel bottom, sides of:	
1) Formed concrete	3
2) Random stone in mortar	2.4 - 3.0
3) Random stone or rubble (riprap)	2.4 - 3.0
d. Brick	3
3) Asphalt	5.4 - 6.0

U. S. Environmental Protection Agency, 1975