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**ADDIS ABABA UNIVERSITY**

**ADDIS ABABA INSTITUTE OF TECHNOLOGY**

**SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING**

**Evaluating the Impact of Land Use Change on Urban Drainage System  
and Proposed Low Impact Development measures in Addis Ababa,  
Ethiopia**

**(Case Study of Megenagna - Bole Ring Road)**

A thesis submitted to School of Civil and Environmental Engineering of Addis  
Ababa University in partial fulfillment for the degree of Masters of Science in Civil  
Engineering Major in Hydraulic Engineering

By

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This is to certify that the thesis prepared by Wondimu Paulos, entitled: Evaluating the Impact of Land Use Change on Urban Drainage System and Proposed Low Impact Development measures in Addis Ababa, Ethiopia (Case Study of Megenagna - Bole Ring Road) and submitted in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering (Major in Hydraulic Engineering) complies with the regulations of the university and meets the accepted standards with respect to originality and quality.

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## **Abstract**

Overflow of water from existing drainage is emerging issue in many cities of Ethiopia and mainly in Addis Ababa. Unplanned growth of urban areas is affecting the urban drainage system in Addis Ababa. The drainage networks in the city do not have sufficient capacity to carry excess runoff due to rapid expansion of built in environment. The roads turning into streams can be easily observed in all parts of the city, especially in Megenagna to Bole Ring Road, during rainy seasons. The purpose of this study was to estimate peak flow rate due to expanding built environment in the study area using SWMM and propose measures that reduce excess stormwater or excess runoff by using generic LID Controls. Stormwater Management Model (SWMM 5.1) was applied to simulate the urban runoff from the Bole Gerji delineated area (about 188 hectares) located in Addis Ababa, Ethiopia in conjunction with other software's like ArcGIS, Google Earth Pro, Excel and so on to analyze the quantitative data. Model calibration and validation process was conducted by using daily observed and simulated runoff at selected outlet near Anbesa Garage. Model calibration and validation at selected outlet gives coefficient of determination ( $R^2$ ) and Nash-Sutcliffe simulation efficiency (NSE) values of 0.91 and 0.86 and 0.95 and 0.88 respectively. This result indicated good agreement between observed values simulated runoffs. Built up area coverage changes from 14% to 94% in the last 25 years, and no substantial rainfall intensity variation was observed. Runoff generated from all sub catchments changed from 4.27 m<sup>3</sup>/sec in the past (1995) to 12.94 m<sup>3</sup>/sec in the present (2019). Conduits C1, C2, C3, C4, C5, C6, C11, C12, C17, C18 and C24 were under surcharge situation and Nodes J3, J5, J17, J18 and J24 on the main road (Megenagna - Bole) were found to be flooding or overflowing with total volume of 11.4\*10<sup>6</sup>, 28.5\*10<sup>6</sup>, 7.6\*10<sup>6</sup>, 12.1\*10<sup>6</sup>, and 12.4\*10<sup>6</sup> liters respectively. Evaluation of performances of the proposed low impact development measures indicated positive results. Changing 3.6 ha paved pedestrian by permeable pavement (Scenario 1) reduced total runoff by 12% and total node flooding by 11%; and changing 1.6 ha paved pedestrian by Bio – Retention Cells (Scenario 2) reduced total runoff by 5% and total node flooding by 4%. Based on the thesis result scenario 1 was recommended. Before, the urban drainage becomes a permanent socio-economic nuisance and brings irreversible damage to the city, this study strongly recommends immediate implementation of Low Impact Development Controls (LID) that are supported by strong institutional setup, policy framework, and the public at large.

**Key words:** Land Use Change, Urban Drainage, SWMM 5.1, LID Controls

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## Abbreviations and Acronyms

**AACRA** – Addis Ababa City Road Authority

**AAiT** – Addis Ababa Institute of Technology

**AASHTO** – American Association of State Highway and Transportation Official

**A2** – Rainfall Region Classification of Ethiopian Road Authority

**Arc-GIS** – Architectural Geographical Information System

**BMP** – Best Management Practice

**Cumecs** – Cubic meter per second

**DEM** – Digital Elevation Model

**ERA** – Ethiopian Road Authority

**ERDAS** – Earth Resources Data Analysis System

**FDRE** – Federal Democratic Republic of Ethiopia

**FHWA** – Federal Highway Administration

**HSG** – Hydrological Soil Group

**IDFC** – Intensity Duration Frequency Curve

**LID** – Low Impact Development

**LU / LC** – Land Use / Land Cover

**R<sup>2</sup>** – Nash and Sutcliffe Coefficient

**SCEE** – School of Civil and Environmental Engineering

**SUDS** – Sustainable Urban Drainage System

AAiT - SCEE

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**SWMM** – Storm Water Management Manual

**US EPA** – United State Environmental Protection Agency

## CHAPTER ONE

### 1. INTRODUCTION

#### 1.1 Background

Urban storm water drainage facilities are part of the urban infrastructure elements and design of these facilities require due attention. In Ethiopian context, where watersheds of many urban centers receive significant amount of annual rainfall and where rainfall intensity is generally high, control of runoff at source, flood protection, and safe disposal of runoff through proper drainage facilities becomes essential. (AASHTO, 1991). Urban conditions in developing countries exacerbate drainage challenges; runoff is increased by increasing impermeable urban surfaces and, due to inadequate development control mechanisms and their incompetent enforcement, settlements are constructed with little consideration for storm water drainage (Parkinson, 2002).

Cities are undergoing deep transformations throughout sub-Saharan Africa. Their populations are growing rapidly and their physical boundaries are expanding. Socioeconomic shifts –including the rise of an urban middle class – are leading to new patterns of production and consumption. In the last decades the high rate of urbanization has resulted in a large increase in impervious coverage in the landscape. Impervious surfaces decrease rainfall infiltration in the soil increasing runoff both in terms of peak flow and volume. Rain water in the urban landscape is therefore mainly directed into the municipal storm sewer system, creating serious problems in case of heavy rains, and reducing water availability and quality.

In Ethiopian context, where watersheds of many urban centers receive significant amount of annual rainfall and where rainfall intensity is produced high, Control of runoff at the source, flood protection, and safe disposal of the excess water/runoff through proper drainage facilities becomes significant (FUPI, 2008). Storm water drain networks in cities are usually designed to effectively collect and convey excess surface runoff in order to avert urban flooding (Gouri and Srinivas, 2015). But, often most of them face reduction of functionality and capacity for transferring the runoff flow, and their level of service reduces due to degradation in time, improper maintenance, inappropriate design, aging, sedimentation and siltation, increase in materials' roughness, and structural deterioration. In addition, urban development and climate change exacerbate the situation (Barreto, 2012). Because, such phenomena are followed by increase in runoff volume and peak flow rates. This means that even when

there is a drainage system with acceptable functionality, the design capacity of the system are in adequate for extreme events and flood occurrence (Barreto, 2012).

The foundation and expansion of Addis Ababa has been associated with the rapid conversion of rural land to urban area. For the last one hundred twenty three years it has been noticed that an intensive conversion of rural land to urban development like buildings, transportation networks and facilities (airports and highways), recreation areas, reservoirs and other manmade structures, where most of them are impermeable structures (Belete, 2009). Currently Addis Ababa has undergoing upgrading of existing roads and constructing of new roads this will highly influence the runoff volume of the city. So mitigation measures should be considered in accommodation of incoming surface runoff (Habtamu, 2011).

Bole Sub-City is a major economic nexus of Addis Ababa City. It is also the most economically significant part of the city. Unfortunately, drainage problems include street flooding, water logging, blockage of drainage facilities, deterioration of roads etc. are common in this area. This is particularly severe in areas where road infrastructure appears to be without adequate storm water drainage infrastructure. Beforehand, there were few researches conducted in line to the topic what this research has conducted to address the above mentioned challenges. In general this particular study is intended to find out the sustainable measure to minimize the runoff occurrence in particular area by evaluating the existing drainage system by known Storm Water Management Model called SWMM, and proposes the Low Impact Development (LID) controls for sustainable solution to handle the recurring flood.

## **1.2 Statement of the problem**

In the last decades, an intense process of urbanization took place, a current worldwide phenomenon leading to significant environmental impacts in urban areas. These impacts are related to quality and quantity variables of the hydrologic cycle. Urbanization along with its impermeable structures and improper design are the major causes of flooding in most developing urban areas including Addis Ababa, this urbanization affects the performance of the drainage lines. The urban area exploitation has increased to a degree where the resources and facilities may not be able to sustain the growing population, the environment and established development systems may collapse and lead to serious environmental challenges.

Some of drainage infrastructures in Addis Ababa are not properly designed, located and maintained and have therefore lost their designed efficiency. Also, the inadequate integration between the urban road and storm water drainage and rapidly springing up of new settlement around the city can be attributed to natural causes such as increasing trends in intensity and duration of rainfall, long sloping topography

with poor soil infiltration and engineering inadequacies. Some of the drainage facilities have outlived their service lives.

During rainy season in Addis Ababa, the storm water is over topping drainage system and it hinders the movement of people for hours and entraps the transportation systems (vehicles and trains). Moreover, it ruptured some dwellers' houses and vast flooding plain through town. Therefore, flooding in the city affects day to day movement of the people and the entire economy of the city. Nowadays, one of the major problem of the city is storm water-related flooding and accumulation of stagnant water, which is related the paucity of proper infrastructure and poorly developed storm water management systems. Thus, there is the need to re-examine the existing drainage system and runoff control mechanisms under the current conditions to address drainage challenges in the rapidly expanding city. Engineers thus have no choice but to consider urban flooding problem to their practice in order to adapt and serve the public interest. This thesis aims to study the Impact of land use change in urban runoff quantity and focused on the potential solutions to handle the excess storm water by implementing Low Impact Development Controls and to make the drainage system sustainable.

### **1.3 Objective of the study**

#### **1.3.1 General objective**

The general objective of this study is to estimate peak flow rate due to expanding built in environment in Bole Gerji area using SWMM; and Propose LID Controls to reduce excess storm water or runoff.

#### **1.3.2 Specific objectives**

In order to achieve the general objective of this study, the following specific objectives are set for milestones of the study:

- a) To develop and explore any changes in the past and recent IDF curve,
- b) To quantify the land use change over the past several years of the selected catchment based on at least two separate maps of the past and present,
- c) To estimate total runoff based on IDF and land use change as combine impact using SWMM
- d) Provide sustainable measures to minimize the recurring floods in Addis Ababa city using generic LID Controls.

## 1.4 Research Questions

This research is intended to answer the following research questions in line with the topic and objectives of this research:

- a) How does developed IDF Curve related to IDF Curve developed by ERA?
- b) How does land use change related to urban storm runoff?
- c) How much is the total runoff of existing urban storm water drainage system?
- d) Does the existing drainage system performs well?
- e) Does implement generic LID Controls minimize runoff to specific study area?

## 1.5 Significance of the study

Quantifying the impact of land use change and following sustainable storm water management is a relatively new area of study and will benefit by more research and application in different locations, taking into consideration the variability of soil permeability, slopes, and depth to bedrock. The implementation of storm water LIDs not only reduces runoff volume of storm water and improves the quality of the water, but reduces the urban heat, creates habitat in the city, and improves upon the aesthetics of the city land scape.

This study, generally has the following major significances:

- i. It will increase the service life of urban infrastructures such as roads, buildings, telephone lines, water supply lines and the existing rivers,
- ii. The sub-cities will use it as a reference while they are preparing their annual plans for urban drainage system.
- iii. It will be an alternative means of ensuring sustainable development in Addis Ababa City by strengthening the environmental and socioeconomic activities regarding to urban drainage system.
- iv. Policy makers and any organization working in the area of urban drainage system can use it as a further reference to fill the existing gap between drainage demand and supply.
- v. It will introduce the LID Controls option, which is not the current practice in the country's management practice in dealing with the challenges and problems of urban drainage system.

## 1.6 Scope and limitation of the study

Evaluating the whole catchment is not necessary to come up with solution for the current storm water problem. Therefore, some representative major flood prone areas were selected. According to the residents these areas have been flooded most of the rainy season and based on field observation Megenagna to Bole were selected as flood prone area. Therefore, this study is geographically limited to Megenagna Gerji area of Bole sub city of Addis Ababa. This thesis includes the modeling of the drainage system using SWMM5.1, calibrate and validate the model for the study area through observed data. The delineated area divided into 12 hydrological similar sub catchments and in these twelve sub catchments the runoff from each sub-catchments were modeled and flood routing was done. Also the drainage networks were simulated by considering each sub catchments. The software needs primary data with high quality to minimize the errors within the data, but to collect the primary data there was a time and financial limitation. The other limitation was problem of secondary data from AACRA, there was no design document founded rather than as built and delineation of the town using DEM 30 \*30m is impossible because it indicated by dote rather than polygon. Thus, delineation of the specific study area was done by Google Earth Pro.

## 1.7 Thesis layout

This thesis is sub divided in to five major chapters and it has a detail information of each sub chapter.

**Chapter-One:** Deals about the statement of the problem, research question, general and specific objectives of this study. This chapter also provides information on its background about the problem of flood in urban environment.

**Chapter-Two:** Briefly reviews the land use change it effects in urban drainage, the historical development and basic concept of urban storm water drainage, concept of sustainable urban drainage and urban drainage design related previous studies in Ethiopia specifically in Addis Ababa.

**Chapter-Three:** Outlines the research materials and methodology employed in this study, and an overview of different urban drainage models including SWMM 5.1 and its application in selected areas in Bole Sub – City.

**Evaluating the Impact of Land Use Change on Urban Drainage System and Proposed Low Impact Development measures in Addis Ababa, Ethiopia: Case Study of Megegnagna - Bole Ring Road**

**Chapter-Four:** Widely deals the SWMM 5.1 simulation results and discussion and Urban flood reduction methods are assessed and appropriate techniques that are remedy measures to the flood problem of the study area are specified here.

**Chapter-Five:** Summarizes the entire study by outlining the main conclusions and recommendations.

## CHAPTER TWO

### 2. LITERATURE RIEVIEW

This chapter discusses concepts and theories which are the back bone of the analysis part of this research work. It starts with describing general review on land use change, Impact land use change on urban hydrology and the historical development and basic concept of urban storm water drainage and then followed by: history of urban storm water drainage problem, Urban storm water drainage experience in Ethiopia and major problems, history and policy of urban storm water drainage in Ethiopia, urban storm water drainage in Addis Ababa, environmental and solid waste management in Addis Ababa, finally best management practices and best management selection tools will be discussed.

Generally, it presents various concepts and theories which have been found out by various Researchers/Authors in different periods of time in relation to this research work.

#### 2.1 General Review on Land Use Change

##### 2.1.1 Land Use and Land Cover Change

Land cover data documents how much of a region is covered by forests, wetlands, impervious surfaces, agriculture, and other land and water types. Water types include wetlands or open water. Land use shows how people use the landscape – whether for development, conservation, or mixed uses. The different types of land cover can be managed or used quite differently. Land use can be defined as how land is utilized. For example, residential and industrial land use would be considered one type of developed land use. Land cover is slightly different. A park could be forest, in this land use is a park and land cover is a forest.

Hydrological effects of land use change manifested in many ways and at different spatial and temporal scales. Most obvious is the immediate and direct effects on the quantity and quality of catchment's runoff. For example, land cover change is the most significant factor driving hydrologic changes such as runoff volume, timing and variability (Fohrer, 2001). The simplest method to assess these effects on hydrologic response of catchment is by comparing stream flow and runoff generated from the catchment areas with the contrasting land use types (Barkhordari, 2003). The main causes for land use change are due to manmade and natural causes, where the manmade causes are mainly attributed to the reach for resources to meet human needs. For instance, deforestation is a result of the need for timber for

construction, fuel wood and clearing for agricultural development and for settling the ever increasing population.

### **2.1.2 Land Use Change in Addis Ababa**

In Ethiopia, land is used for agricultural purposes, for construction of buildings and roads and extra purposes. In the country most of the land is used for subsistence farming. With the population growth and slow technological adoption which can increase production, there is deforestation for more production which means conversion of forest to agricultural land and expansion of urban settlements. As per Ethiopia Land use and Land Cover Data Addis Ababa and its neighboring shows different from the current land cover and use due to different activity on urbanization and industrialization. Land use of the City and Its Watershed Area is highly covered With Residential and Commercial building with some uncovered Area of Vegetation. To accommodate the ever-increasing population, industry concentration, and commercial expansion, Addis Ababa city has been expanding horizontally towards its peri-urban areas. Abdissa (2005), Melesse (2005), and Gete (2007) have documented the effect of urbanization in terms of forest and soil degradation, water pollution, and overall decline in agricultural production, agricultural community displacement, and squatter settlement.

### **2.1.3 Land Use change Impacts on Hydrology**

Water on earth exists in a space called the hydrosphere and lithosphere, circulates and forming hydrologic cycle. The cycle has no beginning and no ending and can be affected by different factors. Among those factors, manmade activities, land use and land cover change can affect hydrological processes such as infiltration, runoff and groundwater recharge. Different studies indicate that land use and land cover change have an impact on hydrologic components. For instance, (Adamu, 2013) concluded that land use and land cover changes have major impacts on hydrological processes, such as runoff and ground water flow, (Melesse, 2012) concluded that the decrease of forest land and grass land was accompanied by the increase in agricultural and built up areas and this change in land use and land cover increased surface run off during wet seasons and reduced base flow during the dry seasons. (Gebrie, 2016) Concluded that the land use and land cover change have a great influence on stream flow especially during wet season than dry season. Cultivation of land exerts a major influence on the relationship between surface and subsurface flow. According to data from long term observations done in paired catchments.

### 2.1.4 Effects of Urbanization in Flooding Events

The evolution of the land use is much related with urban development and the increment of floods derived from it. In the undeveloped areas the water coming from precipitation infiltrate in the soil filling the holes between particles until the storage capacity (saturation) is fulfill. After that, the runoff generation starts on the surface. However, within urbanized areas the paved and other impervious surfaces hinders the capacity of the soil to absorb water. Consequently, the velocity of the runoff is increased leading to sharp peak discharges and greater amount of water in the surface (EPA, 2003). Figure 2.2 show the influence of urbanization in the runoff generation. As can be seen, the water cycle balance is modified since the groundwater table level decreases and the runoff is raised instead But not only the quantity of water is modified, water quality is also affected. Organic and nutrient pollutant concentration in the water increases considerably if the water flows above the surface. As a result, the increasing pollutant concentration decreases the possible water uses (Choi, 2004).

Urbanization is the inevitable trend of the development of the world today. It has been estimated that about 64% of developing countries and 86% of developed countries will be urbanized by 2050 (UN, 2015). In the process of urbanization, large amounts of agricultural or other non-urban land are transformed into impervious land and the land use change totally alters natural hydrological processes. Several studies investigated the effects of urbanization-induced land use changes on runoff. The rapid expansion of urban impervious area increased surface runoff yield amount, peak discharge, reduced runoff\_ response time, and changed hydrological regimes. It also changed the long-term groundwater recharge and water balance. Quantitative assessment of the impact of urbanization on surface runoff is essential for urban planning, water resources management, and for early flood warning in big cities.

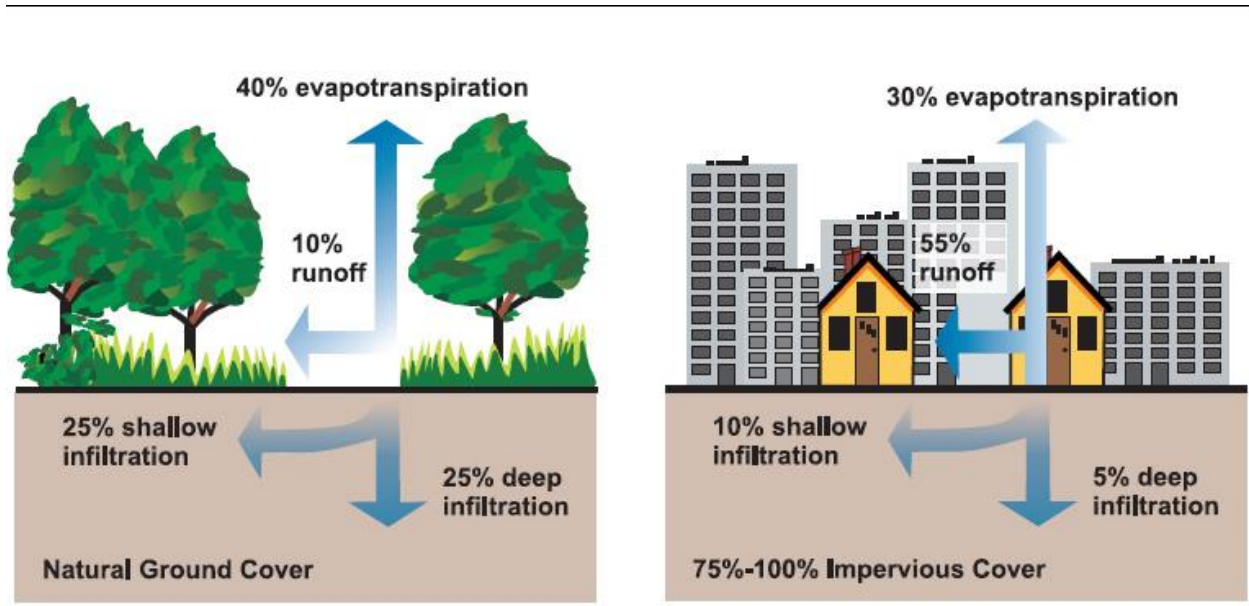


Figure 2. 1: Relation between Urbanization and Hydrological Cycle (US EPA, 2003)

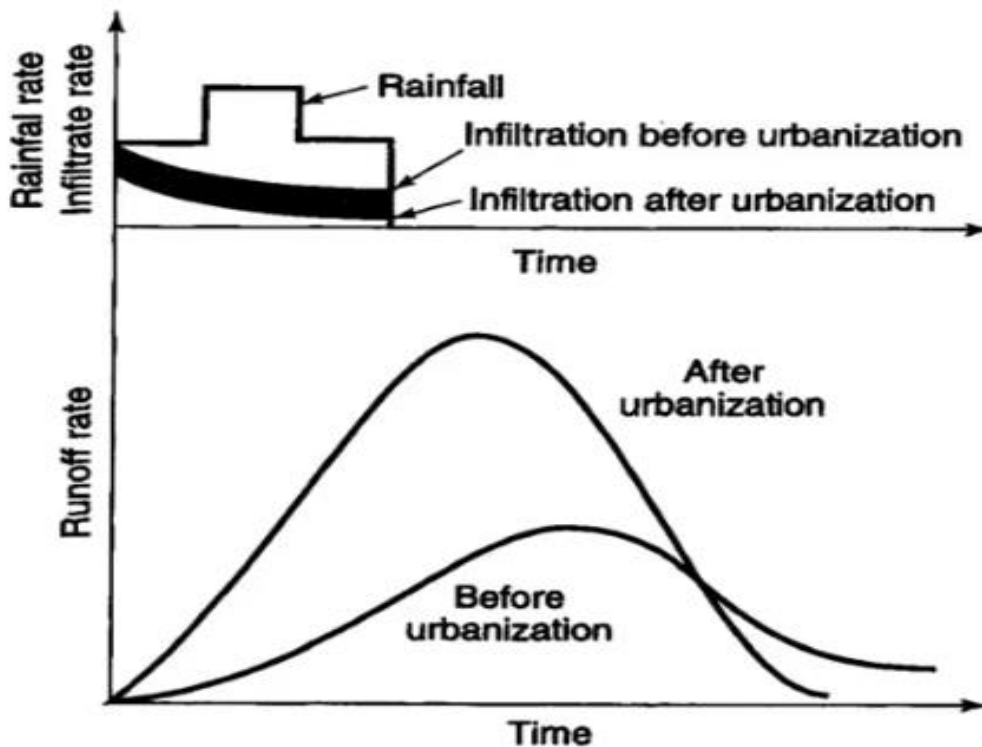


Figure 2. 2: Effect of Urbanization on Surface Runoff (US EPA, 2003)

## 2.2 General Concepts on Urban Drainage

### 2.2.1 Definition of Storm Water

Storm water is the part of precipitation that accumulates on earth's surface, ditch of road side, in culvert, on pavement and generally in drainage system. Also, it is pure rain water plus anything the rain carries along with it's a way. During rain storms, water that falls onto impervious surfaces flows to the nearest storm drain or local water body. This water can come from events other than a rain storm, such as a snow melt or street wash water, all of which are defined as storm water (Jared, 2011). Throughout this research, storm water runoff, will be synonymous with non-point source pollution. Storm water runoff problems are nothing new to local land-use decision-makers. However, the principal concern about runoff has always been safety, with the focus on directing and draining water off of paved surfaces as quickly and efficiently as possible (Jared, 2011).

In urban areas, rain that falls on the roof of our house, or collects on paved areas like driveways, roads and footpaths is carried away through a system of pipes that separated or mixed from sewerage system. Unlike sewage, storm water is not treated. If storm water was not drained properly, it would cause inconvenience, damage, flooding and further health risks (David and John, 2000).

### 2.2.2 Historical Perspective of Urban Drainage

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

### **2.2.3 Development of Modern Urban Drainage Practices**

The beginning of modern urban drainage practices was initiated in European cities during the nineteenth century. One critical turning point in urban drainage occurred during the middle of the nineteenth century. During the first half of the nineteenth century sanitary wastes were discharged from buildings to privy vaults and cesspools. Most sewers were designed exclusively for storm water drainage. Sanitary wastes accumulated in privy vaults and cesspools and were periodically collected by scavengers and transported to a suitable disposal location (e.g., farm, dump outside city). As the nineteenth century progressed the concept of urban drainage changed with the incorporation of water-carriage sanitary waste collection into the urban drainage systems. Sanitary connections to the sewers were made legal and new sewers were constructed to drain storm water and sanitary wastewater (Reynolds, 1946).

The public perspective of urban drainage changed during the nineteenth century from a neglected afterthought to a vital public works system. The public also shifted their stance regarding funding the construction and maintenance of sewer systems. The shift in public perspective was driven by many factors, but the most important was probably the scientific evidence accumulated during the second half of the century linking sanitary wastes and disease transmission (Scullard, 1967). The perspective of urban drainage also changed from a design standpoint during the nineteenth century. Most sewers constructed before the nineteenth century were not planned or designed by an engineer using numerical calculations. Instead a trial-and-error process was executed, which in some cases eventually produced well-functioning systems (Reynolds, 1946).

### **2.2.4 Urban Storm Water Drainage Practices in Ethiopia**

In Ethiopian context, where watersheds of many urban centers receive significant amount of annual rainfall and where rainfall intensity is generally high, control of runoff at the source, flood protection, and safe disposal of the excess water/runoff through proper drainage facilities become essential (NUPI, 2000). Drainage problems in Ethiopian urban centers include flooding, deterioration of roads, land degradation, sedimentation, water logging, blockage of drainage facilities and the like.

With urbanization, impermeability increases with the increase in impervious surfaces (i.e. residential houses, commercial buildings, paved roads, parking lots, etc.), drainage pattern changes, overland flow gets faster, flooding and environmental problems such as land degradation increases. It is a crucial problem facing the existing and future environmental conditions of urban centers. (FUPI, 2008). After its inception, Federal Urban Planning Institute has been involving in planning and design of urban storm

water drainage facilities as part of the Master/Development Plan of a city/town with the objective of keeping the life of urban infrastructure and to protect the urban environment like water pollution from non-point sources of storm water, Air pollution from stagnated water and Soil from erosion and degradation.

Before the establishment of the National Urban planning institute some twenty year ago, there has been no formal working organization in the area of urban storm water drainage system. Even now a day the attention towards urban storm water system is at its immature stage that is why most of the urban storm water drainage structures gets blocked with solid waste of various types after huge amount of money has been invested on them. In some areas they by themselves are sources of environmental problems (FUPI, 2008).

The Federal urban planning institute under the Ministry of Works and urban development has been trying to put a considerable effort in controlling run-off, which is produced as a result of urban structural pavements and external sources, like flooding from Entoto and Yeka mountains in Addis Ababa (Belete, 2009).

The technologies in handling the environmental problems of urban storm water drainage in Ethiopia, which have been practiced, are not in a position to utilize the flood/runoff for various uses, like the treatment/sedimentation of runoff water, construction of detention ponds and other perforated structures for the water to be infiltrated in to the soil, rather the primary aim of urban storm water drainage system in the country is to safely discharge the storm/run-off out of the urban centers( Belete, 2009).

### **2.2.5 Urban Storm Water Drainage Practice in Addis Ababa**

A major trouble of Sub-Sahara African roads are a poor drainage system, which allows storm water to sip through newly tarred surfaces and prematurely to riddle them with potholes. Addis Ababa, the capital city of Ethiopia is one of the places in the country that has been damaged by the rainy season. Faced with a problem of this magnitude, municipal authorities have been doing their level best and stepped up timely and effective road maintenance. The main challenge in this regard remains Addis Ababa's poorly developed drainage system. Only 615 kilometers, or only about 29 percent of the city's road mileage, are equipped with drainage lines, with non-asphalted roads the main victims (Uli, 2008). According to research published in 2009, of the city's 395 kilometers of asphalted roads only 193 kilometers had storm drainage lines, and out of 960 kilometers of no asphalted roads only about 143 kilometers had drainage

channels. More often than not, unlined channels are to be found in areas where ground profiles are steep, which exposes those areas to erosion through high velocities of flow.

No up-to-date data is available regarding to the proportion of housing units that are connected to drainage lines. According to various studies, this was the case for only about 33 percent of them in 1996. A community-based infrastructure upgrading program has done a lot to improve drainage in the city during the last 15 years. However, the difference made by the program is reduce the small size of the problem. According to Mr. Dagnachew Belete's study of Addis Ababa drainage lines, the existing drainages have lower capacity to convey the runoff generated. Apparently the drainage system of Addis Ababa is sadly underdeveloped by any standard.

## **2.3 Urban Storm Water Drainage Problems**

The practice of urban drainage in developing countries encounters more serious problems than those of developed countries, because urban development occurs under more difficult socio-economic, technological and climatic conditions. Developing counties experience accelerated urbanization without adequate investment in infrastructure, and against a background of deficient public services for water treatment, collection and treatment of foul sewage, garbage collection, urban drainage, transport and health. Urban concentrations have environmental consequences in the form of urban flooding and pollution of water courses, soil and air. Settlements are established in inappropriate areas such as those originally set aside for environmental preservation and on steep hillsides and areas liable to flooding (Novatech, Lyon and GRAYE, 2001). The specific factors inhibiting modernization of urban drainage in developing countries, basically by means of infiltration and retention of storm runoff, can be grouped under the following headings: (1) concern for the environment is less familiar than concern for conventional sanitary planning ; (2) there is no effective control over urban development, whether legal or clandestine; (3) runoff from storm rainfall is highly contaminated; (4) runoff transports large quantities of sediment and garbage ; (5) climatic factors can increase risk of epidemics and construction costs; (6) there is a shortage of engineering 'know-how' concerning modern approaches to urban drainage; (7) there is a lack of interaction between the population and public administrators seeking solutions to urban drainage problems(Novatech, Lyon and GRAYE 2001).

### **2.3.1 Uncontrolled Urban Settlement**

Impermeable surfaces and the construction of drains for rapid storm-water removal are the major causes of urban floods due to traditional urban settlement, pursued without regard for the environment (Uli, 2008). Such urbanization patterns make it difficult to control urban drainage, since it not only causes or aggravates local flooding but can also create problems downstream.

The extent of impermeable cover is directly correlated with runoff coefficients and also with population density, so that an indirect method of evaluating the impact of urbanization on drainage is to relate population density with runoff coefficients. There is evidence world-wide that higher urban population density commonly results in greater storm-water generation, (Debo and Reese, 2003) but many urban planners take no account of this important effect and neglect the wider costs of their storm-water control procedures.

Modern urban drainage calls for detention and infiltration areas, contrary to the philosophy of higher population density. Many cities in developing countries have a density index which already causes critical drainage situations. Besides the problems of control in legal settlements, socio-economic problems lead to the invasion of public areas, forming slums with high population density and high rates of impermeable soil surface (Debo and Reese, 2003).

### **2.3.2 Contamination by Foul Sewage**

WHO internet data show that urban sanitation coverage for Africa, Asia, Latin America and the Caribbean, where most developing countries are concentrated, at present of the order of 80%, giving an average minimum level of raw sewage dispersed through the environment as 20%. The amount of raw or inadequately-treated sewage discharged into urban rivers is not recorded at the global level. Primary and secondary treatment levels of domestic and industrial waste do not enter the WHO and World Bank sanitation indicators, and this complicates any global evaluation of the quality of life or of environmental sustainability of urban conglomerations and their surrounding regions. The presence, real or potential, of foul sewage in the storm-sewer network makes it difficult to adopt modern urban drainage practices. The potential for pollution by foul sewage is very high and it cannot be permitted to enter areas set aside for retaining storm runoff or for allowing runoff to infiltrate the soil. Pollution of storm runoff by dirt from streets, and by substances present in the rainfall itself is also important. In general it is expected that storm runoff in developing countries has higher pollution indices in terms of organic material and coliforms.

### **2.3.3 Excess Sediment and Garbage**

Urban areas in developing countries have significant proportions of exposed soil liable to erosion and giving rise to large quantities of sediment. Building sites, whether in areas where the city is expanding or within the developed urban area, do not normally have controls for erosion prevention or for retaining sediment so that it does not reach the streets, storm drains and urban rivers. It is no exaggeration to say that 10 to 15% of urbanized area in developing countries contributes extensively to sediment production and transport. The amount of garbage entering the drainage network is reduced corresponding to a production of 0.4 to 0.8% of total garbage produced (Tucci, 2000). For developing countries, the rate of garbage accumulation in the streets is certainly higher, since in some parts of the cities the storm-drain network is used for garbage disposal. With these high sediment and garbage loads, no modern solution to urban drainage is viable without special retention structures upstream or rigorous maintenance procedures with dredging or mechanical removal of the large volumes carried after every storm (Tucci, 2000). This is a peculiar feature of developing countries which makes control works for storm runoff control even more expensive to implement.

### **2.3.4 Lack of Appropriate Technology**

For the environmental approach to be successful, a change of technical culture is required through training (capacity building at all levels, for district engineers and urban planners) and environmental education for the people (Jones, D.E., Jr. 1967). Academic institutions can play a big role to take on the task of spreading information in repeated seminars and technical-scientific meetings who work in the field of storm-water drainage to increase their knowledge regarding to the subject matter. As Jones D. E. said the trust that develops in such meetings between researchers and technicians opens up communication channels leading to collaboration between municipality and university in technical support services for modernizing urban drainage practice.

### **2.3.5 Absence of Community Participation**

Lack of community participation in the search for enduring solutions for urban drainage problems is one of the main obstacles preventing the success of modern storm runoff control measures, whether by structural or non-structural measures (Novatech, Lyon, and GRAYE 2001). In most developing countries this has been a problem for sustainable storm water drainage management. Lack of community participation leads to the repetition of earlier errors in solving drainage problems, to the discredit of

public action, and lack of concern with environmental questions (ibid). It can also bring about low investment in urban facilities.

## **2.4 Sustainable Urban Drainage System**

### **2.4.1 The concept of Sustainable Urban Drainage System**

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

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

The idea behind SUDS is to try to replicate natural systems that use cost effective solutions with low environmental impact to drain away dirty and surface water run-off through collection, storage, and cleaning before allowing it to be released slowly back into the environment, such as into water courses. This is to counter the effects of conventional drainage systems that often allow for flooding, pollution of the environment – with the resultant harm to wildlife – and contamination of groundwater sources used to provide drinking water. The paradigm of SUDS solutions should be that of a system that is easy to manage, requiring little or no energy input (except from environmental sources such as sunlight, etc.), resilient to use, and being environmentally as well as aesthetically attractive. Examples of this type of system are basins (shallow landscape depressions that are dry most of the time when it's not raining), rain-gardens (shallow landscape depressions with shrub or herbaceous planting), swales (shallow

normally-dry, wide-based ditches), filter drains (gravel filled trench drain), bio retention basins (shallow depressions with gravel and/or sand filtration layers beneath the growing medium), reed beds and other wetland habitats that collect, store, and filter dirty water along with providing a habitat for wildlife (Sharma, D., et al, 2008)

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

### 2.4.2 How Does Sustainable Urban Drainage System, SUDS Work?

Sustainable drainage systems use a sequence of techniques that together form a management train. As surface water flows through the system, flow velocity is controlled and pollutants are removed. The management train may include the following stages:

**A) Source Control:** This method decreases the volume of water entering the drainage/river network by intercepting run-off water on roofs for subsequent re-use (e.g. for irrigation) or for storage and subsequent evapotranspiration (e.g. green roofs).

**B) Pretreatment:** steps, such as vegetated swales (ditches) or filter trenches, remove pollutants from surface water prior to discharge to watercourses or aquifers.

**C) Retention:** systems delay the discharge of surface water to watercourses by providing storage within ponds, retention basins and wetlands.

**D) Infiltration:** systems, such as infiltration trenches and soak ways mimic natural recharge, allowing water to soak into the ground.

### 2.4.3 Generic Type of LID Controls Defined in SWMM

The Low Impact Development (LID) approach has been recommended as an alternative to traditional storm water design. Such techniques are also called Water Sensitive Urban Design (WSUD) in Australia (Lloyd, 2001) and Sustainable Urban Drainage Systems (SUDS) in the United Kingdom (CIRIA, 2000).

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A range of factors to be considered when selecting SUDS such that the most appropriate technique is adopted and SUDS are successfully implemented. Such factors include site suitability, available land space, cost, maintenance issues and community acceptance.

SWMM allows engineers and planners to represent combinations of green infrastructure practices to determine their effectiveness in managing runoff. Although some of these practices can also provide significant pollutant reduction benefits, at this time, SWMM only models the reduction in runoff mass load resulting from the reduction in runoff flow volume.

LID Controls are low impact development practices designed to capture surface runoff and provide some combination of detention, infiltration, and evapotranspiration to it. They are considered as properties of a given sub catchment. SWMM can explicitly model eight different generic types of LID controls:



**Rain Gardens.** A depressed area in the landscape, planted with grasses, flowers, and other plants, that collects rain water from a roof, driveway, or street and allows it to infiltrate into the ground. Rain gardens can also help filter out pollutants in runoff and provide food and shelter for butterflies, song birds and other wildlife. More complex rain gardens with drainage systems and amended soils are often referred to as bio

retention cells.

**Figure 2. 3: Rain Garden (SWMM Manual, 2015)**



**Bio retention Cells (or Bio swales).** Depressions that contain vegetation grown in an engineered soil mixture placed above a gravel drainage bed which slow, infiltrate, and filter runoff. They provide storage, infiltration, and evaporation of both direct rainfall and runoff captured from surrounding areas. As

linear features, bio retention cells are particularly well suited to being placed along streets and parking lots.

**Figure 2. 4: Bio Retention Cells (SWMM Manual, 2015)**



**Vegetative Swales.** Channels or depressed areas with sloping sides covered with grass and other vegetation. They slow down the conveyance of collected runoff and allow it more time to infiltrate the native soil beneath it.

**Figure 2. 5: Vegetative Swales (SWMM Manual, 2015)**



**Infiltration Trenches.** Narrow ditches filled with gravel that intercept runoff from upslope impervious areas. They provide storage volume and additional time for captured runoff to infiltrate the native soil below.

**Figure 2. 6: Infiltration Trenches (SWMM Manual, 2015)**



**Green Roofs.** Green roof is a system of planted vegetation and lightweight soil media used on rooftop areas. A variation of a bio retention cell, green roofs have a soil layer laying atop a special drainage mat material that conveys excess percolated rainfall off of the roof. They contain vegetation that enable rainfall infiltration and evapotranspiration of stored water. Green roofs are particularly cost-

effective in dense urban areas where land values are high and on large industrial or office buildings where storm water management costs are likely to be high.

**Figure 2. 7: Green Roofs (SWMM Manual, 2015)**



**Rooftop Disconnection.** This practice allows rooftop rainwater to discharge to pervious landscaped areas and lawns instead of directly into storm drains. You can use it to store storm water and/or allow storm water to infiltrate into the soil. Downspout disconnection could be especially beneficial to cities with combined sewer systems. SWMM

can also model roofs with directly connected drains that overflow onto pervious areas.

**Figure 2.8: Rooftop Disconnection (SWMM Manual, 2015)**



### Rain Barrels or Cisterns (Rainwater Harvesting).

Containers that collect roof runoff during storm events and can either release or re-use the rainwater during dry periods. Cisterns may be located above or below ground and have a greater storage capacity than a rain barrel.

**Figure 2.9: Rain Barrels (SWMM Manual, 2015)**



### Continuous Permeable Pavement Systems.

Excavated areas filled with gravel that are paved over with a porous concrete or asphalt mix. Normally, rainfall will immediately pass through the pavement into the gravel storage layer below where it can infiltrate at natural rates into the site's native soil. Block Paver systems consist of impervious paver blocks placed on a sand or pea gravel bed with a gravel

storage layer below. Rainfall is captured in the open spaces between the blocks and conveyed to the storage zone and native soil below.

**Figure 2.10: Continuous Permeable Pavement system (SWMM Manual, 2015)**

## 2.5 Urban Storm Water Drainage Models

Combined sewers were constructed in many cities of the United States before 1900 without recognizing the need for segregation and treatment of domestic and industrial wastes from storm runoff (Hall, 1984). Although these systems still exist in older municipalities in the U.S., separate sewers have dominated the construction during the 20th century. Separate systems for storm water drainage and sewerage are almost universal in Australia. The main purpose of urban drainage systems is to collect storm water and convey it to receiving waters, with minimal nuisance, danger or damage, at least in the conventional drainage systems. However, in recent times emphasis has been shifted from disposal of storm water to total management of storm water, considering storm water as a resource (CEPA, 1993). In addition to collection and disposal of storm water, several other objectives are considered in total management of storm water. These objectives include: limiting pollutants entering receiving waters through water quality control measures such as wetlands; minimizing other adverse impacts of urbanization (e.g. erosion and sedimentation); water conservation in semi-arid and arid areas; integration of large-scale drainage works into overall town planning schemes with multipurpose land-use (such as drainage, recreation or transportation), and reuse of storm water. The design methods for urban drainage systems include a wide range from rule-of-thumb methods to computer models. The Statistical Rational method has been commonly used in Australia for computing flows for urban drainage design. However, there is an increased tendency in recent times to use computer models to analyze complex drainage systems (CEPA, 1993).

These models generally consider the major hydrological and hydraulic processes of urban drainage systems such as interception, infiltration (from pervious surfaces), depression storage, overland flow, gutter flow and pipe flow. These computer models can be used for both storm event modeling and continuous simulation. Storm event modeling which considers the generation of flood hydrographs due to a storm is important in urban drainage design. The continuous modeling, which deals with modeling of the drainage system over a long period, is important in estimating storm water yield, which can be reused (CEPA, 1993).

### 2.5.1 Storm Water Management Model, EPA SWMM 5.1

SWMM5 (U.S. Environmental Protection Agency, 1992) is a comprehensive computer model for simulation of urban runoff quantity and quality in storm and combined sewer systems. SWMM 5 stands for Storm Water Management Model. All aspects of the urban hydrologic and quality cycles are

simulated, including surface runoff, transport through the drainage network, storage and treatment. Like most hydrologic models, SWMM5 subdivides the overall catchment into sub catchments, predicting runoff from Sub catchments on the basis of their individual properties, and combining their outflows using a flow routing scheme. SWMM5 can also simulate backwater effects. In SWMM5, sub catchments are represented mathematically as spatially lumped, nonlinear reservoirs, and their outflows are routed via the channel/pipe. Sub catchments are subdivided into three subareas, impervious area with and without depression storage, and pervious areas with depression storage. Flow from one subarea is not routed over another subarea. Overland flow is generated from each of the three subareas by approximating them as nonlinear reservoirs. This nonlinear reservoir is established by combining the continuity equation with Manning's equation. Infiltration from pervious areas can be computed by either Horton or Green-Ampt equation. Flow routing in channel/pipes is also performed through a nonlinear reservoir by combining the continuity equation with Manning's equation. (EPA, 1992).

### **2.5.2 Model for Urban Sewers, MOUSE**

MOUSE (Danish Hydraulic Institute, 1988) stands for Modeling of Urban Sewers and is a hydrologic-hydraulic model applicable only for modeling of urban catchments. This model is used extensively for sewerage design in Australia compared to the design of storm water drainage networks (Lindberg and Car, 1992). The hydrologic part of the model deals with simulation of runoff using two methods: a simple method based on time-area diagram and a complex method based on kinematic wave theory and continuity equation. The hydraulic part of the model simulates flow routing in closed conduits or open channels. Three options are available in MOUSE to compute depth and velocity of flow. The first is the kinematic wave method, which is mostly applied to part full flow conditions. The second is the diffusive wave method, which considers backwater and surcharge in the systems. The last is the dynamic wave method, which provides a full hydrodynamic solution. MOUSE, like SWMM, is well-suited for analyzing the hydraulic performance of complex looped sewer systems including overflows, storage basins and pumping stations. Water quality modeling and prediction is also included in the MOUSE model. (Lindberg and Car, 1992).

### **2.5.3 Civil Computer Aided Design, Civil CAD-Pipes**

CIVILCAD (Surveying and Engineering Software, 1997) is a multipurpose design computer package. It was mainly a design tool for road design, although it provides facilities for drainage design. However,

this package is rarely used only for drainage design by city/shire councils in Victoria (personal communication with R. Silva, Buloke Shire Council, Victoria, 1999).

The drainage module of CIVILCAD performs the following basic functions:

- Perform hydrological calculations to calculate surface runoff, gutter flow and pipe flow.
- Design the pipes interactively to obtain the optimum combination of diameters, slopes and depths of pipes.
- Perform backwater analysis to ensure satisfactory hydraulic performance.
- Produce reports of calculations including tables and figures (both hydrographs and longitudinal sections).

#### **2.5.4 Runoff Analysis and Flow Training Simulation, RAFTS**

The RAFTS (WP Software, 1991) model has been used in Australia since 1980s. RAFTS stands for Runoff Analysis and Flow Training Simulation. RAFTS simulate runoff hydrographs at defined points throughout the catchment for specific rainfall events (both observed and design). RAFTS are suitable for modeling of catchments ranging from rural to fully urbanize. The model is capable of analyzing catchments comprising natural waterways, formalized channels, pipes, retarding and retention basins, and any combination of these. There are no specific limitations on the catchment size. It has been successfully used for on-site detention and on catchments up to 20,000 km<sup>2</sup> (WP Software, 1991).

RAFTS can be used in event or continuous mode, with appropriate rainfall inputs. Like most rainfall-runoff models, RAFTS requires the catchment to be sub-divided into several sub catchments. Each sub catchment is then divided into 10 subareas within RAFTS based on lines of equal travel time or isochrones. Runoff from each subarea is routed using the Laurenson's (1964) runoff routing procedure to obtain the outflow hydrograph of a Sub catchment. RAFTS can model pervious and impervious areas separately. However, it does not consider directly connected impervious area and supplementary area separately as in ILSAX and SWMM. RAFTS use initial loss-continuing loss model or Philip's infiltration equation to simulate the excess runoff. Pipe flow is determined using Manning's equation. Overflow is computed as the portion of the total sub catchment inflow, which cannot flow through the pipe because of inadequate capacity. Pit inlet capacity restriction is not considered in this model. For flood routing through pipes and trunk drainage system, the Muskingum procedure is used. As an alternative to channel routing where physical data is lacking, RAFTS allows a simple channel lagging procedure whereby the flood hydrograph is simply lagged by an appropriate time with zero attenuation. Lag times are calculated

in RAFTS using flow velocity computed from the Manning's equation. Puls' level pool routing procedure is used in the retarding and retention basins.

### **2.5.5 Watershed Bounded Network Model, WBNM**

The WBNM (Boyd *et al.*, 2000) model is an event based nonlinear runoff routing model, capable of modeling runoff from small and large catchments. In WBNM, a catchment is divided into a number of sub catchments and is represented by a separate storage element. Each urbanized sub catchment is divided into pervious and impervious subareas, with separate rainfall losses to compute the rainfall excess. Five alternative loss models (i.e. initial loss-constant loss rate, initial loss-loss rates varying in steps, initial loss-runoff proportion, Horton continually varying loss rate and Green-Ampt varying loss) are available in WBNM to model rainfall losses. Overland flow in each sub catchment is modeled by a nonlinear reservoir with time-lag. Three options available for channel routing are:

- a) Nonlinear routing using a "channel factor" selected to reflect the increased flow velocities in the "improved" channel.
- b) Muskingum routing, with its parameters selected based on the translation and attenuation properties of the reach.
- c) Time-lag method, in which the upstream hydrograph is delayed through the reach by a specified time (but without attenuation) to produce the downstream hydrograph.

### **2.5.6 Storm Water Runoff Model, STORM**

The U.S. Army Corps of Engineers (1977) developed Storm Water Runoff Model (STORM) to analyze quantity and quality of runoff from urban and nonurban catchments. STORM was primarily developed to evaluate the storm water storage and treatment capacity required to reduce untreated overflows below specified values. Computations of treatment, storage and overflow proceed in an hourly basis by simple runoff volume and pollutant mass balance for the entire catchment. Since this model runs on hourly time step, this model is not suitable for small catchments where time of concentration is less than one hour. STORM is a continuous simulation model. This model is basically a planning model and therefore, not suitable for detailed quantity or quality modeling. Runoff can be determined in one of three ways. They are the runoff coefficient method, the U.S. Soil Conservation Service (SCS) curve number technique and a method which combines the above two.

## 2.6 Related Studies on Urban Drainage system issues in Addis Ababa

In order to establish the fact that existence of drainage system problem in the city and town to understand the works that are done, literatures and different manuals should be reviewed. The literatures and manuals showed no doubt on the existence of drainage system problem in the city. The presentation of the problems in the literature are presented either in the form of malfunctioning of specific component of the urban drainage system due to unavoidable distribution of wasted material and missed design of drainage system related to their carrying capacity of peak discharge. As related studies, the following are major studies that are carried out on drainage system problem in Addis Ababa city. All studies were performed by post graduate students in Addis Ababa University. The first thesis is titled, “Study of the Urban Drainage System in Addis Ababa, Yeka Sub city” (Dagnachew, 2009). The second thesis is titled, “Investigation on Storm Drainage Problem of Addis Ababa - Case Study at Gotera – Wollo Sefer, Saris - Gotera and Ring Road” (Desalegn, 2011) in Addis Ababa city. The third thesis is titled, “Sustainable Storm water management by implementing low impact development in Jemo, Addis Ababa (Kidist, 2019).

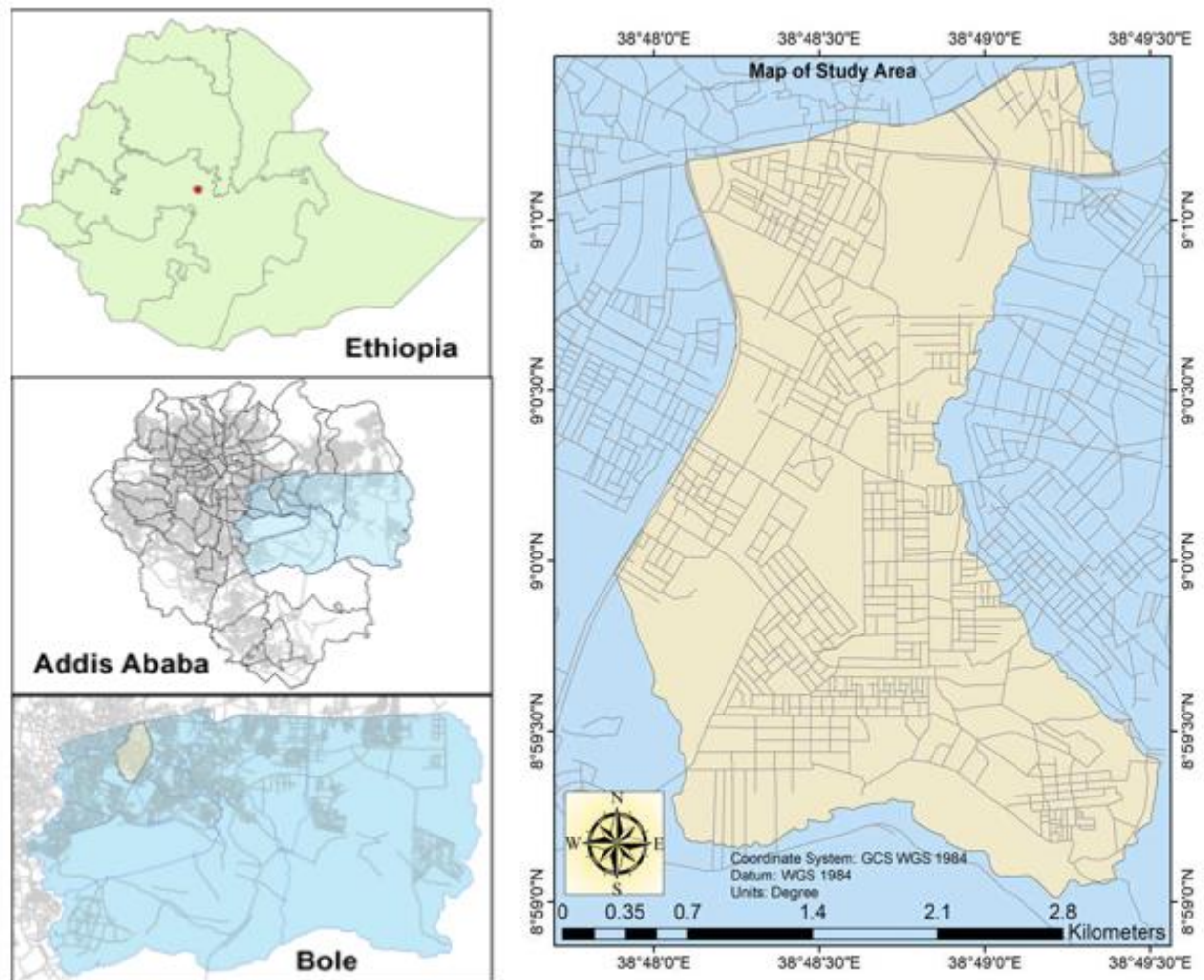
## CHAPTER THREE

### 3. MATERIALS AND METHODS

#### 3.1 General description of the study area

Addis Ababa is located in the central highlands of Ethiopia. Geographically, it is located at  $9^{\circ} 38' 0''$  N between  $38^{\circ} 42' 0''$  E, having an average minimum temperature of  $5^{\circ}\text{C}$ , maximum temperature  $27^{\circ}\text{C}$ , and average annual rainfall of 1188.27 millimeter. Addis Ababa is constituted as a City Government. The city covers a total area of 540 square kilometers (54, 000 ha) (CSA, 2019). Out of Ethiopia's estimated urban population of nearly 13.6 million, about 35.29 per cent of the total lives in Addis Ababa, of which 2,203,200 (45.9%) are male and 2,596,800 (54.1%) female. The average number of persons per household is 4.6, and the total number of households in the city is 671,210. The rate of growth of the population of the city is estimated to be 4.4% (CSA, 2019). The City has gained international status by being the seat of the African Union, several international organizations and numerous embassies. Addis Ababa is located in the central part Ethiopia and belongs to the western highlands. The metropolis located in the highlands is also close to the Rift Valley. Addis Ababa is bordered by all sides by the Regional State of Oromia. Addis Ababa is divided into ten Sub-Cities stemming from the 2003 reforms onwards and every sub-city has its own administrative autonomy.

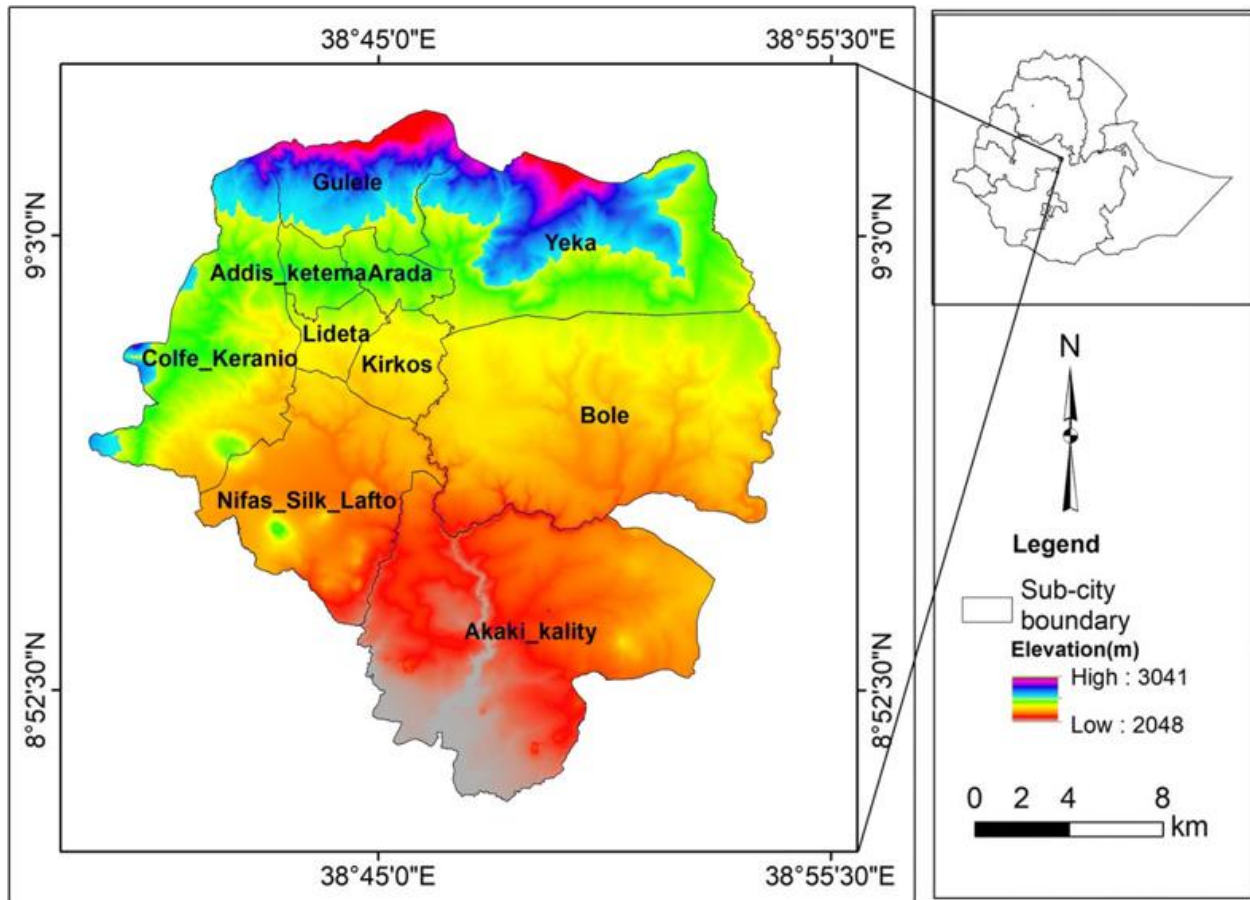
The study area, Bole Sub-city, is one of ten districts in Addis Ababa. It is situated in the southeastern part of Addis with a total Area  $122.08 \text{ km}^2$  and is accessible to all parts of the city, Addis Ababa (Bole Sub City eco-city project). It is relatively well developed and planned. Now, Bole Sub-City is organized into 14 Woreda's. According to the population unit of the Finance and Economy Development Bureau, the population of Bole Sub-City is estimated to be 328,900 in year 2019/20 G.C (2012 E. C).



**Figure 3. 1: Location map of Study area (ArcGIS)**

### **3.1.1 Topography**

Bole sub city district is characterized by a highly flat relief with small hills, see Figure below. The general slope is about 0.9%, dipping from the Northern or upper catchment part towards kality catchment. The highest elevation in Bole sub city is 3500 meters, while the lowest point in the study area is around 2100 meters (DEM). The steeper areas are the south edge and the western part, which consist mainly on small valleys bordering the urbanized area. With the elevation of 2326m above sea level at Bole International Airport, in the southern periphery, and the highest over 3700 m at Yeka Mountains, north of the city. This means, an average elevation of 2408 meters above sea level.



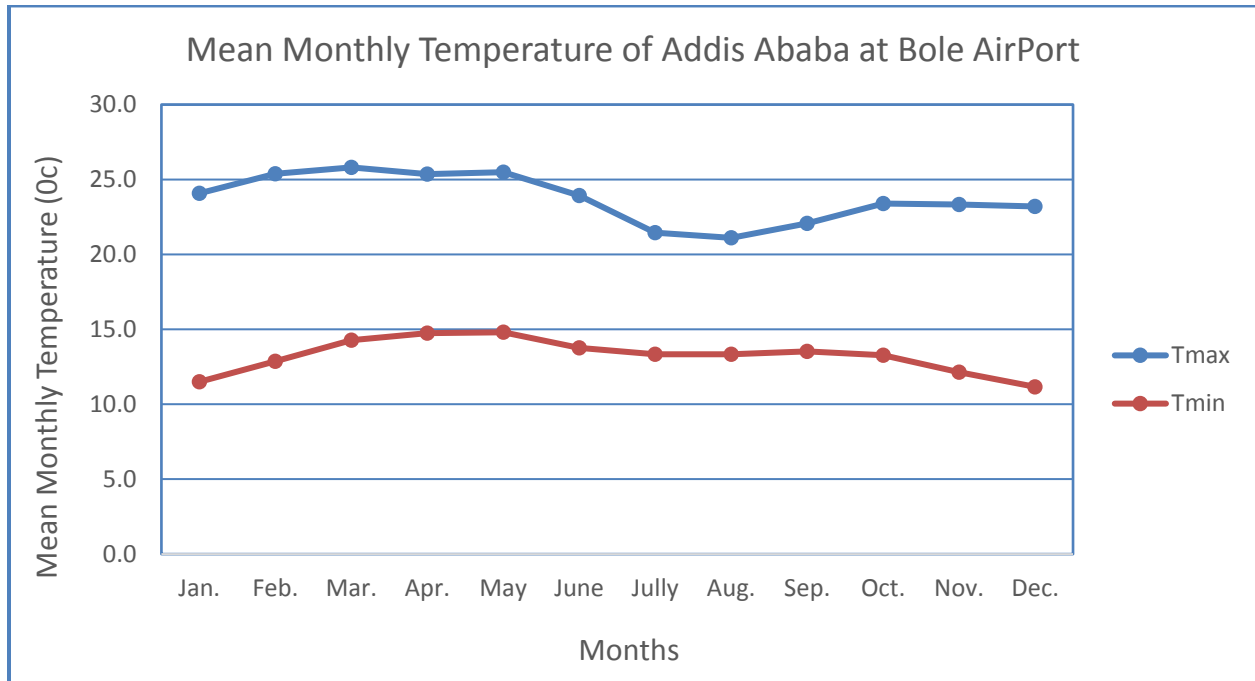
**Figure 3. 2: Analysed digital elevation model (DEM) of Addis Ababa (ArcGIS)**

### 3.1.2 Population

The population of Addis Ababa is currently about 4,800,000 according to the 2019 population projection with annual growth rate of 4.4% (CSA: 2019). The same report shows that 45.9% of the City’s populations are males and the rest 54.1% are females. Bole - Sub city has an area of 122.08 km<sup>2</sup> with 328, 900 Population size and density of 2695 persons per km<sup>2</sup>.

### 3.1.3 Climate

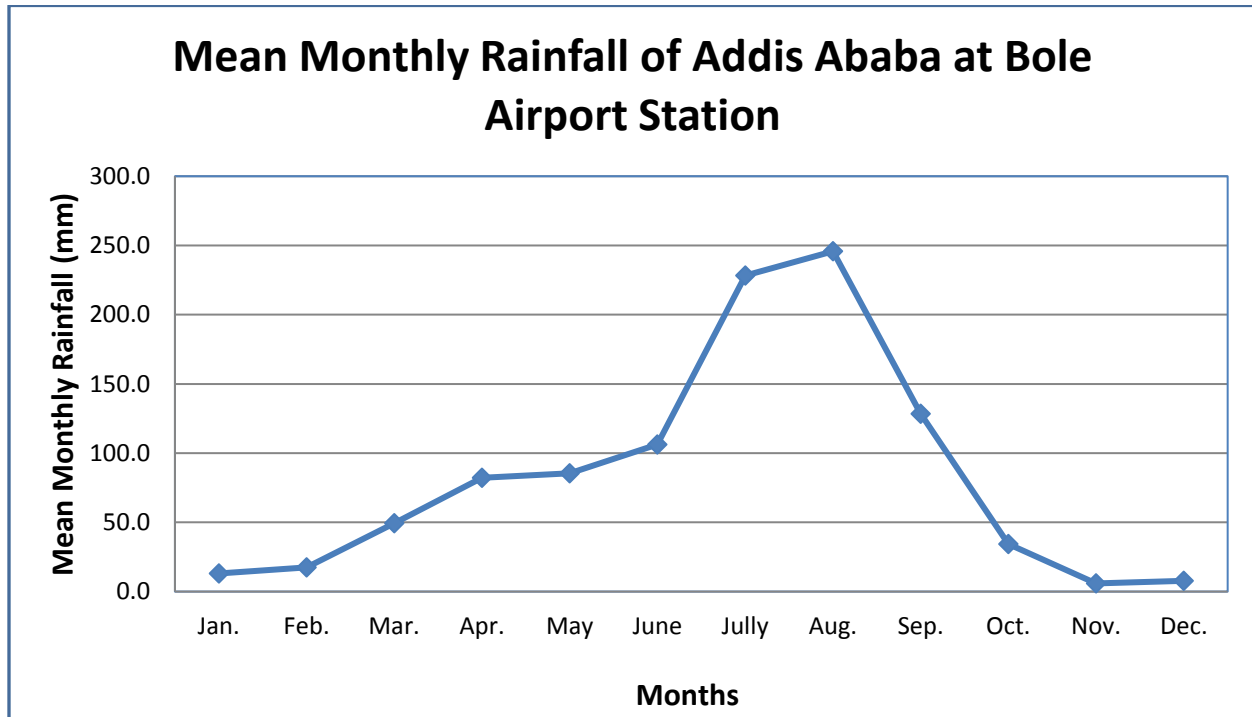
Addis Ababa city is positioned near equator having very constant temperature from month to month. As part of the Ethiopia climate and based on the national atlas of Ethiopia (1981); the city of Addis Ababa in general and that of Bole Sub city falls within Dega and Weina Dega climatic zone.



**Figure 3. 3: Mean Monthly Temperature of Addis Ababa at Bole Airport Station**

### 3.1.4 Rainfall

The seasonality of the rainfall in Addis Ababa is generally governed by the migration of the Inter Tropical Convergence Zone (ITCZ). The forward and the backward migration of the ITCZ produce substantial rainfall bringing moisture from the south Atlantic and Indian Oceans. Addis Ababa receives most of its rainfall (75 %) from June to September. Typical monthly rainfall distribution for Addis Ababa at Bole Airport station is given in Figure 3.4



**Figure 3. 4: Mean Monthly Rainfall of Addis Ababa at Bole Airport Station**

### 3.1.5 Geology

The study area is located in Addis Ababa city, which is found on the central highlands of the Ethiopian plateau. The Addis Ababa area is covered by different types of volcanic rocks. The urban area is composed of younger basalts called Addis Ababa basalts which are also covered with volcanic topsoil materials (Dirk Muschalla, 2001). The western part of Addis Ababa belongs to the younger age stratum; the northern part is mainly composed of Trachey basalts. In the Bole area, a kind of basalt, called ignimbrites, is partly found. The topsoil materials in the western part are thick and soft compared to those of the northern and eastern parts (ibid).

### 3.1.6 Land Use and Soil

Addis Ababa has experienced rapid physical expansion in east, west, south and little in north direction. The city had different land use maps which were developed at different time in the history of the city. The modeled area contains all types of urban area: commercial, administration, business area, asphalt road, green area and also these modeled areas contain areas that are not developed.

The development of soils depends primarily on geologic and climatic conditions. In Ethiopia, 17 major soil units have been identified (EMA, 1988). The FAO Soil Map of Ethiopia classifies 19 soil units, which do not all coincide spatially with the EMA soil map. Since the FAO classification system is recent, the FAO classification (FAO, 1998) is selected. The major types of soil at the study area (Bole Sub-city) were Eutric Vertisols (Hydrological Soil Group - D) and Humic Nitisols (Hydrologic Soil Group - B) according to typical hydrologic soils groups for Ethiopia, it is hydrologic soil group D.

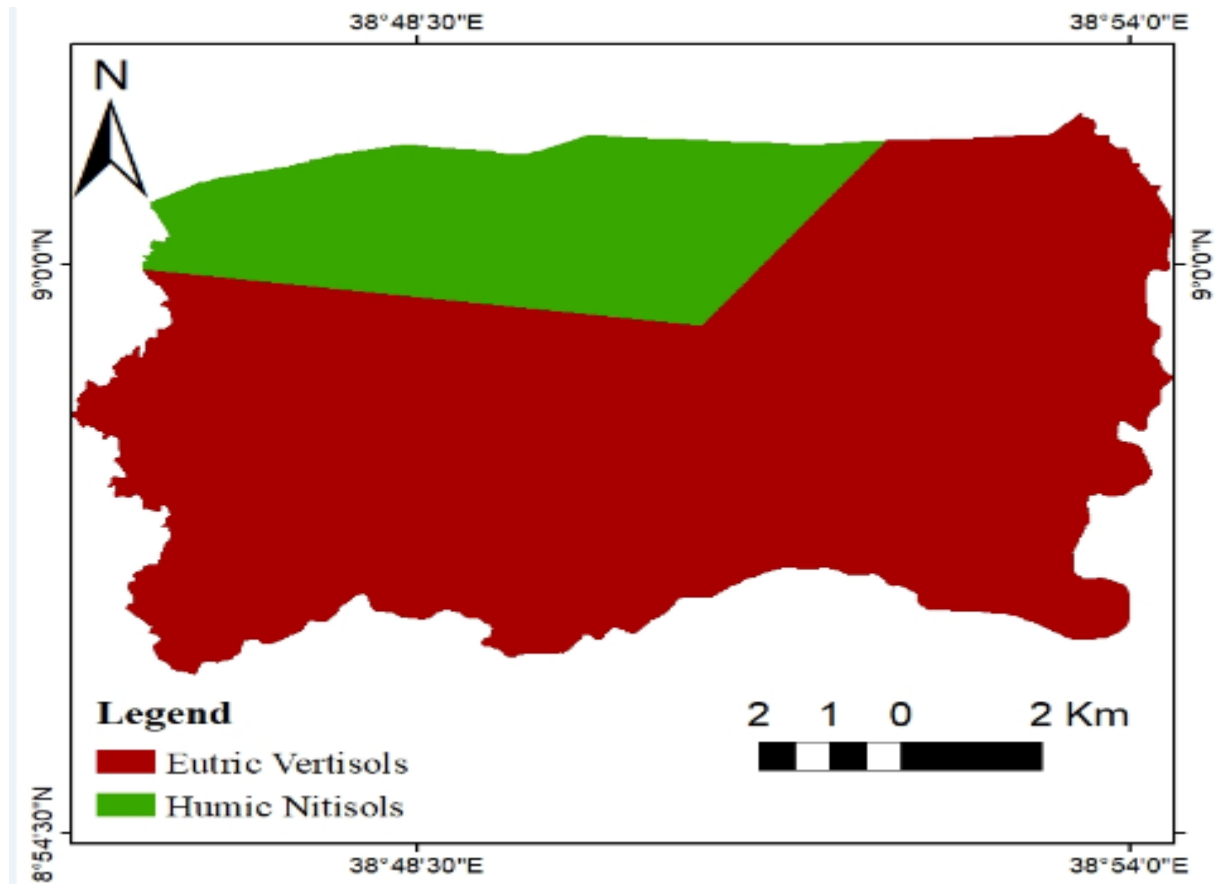


Figure 3. 5: Soil type of Bole Sub City (ArcGIS)

### 3.2 Materials

The following materials have been used to conduct this research:

- **ARC-GIS** to obtain hydrological and physical parameters and spatial information of the catchments of the study area.
- **DEM data** is used as an input data for ARC-GIS software for catchment delineation and estimation of catchment characteristic.

- **Google Earth Pro Software** to verify water shed and divides of catchments of the study area.
- **Storm Water Management Model** to simulate the peak runoff
- **Tape meter** to measure the existing storm water drainage lines depth, width and diameter which helps to evaluate the capacity of the drainage system.
- **ERDASI IMAGINE 2015 Software** used for Preprocessing of land sat images such as layer stacking, mosaic king and band and color combination.
- Hydraulic, Hydrological and meteorological design documents

### 3.3 Data Collection and Analysis

This part contains types and sources of data which were used in this study. Consequently, information and data collection were obtained via two sources which include: Primary and secondary sources.

#### 3.3.1 Data Sources

##### A. Primary sources

For this study, through site observation pictures of different drainage status were taken to show the true state of things in the study area. Observations were also made to identify the status of the drainage that properly functioning, partially blocked and fully blocked. Measuring dimension of drainage system at each sub-catchments. The dimension including depth, width and length each sub –catchment.

##### B. Secondary sources

The criterion for the selection of the Metrological data was based on the availability of data, the data quality and possibly whether the station is within the watershed or not? And if not it is within the Sub-basin or nearby. Rainfall records of different stations were collected from the National Meteorological Services Agency, (A.A Bole, Kotebe, Intoto, and A.A Observatory gauged rainfall data) that used to develop IDF Curve of the study area. Discharge measurement at the outlet near Anbesa garage, Drainage shape and land-use layout, catchment areas, Invert Elevation and dimensions, slopes and roughness parameters of drainage conduits and slope of the drainage profile were collected from Addis Ababa City Road Authority. Some site specific and sensitive parameter that used as input for SWMM5 model were collected from books, journals, manuals etc.

Table 3. 1: Summary of selected Meteorological Stations near the study area

S. No.	Station Name	Latitude	Longitude	Altitude	Year of data obtained
1	Addis Ababa Bole	9.03	38.75	2354	1997 - 2017
2	Addis Ababa Observatory	9.02	38.74	2386	1997 - 2017
3	Intoto	9.08	38.72	2903	1997 - 2017
4	Kotebe	9.06	38.84	2755	1997 - 2017

### 3.3.2 Rainfall Data Analysis

#### 3.3.2.1 Data Quality Analysis

Rainfall is an important part of the hydrological cycle. One of the first steps in any hydrological and meteorological study is accessing reliable data. However, precipitation data is frequently incomplete. The incompleteness of precipitation data may be due to damaged measuring instruments, measurement errors and geographical paucity of data (data gaps) or changes to instrumentation overtime, a change in the measurement site, a change in data collectors, the irregularity of measurement, or severe topical changes in the climate of a zone. Therefore, some correction mechanisms should be applied to the raw data. The common tests are:

##### A. Estimating missing rainfall

Due to the absence of observer or instrumental failure rainfall data record occasionally are incomplete. In the estimation of missing data from a rain gauge station, performance of a group of neighboring stations including the one with missing data are considered. A comparison of the recordings of these stations is made by using their normal rainfall as standard of comparisons. In such a case one can estimate the missing data by using the nearest station rainfall data. The collected rainfall data of Addis Ababa Bole station were checked against fill the missing data, normal ratio method were used Because of the normal annual precipitation of the index stations lies exceeds  $\pm 10\%$  of normal annual precipitation of interpolation station. According to the Normal Ratio Method the missing precipitation is obtained by:

$$P_x = \frac{1}{n} \sum_{i=0}^n \frac{N_x}{N_i} P_x \dots\dots\dots (3.1)$$

Where

**Evaluating the Impact of Land Use Change on Urban Drainage System and Proposed Low Impact Development measures in Addis Ababa, Ethiopia: Case Study of Megegnagna - Bole Ring Road**

- $P_x$  is the missing precipitation for any storm at the interpolation station 'x',
- $P_i$  is the precipitation for the same period for the same storm at the  $i$ th station of a group of index stations,
- $N_x$  the normal annual precipitation value for the 'x' station and  $N_i$  the normal annual precipitation value for  $i$ <sup>th</sup> station.

For three nearby stations in a catchment, the Normal Ratio Method becomes:

$$P_x = \frac{1}{3} \left[ \frac{N_x}{N_1} P_1 + \frac{N_x}{N_2} P_2 + \frac{N_x}{N_3} P_3 \right] \dots \dots \dots (3.2)$$

**B. Checking Precipitation data Consistency**

If the condition relevant to the recording of rain gauged stations has undergone significant change during the period of record, inconsistency would arise in rain fall data of a station. There are a lot of factors that affect the consistency of precipitation record at a given weather station. This factor includes the rain gauge has been installed at different size in the past, the rainfall depth is being recorded continuously between previous and the current site, change of exposure condition of gauge due to growth of trees or construction of tall building in the possibility of gage site.

To check a given weather station data time series observational data is relatively consistent and homogeneous and the periodic data are proportional to an appropriate simultaneous period. This station data proportionality can be tested by construction of double mass curve analysis in which accumulated rainfall/precipitation data is plotted against the mean value of all neighborhood stations. Double mass curve techniques method helped in determining the best realistic correlation of stations located near or within watershed. This technique is based on the principle that when each recorded data comes from the same parent population, they are consistent (Subramanian, 2008). If it has error, it should be corrected by using the following formula:

$$P_{cx} = P_x \left[ \frac{M_c}{M_a} \right] \dots \dots \dots (3.3)$$

Where:

- $P_{cx}$  = correct precipitation at any time  $t_1$  at station  $x$
- $P_x$  = original record precipitation at any time  $t_1$  at station  $x$
- $M_c$  = correct slope of the double mass curve

- $M_a$  = original slope of double mass curve

If the conditions relevant to the recording of a rain gauge stations have undergone a significant change during the period of record, inconsistency would arise in the rainfall data of that station. Double mass curve technique is often used to test the consistency of rainfall record. The procedure is that accumulated rainfall at the gauge station whose record is in doubt is plotted as ordinate versus the average concurrent accumulated average rainfall of nearby stations whose rainfall data are reliable. A double mass curve graph was drawn to check the consistency of the collected data. And it was observed that the data of most of the stations are consistent. Table A.8 attached in appendix part shows all the data used to plot the double mass curve below.

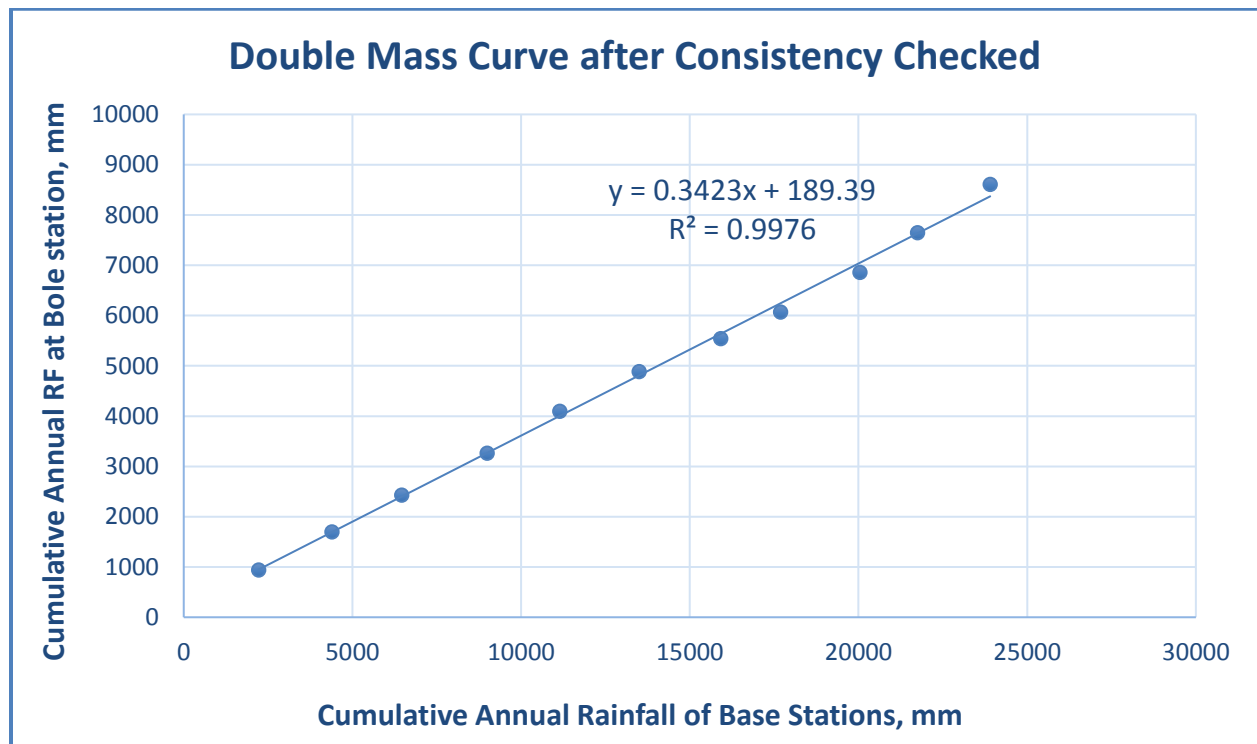


Figure 3.6: Double mass curve for selected rain gauge station

### 3.3.2.2 Frequency Analysis

Extreme rainfall events and the resulting floods can take thousands of lives and cause billions of dollars in damage. Flood plain management and design of flood control works, reservoirs, bridges, and other investigations need to reflect the likelihood or probability of such events. Hydrological studies also need to address the impact of unusually low rainfalls causing low stream flows which affects for example water quality and water supply. The term frequency analysis refers to the techniques whose objective is

to analyze the occurrence of hydrologic variable within statistical framework, by using measured data and basin predictions on statistical laws. The historical rainfall data available is a 24hr duration rainfall hence appropriate IDF reduction methods need to be used to obtain rainfall intensities of shorter duration. Any probability distribution can be used as the model but the reliability of the distribution is checked by the goodness of fit tests. Among many method, Log Pearson Type III methods are used for this research based on as suggested by Ethiopian Drainage Design Manual (ERA, 2013).

Log Pearson Type III distribution expressed as:

$$Y_T = Y_{avg} + K_T * S_y \dots \dots \dots (3.4)$$

Where:

- $Y_T$  - Log  $X_T$  – logarithm of Rainfall depth ( $X_T$ ) at return period T years [mm]
- $Y_{avg}$  - Mean value of logarithmic rainfall data (daily) [mm]
- $S_y$  - Standard deviation (mm)
- $K_T$  = Log Pearson Type III distribution frequency factor (taken from readily available table) and

Coefficient of skewness  $C_s$  is calculated for the logarithms of the data. The frequency factor depends on the return period T and the coefficient of skewness  $C_s$ . When  $C_s=0$ , the frequency factor is equal to the standard normal variable z. When  $C_s \neq 0$ ,  $K_T$  is approximated by Kite (1977) as:

$$K_T = Z + (Z^2 - 1)K + \frac{1}{3}(Z^3 - 6Z)K^2 - (Z^2 - 1)K^3 + ZK^4 + \frac{1}{3}K^5 \dots \dots (3.5)$$

Where:

$$K = \frac{C_s}{6}$$

### 3.3.2.3 IDF Curve developed for Addis Ababa

Intensity duration frequency (IDF) curves describe the relationship between rainfall intensity, rainfall duration and return period (or its inverse, probability of exceedance). Estimation of maximum rainfall depths for different return periods (T) are obtained by statistical technique of frequency analysis. Extreme value type I, Gumbel and Log Pearson Type III distributions was used for modeling storm determination of desired return periods in areas where appropriate IDF curves are not available. Thus, the analysis consists of determining maximum rainfall depths associated with T value of interest. Because of the

absence extreme rainfall values for periods less than 24 hours (12, 6, 3 and etc.) in Bole Airport station IDF curve developed to obtain the depth and intensity of 5 or 10 or 15 etc minute interval that used as input for SWMM5 model. Ethiopian Road Authority (ERA) Drainage Design Manual of 2013 suggests the following equation for calculation of shorter duration rainfall from 24hour duration rainfall.

$$\frac{R_t}{R_{24}} = \frac{t}{24} \left( \frac{(b+24)^n}{(b+t)^n} \right) \dots\dots\dots (3.6)$$

Where:

Rt: R24 -Rainfall ratio, Rt: Rainfall in a given durations (hr.), R24: Rainfall in 24 hours, n: constant, b: constant, t: time (hr.)

Based on studies of a large number of rainfall gauges in East Africa, ERA 2013 manual suggests the average values of b and n are 0.3 and 0.9 respectively. These values have been adopted for this thesis IDF development.

The methods employed to develop IDF curve for the shorter duration events using the above equations are as follows:

Rainfall intensity,  $I_t$  (mm/hr) is obtained by:  $I_t = \frac{R_{24}}{t}$  substituting in the above equation gives:

$$I_t = \frac{R_{24} (b+24)^n}{24(b+t)^n} \dots\dots\dots (3.7)$$

Where:

$I_t$  in mm/hr.,  $t$  in hr. and  $R_{24}$  in mm

### 3.4 Methodology

#### 3.4.1 General Methodology

Basically, the general methodology for the study can be described by the following flow chart.

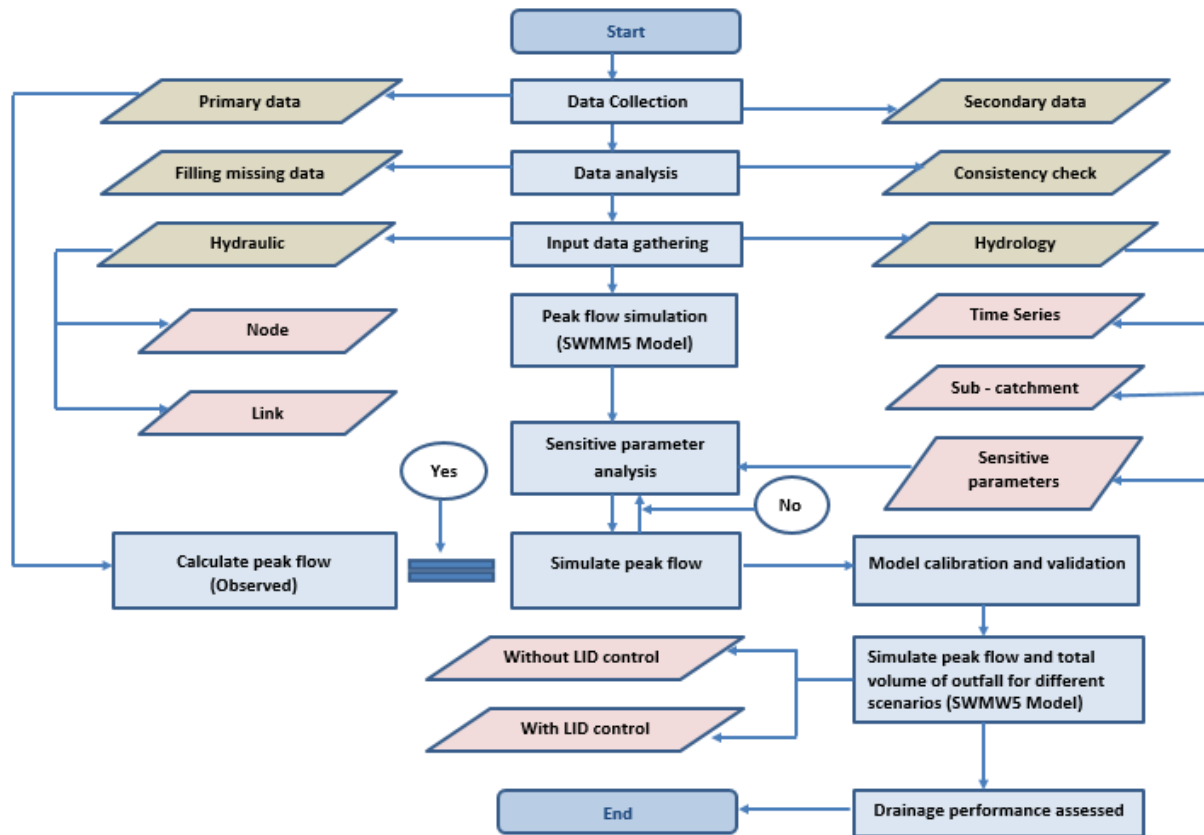


Figure 3. 7: Schematic representation of the methodologies followed in the study

### 3.4.2 Hydraulic and Hydrological modelling using EPA SWMM 5.1

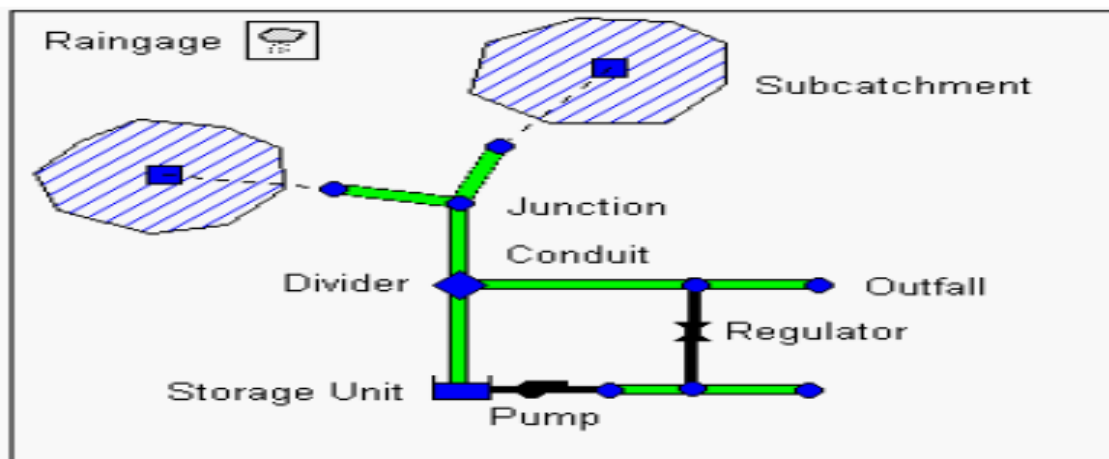
#### 3.4.2.1 Description of SWMM 5.1 model

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

SWMM conceptualizes a drainage system as a series of water and material flows between several major environmental compartments. These compartments and the SWMM objects they contain include:

- **The Atmosphere compartment**, which generates precipitation and deposits pollutants onto the land surface compartment. SWMM uses Rain Gage objects to represent rainfall inputs to the system.
- **The Land Surface compartment**, which is represented through one or more Sub catchment objects. It receives precipitation from the Atmospheric compartment in the form of rain or snow; it sends outflow in the form of infiltration to the Groundwater compartment and also as surface runoff and pollutant loadings to the Transport compartment.
- **The Groundwater compartment** receives infiltration from the Land Surface compartment and transfers a portion of this inflow to the Transport compartment. This compartment is modeled using Aquifer objects.
- **The Transport compartment** contains a network of conveyance elements (channels, pipes, pumps, and regulators) and storage/treatment units that transport water to outfalls or to treatment facilities. Inflows to this compartment can come from surface runoff, groundwater interflow, sanitary dry weather flow, or from user-defined hydrographs. The components of the Transport compartment are modeled with Node and Link objects.

Figure 3.7 below depicts how a collection of SWMM's visual objects might be arranged together to represent a storm water drainage system. These objects can be displayed on a map in the SWMM workspace.



**Figure 3. 8: Physical objects used to model a drainage system**

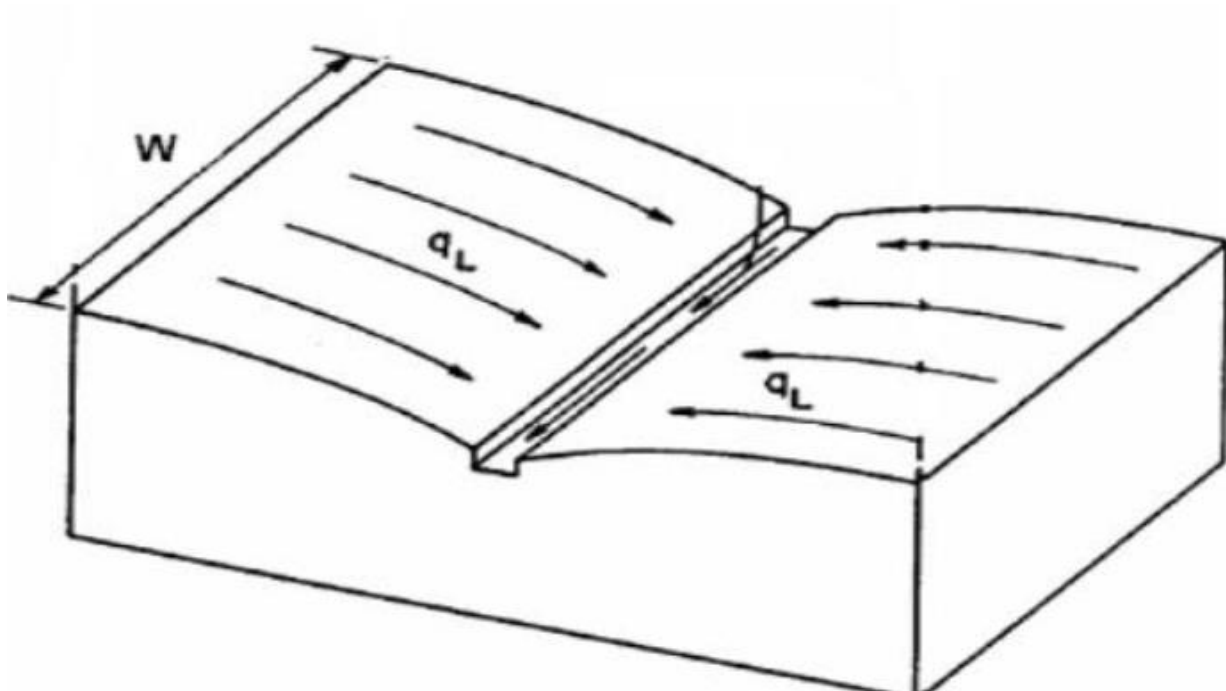
### Model set up procedure

One typically carries out the following steps when using EPA SWMM to model a study area:

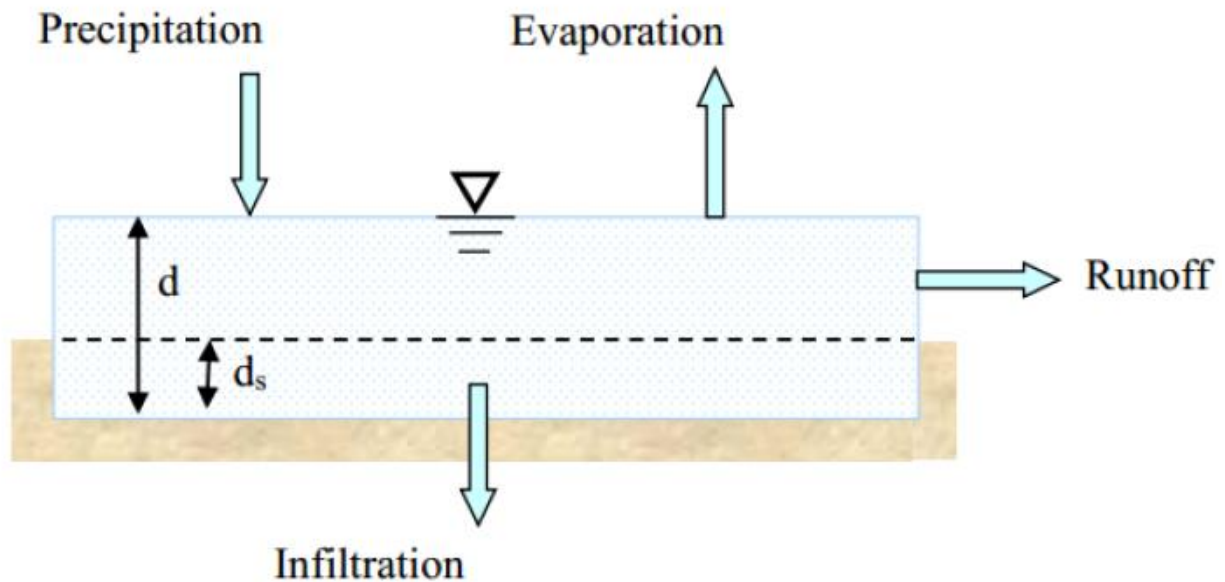
- Specify a default set of options and object properties to use
- Draw a network representation of the physical components of the study area
- Edit the properties of the objects that make up the system
- Select a set of analysis options
- Run a simulation
- View the results of the simulation

### 3.4.2.2 Governing equation of SWMM

SWMM conceptualizes a sub catchment as a rectangular surface that has a uniform slope,  $S$  and a width,  $W$  that drains to a single outlet channel as shown in Figure 3.8. Overland flow is generated by modeling the sub catchment as a nonlinear reservoir, as sketched in Figure 3.9.



**Figure 3. 9: Idealized representation of a sub catchment**



**Figure 3. 10: Non – linear reservoir model as a sub catchment**

In this representation, the sub catchment experiences inflow from precipitation (rainfall and snowmelt) and losses from evaporation and infiltration. The net excess ponds atop the sub catchment surface to a depth d. Pondered water above the depression storage depth ds can become runoff outflow q. Depression storage accounts for initial rainfall abstractions such as surface ponding, interception by flat roofs and vegetation, and surface wetting. From conservation of mass, the net change in depth d per unit of time t is simply the difference between inflow and outflow rates over the sub catchment:

$$\frac{\partial d}{\partial t} = i - e - f - q \dots \dots \dots (3.8)$$

Where:

- $\frac{\partial d}{\partial t}$  = net change in depth per unit time
- i = rate of rainfall + snowmelt (m/s)
- e = surface evaporation rate (m/s)
- f = infiltration rate (m/s)
- q = runoff rate (m/s).

Note that the fluxes i, e, f, and q are expressed as flow rates per unit area (Cumecs/m<sup>2</sup> = m/s).

Assuming that flow across the sub catchment’s surface behaves as if it were uniform flow within a rectangular channel of width W (m), height d–ds, and slope S, the Manning equation can be used to express the runoff’s volumetric flow rate Q (cumecs) as:

$$Q = \frac{1}{n} * S^{\frac{1}{2}} * R_x^{\frac{2}{3}} * A_x \dots \dots \dots (3.9)$$

Where:-

- n is a surface roughness coefficient,
- S the apparent or average slope of the sub catchment (m/m),
- Ax- the area across the sub catchment’s width through which the runoff flows (m<sup>2</sup>), and
- Rx- is the hydraulic radius associated with this area (m).

Referring to Figures 3.9 and 3.10, Ax is a rectangular area with width W and height d-ds. Because W will always be much larger than d it follows that:

$$Ax = (d - ds)W \text{ and } Rx = \frac{d - ds}{2}$$

Substituting these expressions into Equation 3.9 gives:

$$Q = \frac{1}{n} * W * S^{\frac{1}{2}} * \left(\frac{d - ds}{2}\right)^{\frac{5}{3}} \dots \dots \dots (3.10)$$

To obtain a runoff flow rate per unit of surface area (q), equation 3.10 divided by surface area of the catchment.

### 3.4.3 Peak Discharge estimation using Rational Method

In the design of storm water drainage system, the main purpose of hydrologic analysis is to determine the maximum amount of run-off (peak discharge) that can be accumulated at certain storm drainage outlet (usually a ditch) along a highway/access road alignment section. The Rational method, one of the most commonly used simplified models for road storm drainage, is primarily based on the concept that the peak discharge from a watershed will always occur when the rain lasts long enough at its maximum intensity to enable all portions of the basin to contribute to the flow. For this thesis Rational Method is appropriate because of area for each catchment is less than 50 hectare (0.5 km<sup>2</sup>). The peak runoff is given by the following expression:

$$Q = 0.00278 * C * I * A * F.S \dots \dots \dots (3.11)$$

Where:

- Q – Discharge at outlet (m<sup>3</sup>/s)
- C – Rainfall-Runoff Coefficient
- I – Maximum probable rainfall Intensity (mm/hr.)
- A– Catchment Area (hectares)
- F.S – Factor of Safety (factor of ignorance) (Optional)

**The procedures in rational method to determine peak discharge are:**

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

**Runoff coefficient, C determination**

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

$$C_w = \frac{[A_1 C_1 + A_2 C_2 + \dots + A_n C_n]}{[A_1 + A_2 + \dots + A_n]} \dots \dots \dots (3.12)$$

Where

- $C_w$  - Weighted Runoff Coefficient
- $C_1, C_2 - C_n$  = coefficient of runoff for parts of the drainage area.
- $A_1, A_2 - A_n$  = parts of drainage areas with different runoff coefficients.

**Table 3.2: Runoff coefficient value (ERA, 2013)**

Type of Drainage Area	Runoff Coefficient, C
Business: Downtown areas	0.7 – 0.95
Neighborhood area	0.5 – 0.7
Residential : Single - family	0.3 – 0.5
Multi units, detached	0.4 – 0.6
Multi units , attached	0.6 – 0.75
Suburban	0.25 – 0.4
Residential (0.5 hectares lots or more)	0.3 - 0.45
Apartment dwelling areas	0.5 – 0.7
Industrial : Light area	0.5 – 0.8
Heavy areas	0.6 – 0.9
Parks, Cemeteries	0.1 – 0.25
Playgrounds	0.2 – 0.4
Railroad yard areas	0.2 – 0.4
Unimproved areas	0.1 – 0.3

### Rainfall Intensity, I determination

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

### Time of Concentration, $T_c$ determination

The time of concentration was defined as the period required for water to travel from the most hydraulically distant point of the water contributing area to the drainage system under consideration. Use of the Rational Method requires the time of concentration ( $T_c$ ) for each design point within the catchment area. The duration of rainfall is then set equal to the time of concentration and is used to estimate the design average rainfall intensity ( $I$ ). Peak discharge generated from pavement surface and small cross streams has been determined through rational model application. For a specific drainage basin, the time of concentration consists of an overland flow time plus the time of flow in a closed conduit or open channel to the design point. Overland flow is the time required for runoff to flow over the surface to the nearest inlet and is primarily a function of the length of overland flow, the slope of the drainage basin, and surface cover. Pipe or open channel flow time can be estimated from the hydraulic properties of the conduit or channel. To determine time of concentration for over land and channel flow, there are many formulae. Among these the Kerby formulae for over land flow and Kirpich formulae for channel flow are presented respectively.

$$T_c = 0.604 \left( \frac{RL}{S^{0.5}} \right)^{0.467} \dots \dots \dots (3.11)$$

Where:

- $T_c$  - Time of concentration in hours
- $L$  - Length of overland flow in kilometers
- $S$  - Slope in m/m
- $R$  - Roughness coefficient

$$T_c = \left( \frac{0.87L^2}{1000Sav} \right)^{0.385} \dots \dots \dots (3.12)$$

Where:

- $T_c$  - Time of concentration in hours
- $L$  - Length of overland flow in kilometers
- $S$  - Slope in m/m

**Catchment area, A**

In general, the catchment area can be determined from topographic maps DEM data and field surveys. However, for large catchment areas, it is necessary to divide the area into sub-catchment areas to account for major land use changes.

### 3.4.4 Storm Water Management Model, SWMM 5.1 Setup

#### A. Watershed Delineation

The concept of a watershed is basic to all hydrologic designs. Since large watersheds are made up of many smaller watersheds, it is necessary to define the watershed in terms of a point; this point is usually the location at which the design is being made and is referred to as the watershed "outlet." With respect to the outlet, the watershed consists of all land area that fall water to the outlet during a rainstorm. Using the concept that "water runs downhill," a watershed is defined by all points enclosed within an area from which rain falling at these points will contribute water to the outlet. Therefore, watershed delineated using Google Earth Pro and saved in SWMM compatible file mode.



Figure 3. 11: Specific study area delineated using Google Earth Pro

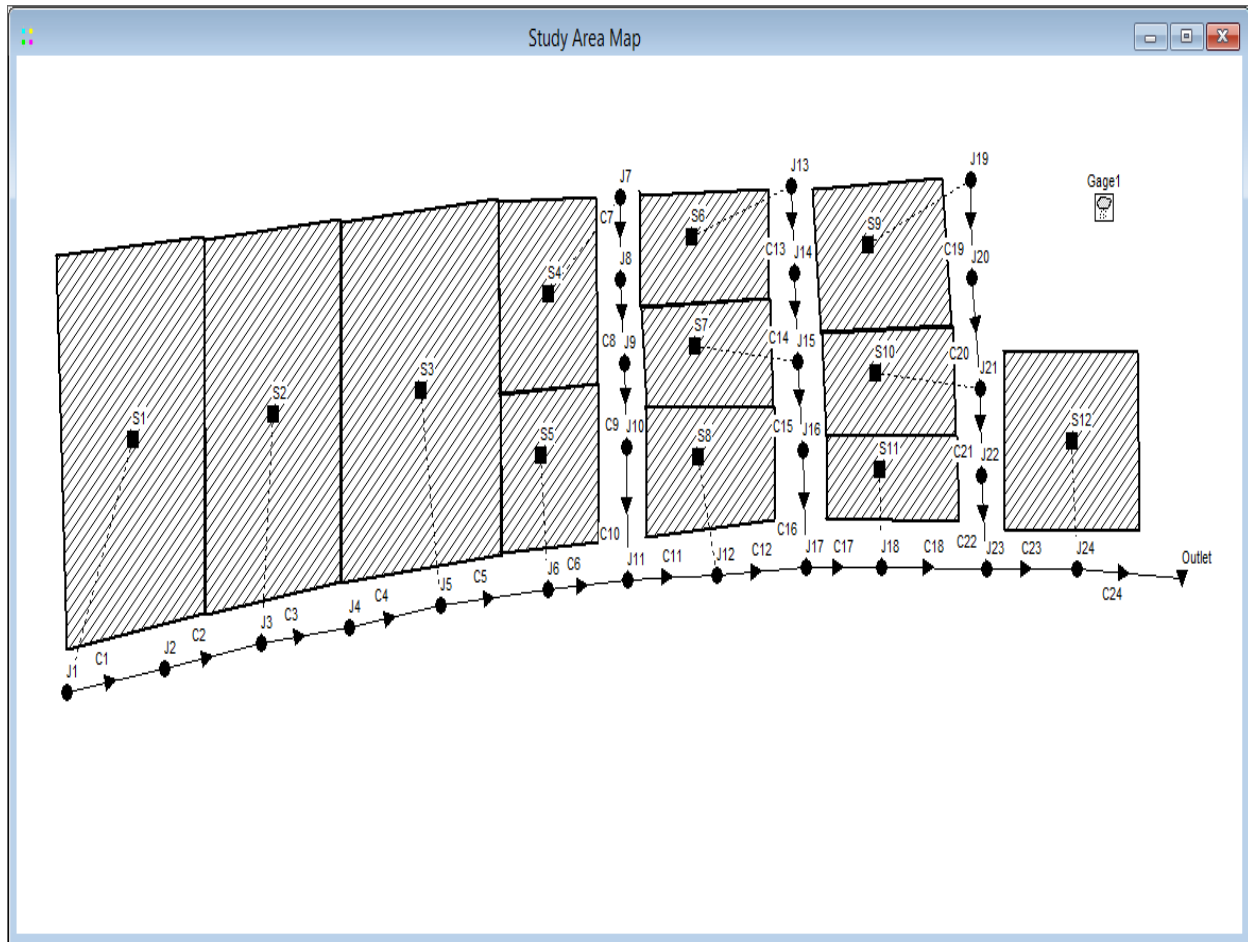
#### B. Calculation of hydrological Sub catchments area

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The total catchment area was calculated as 188 hectares using Google Earth. The elevation of the catchment is 2356.2 m and 2314.8 m at the top and bottom levels of the catchment respectively. The length between top level and bottom level of the catchment is 2780 m. The whole catchment is divided into twelve sub-catchments based on elevation and existing drainage networks. The sub-catchments are presented in Fig. 3.11 and the calculated area for those sub catchments in Table 3.2.

Table 3. 3: Area calculation of hydrological sub catchments

Serial Number	Sub catchment	Area, ha
1	Sub catchment 1, S1	34
2	Sub catchment 2, S2	34
3	Sub catchment 3, S3	34
4	Sub catchment 4, S4	8
5	Sub catchment 5, S5	7
6	Sub catchment 6, S6	8
7	Sub catchment 7, S7	8
8	Sub catchment 8, S9	14
9	Sub catchment 9, S10	12
10	Sub catchment 10, S11	10
11	Sub catchment 11, S12	6
12	Sub catchment 12, S13	13
	Sum	188



**Figure 3. 12: Delineated catchment divided in to twelve hydrological sub catchments**

### **C. Assigning nodes, conduits and outfall**

After the sub catchments are defined, the input parameters for conduits, node and junctions are entered. The input parameters for junction are Invert elevation and maximum depth; for conduits are Inlet and outlet node, Shape and size, Length, Roughness, and Inlet and outlet offset; for outfalls are Invert elevation and maximum depth. The runoff from the corresponding sub catchments are distributed to the respective nodes and finally to the outlet through a conduits. In this project, the drain systems single sides of the road have been provided. On the basis of field Drainage Network the nodes, conduits and outlets are modeled on SWMM. Nodes, Conduits and outfall input values are attached in **the Appendix Table A.1 up to Table A.4.**

#### D. Time Series Rainfall Data

Daily rainfall data obtained from the Bole International Airport station for the period 2007 to 2017 was analyzed first. IDF curves was developed for the Addis Ababa Municipality for 9 different durations (ranging from 5 to 200 minutes) and 6 different (2, 5, 10, 25, 50 and 100 yrs.) return periods. Regression of the intensity values for all the durations against the return periods will give a curve model with Easy fit statistical software. Rainfall intensities for varying durations was plotted against the return periods on normal axes using the Excel. From plotted IDF Curve, the rainfall intensity read using 10 years return period and computed different time of concentrations. Those values were assigned to rain gage and connected to the sub catchments that contribute to runoff.

### 3.4.5 Model Sensitivity Analysis

Sensitivity analysis is a technique of identifying the responsiveness of different parameter involving in the simulation of hydrological process. Sensitivity analysis was performed to choose the most sensitive flow parameters that influence the catchment represented by SWMM to be used for calibration. Sensitivity analysis, however, suggests that the simulations are relatively more sensitive to some parameters than to others. This also allows limiting excessive over parameterization of a model (Beven, 1996). The most parameters used for sensitivity analysis and their allowable range of change proposed by Li et al, 2014 below in Table 3.4.

Table 3. 4: Key parameters for sensitivity analysis (Li et al, 2014)

Sensitive Parameters for SWMM Hydrology and Hydraulic Analysis			
Name of Parameter	Meaning		Value Range
N – Imperv	Manning’s roughness coefficient for impervious area		0.011 – 0.015
N - Perv	Manning’s roughness coefficient for pervious area		0.05 – 0.8
Destore – Imperv	Depth of depression storage in impervious area		0 – 3
Destore - Perv	Depth of depression for pervious area		3 – 10
Conduit Roughness	Manning’s roughness coefficient for conduit		0.011 – 0.024
Infiltration Method	Green Ampt	Suction Head, mm	290

		Conductivity, mm/hr.	0.51
		Initial Deficit	0.25 – 0.26

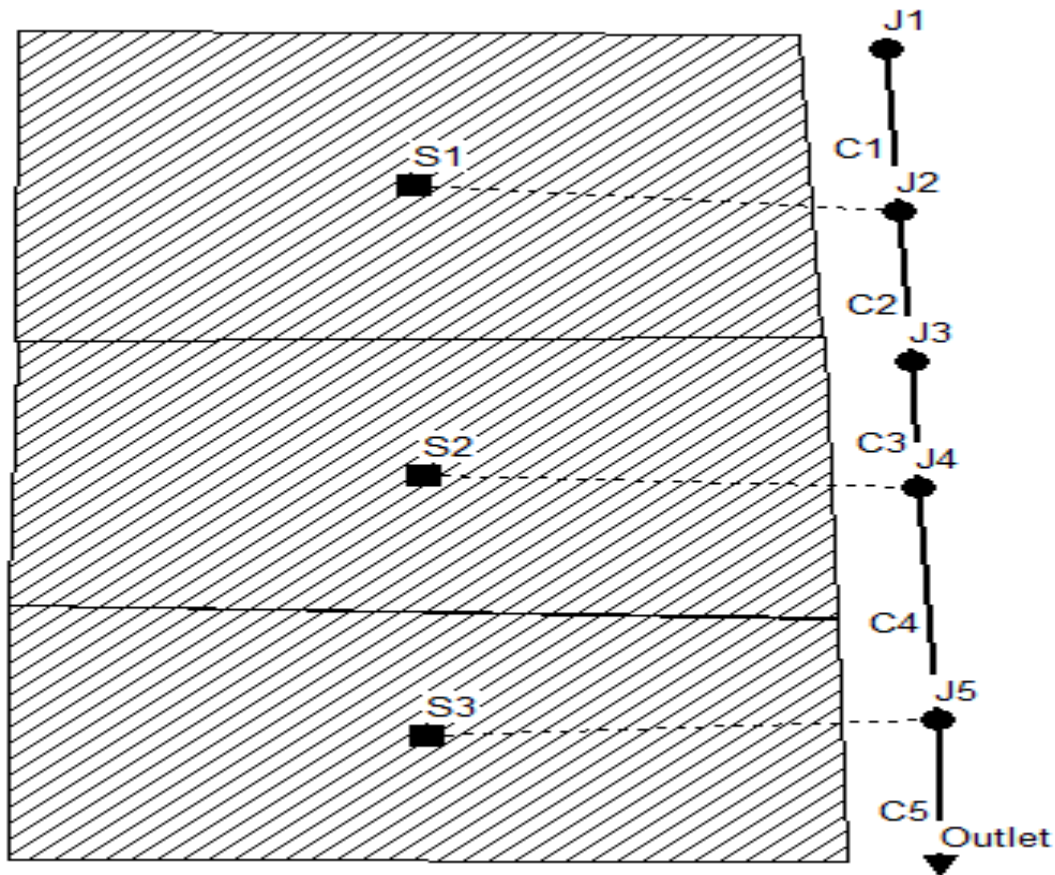
### 3.4.6 Model Calibration and Validation

An important part of any modeling exercise is the model calibration and validation. Model calibration (parameter estimation) involves the automatic and/or manual adjustment of model parameters to minimize the difference between observed and predicted values, which is called the objective function. Physically based models such as SWMM require a large number of parameter values to be calibrated in applications to real catchments. Generally speaking, model calibration is involving determination of model parameters that gives the best possible correspondence between calculated value and simulated runoff from each sub catchment and each conduit. Calibration was accomplished by comparing the output of the SWMM model with the observed data at the same conditions.

In order to utilize any predictive watershed model for estimating the effectiveness of future potential management practices the model must be first calibrated to measured data and should then be tested (without further parameter adjustment) against an independent set of measured data. This testing of a model on an independent data set is commonly referred to as model validation. Model validation involves testing the ability of a model to simulate the hydrologic response of a basin for conditions different from that used during the calibration period (Lorup et al., 1998). Validation ensures that the calibrated parameters set performs reasonably well under an independent data set. In this study, the model simulation period was divided into two (calibration and validation) and the simple split sample test proposed by Klemes (1986) was adopted at daily time steps.

Generally for model calibration and validation observed stream flow or measured depth of flow are required within a watershed or drainage system at known point. It was not a common practice in our country to measure discharge in urban drainage system. Unfortunately, Addis Ababa City Road Authority had road expansion project in Bole Sub city. It was upgrading the road from Unity University to Anbesa Garage. Previously the road was 695 m length, 7m width and with rectangular cross section 0.6 by 0.8 m surface drainage. It was upgraded to 10m width including walkway and drainage system converted to 1m diameter circular subsurface drainage line. For this upgrading project, the office collected depth flow at the outlet point (near Anbesa Garage) from August 1 to August 10, 2019 and

converted the depth to discharge. Therefore, to accomplish model calibration and validation, the same area was modelled using SWMM 5.1 and simulate the flow for specific measured data. The modelled area show in Figure 3.12 with total drainage area of 27.6 hectares. Two time periods were selected denoted by the calibration period (From August 1 to August 5, 2019) and the validation period (From August 6 to August 10, 2019).



**Figure 3. 13: Delineated area for model calibration and validation (From Unity University (J1) to near Anbesa Garage (J5)).**

### 3.4.6.1 Model performance evaluation criteria

The performance of a model must be evaluated on the extent of its accuracy, consistency and adaptability (Negasa, 2013). A forecast efficiency criterion is therefore necessary to judge the performance of the model. Assessing storm rainfall runoff modeling requires subjective and/or objective estimates of the closeness of the simulated behavior of the model to observations.

In this thesis work, the model performance in simulating observed from each sub catchment’s discharge has been evaluated during calibration and validation through using **Nash and Sutcliffe efficiency criteria (NSE) and coefficient of determination (R<sup>2</sup>)**. The R<sup>2</sup> and NSE simulation efficiency measure how well trends in the calculated data are reproduced by the simulated results over a specified time period and for a specified time step.

**Nash-Sutcliffe Efficiency, NSE:**

The Nash and Sutcliffe Efficiency (NSE) is a measure of efficiency that relates the goodness-of fit of the model to the variance of measured data. NSE can range from -∞ to 1 and an efficiency of 1 indicates a perfect match between observed and simulated discharges. NSE value between 0.9 and 1 indicate that the model performs very well while values between 0.6 and 0.8 indicate the model performs well (Abeyou, 2008).

The **NSE efficiency**, proposed by Nash and Sutcliffe (Nash, 1970), is defined as:

$$NSE = 1 - \frac{\sum_{i=1}^n (Q_o - Q_s)^2}{\sum_{i=1}^n (Q_o - \bar{Q}_o)^2} \dots\dots\dots (3.13)$$

Where:

- $Q_o$  = measured discharge,
- $Q_s$  = Simulated flow,
- $\bar{Q}_o$  = Average of measured discharge and

**Coefficient of Determination, R<sup>2</sup>:**

The coefficient of determination R<sup>2</sup> is defined as the squared value of the coefficient of correlation. It is estimated as:

$$R^2 = \frac{\{\sum_{i=1}^n (Q_s - \bar{Q}_s)(Q_o - \bar{Q}_o)\}^2}{\{\sum_{i=1}^n (Q_s - \bar{Q}_s)^2\}\{(Q_o - \bar{Q}_o)^2\}} \dots\dots\dots (3.14)$$

Where:

- $Q_o$  = measured discharge,
- $Q_s$  = Simulated flow,
- $\bar{Q}_o$  = Average of measured discharge and
- $\bar{Q}_s$  = Average of simulated flow.

The range of values for R<sup>2</sup> is 1.0 (best) to 0.0

**Relative Error, RE:**

**Relative Error** is a measure of the uncertainty of measurement compared to the size of the measurement.

**Relative error** is also known as **relative** uncertainty or approximation **error**.

$$RE = \frac{\sum_{i=1}^n (Q_o - Q_s)}{\sqrt{\sum_{i=1}^n Q_o}} \dots\dots\dots (3.15)$$

Where:

- $Q_o$  = measured discharge,
- $Q_s$  = Simulated flow,

### 3.4.7 Sustainable measure to minimize runoff occurrence

Sustainable approach is the additional benefits such as environmental improvement, natural groundwater recharge, runoff reduction as well as energy savings. In UK, generally regard sustainable drainage systems as SUDS (Woods-Ballard et al., 2007 and 2015). Similar green drainage systems are called Low Impact Development (LID) or Best Management Practices (BMPs) in United States (USEPA, 2006).

### 3.4.8 SWMM LID modeling techniques

The purpose of LID is to reduce and/or eliminate the altered areas of the post development hydrograph, as shown by the shaded areas by reducing the peak discharge rate, volume, and duration of flow through the use of site design and storm water quality control measures. The benefits of reduced storm water runoff volume include reduced pollutant loadings and increased groundwater recharge and evapotranspiration rates.

The storm water management model (SWMM5) has also widely used to model SUDS through its low – Impact Development (LID) module (Burszta-adamiak and Mrowiec).

LID practices are designed to capture surface runoff by providing detention, infiltration, evapotranspiration, or some combination of the three. SWMM LID features are attributes of individual sub catchments. SWMM allows the user for placing LID controls:

- Create a new sub catchment dedicated exclusively to a single LID control; or
- Place one or more LID controls within an existing sub catchment, displacing an equal.

Five common types of LID (Bio Retention Cells, Vegetative Swales, Rain Barrel, Porous Pavement and Infiltration Trenches) are programmed in SWMM and are accessed through simple dialog boxes. The

LID technologies were programmed using algorithms that already existed in the SWMM engine and generic LID unit is represented by a number of vertical layers (Rossman, 2010).

### 3.4.8.1 Bio – Retention Cell

In the bio retention cell scenario, runoff from the study area was routed through a bio retention cell in the LID area. (Lucas 2010). Bio retention cells are depressed landscapes into which runoff is directed and allowed to pond, filter, and infiltrate. Some bio retention cells modeled by the LID Sizing consist of the design parameters specified in Section, including a 6” (15.2cm) ponding depth underlain by 18”(45.7cm) of bio retention soil mix and 12” (305mm), 24” (61cm), or 36” (91.4cm) of gravel storage.(a minimum storage depth of 12”). The ponding zone allows for temporary storage of runoff and promotes percolation into the bio retention mix, where runoff is also stored in the mix’s pore structure, as well as filtered and bio treated. The runoff eventually drains into the gravel layer below which provides a third storage component. A perforated underdrain is located at the top of the gravel storage component to prevent overflow of the system. (Sun et al. 2011). The use of bio-retention areas is appropriate in relatively small catchments, typically in the region of 1000-4000 m<sup>2</sup>. Several smaller bio-retention areas can be linked together for larger catchments (Endicott & Walker, 2003; Woods-Ballard et al., 2007).

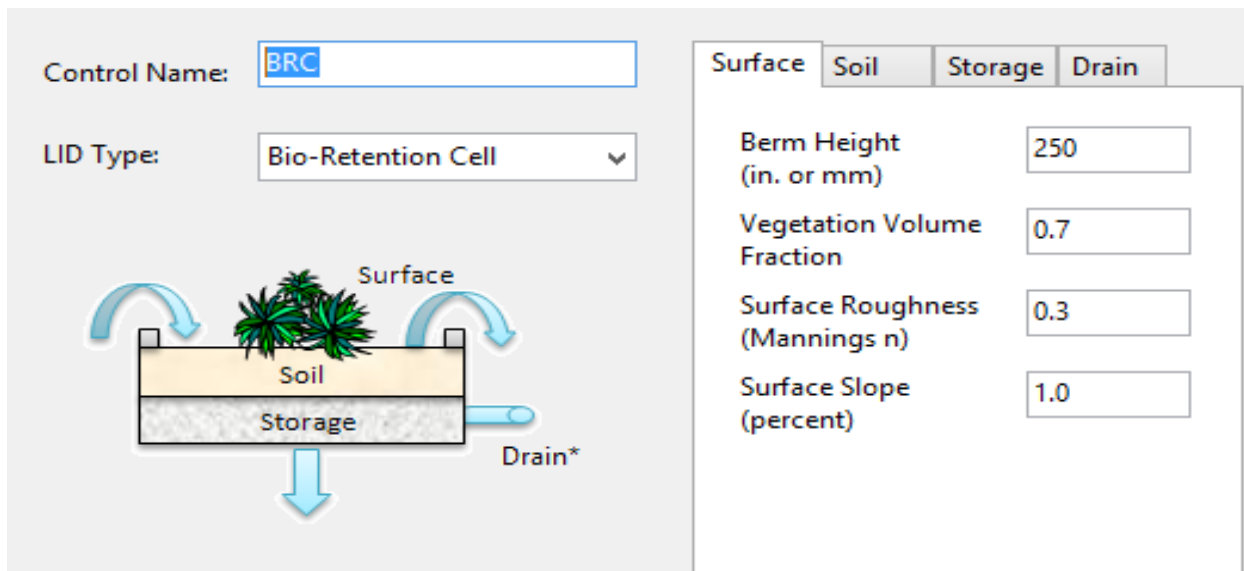


Figure 3. 14: LID Control editor in SWMM 5.1 (Bio – Retention Cell)

Table 3. 5: Design elements of the bio – retention cell (Seema, 2014)

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Layer	Properties	Values
<b>Surface</b>	Berm Height, mm	250
	Vegetative Volume Fraction	0.1
	Surface Roughness	0
	Surface Slope, %	0
<b>Soil</b>	Thickness, mm	500
	Porosity	0.5
	Field Capacity	0.2
	Wilting Point (Volume Fraction)	0.1
	Conductivity, mm/hr.	0.5
	Conductivity slope	40
	Suction Head, mm	290
<b>Storage</b>	Thickness, mm	400
	Void Ratio	0.75
	Seepage Rate, mm/hr.	10.9
	Clogging Factor	0.1
<b>Planning Parameters</b>	Area of each unit, m <sup>2</sup>	300 - 1000
	Number of units	$\geq 1$
	Surface width per meter unit	$>200$

	% of initially saturated	0
	% of impervious area treated	20

### 3.4.8.2 Permeable Pavement

Porous pavement is a paved pervious surface underlain by a gravel storage zone. The pavement consists of less fine aggregates than traditional concrete or asphalt, and the larger pore spaces that result allow for temporary storage of runoff. The runoff eventually drains into the gravel layer below which provides an additional storage component and allows infiltration into the underlying native soils. The porous pavement modeled by the LID assumes a pavement thickness of 5” (12.7cm) with a gravel storage depth dependent on the saturated conductivity of the underlying native soils. (Seema, 2014).

Table 3. 6: Designing and planning parameters of Permeable Pavement used in this study

<b>Planning Parameters</b>	Area of each unit, m <sup>2</sup>	400 - 1500
	Number of units	1
	Surface width per meter unit	>200
	% of initial saturation	0
	% of impervious area treated	11

Control Name:

LID Type:

Soil	Storage	Drain
Surface		Pavement
Thickness (in. or mm)	<input type="text" value="250"/>	
Void Ratio (Voids / Solids)	<input type="text" value="0.15"/>	
Impervious Surface Fraction	<input type="text" value="0"/>	
Permeability (in/hr or mm/hr)	<input type="text" value="100"/>	
Clogging Factor	<input type="text" value="0"/>	

Figure 3.15: LID Control editor in SWMM 5.1 (Permeable Pavement)

## CHAPTER FOUR

### 4. RESULT AND DISCUSSION

#### 4.1 IDF Curve developed for Bole Air Port station

The daily rainfall data of study area was collected from NMSA about 20 years. Since rainfall data is most of time incomplete due to damage of measuring instruments, measurement errors and errors related to personnel in the charge. Therefore, some correction mechanisms should be applied to the raw data to increase the quality of data. Different rainfall quality tests was conducted on the raw data as mentioned section 3.3.2 of this thesis.

Intensity duration frequency (IDF) curves describe the relationship between rainfall intensity, rainfall duration and return period (or its inverse, probability of exceedance). Because of the absence extreme rainfall values for periods less than 24 hours (12, 6, 3 and etc.) in Bole Airport station, IDF curve developed to obtain the depth and intensity of 5 or 10 or 15 etc. minute interval that used as in put for SWMM5 model. The relationship adopted for IDF development at a given station, any probability distribution can be used but the reliability of the distribution were assessed whether a given distribution are suited to a data set or not by the goodness of fit tests. Based on studies of a large number of rainfall gauges in Ethiopia, ERA 2013 manual suggests the Log Pearson Type III distribution for IDF development in Ethiopia. Hence, Log Pearson Type III distribution have been adopted for this thesis IDF development. Table A.6 attached in appendix part shows all the data used to develop the IDF Curve below.

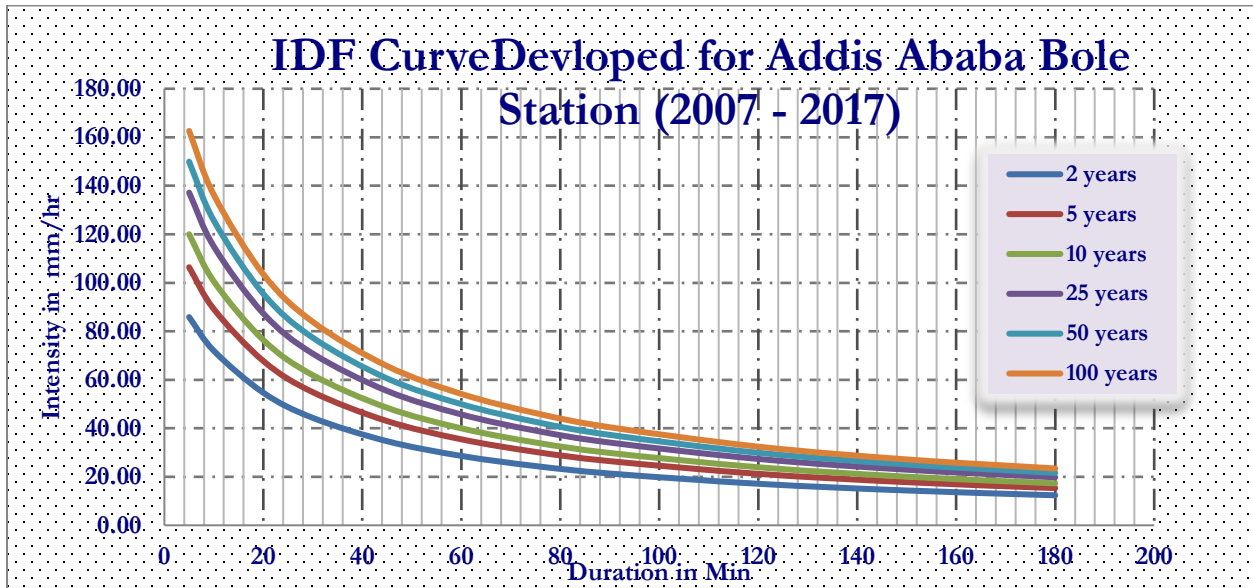


Figure 4. 1: IDF Curve developed for Bole Air Port station, Own analysis 2019

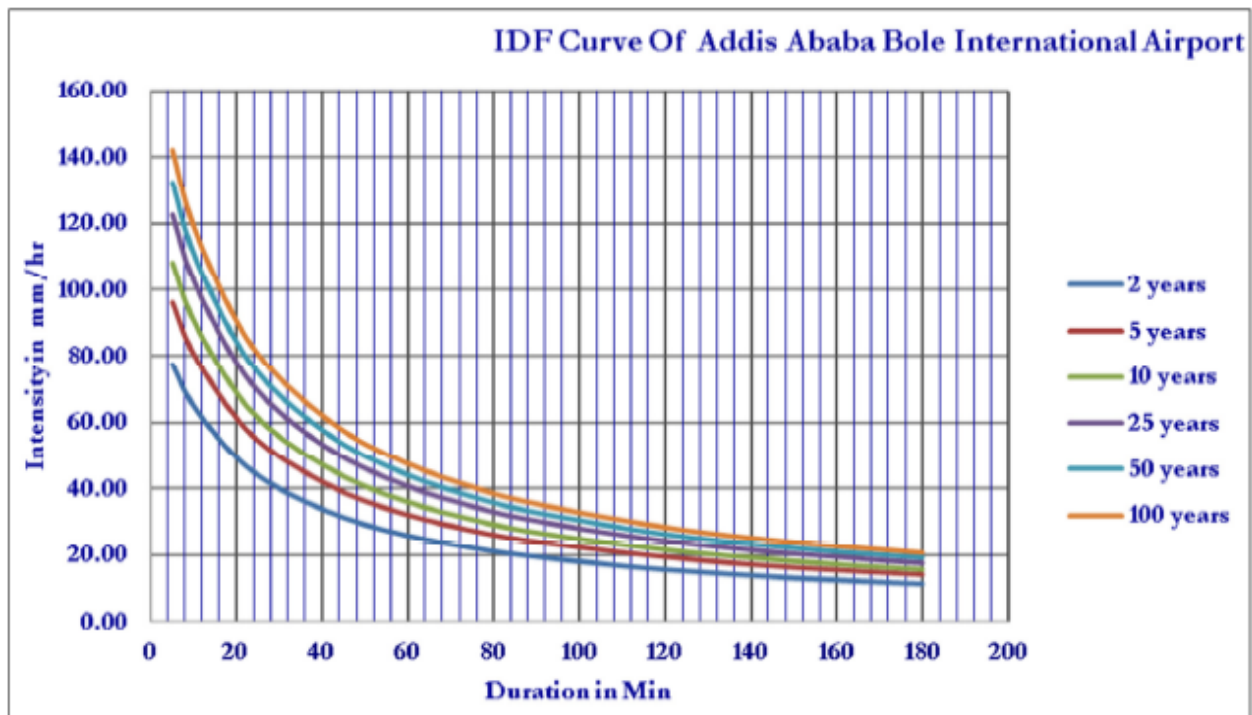


Figure 4. 2: IDF Curve of Region A2 developed by ERA 2013

Since the main objective of this thesis is to consider the runoff difference during the construction period and the recent period and one of the factor that affect the quantity of runoff is rainfall intensity, its analysis is important. Therefore, the variation of this parameter should be consider by comparing IDF

Curve developed by ERA in 2002 (as past) for different rainfall regions and IDF developed by me as present (2019). Based on classification of rainfall regions of ERA, Addis Ababa categorized under Region A2 and IDF Curve developed by ERA in 2002 attached in appendix part of figure B.1. The above two IDF curves shows almost the same value of intensity of rainfall for all duration, except some increase on peak values was observed for shorter duration in recent curve. Based on Gezagn (2017) study on Long term climate description in Ethiopia and based on the above graph, rainfall intensity behaves almost stationary in Addis Ababa, Ethiopia. Almost the same value observed for Bole Airport station for different periods of time.

## **4.2 Land Use / Land Cover of the study area in the past and present**

Mapping and classifying land use and land cover is very important in hydrological study. Before the analysis of land use and land cover change each homogeneous land use and land cover should be defined. Defining of the land use of the study area was done using remote sensing data. Since the land use of the study area not available for 1995, which was the time China Road and Bridge Corporation (CRBC) signed a contract agreement with Addis Ababa City Roads Authority (AACRA) for the project which was 25 years back. So that the preparing land use map the area from the satellite image is required.

Preprocessing of land sat images such as layer stacking, mosaic king and band and color combination was done using ERDAS IMAGINE 2014 software for the past and the present (2020) land use was analyzed using Arc GIS software after delineating the study area from Google Earth Pro. From analysis, it is noted that the land use and Land Cover dramatically changed and the amount of runoff expected to be increased from the Ring road Design Period. As shown below in page 65, Table 4.1 land use change (imperviousness) dramatically changing from 14% in 1995 to 94% in 2019. Since imperviousness reduces the amount of water infiltrate in the soil and increases the amount surface runoff, it is assumed that the amount of total runoff generated in 2019 will be much higher than the runoff of the same area in past or in 2002. That result going to be obtained taking in to consideration this land use change below in section 4.4.1 and 4.4.2 during running Storm Water Management Model.

**Figure 4. 3 True color Display of 2020 Geo-Eye (Google Erath -Pro) Imagery of Bole Gerji Area**

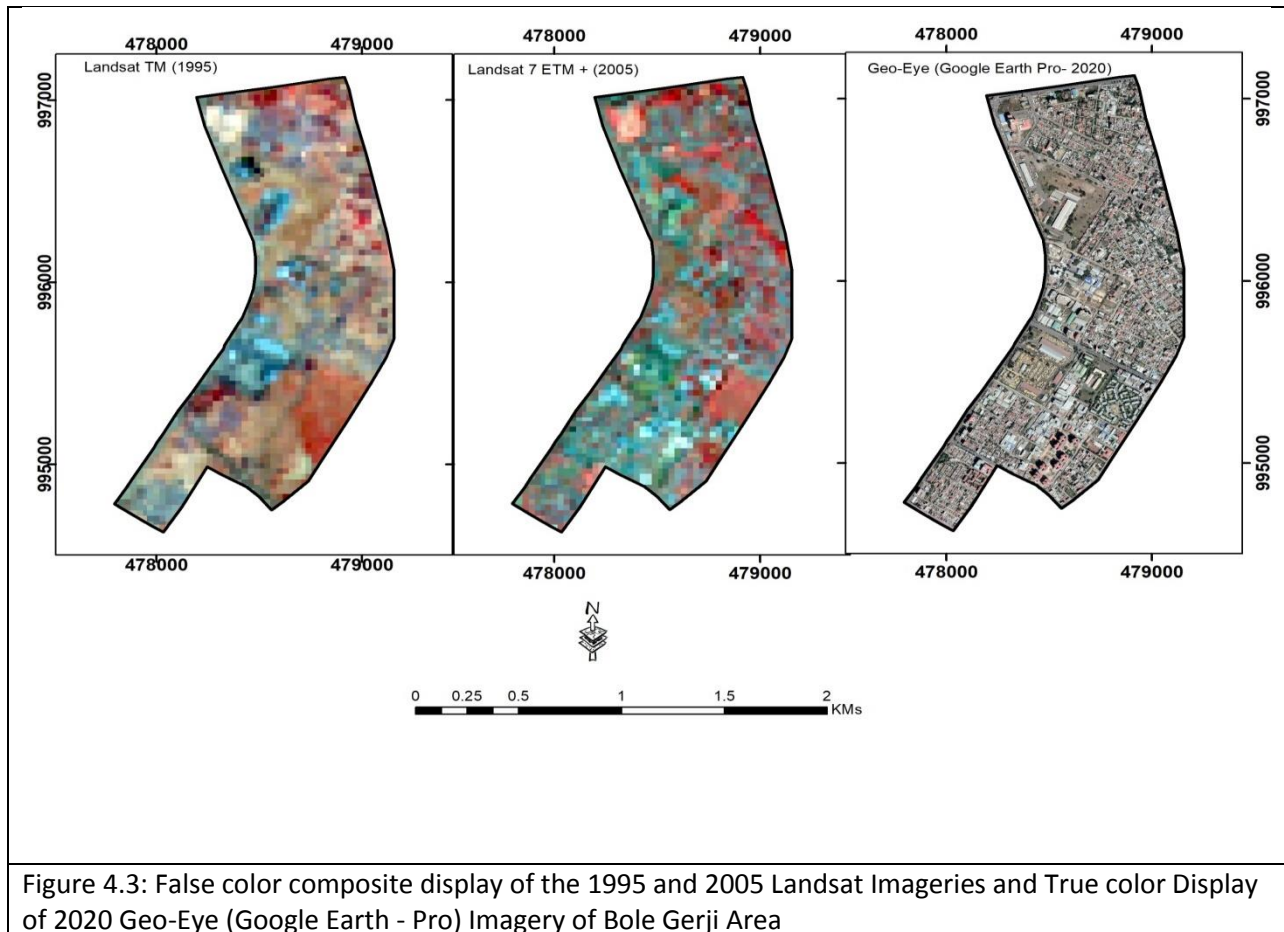


Figure 4.3: False color composite display of the 1995 and 2005 Landsat Imageries and True color Display of 2020 Geo-Eye (Google Earth - Pro) Imagery of Bole Gerji Area

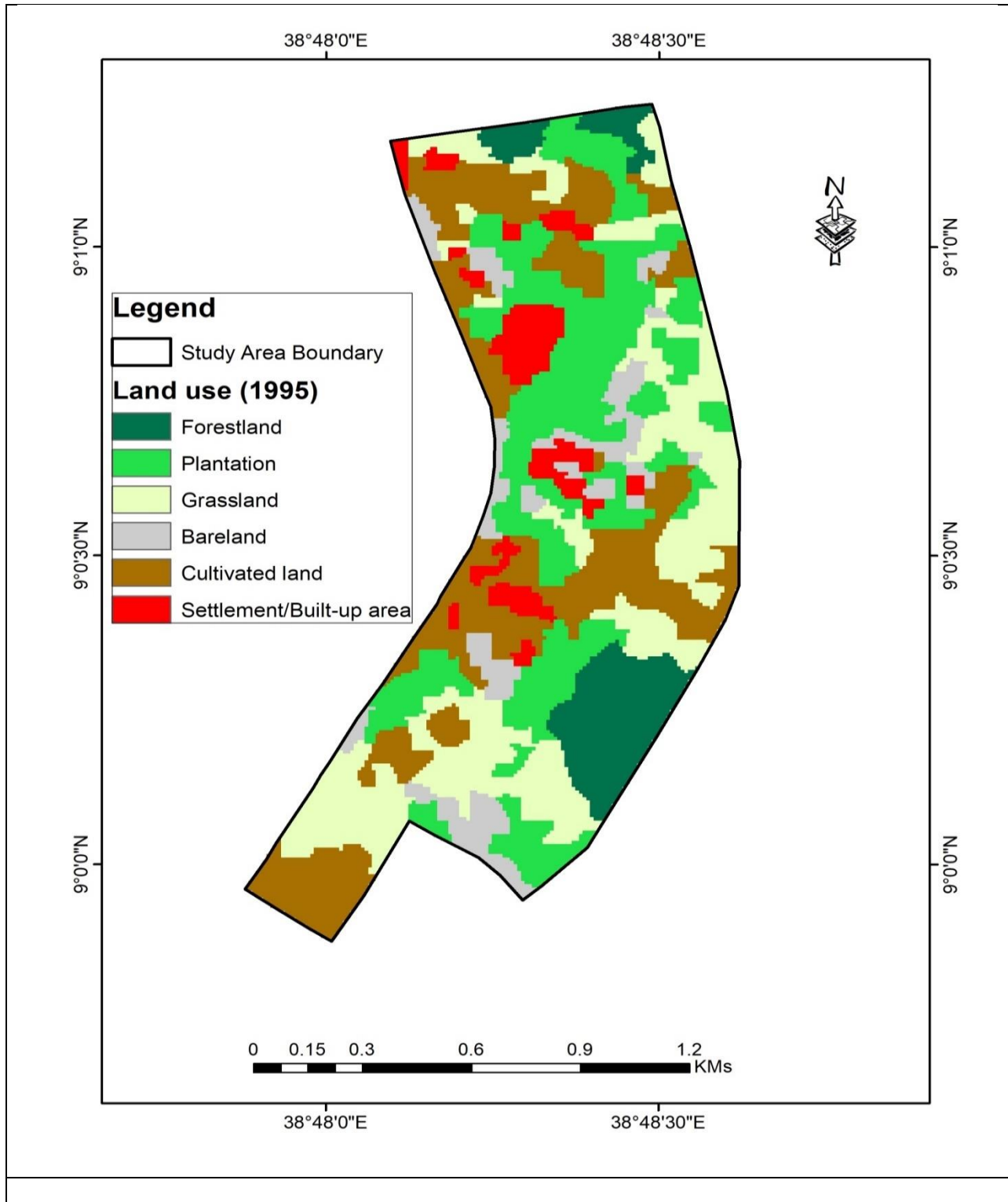


Figure 4. 4 Land Use map of Bole Gerji in 1995

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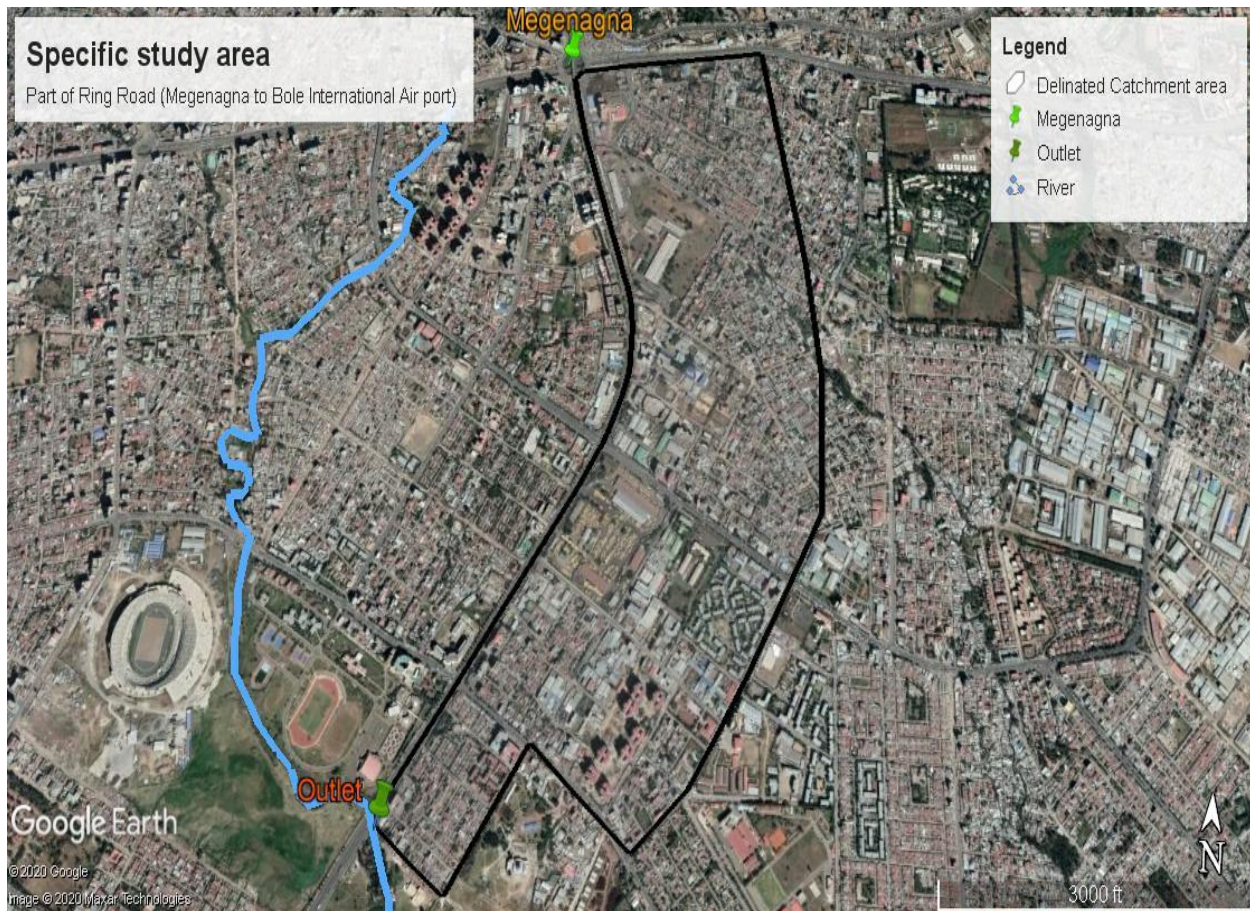


Figure 4. 5: Land use map of Bole Gerji in 2020

Table 4.1: Summary of land use pattern of Bole Gerji area in 1995 and 2020

Land uses	Area (1995)			Area (2020)		
	COUNT (10mx10m)	Ha.	%	COUNT (100mx100m)	Ha.	%
Forestland	1502	15.02	8.0	0	0.00	0.0
Plantation	4710	47.10	25.1	16912	1.69	0.9
Grassland	4193	41.93	22.3	0	0.00	0.0
Bare land	1421	14.21	7.6	80805	8.08	4.3
Cultivated land	4380	43.80	23.3	0	0.00	0.0
Settlement (Built-up area)	2586	25.86	13.7	17816	178.16	94.8
	18792	187.92	100	18792	187.92	100

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In order to assess the accuracy of the classified 1995 Landsat imagery, confusion matrix method used. The study area land use was classified into five land use classes (plantation, grassland, bare land, cultivated land and built-up area), 78 sites were pointed with GPS and some respondents were asked what land use classes these points represented in 1995 for independent reference pixels.

**Table 4.2 Confusion Matrix for assessment of LULC classification from Landsat TM 7 in 1995**

		Land use class (field observed)					Total	Producer Accuracy
		Plantation	Grassland	Bare land	Cultivated land	Urban Built-up Area		
		No.	No.	No.	No.	No.		
Land use Class (Office classified)	Plantation	10	0	1	1	0	12	83.3
	Grassland	0	12	1	0	0	13	92.3
	Bare land	0	1	10	1	0	12	83.3
	Cultivated land	0	1	1	8	1	11	72.7
	Urban Built-up Area	0	0	0	0	30	30	100.0
Total		10	14	13	10	31	78	100.0
User Accuracy		100	86	77	80	97	78	

Observed agreement,  $P_o = (\text{sum of diagonal} / \text{Row marginal}) = ((10+12+10+8+30) / (12+13+12+11+30)) = 0.892$

Expected agreement,  $P_e = ((12*10/78*78) + (13*14/78*78) + (12*13/78*78) + (11*10/78*78) + (30*31/78*78)) = 0.241$

Kappa index,  $K = (P_o - P_e) / (1 - P_e) = (0.892-0.241) / (1-0.241) = 0.863934$  or 86.4%

- Kappa Index > 0.80 Strong accuracy (degree of agreement)
- 0.40 > Kappa Index > 0.80 Moderate accuracy (degree of agreement)
- Kappa Index < 0.40 Weak accuracy (degree of agreement)

## 4.3 Model Calibration and Validation

### 4.3.1 Sensitive parameters selection

It is necessary to conduct a detailed sensitivity analysis to evaluate the main parameters of the SWMM which are the most sensitive parameters affecting the rainfall-runoff-routing simulation in the model. Sensitivity analysis was performed to choose the most sensitive flow parameters that influence the catchment represented by SWMM to be used for calibration. After repetitive iteration, the following values are used to calibrate the SWMM5.1 Model.

Table 4. 3: Values of sensitive parameters for model calibration

Sensitive Parameters for SWMM Hydrology and Hydraulic Analysis					
Name of Parameter	Meaning	Value Range	Initial Values	Used value / Sensitive to peak flow	
N – Imperv	Manning’s roughness coefficient for impervious area	0.011 – 0.015	0.012	0.013	
N - Perv	Manning’s roughness coefficient for pervious area	0.05 – 0.8	0.13	0.8	
Destore – Imperv	Depth of depression storage in impervious area	0 – 3	1	1.5	
Destore - Perv	Depth of depression for pervious area	3 – 10	6	9	
Conduit Roughness	Manning’s roughness coefficient for conduit	0.011 – 0.024	0.014	0.016	
Infiltration Method	Green Ampt	Suction Head, mm	290	290	290
		Conductivity, mm/hr.	0.51	0.51	0.51
		Initial Deficit	0.25 – 0.26	0.25	0.26

### 4.3.2 Model Calibration

The calibration of SWMM model for runoff was done by using the daily - observed runoff data at near Anbesa Garage (outlet of the watershed) for the August 1, 2019 to August 10, 2019. In this study, I have discharge data of ten days. Out of the total discharge data two-third of the data were used for calibration and one-third of the data were used for validation. The calibration period was carried out for six days that is from August 1, 2019 to August 6, 2019 whereas the validation period were carried out for four days from August 7, 2019 to August 10, 2019.

The model was calibrated by using the values of parameters that were identified as highly sensitive to runoff as it was described under sensitivity analysis section. The simulation was done and parameters were calibrated using manual calibration and the calibrated parameters were updated in the model and the final simulation was run.

For model performance evaluation NSE, and  $R^2$  were used as criteria and the objective function. The simulated and observed daily discharge at the gauging site were plotted for visual comparison.

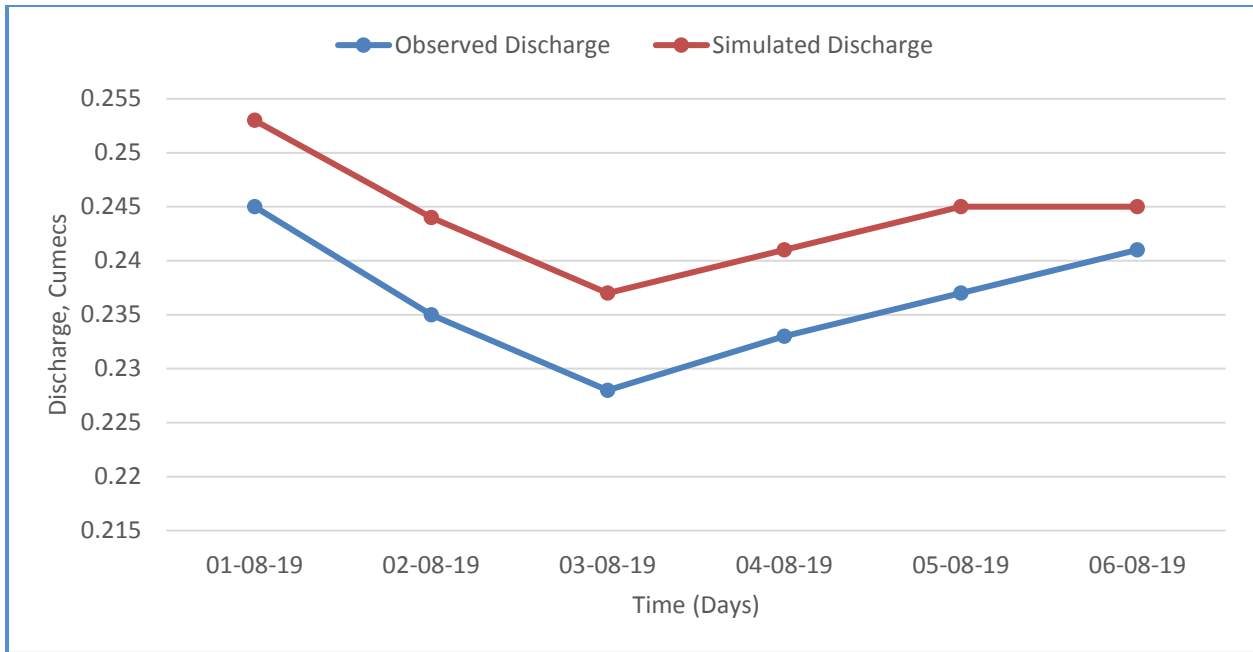
The observed and simulated runoff for the calibration period were also plotted against each other in order to determine the goodness of fit by using the coefficient of determination ( $R^2$ ) and the Nash-Sutcliffe coefficient of efficiency (NSE). NSE can range from  $-\infty$  to 1 and an efficiency of 1 indicates a perfect match between observed and simulated discharges. NSE value between 0.9 and 1 indicate that the model performs very well while values between 0.6 and 0.8 indicate the model performs well; and the range of values for  $R^2$  is 1.0 (best) to 0.0 (Abeyou, 2008). In this study, the coefficient of determination ( $R^2$ ) value for daily discharge for the calibration period was 0.91. In addition, the Nash- Sutcliffe coefficient of efficiency (NSE) for the same period was found to be 0.86. In general, the model performs very well in predicting the discharge from Bole Gerji catchment.

Table 4. 4: Observed and Simulated flow of the gauging station

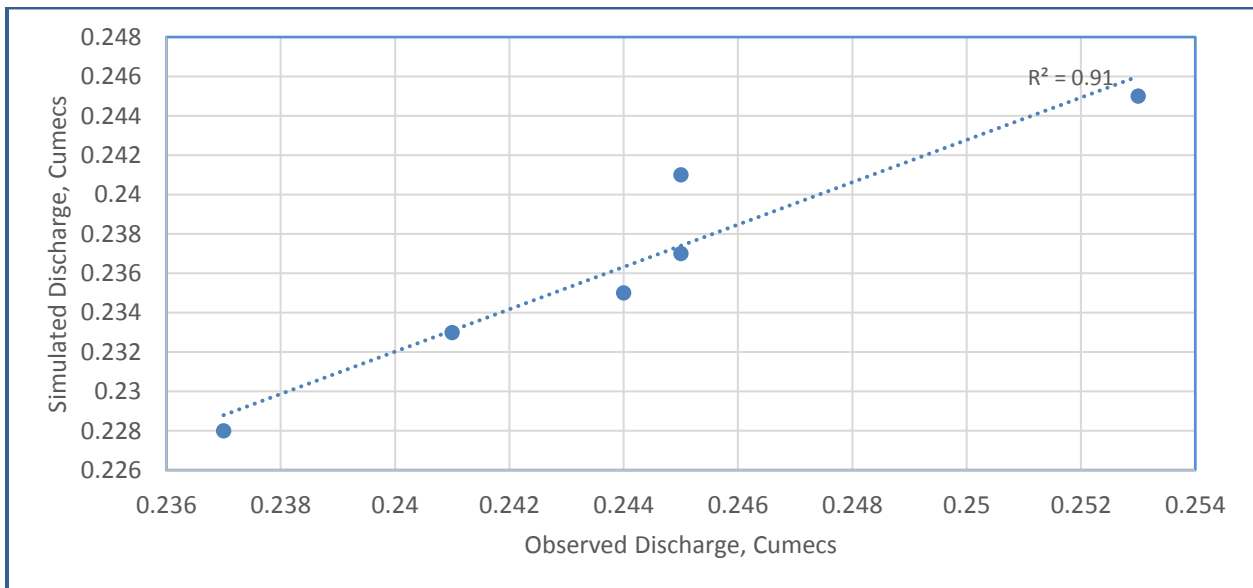
Date	Observed discharge, m <sup>3</sup> /sec	Simulated discharge by SWMM, m <sup>3</sup> /sec
01-08-2019	0.246	0.251
02-08-2019	0.237	0.242
03-08-2019	0.228	0.233
04-08-2019	0.236	0.240

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05-08-2019	0.238	0.241
06-08-2019	0.238	0.241



**Figure 4. 6: Observed and Simulated flow of the gauging station**



**Figure 4. 7: Fit line of Observed and Simulated flow for Calibration Period**

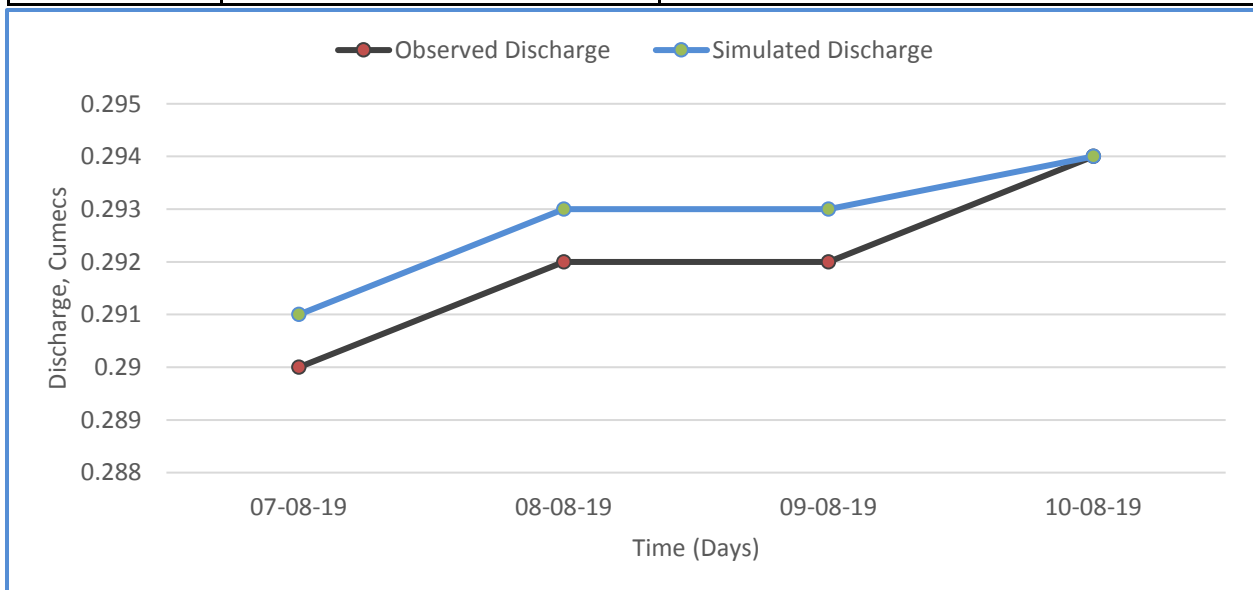
### 4.3.3 Model Validation

Model validation is the process of demonstrating that a given site-specific model is capable of making sufficiently accurate predictions. This implies the application of the calibrated model without changing the parameter values that were set during the calibration, when simulating the response for a period other than the calibration period (Refsgaard, 1997).

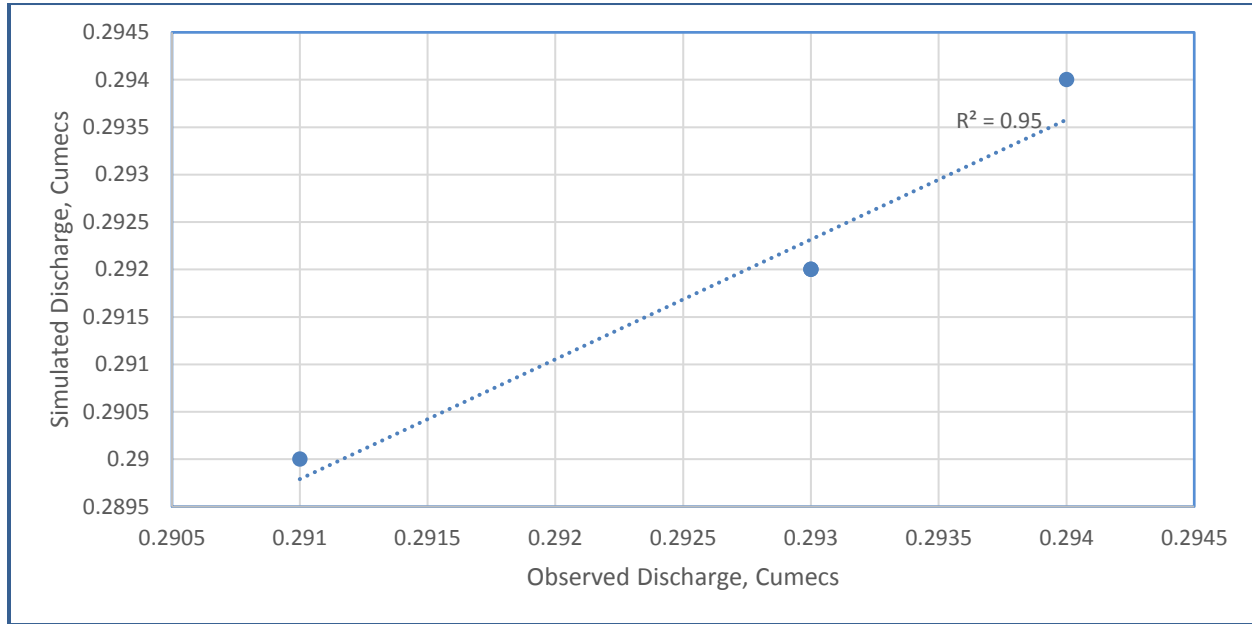
The model validation was carried out for daily discharge of the days August 7, 2019 to August 10, 2019. Model validation involves re-running the model using the input data, which are not used in calibration period and without any adjustment of calibration parameters at different period. The value of coefficient of determination ( $R^2$ ) and the Nash-Sutcliffe efficiency (NSE) for daily discharge of the period August 7 – August 10, 2019 of the gauging station was 0.88 and 0.95 respectively. The observed and simulated daily stream flow for validation of the model is shown in the table 4.8. Below.

Table 4. 5: Observed and Simulated flow at gauging station

Date	Observed discharge, m <sup>3</sup> /sec	Simulated discharge by SWMM, m <sup>3</sup> /sec
07-08-19	0.29	0.291
08-08-19	0.292	0.293
09-08-19	0.292	0.293
10-08-19	0.294	0.294



**Figure 4. 8: Observed and Simulated flow at gauging station**



**Figure 4. 9: Fit line of Observed and Simulated flow for Validation**

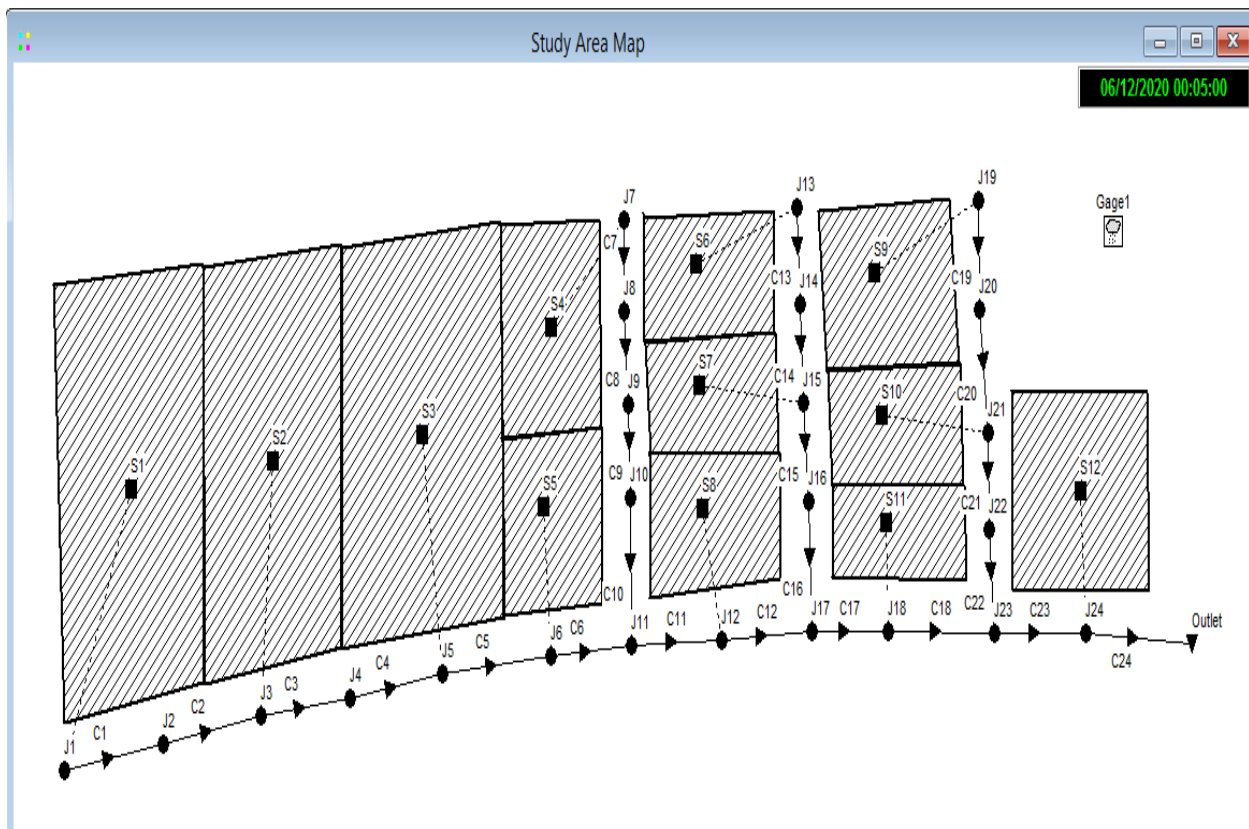
#### 4.4 EPA SWMM 5.1 Runoff Simulation Results

SWMM is one of the most widely used open-source models available for simulating urban drainage systems. The major advantage of SWMM is that it incorporates the capabilities of both hydrological and hydraulic simulation. The existing situation of the modelled area was shown in figure 4.10, represented as J1 (the first manhole in front of Bole sub-city building) to J24 are manholes / junctions, S1 to S12 are sub catchments, C1 to C24 are conduits / pipes, and Gage 1 represents Rain gauge in Bole Airport station. There was many inlets in the modeled area and J1 near Megenagna, In front of Bole Sub City building consider as the starting inlet. The outlet located on the same side as inlet and at distance about 2.8 km in the way to Bole. Total catchment area modeled was 188ha and total length used in modelling was 8km. All the pipes are circular in section with maximum diameter of 1m and junctions are rectangular in section and their invert elevation, offsets etc. are attached in Appendix part from Table A.1 to Table A.4. According to Rossman (2010), This Model uses the Continuous rainfall data or the IDF for corresponding time of concentration of sub-catchments. The time of concentration varies from 51min- 74min for all catchments. The Intensity rainfall for the respected values of Time of concentration which varies

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36mm/hr. – 46mm/hr. for the 10yr design period. The model has been run with the continuous rainfall events with 10 years return periods and the corresponding rainfall intensity used to analyze the past and current performance.

Runoff is a function of catchment size, land use, soil type, slope and climate. All import input parameters from analysis of rainfall and catchment characteristics were prepared in form that the model understand. Different case scenarios have been considered in this study to obtain a fully understanding of the system performance under multiple working conditions. After calibration and validation of the model for flow, running the hydrological model with the intensity data, the runoff generation within the area was obtained. The view of catchment used for simulation in SWMM was shown in Figure 4.10 below.



**Figure 4. 10: Study are map after running the SWMM 5.1 model**

To meet the objective of this study, the model run in two different periods to simulate total runoff peak and node flooding of the catchment in the past and present.

#### 4.4.1 Simulation result of the model in past or before the construction of the road (in 1995)

Since Addis Ababa Ring Road project construction commenced in early 1990's and my study area is part of it, I consider this period as past condition. To run the simulation, land use of Bole Gerji area in 1995 downloaded and analyzed and IDF curve developed by ERA in 2002 were used to obtain input parameters to the model. As show in Table 4.6 column 10, the total runoff from the catchment is 4.27 m<sup>3</sup>/sec. There was no conduit surcharge and node flooding. This was the existing situation in the area just after construction or 25 years back.

Table 4. 6: Simulated runoff of the study area in the past (in 1995)

Topic: Subcatchment Runoff <span style="float: right;">Click a column header to sort the column.</span>									
Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Imperv Runoff mm	Perv Runoff mm	Total Runoff mm	Total Runoff 10 <sup>^6</sup> ltr	Peak Runoff CMS
S1	40.00	0.00	0.00	29.19	3.90	5.66	9.57	8.13	0.55
S2	39.00	0.00	0.00	27.87	5.32	4.97	10.29	8.75	0.57
S3	38.00	0.00	0.00	27.85	5.18	4.42	9.60	8.16	0.54
S4	45.00	0.00	0.00	27.94	6.17	9.46	15.63	3.13	0.24
S5	39.00	0.00	0.00	26.51	5.33	7.04	12.36	2.16	0.16
S6	46.00	0.00	0.00	25.85	6.31	13.72	20.03	4.01	0.40
S7	45.00	0.00	0.00	25.79	6.17	12.92	19.09	3.82	0.36
S8	38.00	0.00	0.00	26.25	5.19	6.43	11.62	4.07	0.31
S9	46.00	0.00	0.00	29.08	4.49	12.33	16.82	5.05	0.41
S10	46.00	0.00	0.00	25.98	4.51	15.43	19.94	2.99	0.33
S11	36.00	0.00	0.00	24.06	4.91	6.90	11.81	1.77	0.16
S12	36.00	0.00	0.00	27.06	3.51	5.34	8.85	2.88	0.24

#### 4.4.2 Simulation result of the model in present (in 2019)

To run the simulation, land use of Bole Gerji area in 2020 downloaded and analyzed and the same IDF curve developed by ERA in 2002 were used to obtain input parameters to the model. As show in Table 4.6 column 10, the total runoff from the catchment is 12.94 m<sup>3</sup>/sec. This increase in runoff from 4.27

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m<sup>3</sup>/sec to 12.94 m<sup>3</sup>/sec almost triple the value in the past was mainly due the substantially change in land use / increase imperviousness in the area and Table 4.7, column 10 clearly shows the runoff from each sub – catchments.

**Table 4. 7: Simulated runoff of the study area in the present (in 2019)**

Topic: Subcatchment Runoff <span style="float: right;">Click a column header to sort the column.</span>									
Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Imperv Runoff mm	Perv Runoff mm	Total Runoff mm	Total Runoff 10 <sup>^6</sup> ltr	Peak Runoff CMS
S1	40.00	0.00	0.00	0.50	34.18	1.37	35.56	30.22	1.92
S2	39.00	0.00	0.00	0.52	34.09	1.13	35.23	29.94	1.85
S3	38.00	0.00	0.00	0.52	33.15	1.09	34.24	29.10	1.79
S4	45.00	0.00	0.00	0.50	41.63	1.53	43.17	8.63	0.84
S5	39.00	0.00	0.00	0.49	36.34	1.28	37.62	6.58	0.51
S6	46.00	0.00	0.00	0.48	43.19	1.65	44.84	8.97	0.98
S7	45.00	0.00	0.00	0.48	42.24	1.60	43.84	8.77	0.92
S8	38.00	0.00	0.00	0.49	35.41	1.23	36.64	12.82	0.94
S9	46.00	0.00	0.00	0.49	42.96	1.62	44.58	13.38	1.41
S10	46.00	0.00	0.00	0.48	43.28	1.67	44.95	6.74	0.74
S11	36.00	0.00	0.00	0.48	33.54	1.14	34.68	5.20	0.33
S12	36.00	0.00	0.00	0.50	33.13	1.08	34.21	11.12	0.71

SWMM also contains a flexible set of hydraulic modeling capabilities used to generate profile plots showing how water surface depth varies across a path of connected nodes and links. The water profile plot along junction J1 to outlet in the left side of Megenagna – Bole ring was shown in figure 4.11 below. As shown in the figure, all the Conduits / pipes (C1, C2, C3, C4, C5, C6, C11, C12, C17, C18, C23 and C24) are under surcharge situation during peak flows and all the Nodes / Joints (J1, J2, J3, J5, J6, J11, J12, J17, J18, J22, J23 and J24) in the main road (Megenagna to Bole) are flooding or overflows during peak flow situations. But, substantial amount of over flow recorded in nodes J3, J5, J17, J18 and J24. Typical value of the flow velocity ranges from 0.1 to 4 m/s, depending on the natural conditions (ERA, 2013). For all pipes the velocities are within the range.

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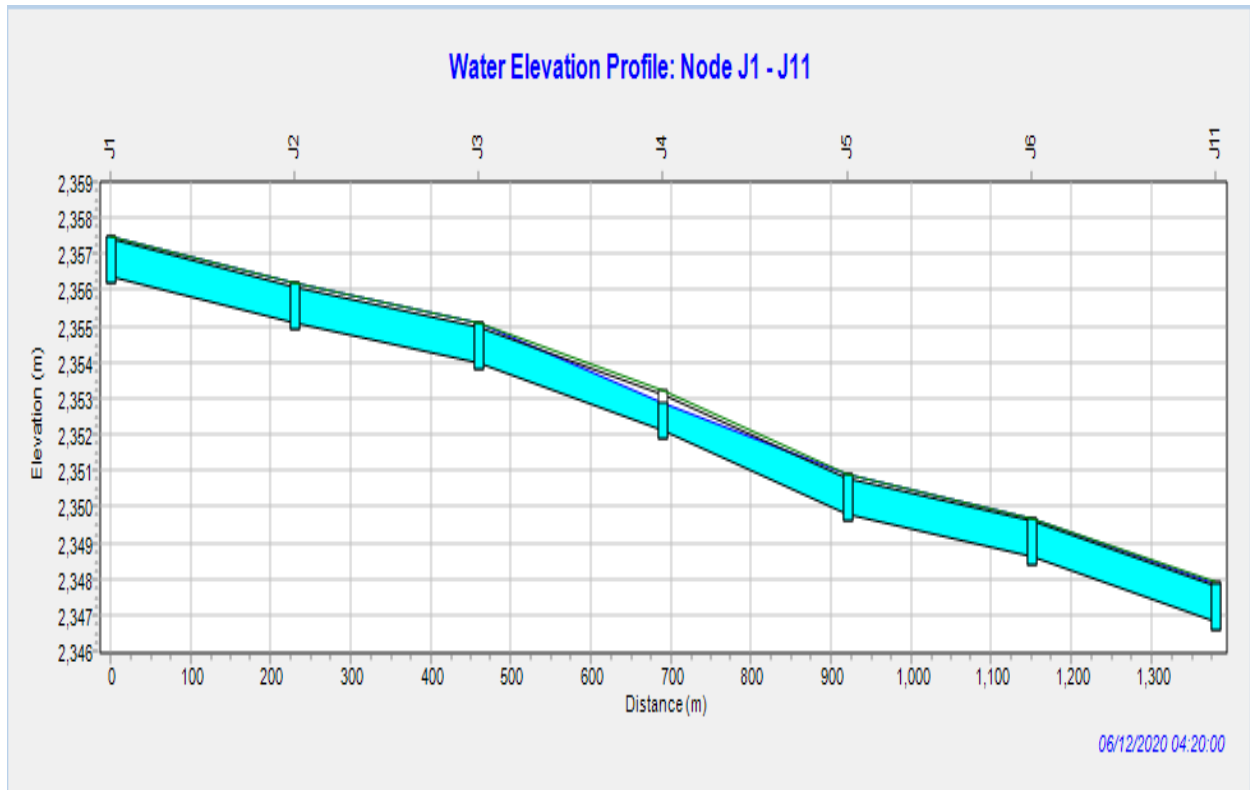


Figure 4.11: Water elevation profile showing node flooding from J1 to J11

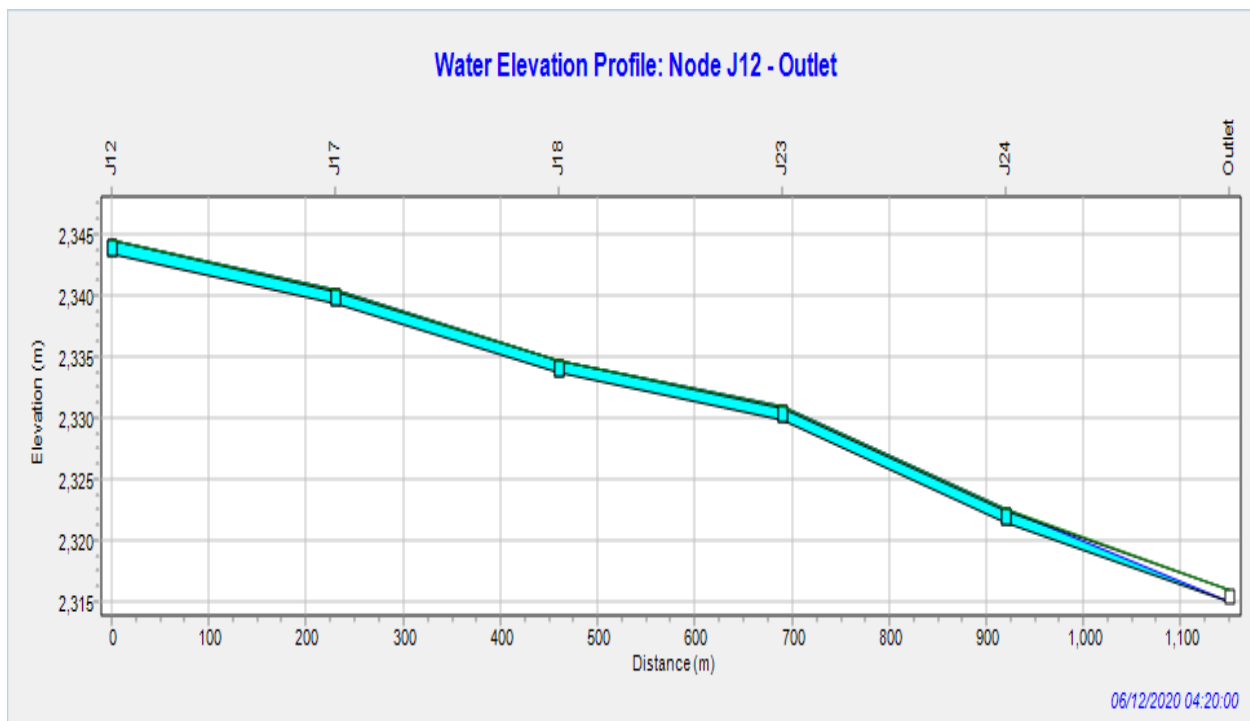


Figure 4.12: Water elevation profile showing node flooding from J12 to Outlet

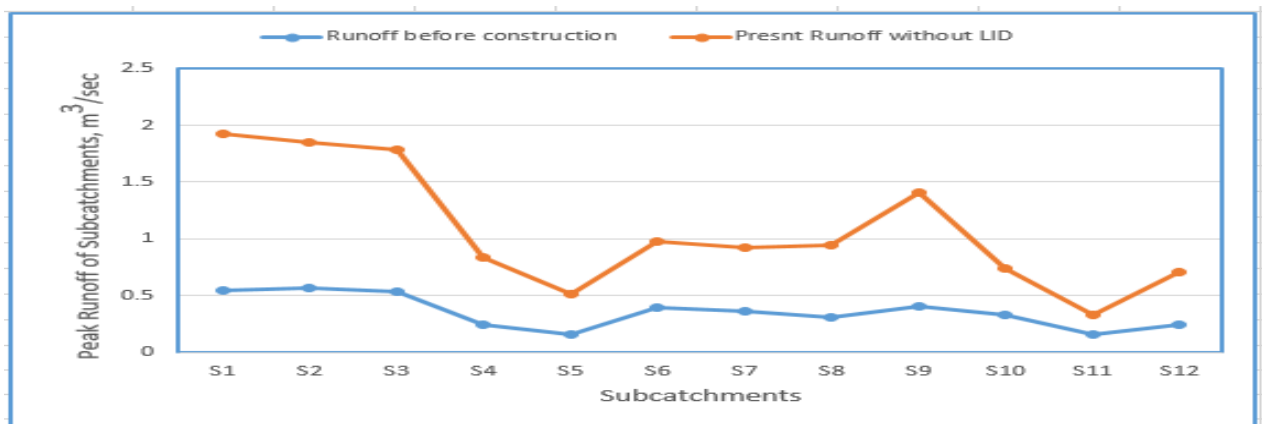
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In SWMM flooding will occur whenever the water surface at a node exceeds the maximum assigned depth. Normally such water was lost from the system. Substantial amount of total flooded volume of water from junction 3, 5, 17, 18 and 24 are  $11.43 \times 10^6$ ,  $28.478 \times 10^6$ ,  $7.567 \times 10^6$ ,  $12.117 \times 10^6$  and  $12.401 \times 10^6$  liters respectively as shown below in Table 4.8 column 6.

**Table 4.8: Node flooding occurring in 2019 before LID application**

Topic: <span style="border: 1px solid black; padding: 2px;">Node Flooding</span> <span style="float: right;">Click a column header to sort the column.</span>						
Node	Hours Flooded	Maximum Rate CMS	Day of Maximum Flooding	Hour of Maximum Flooding	Total Flood Volume $10^6$ ltr	Maximum Poned Depth Meters
J1	1.93	0.246	0	04:00	1.205	0.000
J2	2.43	0.143	0	03:46	1.116	0.000
J3	3.30	1.227	0	05:00	11.430	0.000
J5	4.95	2.300	0	04:27	28.476	0.000
J6	0.61	0.149	0	04:00	0.217	0.000
J11	0.35	0.121	0	03:51	0.082	0.000
J12	2.94	0.824	0	03:49	4.680	0.000
J17	3.42	1.361	0	04:01	7.567	0.000
J18	4.03	1.082	0	02:43	12.117	0.000
J23	1.03	0.681	0	03:52	1.762	0.000
J24	4.09	1.479	0	04:30	12.401	0.000
J22	0.16	0.117	0	04:00	0.054	0.000

Below in figure 4.13, the runoff generated in each sub catchment in the past and present are drawn. As show in figure, there is visible changes between two curves (runoff of the same area in past and present), due the land use change in the study area and the next step will be to minimize the gap between two graphs which means in directly minimize the runoff of each sub catchments and node flooding in critical nodes (J3, J5, J17, J18 and J24).



**Figure 4. 13: Peak Runoff of sub catchment in past and present without LIDs**

## 4.5 Runoff estimation using Rational Method

In this thesis Rational Method was used for comparison of the EPA SWMM model result and comparing the obtained results with others' finding using these two methods. Rational of selecting rational method for comparisons is because of area for each catchment is less than 50 hectare (0.5 sq. km) according to ERA manual 2013. Since the rational method equation and their characteristics have explained in the methodology part in detail. Runoff coefficient determination for the different catchment characteristics, time of concentration computation and corresponding rainfall intensity value for 10 years return period explained in part 4.1 of this thesis and detain analysis attached in appendix part table A.7.

Total runoff obtained from 12 urban sub-catchment using EPA SWMM is 12.94 m<sup>3</sup>/sec whereas from the same catchment using Rational Formula is 11.83 m<sup>3</sup>/sec. Similar study was done in India and the total runoff from seven sub catchments by SWMM is 2.177 m<sup>3</sup>/sec and by Rational Method is 1.109 m<sup>3</sup>/sec (Waikar, 2015) and other similar work done in Shire Endaslasse, Ethiopia and the total runoff from eleven sub catchments by SWMM is 9.28 m<sup>3</sup>/sec and by Rational Method is 7.45m<sup>3</sup>/sec (Mearg, 2019).

Peak runoff from each sub catchment calculated using Rational Method and the maximum values of discharge obtained from EPA SWMM 5.1 model study are given in Table 4.9.

**Table 4.9: Peak runoff values obtained by EPA SWMM and Rational Method**

Sub - catchment	Peak discharge calculated using SWMM (m <sup>3</sup> /sec)	Peak discharge calculated using Rational Method (m <sup>3</sup> /sec)
S1	1.92	1.89
S2	1.85	1.84
S3	1.79	1.62
S4	0.84	0.61
S5	0.51	0.38
S6	0.98	0.83
S7	0.92	0.81
S8	0.94	0.82
S9	1.41	1.23
S10	0.74	0.64
S11	0.63	0.55
S12	0.71	0.65
Sum	12.94	11.83

## 4.6 Scenarios development for mitigation measure

SWMM allows engineers and planners to represent combinations of green infrastructure practices to determine their effectiveness in managing runoff. Although some of these practices can also provide significant pollutant reduction benefits, in this research, SWMM models used to work on reduction in runoff mass load resulting from the reduction in runoff flow volume and peaks.

The Low Impact Development (LID) approach has been recommended as an alternative to traditional storm water design. A range of factors to be considered when selecting LID such that the most appropriate technique is adopted and LID are successfully implemented. Such factors include site suitability, available land space, cost, maintenance issues and community acceptance. SWMM can explicitly model eight different generic types of LID controls: Bio – Retention Cell, Infiltration Trench, Permeable Pavement, Vegetative Swales, Green Roof, Rain Barrels or Cisterns, Rooftop Disconnection and Rain Garden. The ERA claims that pipes in a drainage system must not work under pressure or surcharge and Nodes must not overflow or flooding for all selected return period rainfall event. Having these points in mind and to meet the goal of the research, scenarios can be developed to reduce and / or eliminate excess storm water that cause node flooding and pipe surcharging in section 4.4.2 (previous page) by using generic LID controls.

Analyzing the effect of land use / land cover change or imperviousness and change in intensity of rainfall on the urban drainage system is parts of this study. To do this, it is necessary to develop scenarios that reflect the changes made to the watershed land use. Scenario analysis is the process of evaluating possible future events through the consideration of alternative possible outcomes. Therefore, when scenario is developed it should be able to present several alternative future developments. The scenarios may be developed based on future land use master plan in the watershed if there is any. But, in the absence of future master plan, the scenarios can be developed by changing the land use by a specified percentage and quantify the changes caused by the conversion of impervious land use type to pervious by applying selected LIDs. Based on factors to be consider for selecting types of LIDs for specific area and more than 90 % of imperviousness (land surface) is due to rooftops and paved roads , I select Permeable Pavement, Rooftop Disconnection, Rain Barrels or Cisterns and Bio – Retention Cells as LIDs for the study area. Rooftop Disconnection and Rain Barrels are seems suitable LID for the study area, but insignificant in terms of reducing runoff generation.

Depending on the watershed area delineated by Arc GIS and Google Earth Pro, the land use obtained in this study area, Bole Gerji consists of about 8 km pedestrian road and in average in each sub-catchments more than 70% covered with rooftops. The valuable scenario development was made by changing the

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8km pedestrian road by permeable pavements and by Bio – Retention Cells so that minimize runoff generated at source before entering to drainage system and finally reduce node flooding and peaks by considering the following scenarios.

**Scenario’s developed:**

**Scenario 1: Changing 2% of total catchment area to Permeable pavement**

Considering the study area (Bole Gerji): 2.942 km Megenagna to Bole left side, Nexus hotel to Ahadu Building = 0.722 km two side, Ahadu building to Blue Building = 0.83km one side, Ethio-parent school to Anbesa garage = 0.698 km two sides and Imperial hotel to Blue Building = 0.698 km two sides give total length of paved pedestrian 8 km (its layout shown in Figure 4.14 below ) and Considering 5m width recommended by AACRA manual, total area of 3.6 ha paved pedestrian by permeable pavement reduces total peak runoff by 12 % and reduces total node flooding / total flood volume/ by 11% during peak flow situation.

**Table 4.10: Node flooding occurring in 2019 after LID (Permeable Pavement) applied**

Topic: <input type="text" value="Node Flooding"/> Click a column header to sort the column.						
Node	Hours Flooded	Maximum Rate CMS	Day of Maximum Flooding	Hour of Maximum Flooding	Total Flood Volume 10 <sup>6</sup> ltr	Maximum Poned Depth Meters
J1	1.81	0.225	0	04:00	1.046	0.000
J2	2.38	0.143	0	03:47	1.076	0.000
J3	3.27	1.203	0	05:00	11.131	0.000
J5	4.91	2.262	0	04:19	27.664	0.000
J6	0.30	0.093	0	04:02	0.059	0.000
J11	0.12	0.109	0	04:01	0.019	0.000
J12	2.83	0.759	0	03:58	3.805	0.000
J17	3.10	1.177	0	04:00	5.903	0.000
J18	3.91	1.052	0	02:58	11.149	0.000
J23	0.81	0.570	0	04:00	1.163	0.000
J24	3.93	1.443	0	04:22	11.160	0.000

**Scenario 2: Changing 0.85% of total catchment area to Bio – Retention Cells**

Covering the same place (paved pedestrian) as in scenario 1 by Bio – Retention Cells (0.85% of the total catchment area) and remaining 1.25% of the total area as side walk pedestrian road, reduces total peak runoff by 5% and total node flooding / total flood volume/ by 4% during peak flow situation.

**Table 4.11: Node flooding occurring in 2019 after LID (Bio – Retention Cells) applied**

Topic: <span style="border: 1px solid black; padding: 2px;">Node Flooding</span> <span style="float: right;">Click a column header to sort the column.</span>						
Node	Hours Flooded	Maximum Rate CMS	Day of Maximum Flooding	Hour of Maximum Flooding	Total Flood Volume 10 <sup>6</sup> ltr	Maximum Poned Depth Meters
J1	1.88	0.237	0	04:00	1.140	0.000
J2	2.41	0.143	0	03:47	1.100	0.000
J3	3.29	1.217	0	05:00	11.310	0.000
J5	4.94	2.286	0	04:24	28.170	0.000
J6	0.49	0.131	0	04:00	0.147	0.000
J11	0.27	0.116	0	03:55	0.055	0.000
J12	2.90	0.798	0	03:52	4.306	0.000
J17	3.31	1.286	0	04:00	6.894	0.000
J18	3.98	1.064	0	02:48	11.609	0.000
J23	0.94	0.671	0	04:01	1.543	0.000
J24	4.04	1.461	0	04:27	11.912	0.000

#### 4.6.1 Low Impact Development Controls result summary

As show in the table 4.10 below, the two LIDs are result in significant variation in both node flooding and peak runoff reduction. Scenario 1 almost reduces two fold of scenario 2 both in runoff peak and node flooding. Comparing the above two LIDs In terms of cost of constructions: Scenario 1 is better than scenario 2. Because Bio Retention Cells requires with minimum requirement of depression with vegetation grown in engineered soil of depth 50cm placed over 40cm gravel with drain system attached to it (Seema, 2014), whereas permeable pavement is a paved pervious surface underlain by a gravel storage zone. The pavement consists of less fine aggregates than traditional concrete or asphalt and a pavement thickness of 12.7cm with a gravel storage depth (Seema, 2014). Again comparing to LIDs In terms of serviceability, permeable pavement are used as parking lots, low-traffic roads, sidewalks, and driveways. Due its nature of construction, Bio – Retention Cells requires regular follow up and frequent maintenance. Based on the above three comparison, scenario 1 was more preferable than Scenario 2 for this study.

Table 4.12: Low Impact Development Controls result summary

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Scenarios	Area coverage (ha)	Runoff from all sub catchments (m <sup>3</sup> /sec)	Node flooding from critical nodes (10 <sup>6</sup> ltr)	% of Runoff Reduction	% of Node flooding Reduction
Base Scenario	188	12.9	81.107		
Scenario 1	3.6	11.5	72.535	12	11
Scenario 2	3.6	12.34	77.916	5	4

### 4.6.2 Area where LID Controls applied

Both LIDs located in the same place with the same length but with different width size. For both cases the paved pedestrian mentioned above in Scenario 1 was used. Their corresponding area coverage indicated here in Table 4.13.

Table 4.13: Area Proposed for Construction of Permeable Pavement and Bio – Retention Cell

LID Type			
Bio – Retention Cell		Permeable Pavement	
Code of Area	Area Coverage, m <sup>2</sup>	Code of Area	Area Coverage, m <sup>2</sup>
BRC01	988	PP1	2470
BRC02	988	PP2	2470
BRC03	988	PP3	2470
BRC04	770	PP4	1925
BRC05	674	PP5	1685
BRC06	860	PP6	1040
BRC07	372	PP7	930
BRC08	860	PP8	930
BRC09	800	PP9	1630
BRC10	500	PP10	2150
BRC11	800	PP11	2150
BRC12	844	PP12	930
BRC13	372	PP13	930
BRC14	652	PP14	1630
BRC15	600	PP15	1500
BRC16	500	PP16	1250
BRC17	300	PP17	750
BRC18	800	PP18	2000
BRC19	800	PP19	2000
BRC20	600	PP20	1500
BRC21	500	PP21	1250
BRC22	300	PP22	750

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BRC23	616	PP23	1540
BRC24	844	PP24	2110

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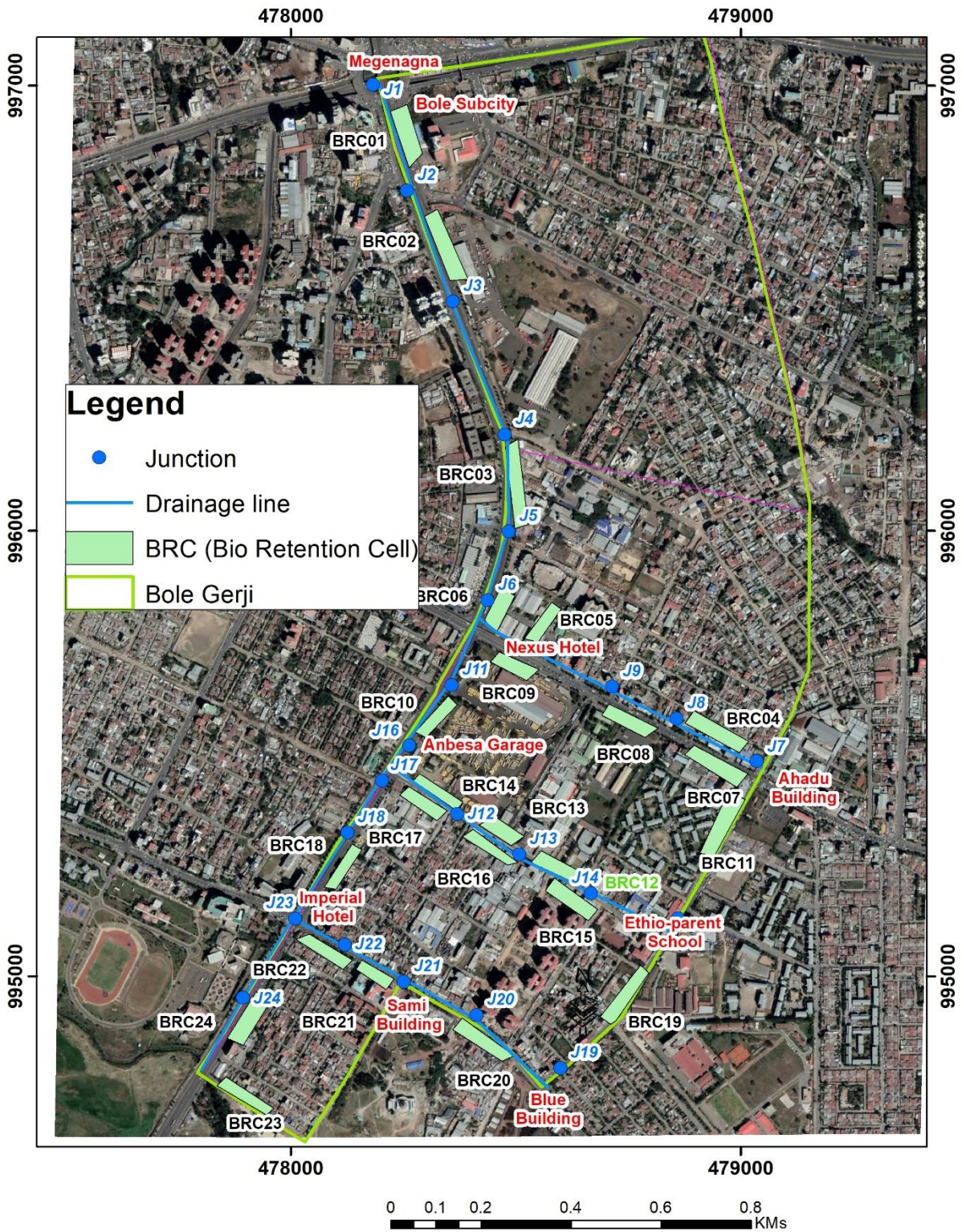


Figure 4.14: Layout of proposed Bio – Retention Cells and Permeable Pavement

## CHAPTER FIVE

### 5 CONCLUSION AND RECOMMENDATION

#### 5.1 CONCLUSION

In this study Impact of land use change on urban drainage system was analyzed and Sustainable Storm Water management concept with help of Low Impact Development Controls has been applied to reduce excess storm water from urban drainage around Bole Gerji area. SWMM which is the robust and powerful tool for analyzing the effect of urban flood water control & management in conjunction with other software's like ArcGIS, Google Earth Pro, Excel were used to analyze the quantitative data. As part of Ring Road, Megenagna - Bole Ring Road part was evaluated. It was found that the road segment has been facing problems of overflowing and flooding which was mainly caused by the increment of built up area. The results of this study showed that the drainage system has been hindered to function properly in rainy season because of the challenges that have been faced over years. After analyzing the results, the following conclusions were drawn:

- ❖ Mapping and classifying land use / land cover is very important in drainage study. To detect the impact of imperviousness in drainage system, Land use / land cover of the study area was analyzed in past (1995) and present (2019). It was observed that the Built up area coverage in past was around 14% and it increased to 94% in recent year. This is due to urbanization and lack of uncontrolled development. Due to this dramatic change in land use, the recent runoff becomes three fold higher than the flood of the Ring road during construction.
- ❖ No Substantial rainfall intensity variation was observed in the study area. The result shows that the rainfall intensity for different time of concentration was similar in 1995 and 2019, except some increase on peak values was observed for shorter time of concentration value in recent curve.
- ❖ Capacities of the drainage system cannot handle the current runoff that flows over the area. Due to dramatic change in land use, runoff generated from all sub catchments increase from 4.27 m<sup>3</sup>/sec to 12.94 m<sup>3</sup>/sec, almost three folds. Most of the conduits / Pipes under surcharge situation and five Nodes / Joints in the main road (Megenagna – Bole) were flooding or overflowing. These five nodes, J3, J5, J17, J18 and J24, with total volume of 11.4\*10<sup>6</sup> liters, 28.5\*10<sup>6</sup> liters, 7.6\*10<sup>6</sup> liters, 12.1\*10<sup>6</sup> liters and 12.4\*10<sup>6</sup> liters, respectively.
- ❖ Based on detail site visit and interview of Addis Ababa residents near the ring road, the amount of excess flood in the area, especially around Imperial roundabout during rainy season was higher that

node flooding obtained in this thesis. It was observed that most of the storm drain inlet type on the ring road is grate inlet that are blocked with debris and not functioning properly.

- ❖ The node flooding or flooding in the study area can be minimized by using selected Low Impact Development controls. Two scenarios were developed: the first one was changing 2% of the total catchment area (Paved pedestrian) to permeable pavement, which means replacing total area of 3.6 ha paved pedestrian by permeable pavement. This reduces total runoff by 12 % and total node flooding by 11%. The second scenario developed was changing 0.85% of total catchment area (Paved pedestrian) to Bio – Retention Cells, which is changing total area of 1.6ha paved pedestrian by Bio – Retention Cells. This reduces total runoff by 5% and total node flooding by 4%. Based on the three comparison points considered in the analysis part, scenario 1 was more preferable than Scenario 2 for this study.

Generally, the performance of the storm water drainage system was not satisfactory. Therefore, the capacity of the drainage system needs to be improved to give the best services. In addition to providing selected LIDs, it needs a serious of regular maintenance and clearing service.

## **5.2 RECOMMANDATION**

- In order to handle the increased runoff due to the increment of built up area, the existing storm water management system needs to be improved or modified by proposed LID Controls or other appropriate measures.
- Increasing Pipe diameters can be consider as one the solution to reduces excess runoff in urban drainage system, but in case of Addis Ababa Ring Road the option could not be recommended. Since the area is in the economically active zone of the city, any upgrading process interrupts the transportation system of the area.
- Since most of the storm drain inlets along Ring Road are not functioning properly because of different reasons such as inlet type, damage and lack of routine maintenance, improper usage of openings, and others. This could be reduce by cleaning and by providing efficient storm drain inlets.
- Regular maintenance and frequent cleaning of drainage lines should be on regular basis for sustainable stormwater drainage system.

**Evaluating the Impact of Land Use Change on Urban Drainage System and Proposed Low Impact Development measures in Addis Ababa, Ethiopia: Case Study of Megegnagna - Bole Ring Road**

- Create awareness within the community to use the drainage systems in a way that they could serve for long. The community should properly handle solid and liquid wastes, because dumping the solid waste on manholes and side inlets were commonly observed during site visit.
- Addis Ababa City Road Authority must plan and prepare to implement Low Impact Development practices in the whole city. LID is the new and innovative solution for urban drainage management which can reduce the load of pollutants entering receiving water bodies and contribute to restoring the health and vitality of the watershed of the city.

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## Appendixes

### A - Tables

Table A. 1: Node input data

Node Name	Node Type	Maximum Depth (m)	Invert Elevation (m)
J1	Junction	1.3	2356.2
J2	Junction	1.3	2354.9
J3	Junction	1.3	2353.8
J4	Junction	1.3	2351.9
J5	Junction	1.3	2349.6
J6	Junction	1.3	2348.4
J7	Junction	1.3	2349.4
J8	Junction	1.3	2348.0
J9	Junction	1.3	2346.0
J10	Junction	1.3	2344.2
J11	Junction	1.3	2342.6
J12	Junction	1.3	2339.4
J13	Junction	1.3	2341.0
J14	Junction	1.3	2336.1
J15	Junction	1.3	2331.7
J16	Junction	1.3	2326.6
J17	Junction	1.3	2322.8
J18	Junction	1.3	2319.5
J19	Junction	1.3	2337.0
J20	Junction	1.3	2332.1
J21	Junction	1.3	2327.7
J22	Junction	1.3	2322.6
J23	Junction	1.3	2318.5
J24	Junction	1.3	2312.1
Outlet	Outfall		2308.6

Table A. 2: Conduit input data

Conduit Name	Conduit Shape	Maximum Depth (m)	Conduit Length (m)	Roughness	Inlet Offset (m)	Outlet Offset (m)
C1	Circular	1.0	230	0.014	0.2	0.2
C2	Circular	1.0	230	0.014	0.2	0.2
C3	Circular	1.0	230	0.014	0.2	0.2

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C4	Circular	1.0	230	0.014	0.2	0.2
C5	Circular	1.0	230	0.014	0.2	0.2
C6	Circular	1.0	230	0.014	0.2	0.2
C7	Circular	1.0	180	0.014	0.2	0.2
C9	Circular	1.0	180	0.014	0.2	0.2
C10	Circular	1.0	180	0.014	0.2	0.2
C11	Circular	1.0	230	0.014	0.2	0.2
C12	Circular	1.0	230	0.014	0.2	0.2
C13	Circular	1.0	180	0.014	0.2	0.2
C14	Circular	1.0	180	0.014	0.2	0.2
C15	Circular	1.0	180	0.014	0.2	0.2
C16	Circular	1.0	180	0.014	0.2	0.2
C17	Circular	1.0	230	0.014	0.2	0.2
C18	Circular	1.0	230	0.014	0.2	0.2
C19	Circular	1.0	180	0.014	0.2	0.2
C20	Circular	1.0	180	0.014	0.2	0.2
C21	Circular	1.0	180	0.014	0.2	0.2
C22	Circular	1.0	180	0.014	0.2	0.2
C23	Circular	1.0	230	0.014	0.2	0.2
C24	Circular	1.0	230	0.014	0.2	0.2

Table A. 3: Sample rainfall input data format

<b>T10 – 45min</b>		
<b>Date</b>	<b>Time (H : M)</b>	<b>Value (mm)</b>
	0.0	0
	1.0	6
	2.0	8
	3.0	12
	4.0	8
	5.0	6
	6.0	0

## B – Figures

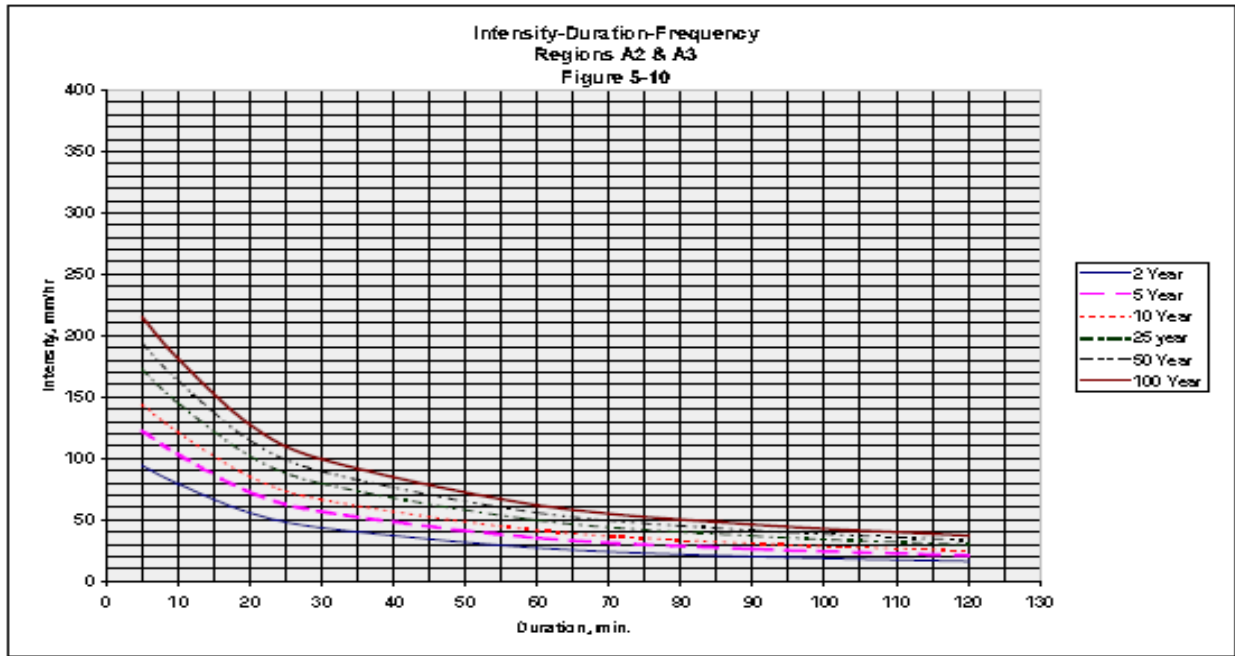


Figure B.1: IDF Curve of Region 2 developed by ERA 2002

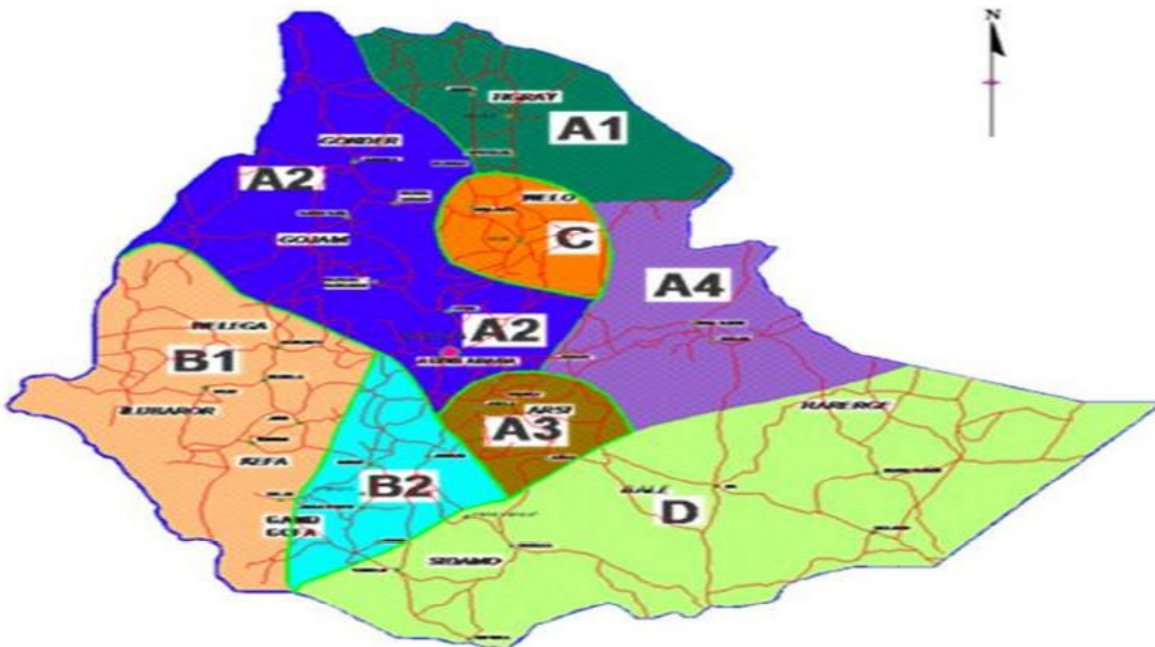


Figure B. 2: Rainfall Region of Addis Ababa, A2







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Bole	9.03	38.75	2354	PRECIP	2015	8	9:00	8.7	3	0.7	2.2	7	18	14	11	7	8.6	17	7	12	3	12	5	3.8	11	9	5.3	4.8	4	17	5.5	18	13	1.4	6.8	0	1.5	5
Bole	9.03	38.75	2354	PRECIP	2015	9	9:00	2.5	3	2.5	8.6	0.4	0	1.5	2.8	3	0	1	6	6.3	2	3	0	7.6	2	2	6.9	0.6	0	12	6.3	1.6	12	1.3	0	5	14	
Bole	9.03	38.75	2354	PRECIP	2015	10	9:00	8.4	0	2	3.2	5	0	0	0	0	5.25	1	2	0	0	0	0	0	0	0	0	0.3	0	0	0	0	0	0	0	0	0	0
Bole	9.03	38.75	2354	PRECIP	2015	11	9:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.6	0	0	0.5	0.6	0	0	
Bole	9.03	38.75	2354	PRECIP	2015	12	9:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Bole	9.03	38.75	2354	PRECIP	2016	1	9:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Bole	9.03	38.75	2354	PRECIP	2016	2	9:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Bole	9.03	38.75	2354	PRECIP	2016	3	9:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	9	0.2	0.2	0	9.1	4.5	1.3	0	11	4	5	0	0
Bole	9.03	38.75	2354	PRECIP	2016	4	09:00	1.4	0	1.8	0	0	0	0	0	0	0	0	2	17	8	12	0	0	3	0	4.9	0	0	0.6	0	9.5	12	20	0	18	26	
Bole	9.03	38.75	2354	PRECIP	2016	5	09:00	14	14	0	0	4.7	8.4	5.4	0	2	13.2	33	0	0	0	0	6.1	2	8	0	0	0	1.3	3.4	6	2.8	0	0	0	0	0	
Bole	9.03	38.75	2354	PRECIP	2016	6	09:00	0	0	0	0	0	0.4	0	0	19.3	0	0	1	2	4	0	9	0	2	4	10	7	4.2	8.1	5.2	4.7	13	2	6	2.2		
Bole	9.03	38.75	2354	PRECIP	2016	7	09:00	5.2	1	5	9	0	7.5	1.7	1.4	0	5.4	3	4	16	5	0	9	8	0	3	20	9.6	5	0	7.2	14	6.6	6.7	4.4	0	3.6	2
Bole	9.03	38.75	2354	PRECIP	2016	8	09:00	14	4	1.3	2.5	8.6	14	4.8	12	2	8.2	14	3	0	6	0	5	4.6	15	11	1.7	4.6	3	32	2	8.7	22	0	14	0	0	9
Bole	9.03	38.75	2354	PRECIP	2016	9	09:00	0	5	5	0	0	0	1.6	0	6	0	2	11	1.2	0	0	0	15	4	5	0	0	0	12	13	2.2	12	0	0	10	28	
Bole	9.03	38.75	2354	PRECIP	2016	10	09:00	2.2	0	0	0	0	0	0	0	0	10.5	1	3	0	0	0	0	0	0	0	0	0.5	0	0	0	0	0	0	0	0	0	0
Bole	9.03	38.75	2354	PRECIP	2016	11	09:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Bole	9.03	38.75	2354	PRECIP	2016	12	09:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Bole	9.03	38.75	2354	PRECIP	2017	1	09:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Bole	9.03	38.75	2354	PRECIP	2017	2	09:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Bole	9.03	38.75	2354	PRECIP	2017	3	09:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	17	0.4	0	0	0	8.9	0.6	0	22	7.2	10	0	0
Bole	9.03	38.75	2354	PRECIP	2017	4	09:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	17	0	7	0	9		
Bole	9.03	38.75	2354	PRECIP	2017	5	09:00	3.8	13	0	0	0	0	0	0	0	0	2	2.8	49	15	22	1.4	6	0	4.9	0	3	0	0	2.3	6.8	16	26	0.8	0		
Bole	9.03	38.75	2354	PRECIP	2017	6	09:00	0	0	0	1.2	1.2	0	0	0	1.2	0	4	12	0	0	0	2.2	2	0	0	0.8	0	0	1.3	4.8	4.4	1	0.3	4	3		
Bole	9.03	38.75	2354	PRECIP	2017	7	09:00	2.6	23	1.6	8.2	6.3	29	20	2.4	0	6.7	5	5	6.2	3	6	1	15	2	15	4.6	4.4	5	10	3.5	13	5.7	4.5	2	5	0.2	0
Bole	9.03	38.75	2354	PRECIP	2017	8	09:00	4	0	9.2	0	7	23	4.8	17	7	5.5	1	22	0.3	4	6	0	23	0	6	5	9.7	13	15	0	3.3	8.5	12	8.4	6	13	10
Bole	9.03	38.75	2354	PRECIP	2017	9	09:00	0	2	1.2	18	15	13	14	10	17	8	4	0	13	48	9	0	0.6	24	13	9.7	10	1	1.5	0	0	0	0	0	5	25	

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Table A.5: Cumulative of Annual Rainfall of Bole station and Cumulative of Sum of Annual Rainfall of Base stations

Year	Annual Rainfall of Bole station, mm	Annual Rainfall of Kotebe station, mm	Annual Rainfall of Intoto station, mm	Annual Rainfall of Observatory station, mm	Cumulative of Annual Rainfall of Bole station, mm	Sum of Annual Rainfall of Base stations, mm	Cumulative of Sum of Annual Rainfall of Base stations, mm
2007	741.029	647.60	810.30	760.39	741.03	2218.29	2218.29
2008	592.720	603.90	879.50	687.30	1333.75	2170.70	4388.99
2009	576.920	486.43	927.64	657.50	1910.67	2071.56	6460.55
2010	653.290	739.60	957.20	838.40	2563.96	2535.20	8995.75
2011	651.715	725.50	753.11	671.00	3215.67	2149.61	11145.36
2012	620.809	603.97	1130.86	614.39	3836.48	2349.22	13494.58
2013	654.470	629.06	1065.20	732.40	4490.95	2426.66	15921.23
2014	529.180	390.98	702.00	676.80	5020.13	1769.78	17691.01
2015	792.135	594.94	1095.54	662.89	5812.27	2353.37	20044.38
2016	788.003	244.70	645.60	817.20	6600.27	1707.50	21751.88
2017	962.379	609.76	757.22	784.95	7562.65	2151.92	23903.80

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Table A.6: Data for Intensity Duration Frequency Curve development for Bole Airport Station

IDF Curve Developed from Addis Ababa Bole Meteorological Station														
Summary of the three Probability Distribution for Addis Ababa Bole Meteorological Station														
Return Period T(yr)	Probability (P)	Mean(X)	SD(S)	Mean(Yt)	SD(Sy)	Log Normal			Log Pearson Type III			EVI (Gumbel)		ERA X <sub>T</sub> for A2
						K <sub>T</sub>	Y <sub>T</sub>	X <sub>T</sub>	K <sub>T</sub>	Y <sub>T</sub>	X <sub>T</sub>	K <sub>T</sub>	X <sub>T</sub>	
500	0.002	51.41	13.32	1.70	0.11	2.88	2.02	103.91	3.54	2.09	122.78	4.40	109.98	122.27
200	0.005	51.41	13.32	1.70	0.11	2.58	1.98	96.25	3.08	2.04	109.22	3.68	100.44	110.91
100	0.010	51.41	13.32	1.70	0.11	2.33	1.96	90.35	2.71	2.00	99.55	3.14	93.21	102.45
50	0.020	51.41	13.32	1.70	0.11	2.05	1.93	84.33	2.32	1.96	90.31	2.59	85.96	94.07
25	0.040	51.41	13.32	1.70	0.11	1.75	1.89	78.09	1.92	1.91	81.43	2.04	78.65	85.70
10	0.100	51.41	13.32	1.70	0.11	1.28	1.84	69.34	1.32	1.85	70.03	1.31	68.80	74.45
5	0.200	51.41	13.32	1.70	0.11	0.84	1.79	62.02	0.80	1.79	61.41	0.72	61.00	65.52
2	0.500	51.41	13.32	1.70	0.11	0.00	1.70	50.12	-0.09	1.69	49.02	-0.16	49.22	51.92

$$RR_t = \frac{t}{24} \left[ \frac{(b+24)^n}{(b+t)^n} \right]$$

b = 0.3  
n = 0.9

RR<sub>t</sub> = Rainfall Ratio R<sub>t</sub>:R<sub>24</sub>  
 R<sub>t</sub> = Rainfall in a given duration 't' in hours  
 R<sub>24</sub> = Rainfall in 24 hours  
 t = time in hours

Based on studies of a large number of gauges in East Africa,

b = 0.3  
n = 0.9

b = 0.3  
n = 0.9 (0.78 ≤ n ≤ 1.09)

Duration (Minutes)	Duration (hr)	Rainfall Ratio(RR <sub>t</sub> )	Depth for given Return periods(mm)						Intensity for given return periods(mm/hr)					
			2 years	5 years	10 years	25 years	50 years	100 years	2 years	5 years	10 years	25 years	50 years	100 years
5	0.08	0.15	7.15	8.87	10.00	11.43	12.49	13.55	85.85	106.40	120.00	137.18	149.93	162.59
10	0.17	0.24	11.99	14.86	16.75	19.15	20.93	22.70	71.92	89.13	100.53	114.93	125.61	136.21
20	0.33	0.37	18.21	22.57	25.46	29.10	31.81	34.49	54.64	67.71	76.37	87.31	95.42	103.48
30	0.50	0.45	22.14	27.44	30.94	35.38	38.66	41.93	44.28	54.87	61.89	70.75	77.33	83.85
45	0.75	0.53	26.00	32.22	36.34	41.54	45.41	49.24	34.67	42.96	48.45	55.39	60.54	65.65
60	1.00	0.58	28.60	35.45	39.98	45.71	49.95	54.17	28.60	35.45	39.98	45.71	49.95	54.17
75	1.25	0.62	30.52	37.82	42.66	48.77	53.30	57.80	24.42	30.26	34.13	39.01	42.64	46.24
90	1.50	0.65	32.01	39.67	44.74	51.15	55.91	60.62	21.34	26.45	29.83	34.10	37.27	40.42
120	2.00	0.70	34.23	42.43	47.85	54.70	59.78	64.83	17.12	21.21	23.92	27.35	29.89	32.42
150	2.50	0.73	35.85	44.43	50.11	57.28	62.61	67.89	14.34	17.77	20.04	22.91	25.04	27.16
180	3.00	0.75	37.11	45.98	51.86	59.29	64.80	70.27	12.37	15.33	17.29	19.76	21.60	23.42
210	3.50	0.77	38.13	47.25	53.29	60.92	66.59	72.21	10.89	13.50	15.23	17.41	19.02	20.63
240	4	0.79	38.99	48.32	54.49	62.30	68.09	73.83	9.75	12.08	13.62	15.57	17.02	18.46
270	4.5	0.81	39.73	49.23	55.53	63.48	69.38	75.23	8.83	10.94	12.34	14.11	15.42	16.72
300	5	0.82	40.37	50.03	56.43	64.51	70.51	76.46	8.07	10.01	11.29	12.90	14.10	15.29
330	5.5	0.83	40.95	50.75	57.24	65.43	71.51	77.55	7.45	9.23	10.41	11.90	13.00	14.10
360	6	0.84	41.47	51.39	57.96	66.26	72.42	78.53	6.91	8.57	9.66	11.04	12.07	13.09

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Table A.7: Peak Runoff computation using Rational Method

Pipe n value = 0.014																	
Pipe Name	Lengt h (m)	U/S Elev. (m)	D/S Elev. (m)	Slope	U/S Junc.(Ca tchment)	D/S Junc.	Sub - Catchmnet	Area (km <sup>2</sup> )	C	C*A	Time to Inlet / U/S Td (min)	Flow Route	U/S Flow Time (min)	Tc (min)	Td (min)	I (mm/hr)	Catchment Discharge, Qp (m <sup>3</sup> /sec)
C1	230	2356	2355	0.006	J1	J2	S1	0.34	0.5	0.17	59	Overland	0	59	59	40	1.890
C2	230	2355	2354	0.005	J2	J3											
C3	230	2354	2352	0.008	J3	J4	S2	0.34	0.5	0.17	59		4	63	63	39	1.843
C4	230	2352	2350	0.01	J4	J5											
C5	230	2350	2348	0.005	J5	J6	S3	0.34	0.5	0.15	59		6.5	65.5	65.5	38	1.616
C6	230	2348	2343	0.025	J6	J7	S5	0.07	0.5	0.04	59		8.5	67.5	67.5	39	0.379
C7	180	2349	2348	0.008	J7	J8	S4	0.08	0.6	0.05		Overland			54	45	0.600
C8	180	2348	2346	0.011	J8	J9											
C9	180	2346	2344	0.01	J9	J10											
C10	180	2344	2343	0.009	J10	J11											
C11	230	2342	2339	0.013	J11	J12											
C12	230	2339	2323	0.072	J12	J17	S8	0.14	0.6	0.08	59		10.5	69.5	69.5	38	0.813
C13	180	2341	2336	0.027	J13	J14	S6	0.08	0.8	0.06		Overland			51	46	0.818
C14	180	2336	2332	0.024	J14	J15											
C15	180	2332	2327	0.028	J15	J16	S7	0.08	0.8	0.06	51		1.5	52.5	52.5	45	0.801
C16	180	2327	2323	0.021	J16	J17											
C17	230	2323	2320	0.014	J17	J18											
C18	230	2320	2319	0.004	J18	J23	S11	0.09	0.6	0.05	59		12	71	71	36.5	0.548
C19	180	2337	2332	0.027	J19	J20	S9	0.12	0.8	0.1		Overland			50	46	1.228
C20	180	2332	2328	0.024	J20	J21											
C21	180	2328	2323	0.028	J21	J22	S10	0.1	0.5	0.05	50		1.5	51.5	51.5	46	0.639
C22	180	2323	2319	0.023	J22	J23											
C23	230	2319	2312	0.028	J23	J24											
C24	230	2312	2309	0.015	J24	Outlet	S12	0.13	0.5	0.07	59		15	74	74	36	0.651
																	<b>11.828</b>