STORM WATER DRAINAGE SYSTEM ANALYSIS OF HAWASSA CITY, ETHIOPIA

MASTER OF SCIENCE THESIS IN WATER RESOURCE ENGINEERING AND MANAGEMENT

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We, the undersigned, members of the Board of Examiners of the final open defense by Girma Yane Rikiba have read and evaluated his thesis entitled "storm water drainage system analysis of Hawassa city", and examined the candidate. This is, therefore, to certify that the thesis has been accepted in partial fulfillment of the requirements for the degree

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ABSTRACT

Intense rainfall and urbanization are the main factors that affect the storm drainage system. In addition to rainfall intensity and impermeable layers, system blockage by solid wastes is the common problem in Hawassa city. The study achieved the objective of storm water drainage system analysis of Hawassa city. To analyze the level of the problem models like Arc GIS10.2, Google earth Pro, Generalized Extreme Value distribution and rational method are used to delineate the drainage system, classify of the sub watershed, simulate the design rainfall depth and runoff volume estimation respectively. The peak discharge generated in 2002 was 54.45m^3*/s. Due to both factors the peak discharge generated counts 102.80m*^{3}*/s in 2019. Following the drainage system networking capacity of 13drains (59.09%) exceeded by design discharge, 20(90.91%) by 25 years discharge and 21(95.45%) drains capacity exceeded by 100 years discharge that contributes to pedestrian mobility interruption, traffic congestion , chronic distress of asphalt road and environmental health problem due to over flooding. Communication between professionals responsible for storm water drainage system, collaboration of institutions such as water supply, sewerage and sanitation, solid waste management and infrastructure department should be maintained. SUDs measures such as sizing drain and retention pond strategies to cope with intense rainfall and urbanization impact on storm drainage system and environment were recommended.*

Key words: Intense rainfall, urbanization, storm water drainage system, rational method, Arc GIS10.2

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LIST OF ACRONMYS AND ABBREVATIONS

AASHTO:	American Association of State Highway and Transport Officials
AUDACIOUS:	Adaptable Urban Drainage Addressing Change in Intensity Occurrence and Uncertainty of Storm
BKCC:	Building Knowledge for a Climate change
BMP:	Best Management Practices
CDF:	Cumulative Distribution Function
CH:	Catchment
Co:	Carbon monoxide
Co2:	Carbon dioxide
DCC:	Dhaka City Corporation
DDM:	Drainage Design Manual
DEM:	Digital Elevation Model
E.C:	Ethiopian Colander
EMA:	Ethiopian Mapping Agency
EPA	Environmental Protection Agency
ERA:	Ethiopian Road Authority
ET:	Ethiopian
Ff:	Following
GDP:	Growth Development Plan
GEV:	Generalized Extreme Value Distribution
GHG:	Green House Gases
GIS:	Geographic Information System
GPS:	Geographical Position System
IDF:	Intensity, Duration, Frequency
IPCC:	Intergovernmental Panel on Climate Change
LID:	Low impact development
LULC:	Land use land cover
MOM:	Method of Moment
NGO:	Nongovernmental Organizations
PDF:	Probability Density Function
PMF:	Probable Maximum Flood
PPM:	part per million
PWMs:	Probability Weighted Moments
SNNPRS:	South Nation's Nationalities and People Regional State
SUDS:	Sustainable Urban Drainage Systems
Sq.kms:	Square Kilometers
SWMM:	Storm Water Management Model
UK:	United Kingdom
U.S:	United States
USWM:	Urban Solid Waste Management
VOC:	Volatile organic compounds

1 INTRODUCTION

1.1 Background of the Study

The review on influencing factors and strategies for Sustainable Urban Drainage System, defines that, “drainage system is a major part of city infrastructure, which is vital and essential for the needs of interactions between human’s civil activities and the water circulation within and even away from an urban area”. These infrastructures fixed at paved streets, parking lots, sidewalks, and building roofs during storm events (Fengxiang Q. et al. 2018). According to the review of (Water, 2014) on Sustainable Urban Drainage Systems Considering the Climate Change and Urbanization Impacts conclusion, for long time, urban drainage has played a different role in cities. Prior objectives of urban drainage include provision of a suitable cleaning mechanism of wastes for public hygiene and proficient conveyance facility for flood protection and recently, additional focus has been on environmental protection and the recreational benefits of urban drainage. These infrastructures account all parts of the urban water cycle in management to ensure economic, social, ecological and environmental sustainability (Jurijs Kondratenko et.al. 2007-2013). The efficiency of an urban drainage system could be affected by many factors such as: climate, topography, geology, scientific knowledge, engineering and construction capabilities, societal values, religious beliefs etc (Fengxiang Q. et al. 2018). Urban development modified the living environment and affect water drainage in a number of ways. The increase in impermeable surfaces such as parking lots, highway systems and conversion of woodlands in to high-dense commercial and residential uses greatly reduce water infiltration in to the ground (Fengxiang Q. et al. 2018). In other hand, climate change induced increasing storm rainfall intensity has direct impacts on surface runoff. The combined effects lead to more surface runoff, faster runoff concentration and high flow rate (Fengxiang Q. et al. 2018). Lacks of urban storm water drainage management represent one of the most common sources of compliant from the dwellers in many urban center of Ethiopia. In addition to intense rainfall and impermeability of the urban landscape, the planning as well as implementation of storm water protecting is insufficient (Biniam A., 2016). The question of intense rainfall and impermeable layers and their effect on the design and operation of urban- drainage systems of Hawassa city is addressed.

1.2 Statements of the Problem

Hawassa city is the developing city in SNNPR of Ethiopia. To assure development and advance of living standard of the dwellers, a lot of infrastructures such as buildings, roads, etc. are constructed and several industries are planted. Hence, the land use land cover of the city is changing to impermeable layers. Rainfall intensity is also increasing. There is the increment in urban dwellers in the city that contribute for ineffectiveness of urban drainage system through disposing litter. The Litter disposed to find their way in to drain system during the rain which could block the flow of water. Lack of sewerage system is also the main problem in the area. Garages, car washes or service centers beside the road of the city and urban dwellers contribute extra load to the storm drainage system. Though there is storm drainage system, it experiences frequent over flooding in most parts of the city. Property damage, closing of service centers during high rain event because of the fear of property damage by the resulting flood, interruption of pedestrians' mobility etc. are common. Hence, it needs to know the major causes of this problem and their level of importance.

1.3 Objectives

1.3.1 General Objectives

The study aims to analyze the storm water drainage system of Hawassa city.

1.3.2 Specific Objectives

- To estimate the volume of runoff
- To evaluate the performance of storm water drainage system
- To assess the possible impacts of the resulting flood and stagnant water on socio- economic activities, transport system, environmental health of the city.

1.4 Research Questions

1. How do intense rainfall and impermeable layers affect runoff volume?
2. How does the performance of urban drainage system will be affected by increased runoff volume?
3. What the adverse effect the flood and stagnant water has on the, socio-economic activities, transportation system, and environmental health of the city?

1.5 Significance of the Study

The study gives insight for urban planners and managers as well as urban dwellers about pros and coins of proper storm drainage system management in urbanized city Hawassa. It contributes towards providing the city with good aesthetics; make roads safe for driving and for pedestrians, healthy environment, valued amenity aspect, economic and sustainable storm drainage systems.

1.6 The Scope of the Study and limitations

The study embrace's estimating flood volume; evaluate effectiveness of urban drainage system through assessing deviation in design rainfall depth, rainfall intensity and imperviousness of catchment area, determining capacity of drainage system to carry design discharge (adequacy), mapping drainage system networking to find flow concentration in a junction, determine the volume of solid waste that shared the depth of the drains and results in poor performance of storm water drainage system. Effects of resulting flood on service centers and telecom establishment, transportation system (specially pedestrian and open vehicles users) and the road structure itself, and environmental health of the city was assessed. Recommend the effective strategies to manage storm runoff. This study limited to five sub- cities such as Menehariya (Guwe and Millennium), Misrak, Mehal, Hayk-Dar (Gebeya-dar), Bahil-Adarash(Anidnet and Harar) because the sub cities are flood prone areas.

2 LITERATURE REVIEW

2.1 Intense rainfall and impermeable layer effect on runoff volume

As IPCC special report on Climate Change, Desertification, Land Degradation, Sustainable Land Management, Food Security, and Greenhouse gas fluxes in Terrestrial Ecosystems, Changes in land conditions, either from land use or climate change, affect global and regional climate. At the regional scale, changing land circumstances can reduce or emphasize warming and affect the intensity, frequency and duration of extreme events. These changes differ from place to place and season to season. Climate change creates accompanying stresses on land; make worse existing risks to livelihoods, biodiversity, human and ecosystem health, infrastructure, and food. Increasing impacts on land are anticipated under all future GHG emission scenarios. Some regions will face higher risks, while some regions will face risks previously not expected, (IPCC, 2019). According to IPCC fifth assessment report, at the global scale, climate change is expected to lead to significant to sea level rise and to change in frequency, intensity and spatial patterns of temperature, precipitation and other meteorological factors. Climate change has significant impact on ecosystem functioning and welfare of people. Climatic stress leads to a decrease in the distribution of typical native species and influences environment through health-related effects and socio-economic impacts by increased numbers extreme events. In addition to climate change, urbanization and the associated increases in the number and size of cities are impacting ecosystems with a number of interlinked pressures. These include loss and degradation of natural areas, imperviousness and the densification of built-up areas, which pose additional significant challenges to ecosystem functionality, the provision of ecosystem services and human safety in cities around the world (IPCC, 2014).

2.1.1 Urban Temperatures

According to the report of Intercontinental Panel on Climate Change, human activities are estimated to have caused approximately 1.0°C of global warming above pre-industrial levels, with a likely range of 0.8°C to 1.2°C. Global warming is likely to reach 1.5°C between 2030 and 2052 if it continues to increase at the current rate, (IPCC, 2018). The increment in temperature is due to emission of GHG. This increment in temperature brings the increment on extreme events such as drought and flood which affect the urban drainage system. Emission of Co, Co_2 and VOC for Hawassa city is given in table 2-1 below

Table 2—1: GHG emission (ppm) in the study area

Land Use	**Location**	**Co**	**Co_2**	**VOC**
Residential	Condominium	403.80±17.23	1.55±0.46	1668.8±518.23
	Hitata kebele	407.80±22.33	1.68±0.63	2001.2±680.36
	Gezahegn resort	429.00±20.69	1.79±0.92	1442.8±30.34
	Outskirt	417.40±18.00	1.40±0.64	1447±53.69
	Core area	429.4±21.13	1.21±0.38	1488.2±12.36
	Mean	417.48±21.16	1.51±0.61	1609.76±411.79
Industrial	Ceramic factory	467.60±69.37	4.03±1.07	2092.0±646.42
	Industrial park	420.40±19.14	5.30±1.53	1731.4±661.45
	Mean	444.50±54.05	4.66±1.41	1911.70±645.21
Commercial	Market	416.2±16.27	1.03±0.22	1443.4±81.72
Circulation	Roundabout	426.00±30.95	3.44±1.30	2655.8±591.59
	Bus station	421.80±30.95	2.23±1.16	2095.0±906.75
	Main entrance	435.80±17.60	4.32±0.58	1646.2±321.35
	Mean	427.87+20.64	3.33±1.32	2132.33±739.71
Institutional	Hawassa university	386.80±8.41	0.17±0.03	1243.0±253.48
	Referral hospital	394.8±17.54	0.18±0.04	1511.6±114.23
	Mean	390.80±13.63	0.18±0.03	1377.30±233.23
Recreational	Amoragedel	408.20±11.84	0.24±0.13	2010.6±275.86
	Tabor Terara	380.40±16.44	0.07±0.03	1634.2±29.50
	Haile resort	375.6±13.64	0.06±0.04	1231.0±269.88
	Grand mean	388.07±19.79	0.123±0.11	1625.27±529.31

(Source: Oluwasinaayomi Faith Kasim et.al, 2018)

2.1.2 Urban Hydrology

The study in south Sweden with a catchment area 54ha of which 20ha is impervious by employing GIS, Mike Urban(MOUSE) and SWMM for climate change and urbanization impact on urban drainage system, tells' us, Hydrological changes, particularly heavier precipitation due to an increasing global mean temperature, will expected in the 21st century. These changes will have an enormous impact on urban environments and infrastructures, especially urban drainage systems whose capacities are closely related to rainfall events (Olofsson, 2007). The estimated variation of temperature, precipitation and extreme events according to Ethiopia is given in table 2-2.

Table 2—2: Ethiopian's changing climate, historical trend and future projection:

	Temperature	**Rainfall**	**Extreme event**
Historical trend	Mean temperature increased By 1.3°c from 1960 – 2006,more hot days and cold nights, fewer cold days and nights	Highly variable From year to year, season to season, decade to decade, no significant trained	Regular sever flood and drought event, no evidence of changes in frequency or intensity of extremes
2020's	1.2°c (0.7 – 2.3°c)	0.40%	Greater increase of rain fall from Oct – Dec. especially in the south and east
2050's	2.2°c (1.4 -2.9°c)	1.10%	Heavier rain fall events, Uncertain future El- Nino behavior brings large uncertainties
2090's	3.3°c (1.5 – 5.1°c)	Wetter conditions	Flood and drought events likely to increase Heat waves and higher evaporation

(Source: N Souverijns, et al 2016)

2.1.3 Design of Urban Drainage Systems

The study of urban drainage in under-developed countries by Jonathan Parkinson says that, the basic problem of drainage system design in under-developed countries is a lack of approval of the differences in rainfall distribution patterns and urban hydrology in several parts of the world. Elevated rainfall intensities and the rain concentrated during a few months cause the flooding problem. Global warming may result in further impacts upon rainfall distribution adding further uncertainty to drainage system design and flood prediction. Urban catchments in developing countries cause wide variations in the rainfall – runoff response and the resultant volume of runoff and peak flows (Jonathan Parkinson, 2002). The study entitled Advanced in urban drainage management and flood protection by Hans-Rein hard Verworn in University of Hannover/ Germany says that, until now; however, systems have never been designed with respect to flooding or runoff frequencies but rather with respect to rainfall frequency. The assumption that rainfall and runoff frequencies are equivalent is the basic principle of the rational formula, and it is still a principle commonly adopted with the use of the design storm concept. The design rainfall intensity is a function of duration and frequency adopted from the analysis of long time-series of rainfall recordings. With the design storm concept, the rainfall load normally no longer has a uniform intensity, and the calculation of the runoff data is more sophisticated, using models for the simulation of the dynamic hydraulic processes within the drainage system. A better solution may be to collect all existing time-series, analyze them for their meteorological characteristics and hydrological responses. Due to its simplicity and the availability of statistical rainfall data, the design storm concept is mainly used (H.R. Verworn, 2002). According to Mats Olofsson, for both climate change and urbanization, Even though summer precipitation decreases in the study area, precipitation intensity will increase, hence more flooding can be expected, maximum water levels in the urban drainage system will increase, hence more and longer lasting floods will occur. The frequency of floods will also increase (Olofsson, 2007).

2.2 Operational Performance

2.2.1 Performance Analysis

According to the journal of loyal society in its title “Advance in Urban Drainage Management and Flood Protection in Germany” performance is defined as the frequency with which a defined boundary condition is exceeded. The boundary condition may be the occurrence of flooding or the water level reaching the ground level or the spilling over of a detention basin. With the appearance of the special computer, models emerged that made it possible to simulate the runoff processes for a definite rainfall input. A more reliable way to assess the performance is to use continuous or time-series simulation with a statistical analysis of the results instead of the rainfall. A rainfall- runoff simulation model is fed with long time-series of recorded rainfall, thus producing equally long time-series of runoff and water level data. In principle, the results of the simulation are a replacement for long sequences of measured data (H.R. Verworn, 2002).

2.2.2 Urbanization and Urban Drainage System

Dibaba’s finding reveals that, aging, unintentional urban settlements, population growth, rising impervious surface and poor waste management infrastructure can be considered as the major factors that affecting the performance of drainage systems in the town. The ongoing construction activities and unsafe waste disposal in the city will increase surface water runoff. In the other hand, lack of appropriate storm water drainage results in uncontrolled surface flow of the water creating enormous flooding problems. Inadequate capacity of the infrastructures, clogging, low storage capacity, waste disposal, poor integration with road, surprising rate of urbanization, land use change, population pressure, lack of periodical inspection and maintenance, carelessness of the community, Poor workmanship and cleaning solid wastes are also the causes of ensuring malfunction of the existing drainage structure (Dibaba, 2018). Jonathan Parkinson in his study find that, the technical and managerial defects and state's like, consecutive operation and maintenance stay behind major challenges to urban authorities who are often ineffective in dealing with the scale of the problem. Uncollected solid waste often finds its way into surface drains and sewers; these drains, blocked with debris, then have less capacity than clean ones, and are more likely to flood during large storms. The problem of litter in the storm water drainage systems is at its worst where modern technologies, such as the plastics industry, have been introduced before the development of a strong environmental lobby or influence to police the waste (Jonathan Parkinson, 2002).

2.2.3 Institutional Arrangements for Urban Drainage Planning

Ineffective solid waste management and lack of effective arrangements for drain cleaning which is likely to be related to a lack of resources and manpower, and inappropriate equipment aggravated operational problems of drainage system. But, to make matters worse, the department responsible for solid waste management is often separate from that responsible for drain cleaning and coordination between different urban authorities is generally very poor. In several cities within the developing world, there is usually no real management over new developments due to insufficiency within the administration systems for urban drainage system, (Jonathan Parkinson, 2002).

2.2.4 Implications for Policy and Practice

The research in thirty cities in Ethiopia including capital city Addis Ababa with its title, Situational Analysis of Urban Sanitation and Waste management with cross-sectional study, qualitative/ group focus discussion, depth interview and document review finds that Urban sanitation and waste management policies, strategies, plans, duties, and responsibilities in Ethiopian towns have definite gaps and overlaps. Waste management in urban areas is poorly harmonized and hard to systematically regulate due to replication of effort and indistinct roles among various stakeholders. At the town/city level there is overlap in responsibilities for activities, low assessment mechanisms, and poor supportive mechanisms that could bring into line the national and regional policies, strategies, and plans for USWM services (John Snow. Inc., 2015). Jonathan Parkinson said that, for investments in urban drainage systems to be gainful and operationally sustainable, greater emphasis needs to be placed upon communication between professionals responsible for drainage, to promote the effectiveness of storm water management systems can be directly linked to the effectiveness of urban management. Effective drainage area and catchment planning require careful coordination between the related institutions accountable for water supply, sewerage, drainage and solid waste management. The overall planning framework needs to be considered in relative to land use in the urban areas and, in particular to the community that dwell in this land. In order to achieve this teamwork between government agencies and non-governmental organizations; as well as discussion with the community is fundamental (Jonathan Parkinson, 2002). In the international Journal of Water Resource “An Approach to Drainage system sustainability in Wolaita Soddo town, A case study from Southern Ethiopia”, by using Descriptive survey, structured questions and interview, lack of skilled man power, poor integration of stakeholders, low level community participation, constraints of budget and absence of drainage system network plan in the town were prerequisite for drainage ineffectiveness, (Mitiku and Mekdes, 2017).

2.3 Impact of Poor Urban Drainage

2.3.1 Environmental Health

The study in developing countries, particularly in Brazil, by exploitation of GIS revels that, the consequences of poor drainage have major implications on the health and livelihood of urban populations. In poorly drained area urban runoff mixes with waste products from overflowing latrines and sewers inflicting pollution and resulted in to the high risk of water born diseases, gastrointestinal malady, enteric worm infections like roundworm, hookworm and breading site for mosquitoes that are responsible for a number of variety of disease (Toryila T.M et.al, 2016) and (Jonathan Parkinson, 2002) and In developing countries e.g. Sululta town in Ethiopia, Upper respiratory tract infection, diarrhea, skin diseases, eye diseases, and typhoid were primary causes of death (John Snow. Inc., 2015). In the UK, by using the models like GIS, computational local models, the roof drainage model, the study reveals that, the impact of flooding on human health is significant, especially where property is flooded. Categories for this are: direct mortality and injuries; infection; and mental health effects (Richard Ashley et.al, 2008).

2.3.2 Infrastructure

According to Najda Kasich, One of the chief functions of urban infrastructure services is to try to isolate human settlements from climate influences. So as ecosystems respond to climate change, infrastructure will be stuck by that response. The potential exists for disruption of one variety of service if another is non-continuous as a result of hydrologic cycle. Infrastructure and its users can involve increasing vulnerabilities due to climate change; they involve flooding associated with rising sea levels and intense precipitation as well as persistent heat from rising temperatures (Najda Kasich et.al, 2017).

2.3.2.1 Transportation System

The review on the result of poor drainage system on road pavement tells as, acute case happens on the road; it results accidents on the road users and harm the road infrastructure itself if structural section contains free water. The gradation and properties of layer materials rarely allow the layer to be a good drain layer resulting in defense of water layer inside the pavement inflicting a "bathtub" state, leading to early failure and chronic pavement distresses (Toryila T.M et.al, 2016). Urban voidance issue in capital of Bangladesh city People's Republic of Bangladesh, Roadways made from concrete was buckle or "explode" and roads with asphalt was soften and deteriorate earlier. Interruption of traffic movement is a crucial known impact that arises because of standard water work drawback created from inappropriate system (Alom MM., 2014). and also the study entitled Building Knowledge for a Climate change (BKCC) Adaptable Urban Drainage- Addressing change in intensity occurrence and Uncertainty of Storm (AUDACIOUS) in UK by exploitation GIS for flood mapping, rational method to estimate the runoff shows that, There might also be indirect effects like disruption to infrastructure networks resulting in associated degree inability to trip work, etc. (Richard Ashley et.al, 2008)

2.3.3 National Economy

Poor avoidance quality on roads leads to a substantial quantity of pricey repairs or early replacements. The poor urban drainage causes the injury in road and an urban drainage system itself that have a consequence on the financial system of the country. The research conducted on performance assessment of urban drainage system/ case study: District 10 of Tehran Municipality/Iran by using SWMM version 5, IDF curve to draw rainfall and Horton equation to have rainfall-runoff relation and dynamic wave models says that, the drainage network reliability is also acceptable, and although should be improved through rehabilitation. Applying the performance indicators in rehabilitation process is accommodating to understand how much of a measure is required for improving the circumstance of each part of the system, and to give a sight to decision makers about the fitting costs of each measure and budgets for rehabilitation (Begin Dinesh et.al, 2016).

2.4 Review of Hydrologic/ hydraulic models

To select the appropriate model for analysis, hydrologic and hydraulic models are reviewed in table 2-3.

Table 2—3: review of the hydrologic/ hydraulic models

Types of models	**Introduction, description/input parameters, application and limitations**
Rational method	Introduced by Kuichling in 1889, Input parameters: Catchment area, watercourse length, average slope, catchment characteristics, and rainfall intensity. Application: Suitable for small impermeable urban area, recommended maximum area 50ha. PMF Limitations: Pervious area, Return period of flood that could be determined (years)2-200
SCS- CN method	Introduced in USDA, SCS, Input parameters: Catchment area, watercourse length, length to catchment centroid (centre), mean annual rainfall, vegetation type, soil cover and synthetic regional unit hydrograph. Application: Suitable for, areas with any abstraction Recommended maximum area ($0.5 – 65km^2$). Limitations: Basin should have fairly homogeneous CN values• CN should be 40 or greater, tc should be between 0.1 and 10 hr, Ia/P should be between 0.1 and 0.5, Basin should have one main channel or branches with nearly equal times of concentration, Neither channel nor reservoir routing can be incorporated, Fp factor is applied only for ponds and swamps that are not in the tc flow path
HEC-HMS	Description: Conceptual model, hydrologic flood event routing, Application: Calculate water surface profiles for steady state, gradually varied flow in natural and manmade channels, sub/super critical flow profile, effects of obstructions such as, bridges, culverts, weirs, structures in flood plans, hydraulic routing(storage), Limitations: Water quality, BMP evaluation, integration with GIS/CAD
HEC-RAS	Introduced: U.S corps of Engineers hydrologic Engineering center 1998, description: One dimensional open channel steady state hydraulics, Application: Hydraulics of Open channel, waterways, culvert, bridge, structures. Integrate with GIS, Limitation: Flood routing, hydraulics of pipe system, storage routing, water quality, planning BMP

MOUSE	Description: Pipe hydrodynamics, water quality, application: Hydrology: flood routing event or continuous Hydraulics: pipe system, culvert, bridge, structure, limitation: Hydraulics: storage routing, Water quality: pollutant estimation and water quality control, BMP evaluation, interaction with GIS
SWMM	Introduced: EPA 1971, Application: Integrated hydrology, pipe and open channel hydraulics, pumps, water quality, BMPs, Hydrology: event or continuous flood routing Hydraulics: open channels, waterways, pipe systems, storage routing Water quality: pollutant estimation and water quality control, BMP evaluation, Limitation: Hydraulics: culvert and bridge structure
GIS	Introduced: U.S. and Canada in the mid-1960s, Input parameters: Satellite images, DEM, spatial data from the real world, Application: Provide water shed physical feature, Compute hydrologic model input parameters such as catchment area, model the rainfall/runoff process to determine design flow, provide the capability for screen design of the system including conveyance structures and appurtenances, optimize the final design, map or draw the system as designed, including plan and profile drawings of all structural components. Limitation: Use large set of data base
DRAINS/ILSAX and TRRL	Introduced: Australia, 1998/1993, Input parameters: Time area hydrology, storage, pipe or open channel hydraulics Catchment characteristics, travel time, characteristic of the assumed concentrated storage at the sub-area outlet, Application: Hydrology: flood routing: event, Hydraulic: open channels, waterways, pipe system, storage routing. Can model pit entry capacities, bypass flows, over flows from inlets, and rout them from one entry pi to another. Behavior of detention basin, non circular conduits and open channel. Limitations: Hydraulics of culverts, bridge, structure, water quality, BMP, interaction with GIS
AQUALM-XP	Description: hydrology , water quality, BMPs Application: Hydrology: continues flood routing, Hydraulics: daily storage routing, Water quality: pollutant estimation and water quality control, BMP planning, Limitations: Hydraulics: open channels, waterways, pipe system, culvert bridge, structure,

NB. The list does not include all available models. Other models that are not listed may be equally or more suitable for particular application.

Among these models the most used and the simplest one for urban areas runoff volume estimation better accounting the intense rainfall and urbanization is the rational formula. And arc GIS10.2, Microsoft excel, professional easy fit, Google earth pro the computer programs were used to delineate the drainage system, make calculation easy, test goodness of fit and sub-watershed divide respectively. Descriptive statistics model (GEV) used to simulate design rainfall depth.

3 MATERIALS AND METHODS

3.1 Description of Study Area

Hawassa is established in 1952 E.C during the period of Emperor Hailesilasie. The city got both its name and beauty from Lake Hawassa. The city administration has an area of 157.2 sq.kms divided in to 8 sub-cities and 32 Kebeles. These sub cities are Hayek Dare, Menehariya, Tabore, Misrak, Bahile Adarash, Addis Ketema, Hawela-Tula and Mehal sub city. Currently the city is serving as the Capital of southern Nations Nationalities & peoples Region, the Sidama Regional state & Hawassa city administration (socio-economic profile of Hawassa city, 2010E.C).

Figure 3-1: Map of Hawassa city administration (source: Google map)

3.1.1 Location

Hawassa is located in the Southern Nation’s Nationalities and Peoples Region on the shores of Lake Hawassa in the Great Rift Valley; 273 km south of Addis Ababa and 1,125 km north of Nairobi, Kenya. The city lays on the Trans-African High Way- 4: an international road that stretched from Cairo (Egypt) to Cape Town (S. Africa). The city is bounded by Lake Hawassa in the West, Oromia Region in the North, Wendo genet woreda in the East and Shebedino woreda in the south.

Geographically, the City lays between 6°55’0’’ to 7°6’0’’ latitude North and 38° 25’0’’ to 38° 34’0’’ longitudes east. The urban part of the City lies between $7^{o}0^{'}30^{"}$ to $7^{o}5'30"$ latitude North and $38^{o}28'0"$ to $38^{o}31'0"$ longitude East and specific study area lays between $7^{o}2'0"$ to $7^{o}4'0"$ latitude North and $38^{o}28'30"$ to $38^{o}30'0"$ longitude East.

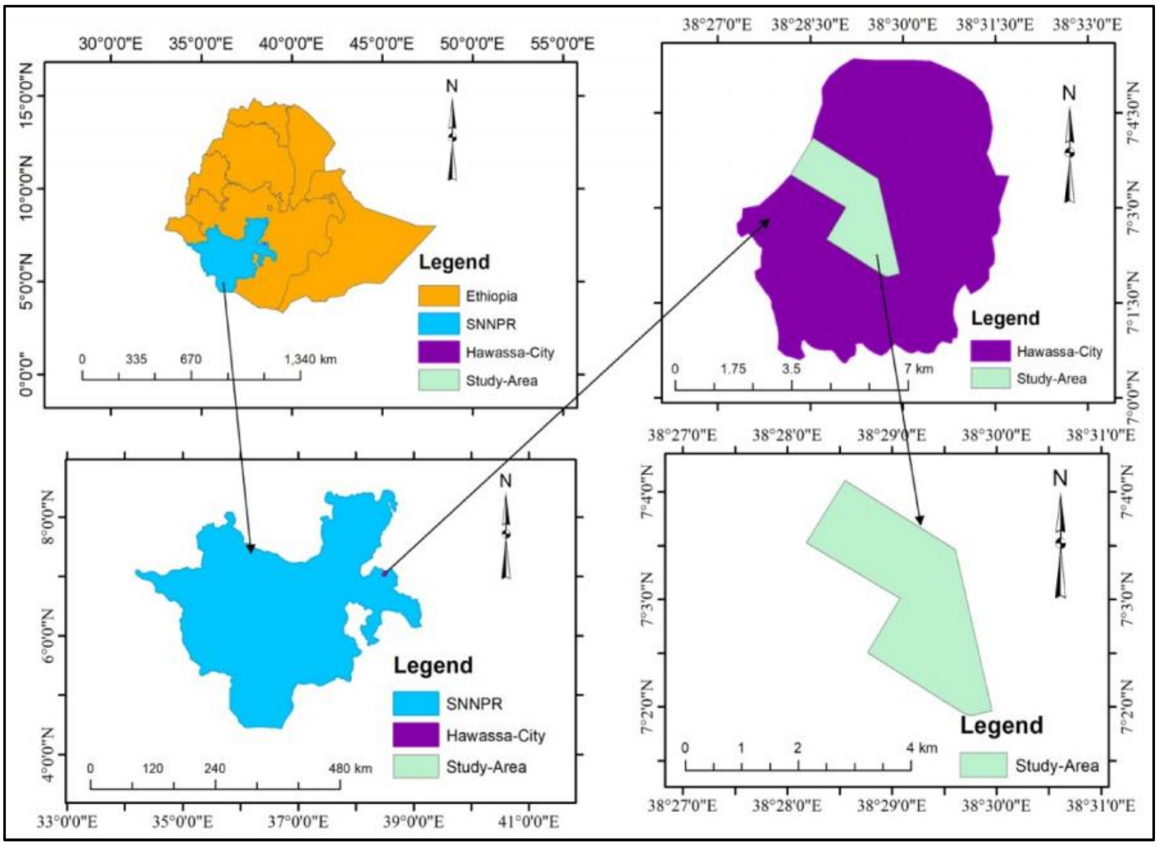


Figure 3-2: Location map of the study area

3.1.2 Climate

Rainfall

Integrated weather condition over several years is generally termed as climate. Therefore climate represents the accumulation of daily and seasonal weather events (from average range of weather) over a long period of time. So climate change means the deviation of the annual rainfall from the long term mean; and its deviation using the statistics model (standard deviation) for 31 years (1989-2019) time series rainfall and presented by figure 3-2 below.

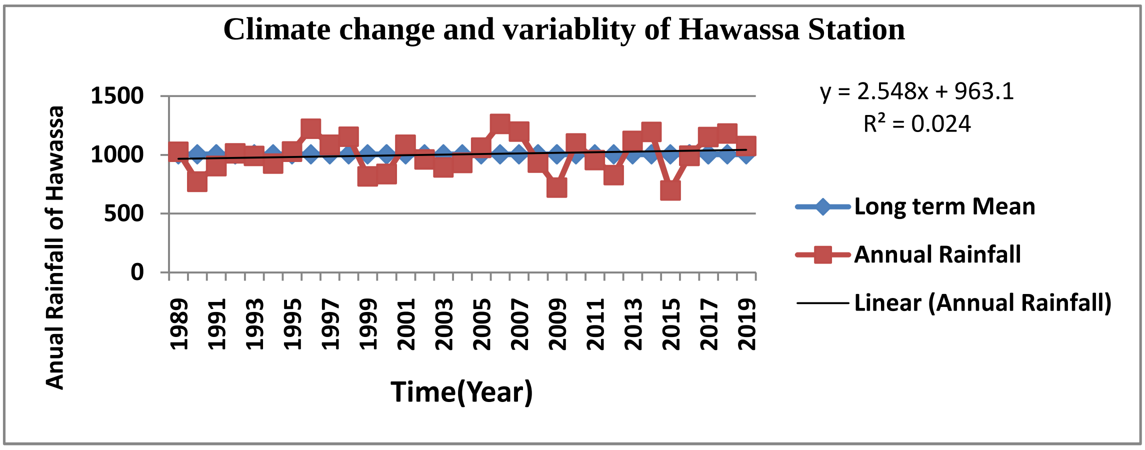


Figure 3-3: climate change and variablity of Hawassa

The statistics of the study area shows that the mean annual rainfall of the 31 years data recorded in Hawassa varies 149.638mm from the mean. I.e. there is climate variation of 149.638mm from the long term mean.

Temperature

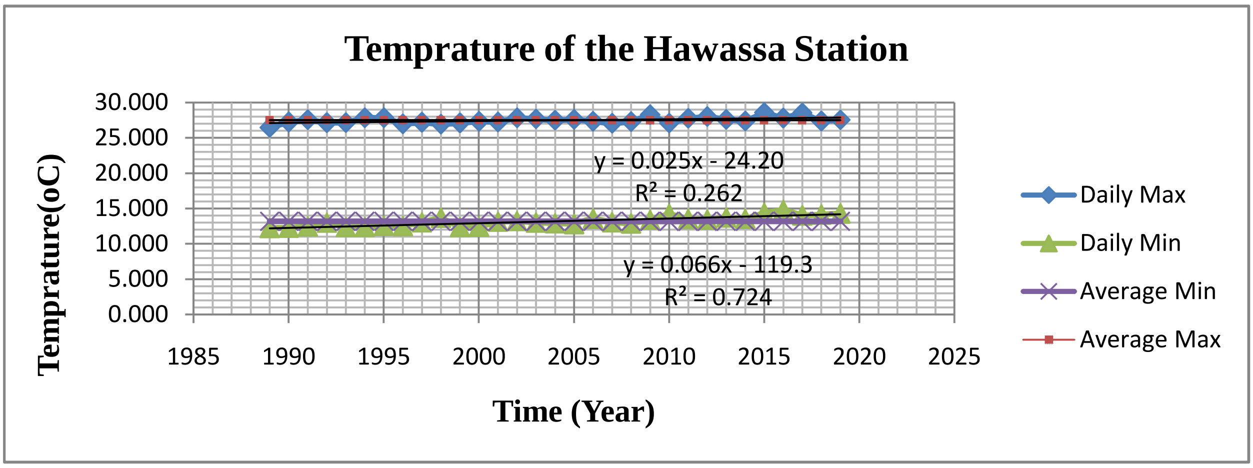


Figure 3-4: Temperature of Hawassa station from 1989 to 2019

Average daily maximum temperature over 31 years (1989-2019) is 27.486^{o}C and the standard deviation of this daily maximum is 0.458. While average daily minimum temperature is 13.202^{o}C and standard deviation is 0.706. Daily minimum temperature is more variable than that of daily maximum temperature in the study area.

3.1.3 Geology and Soil

The geology of the Hawassa area comprises pyroclastic deposits, trachyte, lacustrineand alluvial deposits, unsorted gravels, sandy gravel, clay and underlying basaltic rocks. Pyroclastic deposits are made up of ignimbrites, volcanic ash, tuff and rhyolites, which are considered to be part of the Nazareth series, Belongs to the Dino formation, which erupted following fissuring of the uplifted land mass. The urban area consists of soils with poor to moderate drainage characteristics. The prevailing soil distribution consist 40% black cotton soil, 30% red soil, 20% sandy soil and 10% silt soil which are derived from the weathering of basaltic lavas (Kirubel, 2015).

3.1.4 Land use Land Cover

Nowadays the city divided in to industrial, commercial and residential zones. Administration, recreational, asphalt, gravel and coble stone etc. are the most dominant types of land use land cover of the city. The coverage of the features is changing over the time. Land use land cover transition of the study area is given in table 3-1

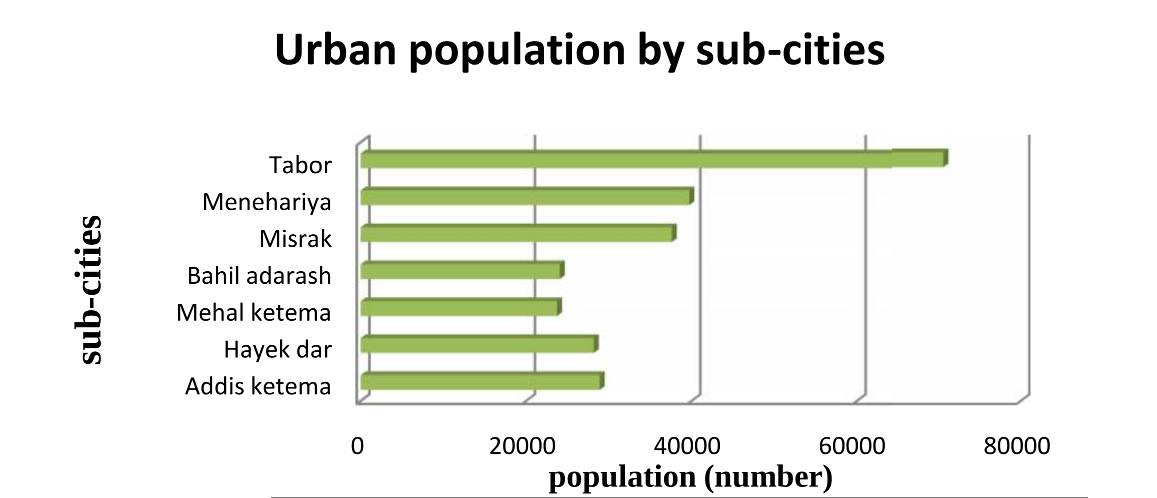
Table 3—1: land use land cover transition of the study area

Land use Land cover transition of the study area					Change of land use	
land use type	Area (ha) (2002)	Percentage (%)	Area (ha)(2019)	Percentage (%)	Area (ha)	Percentage (%)
Residential	133.14	23.22	145.75	25.41	12.61	9.47
Commercial	35.59	6.21	84.87	14.8	49.28	138.47
Mixed resident	80.13	13.98	87.79	15.3	7.66	9.56
Administration	53.16	9.27	48.36	8.43	-4.8	-9.03
Schools	6.55	1.14	20.03	3.49	13.48	205.8
Health centres	0.61	0.11	2.66	0.46	2.05	336.07
Recreational area	5.39	0.94	36.27	6.32	30.88	572.91
Parks-cemeteries	13.26	2.31	5.89	1.03	-7.37	-55.58
Unimproved	120.5	21.02	1.14	0.2	-119.36	-99.05
Asphalt	13.36	2.33	61.31	10.69	47.95	358.91
Coble stone	0	0	56.855	9.91	56.855	100
Gravel Road	101.47	17.7	9.24	1.61	-92.23	-90.89

Swampy area	7.38	1.29	13.47	2.35	6.09	82.52
Detention storage	2.81	0.49	0	0	-2.81	-100
	573.35	100	573. 35	100		

3.1.5 Population Characteristics

Much of the population growth in Hawassa has been the result of internal migration and expansion of educational and other facilities, widening of the city’s boundaries has caused some of the increase. According to projections of the central statistics authority of Ethiopia, Hawassa’s population is estimated to be 403,025 in 2010E.C. The City’s population gender breakdown will be relatively evenly split between male (207,416, 51.4 %) and female (195,609, 48.6%). Out of the total population 266,331(66.08%) people live in urban area, while 136,694(33.92%) live in the rural area of the administration. The annual population growth rate is 4.02. 4.8% growth rate in urban and, 2.8% growth rate in rural areas of the city. The urban population by sub- cities is given in the figure 3-3



	Addis ketema	Hayek dar	Mehal ketema	Bahil adarash	Misrak	Menehariya	Tabor
Series1	28908	28186	23745	24081	37624	39737	70665

Figure 3-5**:** urban population of Hawassa city

(Source: socio economic profile of Hawassa city, 2010E.C)

3.1.6 Socio- Economic Activity

The City of Hawassa offers a remarkable array of career opportunities. The nearly 10,033 employees of the City of Hawassa engaged in different works that makes Hawassa a great place to live, works, and engage in play. Among the employees 4,813 male, 4,317 female, totally 9130 are permanent; And 562 male, 341 female, 903 totals are temporal employees. The unemployment rate measures the number of people actively looking for a job as a percentage of the labor force. These are 5308 male, 13265 female, 18573 totals.

Hawassa city has relatively an extensive and developed transport network which includes both private and public services. Roads are a critical component of any city's transportation infrastructure. The city Administration is building an improved road and transport network across the city through a range of key projects including major Asphalt road with pedestrian walkways and Coble stone road construction.

Markets, super markets, mini markets, shops and like centers in the city provide the goods for the users with plastic packs and plastic bottles. Chat and fruit trade is common in the city especially around old bus station. Hotels, cafes and restaurants, telecom, Car wash, Garages, etc. along the road sides are also the part of socio- economic activity of the city.

3.2 Data Collection and Analysis

3.2.1 Data Collected and Sources

The data used for this study was collected from Ethiopian National Meteorology agency, Ethiopian Mapping agency, Hawassa city (infrastructure, Health, urban planning, solid waste and city beautification) departments, Hawassa city water supply, sewerage and sanitation enterprise, and conducting survey in the study area. These data which used as input parameters for the selected models, types and their sources are given in the table 3-2.

Table 3—2: data collected, user models and sources of data

Data collected	Type	input for	Sources
DEM	secondary	GIS	Ethiopian Mapping Agency
LULC map/ land sat image	Primary/secondary	GIS	Remote sensing/ Google Earth
Coordinate points/topographic map	Primary/ secondary	GIS	GPS/Ethiopian Mapping Agency
Time series Rainfall / IDF curve	secondary	Rational method	National Meteorology Agency/ Ethiopian Roads Authority
Catchment area,	Primary	Rational method	Field survey/ GIS
watercourse length,	Primary	Rational method	Field survey/ GIS
average slope,	Primary	Rational method	Field survey/ contour-map
catchment characteristics,	Primary	Rational method	Field survey, GIS, Google Earth
Dimensions /geometry/	Primary	Manning	Field survey
Note: secondary Data related with Health is collected by reviewing the recorded documents from Hawassa city Health department			

3.2.2 Data Analysis method

The data gathered in above section are analyzed and presented in suitable forms such as tabular, graphical, figurative, etc. approach with the conclusions and recommendations

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

- Check the quality of data
- Design rainfall frequency analysis by using statistical model known as generalized extreme value method
- Estimate rainfall for shorter duration using equation provided by ERA DDM (2013)
- Develop IDF curve for 2, 5, 10, 25, 50, 100 and 200 years return period
- Determine the time of concentration for each segment of drainage system
- Determining the rainfall intensity for the determined time of concentration for each segments
- Delineating watershed using DEM and ArcGIS10.2
- Determine the weighted runoff coefficient for the representative area and Chose the representative frequency factors for return periods.
- Apply the hydrologic model (rational method) to compute the peak flood.

- Mapping and modeling drainage system flow path networks
- Finally check the adequacy, size the structures by using manning's equation and design regional retention pond at the shore of Lake Hawassa.

3.3 Materials and Computer Programs

Materials used for this study are:

- GPS: to collect the coordinate points GPS named Garmin-72H is used.
- DEM: is an input data for ArcGIS10.2 for catchment delineation and estimation of catchment characteristics
- Contour map: determine elevation, flow direction and length, of the catchment area
- Cell phone: to capture the images during site observation
- Measuring tape: to measure the geometry of existing drainage system

Computer models and software's

- Professional easyfit5.6: to estimate the parameters and statistics of the rainfall distribution
- ArcGIS10.2: to obtain hydrological and physical parameters and spatial information of the catchment area.
- Google Earth pro: To verify watershed and divides of catchments of study area.
- Microsoft excel spread sheet : to make calculations easy

3.4 Estimation of the runoff volume

3.4.1 Hydro climatic data and its quality

3.4.1.1 Rainfall data

To estimate the runoff volume with respect to climate change and urbanization, the time series rainfall data of 31 years (1989-2019) is collected from the National Meteorological Agency in Hawassa, this is registered in database used as input parameter for statistical models. Anyhow in the city there are two stations one Hawassa synoptic at Mobill and the other one at mount Tabor. For this research Synoptic one is used due to its year of establishment which fulfills the minimum requirement for climatic analysis (30 years). To fill the missed rainfall at station other three stations fulfill the minimum requirement such as Halaba Kulito, Morocho and Wendo Genet were used.

3.4.1.2 Missing data filling

Due to the absence of data attendant or instrumental failure rainfall data record occasionally are incomplete. In such a case one can estimate the missing data by using the nearest stations rainfall data. Among different approaches normal ratio method has chosen for the regions where annual rainfalls between stations differ by more than 10%, (Biniyam A., 2016).

$$P_X = \frac{1}{3}\left\{P_1 \frac{N_X}{N_1} + P_2 \frac{N_X}{N_2} + P_3 \frac{N_X}{N_3}\right\} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 3.1$$

Where

P_X= the missing rainfall data (daily, monthly, yearly)

P_1 , P_2 P_3 Rainfall data at nearest station (daily, monthly, yearly)

N_X - mean annual rainfall data at missed station

N_1, N_2, N_3 - Mean annual rainfall at different nearest stations

3.4.1.3 Data quality checking

Checking for outliers

Outliers in maximum rainfall can play a considerable role in unreal analysis leading to unreal predictions. An outlier can be considered an observation which abnormally or haphazardly deviates from normal status of data and analyses. As a matter of fact, one of necessary factors in abundance analysis is existence of real and long –term data where outliers may have considerable adverse effect in data prediction. Such effect leads to unreal estimation in data analysis for a given variable (Sajad Mirzael et.al, 2014).

For instance, Outlier lead to unreal probabilistic distribution, Estimation of probabilistic distributions parameters is influenced by outliers leading to faulty estimation. Low outliers cause underestimation leading to structural collapse in big floods. High outliers cause overestimation leading to higher costs of water structure building. Thus, false inputs result in false outputs so that an outlier in some statistical methods can be specifying final results. Therefore accurate statistical determination of data find outliers is very important. Among the methods the 3SD$\pm$M is used in this study (Sajad Mirzael et.al, 2014).

Checking for consistency

Adjusting for gage consistency involves the estimation of an effect rather than a missing value. An inconsistent record may result due to changes in observation procedures, change in exposure of the gage, change in land use and where vandalism frequently occurs. Double mass curve is the method that is used to check for consistency in a gage record for this study. It was determined by plotting the cumulative values of observed time series rainfall of the station for which consistency need to be checked on the Y-axis Vs the cumulative values observed time series rainfall of neighbor stations. A break in the slope of the curve would indicate that conditions have changed (inconsistency), the values of the inconsistent station adjusted by multiplying the value with the slope of change. In this study the gage was consistent and no need to adjust.

Checking for Homogeneity

A homogenous time series is defined as one in which variations caused only by the weather and climate. Climate change, flood, rainfall-runoff relationship etc. vary according to the quality of data used. Therefore stations should be checked for homogeneity (Alaa M. et.al, 2014). To check the homogeneity of stations the average monthly rainfall in the vertical axis vs. months of a year on the horizontal axis was plotted. The plot followed the same pattern and chosen stations considered homogeneous and used for analysis.

3.4.2 Design rainfall depth and frequency analysis

3.4.2.1 Estimation of average depth of rainfall over a catchment

Variety of methods are used to estimate average depth of rainfall over a catchment, chose of method requires judgment in consideration of quality and nature of data, importance, use and required precision of the result. These methods are arithmetic mean, Thiessen polygon, and Isohyetal. Due to the year of establishment, the only used station is Hawassa synoptic from (1989 to 2019) but Tabor station established in 2006 does not fulfill the minimum requirement (30 years) for climatic analysis. Therefore the point rainfall depth represented as average depth for the study area.

3.4.2.2 Frequency analysis

Occurrence of flood is governed by chance. This chance of flooding is described by statistical analysis of flooding in subject watershed. A design frequency shall be selected to match facilities coast, amount of traffic, potential flood hazard to property, expected level of service, political consideration and budgetary constraints, considering the magnitude and risk associated damages from large flood events (ERA DDM, 2013). To chose this appropriate distribution model, goodness of fit test employed.

Goodness of Fit Test

Goodness of fit test can be consistently used in climate statistics to help in finding the best distribution to use to fit the given data. These tests cannot be used to select the best distribution, rather than to eliminate possible distributions. These tests compute test-statistics, which are used to analyze how well the data fits the given distribution. These tests describe the differences between the observed data values, and the expected values from the distribution being tested (Nick Millington et.al, 2011). The Anderson-Darling (AD), Kolmogorov-Smirnov (KS) and Chi-Squared($x^{2)}$ tests were used for the goodness of fit tests in this study.

I. Anderson-Darling (AD) Test

The AD test compares an observed CDF to an expected CDF. This method gives more weight to the tail of the distribution than KS test, which in term leads to the AD test being strengthen and having more weight than the KS test. The test rejects the hypothesis regarding the distribution level if the statistics obtained is greater than a critical value at a given significance level (α) (Nick Millington et.al, 2011).

II. Kolmogorov-Smirnov Test (KS)

The KS analysis statistics is based on the maximum vertical distance from the empirical and theoretical CDFs. Similar to the AD analysis statistics, a hypothesis is rejected if the analysis statistics is greater than the critical value at a chosen significance level of α = 0.05. The samples are assumed to be form a CDF F(x) (Nick Millington et.al, 2011).

III. Chi-Square Test

This test is used to determine if a sample comes from a given distribution. It should be noted that this is not considered a high power statistical test and is not very useful (Nick Millington et.al, 2011). The test result of this method is rejected.

3.4.3 Statistical Distribution

The GEV distribution used in this study has a wide variety of applications for estimating extreme value of given data sets. This distribution is commonly used in hydrological application (Nick Millington, 2011). The following sections explain the theory of distribution.

Generalized Extreme Value Distribution (GEV)

Nick Millington described the GEV distribution as a family of continuous probability distribution that combines the Gumble (EV1), Frechet (EV2) and Weibull (EV3) distribution. Generalized Extreme Value distribution uses parameters such as location, scale and shape. The location parameter describes the change of a distribution in a given direction on horizontal axis. The scale parameter describes how extend out the distribution is, and defines where the mass of the distribution lies. As scale parameter increases, the distribution will become more spread out. The third parameter in GEV family is the shape parameter, which severely affects the shape of the distribution and governs the tail of each distribution. The shape parameter is resulting from skewness as it represents where the bulk of the data lies, which creates the tail(s) of the distribution. When shape parameter ($k = 0$) this is EV1, when ($k > 0$) this is EV2 and when ($k < 0$) this is EV3. The shape parameter for GEV can greatly affect the results. A positive shape parameter will result in the distributions being upper bounded; this parameter is undesirable in practical applications as this produce very minimal difference in extent between large return periods. A negative shape parameter assures that the distribution is unbounded and results in an increase in magnitude, as the return period gets larger (Nick Millington et.al, 2011). When designing for extreme events, we are looking for these large values. The CDF and PDF are defined as

$F(x) = exp\{-(1 - k\frac{(x-£)}{\alpha})^{\frac{1}{k}}\}$ ……………………………………… 3.7

$f(x) = \alpha^{-1}exp\{-(1-k)y - \exp(-y)\}$ …………………………...3.8

Where $y = -k^{-1}log\left\{1 - k\frac{(x-£)}{\alpha}\right\}$ when $k \neq 0$

Where

$£\ is\ location\ parameter\ \alpha\ is\ scale\ parameter\ and\ k\ is\ the\ shape\ parameter$

Parameter Estimation Techniques

L- Moment parameter estimation method was used. These are based on probability weighted moments, however provide a greater degree of accuracy and easy. Probability weighted moments use weights of the cumulative distribution function (CDF). L-moments are the modification of Probability Weighted Moments as they use the Probability Weighted Moments to analyze parameters that are easy to interpret and that can be used in calculation of parameters for statistical distributions. L-moments are based on linear combinations of data that have been arranged in ascending order. They provide an advantage, as they are easy to work with and more reliable as they are less sensitive to outliers. The method of L-moments calculates more accurate parameters than that of Method of Moment techniques for smaller sample sizes. And also L-moments can be more widely used, and are also nearly unbiased (Nick Millington et.al, 2011).

Probability Weighted Moment Equations

According to (Nick Millington et.al, 2011), Probability Weighted Moments are needed for computation of L-moments. The data first must be arranged in ascending order, and then apply the following equations.

$M100 = sample\ mean = \frac{1}{N}\sum_{i=1}^{N} Pi$ ………………………………… 3.9

$M110 = \frac{1}{N}\sum_{i=1}^{N} \frac{(i-1)}{(N-1)} Pi$ ……………………………………………… 3.10

$M120 = \frac{1}{N}\sum_{i=1}^{N} \frac{(i-1)(i-2)}{(N-1)(N-2)} Pi$ …………………………………………3.11

$M130 = \frac{1}{N}\sum_{i=1}^{N} \frac{(i-1)(i-2)(i-3)}{(N-1)(N-2)(N-3)} Pi$ ……………………………………3.12

In which N is the sample size, Pi is the data value and i is the rank of the value in ascending order.

L- Moment equations

L-moment equations are as given below

λ1= L1 =M100 ………………………………………………………… 3.13

λ2= L2 =2M110 - M100 ………………………………………………..3.14

λ3= L3 =6M120 - 6M110 + M100 …………………………………..3.15

λ4= L4 =20M130 - 30M120 + 12M110 - M100 …………………….. .3.16

τ2=L2/L1 (L-CV) …………………………………………………… 3.17

τ3= L3/L2 (L-Skewness) …………………………………………… 3.18

$\tau4 = L4/L2$ (L-Kurtosis) ……………………………………………….. 3.19

Kurtosis is the measure of the "peakdness" of distribution

The 4L-moments (λ1, λ2, λ3 and λ4) are all derived using the 4PWMs.

Generalized Extreme Value Distribution (GEV) model

As stated, the GEV distribution uses these parameters £, is the location parameter α, the scale parameter and k, the shape parameters. The parameters are defined as

$$k = 7.8590c + 2.955\,c^2 \quad \text{……………………………………….. 3.20}$$

in which $c = \frac{2}{3+\tau3} - \frac{ln2}{ln3}$

$$\alpha = \frac{\lambda 2k}{(1-2^{-k})\Gamma(1+k)} \quad \text{…………………………………..………..3.21}$$

$$£ = \lambda 1 - \alpha\left[\frac{1-\Gamma(1+k)}{k}\right] \quad \text{……………………………………………….. 3.22}$$

In which Γ = the gamma function

Once all parameters have been estimated, calculating the T- year return precipitation (Pt) can be done using

$$P_t = £ + \left(\frac{\alpha}{k}\right)\left[1 - (-log\,\frac{(T-1)}{T})^K\right] \quad \text{……………………………………… 3.23}$$

Step by step procedure for the estimation of the GEV parameter

- Sort the data set by ordering all of the data points in ascending order(lowest to highest)
- Calculate the 4PWMs (M100, M110, M120 M130)
- Calculate the 4L-Moments (λ1, λ2, λ3 and λ4) using the PWMs
- Calculate K, the shape parameter
- Calculate £- the location parameter and α-scale parameter
- Using the desired return period, apply all parameters to the return period equation to calculate the estimated return value.

To choose suitable statistical distribution model the computer program called Easy fit 5.6 available at htpp://www.Mathwave.com/easyfit-distrbution-fitting.html. Was used

3.4.4 Rainfall Intensity

The rainfall intensity (I) is the average rainfall rate in mm/hr for duration equal to the time of concentration for a selected return period. Once a particular return period has been selected for design and a time of concentration calculated for the catchment area, the rainfall intensity can be determined from Rainfall-Intensity-Duration curves. For drainage areas in Ethiopia, you may compute the rainfall intensity at any required time using the 24hr rainfall depth, which is known as a rainfall intensity-duration-frequency (IDF) relationship (ERA DDM, 2013).

$$R_{Rt} = \frac{t}{24} \frac{(b+24)^n}{(b+t)^n} \qquad \text{3-24}$$

Where:

R_{Rt}: Rainfall depth Ratio R_t: R_{24}

R_t: Rainfall depth in a given duration (t)

R_{24}: 24 hr rainfall depth

b and n: coefficients b=0.3 and n=(0.78-1.09)

Rearranging equation 3-24 we obtain R_t as

$$R_t = \frac{t(b+24)^n}{24(b+t)^n} * R_{24} \qquad \text{3-25}$$

Now by definition, rainfall intensity (I) is the ratio of rainfall to the duration of that rainfall,

$$I = \frac{R_t}{t} = \frac{R_{24}(b+24)^n}{24(b+t)^n} \text{ (mm/hr)} \qquad \text{3-26}$$

Using b = 0.3 and n = 0.92 as suggested by ERA drainage design manual, results are tabulated for rainfall duration 5, 10, 15, 20, 25, 30, 60, 90, 120, 150 and 180 minutes in the result and discussion part of this paper.

3.4.5 Time of Concentration

Description

Time of concentration (T_c) is used in the rational method to determine the critical rainfall duration, which can then be combined with appropriate rainfall intensity duration (IDF) relation to establish the required design rainfall intensity. The T_c is the time required for water to flow the most remote point of the basin to the location being analyzed. A storm equal to this duration will permit direct runoff to arrive from all points in the water shed concentrating at the outlet. This time measure is taken to be the critical time by many flood estimating approaches. A shorter time, although resulting in higher rainfall intensity, will not permit the entire basin to contribute flow simultaneously. A longer duration allows the entire basin to contribute, but with a lower intensity (ERA DDM, 2013).

Flow path consideration

There may be a number of possible paths to consider in determining the longest travel time. Identify the flow path along which the longest travel time is likely to occur. Generally; it is reasonable to consider the three following components of flow that can characterize the progression of runoff along a travel path: overland flow (sheet flow), shallow concentrated flow, and conduit and open channel flow (concentrated channel flow) (ERA DDM (2013).

Peak discharge adjustments

In few cases, the runoff from a portion of the drainage area that is tight may result in greater peak discharge than would occur if the whole area were considered. In these occasions, it is possible to adjust the drainage area and time of concentration by disregarding those areas where flow time is too slow to add to the peak discharge. Sometimes it is essential to estimate several different contributing areas and associated times of concentration to determine the design flow that is critical for a particular application (ERA DDM, 2013).

Overland flow path selection

In drainage system design, the over land path is not necessarily perpendicular to the contours shown on available mapping, often the land will be graded and swales and streets will intercept the flow that reduces the time of concentration. Care should be exercised in selecting overland flow paths in excess of 60m in urban areas and 120m in rural areas (ERA DDM, 2013).

Calculation of time of concentration

There are several methods that can be used to estimate time of concentration (t_c), some of which are proposed to calculate the flow velocity which in individual segments of the flow path, e.g. overland (sheet) flow, shallow concentrated flow, open channel flow. The time of concentration can be calculated as the sum of the travel times within the various consecutive flow segments (ERA DDM, 2013).

I. Sheet flow travel time

According to (ERA DDM, 2001) Sheet flow is the shallow mass of runoff on a plane surface with a uniform depth across the sloping surface. This usually occurs at the headwater of streams over relatively short distances, rarely less than about 100m, and possibly less estimated with a version of the kinematic wave equation, derivative of Manning's equation as

$$T_{ti} = \frac{0.091}{P_2^{\,0.5}} \frac{(nL)^{0.8}}{S^{0.4}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..3.27$$

Where

T_{ti}= sheet flow travel time (hr), n= manning's roughness coefficient

L = flow length (m), P_2= 2 year 24 hour rainfall (mm)

S= surface slope (m/m)

II. Shallow concentrated flow velocity

After a maximum distance of at most 100m, sheet flow tends to concentrated in rills and then gullies of increasing proportions flow is usually referred to as shallow concentrated flow (ERA DDM 2001).The velocity of such flow can be estimated using a relationship between velocity and slope as follows.

Unpaved surface $V = 4.9178S^{0.5}$ ……………………………………3.28

Paved surface $V = 6.1961S^{0.5}$ …………………………………….. 3.29

Where

V= velocity (m/s)

Sp = slope (percent)

After calculating flow velocity, shallow concentrated flow time is calculated using equation 3.31

III. Open channel and pipe flow velocity

Cross-section, geometry and roughness should be obtained for all channel reaches in the watershed (ERA DDM, 2013). Manning's equation can be used to estimate average flow velocities in pipes and open channels as follows

$$V = \frac{1}{n} R^{\frac{2}{3}} S^{0.5} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..3.30$$

Where

n = roughness coefficient (table 3-4)

V = velocity (m/s)

R = hydraulic radius (defined as the flow area developed by the wetted perimeter (m)

S = slope (m/m)

For a circular pipe flowing full, the hydraulic radius is one- fourth of the diameter. For a wide rectangular channel ($W > 10d$), the hydraulic radius is approximately equal to the depth, the travel time is then calculated as follows

$T_{ti} = \frac{L}{60V}$ …………………………………………………………...3.31

Where Tti = travel time for segment (min)

L= flow length for segment (m)

V = velocity for segment (m/s)

3.4.6 Hydrological modeling using ArcGIS10.2 and DEM

3.4.6.1 Catchment area delineation

Catchment characteristics were determined by delineating the area by incorporating DEM and Arc GIS10.2. Geographical information system (GIS) introduced in U.S. and Canada in the mid-1960s enables the user to incorporate a wide range of spatial information about the physical system into a computer database. This can include not only information about ground surface, but details of the urban infrastructure (water, waste water, electricity, streets etc.). Rapid developments are occurring in the GIS field in order to integrate all the elements described above into a computer mapping; and hydrology analysis and design package that can, Provide water shed physical feature; Compute hydrologic model input parameters such as catchment areas; model the rainfall/runoff process to determine design flows; Provide the capability for screen design of the system including conveyance structures and appurtenances; Optimize the final design; Map or draw the system as designed, including plan and profile drawings of all structural components.

Runoff coefficient (C_w) determination

Proper selection of the runoff coefficient which is variable of rational method requires decision and skill of the hydrologist. The proportion of the total rainfall that will reach the storm drains depends on the percent imperviousness, slope, and detention character of the surface. The runoff coefficient accounts for the effect of infiltration, detention storage, surface retention, Evapotranspiration, flow routing and interception. Google earth pro Software used to divide sub-watershed. The weighted runoff coefficient is determined as follows

$$C_w = \sum_{i=1}^{N} \frac{A_i c_i}{A_i} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \; 3.32$$

Where C_w = weighted runoff coefficient

C_i = coefficient of runoff for part of the drainage area

A_i = parts of drainage areas with different runoff coefficients.

Table 3—1: recommended runoff coefficient C for various land uses

Description of area	Runoff coefficient
Business: downtown area	0.7-0.95
Neighbourhood areas	0.5-0.7
Residential: single family areas	0.3-0.5
Residential: multi units, detached	0.4-0.6
Residential: multi units, attached	0.6-0.75
Suburban	0.25-0.4
Residential (0.5hectare lots or more)	0.3-0.45
Apartment dwelling areas	0.5-0.7
Industrial: light areas	0.5-0.8
Industrial: heavy areas	0.6-0.9
Parks, cemeteries	0.1-0.25
Play grounds	0.2-0.4
Railroad yard areas	0.2-0.4
Unimproved areas	0.1-0.3

(Source: ERA DDM 2001)

Table 3—2: coefficients for composite runoff analysis

Surface	Runoff coefficient
Street: Asphalt	0.7 - 0.95
Concrete	0.8 - 0.95
Drives and walks	0.75 - 0.85
Roofs	0.75 - 0.95

(Source: ERA DDM, 2001)

3.4.7 Hydrology model to better account for intense rainfall and urbanization effects

There is a need to relation better for the changes in the way in which rainfall will runoff surfaces in urban areas as the intense rainfall and urban tight area. Presently the rational runoff equation for the runoff from impervious surfaces is logically well developed and tested. However, the application of even this type of simple relationship to drainage in small urban areas is complex, especially where there are permeable areas and local surface storage. Where large areas are being drained, changes in the soil moisture between and during storms and longer term changes in flora, will affect both the volumes and rates of runoff. Consequently until these have been better defined, it is recommended that simple rational equation type approaches are used for local area drainage assessments, with runoff coefficients being selected. The simplest form of relationship between rainfall intensity and the rate at which rainfall runs off a surface is the rational formula (Ashley et.al, 2008). This Method is most accurate for estimating the design storm peak runoff for areas up to 50 hectares (0.5 km2) (ERA DDM, 2013). First the model introduced since 1889 by Kuichling, is still widely used. Even though it has come under frequent appreciation for its simplistic approach, no other drainage design method has achieved such wide use. The rational formulas estimates the peak rate of runoff at any location in a catchment area as a function of the catchment area, runoff coefficient, and mean rainfall intensity for a duration equal to the time required for water to flow from the most remote point of the basin to the location being analyzed (time of concentration).

The rational formula is expressed as:

$$Q = 0.00278\, CIA \quad \text{………………………………………………………. 3-33}$$

Where:

Q = maximum rate of runoff, m^3/s

C = runoff coefficient representing a ratio of runoff to rainfall

I = average rainfall intensity for a duration equal to the time of concentration, for a selected return period, mm/hr

A = catchment area tributary to the design location, ha

Infrequent Storm

Adjustment of the coefficient for less frequent and higher intensity storms should be necessary, for infiltration and other losses have proportionally minor effect on runoff. The adjustment of the Rational Method for use with major storms can be made by multiplying the right side of the rational method by a frequency factor Cf, ERA DDM (2013). The rational formula now becomes:

$Q = 0.00278\, C C_f\, IA$ …………………………………………………….. 3-34

Note: The product of C_f times C shall not exceed 1.0.

Table 3—3: frequency factor (Cf) for rational formula

Recurrence interval(years)	frequency factor(Cf)
< 25	1
25	1.1
50	1.2
100	1.25

(Source: ERA DDM, 2001)

Steps to Peak flood Estimation using the Rational Method

The following procedure outlines the rational method for estimating peak discharge (ERA DDM, 2013)

1. Determine the watershed area in hectares (km^2).
2. Determine the time of concentration, with consideration for future characteristics of the watershed.
3. Assure consistency with the assumptions and limitations for application of the Rational Method.
4. Determine the rainfall IDF coefficients. Extract the Rainfall Intensity-Duration Frequency Coefficients b, and n values from the list in Hydrology according to the locality in Ethiopia and the design frequency.
5. Use Equation 3-26 to calculate the rainfall intensity in mm/hr or use developed IDF.
6. Select or develop appropriate runoff coefficients for the watershed. Where the watershed comprises more than one characteristic, you must estimate C values for each area segment individually. You may then estimate a weighted C value.

7. Calculate the peak discharge for the watershed for the desired frequency using Equation 3-34.

3.5 Evaluation of the performance of drainage system

3.5.1 GIS based flow pathway mapping and modeling

Drainage system network was modeled incorporating ArcGIS10.2 and collecting the coordinates by GPS named Garmin 72H. Where detailed topographical and drainage information exists, it is recommended that GIS approaches be used, with satellite images, or detailed local land survey information to analyze flood flow pathways and for land use changes, using Regional plans or Local Development Frameworks. Where the drainage system has already been exceeded by high flows, pathway mapping can be used to show where the rainfall excess flow will go to, providing information to assess if this is, and will in the future, remain acceptable (Richard Ashley et.al, 2008).

3.5.2 Hydraulic capacity determination

Water flows in an open channel due to force of gravity and this flow resisted by the friction between the water and channel boundary. In steady, uniform flow there are no accelerations, stream lines are straight and parallel, and the pressure distribution is hydrostatic. The most commonly used equation for checking adequacy of the existing drainage capacity for steady, uniform flow is the manning's equation given (ERA DDM, 2001).

$$Q = \frac{1}{n} A R^{\frac{2}{3}} S^{\frac{1}{2}} \quad \ldots\ldots 3.35$$

Where

Q = discharge rate m^3/s A = cross- sectional flow area m^2

R = hydraulic radius R= A/P P = wetted perimeter (m)

S = energy grade line slope (m/m) n = manning's roughness coefficient

3.6 Assessment of the possible impacts of the resulting flood

The impact of the resulting flood on socio-economic activities, transport system, environmental health is analyzed by using tables in percentage and graphs. To do so the data of property damage was collected by field survey and environmental health by reviewing the recorded documents from Hawassa city Health department. Total population who dwell in this study area (five sub-cities) is 153,373. For population that are large (>10, 000) the formula developed by (Cochran, 1963) as cited by (Glenn D. et.al, 1992) yield a representative sample for proportions.

$$n_o = \frac{Z^2 pq}{e^2}$$

If the population is small (<10,000) then the sample size can be reduced slightly. This is because a given sample size provides proportionately more information for small population than large population. Sample size can be adjusted using

$$n_o = \frac{n_o}{1 + (\frac{n_{o-1}}{N})}$$

Where

n_o - Desired sample size

Z – Standardized normal variable and its value that corresponds to 95% confidence interval equals 1.96.

P – Proportion of population to be included in the sample say (50%).

q= 1-p

N = total number of population

e- Margin of error or degree of accuracy (taken within 1% to 5%)

$n_o = \frac{1.96^2 * 0.5 * 0.5}{0.05^2} = \frac{0.9604}{0.0025} = 384.16\ say\ 385$. But according to (Glenn D. et.al,1992) for population larger than 100,000 the desired sample size 400 represent the population with e = $\pm$5%.

4 RESULTS AND DISCUSSION

4.1 Runoff volume with respect to intense rainfall and urbanization

4.1.1 Data Quality

Outlier detection

Outliers in maximum rainfall can play a considerable role in unreal analysis leading to unreal predictions .Therefore, accurate statistical determination of data find outliers is very important. Among registered time series rainfall data presented in appendix 1.1 table 1-1, and showed in fig. 4-1, in 2000 the daily max. Rainfall depth 110.4 was registered and identified as an outlier and rejected out of the data list.

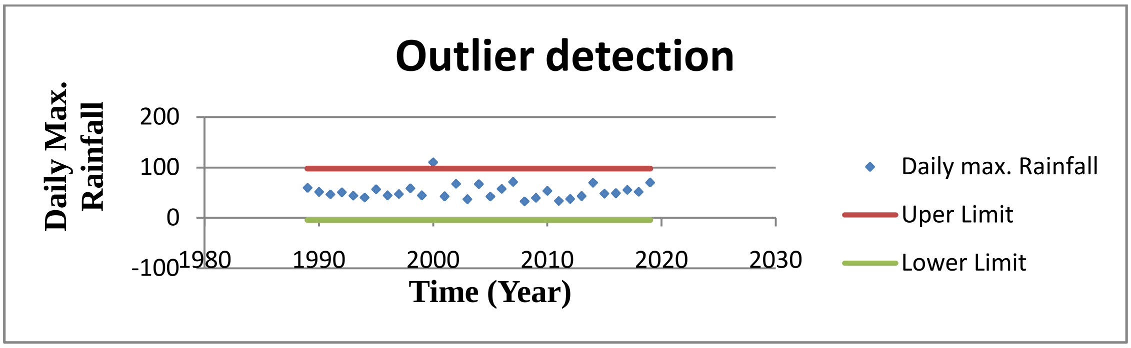


Figure 4-1: outlier detection

Consistency

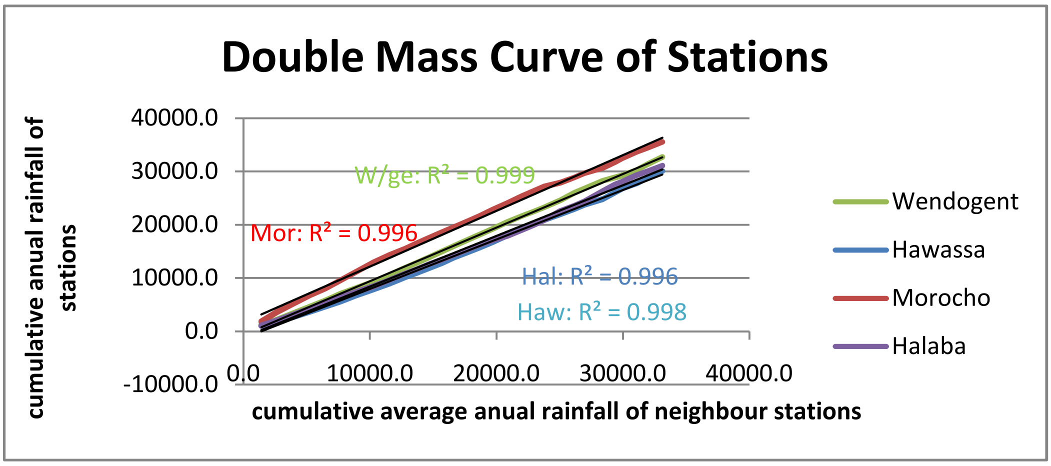


Figure 4-2: double mass curve of the stations

The double mass curves of the stations in figure 4-2 above are more of straight with no break in slope. The R^2 values for all stations are nearly 1. The change slope is not substantial. So, the data collected is consistent. The process of consistency determination is given in appendix1.1 table 1-2 and figure 1.2.

Homogeneity

The homogeneity of the selected stations for data is checked by average monthly rainfall data of the stations presented in appendix 1.1.table 1-3. Homogeneity of data is given in the figure 4-3.

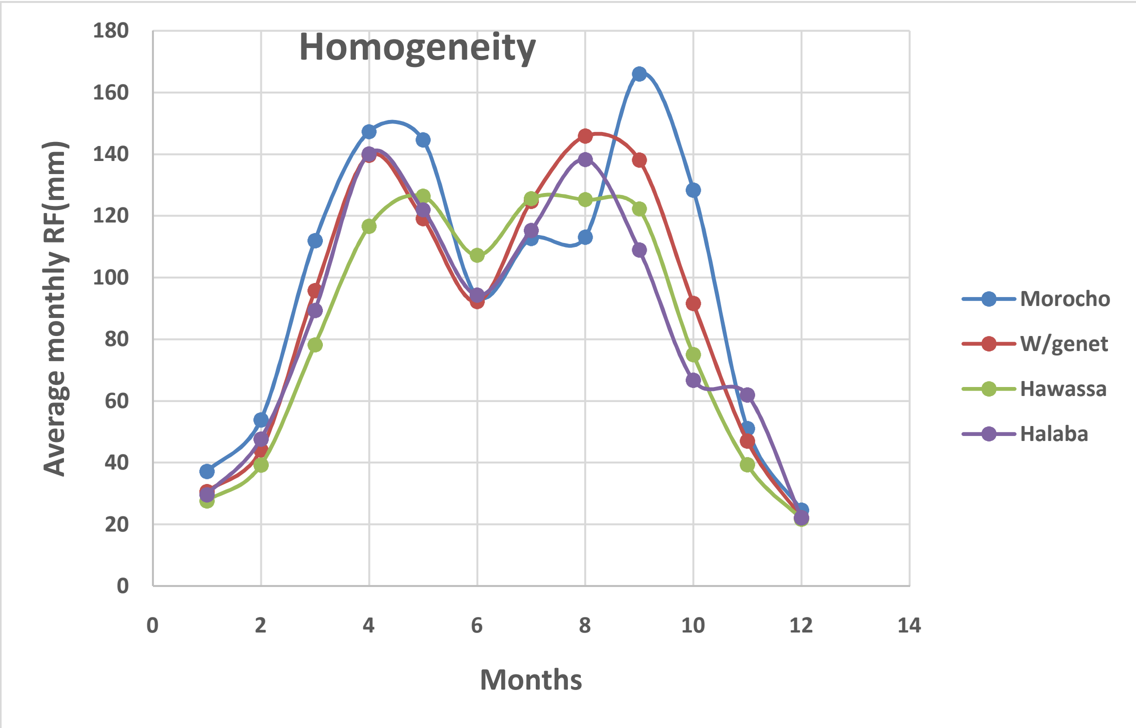


Figure **4-3:** Homogeneity of the selected stations

The data from all stations are homogeneous and Bi-modal and selected stations for analysis are assured being used for this analysis.

4.1.2 Design rainfall and frequency

4.1.2.1 Goodness of fit test

The 31 years time series rainfall data set for shorter storm durations were used in the goodness of fit test. The test results were calculated using the method described in chapter 3.2 of this report. Analyzing the goodness of fit result is a way to determine which of the distribution should not be considered. The kolmogorov-Smirnov and Anderson Darling methods are used in this study and their test result presented in table 4-2 below.

Table 4—1**:** fitting test summary

#	Distribution	Kolmogorov Smirnov		Anderson Darling	
		Statistic	Rank	Statistic	Rank
21	Gen. Extreme Value	0.066	1	0.16686	1
15	Fatigue Life	0.07061	2	0.17713	3
3	Burr (4P)	0.07177	3	0.19087	15
60	Weibull (3P)	0.07193	4	0.1917	16
48	Pearson 6	0.07213	5	0.19256	17
38	Log-Pearson 3	0.07233	6	0.17089	2
40	Lognormal	0.07238	7	0.17718	4

As described in section 3.2 above AD test compares an observed CDF to expected CDF. The test rejects the hypothesis regarding the distribution level of the statistics obtained is greater than a critical value at a given significance level ($\alpha = 0.05$). In table 4-2 above AD test statistic value for GEV is (0.16686) which is less than other statistics of statistical distributions. The test rejected GEV least frequently and is acceptable.

KS test is based on the greater vertical distance from the empirical and theoretical CDFs. The test rejects the hypothesis as the same as AD test. In the above table KS test statistic value is (0.066) which approaches to the significance level and the test rejects GEV distribution only one time and GEV is the best distribution statistic which the used data fits. Quantile-Quantile (Q-Q) plot is the graphical technique for determining if two data sets come from population with a common distribution. And it is the plot of the quantiles of the first data set against the quantiles of the second data set. A 45 – degree reference line is also plotted. If the two sets come from a population with the same distribution, the points should fall approximately along this reference line. Q-Q plot is Similar with probability plot (P-P) (R. Wayne Old ford, 2016).

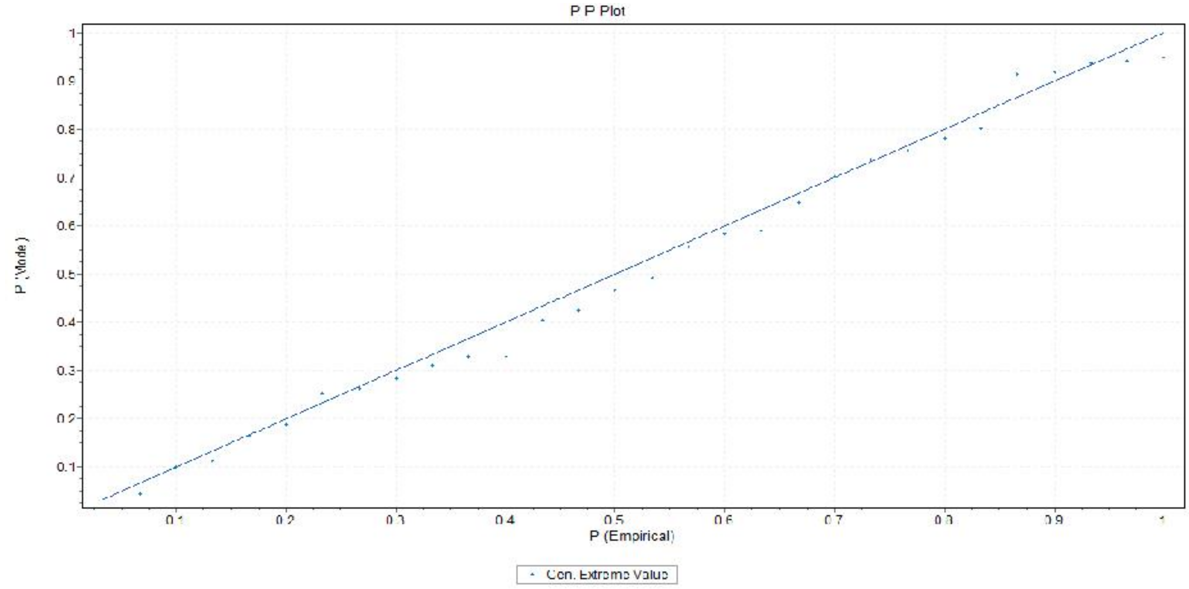


Figure 4-4: P-P plot of the study area

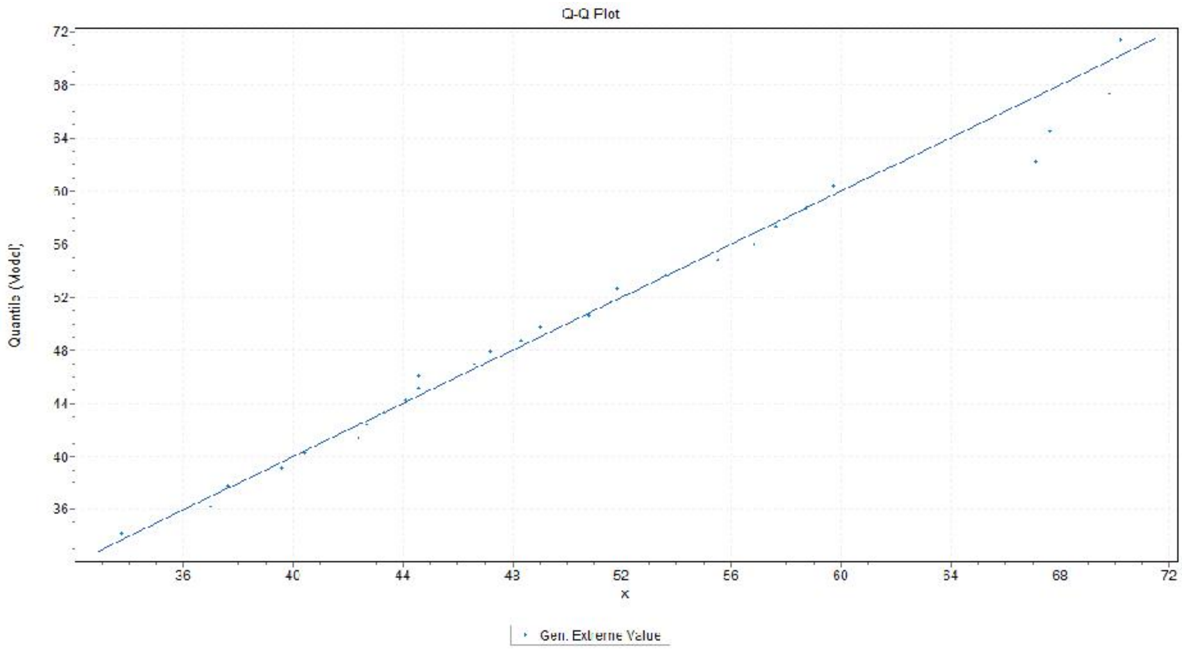


Figure 4-5: Q-Q plot of the study area

So the two plots in figure 4-3 and 4-4 are not curved and assure that

- Two data sets come from populations with a common distribution.
- Two data sets have common location and scale
- Two data sets have similar tail behavior
- Two data sets have similar distribution shape

4.1.2.2 Parameters of statistical distribution

As stated in section 3.2. GEV is three parameters model, time series rainfall data sorted in increasing order and entered in to the software named professional EASY FIT 5.6 Model give's the location (£), scale (**α**) and shape (k) parameters and statistics of the study area as given in figure 4-6 below.

Figure 4-6: statistics and GEV parameters of the study area

As presented at right top of above figure 4-5, the three parameters of generalized extreme value distribution are; location parameter (£) = 45.65 describes the shift of a distribution in a given direction on the horizontal axis. Scale parameter (α) = 9.9202 describes how spread out the distribution is, and defines where the bulk of distribution lies. As scale parameter increases the distribution will become more spread out. The third parameter of the GEV distribution is (K = _ 0.09227). This parameter affects the shape of each distribution and greatly affects the result. These parameters are given in table below in comparison with other periods.

Table **4—2**: comparison of GEV parameters for different periods

	Year		
Parameters	2002	2011	2019
Shape(K)	-0.0199	-0.0890	-0.0923
Scale(α)	5.3570	9.8455	9.9202
Location(£)	45.7840	44.6960	45.6500

Here a negative shape parameter assures that the distribution is unbounded and results in an increase in magnitude, as the return periods get large. So having this negative shape parameter is a good opportunity for analysis of design rainfall depth for different return periods with large value differences between return periods.

4.1.2.3 Design Rainfall Depth for Different Return Period

As stated earlier the 24-hour rainfall depth for different return period by GEV using its Parameters given in table 4-2 and equation 3.23 are presented in table 4-3 with that of R24 of ERA for region B2 for comparison. The value of design depth rainfall is increases as the time goes.

Table **4—3:** simulated rainfall depth for different return period and R24 of ERA for RR-B2

T(Years)	2	5	10	25	50	100	200
R24(2002)	52.29	58.58	62.82	68.28	72.39	76.53	80.71
R24(2011)	57.17	70.24	79.64	92.46	102.69	113.5	124.96
R24(2019)	58.24	71.49	81.05	94.11	104.57	115.65	127.41
R24 of ERA for B2(2013)	55.26	69.95	79.68	92.03	101.29	110.61	120.07

The return period value of GEV is larger than that of rainfall depth of RR-B2. Not only the design rainfall depth differs from the RR-B2 but also differs from time to time

4.1.3 Intensity duration frequency (IDF) curve for different return periods

The IDF developed from 24Hr time series rainfall data of 31 Years (1989 to 2019) obtained from Ethiopian meteorological agency located at Hawassa city. Rainfall intensity for the design storm is needed to calculate peak runoff rate from a drainage area for the design of storm water structures using rational method. For this study self developed IDF curve of the Hawassa station is used and the IDF for region B2 by ERA is presented for comparison in figure 4-6 and 4- 7 respectively.

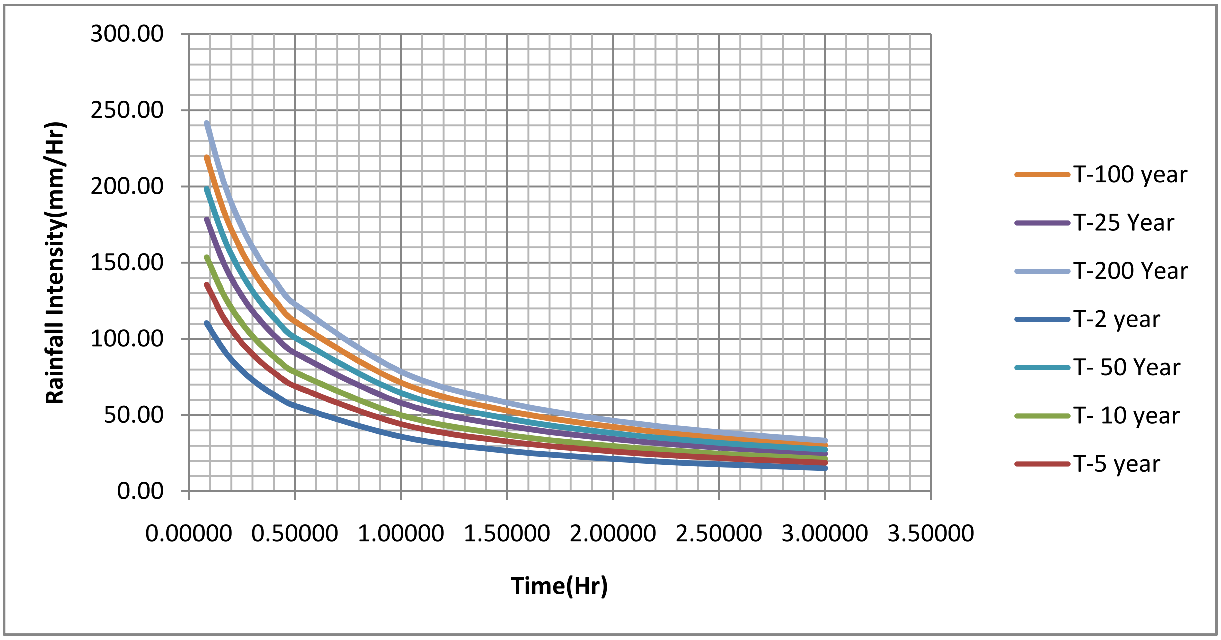


Figure 4-7: IDF curve of the study area

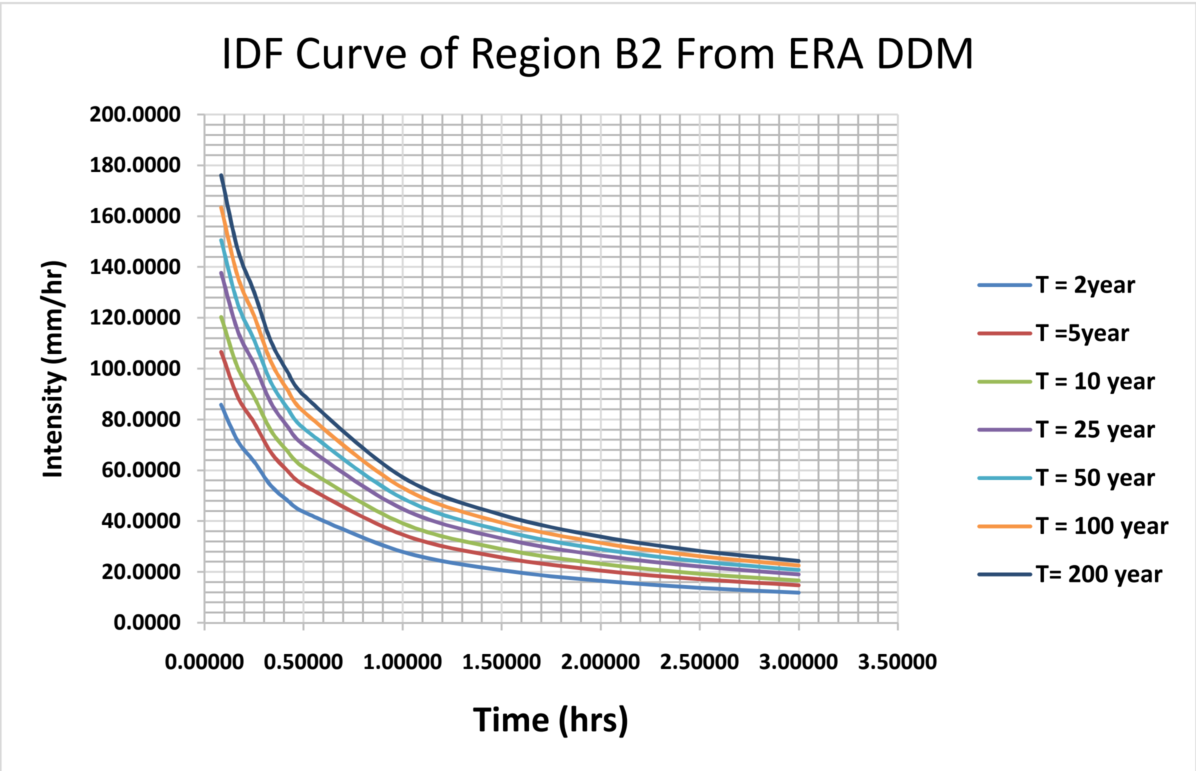


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Figure 4-8: IDF curve for region B2 from ERA drainage design manual (2013)

Table 4—4: Comparison of the frequency analysis with that of frequency analysis given by ERA for region B2.

Intensity (mm/hr)=Rt/duration												
Duration	Self	ERA	Self	ERA	Self	ERA	Self	ERA	Self	ERA	Self	ERA
T(min)	T-2	T-2	T-5	T-5	T-10	T-10	T-25	T-25	T-50	T-50	T-100	T-100
5	110.4	104.7	135.5	132.6	153.6	151	178.4	174.4	198.2	192	219.2	209.6
10	92.1	87.4	113	110.6	128.2	126	148.8	145.6	165.4	160.2	182.9	174.9
15	79.2	77.4	97.2	97.9	110.2	111.6	128	128.8	142.2	141.8	157.2	154.9
20	69.5	66	85.4	83.5	96.8	95.1	112.4	109.9	124.9	120.9	138.1	132.1
25	62.1	58.9	76.2	74.5	86.4	84.9	100.3	98.1	111.4	107.9	123.2	117.9
30	56.1	53.2	68.9	67.4	78.1	76.7	90.6	88.6	100.7	97.6	111.4	106.5
60	35.9	34.1	44	43.1	49.9	49.1	58	56.7	64.4	62.4	71.3	68.2
90	26.6	25.2	32.7	31.9	37	36.4	43	42	47.8	46.3	52.8	50.5
120	21.2	20.1	26.1	25.5	29.5	29	34.3	33.5	38.1	36.9	42.2	40.3
150	17.7	16.8	21.7	21.3	24.7	24.2	28.6	28	31.8	30.8	35.2	33.6
180	15.2	14.5	18.7	18.3	21.2	20.8	24.6	24.1	27.3	26.5	30.2	28.9

The values of rainfall intensity given in table 4-4 differ from 0.4 mm/hr for the duration of 180 minutes to 2.6 mm/hr for the duration of 5 minute for the return period of 10 years and 0.5 mm/hr to 4 mm/hr for the return period of 25 years for the same 180 and 5 minutes durations respectively from the value given by ERA. This is because of the rainfall used to develop the IDF curve by ERA is up to 2013 and the average value from homogeneous rainfall regions (RR-B2). While rainfall used for self developed IDF curve is up to 2019 and the specific station (Hawassa). So the self developed IDF account higher value. And also by plotting the design rainfall depth results computed by applying the GEV and ERA provided for RR-B2 and applying trend line equation ($R^2 = 0.769$) has better value for the used distribution than that of ERA for RR-B2 ($R^2 = 0.723$).

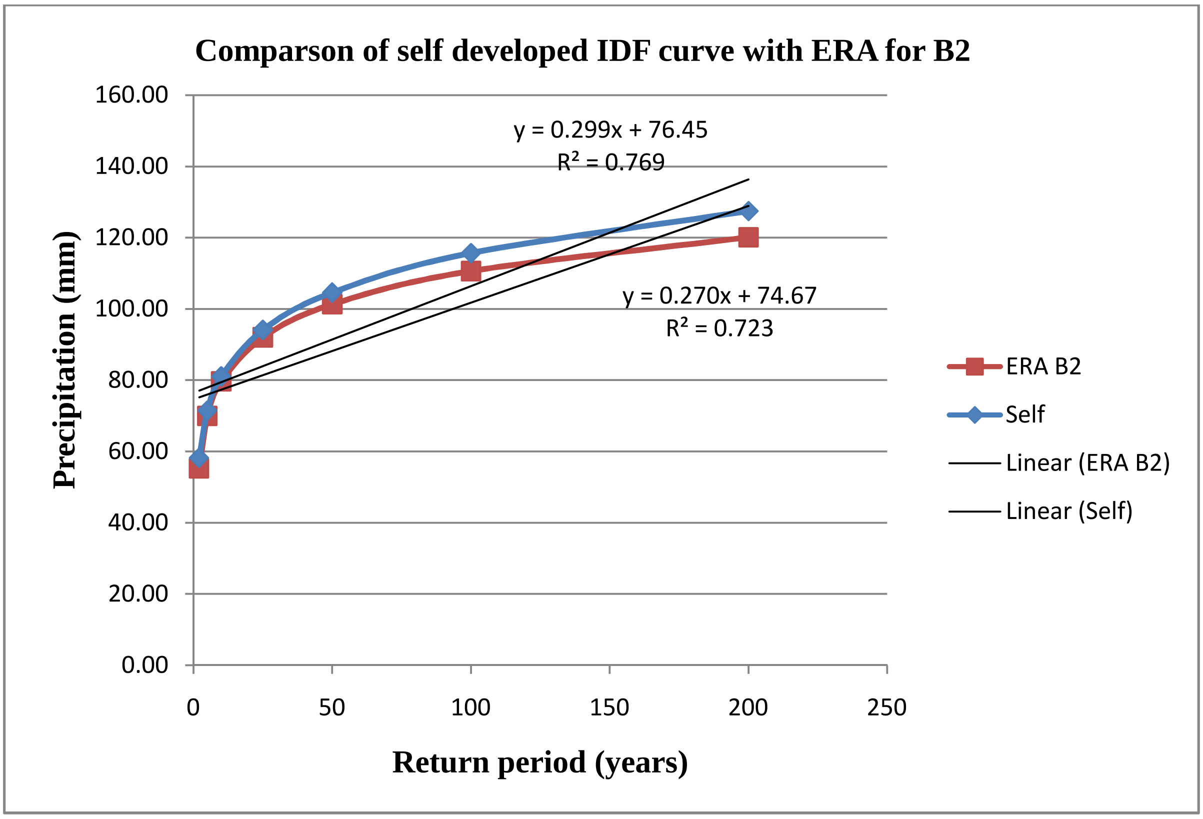

Figure 4-9: plots of frequency analysis results

From frequency analysis GEV has better R^2 value and used for this study for the analysis of design rainfall for required return period or 10 year for design and 25 year for check. Because of the flood hazard happened per 10 year in once we need to check the maximum flood hazard for the longest period (25 year). In some cases a flood event larger than the specified review flood larger than the 100 years (super-flood) might be used to ensure the safety of drainage structure and downstream communities based on ERA drainage design manual(ERA DDM 2013).

4.1.4 Catchment delineation and stream networks

Terrain processing has been resulted to flow direction and flow accumulation grids. Stream networks are obtained base area to define stream parameter that specifies how fine the network should be. Incorporating the digital elevation model (DEM) and ArcGIS10.2, the flow direction and flow accumulation is modeled. Flow direction and flow accumulation result is given in figure 4-10 below.

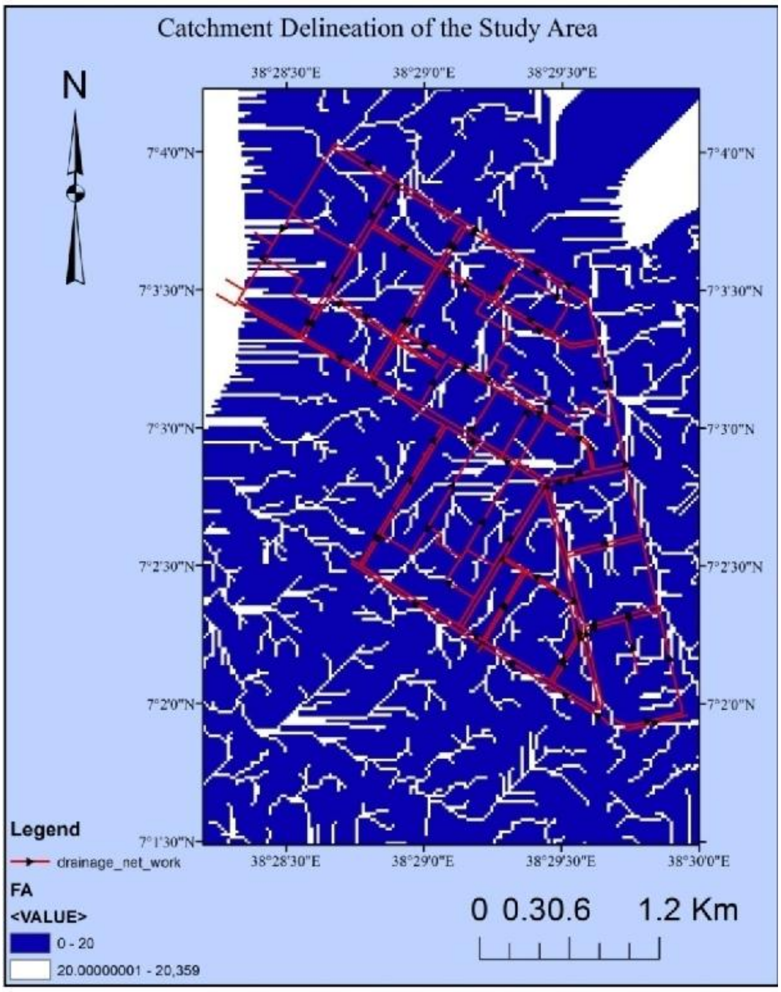

Figure 4-10: flow direction and flow accumulation modeling of the study area

As DEM result in figure 4-10 above, the study area does not receive runoff from external area rather than the catchment classified below.

Flow from Eastern part of the study area blocked by Highway from Addis to Kenya. Flow at Western part of study area accumulates to Piazza. From Shiloh Bible College side (South) flow passes to cheap wood plant through two culvert structure each dimensioned (ϕ = 1.2m) and Trufat to Hawassa compressive referral Hospital and to regional health bureau by drain structure. The Northern side flow accumulates to Haile Resort Hotel. Natural flow in the catchment area accumulated to Gezahegn & Elfinesh Resort Hotel, Daite Cultural restaurant, Yesh traffic light, shell diesel, Sema Hotel, (Hawassa University President Entrance), Lewi Campus Café & Restaurant, Regional Education Bureau, Guadguada Sefer, 22 Mazoria, Arebsefer mosque, Mobill, and Swampy area.

4.1.5 Land use land cover transition of the study area

Land use land cover of the study area is divided into thirteen land use types, based on the master plan of city administration. Classification compiles the similar feature; e.g. government compounds with religious institutions, business areas with service centers etc. to select the representative runoff coefficient. Classification results for two different periods 2002 and 2019 are given in figure 4.11 and 4.12 respectively.

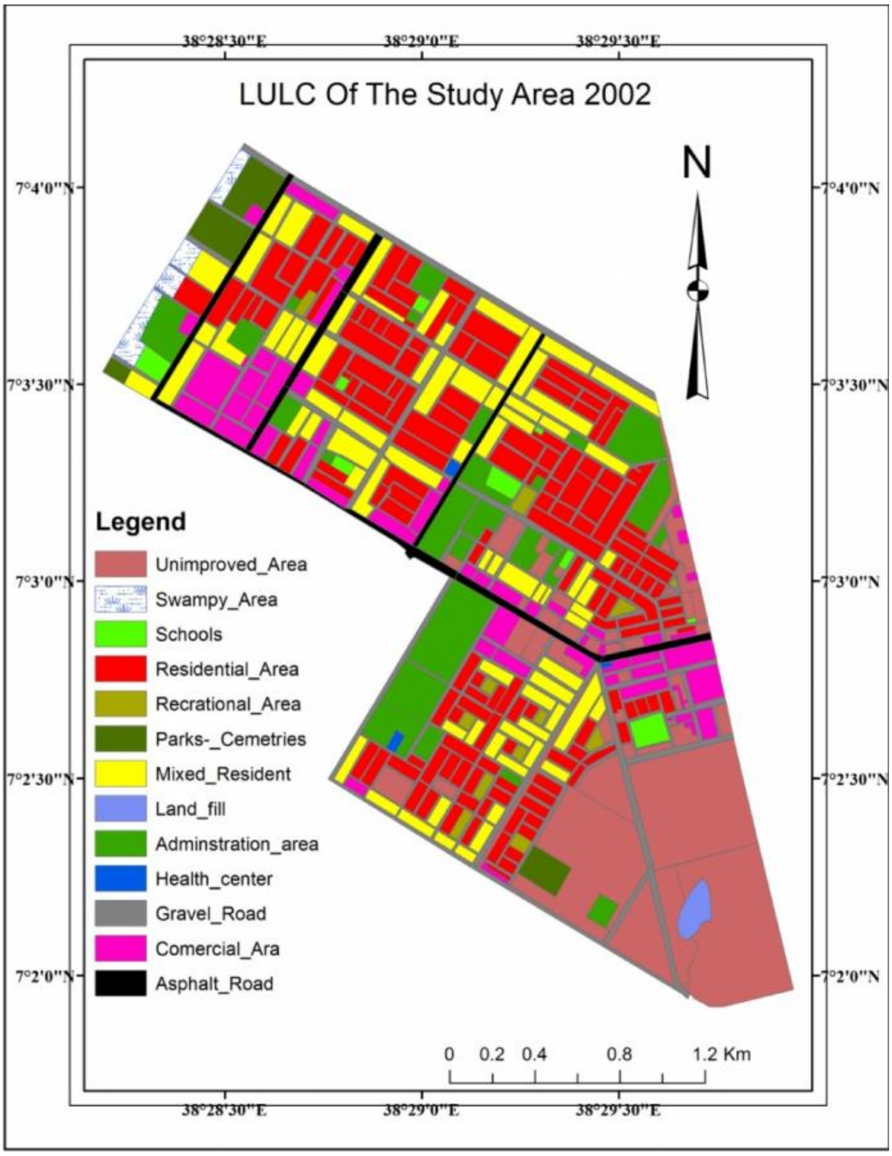

Figure 4-11: land use land cover of the study area in 2002

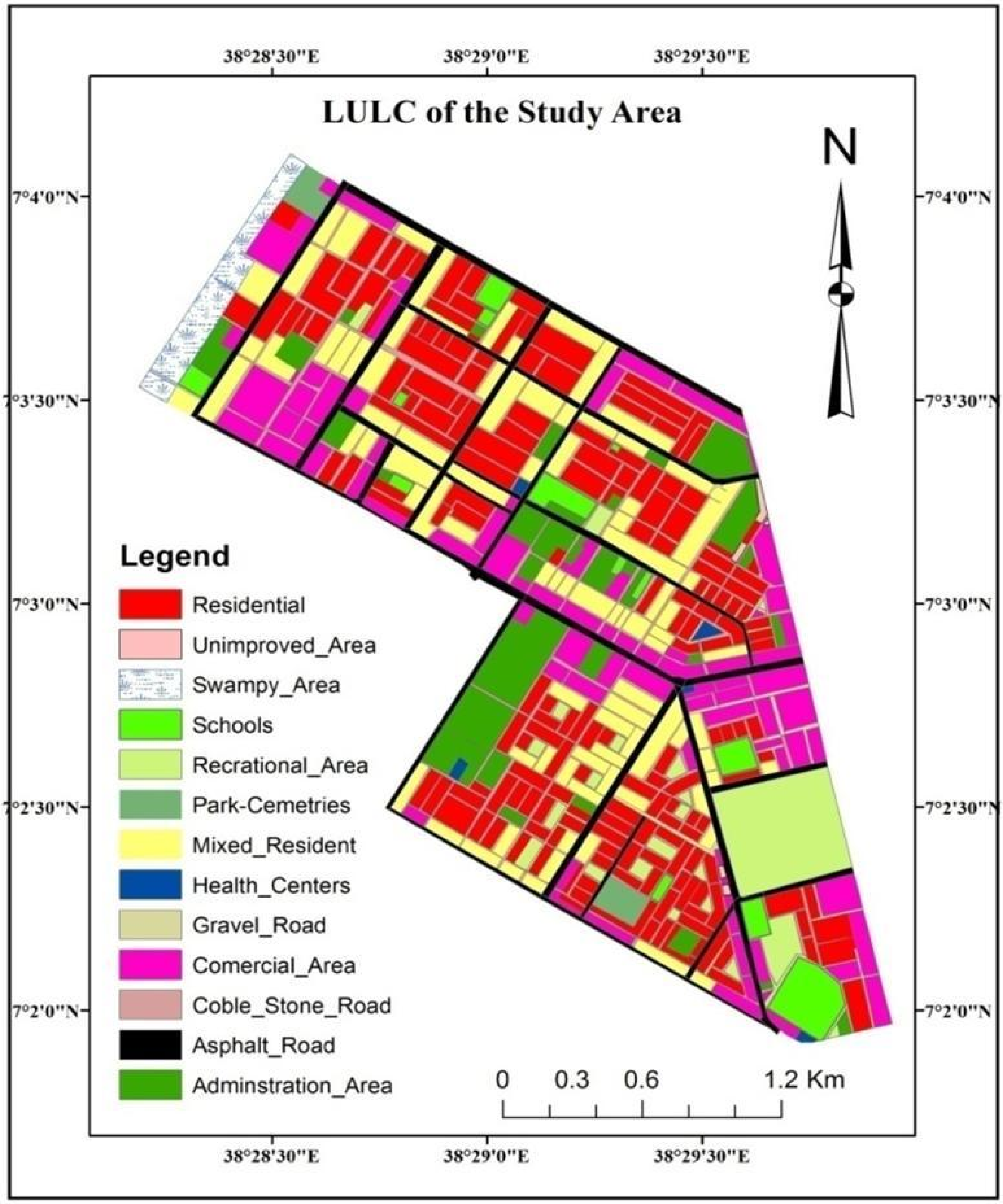

Figure 4-12: land use land cover of the study area in 2019.

In 2002 the land use land cover of the study area above Wanza square dominated by unimproved area with low runoff coefficient 0.2. In 2019 this area dominantly covered with residential, commercial, asphalt, and coble stone etc. which have higher runoff coefficients range from 0.5 to 0.95. The specific shift in land use land cover of the area is given in table 3-1 above.

4.1.6 Hydrologic Analysis

4.1.6.1 Estimation of catchment parameters

The catchment area was divided into twenty-four sub-catchments and their weighted runoff coefficient is determined for each sub-catchment by using equation 3-32. Runoff coefficient of each catchment increased, time of concentration is shortened, and Rainfall intensity is increased. All these above increments result the increased runoff of the study area. The drainage area, length and representative runoff coefficient for each segment is given in table 4-6 below.

Catchment parameter estimation at Daite-stadium junction (CH4)

Slope of the watershed

The slope of overland and shallow concentrated flow is determined by definition of the slope; as $s = \frac{H}{l}$ where S is slope, H is elevation difference and L is catchment length

$= (\frac{1730-1724}{476.93}) = 0.0125 = 1.26\%$

Time of concentration

In the study area the time of concentration for every segment is computed as the sum of inlet time and system time. The inlet time is the two categories such as sheet flow time up to 100m and shallow concentrated flow time for more than 100m. These two's and system time is computed and summed as,

Sheet flow

The sheet flow occurs up to 100m, short grass covered and manning's coefficient of over land flow for short grass cover less than 0.03m is given as 0.15 and using the slope determined above = 1.26%,

$$T_{ti} = \frac{0.091(\text{nl})^{0.8}}{\text{P}_2{}^{0.5}\text{S}^{0.4}} = \frac{0.091(0.15*100)^{0.8}}{58.24^{0.5}0.0126^{0.4}} = 35.94\text{min}$$

Shallow concentrated flow

For shallow concentrated flow occurs at a length greater than 100m and the unpaved surface using the equation given in below.

$$V = 4.9178S^{0.5} = 4.9178 * 0.0126^{0.5} = 0.552\text{m/s}$$

$$T_{tii} = \frac{L}{60*V} = \frac{376.93L}{60*0.552} = 11.39\text{ min}$$

Channel flow

For channel flow the manning's equation is used to calculate the velocity of flow in the channel and the system time is computed then after. The channel is constructed by masonry and chose the manning's roughness coefficient 0.032 as recommended in US Urban drainage design manual,

Having all the channel profile width 0.8 m, depth 0.33 m, upper elevation 1724, lower elevation 1719 m and channel length 517.21 m;

Cross sectional area of the channel

$A = W * D = 0.8 \times 0.35 = 0.280m^2$

Weighted perimeter of the channel

$P = 2D + W = 2 \times 0.35 + 0.8 = 1.500m$

Hydraulic radius of the channel

$R = \frac{A}{P} = \frac{0.280m^2}{1.500m} = 0.187\text{m}$ and $R^{\frac{2}{3}} = 0.327$

Channel slope

$S = \frac{\Delta H}{L} = \frac{(17124-1719)}{517.21} = 0.010\text{(m/m)}$ and $S^{\frac{1}{2}} = 0.098$

Velocity of the flow

$V = \frac{R^{\frac{2}{3}} S^{\frac{1}{2}}}{n} = \frac{0.327*0.098}{0.032} = 1.001\text{m/s}$

$T_{tiii} = \frac{L}{60V} = \frac{517.21}{60*1.001} = 8.612min$

So the travel time is the sum of the above three.

$$T_c = T_{ti} + T_{tii} + T_{tiii} = 35.94 + 11.39 + 8.612 = 55.942 \; say \; 56 \; min$$

The time of concentration is computed in the same fashion for every segment and minimum threshold value (15min) used for rational method as suggested by ERA DDM (2013). The computed results of time of concentration are given in Appendix1.2, table 1.13.

Rainfall intensity

For the computed time of concentration, the rainfall intensity was read from the developed IDF curve in section 4.1.6. For the time of concentration different from time used for plotting the IDF curve the rainfall intensity values obtained by interpolating between the larger and smaller values. A result of rainfall intensity for every segment is presented in Appendix1.2 table 1-14.

Runoff coefficient determination

This sub-catchment is composed of two land use types such as; recreational area and asphalt road. As these features have different runoff coefficient values, the representative weighted runoff coefficient (Cw) should have to determine.

Recreational area (RE3) – A = 24.3ha, C = 0.3,

Asphalt road (AS5) – A = 1.5ha, C = 0.825

$C_w = \sum \frac{C_i}{A_i} A_i \quad = \frac{0.3*24.3+0.825*1.5}{024.3+1.5} = 0.330523$ runoff coefficients for other segments are calculated in the same way and presented in table 4.6.below.

Peak flood computation

From the procedures followed above all parameters to compute peak flood by incorporating rational method described earlier are obtained. Time of concentration and sub-catchment area are in the rage recommended for rational method. Now applying the method the peak flood for 10-year return period for design, 25-year return period for review and even 100-year return period for structure safety and downstream community is computed as follows;

T = 10 year $Q = 0.00278 * C * C_f * I * A = 0.00278$ 0.331*1*53.69*25.8 = 1.273

T = 25 year $Q = 0.00278$ $C * C_f * I * A = 0.00278$ 0.331*1.10*62.35*25.8 = 1.628

T =100year $Q = 0.00278$ $C * C_f * I * A = 0.00278$ 0.331*1.25*76.62*25.8 = 2. 274

Computation has been carried in the same fashion for every segment and the values for different return period is given in table 4-5 and the peak discharge results compared in table 4-6 below for both periods of 2002 and 2019.

Table 4—5**:** current (2019) peak discharge of different return period of each segment

Runoff(Q)(m^3/s)							
Drainage Code	Existing capacity	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year
DL1	2.74	0.873	1.072	1.215	1.552	1.881	2.167
DL2	2.11	1.246	1.529	1.733	2.214	2.684	3.092
DL3	2.02	3.174	3.895	4.416	5.641	6.838	7.877
DL4	0.62	0.915	1.123	1.273	1.628	1.971	2.274
DL5	1.95	3.056	3.751	4.252	5.432	6.584	7.585
DL6	1.38	1.921	2.358	2.673	3.415	4.139	4.721
DL7	3.58	4.39	5.388	6.109	7.803	9.458	10.896
DL9	0.54	6.042	7.415	8.407	10.739	13.017	14.996
DL10	0.13	4.889	6.001	6.803	8.69	10.534	12.135
DL11	0.1	1.372	1.684	1.91	2.439	2.957	3.406
DL12	2.183	0.712	0.874	0.991	1.265	1.534	1.767
DL13	3.33	6.148	7.545	8.554	10.927	13.245	15.258
DL14	4.91	2.89	3.547	4.021	5.136	6.226	7.172
DL16	5.2	6.837	8.392	9.514	12.153	14.731	16.97
DL17	3.96	3.776	4.635	5.255	6.712	8.136	9.372
DL18	3.48	2.473	3.035	3.441	4.395	5.328	6.138
DL19	6.129	4.666	5.726	6.492	8.293	10.052	11.58
DL20	8.87	4.594	5.638	6.392	8.165	9.897	11.402
DL21	3.72	2.845	3.492	3.959	5.057	6.129	7.061
DL25	4.79	3.943	4.84	5.487	7.009	8.495	9.787

Table 4—6: Comparison of generated runoff from each catchment between 2002 and 2019

Sub Catchments	Length	Area	C 2002	C 2019	I(mm/ Hr)2002	I(mm/ Hr) 2019	Q(m^3/ s) 2002	Q(m^3/ s) 2019	Q(m^3/s) for constant	
									C	I
Catchment 1(CH1)	1122.06	27.3	0.2	0.671	66.95	75.25	1.02	3.83	1.14	3.41
Catchment 4(CH4)	740.25	27.2	0.227	0.331	60.51	58.38	1.04	1.46	1.00	1.51
Catchment 2(CH2)	562.48	6.63	0.331	0.758	99.35	110.19	0.61	1.54	0.67	1.39
Catchment 3(CH3)	689.44	7.83	0.257	0.764	94.48	110.19	0.53	1.83	0.62	1.57
Catchment 5(CH5)	502.37	13.35	0.727	0.808	58.33	107.51	1.57	3.22	2.90	1.75
Catchment 6(CH6)	981.02	23.3	0.538	0.725	70.17	78.06	2.45	3.67	2.72	3.30
Catchment 7(CH7)	1139.97	46.97	0.397	0.667	64.37	102.14	3.34	8.89	5.29	5.60
Catchment (CH8A)	624.6	33.1	0.647	0.720	79.88	110.19	4.76	7.30	6.56	5.29
Catchment (CH8B)	1234.77	42.86	0.596	0.686	38.04	110.19	2.70	9.00	7.83	3.11
Catchment (CH8C)	791.8	16.79	0.521	0.540	64.37	110.19	1.57	2.78	2.68	1.62
Catchment (CH13)	523.09	13.75	0.531	0.714	66.95	107.51	1.36	2.94	2.18	1.83
Catchment 9 (CH9)	926.29	34.22	0.598	0.714	40.89	107.51	2.33	2.78	6.12	2.78
Catchment (CH10)	365.65	7.33	0.494	0.763	79.88	110.19	0.80	1.71	1.11	1.24

Catchment (CH11)	303.85	4.98	0.438	0.521	75.01	110.19	0.45	0.80	0.67	0.54
Catchment (CH12A)	1551.54	37.36	0.663	0.703	29.69	41.76	2.04	3.05	2.88	2.17
Catchment (CH12B)	462.83	9.79	0.739	0.739	99.35	110.19	2.00	2.22	2.22	2.00
Catchment (CH12C)	739.09	24.53	0.659	0.647	60.51	110.19	2.72	4.86	4.95	2.67
Catchment (CH14)	653.65	15.86	0.716	0.716	39.44	110.19	1.24	3.48	3.48	1.24
Catchment (CH15)	1207.68	33.08	0.697	0.725	49.61	110.19	3.18	7.34	7.06	3.31
Catchment (CH16)	1023.43	30.11	0.723	0.737	58.33	110.19	3.53	6.80	6.67	3.60
Catchment (CH17)	855.07	28.7	0.708	0.725	53.24	110.19	3.01	6.37	6.22	3.08
Catchment (CH19)	569.23	24.05	0.737	0.758	84.75	110.19	4.18	5.59	5.43	4.30
Catchment (CH18)	681.36	28.76	0.708	0.727	66.95	110.19	3.79	6.41	6.24	3.89
Catchment (CH20)	181	25.04	0.512	0.647	119.0	110.19	4.24	4.96	3.93	5.36
Total Runoff generated from the Catchment							54.45	102.8	90.56	66.5

As presented in table 4-6 above the 10 years peak discharge generated in 2002 was 54.45 m^3/s. due to both climate change and urbanization the peak discharge generated counts 102.80 m^3/s(i.e. the peak discharge is doubled) in 2019. The climate change effect and urbanization effect on peak discharge is compared with keeping runoff coefficient (Cw) and rainfall intensity (I) constant. When the urbanization (Cw) value kept constant 91.21 m^3/s discharge is computed (i.e. the surplus of discharge by climate change from the base year 2002 is 36.76m^3/s. While the climate (rainfall intensity) value kept constant the peak discharge generated from the catchment was 66.55m^3/s. The surplus of discharge by urbanization from the base year 2002 is 12.1m^3/s. Therefore the peak discharge increased by climate change (67.51%) than urbanization (22.49).

4.2 Performance of Urban Drainage System

4.2.1 GIS based flow path mapping and modeling

Urban runoff flow path following the drainage system of the study area is modeled by GIS as given in figure 4-13. Flow concentrated at a junction from several segments. This measure exacerbated the over flooding and affect assets at that junction.

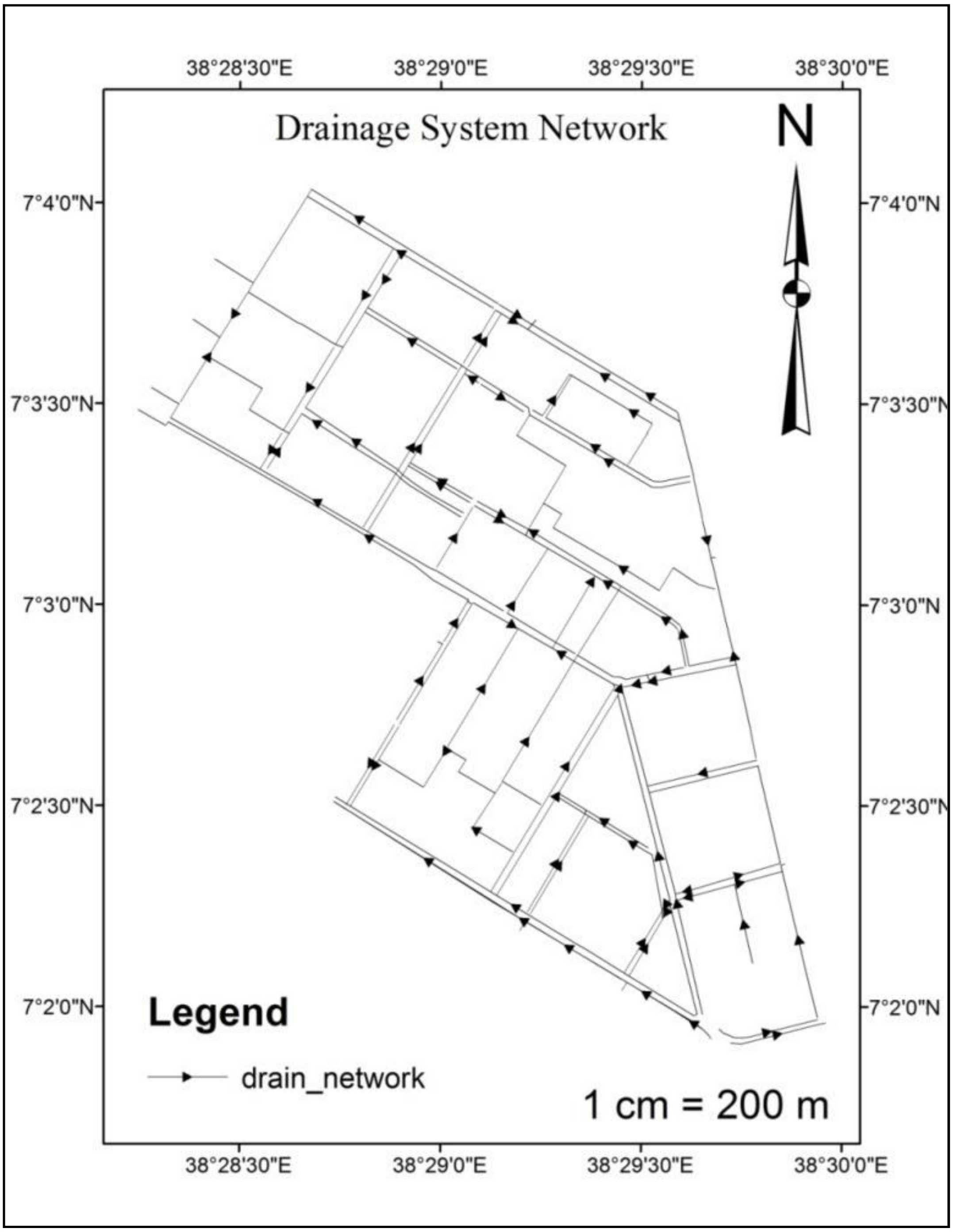

Figure 4-13 drainage system network of the study area

The most vulnerable junctions that counter over flooding are Wanza Square, Yesh traffic light junction, Guadguada square, Piazza – Mobill road junction near artistic printing press and South Star International hotel to Hawassa global automotive services segments. Adequacy of these and other junctions to convey peak discharge to downstream and peak flood that occurs at every junction is computed following drainage system network.

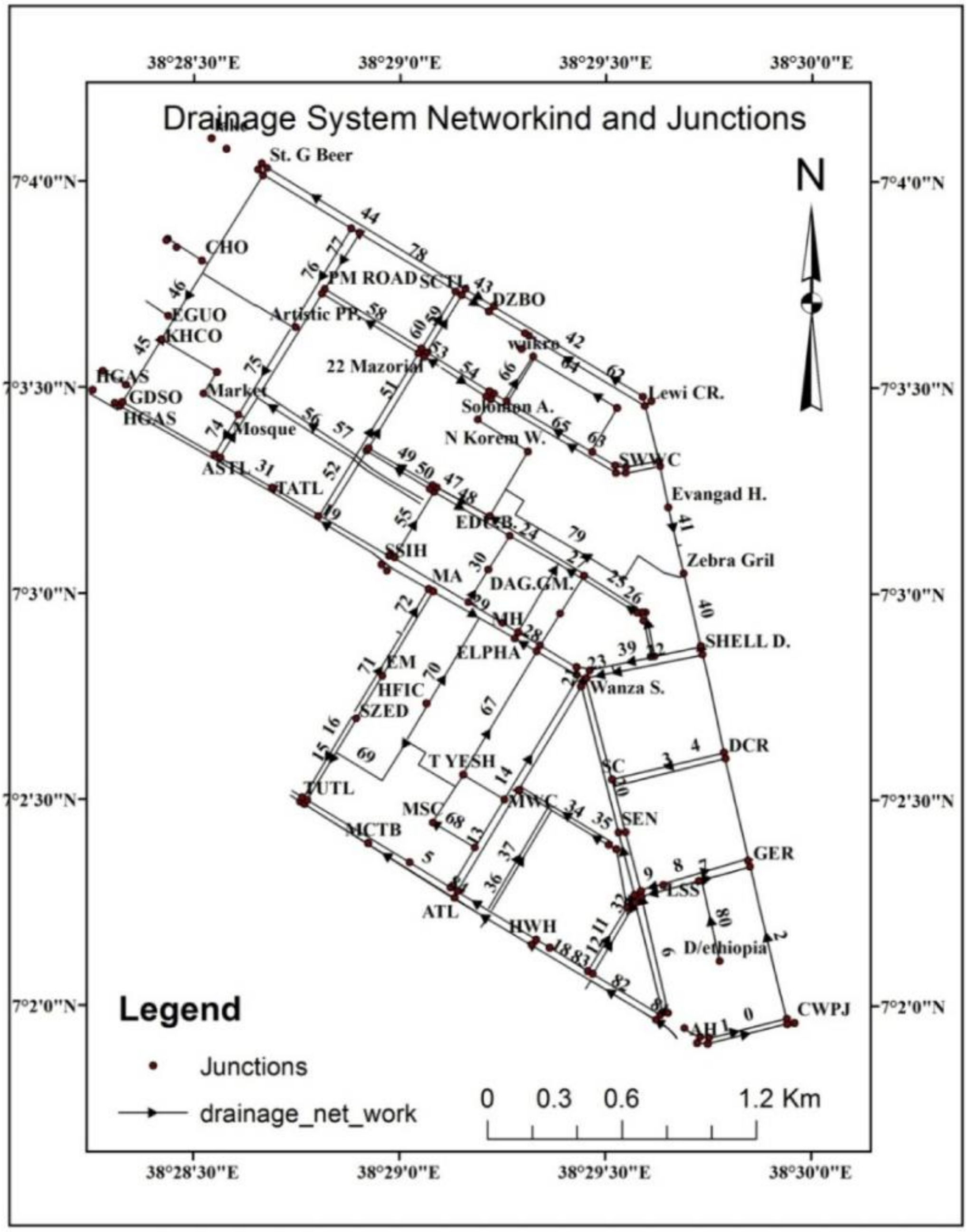

Figure 4-14: drainage system networking and junctions that flow overtops

4.2.2 Adequacy of the drainage system following the system networking

The adequacy of each drain in the study area analyzed, hydraulic capacity of each drain was computed first by manning's as stated earlier and given in appendix2 table 2.1. Then the adequacy of each drain presented in table 4-7 below.

Table 4—7: existing capacity, proposed and excess discharge following flow path

Peak Discharge Reaching Wanza Square							
Drainage segments		Proposed discharge Q(m/s)			Excess discharge Q(m/s)		
Drainage Code	Existing capacity	10 Year	25 Year	100 Year	design	check	safety
DL1	2.74	1.215	1.552	2.167			
DL2	2.11	1.733	2.214	3.092		0.104	0.982
DL4	0.62	1.273	1.626	2.27		1.006	1.65
DL5	1.95	4.252	5.432	7.585		3.482	5.635
DL6	1.38	2.673	3.415	4.721	1.293	2.035	3.341
DL7	3.58	6.109	7.803	10.896		4.223	7.316
DL8	3.48	17.256	22.041	30.731	13.776	18.561	27.251
Peak Discharge Reaching Yesh Trafic Light							
DL8	3.48	17.256	22.041	30.731			
DL9	0.54	6.803	8.69	12.135	6.263	8.15	11.595
	0.471	24.059	30.732	42.865	23.588	30.261	42.394
Peak discharge reaching Trufat traffic light							
DL10	0.13	6.803	8.69	12.135	6.673	8.56	12.005
Peak Discharge Reaching Global Automotive Service							
DL11	0.1	1.91	2.439	3.406	1.81	2.339	3.306
DL12	2.183	0.991	1.265	1.767			
		2.9	3.705	5.173	0.717	1.522	2.99
Peak Discharge Reaching Korem Meda							
DL13+DL23	4.088	8.554	10.927	15.258	4.466	6.839	11.17
peak discharge reaching Guadguada Square							
DL13+DL23 DL14	4.088 4.91	4.088 4.021	4.088 5.136	4.088 7.172		0.226	2.262
DL25	4.79	5.487	7.009	9.787	0.697	2.219	4.997
DL15	10.382	13.596	16.233	21.047	3.214	5.851	10.665
Peak Discharge Reaching Piazza- Mobill Road							
DL15	10.382	10.382	10.382	10.382			
DL16	5.2	9.514	12.153	16.97	4.314	6.953	11.77
DL22	1.67	19.896	22.535	27.352	18.226	20.865	25.682
Peak Discharge Reaching Chambalalla Hotel (Outfall)							
DL17+DL22	5.63	5.255	6.712	9.372		1.082	3.742
DL20+DL24	10.136	6.392	8.165	11.402			1.266
		11.647	14.877	20.774	1.511	4.741	10.638
Peak Discharge Reaching Gezahegn And Elfinesh Resourt(Outfall)							
DL3	5.001	4.416	5.641	7.877		0.64	2.876

As seen above in table 4-5, without considering the external drain effect, about 12 drains/ditches (60%) were exceeded by design discharge (10 years), 16 drains/ditches (80%) were exceeded by 25 years discharge and 19 drains (95%) were exceeded by 100 years peak discharge. But following the drainage system networking 13drains (59.09%) exceeded by 10 years discharge, 20(90.91%) by 25 years discharge and 21(95.45%) drains exceeded by 100 years discharge. I.e. number of drainage lines concentrated in a junction exacerbated flooding at that junction.

4.2.3 Societal value

Practices that affect the effectiveness of the urban drainage system are the disposal of solid wastes and sewage in to and around the drains. The solid wastes disposed around the drain found its way in to the drainage system during the rain and block the water passageway. The city has no sewerage system; incase the city use Vacuum truck to dispose the sewage at mount Alamura. These trucks are not accessible. Therefore urban dwellers, business centers, especially car washes and garages dispose Sewage into storm drain system. Solid waste disposing habit and its effect on urban drainage system in the area is visualized in figure 4-15 below.

Figure 4-15: solid waste and sewage management problem in the study area

Depth shared by solid wastes efficiency and sewage condition in drainage system is given in table 4-8 below.

Table **4—8**: Depth shared by solid wastes, efficiency and sewage condition of drains

Drainage Code	Average Depth(m)		Depth Shared By Solid Waste(m)		Percentage of Shared Volume		Efficiency of The System		Sewage Condition	
	Left	Right	Left	Right	Left	Right	Left	Right	Left	Right
DL1	0.5	0.5	0	0	0	0	100	100	sever	sever
DL2	0.7	0.6	0.1	0.1	14.3	16.7	85.7	83.3	wet	wet
DL3	0.8	0.8	0	0	0	0	100	100	clean	clean
DL4	0.4	0.5	0	0	0	0	100	100	clean	clean
DL5	0.5	0.5	0	0	0	0	100	100	sever	sever
DL6	0.8	0.8	0.2	0.4	25	50	75	50	sever	sever
DL7	0.5	1.5	0.3	0.1	60	6.7	40	93.3	wet	wet
DL8	1.2	0.8	1.2	0.2	100	25	0	75	wet	wet
DL9	-	0.6	-	0.1	-	16.7	-	83.3	-	wet
DL10	0.3		0.1	-	30.3	-	69.7	-	clean	-
DL11	0.3	-	0		0	-	100	-	clean	-
DL12	0.6	1	0.6	0.4	100	40	0	60	wet	wet
DL13	1.2	1.5	0.1	-	8.3	0	91.7	100	wet	sever
DL14	1.8	-	-	-	0	-	100	-	wet	-
DL15	0.6	2.5	0.2	0.3	33.3	12	66.7	88	wet	wet
DL16	0.6	2.2	0.2	0.2	33.3	9.1	66.7	90.9	wet	sever
DL17	-	0.5	-	0.1	-	22.2	-	77.8	-	wet
DL18	0.5	-	0.1	-	22.2	-	77.8	-	wet	-
DL19	1.4	1.4	1.4	0.2	100	14.3	0	85.7	sever	sever
DL20	-	0.9	-	0.2	-	22.2	-	77.8	-	wet
DL21	0.6	2.5	0.3	0.3	50	12	50	88	wet	wet

From table 4-8 above drainage lines 1, 3, 4, 5, 11,13R and 14, (11/35) or 31.43% are free from litter effect. Drainage lines 2, 7R, 9, 13L, 15R, 16R, 19R and 21R (9/35) or 25.71% are the structures with their efficiency lies between 81-99%. I.e. the litter effect on these structures is mild. Drain lines 6, 8R, 10L, 12R, 15L, 16L, 17R, 18L, 19R, 20R and 21L, (11/35) or 31.43% are with their efficiency ranging from 50 to 80 which are moderately affected. But drainage lines 7L, 8L, 12R and 19L (4/35) or 11.43% are severely affected by litters and their efficiency rests below 50%. So the litters exacerbate the overtopping of the flood. In other hand the sewage released in to drainage system ranges (6/35) or 17.15% clean (i.e. water removed immediately after rainfall). (19/35) or 54.28% wet (water clogged due to litter) and (10/35) or 28.57% severely affected by extra load of sewage (I.e. these drains have point source of the sewage from garages and carwashes).

4.2.4 Institutional arrangements for urban drainage planning

At the city level there is gape in commitment and responsibilities for activities; Low surveillance mechanisms, and poor supportive mechanisms that could harmonize the national and regional policies, strategies, and plans for urban solid waste and storm management services. There is a gap in institutional collaboration in urban drainage planning and management. Planning and managing the urban drainage lied on infrastructure department and beautification department under city municipality. The great gap here is Environmental protection department, water supply, sanitation and sewerage enterprise and spatial plan departments have lost being the main actors in urban drainage planning and management.

4.2.5 Implication for Policies and practice

Storm water in the city is managed by the storm water drainage system and the final destination of the storm water is Lake Hawassa. Institutions having more than 1ha areas are without SUDs measures or LID controls, therefore, these practices exacerbate the flooding in the study area. Though Proclamation of solid waste management pro No 513/2007 aims to enhance at all levels capacities to prevent the possible adverse impacts while creating economically and social beneficial assets out of solid waste there is a great gap in solid waste management. Among the total number of 1777 of the solid waste cleaning labors only 90(5.1%) are the ditch cleaners. This labor force is incapable to remove the solid waste disposed in the drainage system. Commitment and capacity of the labor is also the challenging issue. The waste disposed in the drain system persists above a year, which shares the large volume as given in table 4-8 of the drains and blocks the passageway of the storm water. 78 machineries are used to haul the solid waste either for recycling or to landfill. Out of 78 hauling machineries 17 are donkey pulled carts and ride by children below the age of 15 years. These children drop the waste here and there. 28 machines are three wheel drives having insignificant capacity to carry the solid wastes. And also there is no technology to remove the waste from the closed conduits.

Integrating all impacts above table 4-7 and table 4-8 the total resulted flood at the most vulnerable junctions in the study area is presented in table 4-9.

Table **4—9**: integrated impact at most vulnerable junctions

Junctions	Existing capacity m^3/s	Effective capacity m^3/s	Total discharge m^3/s	Total flood m^3/s
DL6	1.380	0.920	2.673	1.753
DL7	3.580	3.191	6.109	2.918
DL8	3.480	0.668	17.256	16.588
DL26	0.471	0.471	25.663	25.192
DL15	10.382	9.137	13.596	4.459
DL16	5.200	4.610	9.514	4.904
DL17	3.960	3.081	5.255	2.174
DL18	3.480	2.707	3.441	0.734
DL19	6.129	5.253	6.492	1.239
DL20	3.600	2.801	6.392	3.591
DL21	3.720	1.037	3.959	2.922
DL22	1.670	1.670	16.874	15.204

Note that: at these junctions flood is exacerbated by litters in addition to urban tight areas and intense rainfall.

4.3 Adverse Effect of Poor Urban Drainage

4.3.1 Transportation System

The vulnerable bodies among the travelers in the city are pedestrians and open vehicle users. Pedestrians and open vehicle users suffer crossing the flood prone areas during the rainy season. The pedestrians cross the road with hanging their shoes on their shoulder; and the sharp materials unsafely disposed on the streets and elsewhere could be the possible causes of body injury. Pedestrians may fail in the open ditches and resulted in fractures. They obligated to pay 10-50 ETB to cross inundated road (max. 30 m) either on the back of straight boys or other vehicles. The problem that pedestrians face during flood is given in figure 4–16 below.

Figure 4-16: flood effect on pedestrians(source: Girma Yane)

The flood could be the cause for the electro-mechanical damage of vehicles. Flood on the road is also causes the traffic delay and congestion. The effect of flood on the open vehicles and other type of vehicles is given in figure 4–17 below.

Figure 4-17: flood effect on open and other type vehicles (source: Girma Yane)

Bad drainage has damage and loss in serviceability of both rigid and flexible types of pavement much greater when structural section contains free water. The quality of drainage is an important parameter which affects the performance of the road pavement. Poor drainage quality leads to a large amount of costly repairs or replacements long before reaching their design life. Asphalt road in the area is in chronic distress. Chronic pavement distress at the route old bus station to market specifically at Tabor primary school and Arebsefer traffic light is presented in figure 4-18.

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Figure 4-18: chronic pavement distress in the route old bus station to old market. (Source: Girma Yane)

4.3.2 Environmental health

In the study area the drainage system operational performance is rated as **poor** and clogging of water in drainage system was identified. The clogged water was the cause for breading of mosquitoes and bad smell. These cause health problem for urban dwellers. Top ten diseases that caused by the performance failure of urban drainage system such as: malaria, typhoid, respiratory diseases, diarrhea, parasite etc. for different years in comparison with other causes are given in table 4-10 below.

Table 4—10: disease caused by poor urban drainage and other causes

Year	All above listed causes					Other causes		
	Male	Female	Total	%	Male	Female	Total	%
2010-2011	3505	3533	7038	59.64	2330	2433	4763	40.36
2011-2012	112519	104811	217330	65.98	55970	56093	112063	34.02
2012-2013	105456	95561	201017	62.18	63337	58914	122251	37.82
2013-2014	109357	101408	210765	60.08	71288	68752	140040	39.92
2014-2015	94230	86878	181108	54.45	76267	75215	151482	45.55
2015-2016	116867	111904	228771	56.65	91049	83986	175035	43.35
2016-2017	129120	116567	245687	57.23	93710	89874	183584	42.77
2017-2018	144456	129014	273470	57.85	101782	95054	196836	42.15

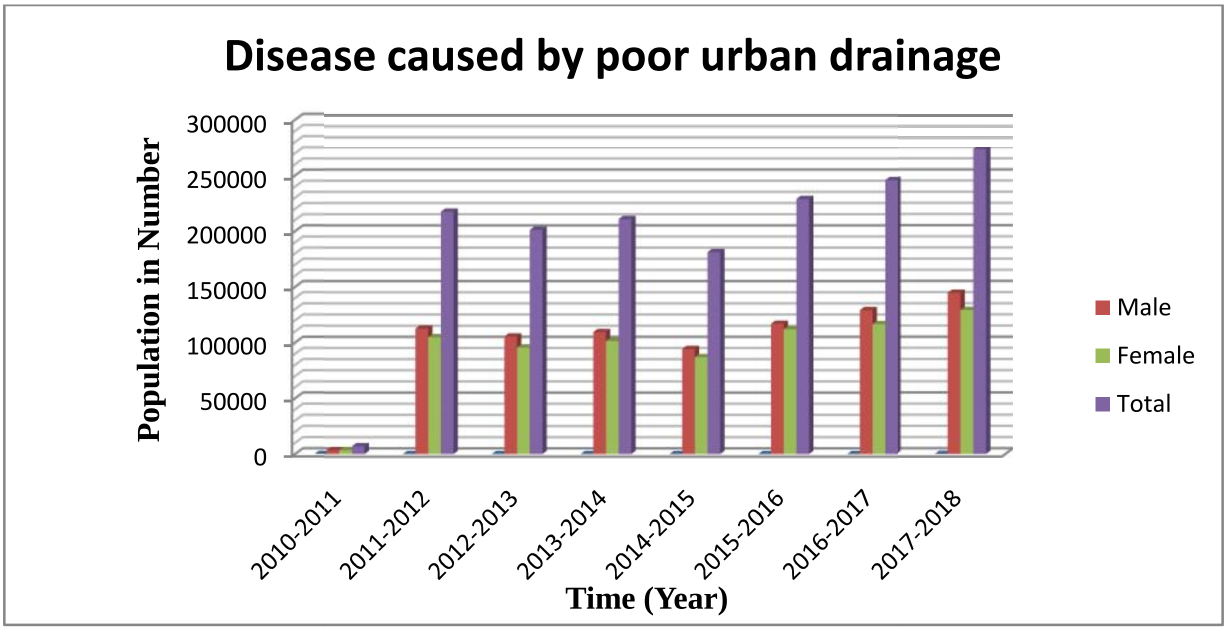

Figure 4-19: disease caused by poor urban drainage system in the study area

4.3.3 Establishments like telecom and other service centers

Meneharia and Guadguada are the hearts of city, next to piazza are much more vulnerable. It was hard to give the telecom services (sells) during the heavy rain event due to flooding. Potentially income of the Telecom establishment, service centers like electronic shops, café and restaurants, Hotels, Banks, etc. is reduced in this flood prone area. For example Abezash Dinku is the Hotel at Wanza Square which is victim of flood. The flood enters even into bed rooms, and damages the electronic device like refrigerator. No bed rent during the flooding days. During observation displacing of road side lights was observed and this has an effect on Ethiopian electric utility department. Due to daily income variability and poor cash registering habit of the mentioned centers, it was impossible to estimate economic loss of the centers.

Figure 4-20: flood effect on Telecom and different service centers (source: Girma Yane)

4.3.4 National Economy

Failure of transportation system, health, telecom and other service centers highly affect the national economy. Nowadays the rehabilitation measure of urban drainage system for empting purpose of the storm water starting from Wanza square to the lake Hawassa is under construction. The depth of cut is more than 7m and the width and the depth of the structure is 2.5m and 2.5m respectively and the top of the structure lays more than 4m below the ground level. and storm water collection pond dimensioned length-500m, width-36m and depth 9m at the former air lines is under construction; this measure costs 271,648,631.96(2.72) millions of ETB. The rehabilitation and sizing measure taken by city administration is given in figure 4-21 below.

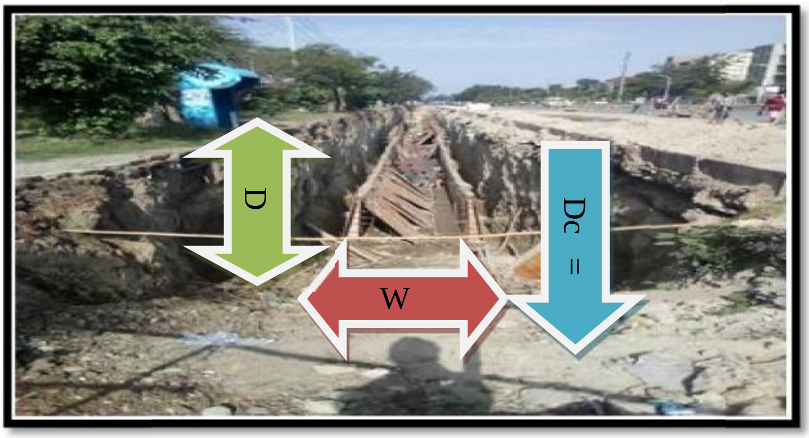

Figure 4-21: sizing and rehabilitation measure in the route Wanza square to Lake (source: Girma Yane)

5 CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

Intense rainfall and urbanization are the main factors that affect the storm water drainage system through intense rainfall, impermeable areas, and system blockage by solid wastes and sewage in Hawassa city this time. The study was aimed to analyze storm water drainage system of Hawassa city. To analyze the level of the problem models like Arc GIS10.2, Google earth Pro, Generalized Extreme Value distribution and rational method were used to delineate the drainage system, classify of the sub watershed, and simulate the design rainfall depth and runoff volume estimation respectively. the peak discharge generated in 2002 was 54.45m^3/s. Due to both intense rainfall and urban tight area the peak discharge generated counts 102.80m^3/s(i.e. the peak discharge is doubled) in 2019. The surplus of discharge by intense rainfall from the base year (2002) is 36.76m^3/s (67.51%); while the surplus of discharge by urbanization is 12.1m^3/s (22.49%). The peak discharge increased by intense rainfall than urban tight area. This factor exacerbated the drainage system malfunctioning in the area. Following the drainage system networking, capacity of 13drains (59.09%) exceeded by design discharge, 20(90.91%) by 25 years discharge and 21(95.45%) drains exceeded by 100 years discharge. Solid waste and sewage made the drainage system **new tomb**. Water in drainage system persists above a week in drains and the drainage system of the study area is rated as **poor** based on AASHTO relationship between layer's drain ability and subjective drainage quality rating**.** Pedestrian's mobility was interrupted by the resulted flood. Traffic congestion and delay are the common problems in the area. Asphalt road in the study area is in chronic distress. Income of Telecom establishment and other service centers is shortened. Sanitary disease in the area is above 54.45% starting from 2010 till now. Sizing and Rehabilitation measure of urban drainage system and retention pond construction in the area highly affected national economy.

5.2 Recommendations

- Most of storm water drainage systems are under capacity. So, SUDs measures such as sizing drain and retention pond should be implemented as the mitigation strategies to cope with intense rainfall and urbanization impact on urban drainage system and environment.

- In most part of the city litter and sewage disposed in to storm water drainage systems. Storm water drainage systems to be cost-effective and operationally sustainable, greater emphasis needs to be placed upon communication, between professionals responsible for drainage. Effective drainage area and catchment planning need careful coordination between the related institutions accountable for water supply, sewerage, drainage and solid waste management.
- Storm water Drainage system planning is poorly integrated with local planning and development policies. Therefore, it should be integrated with local planning documents and policies (development program, construction regulations, spatial plans etc). These plans then can be arranged and united with flood prevention plan to make a whole system that works. It is recommended to implement a strategy that all the areas of new development which exceeds 1ha should include BMPs solutions. Storm water management must be executed within the given parcel to not to increase load for the existing drainage system.

For future study:

- As urbanization is the headache for developing country including Hawassa city/Ethiopia, to compute effective imperviousness area (EIF) either increasing or decreasing, an area factor (fA) should be determined to predict urbanization impact.
- To suggest other BMPs as the mitigation strategies for intense rainfall and urbanization, it needs due study(experiments) on the effectiveness of the LID controls and SUDs measures such as bio-retention cells, green roofs, pervious areas, roof top disconnect, rain gardens, infiltration trenches, rain barrels.
- As the urban runoff contains the pollutant load, runoff's quality (physic-chemical) experiments and its effect on Lake Biodiversity should be conducted.
- Lake shore receives the 25 years peak discharge which is greater than $84m^3/s$ or 100 years peak discharge$130m^3/s$ in a single rain event. During heavy rain season the lake level may rise and could possibly flow back in this region. This dual effect will cause the region more stressed. So municipality should conducted study considering this issue and have to take remedial measures.

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APPENDICES

Appendix1.1: an outlier detection, rainfall consistency, homogeneity, and climate variability computation of the study area.

Table **1—1** maximum rainfall data and its outlier detection

		outlier test within					within	
year	Daily max. pr.		3SD+M	3SD-M	Year	D. max.	3SD+M	3SD-M
1989	59.7		97.96	-4.18	1989	59.7	82.94	-15.24
1990	51.6		97.96	-4.18	1990	51.6	82.94	-15.24
1991	46.6		97.96	-4.18	1991	46.6	82.94	-15.24
1992	50.8		97.96	-4.18	1992	50.8	82.94	-15.24
1993	44.1		97.96	-4.18	1993	44.1	82.94	-15.24
1994	40.4		97.96	-4.18	1994	40.4	82.94	-15.24
1995	56.8		97.96	-4.18	1995	56.8	82.94	-15.24
1996	44.6		97.96	-4.18	1996	44.6	82.94	-15.24
1997	47.2		97.96	-4.18	1997	47.2	82.94	-15.24
1998	58.7		97.96	-4.18	1998	58.7	82.94	-15.24
1999	44.6		97.96	-4.18	1999	44.6	82.94	-15.24
2000	110.4	outlier	97.96	-4.18	2000	rejected	82.94	-15.24
2001	42.7		97.96	-4.18	2001	42.7	82.94	-15.24
2002	67.6		97.96	-4.18	2002	67.6	82.94	-15.24
2003	37		97.96	-4.18	2003	37	82.94	-15.24
2004	28.8		97.96	-4.18	2004	28.8	82.94	-15.24
2005	42.4		97.96	-4.18	2005	42.4	82.94	-15.24
2006	54.6		97.96	-4.18	2006	54.6	82.94	-15.24
2007	71.5		97.96	-4.18	2007	71.5	82.94	-15.24
2008	32.8		97.96	-4.18	2008	32.8	82.94	-15.24
2009	37.4		97.96	-4.18	2009	37.4	82.94	-15.24
2010	53.6		97.96	-4.18	2010	53.6	82.94	-15.24
2011	33.7		97.96	-4.18	2011	33.7	82.94	-15.24
2012	37.6		97.96	-4.18	2012	37.6	82.94	-15.24
2013	43.3		97.96	-4.18	2013	43.3	82.94	-15.24
2014	69.8		97.96	-4.18	2014	69.8	82.94	-15.24
2015	48.3		97.96	-4.18	2015	48.3	82.94	-15.24
2016	49		97.96	-4.18	2016	49	82.94	-15.24
2017	55.5		97.96	-4.18	2017	55.5	82.94	-15.24
2018	51.8		97.96	-4.18	2018	51.8	82.94	-15.24
2019	70.2		97.96	-4.18	2019	70.2	82.94	-15.24
Mean	51.0677				Mean	49.09		
St. dev	15.6298				St.dev	11.2818		

Table: 1—2.annual and cumulative rainfall of stations

annual rainfall of the stations							cumulative rainfall of the stations				
Year	Hawass	Moroch	wendo	halaba	S.neigh	Av.neig	Hawasa	Moroch	we com	ha com	Av.neig
1989	1026.3	1861.9	1240.5	1083.5	4185.9	1395.3	1026.3	1861.9	1240.5	1083.5	1395.3
1990	772.1	1757.6	931	968	3656.6	1218.9	1798.4	3619.5	2171.5	2051.5	2614.2
1991	904.9	1572.4	1213.5	1045.7	3831.6	1277.2	2703.3	5191.9	3385.0	3097.3	3891.4
1992	1013.6	1580	1296.4	1162.6	4039	1346.3	3716.9	6771.9	4681.4	4259.9	5237.7
1993	991	1298.1	1124.1	1272.2	3694.4	1231.5	4707.9	8070.0	5805.5	5532.1	6469.2
1994	925.8	1546	1288.2	806.1	3640.3	1213.4	5633.7	9616.0	7093.7	6338.2	7682.6
1995	1028	1490.5	1024.5	975.6	3490.6	1163.5	6661.7	11106.	8118.2	7313.8	8846.2
1996	1221.7	1862.5	1253.1	1160	4275.6	1425.2	7883.4	12969.	9371.3	8473.8	10271.
1997	1085.3	1384.3	1253.8	1193.5	3831.6	1277.2	8968.7	14353.	10625.	9667.3	11548.
1998	1154.9	1194.1	1304.6	1221.2	3719.9	1240.0	10123.	15547.	11929.	10888.	12788.
1999	816.9	950.7	916.5	763.4	2630.6	876.9	10940.	16498.	12846.	11651.	13665.
2000	837.2	959.9	943.1	876	2779	926.3	11777.	17458.	13789.	12527.	14591.
2001	1086	1214.6	1290.5	944.2	3449.3	1149.8	12863.	18672.	15079.	13472.	15741.
2002	961.8	856.9	888.8	796	2541.7	847.2	13825.	19529.	15968.	14268.	16588.
2003	893.2	999.9	1089.9	943.3	3033.1	1011.1	14718.	20529.	17058.	15211.	17599.
2004	932.5	1054.5	1034.2	992.6	3081.3	1027.1	15651.	21583.	18092.	16204.	18626.
2005	1060.8	1287.7	1144.2	866.5	3298.4	1099.5	16712.	22871.	19237.	17070.	19726.
2006	1262.6	1217.9	1288.5	873.8	3380.2	1126.7	17974.	24089.	20525.	17944.	20853.
2007	1197.1	1298.9	1133.9	1081.6	3514.4	1171.5	19171.	25388.	21659.	19025.	22024.
2008	935.5	992.3	860.6	950.3	2803.2	934.4	20107.	26380.	22520.	19976.	22958.
2009	722	747.6	694.8	845.8	2288.2	762.7	20829.	27128.	23214.	20822.	23721.
2010	1096.6	785.5	1423.9	1800.4	4009.8	1336.6	21925.	27913.	24638.	22622.	25058.
2011	955.9	937.5	1362.2	956	3255.7	1085.2	22881.	28851.	26000.	23578.	26143.
2012	828.1	827.9	793.56	752.3	2373.7	791.3	23709.	29679.	26794.	24330.	26934.
2013	1118.1	1037.6	1451.7	2017.6	4506.9	1502.3	24827.	30716.	28246.	26348.	28437.
2014	1194.8	1017	615.7	1181.1	2813.8	937.9	26022.	31733.	28861.	27529.	29375.
2015	697.6	771.3	344.3	531.3	1646.9	549.0	26720.	32505.	29206.	28060.	29924.
2016	992.6	1167.3	1021.4	1214.8	3403.5	1134.5	27712.	33672.	30227.	29275.	31058.
2017	1152.1	808.9	1152.4	800.1	2761.4	920.5	28865.	34481.	31380.	30075.	31979.
2018	1180.6	1059.7	1340.5	1000.5	3400.7	1133.6	30045.	35541.	32720.	31076.	33112.

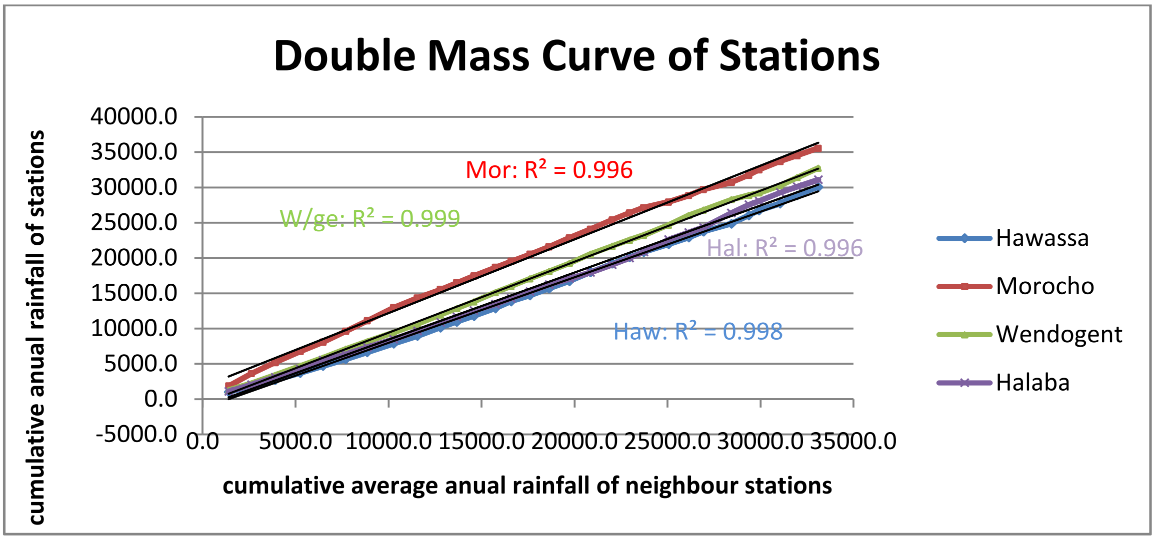

Figure **1-1** consistency of the stations

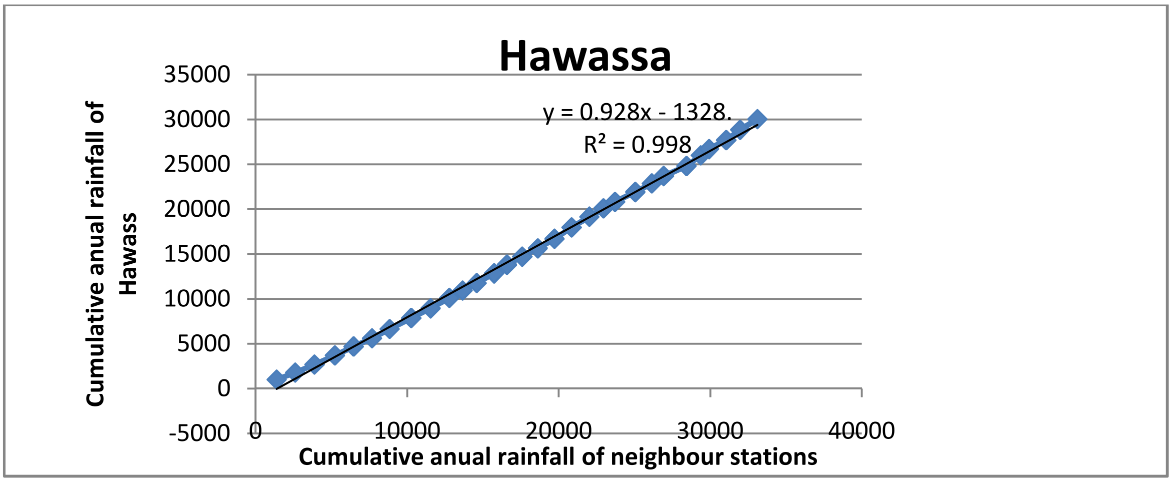

Figure 1-2 consistency of Hawassa station

Table 1—3 Monthly average rainfall of stations

Average monthly rain fall of the stations													
Months		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Stations	Moro	37.1	53.8	111.9	147.2	144.6	94.1	112.6	113.1	166	128.3	51	24.5
	w/gen	30.6	44	95.72	139.5	119.1	92.1	124.7	145.8	138	91.61	47	22.1
	hawa	27.5	39.2	78.16	116.5	126.3	107	125.5	125.2	122.2	74.98	39.3	21.7
	halab	29.5	47.5	89.31	140	121.9	94.2	115.2	138.2	108.9	66.71	61.8	22

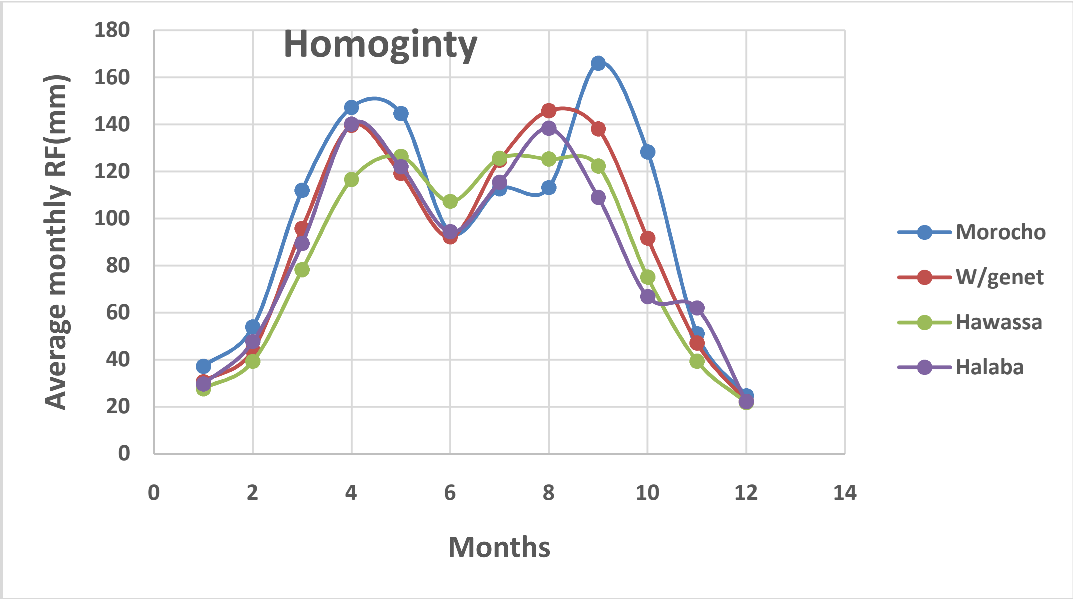

Figure 1- 3 homogeneity of the stations

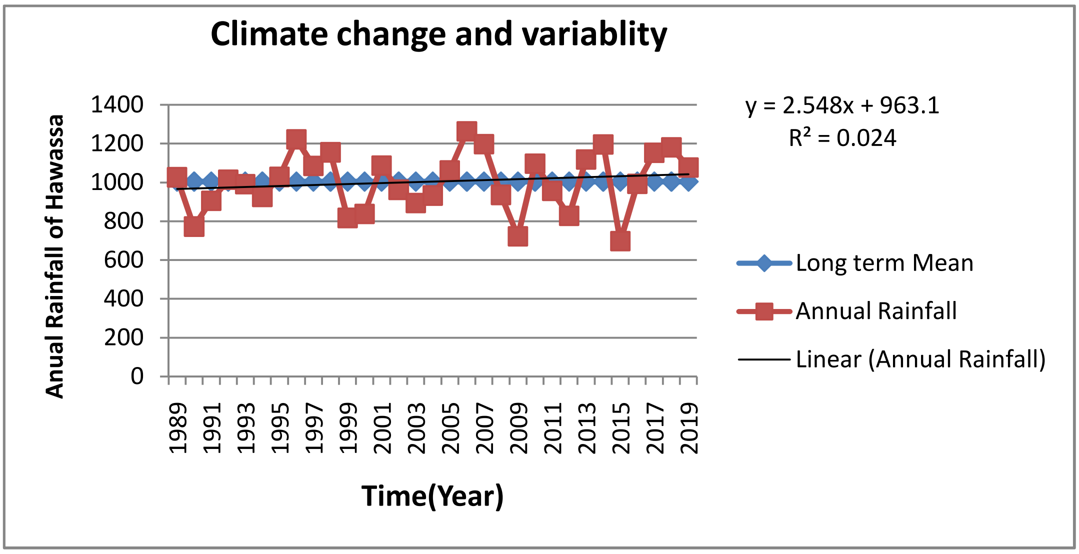

Figure1- 4 climate change and variablity of study area

Table 1—4 statistics of climate change and variability

Mean	1003.929
Variance	22391.52
Standard Deviation	149.638
Coefficient Of Variation	0.149
Skew	-3.1E+12

Appendix 1.2 design rainfall frequency**,** IDF curve and its comparison with ERA for rainfall region B2

The value of RRt by using $\boldsymbol{R}_{\mathbf{Rt}} = \frac{t}{24}\ \frac{(b+24)^n}{(b+t)^n}$ for n = 0.92 and b = 0.3

t(min)	5 min	10 min	15min	20min	25 min	30 min	60 min	90 min	120 min	150 min	180 min
t(hr)	0.08	0.17	0.25	0.33	0.42	0.5	1	1.5	2	2.5	3
b+24	24.3	24.3	24.3	24.3	24.3	24.3	24.3	24.3	24.3	24.3	24.3
(b+24)^n	18.83	18.83	18.83	18.83	18.83	18.83	18.83	18.83	18.83	18.83	18.83
b+t	0.38	0.47	0.55	0.63	0.72	0.8	1.3	1.8	2.3	2.8	3.3
(b+t)^n	0.41	0.5	0.58	0.66	0.74	0.81	1.27	1.72	2.15	2.58	3
t/24	0	0.01	0.01	0.01	0.02	0.02	0.04	0.06	0.08	0.1	0.13
RRt	0.16	0.26	0.34	0.4	0.44	0.48	0.62	0.69	0.73	0.76	0.78

$P_t = £ + (\frac{\alpha}{k})\left[1 - (-log\frac{(T-1)}{T})^K\right]$

Since 2002 $£ = 45.785, \alpha = 5.357$ and $k = -0.0199$

Since 2011 $£ = 44.696, \alpha = 9.8455$ and $k = -0.08899$

Since 2019 $£ = 45.65, \alpha = 9.9202$ and $k = -0.09227$

Table 1—5 design rainfall and return period

T(Years)	2	5	10	25	50	100	200
R24(2002)	52.29	58.58	62.82	68.28	72.39	76.53	80.71
R24(2011)	57.17	70.24	79.64	92.46	102.69	113.5	124.96
R24(2019)	58.24	71.49	81.05	94.11	104.57	115.65	127.41
R24 of ERA for B2(2013)	55.26	69.95	79.68	92.03	101.29	110.61	120.07

Table 1—6 rainfall for shorter duration

Rt =RRt * R24(2019)								
T(year)	2	5	10	25	50	100	200	RRt
R24	58.244	71.486	81.046	94.112	104.571	115.646	127.414	
	9.198	11.29	12.8	14.863	16.515	18.264	20.122	0.158
	15.352	18.842	21.361	24.805	27.562	30.481	33.583	0.264
	19.797	24.298	27.548	31.989	35.544	39.308	43.308	0.34
	23.183	28.454	32.259	37.46	41.623	46.031	50.715	0.398
	25.864	31.744	35.989	41.792	46.436	51.354	56.58	0.444
	28.05	34.426	39.031	45.323	50.36	55.694	61.361	0.482
	35.89	44.049	49.94	57.991	64.436	71.261	78.512	0.616
	39.906	48.979	55.529	64.481	71.647	79.235	87.298	0.685
	42.466	52.12	59.091	68.617	76.243	84.318	92.897	0.729
	44.295	54.365	61.636	71.573	79.527	87.949	96.899	0.761
	45.697	56.086	63.587	73.838	82.044	90.733	99.966	0.785

Table 1—7 rainfall intensity for shorter duration

Intensity (mm/hr)=Rt/duration(2019)								**DURATION**	
T	2	5	10	25	50	100	200	minutes	in hours
	110.38	135.48	153.59	178.36	198.18	219.17	241.47	5 min	0.08333
	92.11	113.05	128.17	148.83	165.37	182.89	201.5	10 min	0.16667
	79.19	97.19	110.19	127.96	142.18	157.23	173.23	15min	0.25
	69.55	85.36	96.78	112.38	124.87	138.09	152.15	20min	0.33333
	62.07	76.19	86.37	100.3	111.45	123.25	135.79	25 min	0.41667
	56.1	68.85	78.06	90.65	100.72	111.39	122.72	30 min	0.5
	35.89	44.05	49.94	57.99	64.44	71.26	78.51	60 min	1
	26.6	32.65	37.02	42.99	47.76	52.82	58.2	90 min	1.5
	21.23	26.06	29.55	34.31	38.12	42.16	46.45	120 min	2
	17.72	21.75	24.65	28.63	31.81	35.18	38.76	150 min	2.5
	15.23	18.7	21.2	24.61	27.35	30.24	33.32	180 min	3

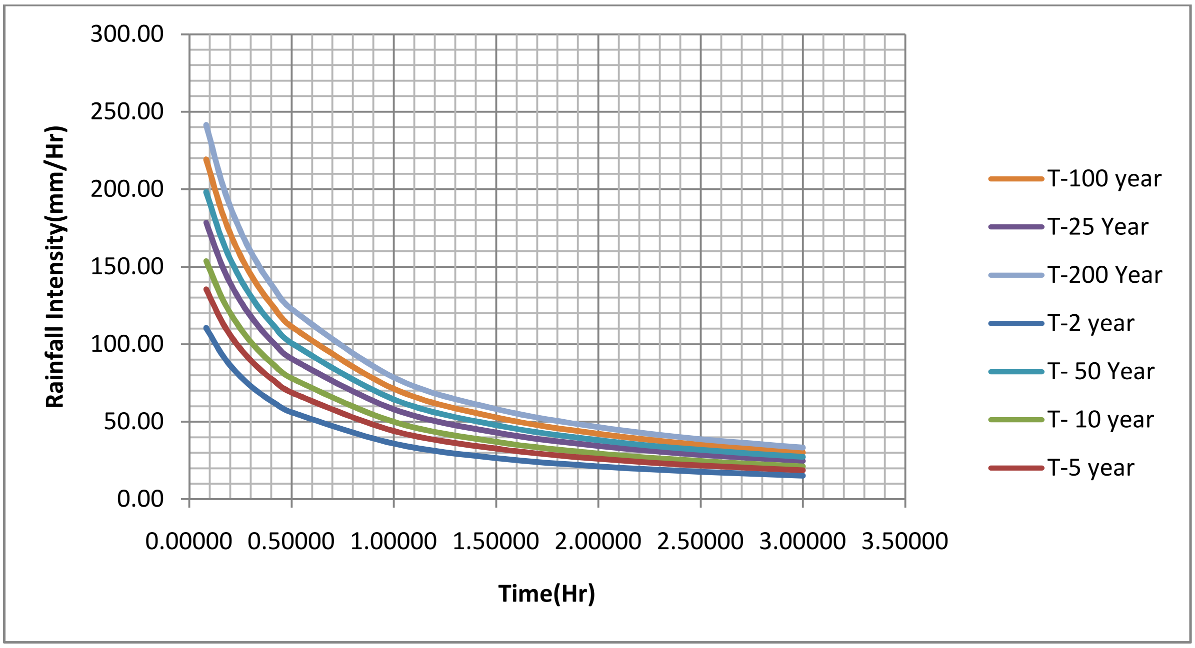

Figure 1-5 IDF curve of Hawassa station

IDF curve for region b2 from era drainage design manual (2013)

Table 1—8 24hr rainfall depth Vs Frequency

24hrs rainfall depth(mm) Vs Frequency(year)								
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

(Source ERA DDM 2013)

Table 1—9 rainfall depth for shorter duration per ERA drainage design manual for region B2

$R_t = RR_t * R_{24}$									
T (years)	2	5	10	25	50	100	200	RRt	DURATION
R24	55.26	69.95	79.68	92.03	101.29	110.61	120.07		in minutes
Rt	8.7	11	12.6	14.5	16	17.5	19	0.1579	5 min
	14.6	18.4	21	24.3	26.7	29.2	31.7	0.2636	10 min
	19.3	24.5	27.9	32.2	35.5	38.7	42	0.3399	15min
	22	27.8	31.7	36.6	40.3	44	47.8	0.3980	20min
	24.5	31.1	35.4	40.9	45	49.1	53.3	0.4441	25 min
	26.6	33.7	38.4	44.3	48.8	53.3	57.8	0.4816	30 min
	34.1	43.1	49.1	56.7	62.4	68.2	74	0.6162	60 min
	37.9	47.9	54.6	63	69.4	75.8	82.3	0.6852	90 min
	40.3	51	58.1	67.1	73.9	80.6	87.5	0.7291	120 min
	42	53.2	60.6	70	77	84.1	91.3	0.7605	150 min
	43.4	54.9	62.5	72.2	79.5	86.8	94.2	0.7846	180 min

Table 1—10 rainfall intensity for shorter duration for region B2

Intensity (mm/hr)=Rt/duration								**DURATION**	
T	2	5	10	25	50	100	200	in minutes	in hours
	104.73	132.57	151.01	174.41	191.96	209.62	227.55	5 min	0.08333
	87.4	110.63	126.02	145.55	160.2	174.94	189.9	10 min	0.16667
	77.36	97.93	111.55	128.84	141.81	154.85	168.1	15min	0.25
	65.98	83.52	95.14	109.88	120.94	132.07	143.36	20min	0.33333
	58.89	74.54	84.91	98.07	107.93	117.87	127.95	25 min	0.41667
	53.23	67.38	76.75	88.64	97.56	106.54	115.65	30 min	0.5
	34.05	43.1	49.1	56.71	62.41	68.16	73.99	60 min	1
	25.24	31.95	36.39	42.03	46.26	50.52	54.84	90 min	1.5
	20.15	25.5	29.05	33.55	36.93	40.32	43.77	120 min	2
	16.81	21.28	24.24	28	30.81	33.65	36.53	150 min	2.5
	14.45	18.29	20.84	24.07	26.49	28.93	31.4	180 min	3

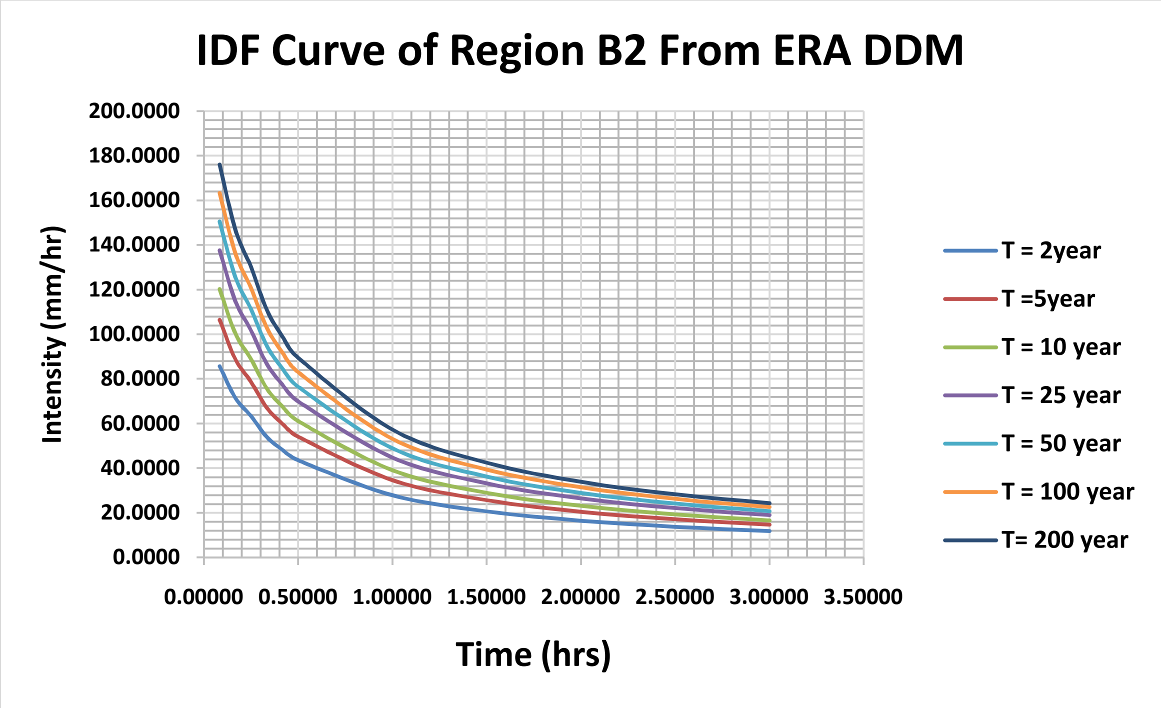

Figure 1-6 IDF curve of region B2 from ERA DDM

Table 1—11 Comparison of the IDF curve result with that of region B2

Intensity (mm/hr)=Rt/duration												
DURA TION	Self	ERA	Self	ERA	Self	ERA	Self	ERA	Self	ERA	Self	ERA
T(min)	T-2	T-2	T-5	T-5	T-10	T-10	T-25	T-25	T-50	T-50	T-100	T-100
5	110.4	104.7	135.5	132.6	153.6	151	178.4	174.4	198.2	192	219.2	209.6
10	92.1	87.4	113	110.6	128.2	126	148.8	145.6	165.4	160.2	182.9	174.9
15	79.2	77.4	97.2	97.9	110.2	111.6	128	128.8	142.2	141.8	157.2	154.9
20	69.5	66	85.4	83.5	96.8	95.1	112.4	109.9	124.9	120.9	138.1	132.1
25	62.1	58.9	76.2	74.5	86.4	84.9	100.3	98.1	111.4	107.9	123.2	117.9
30	56.1	53.2	68.9	67.4	78.1	76.7	90.6	88.6	100.7	97.6	111.4	106.5
60	35.9	34.1	44	43.1	49.9	49.1	58	56.7	64.4	62.4	71.3	68.2
90	26.6	25.2	32.7	31.9	37	36.4	43	42	47.8	46.3	52.8	50.5
120	21.2	20.1	26.1	25.5	29.5	29	34.3	33.5	38.1	36.9	42.2	40.3
150	17.7	16.8	21.7	21.3	24.7	24.2	28.6	28	31.8	30.8	35.2	33.6
180	15.2	14.5	18.7	18.3	21.2	20.8	24.6	24.1	27.3	26.5	30.2	28.9

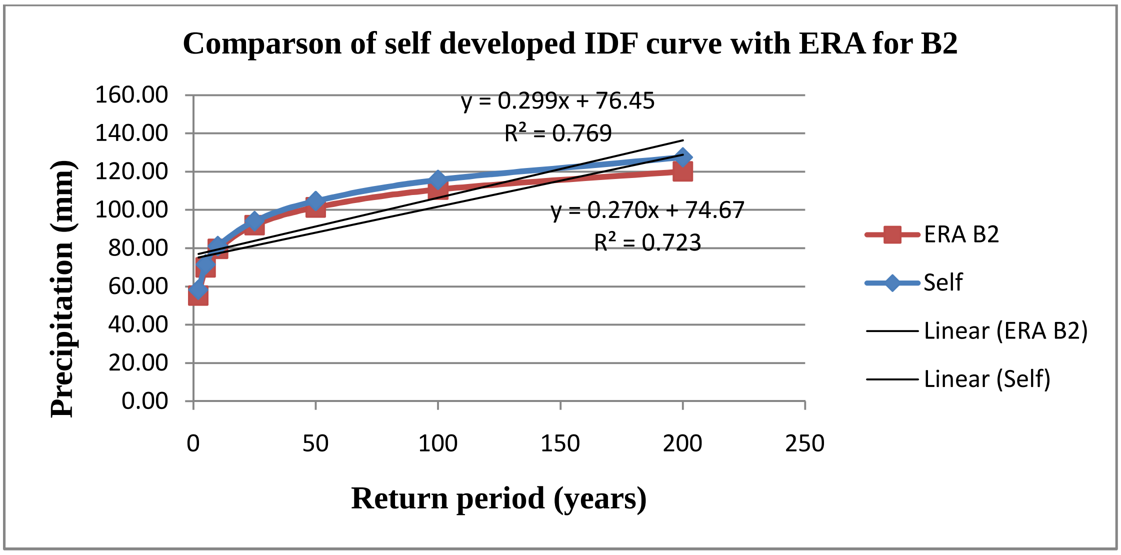

Figure 1-7 plots of frequency analysis results

Table 1—12 roughness coefficients for sheet flow

Roughness coefficients (manning's n) for sheet flow	
surface description	n
smooth surfaces (concrete, asphalt, gravel or bare soil	0.011
fallow no residue	0.05
cultivated soils	
residue cover ≤ 20%	0.06
residue cover >20%	0.17
grasses	
short grass	0.15
dense grasses	0.24
range(natural)	0.13
woods	
less underbrush	0.4
dense underbrush	0.8
Note: when selecting n, consider a cover to a height of about 0.03m. This is the part of only plant cover that will obstruct sheet flow	

(Source: ERA DDM 2001)

Table 1—13 time of concentration for drainage segments

Drainage Code	TIME OF CONSENTRATION				
	sheet flow(min)	shallow conc. Flow(min	System(min)	Tc	decision
CH2	3.851	0.222	1.566	5.638	15
CH3	2.654	3.594	1.791	8.039	15
CH1	23.745	3.406	6.41	33.562	34
CH4	35.94	11.389	8.589	55.918	56
CH6	25.677	0.069	10.111	35.857	36
CH5	4.367	8.358	8.81	21.534	22
CH7	2.385	6.432	22.28	31.097	31
CH8A	6.698	3.444	8.39	18.533	19
CH8B	5.166	3.14	3.595	11.9	15
CH8C	4.997	3.14	22.17	30.307	30
CH12			8.271	8.271	15
CH13	5.017	2.541	16.69	15.689	24
CH12A	58.814	7.432	12.679	78.924	79
CH15	2.798	4.615	3.906	11.318	15
CH17	2.564	0	15.67	18.234	19
CH16	5.511	5.405	10.67	21.586	22
CH19	6.527	4.179	5.67	16.376	16
CH18			7.1	7.1	15
CH14	4.528	3.678	5.301	13.508	15
CH12C	4.336	2.889	4.37	11.6	15
	Average Time of Concentration				25.45

Table 1—14 rainfall intensity for each segment

Drainage Code	Intensity(mm/hr)					
	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year
CH2	79.19	97.19	110.19	127.96	142.18	157.23
CH3	79.19	97.19	110.19	127.96	142.18	157.23
CH1	53.4	65.55	74.31	86.29	95.88	106.04
CH4	38.59	47.36	53.69	62.35	69.28	76.62
CH6	52.06	63.89	72.44	84.12	93.46	103.36
CH5	66.56	81.69	92.62	107.55	119.5	132.16
CH7	55.43	68.03	77.12	89.56	99.51	110.05

CH8A	71.48	87.73	99.46	115.49	128.33	141.92
CH8B	79.19	97.19	110.19	127.96	142.18	157.23
CH8C	55.43	68.03	77.12	89.56	99.51	110.05
CH12	79.19	97.19	110.19	127.96	142.18	157.23
CH13	63.57	78.02	88.46	102.72	114.13	126.22
CH12A	30.01	36.83	41.76	48.49	53.88	59.58
CH15	79.19	97.19	110.19	127.96	142.18	157.23
CH17	71.48	87.73	99.46	115.49	128.33	141.92
CH16	66.56	81.69	92.62	107.55	119.5	132.16
CH19	77.26	94.83	107.51	124.84	138.71	153.41
CH18	79.19	97.19	110.19	127.96	142.18	157.23
CH14	79.19	97.19	110.19	127.96	142.18	157.23
CH12C	79.19	97.19	110.19	127.96	142.18	157.23

Appendix 2 Existing capacity of the drainage system and AASHTO relationships between layers drain ability and subjective drainage quality rating.

Table 2—1 drainage segments and their capacity

Drainage Segments and their existing Capacity																					
From	To	geometry	position	L	Wt	Wb	D	A	P	R	R^2/3	HE	LE	H	S	S^1/2	n	V	Q	Qt	Tc(min)
Atote Traffic Light	Wanza Square	U- Shape	R	1080.8	1.500		1.500	2.250	4.500	0.500	0.630	1721	1715	6.000	0.006	0.075	0.032	1.467	3.300		12.28
			L	1080.	0.700		0.500	0.350	1.700	0.206	0.349	1721	1715	6.000	0.006	0.075	0.032	0.812	0.284	3.584	22.18
Welde amanuel	Lake Side	U- Shape	R	528.7	1.200		0.500	0.600	2.200	0.273	0.421	1750	1734	16.00	0.030	0.174	0.032	2.286	1.372		3.85
			L	528.7	1.200		0.500	0.600	2.200	0.273	0.421	1750	1734	16.00	0.030	0.174	0.032	2.286	1.372	2.743	3.85
Lake Side School Trafic	wanza squer	U- Shape	R	984.2	1.200		0.500	0.600	2.200	0.273	0.421	1734	1719	15.00	0.015	0.123	0.032	1.622	0.973		10.11
			L	984.26	1.200		0.500	0.600	2.200	0.273	0.421	1734	1719	15.000	0.015	0.123	0.032	1.622	0.973	1.947	10.111
Lake Side	Gezah	U-	R	515.97	0.900		0.800	0.720	2.500	0.288	0.436	1736	1731	5.000	0.010	0.098	0.032	1.342	0.966		6.4099

sidama buna	Hawassa High Court	wanza squar		total Diesel	shell Diesel	daite			Hayole Trafic Light	
Mebrathavil	Yesh trafic	Yesh trafic light		wanza	wanza	stadium			Lake Side	
	U- Shape							U- Shape	U- Shape	
R	R	L	R	R	L	L	R	R	L	L
528.470	642.096	268.140	259.32	328.280	507.850	517.21	517.2	370.79	370.79	515.970
0.300	0.700	1.200	0.800	0.800	1.200	0.650	0.800	0.8	1.2	0.900
0.500	0.600	1.200	0.800	0.800	0.800	0.500	0.350	0.6	0.7	0.800
0.150	0.420	1.440	0.640	0.640	0.960	0.325	0.280	0.480	0.840	0.720
1.300	1.900	3.600	2.400	2.400	2.800	1.650	1.500	2.000	2.600	2.500
0.115	0.221	0.400	0.267	0.267	0.343	0.197	0.187	0.240	0.323	0.288
0.237	0.366	0.543	0.414	0.414	0.490	0.339	0.327	0.386	0.471	0.436
1716	1720	1715	1715	1716	1717	1724	1724	1736	1736	1736
1712	1712	1712	1712	1715	1715	1719	1719	1731	1731	1730
4.000	8.000	3.000	3.000	1.000	2.000	5.000	5.000	5.000	5.000	6.000
0.008	0.012	0.011	0.012	0.003	0.004	0.010	0.010	0.013	0.013	0.012
0.087	0.112	0.106	0.108	0.055	0.063	0.098	0.098	0.116	0.116	0.108
0.032	0.032	0.032	0.032	0.032	0.032	0.032	0.032	0.032	0.032	0.032
0.644	1.275	1.794	1.393	0.715	0.961	1.040	1.004	1.401	1.709	1.470
0.097	0.536	2.584	0.891	0.457	0.922	0.338	0.281	0.673	1.435	1.058
		3.475				0.619		2.108		2.024
13.668913	8.3918276	2.490427	3.1036	7.6568983	8.810721	8.2873	8.589	4.4095904	3.6168885	5.85146

Guadguada Square		Solomon Abegaz	Korem Warka	Korem Meda	Tesso Secondary School		Total Diesel		shell	yesh trafic light
22Mazoriya	AVERAGE	Guadguada	Solomon	Korem	Korem		Korem Meda		total	mahilet bld
U- Shape		U- Shape	U- Shape	U- Shape	U- Shape		U- Shape		U- Shape	
R		R	L	L	L	R	L	R	R	R
370.800	729.570	117.440	272.320	339.810	264.87	264.87	1019.6	1019.6	211.840	368.140
1.500	1.600	1.300	2.000	1.500	1.000	0.450	1.150	1.000	0.800	0.850
2.500	1.767	1.200	2.500	1.600	0.500	0.600	1.450	1.500	0.800	0.800
3.750	2.827	1.560	5.000	2.400	0.500	0.270	1.668	1.500	0.640	0.680
6.500	5.133	3.700	7.000	4.700	2.000	1.650	4.050	4.000	2.400	2.450
0.577	0.551	0.422	0.714	0.511	0.250	0.164	0.412	0.375	0.267	0.278
0.693	0.672	0.562	0.799	0.639	0.397	0.299	0.553	0.520	0.414	0.425
1707	1712	1708	1709	1712	1714	1714	1716	1716	1717	1712
1706	1707	1707	1708	1709	1712	1712	1712	1712	1716	1711
1.000	5.000	1.000	1.000	3.000	2.000	2.000	4.000	4.000	1.000	1.000
0.003	0.007	0.009	0.004	0.009	0.008	0.008	0.004	0.004	0.005	0.003
0.052	0.083	0.092	0.061	0.094	0.087	0.087	0.063	0.063	0.069	0.052
0.032	0.032	0.032	0.032	0.032	0.032	0.032	0.032	0.032	0.032	0.032
1.125	1.738	1.621	1.513	1.876	1.078	0.812	1.083	1.018	0.890	0.693
4.218	4.913	2.529	7.566	4.502	0.539	0.219	1.806	1.527	0.569	0.471
10.382	4.913				0.758		3.333			
5.4949411	6.996230	1.2071861	2.9994162	3.0191411	4.0964	5.4339	15.687	16.695	3.9691562	8.8536637

mobil		silasie		Artistic Printing		AIB	irsha tabia	mobil		22Mazoriya	
st.George beer		mobil		Chambala		Artistic	AIB	Artstic printing		Piazza-Mobill Road	
U- Shape		U- Shape		U- Shape	U- Shape	U- Shape	Trapezoidal			U- Shape	
L	R	L	R	R	l	R	R	L	R	L	R
477.2	477.270	555.690	555.69	471.650	335.210	335.210	880.120	499.93	499.93	542.200	542.200
1.000	1.000	1.000	1.000	1.900	1.100	1.100	1.500	1.100	0.800	0.900	1.400
							0.450				
1.000	1.000	1.000	1.000	0.900	1.500	1.500	0.600	1.100	0.700	0.600	2.200
1.000	1.000	1.000	1.000	1.710	1.650	1.650	0.585	1.210	0.560	0.540	3.080
3.000	3.000	3.000	3.000	3.700	4.100	4.100	2.045	3.300	2.200	2.100	5.800
0.333	0.333	0.333	0.333	0.462	0.402	0.402	0.286	0.367	0.255	0.257	0.531
0.481	0.481	0.481	0.481	0.598	0.545	0.545	0.434	0.512	0.402	0.404	0.656
1703	1703	1712	1712	1702	1702	1702	1713	1703	1703	1706	1706
1698	1698	1703	1703	1696	1701	1701	1702	1701	1701	1703	1703
5.000	5.000	9.000	9.000	6.000	1.000	1.000	11.000	2.000	2.000	3.000	3.000
0.010	0.010	0.016	0.016	0.013	0.003	0.003	0.012	0.004	0.004	0.006	0.006
0.102	0.102	0.127	0.127	0.113	0.055	0.055	0.112	0.063	0.063	0.074	0.074
0.032	0.032	0.032	0.032	0.032	0.032	0.032	0.032	0.032	0.032	0.032	0.032
1.538	1.538	1.912	1.912	2.107	0.930	0.930	1.517	1.013	0.794	0.940	1.524
1.538	1.538	1.912	1.912	3.603	1.535	1.535	0.887	1.225	0.445	0.508	4.695
3.075		9.413		8.868	3.070			1.670		12.806	
5.172	5.17297	4.84403	4.8440	3.730983	6.0049231	6.0049231	9.6695401	8.2287	10.495	9.6137847	5.92828

chambalala	kalehiywot church office	Right Corner of Market	At the market	Areb sefer Mosque		AIB	irsha tabia	st.george
ermi guest	ermi guest	Kale Hiywot	Right corner	Market		Areb sefer	AIB	chambalala(
U- Shape	U- Shape	U- Shape	U- Shape	U- Shape	U- Shape	U- Shape	Trapezoidal	U- Shape
L	R	R	R	L	R	L	L	L
255.060	96.140	299.130	103.250	211.360	98.620	98.620	880.120	468.310
2.000	2.000	2.000	2.000	2.000	0.800	1.000	1.500	1.100
							0.450	
1.400	1.400	1.400	1.400	1.400	0.700	1.200	0.600	1.100
2.800	2.800	2.800	2.800	2.800	0.560	1.200	0.585	1.210
4.800	4.800	4.800	4.800	4.800	2.200	3.400	2.045	3.300
0.583	0.583	0.583	0.583	0.583	0.255	0.353	0.286	0.367
0.698	0.698	0.698	0.698	0.698	0.402	0.499	0.434	0.512
1696	1695	1697	1698	1701	1702	1702	1713	1698
1694	1694	1695	1697	1698	1701	1701	1702	1696
2.000	1.000	2.000	1.000	3.000	1.000	1.000	11.000	2.000
0.008	0.010	0.007	0.010	0.014	0.010	0.010	0.012	0.004
0.089	0.102	0.082	0.098	0.119	0.101	0.101	0.112	0.065
0.032	0.032	0.032	0.032	0.032	0.032	0.032	0.032	0.032
1.932	2.225	1.784	2.147	2.599	1.264	1.572	1.517	1.046
5.409	6.230	4.995	6.012	7.278	0.708	1.886	0.887	1.266
	6.129				2.594			
2.2004056	0.7201291	2.7946611	0.8014737	1.3552761	1.300477	1.0458707	9.6695401	7.4605284

arebsefer trafic light		tabor trafic light		south star		Beshu Hotel	sidama buna	Sidama zone F/departme	Sidama zone F/departme	Bunena Masamo
global auto service		arebsefer trafic		tabor trafic		south star	south star	Sidama Buna	Trurat Trafic	Gabriel
CIRCULAR		CIRCULAR		CIRCULAR		CIRCULAR	CIRCULAR	Trapezoidal	Trapezoidal	U- Shape
L	R	L	R	L	R	R	L	R	L	R
485.160	485.1	541.76	541.76	382.26	382.2	390.470	232.250	661.197	426.040	755.000
								0.730	0.730	0.500
								0.500	0.500	
0.600	1.000	0.600	1.000	0.600	1.000	1.000	0.400	0.330	0.330	0.600
0.283	0.785	0.283	0.785	0.283	0.785	0.785	0.126	0.203	0.203	0.300
1.884	3.140	1.884	3.140	1.884	3.140	3.140	1.256	1.304	1.304	1.700
0.150	0.250	0.150	0.250	0.150	0.250	0.250	0.100	0.156	0.156	0.176
0.282	0.397	0.282	0.397	0.282	0.397	0.397	0.215	0.289	0.289	0.315
1700	1700	1711	1711	1715	1715	1716	1716	1718	1718	1721
1693	1693	1700	1700	1711	1711	1715	1715	1716	1716	1717
7.000	7.000	11.000	11.000	4.000	4.000	1.000	1.000	2.000	2.000	4.000
0.014	0.014	0.020	0.020	0.010	0.010	0.003	0.004	0.003	0.005	0.005
0.120	0.120	0.142	0.142	0.102	0.102	0.051	0.066	0.055	0.069	0.073
0.013	0.013	0.013	0.013	0.013	0.013	0.013	0.013	0.032	0.032	0.032
2.608	3.667	3.094	4.350	2.221	3.123	1.545	1.087	0.497	0.619	0.716
0.737	2.878	0.874	3.415	0.628	2.451	1.213	0.137	0.101	0.126	0.215
3.616		4.289		3.079						
3.099867	2.205	2.9179	2.0757	2.8679	2.040	4.2125681	3.5595275	22.165657	11.464625	17.583592

Water Works Construction		Lewi Cumpass		22mazoria	kalehiywot church office
Guadguada		Dawit Zewudu		Guadguada	gebeya dar
U- Shape		U- Shape		U- Shape	U- Shape
L	R	L	R	R	L
823.64	823.64	788.10	788.10	346.340	260.080
0.6	1.6	0.900	1.200	0.900	1.000
0.7	1.2	1.200	2.500	0.600	1.000
0.420	1.920	1.080	3.000	0.540	1.000
2.000	4.000	3.300	6.200	2.100	3.000
0.210	0.480	0.327	0.484	0.257	0.333
0.353	0.613	0.475	0.616	0.404	0.481
1709	1709	1709	1709	1710	1695
1698	1698	1707	1707	1708	1694
11.000	11.000	2.000	2.000	2.000	1.000
0.013	0.013	0.003	0.003	0.006	0.004
0.116	0.116	0.050	0.050	0.076	0.062
0.032	0.032	0.032	0.032	0.032	0.032
1.276	2.214	0.748	0.970	0.960	0.932
0.536	4.251	0.807	2.911	0.519	0.932
4.78671		3.718			
4.37074	2.5188	17.569	13.537	6.0111108	4.6530744

Table 2—2: AASHTO relationship between layer's drain ability and subjective drainage quality rating

s/No.	Drainage Quality	Water removed from drainage structure within
1	Excellent	2 hours
2	Good	1 day
3	Fair	7 days
4	Poor	1 month
5	Very poor	Water will not drained

(Source: Toryila TM.et.al, 2016)