ADDIS ABABA UNIVERSITY

INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

PERFORMANCE ASSESSMENT OF STORM WATER DRAINAGE SYSTEMS

(Case study of Debere Berehan Town)

A thesis submitted to the school of Civil and Environmental Engineering of Addis Ababa University in partial fulfillment of the Degree of Masters of Science in Hydraulic Engineering

By

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Advisor

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Addis Ababa, Ethiopia

Dec.10, 2018

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Declaration

I declare that the thesis entitled "**Performance assessment of storm water drainage systems (Case study of Debere Berehan Town)**" is my own work under close direction and instruction of my advisor and all sources of materials used for this thesis have been duly acknowledged. This thesis submitted in partial fulfillment of the requirements for MSc degree in Civil Engineering (major in Hydraulic Engineering) at Addis Ababa University.

Acknowledgement

First, I would like to thank the almighty God for his unspeakable gift, help and protection during my work.

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Abstract

A drainage system degrades by multiple factors and its performance reduces in time. Since such infrastructure play a key role in preventing urban floods, their performance should be monitored and quantified. Debere Berehan town is one of the rapidly expanding and selected as factories center now a day in Ethiopia. Unfortunately, street flooding and over topping drainage system problems are occurring at the rainy season in a town. Because of flooding the area are becomes water soaked soil which difficult to move easily for Transport. The objective of the study is to assess the Performance of drainage systems of Debere Berhan Town. The study employed both primary and secondary data collection. To achieve the specific objective SWMM5 Model and LID control used as method. Model calibration and validation were done and The performance of SWMM5 was carried out using Coefficient of Determination (R^2) ,The Nash-Sutcliffe coefficient (NSE) and Relative Error (RE).According to simulation almost all of the modeled drainage systems are flooded for each event scenario rain. For T-25 event 35% of closed rectangular channel and 32.5% of Open rectangular channel of the Study area were flooded. More over the total average flow1.64 m^3/s and total volume to outfall $87.75*10^3$ m^3 were occurred from all 17 sub catchments. After LID control provided the discharge values were lesser than that before LID provided. In comparison to the current conditions, bio-retension reduced total outfall volume by 46.2% and average peak flow by 25.4%, the infiltration trench scenario as well reducing total volume to outfall 34.4% and average peak flow by 23.7% as compared to current conditions. In general, the LID results shows bio-retension are best alternative to minimize the peak flow rate and total volume of the area. And applied to the selected area 3.6 ha at S8, S9, S10, S11, S13, S14, S15, S16 and S17 catchments were recommended.

Keywords: urban drainage, SWMM5, model calibration and verification and LID control

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Abbreviations

AAIT---------------------Addis Ababa Institute of Technology

AASHTO----------------American Association of State Highway and Transportation Officials

ASTM--------------------American Society for Testing and Material

A2-------------------------Rainfall Region Classification of Ethiopian Road Authority

DEM ---------------------Digital Elevation Model

EGL-----------------------Energy Grade Line

EP-------------------------Environmental protection

EPA-----------------------Environmental Protection Agency

ERA-----------------------Ethiopian Road Authority

FFG-----------------------Flash Flood Guidance

FHWA--------------------Federal Highway administration

GIS------------------------Geographical Information System

HGL-----------------------Hydraulic Grade Line

HSPF----------------------Hydrologic Simulation Program-Fortran

LID------------------------Low Impact Development

MOUSE ------------------**M**odeling **of U**rban **Se**wers

RAFTS--------------------Runoff Analysis and Flow Training Simulation

SCS ----------------------Soil Conservation Service

STORM-------------------**Sto**rm water **R**unoff **M**odel

SWAT -------------------Soil and water Assessment Tool

SWMM ------------------Storm water management model

1. INTRODUCTION

1.1. General background

Flood is a hydrologic phenomenon that is characterized by both precipitation and soil water contributions (Simono vic et al., 2001). Flooding in urbanized areas has become a very important issue around the world (Barreto Cordero, 2012). Cities are growing fast and large amounts of impervious surfaces replace the natural landscape as a result of urban development. Impervious surfaces can have an effect on local streams and flooding characteristics (Barreto Cordero, 2012).

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 Cordero, 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 Cordero, 2012).

Hydrological and hydrodynamic modeling plays a key role for the hydraulic, structural and environmental assessment. Sustainable approaches, oriented to the control of runoff volumes from the beginning of the rainfall are preferable than methodologies based on conveyance. These sustainable approaches are also oriented to keep environment, social and economic values in balance. (Barreto Cordero, 2012).Since such infrastructure play a key role in preventing urban floods, their performance should be assessed after being quantified. (Negin Binesh,2016).

The main goal of this study is to investigate the ability of the existing urban drainage network in Debere Berhan town in operating satisfactorily in collecting and conveying runoff from typical rainfall, without inundation. The appropriate performance of urban drainage systems plays a key role in preventing urban flooding. There are no studies done before for performance assessment of the storm water drainage system in Debere Berhan town. Many Studies done on drainage system are reviewed to learn a gap regarding to the storm water drainage system problem and the sustainable measurement. Almost all the research was done on the performance assessment of storm drainage system by using the rational method, SCS Method and the model SWMM5 to determine the runoff occurring at the area for comparison. But under this study area was modeled with SWMM5 and the sustainable measure to minimize the run off occurrence at the Debre Berhan town was done using Low Impact Development (LID) that compatible with SWMM5 model.

1.2. Statement of the problem

Lack of urban Storm water drainage (USWD) management represent one of the most common sources of compliant from the residents in many urban centers of Ethiopia, and this problem gets worse and worse with the rate of urbanization. In addition to increased densification and impermeability of the urban landscape, the planning as well as implementation of storm water protecting structures is insufficient.

The road that has no proper drainage systems which results in gully erosion may occur anywhere in the world. The problem is particularly several in developing countries like Ethiopia.

Debere Berhan town is one of the expanding rapidly and selected as factories center now a day in Ethiopia. Unfortunately, there is a problem of drainage such as overtopping and Flooding of the area during intensive rainfall and this may be due to either small drainage dimension, an incremental of rainfall, the increasing of pavements in the overall catchment or urbanization in a town, Due to improper management of the sewage system like, using drainage system as solid waste removal. Because of no mentainance some of the top element of the manhole has been broken.it may increase flood risk due to the entrance of the sewerage to the drainage system and also there are secondary road that not provided with drainage system.

The problems are more faced from Debre Berhan bus station to Beressa River, megenteya to Beressa and Debre Berhan Tena science to Beressa river of Debre Berhan Town.

1. 3 Objectives of the study

1.3.1 General objective

The main objective of this work was simulate the peak flow rate and total volume of 143 ha urban area and Assessing the performance of storm water drainage system in Debere Berehan town.

1.3.2. Specific objectives

The following are the specific objectives of the study:

I. Simulating the peak flow rate and total volume of storm water drainage system of Debere berehan town, using SWMM5 model

II. Reanalyzing the current drainage system performance regarding storm water excess management.

III. Providing sustainable measures to minimize the runoff occurrences in the town using LID (low Impact development) with SWMM5.

1.4. Scope and limitation of the thesis

This thesis includes the modeling of the drainage system using SWMM5, calibrate and validate the model for the study area through observed data. These software has a limitation on the large catchment delineation, and for the large catchment it is very tedious and the aim of this thesis was to model the drainage for Debre berehan town but due to the limitation of software the model only covers around 6 km drainage length .modeling was done from Debre Berhan bus station to Beressa, megenteya to Beressa and Debre Berhan Tena science to Beressa where totally about seventeen sub catchment covered.. In this seventeen 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 ERA, there was no design document founded rather than as built and delineation of the town using DEM 30 is impossible because it indicated by dote rather than polygon.

1.5. Outline of the thesis

This report is divided into 5 Chapters. Chapter one contains the introduction, statement of the problem and objective of the Research, Chapter two contains Literature review information regarding storm water management and flooding, description of the current drainage system and description of different models. Chapter three presents a general description of the study area, modeling of drainage utility using SWMM5 model and a description of SWMM5 software the base tool of this study. In Chapter four the obtained results from the simulations are presented and discussed, both the runoff and the network results. The results obtained by using SWMM5 model reduced by Appling of the Low Impact Development (LID) control and finally chapter 5 presents, the conclusion and recommendation about the obtained results.

2. LITERATURE REVIEW

2.1. Storm water and urban storm water drainage models

2.1.1. Storm water and Flooding

Storm water is the water draining off a site from the rain that falls on the roof and land, and everything it carries with it. The soil, organic matter, litter, fertilizers from gardens and oil residues from driveways it carries can pollute downstream waterways. Rainwater refers only to the rain that falls on the roof, which is usually cleaner. However, storm water can be a valuable resource. Reusing storm water can save potable water and reduce downstream environmental impacts. . (Hatt, B, et al 2004).

In urban areas storm water is generated by rain runoff from roofs, roads, driveways, footpaths and other impervious or hard surfaces. In Australia the storm water system is separate from the sewer system. Unlike Sewage, storm water is generally not treated before being discharged to waterways and the sea. Poorly managed storm water can cause problems on and off site through erosion and the transportation of nutrients, chemical pollutants, litter and sediments to waterways. Well-managed Storm water can replace imported water for uses where high quality water is not required, such as garden watering. (Hatt, B, et al 2004).

In Australia the storm water system is separate from the sewer system. Unlike Sewage, storm water is generally not treated before being discharged to waterways and the sea. Poorly managed storm water can cause problems on and off site through erosion and the transportation of nutrients, chemical pollutants, litter and sediments to water ways. Floods generally develop over a period of days, when there is too much rainwater to fit in the rivers and water spreads over the land next to it (the floodplain). However, they can happen very quickly when lots of heavy rain falls over a short period of time. These flashfloods occur with little or no warning and cause the biggest loss of human life than any other type of flooding. (Vent cow, 1988)

2.1.2. Causes and effects of flooding

Floods are natural phenomena which cannot be prevented (EC 23 October 2007).Flooding‟ is described as a condition where wastewater and/or surface water escapes from or cannot enter a drain or sewer system and either remains on the surface or enters buildings‟. Flooding is often thought of as a result of heavy rainfall, but floods can arise in a number of ways that are not directly related to ongoing weather events. Thus, a complete description of flooding must include processes that may have little or nothing to do with meteorological events. Flooding, by its very nature, is usually a result of both meteorological and hydrologic processes; the character of a flood is determined both by the detailed behavior of the precipitation and by the nature of situation in which the event is likely to occur (soil conditions, amount of antecedent rainfall, and so on).It is not likely that precisely detailed forecasts of flooding events will ever be possible, although it is certainly well within our capability to anticipate the possibility of most flood events.(Deswell ,1993)

Floodwater can seriously disrupt public and personal transport by cutting off roads and railway lines, as well as communication links when telephone lines are damaged. Floods disrupt normal drainage systems in cities, and sewage spills are common, which represents a serious health hazard, along with standing water and wet materials in the home. Bacteria and viruses cause disease, trigger allergic reactions, and continue to damage materials long after a flood. Floods can distribute large amounts of water and suspended sediment over vast areas, restocking valuable soil nutrients to agricultural lands. In contrast, soil can be eroded by large amounts of fast flowing water, ruining crops, destroying agricultural land, buildings and drowning farm animals.(Deswell ,1993)

2.1.3. Effects of urbanization

A direct consequence of urban development is that rainfall which was previously captured by the land now falls on impervious surfaces and is converted into surface runoff (Roesner, et al., 2001). Before a watershed is developed, pervious fallow land is able to store, infiltrate, and evapotranspiration a majority of the rainfall. After development, much of this pervious land is covered over with impervious surfaces that the rainwater cannot pass through, such as roads, sidewalks, parking lots, driveways, and rooftops. In addition, much of this impervious area is directly connected to a drainage system and the rainwater has no chance to infiltrate by passing over pervious ground; this is called Directly Connected Impervious area (DCIA). The typical storm drainage system is designed to convey runoff quickly and efficiently to the receiving stream or water body. Combined, these result in larger, more frequent peak flows with more total volume (Jones, et al., 2005; Roesner, et al., 2001; WEF & ASCE, 1998). Furthermore, since a large portion of the impervious area is directly connected to the drainage system, biological processes are also largely removed; as a result, pollutants are also conveyed quickly and untreated to the receiving water bodies.

2.1.4. Storm water drainage system

2.1.4.1. Functions of storm water drainage system

One of the drainage system's functions is to collect surface water and/or ground water and direct it away, thereby keeping the ballast bed drained. The drainage system must also protect the substructure from erosion, from becoming sodden, and from losing its load-bearing capacity and stability. Another main objective of storm sewer is to protect; Public health and safety, Environmental protection and Sustainable development. Drain and Sewer systems are provided in order to prevent spread of disease by contact with fecal and other waterborne waste, to protect drinking water sources from contamination by waterborne waste and to carry runoff and surface water away while minimizing hazards to the public. Additionally, the impact of drain and sewer systems on the receiving waters shall meet the requirements of any national or local regulations or the relevant authority. (AASHTO, 1991)

2.1.4.2. Types of storm water drainage system

The primary purpose of road drainage structure is to serve as conveyance structures preventing water from pooling on the roadway surface. Effective drainage structures prevent overland runoff from reaching the roadway, as well as drain water from the road surface. (ERA, 2002).A drainage system will include all the components needed to ensure that the substructure is properly drained, and may be formed of components such as, Open ditches, closed ditches with pipe drains, Drainage through storm water drainage pipes, Channels and culverts. (AASHTO, 1991 et al).

Provision shall be made to remove runoff from streets into drainage channels, watercourses, and pipe systems at low points and at intervals that will assure that ponding of storm water on streets does not occur for long durations. The maximum depth of storm water flow on any street shall not exceed 0.3 m, with a maximum flow velocity of 2 m/s. **(**Storm Water Design Criteria Manual for Municipal Services, May 2011).

For storms greater than the design storm of the minor drainage system (i.e. a storm event with a return period in excess of 5 years), streets could be designed to temporarily convey flow as part of the major drainage system. The flow conveyance capacity of street shall be determined using the Manning Equation, with a Manning's resistance coefficient of 0.013 (asphalt surfaces) or 0.015 (concrete surfaces).For storms up to and including the 5-year-return-period storm, the Designer must ensure that, for all roads, a travelled way of adequate width is maintained to ensure the safe passage of all vehicles in both directions. For residential streets and local collector streets, the Designer must ensure that during storms up to and including the major design storm (1.2 times the 100-year-return-period storm), the depth and spread of flow does not exceed the curb height and does not exceed the right-of-way width. **(**Storm Water Design Criteria Manual for Municipal Services, May 2011).

For major collector streets and arterial streets (emergency access routes), the Designer must ensure that during storms up to and including the major design storm (1.2 times the 100-year-return-period storm), a travelled way of adequate width is maintained to ensure the safe passage of vehicles in both directions. For designated highways within the Town of Riverview, the more stringent of the New Brunswick Department of Transportation design criteria or the Town of Riverview specifications for drainage infrastructure shall apply. **(**Storm Water Design Criteria Manual for Municipal Services, May 2011).

A storm water drainage system describing processes and components were

The main parts of an urban drainage system are:

- Property drainage,
- Street drainage (including both piped and surface flows),
- Trunk drainage (consisting of large conduits, usually open channels located on lands reserved for drainage purpose), and
- Receiving water bodies.

2.1.5. Drainage network of Debre Berhan Town

In principle provision and analysis of a city drainage system are categorized by tributary areas which finally drop out into natural drainage system. The rolling and undulating topography of the city was good opportunity for good urban drainage system. However, there are no proper drainage channels as well as well-designed drainage systems in most part of the city. This is mainly attributed due to the poor road network system of the city. The existing roads are not properly planned and designed and have no sufficient width for accommodation of other infrastructures. Up until now there was no proper design document of the drainage system as well as proper master plan of road network of the city.

The previous Master Plan (1996) didn't recommend anything about drainage system and network for the town. In connection to this no plan design drawings are found in the Municipality regarding the existing drainage network. However, the existing asphalt roads (the main high way to Dessie, the two asphalt roads on left and right sides of this main high way, those which connect these two asphalt roads with the main high way, and the road that dissects to Debre Berhan University, have improved the drainage and flood protection conditions of the town. (Dagne Amdetsion, 2016).

2.1.5.1. Drainage pattern of Debre Berhan

The drainage pattern of the city is done on the available maps following the general topographic nature of the city. This has been confirmed on the site and some adjustment has been made on the final maps. The drainage master plan should be related with the existing road width and the city road network master plan including all utility lines and infrastructures. However, such master plan was not available for Debre Berhan town. The drainage pattern is to indicate the center line position of the drainage system relative to the road section. The aim of the drainage pattern is also to indicate the general drainage pattern of major roads within the city and their disposal points. In the process of drainage network design, it is recommended drainage on both sides of the road network system. Catchment's area for any recommended drainage system may be either the road surface runoff or the immediate adjacent catchment area, depending on the topographic nature of the area. There are small streams and gullies within the city. Those can be utilized as a disposal point of the drainage system. Still the existing drainage systems are dropping out into those small streams and rivers. Therefore, streams and rivers within the city are serving as part of the city drainage system. The drainage system of the town is explicitly guided with the two rivers locally known as Beressa and Dalecha rives though some other local streams have also contributed little in such regard. Beresaa River, which is a perennial stream, bisects the town in to two (Debre Berhan proper and Tebassie) has contributed most. It, therefore, drains most parts of the town (the central, southeast, western half including the northwestern and the southwestern). Dalecha, on the other hand, drains the northern parts. Moreover, there are other intermittent streams which end up flowing to one of these two main rivers. And these small streams have also their own contribution to facilitate the drainage though limited in their local areas. All these represent the natural drainage networks of the town which facilitate the surface drainage system of the town in almost all parts having a significant positive implication. And besides, those two rivers are also used as major source of drinking water for livestock, washing, greenery and small scale irrigation. (Dagne Amdetsion, 2016).

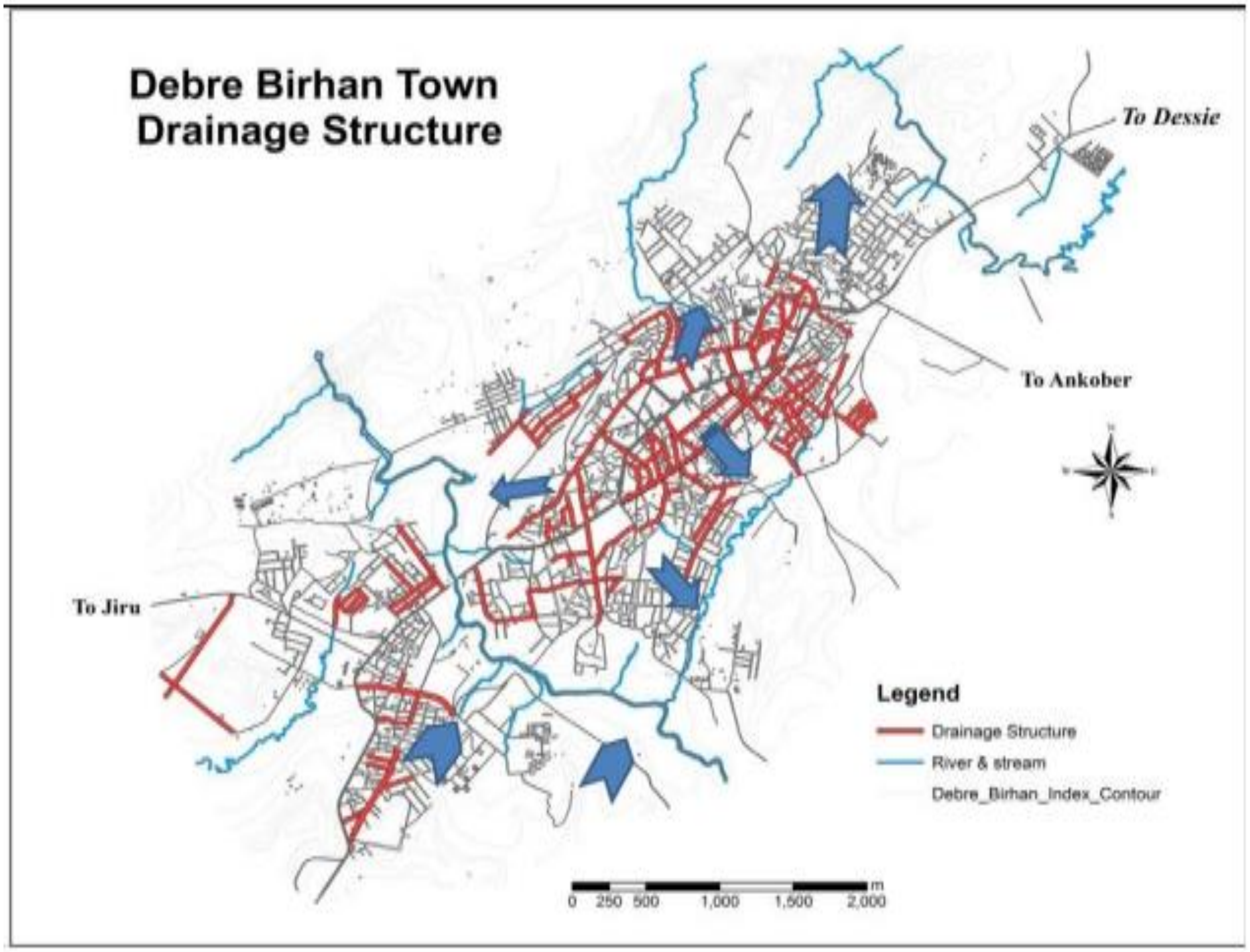

Figure 1: Drainage Pattern of the Town (Dagne Amdetsion, 2016).

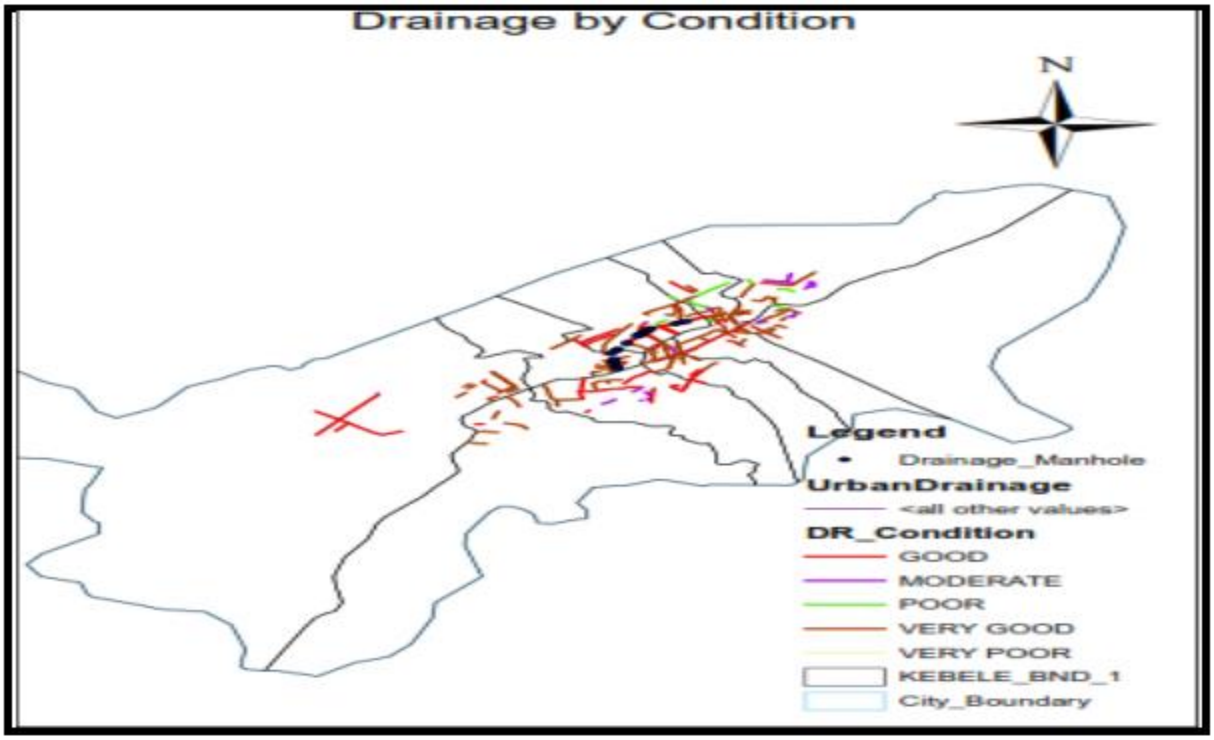

Figure 2: Drainage Condition (Dagne Amdetsion, 2016).

Some parts of the town, where there is a flat topography which is observed in the newly expanded area, this needs a careful drainage system design.

2.1.5.2. Existing manmade drainage

These manmade road drainage structure networks divided broadly into two categories that include: first, the masonry channels (constructed "in-situ") represent most of the draining in the major asphalt and cobble stone roads either in the form of open system or closed system (covered by pre-cast concrete slabs with drainage slots). The width of the masonry drain is 80cm with 80 to 100cm depth. Second, excavated earth channels which are predominantly constructed for other road networks in the town with only limited sections being lined and which are rarely compacted. The asphalt roads that have been mentioned above are provided with manmade drainage ditches of open or closed types (though it doesn't cover all parts) either in both sides or in one side. In this regard, one can mention some like the main high way from the inlet off Addis Ababa to the Dessie outlet, the two sub arterial asphalt roads on the left and right sides of this main high way, those connector asphalt roads dissected from the main high way, and the asphalt road that runs to Debre Berhan University, the road to Blanket factory, and some cobble stone roads, have relatively better drainage systems. These drainage networks run in different directions of the town (in the central part it runs with the main road from southwest to northeast extremes, in the southern part it runs from north to south direction, and in the northern part of the town it runs from south to north direction).

In general, from the overall observation and assessment, it is realized that:

- Most of local and collector road have no road drainage structure on both side. Some of cobble stone road have only one side drainage structure;
- Some of drainage structure doesn't follow the road alignment;
- Most of the drainage systems have no proper start and end point to be terminated and hence dead ends observed on most of the drainage structures construct.

Figure 3: Views of poor drainage network (During site visited 2018).

Figure 4 : Conditions of drainage structure. (During site visited 2018).

While most of the asphalt roads have rectangular masonry structural drains, most of the local roads have no any drainage structure at all, some have the natural and excavated earth channels. As a result, the storm water flows everywhere and in ever00.0y direction creating its own way (without properly designed drainage path). That is why, this mostly observed on the road network itself, near the fences, and even standing still for some times. However, it is important to note that the drainage network does not cover all sides (even the most requiring once) and all except some (those found along the principal and arterial asphalt roads) are sub-standard and low quality networks. This, therefore, implies how much the drainage coverage of the town is still very much low.

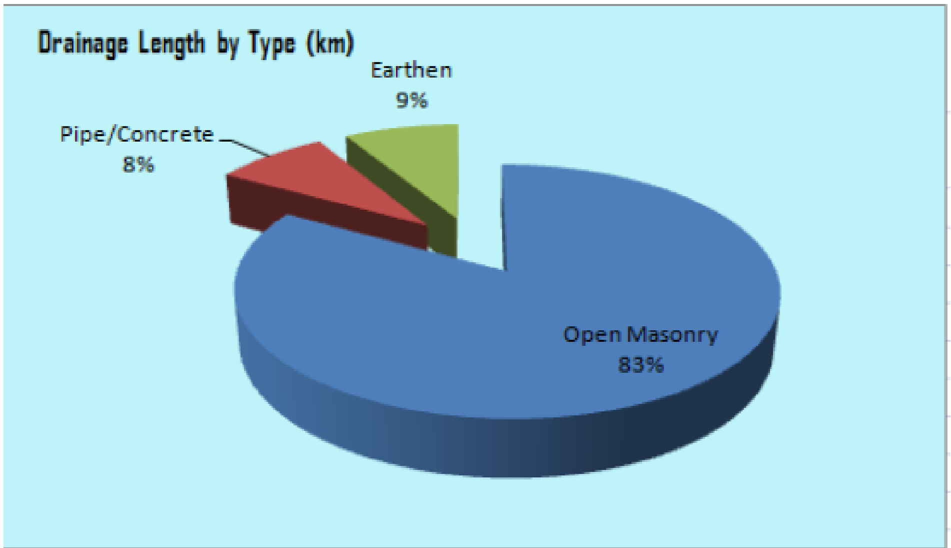

Figure 5: Drainage length by type. (Dagne Amdetsion, 2016).

2.1.6. The hydrological cycle and urban stormwater

Today, the natural hydrological cycle (i.e. the description of the movement of water below, above and on the earth's surface) (USGS, 2014) has to a large part been replaced by an artificial water system (i.e. urban drainage) within city areas. In nature, when precipitation falls upon the natural soil surface a large amount of the water is absorbed by the local vegetation. The majority of the water (that has not been absorbed by vegetational /plant roots) is either transported and discharged into downstream surface water recipients (such as rivers, streams and lakes) or becomes a part of the groundwater through continuous infiltration of the soil and bedrock. The amount of surface water runoff in a specific area will be influenced by the geological conditions on site (i.e. type of bedrock and soil layer that exist in the area and how fast the soil gets saturated) as well as climate conditions (i.e. intensity and duration of precipitation). Since precipitation varies over the year the flow rates will also vary with respect to seasons. Soil has some capacity to bind water but when saturated the runoff tends to increase rather rapidly, resulting in a higher and more intensive water flow (Butler and Davis, 2004).

Storm water is the water runoff created by precipitation and snow melting events that comes into contact with different urban surfaces and materials without being absorbed by the natural ground (i.e. soil). Due to this storm water is often contaminated by a broad range of different pollutants since the water moves through and absorbs substances from a variety of urban surfaces (i.e. parking lots, roads with heavy traffic and eroded urban surfaces such as rooftops) (EPA, 2014b). Furthermore, pollution originating from atmospheric deposition will also find its way into the storm water runoff (Butler and Davies, 2004).

2.1.6.1. Urban stormwater runoff

In the following subtopic a description of urban stormwater runoff with regard to transport, ground characteristics, precipitation (including effective rainfall) and pollution will be described. (EPA, 2009).

2.1.6.2. Stormwater runoff transport

The development of urban areas has had a significant impact on urban stormwater runoff and generation due to the replacement of natural green infiltration surfaces (i.e. natural soil cover) with impervious surfaces (such as concrete roads, rooftops and buildings) within cities (EPA, 2009). Due to this, stormwater is transported downstream at a much faster rate (since water moves faster over hard surfaces in comparison to natural surfaces). The result will be that urban areas experience a faster moving runoff flow (with a higher peak flow) that will enter the urban drainage system at a faster rate. But the urban runoff flow will also die away much faster (compared to natural green areas) which will result in a higher peak flow (Butler and Davies, 2004).

The figure below illustrates the difference in runoff volume before urbanization and after urbanization has taken place

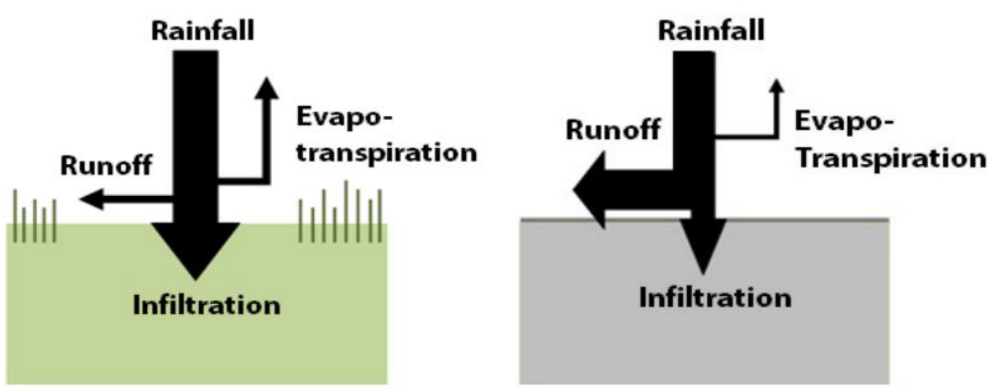

Figure 6: water transport as a result of precipitation before urbanization natural surfaces and after urbanization.

2.1.6.3. Precipitation characteristics and evaluation

As mentioned above the peak flow will be higher in urban areas due the increased amount of impervious surfaces and general lack of natural surface areas. Most stormwater is the direct result of precipitation (but could also be influenced by snow melting events in cold regions) occurring in the area and the urban drainage system is required to deal with this kind of water. The urban effect on stormwater runoff peak flows (as compared with natural/rural surfaces) is illustrated below.(Butler and Davis, 2004).

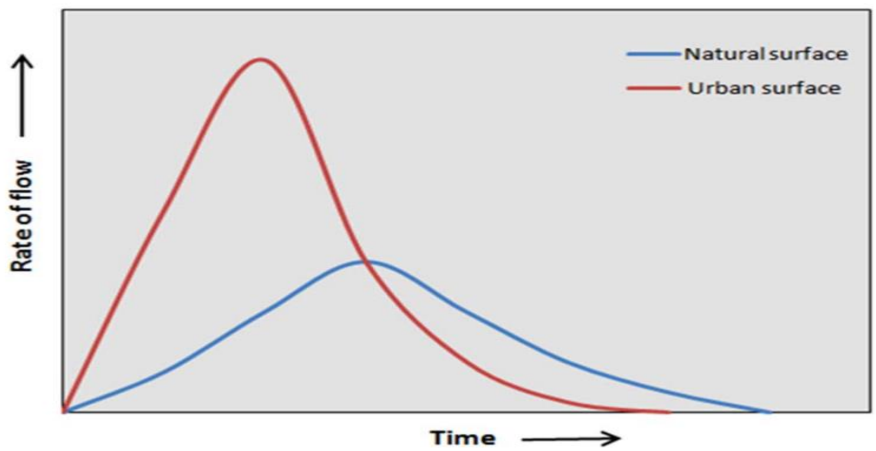

Figure 7: Effect of urbanization volume and rates of surface water runoff

Rainfall is normally measured as intensity (mm/hour) and is representative on a specific location and often recorded together with duration and frequency. Rainfall duration refers to the specific time period for which the rainfall lasts. Rainfall frequency is an expression of the return period of a similar rainfall event with the same magnitude rate and is normally expressed in years. As an example, if a specific rainfall occurs 25 times during a 100 year period then it has a return period of 4 years) (Butler and Davis, 2004). During a rainfall event the intensity is typically the largest at the beginning (i.e. the first hour) and then diminishes with every hour after that (i.e. the intensity reduces with duration). This could be illustrated in an IDF graph (i.e. how rare or frequent a certain rainfall event is) (Butler and Davis, 2004).

2.1.7. 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).

2.1.7.1. EPA SWMM5

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, SWMM5subdivides 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).

2.1.7.2. MOUSE

MOUSE (Danish Hydraulic Institute, 1988) stands for **M**odeling **of U**rban **Se**wers 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 stormwater 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).

2.1.7.3. CIVILCAD

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).

2.1.7.4. 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 km2 (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 Subcatchment. 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.

2.1.7.5. 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.

2.1.7.6. STORM

The U.S. Army Corps of Engineers (1977) developed **Sto**rm water **R**unoff **M**odel (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.

2.1.8. Selection of storm water management model

There are various number of techniques for evaluation of storm water runoff on bases of water balance equation, empirical equation and viable models like SCS curve number, STORM model, CIVIL CAD, RAFTS, MOUSE and SWAT by such strategies calculations for penetration, overflow of surface, routing of flow, and slacking of surface overflow have been re arranged to permit simulation of flow with a hydraulic and Hydrologic Simulation Program For each one of those number of techniques they have some limitations and advantages .however for our project targets the SWMM5 were favored for urban runoff estimation. Because of the following consideration and it was validated in our country, jimma town. (Getachew K. and Tamene A, 2015).

2.1.9. Low Impact Development (LID) controls

Basically, LID is a land re-development approach to manage storm water. The main goal of LID is to reduce the negative effects of precipitation flooding waters by maintaining the pre-development hydrology of a site by decentralizing micro-scale controls (Coffman, 2000; Shafique & Kim, 2015).LID practices effectively reduce water-related problems through infiltration and evaporation of the storm water resulting environmental, social, and economic benefits. 8 The common LID practices are bio-retention, green roofs, permeable pavements, rain gardens, vegetative swales, and rain cisterns (a.k.a. rain barrel) that are used to create a functionally equivalent hydrologic landscape (Coffman, 2002).

2.1.10. LID deployment in SWMM5

Overland flow from LID controls can be modeled in three ways. The first approach is to route impervious subcatchment to pervious subcatchment to receiving node as shown on Figure 9. Pervious area properties are to be matched to LID control design. The pervious area of the subcatchment acts as LID control. This approach is not realistic and does not give accurate results.

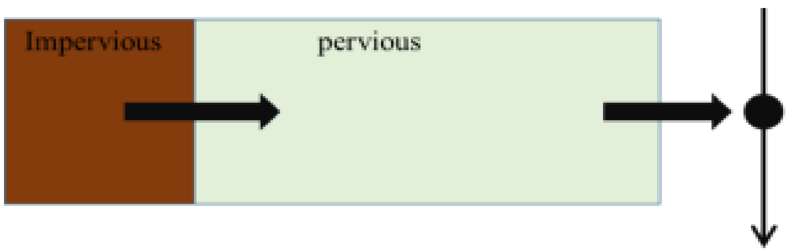

Figure 8: Route impervious to pervious area

The second approach is to create LID subcatchment as a separate subcatchment and to route the original subcatchment to the LID subcatchment to receiving node as shown on Figure 10. The LID design is to be matched to subcatchment properties. LID area is to be extracted from original pervious or impervious area.

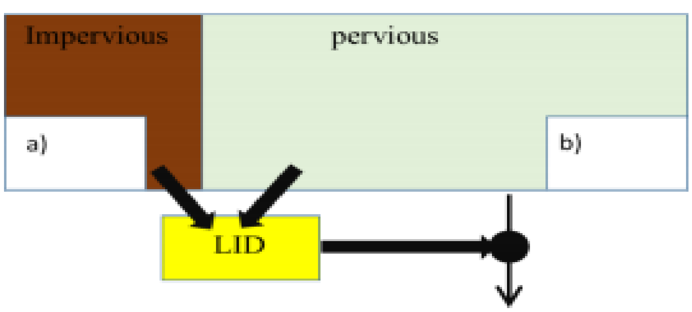

Figure 9: LID as separate Catchment

The last approach is to create LID as part of original subcatchment and to route runoff through LID prior to receiving node as shown on Figure 11 (USEPA, 2000). LID area is to be added to original pervious or impervious area.

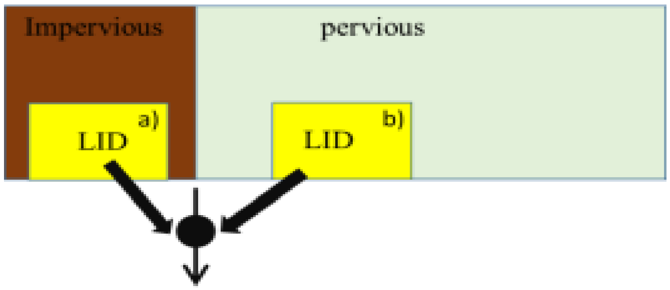

Figure 10: LID area included in the subcatchment

If multiple LID units are placed in a subcatchment, then the LID units take the impervious area runoff of a subcatchment. Different capture ratios can be given to different LID units. The options for routing the surface flow and underdrain flow of the LID units are as follows: a) both surface overflow and underdrain flow is routed to the sub catchment's outlet; and b) Underdrain flow can be routed to a separate outlet other than its subcatchment pervious/impervious area. (USEPA, 2000).

Sub catchment of Debre Berhan town are contain both pervious and impervious and no area used separately for LID be for receiving node to control the stormwater. because of this the area were LID control applied are considered as part of original subcatchment.so the second alternative shown on figure 10 selected to route runoff through LID prior to receiving node .

3. MATERIALS AND METHODS

3.1. Description of the study area

3.1.1. Location and topography

The Town of Debre Berhan is located at $09^0 41'$N latitude and $39^0 31'$E longitude, 130 km on the main road Addis Ababa- Dessie -Mekele road in the Amhara region, north shoa administrative zone, and district of Debre Berhan zuria. Debre Berhan is situated on a plateau in the central Ethiopia highland system about 15km west of the great rift escarpment at an average elevation of between 2800 and 2845 m a.s.l. with a total area of 1471ha.

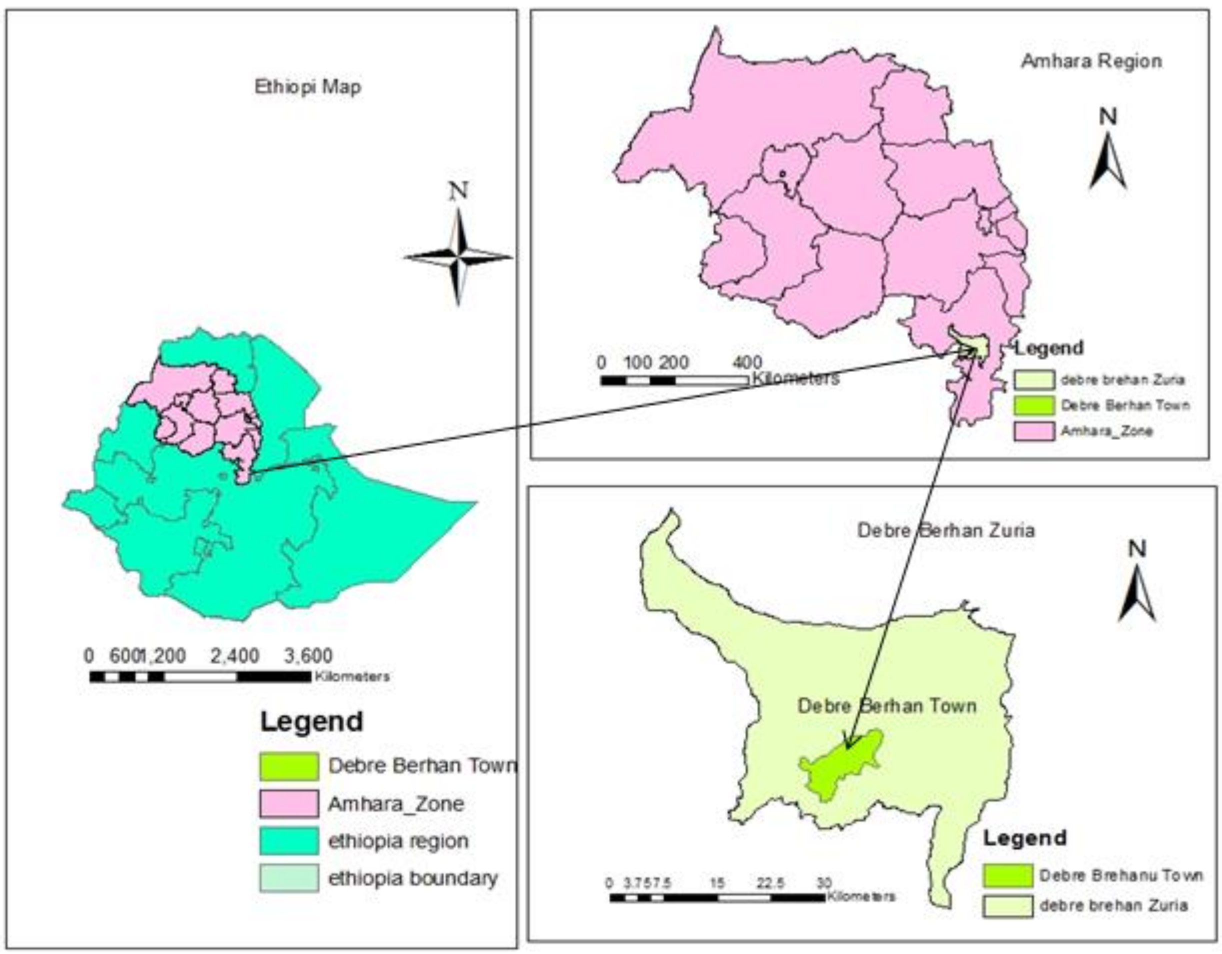

Figure 11: Location of the study area Debere Berhan town (by Arc GIS)

Debre Berhan town is characterized by a highly flat relief with small hills, see Figure below. The general slope is about 0.9%, dipping from the southern part or upper catchment part towards mendida catchment. The highest elevation in Debre Berhan is 2800 meters, while the lowest point in the study area is 2845 meters .The steeper areas are the north edge and the western part, which consist mainly on small valleys bordering the urbanized area.

Figure 12: Topography of Debre Berhan town (google earth)

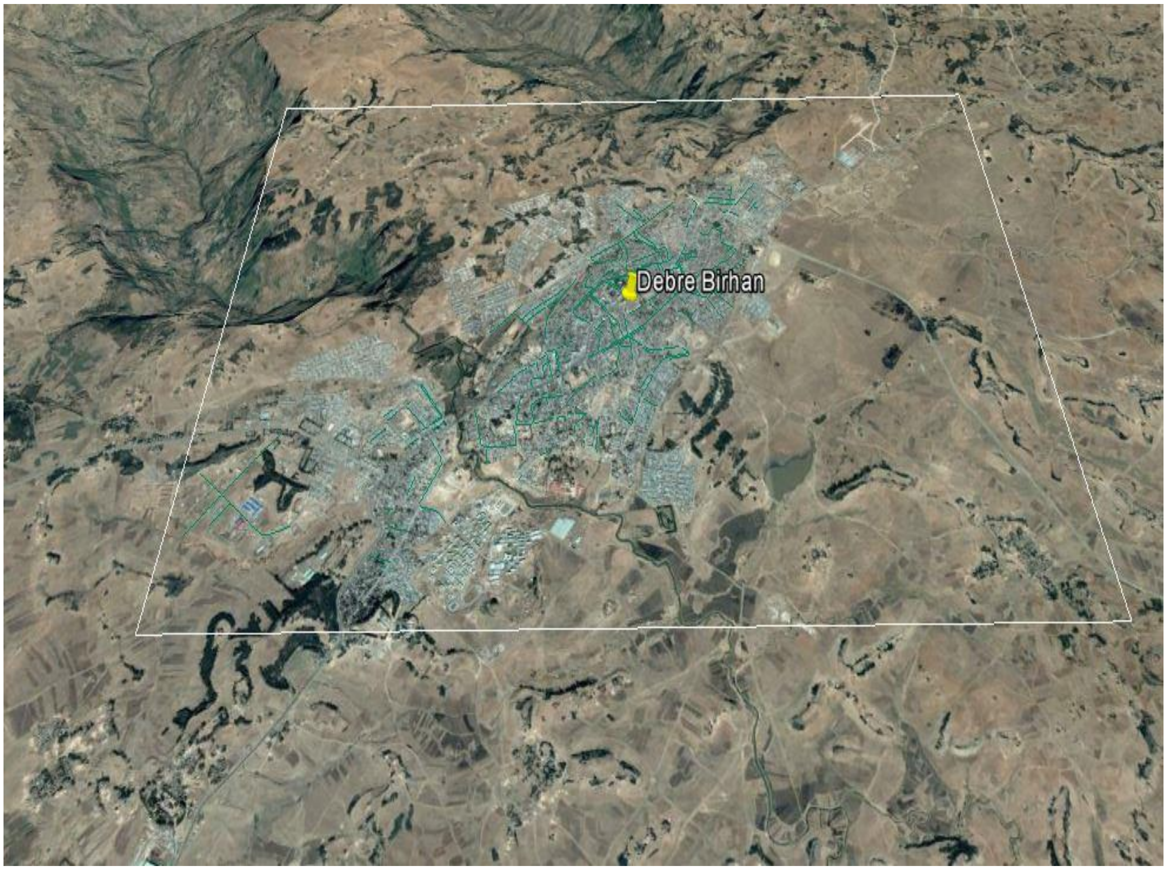

Figure 13: Border of Debre Berhan town and Debre Berhan zuria (google earth)

3.1.2. Population

The population of Debre Berhan town was estimated in 2006 E.C by central statistics Authority are 79,832. Rural dwellers are 13,261 and the remaining 66,571 live in the town. The population of the town becoming increasing from time to time in relation with the town development in investment, trade and other activities (source :population by wereda ,central statistics Authority 2006).

3.1.3. Climate

Debre Berhan lies between elevation of 2800 and 2845m a.s.l. From the elevations it belongs to the dega climatic zone. The mean annual temperature ranges between 5^0C and 23^0C which is presented in Table 1 from the available records. (Ermias S, 2007)

Table1: The mean monthly temperature fluctuation based on Seven years of data's in Debre Berhan.

Month	Jan	Feb	Mar	Apr	May	Jun
Mean Temp ºC	12	13.2	14.4	15.4	17	16.9

Jul	Aug	Sep	Oct	Nov	Des
14.5	15.7	15.4	13.4	13.1	11.9

3.1.4. Precipitation

The mean annual precipitation is 874mm shown by Table 2 which corresponds with the isohyetal map. The precipitation regime of Debre Berhan is classified in the II-D regime. The rain fall distribution is for six months from March to April and from June to September with high concentration occurring in July and August. The rainy seasons are designated small rain falls in March, April and June and big rain falls from July to September.

Table 2: monthly mean Rainfall of Debre Berhan.

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
mean (mm)	15.56	16.75	38.97	55.03	29.02	62.53	333.39	289.50	77.50	19.86	53.02	8.19

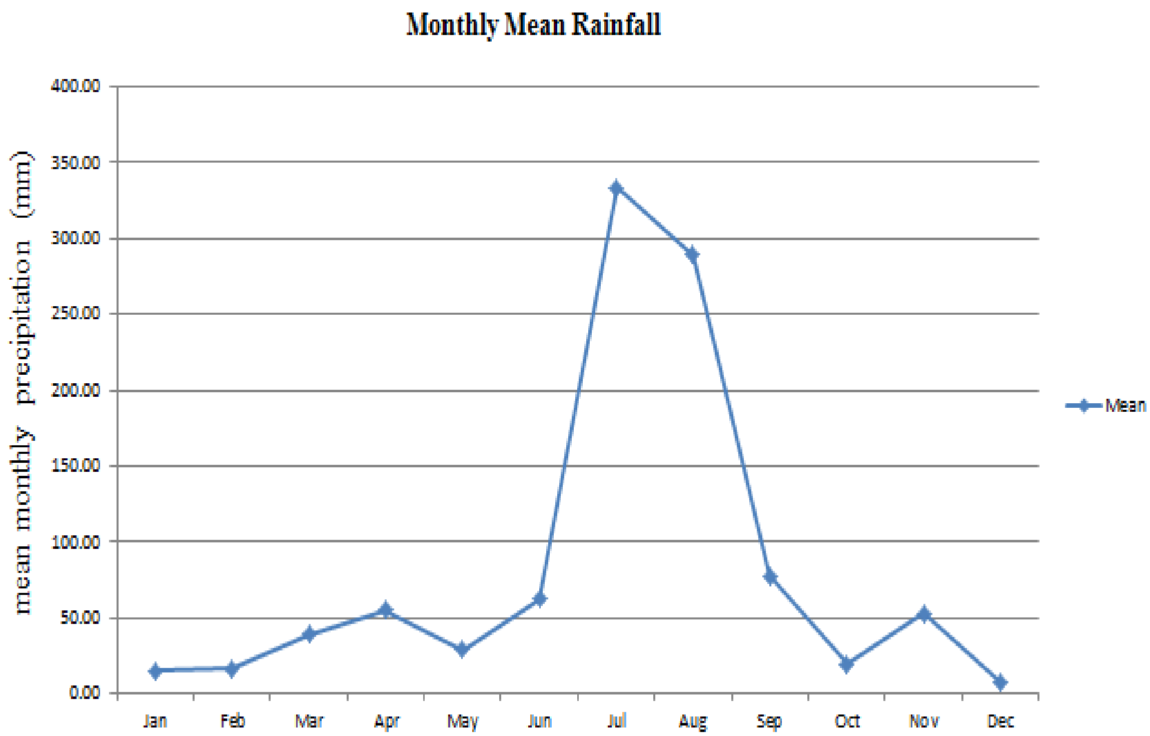

Figure 14: Annual average monthly rainfalls in Debre Berhan

3.1.5. Land Use, Land Cover and soil type

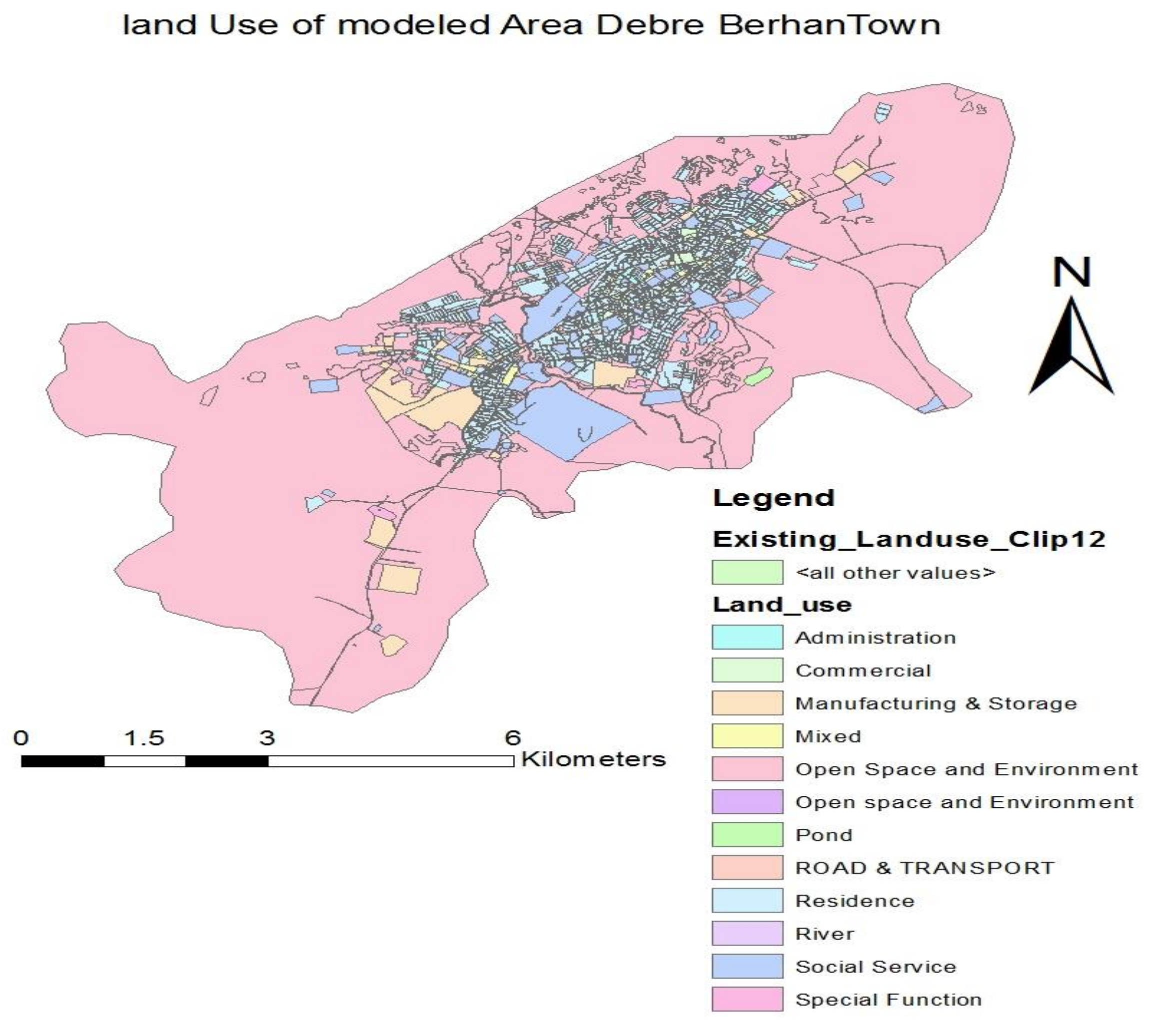

Figure 15: Land use of modeled area Debre berehan town (by Arc GIS, 2018)

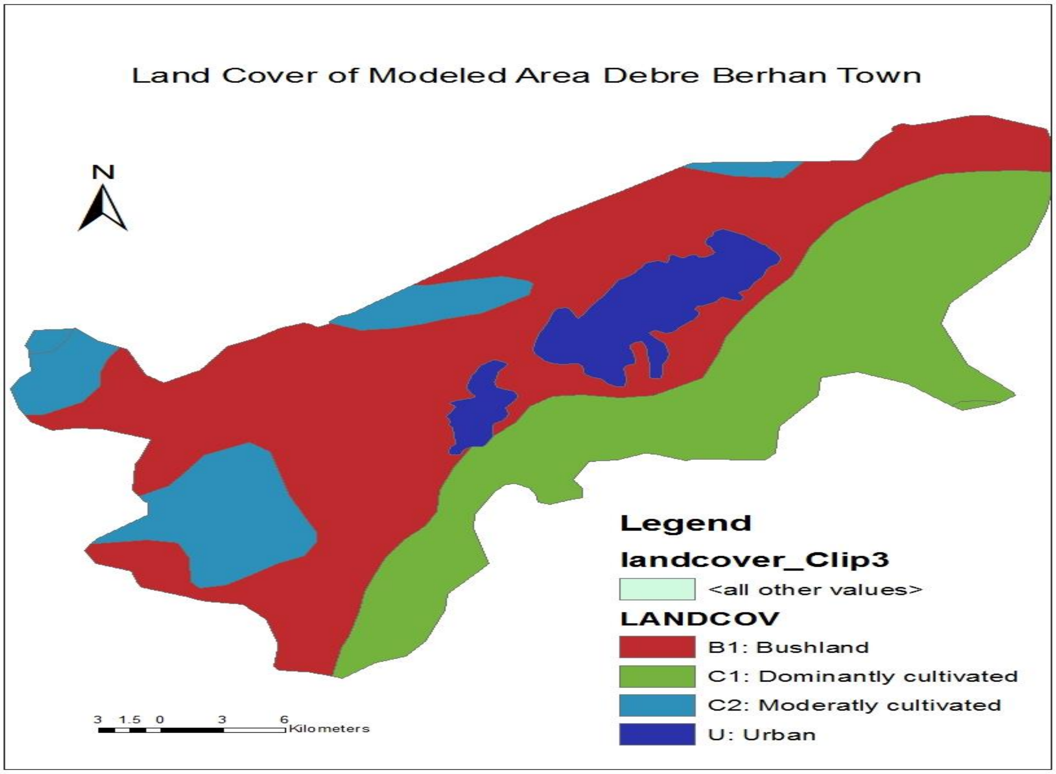

Figure 16: Land covers of Debre berehan town by Arc GIS, 2018

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 types of soil at the study area Debre Berhan town were Eutric Cambisols, Vertic Cambisols and Lithosols of hydrologic soil group D.

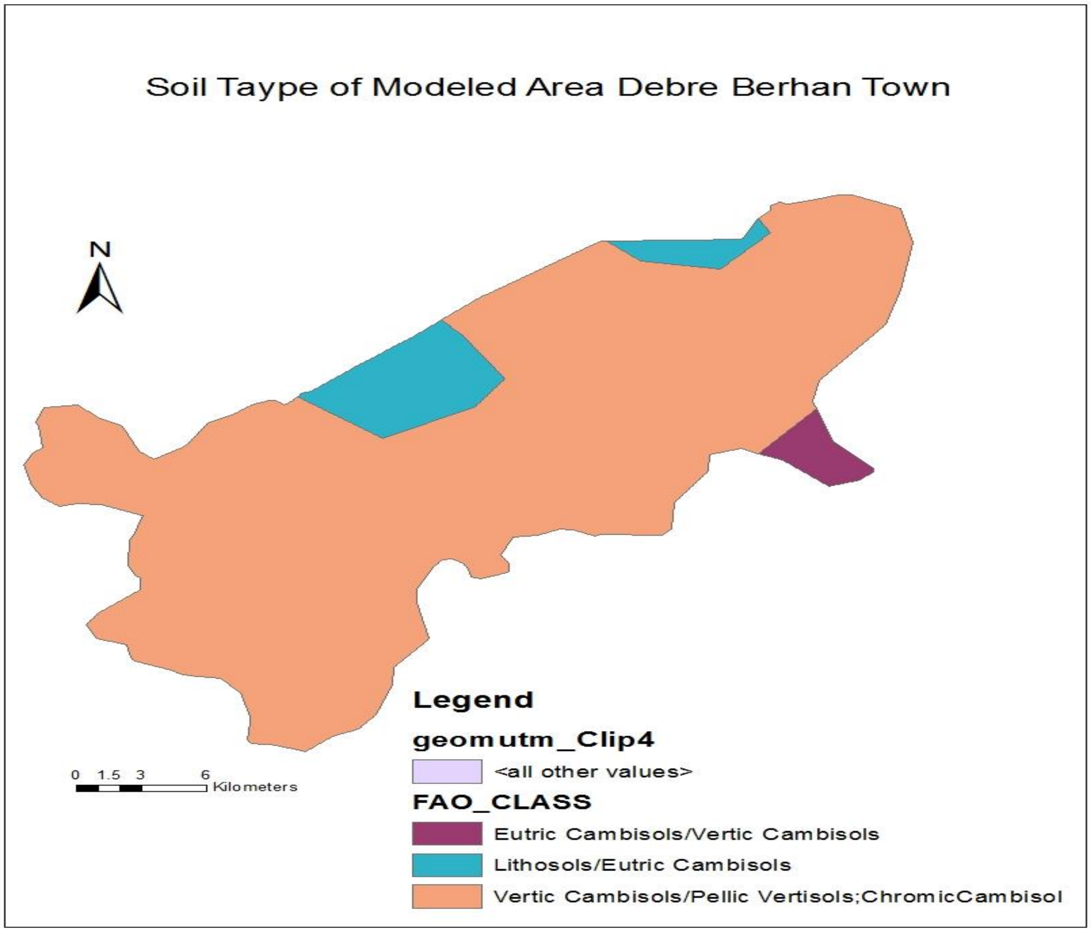

Figure 17: Soil type of Debre berehan town by Arc GIS, 2018

The modeled area contains all types of urban area means, there is commercial, administration, and business area and also these modeled areas contain areas that are not developed. The modeled area has only three types of soils these are Eutric Cambisols, Lithosols and Vertic camblsoils according to typical hydrologic soils groups for Ethiopia.

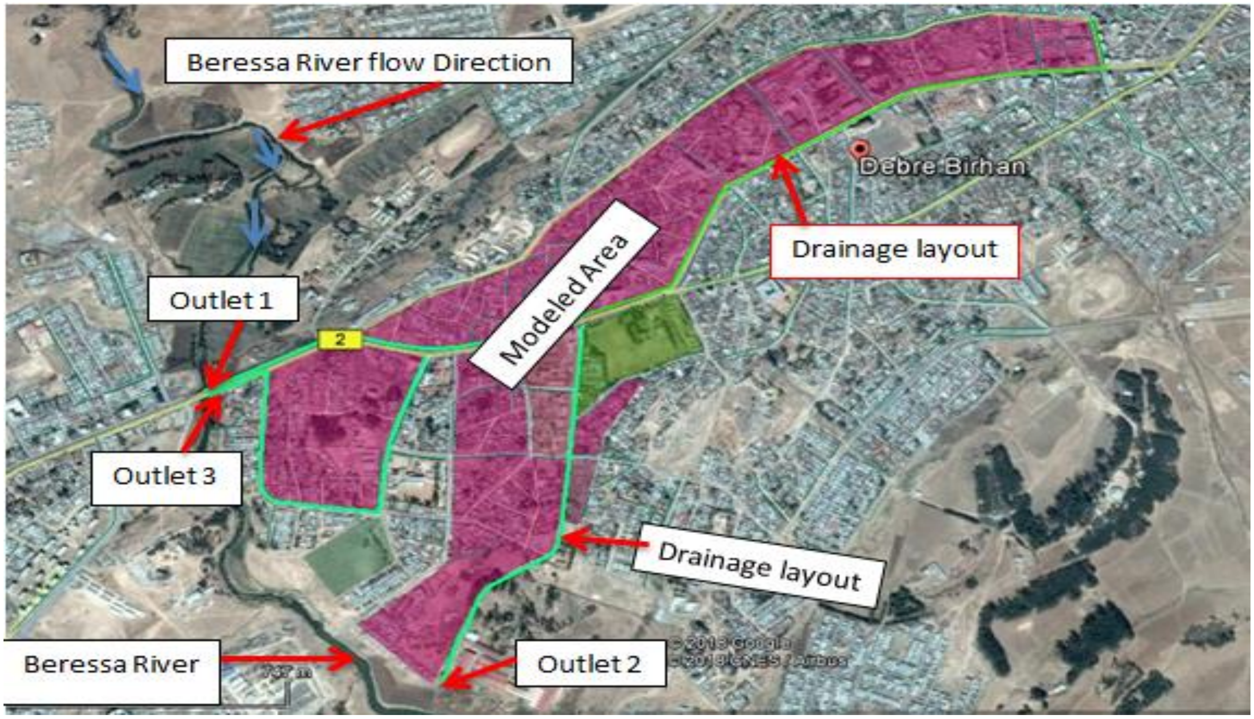

Figure 18:Modeled study area top view (Source: Google Earth)

Figure 19: present drainage and flooding condition (site visit).

3.2. Data sources

For this study, information and data collection were obtained via two sources which include: Primary and secondary sources.

A. Primary sources

Study Area Observations:

For this study, 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. A measured Flow depth which used for model calibration and validation was done.

B. Secondary Sources

The physical data of the catchments and their storm water drainage systems were also collected for modeled of study catchments. 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 as built gained from Ethiopia road authority sebeta district and Debre Berhan town municipality. Rain fall data of 10 Years which used for IDF curve developed were collected from Ethiopia metrological agency and sensitive parameter that used as input for SWMM5 model were collected from books, journals, manuals etc.

3.3. Rainfall data analysis

3.3.1. Fill missing data and consistency check

Hydrologic data (i.e. rainfall data) were carefully checked for accuracy and consistency as part of the program. Whenever Possible, the collected rainfall data of Debre Berhan were checked against fill the missing data normal ratio method were used Because of the normal annual precipitation of the index stations lies exceeds ±10% of normal annual precipitation of interpolation station. According to the normal ratio method the missing precipitation is given as:

$$P_X = \frac{1}{n}\sum_{i=1}^{i=n} \frac{N_X}{N_I} P_x \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.1)$$

Where P_x is the missing precipitation for any storm at the interpolation station 'x', Pi is the precipitation for the same period for the same storm at the "ith" station of a group of index stations, N_x the normal annual precipitation value for the 'x' station and Ni the normal annual precipitation value for 'i_{th}' station.

For three index stations in a catchment area were become.

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

And a double mass curves were used to check the consistency of rain gauge record and the correction factor were 0.832. Independent rainfall data obtained from other 3 nearby stations Chacha station, Mendida station and Debre Sina station for selected storm events were used.

3.4. IDF curve developed for Debre Berhan

Intensity duration frequency (IDF) curves describe the relationship between rainfall intensity, rainfall duration and return period (or its inverse, probability of exceedence).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 Debre Berhan station IDF curve developed to obtain the depth and intensity of 15 minute interval that used as in put for SWMM5 model. The following equation were used to developed Rainfall in a given durations (hr).

$$\frac{R_t}{R_{24}} = \frac{t}{24}\left[\frac{(b+24)^n}{(b+t)^n}\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.3)$$

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, the average values of b and n are found to be 0.3 and 0.9 respectively. These values have been adopted for this thesis project IDF development. (M. L. Waikar* and Undegaonkar Namita U, January, 2015).

The relationship adopted for IDF development at a given station, any probability distribution can be used but the reliability of the distribution is checked the goodness of fit tests by chi-square test and Log Pearson Type III methods are used for this research.

$Y_T = Y_{avg} + K_T * S_y$ ………………………………………………… (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]

Sy - Standard deviation (mm)

KT = Log Pearson Type III distribution frequency factor (taken from readily available table) and Coefficient of skewness Cs is calculated for the logarithms of the data. The frequency factor depends on the return period T and the coefficient of skewness Cs. When Cs= 0, the frequency factor is equal to the standard normal variable z. When Cs ≠ 0, KT is approximated by Kite (1977) as:

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

Where:- K = $\frac{C_S}{6}$.

3.5. Hydraulic and hydrological modeling using SWMM5

3.5.1. Description of SWMM 5 model

SWMM (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 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, SWMM subdivides the overall catchment into subcatchments, predicting runoff from the subcatchments on the basis of their individual properties, and combining their outflows using a flow routing scheme. SWMM can also simulate backwater effects. (EPA, 1992).

In SWMM, subcatchments are represented mathematically as spatially lumped, nonlinear reservoirs, and their outflows are routed via the channel/pipe. Subcatchments 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).

Before any analysis work could be conducted, a hydrologic model of the Duck Pond watershed needed to be constructed. No one had previously assembled a detailed model of the entire watershed, and no single database existed that contained all of the necessary information. Data had to be gathered from numerous sources and compiled together into a coherent body of information. The program selected to perform the modeling was SWMM5. This chapter presents the efforts to compile and process the data required for the model.

Model set up procedure

- Set the coordinates of area map/image
- Draw network representative and describe sub catchments
- Edit the properties of the object that make up the system
- Describe how the system is operated
- Select a set of analysis options Run Simulation for Rainfall/Runoff and Flow routing

3.5.2. Governing equations

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 20. Overland flow is generated by modeling the sub catchment as a nonlinear reservoir, as sketched in Figure 21. (EPA, 1992)

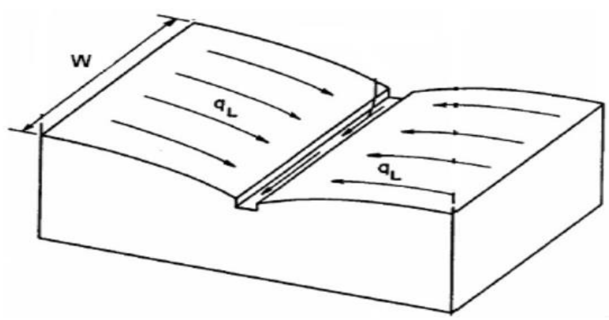

Figure 20: Idealized representation of a sub catchment.

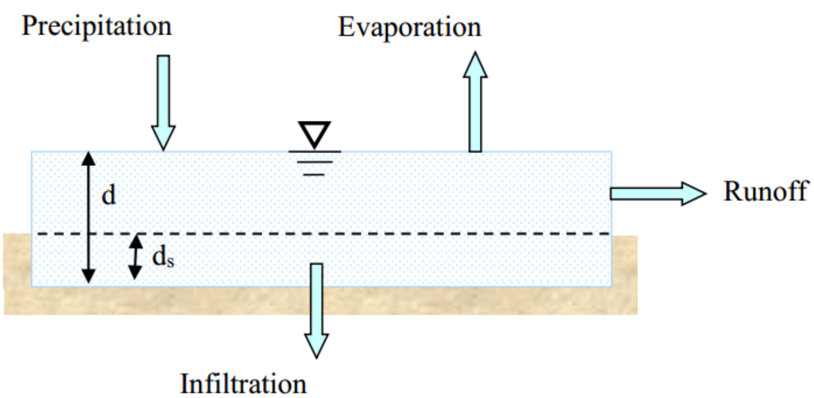

Figure 21: nonlinear reservoir model of a sub catchment.

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. Ponded 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:

$\frac{\partial d}{\partial t} = i - e - f - q$ ……………………….……... (3.5)

Where:

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 (cms/m^2 = 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 (cms) as:

$Q = \frac{1}{n} S^{1/2} {R_x}^{2/3} A_x$ ……………………….…… (3.6)

Where:-

n -is a surface roughness coefficient, *S* the apparent or average slope of the sub catchment (m/m), A_x- the area across the sub catchment's width through which the runoff flows (m^2), and
R_x- is the hydraulic radius associated with this area (m).
Referring to Figures 20 and 21, A_x is a rectangular area with width W and height d-ds. Because W will always be much larger than d it follows that

$A_x = (d - d_s)$ and $R_x = (d - d_s)$

Substituting these expressions into Equation 3.6 gives:

$Q = \frac{1}{n} W S^{1/2} (d - d_s)^{5/3}$ ………………….…… (3.\7)

To obtain a runoff flow rate per unit of surface area (q), equation 3.7 divided by surface area of the catchment.

3.5.3. Preparation of modeled area

The surface runoff, water elevation profile was obtained by simulating the SWMM5. The Modeled area was divided into 17 different regions called as sub-catchments (S). Each subcatchment is designed with storm water lines by providing proper slope at intermediate junctions by connecting with conduits.

The overall runoff which was delivered from all the sub-catchments is discharging to outfalls through conduits with required slope. The present simulated model S1 to S17 denotes seventeen sub-catchments, J-indicates junctions between the nodes and C- stands for conduits which connects the flow between successive junctions. The modeled area of Debre Berhan town drainage network consists of 59 nodes means 56 Junctions, 59 links and 3 outfalls. See Figure 22, 23 and 24.

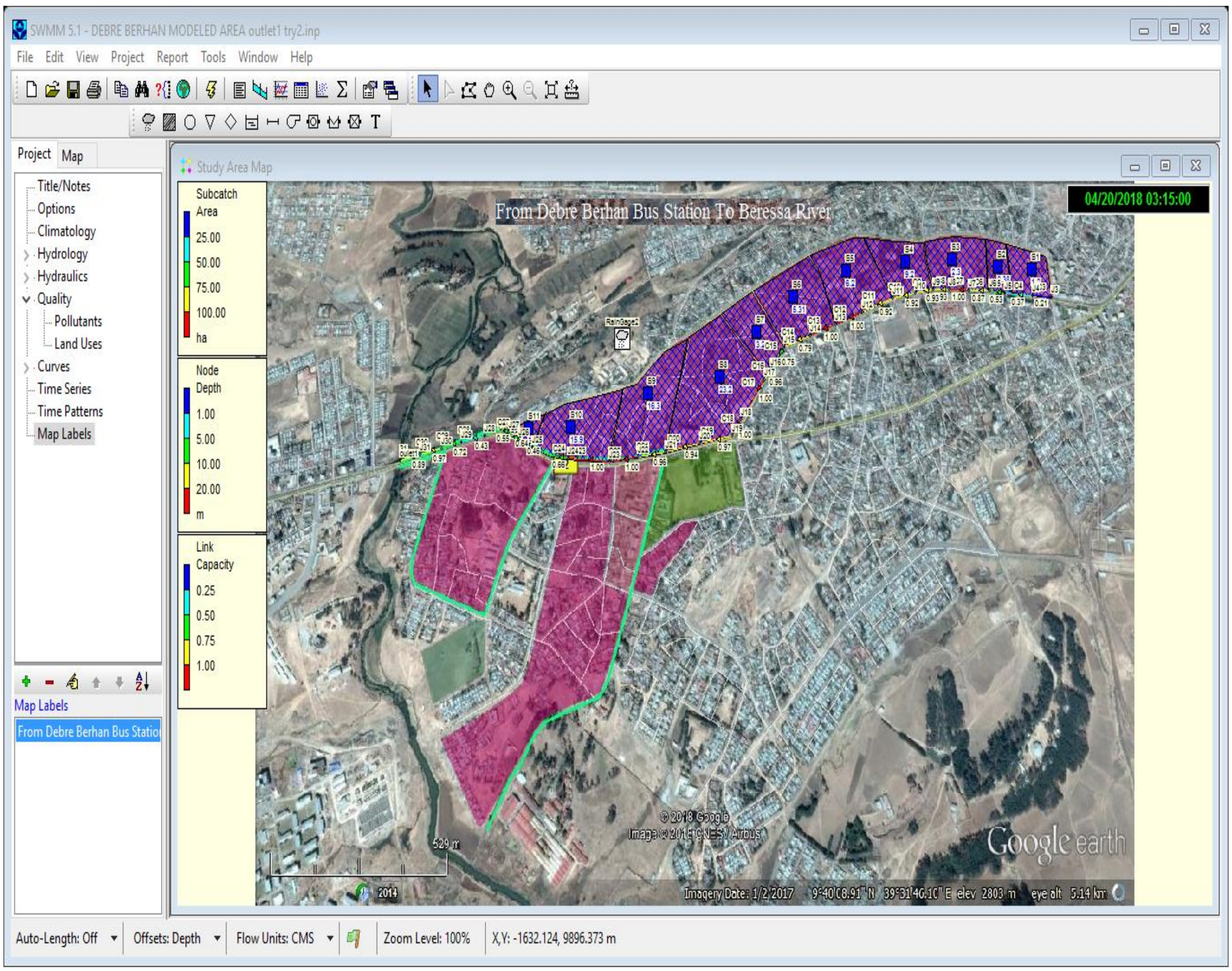

Figure 22: map of flood modeling from Debre Berhan bus station to Beressa River (SWMM5).

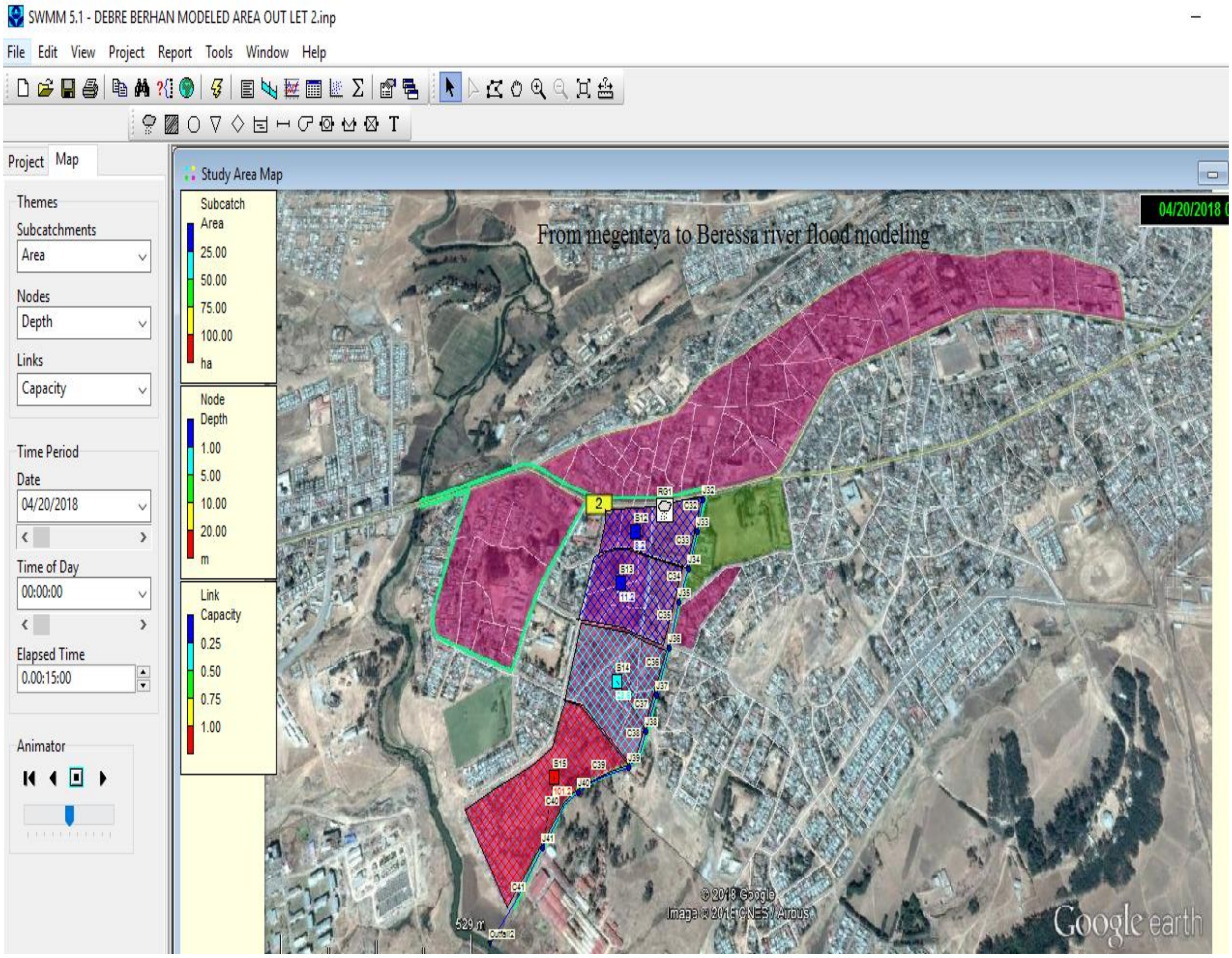

Figure 23: map of flood modeling from megenteya to Beressa River.(SWMM5)

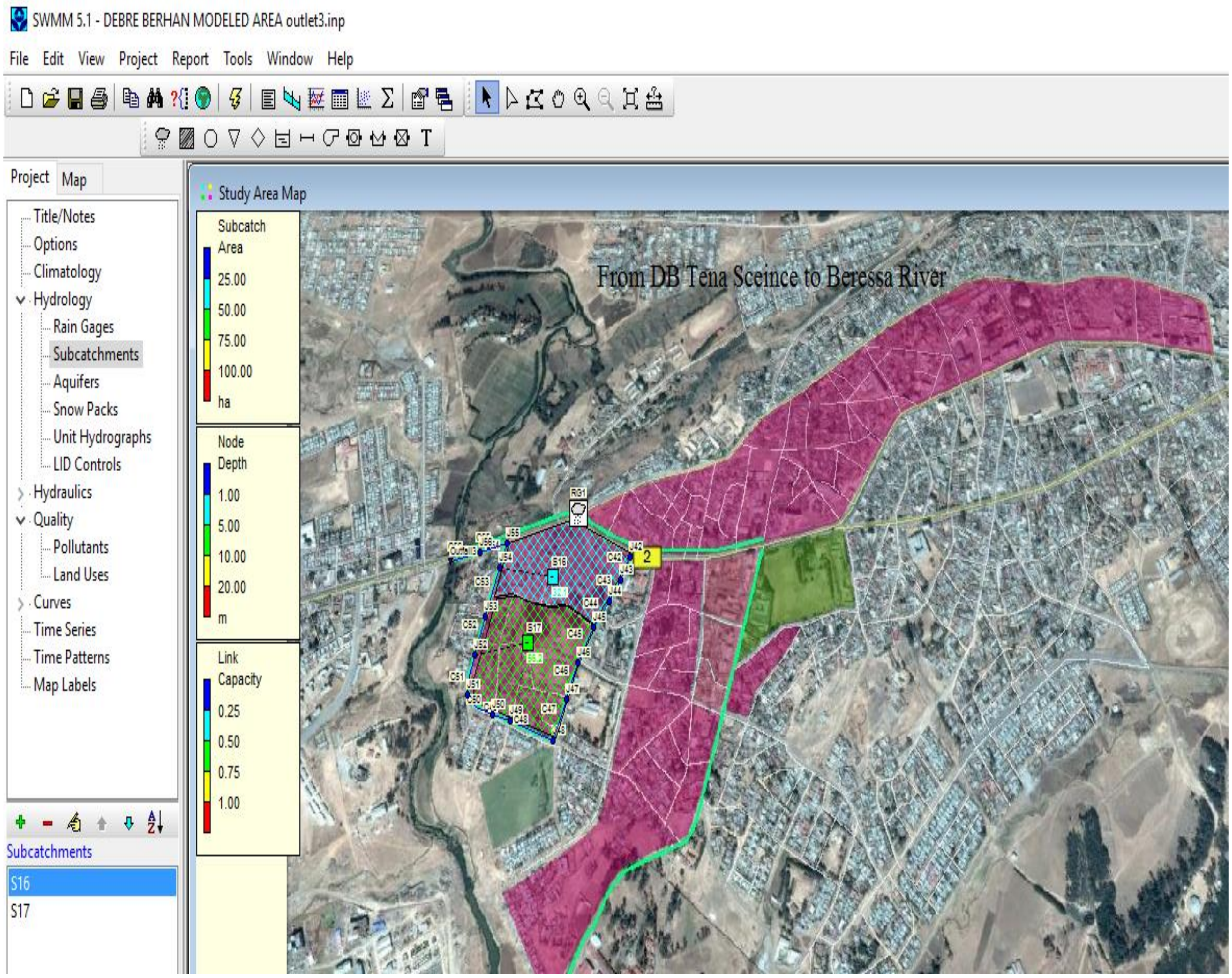

Figure 24: map of flood modeling from Debre Berhan Tena science to Beressa River. (SWMM5)

The manholes/Junctions are modeled as closed rectangular 1 meter depth by 0.6 widths for Debre Berhan Bus station to Beressa River and open rectangular channel 0.8 depth by 0.5 widths for the other two site megenteya to Beressa River and Debre berehan Tena science to Beressa river .It had assumed that there are no energy losses in the manholes. The precipitations are introduced into the model by associating each sub-catchment to the rainfall time-series.

3.5.4. Model calibration and validation

Calibrations were a procedure of parameters evaluation and adjusting sensitive parameter by comparing calculated and simulated. Sensitivity Analysis, Model Calibration, and Rainfall-Runoff Simulation Sensitivity analysis were done for Debre Berhan modeled area. Sensitive Parameters that used for Debre Berhan modeled area were shown in table 4.The most parameters used for sensitivity analysis and their allowable range of change proposed by (Li et al ,2014).

Table 3: some keys parameters used for sensitivity analysis.

parameter	description	allowed range of change
N-Imperv	Maning's roughness coefficient for impervious areas	0.005-0.05
N-Perv	Maning's roughness coefficient for pervious areas	0.05-0.5
Dstore-Imperv	Depth of surface storage in impervious areas (mm)	1.3- 2.5
Dstore-Perv	Depth of surface storage in pervious areas (mm)	2.5- 7.6
Zero-Imperv	Impervious areas without surface storage (%)	50-80

It should be mentioned that different allowed ranges for above parameters has offered by several researchers; however the current study mostly used the values represented in table 4.

3.5.4.1. Currently available data for model calibration and validation

To support the SWMM5 model calibration and validation monitored rainfall through the basin and recorded stream flow at drainage system was needed. But, because of no flow gauges installed on Debre Berhan drainage system (devices, like flow meters) and unavailability of recorded data it was difficult to obtain observed data. Therefore, In order to accomplish this, open rectangular channel (from megenteya to Beressa River, which drains the 63.01 (ha) of Debre Berhan town) was selected. And depth was recorded for 10 days to calibrate sensitive parameters and validate SWMM5 model for the area. And also 10 day rainfall data parallel to the day that the depth recorded are taken from metrological agency and used for model simulation.

Therefore, 5 day rainfall data used to calibrate the model with the flow that calculated using 5 day average depth. Where the left 5 day rainfall data used to validate the model with the flow calculated using the parallel recorded depth.

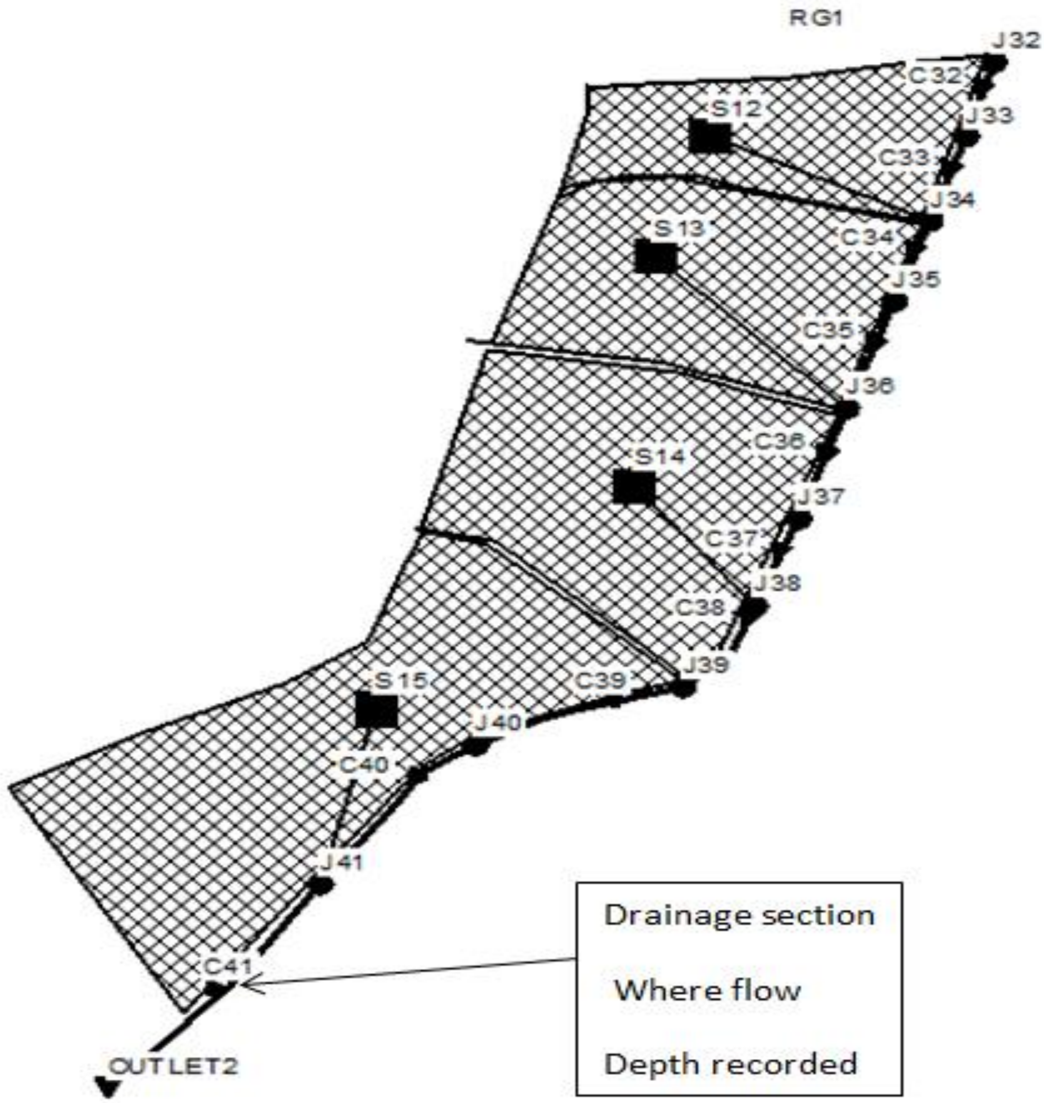

Figure 25: Site where flow depth gauged.

Table 4: Recorded depth used for model calibration

Gauge site	Date	Recorded Flow depth (m)	Avrg. flow depth (m)
near outlet 2	29-05-18	0.65	0.632
	12-06-18	0.76	
	14-06-18	0.45	
	21-07-18	0.72	
	27-07-18	0.58	

Table 5: Recorded depth for model validation

Gauge site	Date	Record depth (m)
Near Outlet 2	04-05-2018	0.71
	06-05-2018	0.64
	18-06-2018	0.72
	19-06-2018	0.59
	25-07-2018	0.66

3.5.5. Model performance evaluation criteria

The reliability of the flow monitoring data is assessed as part of the model calibration and validation process. And also, model simulation errors were quantified by measuring the difference between observed and simulated hydrographs. Results of all comparative studies were analyzed with three error functions

- Coefficient of Determination (R^2)
- The Nash-Sutcliffe Efficency (NSE)
- Relative Error (RE)

$$\text{Coefficient of determination } (R^2) = \left[\frac{\sum_{t=1}^{n}(q_t{}^{obs} - q_t{}^{avrg.obs})(q_t{}^{sim} - q_t{}^{avrg.sim})}{\sqrt{\sum_{t=1}^{n}(q_t{}^{obs} - q_t{}^{avrg.obs})^2}\sqrt{\sum_{t=1}^{n}(q_t{}^{sim} - q_t{}^{avrg.sim})^2}}\right]^2 \quad \text{……….} \ (3.8)$$

$$\text{The Nash} - \text{Sutcliffe Efficency (NSE)} \;=\; 1 - \frac{\sum_{t=1}^{n}({q_t}^{obs} - {q_t}^{sim})^2}{\sqrt{\sum_{t=1}^{n}({q_t}^{obs} - {q_t}^{avrg.obs})^2}} \;\ldots\ldots\ldots (3.9)$$

$$\text{Relative Error (RE)} = \frac{\sum_{t=1}^{n}|{q_t}^{obs} - {q_t}^{sim}|}{\sqrt{\sum_{t=1}^{n}{q_t}^{obs}}} \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.10)$$

Where ${q_t}^{obs}$ and ${q_t}^{avrg.obs}$ are the calculated and average flow respectively and ${q_t}^{sim}$ and ${q_t}^{avrg.sim}$ are the simulated and average flow respectively at time t, t is time, and n is the total number of time steps.

3.5.5.1. Acceptable level of calibration

RNS Between 0 and 1 indicates acceptable models < 0 indicates poor models, = 1 perfect models, = 0 Model is no better than using as an Estimator RE < 30%, R^2- Approach to one and indicates acceptable models .it’s shown on the graph relating the differences between observed and simulated hydrograph peak, volume and time to peak. These three attributes are important in design and analysis of urban drainage systems. Peak discharge is required in urban drainage design for sizing pipes, culvert and bridges. Runoff volume is required for design and operation of flood control structures such as retarding basins. Time to peak discharge is required for flood forecasting and operation of control structures during storm events. Flow data was calibrated for daily flows.

3.6. 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).Similar green drainage systems are called Low Impact Development (LID) or Best Management Practices (BMPs) in United States (USEPA, 2006).

3.6.1. 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 stormwater quality control measures. The benefits of reduced stormwater 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, providing detention, infiltration, evapotranspiration, or some combination of the three. SWMM LID features are attributes of individual subcatchments. SWMM allows the user for placing LID controls:

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

Five common types of LID (bio retention cells, vegetative swales, rain barrel, and 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).

3.6.1.1. Bioretention cell

In the bioretention cell scenario, runoff from the study area was routed through a bioretention cell in the LID area. (Lucas 2010). Bioretention cells are depressed landscapes into which runoff is directed and allowed to pond, filter, and infiltrate. Some bioretention cells modeled by the LID Sizing consist of the design parameters specified in Section, including a 6’’ (15.2cm) ponding depth underlain by18’’(45.7cm) of bioretention 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 bioretention 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 appropriates in relatively small catchments, typically in the region of 1000-4000 m^2. Several smaller bio-retention areas can be linked together for larger catchments (Endicott & Walker, 2003; Woods-Ballard et al., 2007).

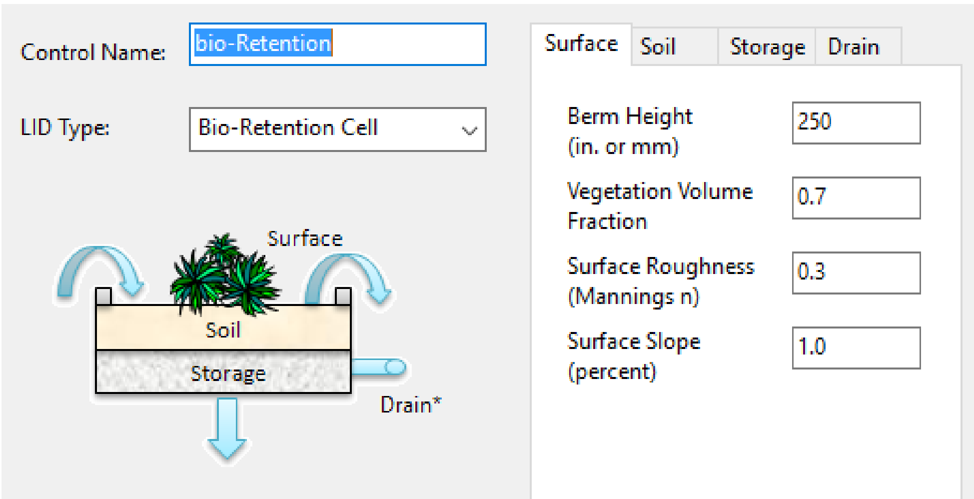

Figure 26: LID control editor in SWMM5 (bio-retention Cell

Table 6: Lists the design elements of the bioretention cell.(Seema Bardhipur.2014).

Layer	**Properties**	**Values**
Surface	Berm Height (mm)	250
	Vegetative Volume Fraction	0.7
	Surface Roughness	0.3
	Surface Slope (%)	1
Soil	Thickness (mm)	1075
	Porosity	0.5
	Field Capacity	0.4
	Wilting Point (volume fraction)	0.1
	Conductivity (mm/hr)	10.9
	Conductivity Slope	10
	Suction Head (mm)	110

Storage	Thickness (mm)	400
	Void Ratio	0.75
	Seepage Rate (mm/hr)	5.08
	Clogging Factor	0.1
Planning parameters	Area of each unit (m^2)	4000
	Number of units	2
	Surface width per (m) unit	1
	%initially saturated	0
	% of impervious area treated	30

3.6.1.2. Infiltration trench

Infiltration trenches are engineered structures that provide storage and facilitate infiltration of runoff into the subsurface. Infiltration trenches are typically long and narrow and filled with aggregate. Runoff from the study area was routed through an infiltration trench in the LID area. Infiltration trenches are excavations backfilled with stone aggregate used to capture runoff and infiltrate it into the ground. They can be simulated as a rectangular, fully pervious sub-catchment whose depression storage depth equals the equivalent depth of the pore space available within the trench.

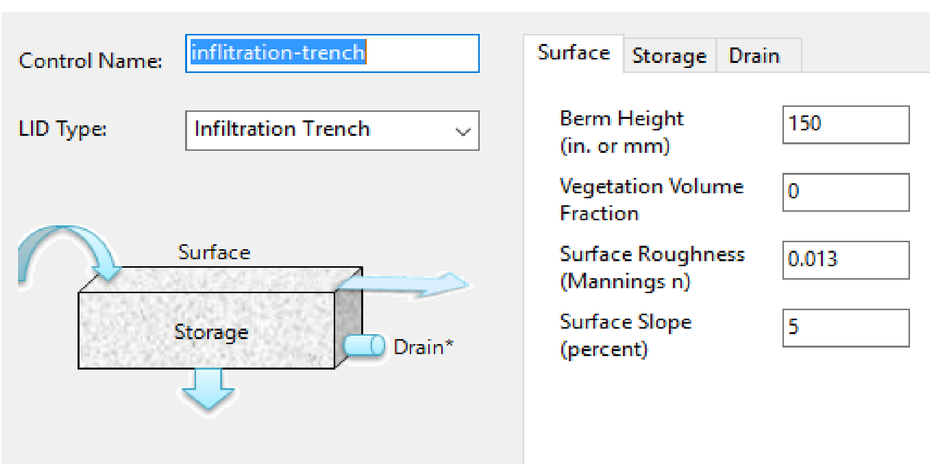

Figure 27: LID control editor in SWMM5 (infiltration trench Cell)

Table 7: Designing and planning parameters of infiltration trench used in this study.

	Layer	Parameter	Unit	Value
Design parameters	Surface	Berm height	Mm	150
		Vegetation volume fraction		0.4
		Surface roughness	Manning's n	0.013
		Surface slope	%	5
	Storage	Thickness	Mm	750
		Void ratio		0.4
		Seepage rate	mm/h	210
		Clogging factor		0.1
	Drain	Flow coefficient		0
		Flow exponent		0
		Offset height	Mm	0
Planning parameters		Area of each unit	m^2	4000
		Number of units		2
		Surface width per unit	M	1
		%initially saturated	%	0
		% of impervious area treated	%	30

3.6.1.3. 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 Bardhipur.2014).

Table 8: Designing and planning parameter of permeable pavement used in this study.

Planning parameters	Area of each unit (m^2)	4000
	Number of units	2
	Surface width per (m) unit	1
	%initially saturated	0
	% of impervious area treated	30

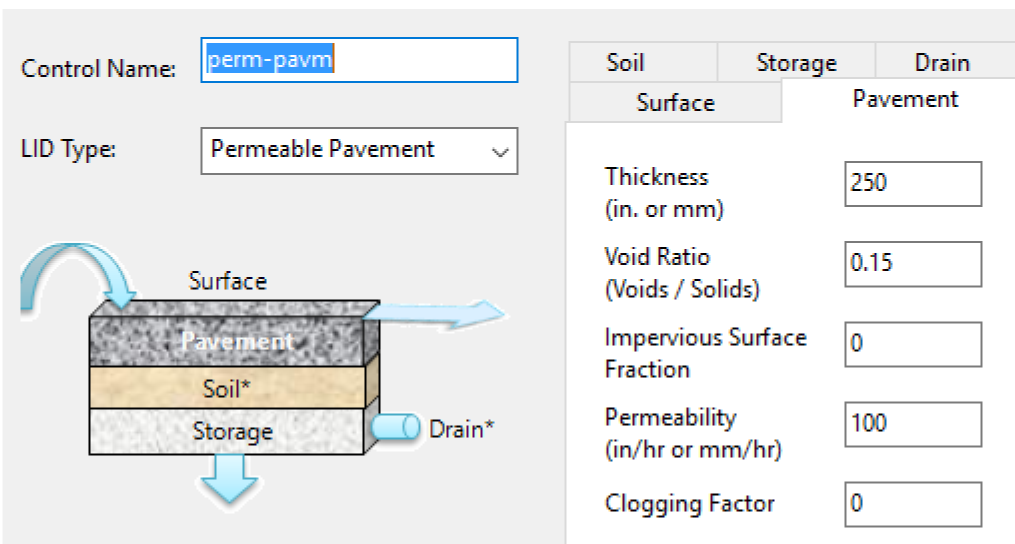

Figure 28 : LID control editor in SWMM5 (permeable pavement Cell)

3.6.1.4. Vegetative swales

Consists of a gently sloped, vegetated channel through which runoff is allowed to sheet flow. The soil underlying the swale is amended with compost to increase its porosity and infiltration capacity, thereby increasing the storage volume within the underlying soils and the infiltration rates into the native soil below. The vegetative swales modeled by the LID include amended depths of 6” (15cm), with a 2% longitudinal slope (in the direction of flow) and side slopes of 3:1 (run: rise). (Seema Bardhipur.2014).

Table 9: Design and planning parameters of Vegetative swales used in this study

Planning parameters	Area of each unit (m^2)	4000
	Number of units	2
	Surface width per (m) unit	3.5
	%initially saturated	0
	% of impervious area treated	30

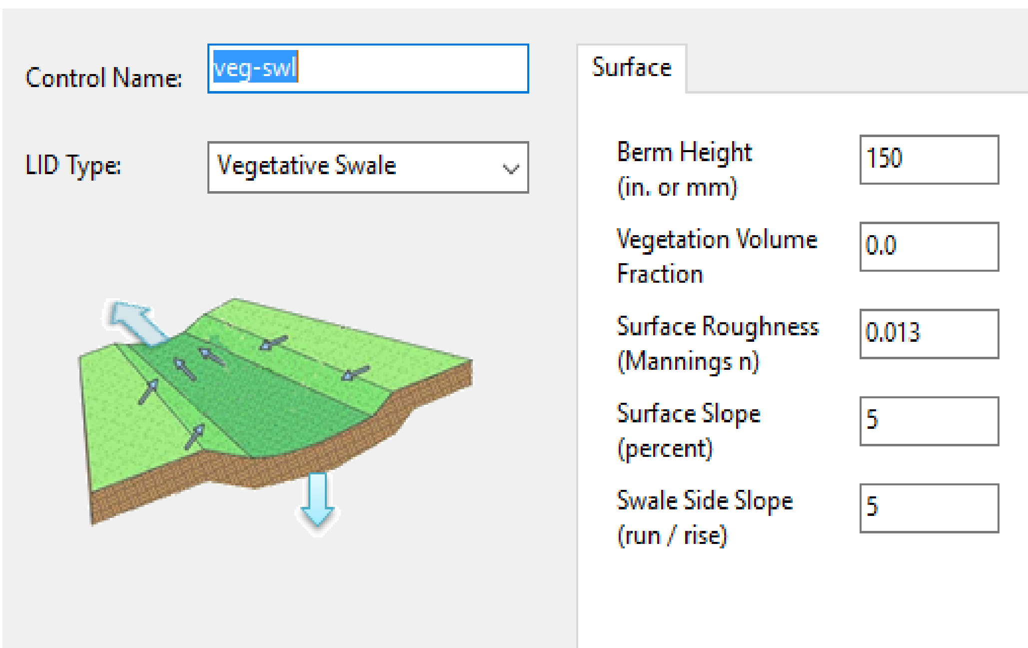

Figure 29: LID control editor in SWMM5 (vegetative swales Cell

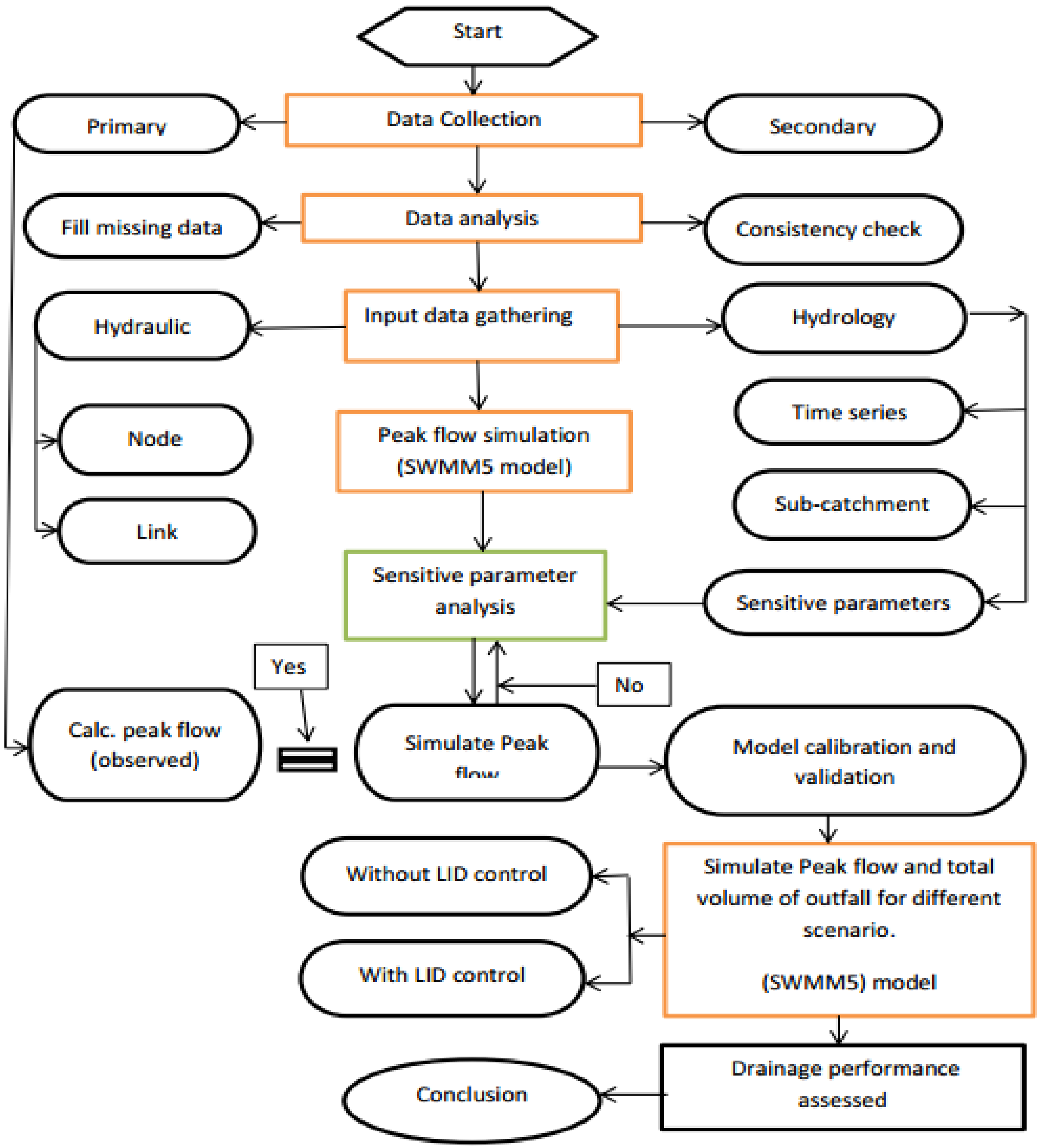

Figure 30: Schematic representation of the methodologies followed.

4. CHAPTER FOUR

4.1. RESULTS AND DISCUSSIONS

4.1.1. Consistency check

Hydrologic data (i.e. rainfall data) were carefully checked for accuracy and consistency as part of the program. And a double mass curves were used to check the consistency of rain gauge record and the correction factor were 0.832. Independent rainfall data obtained from other 3 nearby stations Chacha station, Mendida station and Debre Sina station for selected storm events were used. Shown on, (appendix c).

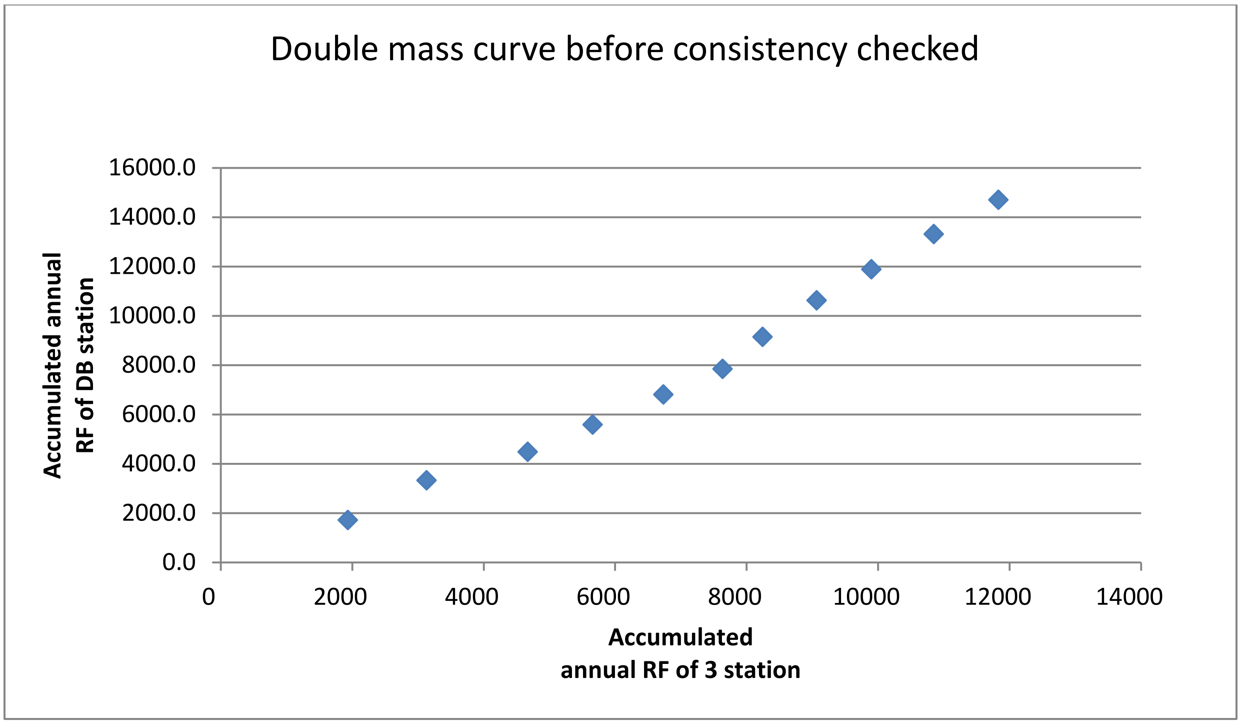

Figure 31: Shows double mass curve before consistency of rainfall data was done.

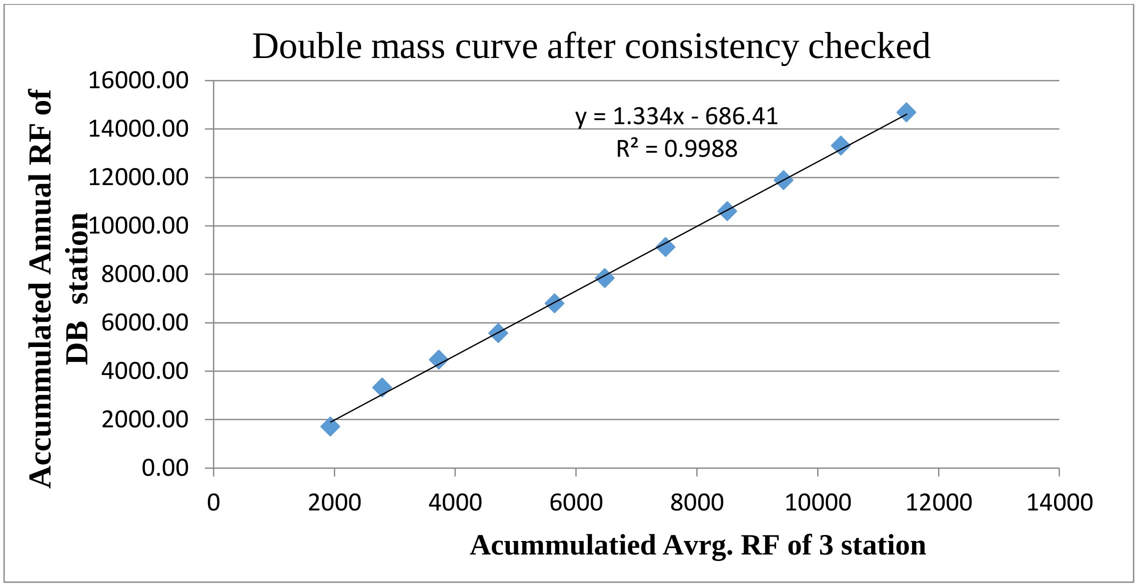

Figure 32 shows double mass curve after consistency of rainfall checked.

4.1.2. IDF curve developed for Debre Berehan station

Debre Berehan drainage system were a 24hr duration rainfall, hence IDF needed to obtain rainfall intensities of shorter duration that SWMM5 used. The input properties of rain gages for SWMM5 were Rainfall data type Intensity, volume or cumulative volume and time interval (.daily, hourly, 15 minute and 5 minute.). For model accuracy 15 minute intensity selected and to have intensity with 15 minute interval IDF developed for different return period to simulate peak flow in the study area for different scenario. As shown in figure 31.

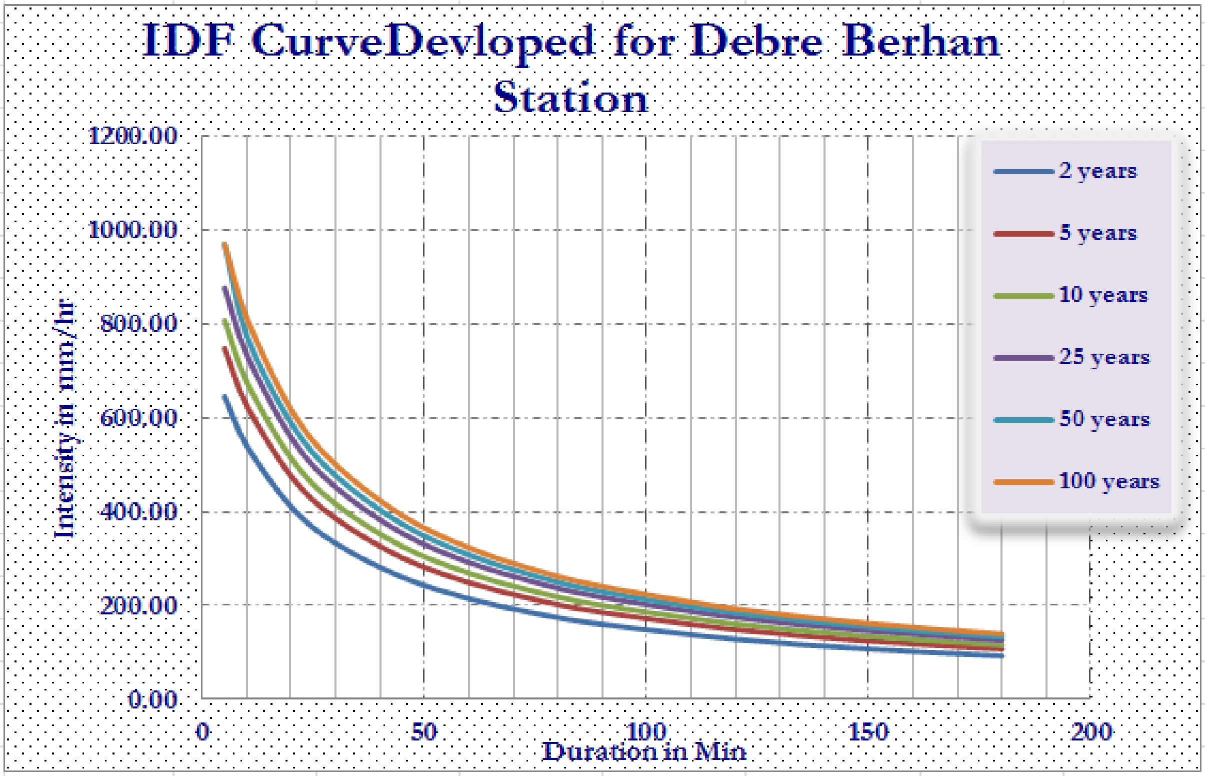

Figure 33: Graph of IDF curve developed for Debre Berhan meteorological station.

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. The IDF curve developed by ERA under rainfall region A2 and the new developed by different distribution method checked by the goodness of fit tests And Log Pearson Type III and EV1 (Gumbel) were more fit and as shown on figure 32. EV1 (Gumbel) was more maximum and used for IDF curve developed.

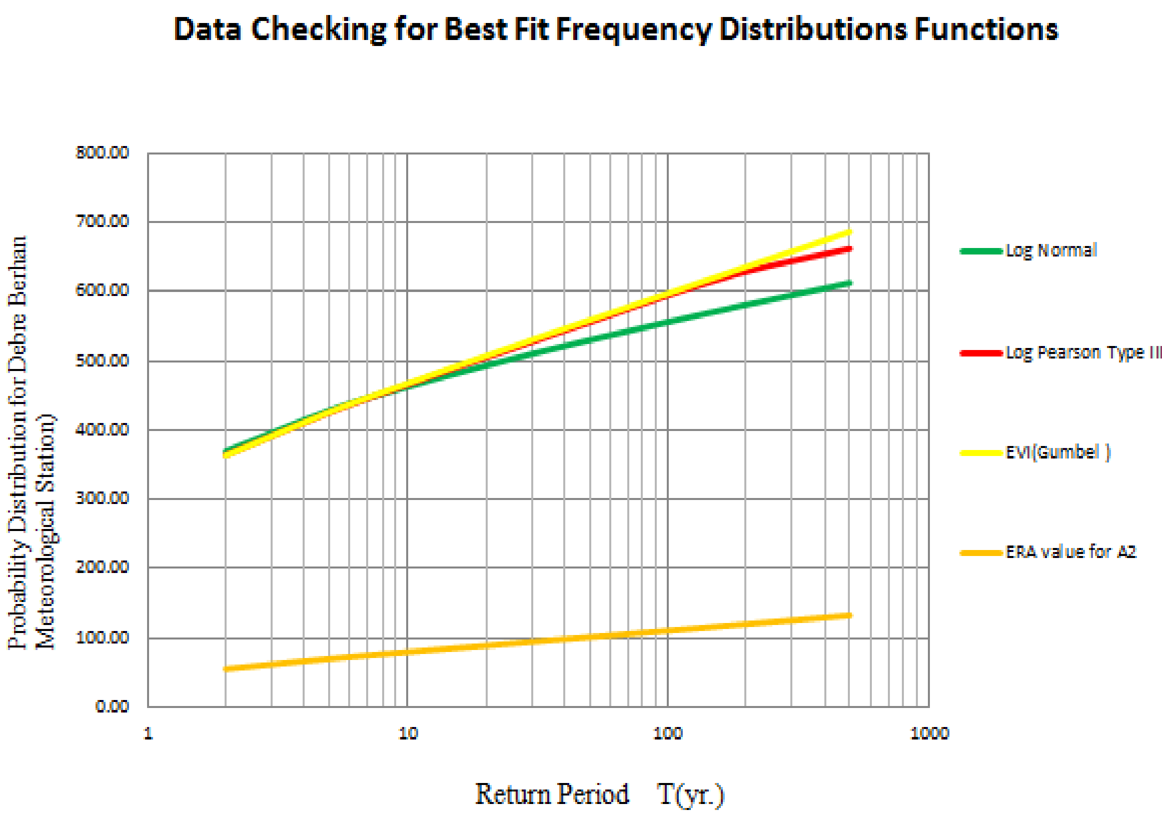

Figure 34: Graph best fit frequency distributions function.

4.1.3. Model calibration and validation

For model calibration sensitive Parameters evaluation and adjusting sensitive parameter was done and the last sensitive values used for the SWMM5 model were show in the table 11.

Table 10: Calibration parameter for SWMM5 hydrology and hydraulic. (Li et al ,2014).

Name Of Parameter	Meaning		Value Range	Initial Values	Used Values/Sensitivity parameters
N-Imperv	Manning's roughness Coeff. For Impervious Area		0.011-0.015	0.011	0.014
N-perv	Manning's roughness Coeff. For pervious Area		0.05-0.8	0.1	0.7
Destore- Imperv	Depth of depression storage on Imperv Area		0-3	1	2.5
Destore- perv	Depth of depression storage on perv Area		3-10	3	8
Smooth concrete Roughness	Manning's roughness Coeff. For open channel with smooth concrete		0.012	0.012	0.012
Infiltration method	Green Ampt	Suction	3.5	3.5	3.5
		Conductivity	0.5	0.5	0.5
		Initial deficit	0.25 – 0.26	0.25	0.26

The model calibrated with the peak flow rate value that obtained by manning equation at the gauged site 0.221 m3/s and the value simulated with SWMM5 model after the sensitive parameters fixed, were 0.277m3/s. and the depth recorded at selected site near out let2 was used for the determined velocity and the flow rate in the drainage for the validation of the model for the area as show in table 12.

Table 11: Determined velocity and flow rate for model validation.

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

for validation

(date)	Gauged site	width(m)	Recorded flow Avrg. depth (m)	elevation defrence (m)	drainage lenth (m)	slope (S)	roughness (n)	Area (m2)	peremeter (m)	hydrulic radius (R)	veloccity (m/s)	discharge (Q) (m3/sec.)
04-05-18		0.5	0.63					0.315	1.76	0.179	0.763	0.220
06-05-18		0.5	0.64					0.32	1.78	0.180	0.765	0.224
18-06-18	Near Outlet 2	0.5	0.72	7.2	2356.7	0.003055	0.023	0.36	1.94	0.186	0.782	0.258
19-06-18		0.5	0.59					0.295	1.68	0.176	0.754	0.203
25-07-18		0.5	0.66					0.33	1.82	0.181	0.770	0.233

Table 12: Simulated and calculated flow rate for validation

(date)	Gauged site	Record avrg.(Head) (m)	Simulated Flow rate (using SWMM5) CMS	Calc.flow rate (CMS) (using Manning's) (observed) CMS
04-05-18		0.71	0.286	0.253
06-05-18		0.64	0.244	0.224
18-06-18	Near Outlet 2	0.72	0.288	0.258
19-06-18		0.59	0.225	0.203
25-07-18		0.66	0.281	0.233

After the model calibrated, 5 day rain fall data used for the model without the sensitive parameter changed and flow rate simulated by model were compared with the flow rate calculated (observed) at the selected site for five days .therefore the result obtained approximately equal and the model was validated. Which, shown on table 12 and graphically on figure 33.

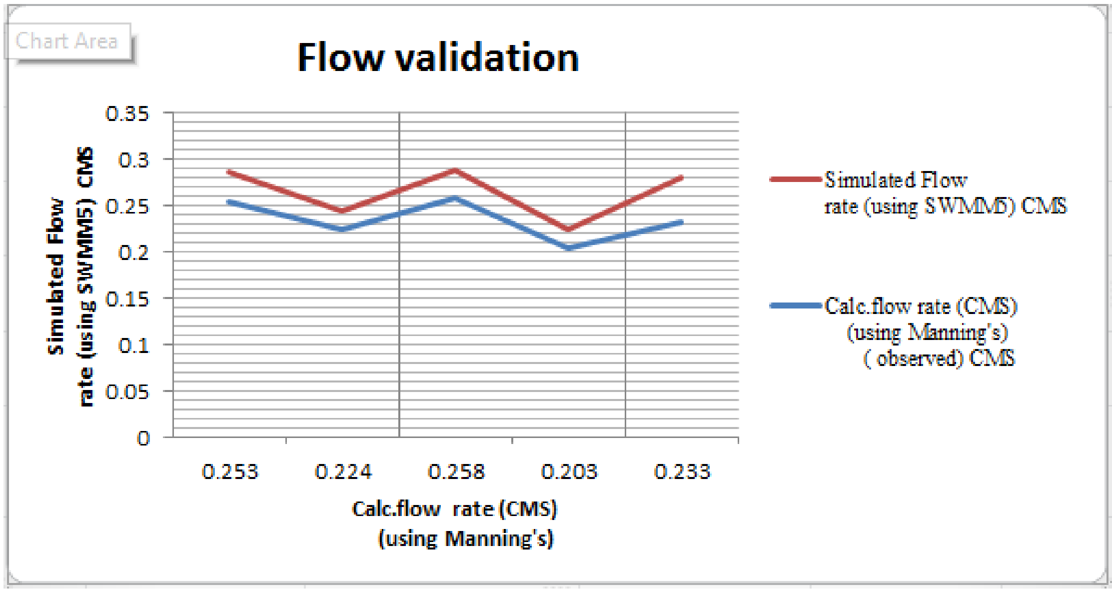

Figure 35: Calculated (Observed) flow rate with Simulated (modeled)

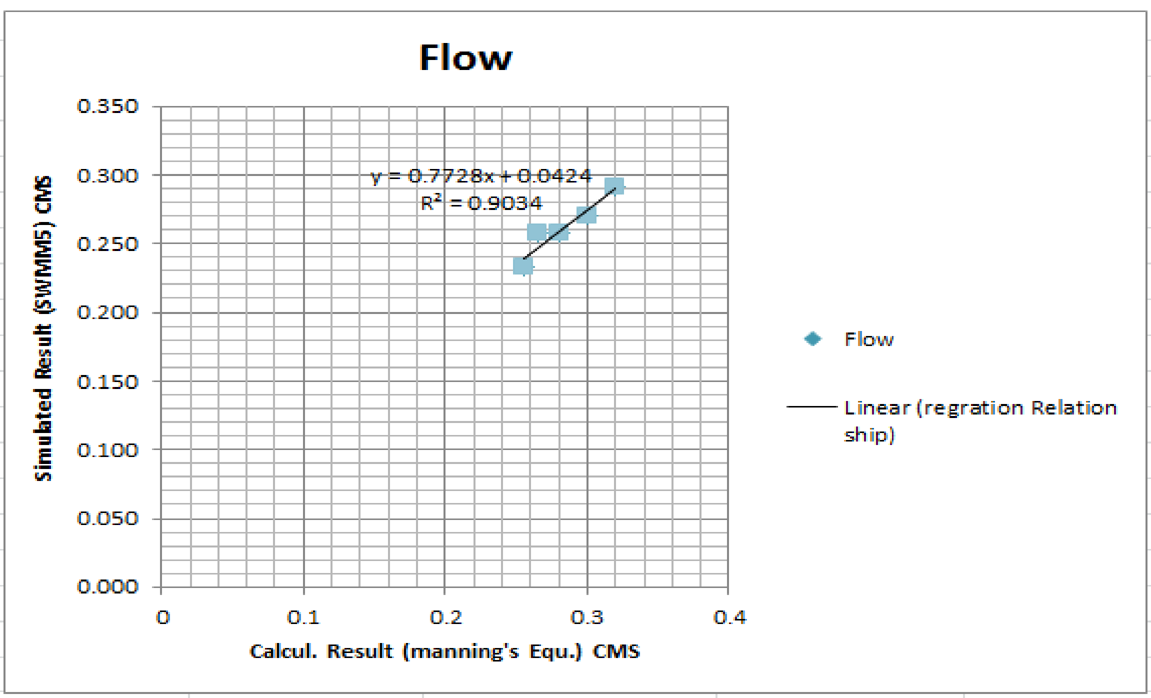

Figure 36: Correlated plot of calculated and simulated Flow

Model performance was measured using the coefficient of determination (i.e. R^2) ,Nash–Sutcliffe model efficiency (Nash and Sutcliffe 1970), and Relative Error .Overall, the calibration resulted in a good fit with observed daily flows for the simulation period, with R^2 = 0.852 ,NSE = 0.935 and relative error (RE) =9.7%.

4.1.4. Runoff simulation for different rainfall event

The amount of runoff generated during the different rainfall events is included in Table 14 as expected; the continuous rainfalls with variable intensity produce more runoff.

Table 13: Total runoff generation for continuous rainfalls with different return period

	Rain Event				
Runoff from each sub- catch. With d/t event(CMS)	T5	T10	T25	T50	T100
S1	0.52	0.58	0.65	0.7	0.75
S2	1.16	1.27	1.4	1.49	1.58
S3	1.17	1.27	1.38	1.46	1.53
S4	4.54	4.93	5.39	5.7	6
S5	3.1	3.37	3.67	3.88	4.08
S6	2.73	2.96	3.21	3.39	3.56
S7	1.37	1.52	1.69	1.82	1.93
S8	3.15	3.42	3.73	3.94	4.14
S9	2.16	2.34	2.55	2.69	2.83
S10	3.27	3.54	3.86	4.07	4.28
S11	1.83	1.99	2.18	2.31	2.43
S12	4.17	4.52	4.92	5.19	5.46
S13	6.53	7.19	7.97	8.51	9.02
S14	8.6	9.47	10.49	11.8	11.88
S15	25.51	28.16	31.26	33.43	35.49
S16	13.82	15.06	16.49	17.49	18.43
S17	22.59	24.72	27.2	28.92	30.54

By comparing both maps it is noticeable that the total discharge is considerably greater in the T25 scenario when compared with T10 scenario. The sub-catchments with higher runoff values are located in the south-eastern and central part of the area, along with a group of 4 sub-catchments in the western part. Both runoff discharges follow the same distribution with the singularity that the T10 discharge is slightly lower. Indeed, the peak discharge is greater during the T25 rain. This means that it is best to consider T25 peak flood occurrence for design of Debre Berhan Drainage system.

4.1.5. Simulation surface runoff results

Different case scenarios have been considered in this study to obtain a fully understanding of the system performance under multiple working conditions. Firstly, the model had been run with the continuous rainfall events with different return periods to analyze the current performance, as shown in Table below shows a summary of the simulations carried out with 25year return period event.

Table 14: Peak runoff at each sub catchment

Sub-catchments	Total Infiltration (mm)	Total Runoff (mm)	Total Runoff 10^6 (Ltr)	Peak Runoff (CMS)	Runoff coeff.
S1	19.11	289.3	8.97	0.76	0.901
S2	16.24	294.45	9.13	1.60	0.917
S3	15.62	295.05	6.79	1.27	0.919
S4	16.27	293.26	26.98	4.93	0.914
S5	1626	293.16	18.18	3.37	0.913
S6	15.19	295.80	15.71	2.96	0.922
S7	15.92	295.50	9.60	1.52	0.921
S8	16.76	289.99	18.27	3.42	0.904
S9	15.22	297.17	12.57	2.34	0.926
10	14.99	297.25	19.02	3.54	0.926
11	16.83	290.73	11.05	1.99	0.906

Table 15 time of peak runoff occurring at each sub catchment

Hours(hr)	S4 Runoff (CMS)	S5 Runoff (CMS)	S6 Runoff (CMS)	S7 Runoff (CMS)	S8 Runoff (CMS)	S9 Runoff (CMS)
3:15:00	6.61	4.45	3.6	2.44	24.09	15.25
6:30:00	0.39	0.26	0.14	0.06	4.07	1.99
9:45:00	0.1	0.07	0.03	0.01	1.45	0.65
13:00:00	0.04	0.03	0.01	0	0.69	0.3

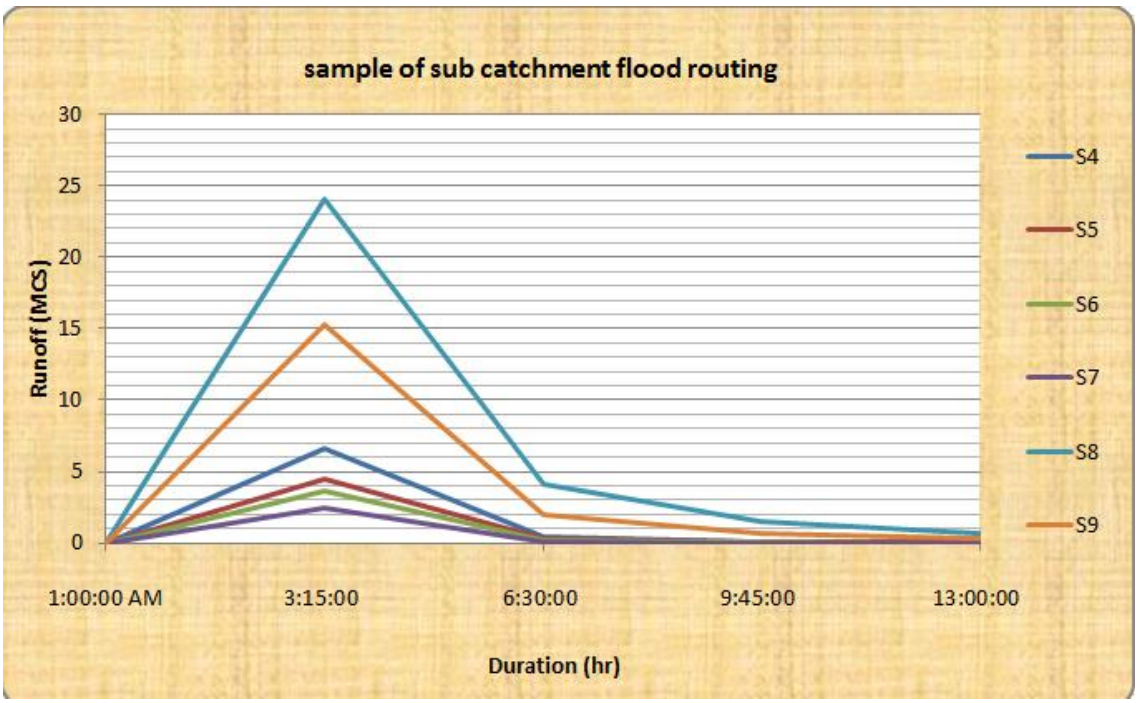

Figure 37: graph of some sub catchment flood for comparison

4.1.6. Network simulation

The drainage systems modeled to cope with a 25- years return period rainfall in terms of water level below the surface in drainage systems. This implies that the flooding risk must be verified in the nodes (manholes) in the systems, whereas water level at each lateral connection must be checked independently with a longitudinal profile in parts of the system which are connected.

The general network performance is determined by infiltration rates and the average water flow production. The water elevation profile in each node was over flooded as we see figure 36, 37 and 38.

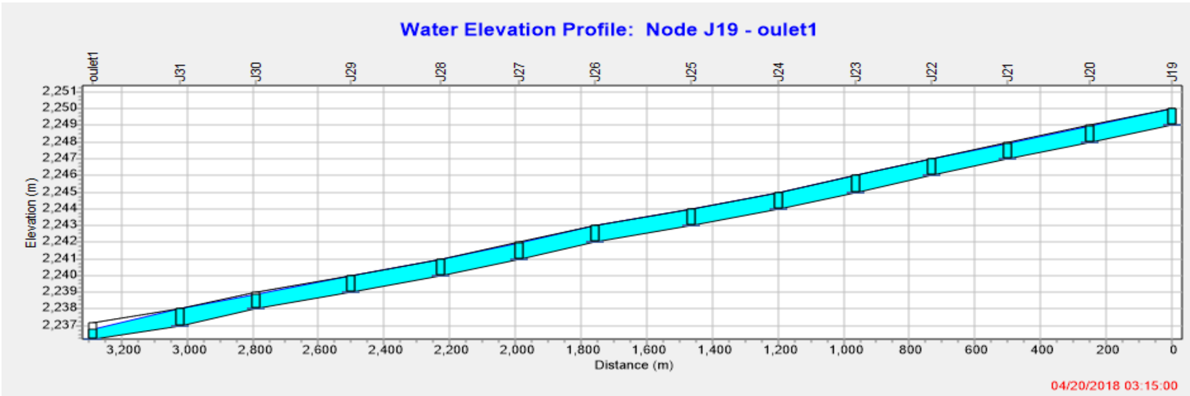

Figure 38: water elevation profile of flooded junction to outlet 1

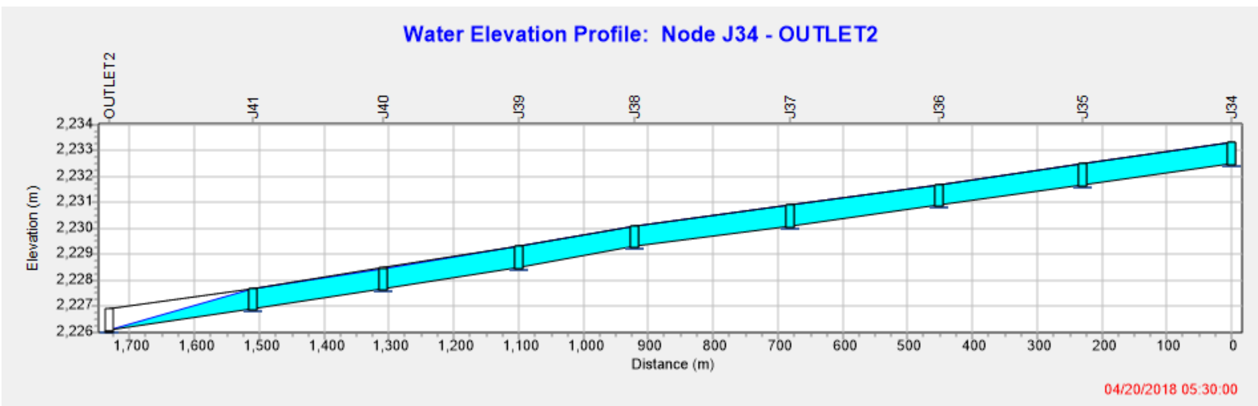

Figure 39: water elevation profile of flooded junction to outlet 2

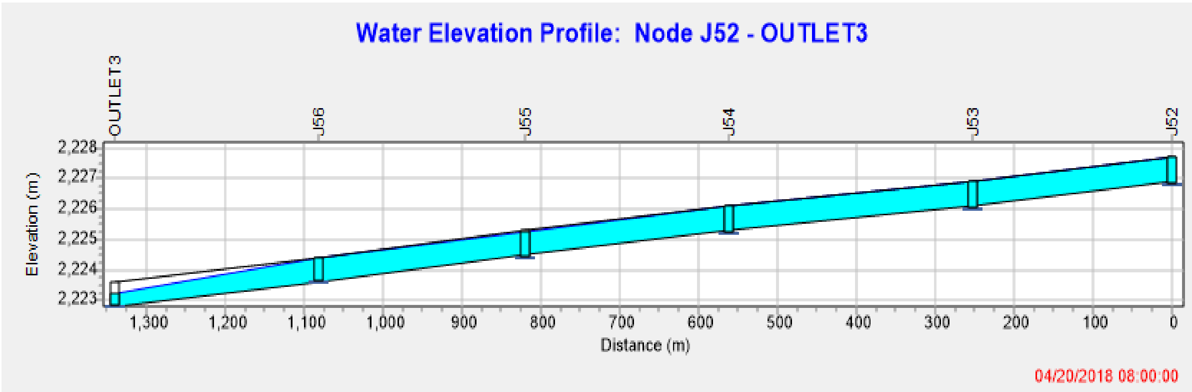

Figure 40: water flow profile of flooded junction to outlet 3

The water profile plot is obtained for nodes from junction J19 to J31, J34 to J41 and J52 to J56 as shown in figure 36, figure 37 and figure 38 respectively. The simulation status report shows that sections between these junctions are surcharged (flooded).

Nevertheless, in order to facilitate the analysis of the flooding risk in the whole area it was assumed that the maximum drainage flooded (water level minus ground level) are representative for the entire junction connected to the link under consideration.so that the link and the junctions are classified in three regions for closed and open rectangular channel.

Closed Rectangular channel water level above surface flooded area (link flooded greater than zero), risky area (link flow varies between zero and minus 1/3 meter) and safe area (link flow smaller than minus 1/2 and minus one meter) and open rectangular channel water level above surface flooding area (link flooded greater than zero), risky area (link flow varies between 0 to -0.8/3 meters) and safe area (link flow smaller than minus 0.4 and minus one meter).

The results obtained in the simulations are gathered in Table 17.

Table 16: Drainage system flooding status in present for different rain event scenario

	Depth Interval	**T5**	**T10**	**T25**	**T50**	**T100**	**Status**
Closed Rectangular channel flooding	b/n (-1/2 and-1)m	59%	60%	40%	20%	14%	Safe
	b/n (-1/3 and 0)m	28%	31.7%	31%	28%	4%	Risky
	>0m	16.2	18.3	35%	59%	72.4%	Flooded
Open Rectangular channel flooding	b/n (-0.4 and-0.8)m	62%	66%	33%	23%	21%	Safe
	b/n (-0.8/3 and 0)m	30%	31.7%	31%	28%	9%	Risky
	>0m	18.4	20.5	32.5%	42%	75.4%	Flooded

According to simulation almost all of the modeled drainage systems are flooded for each event scenario rain. For T25 event 35% of closed rectangular channel and 32.5% of Open rectangular channel were flooded. And also total average flow1.646 m^3/s and total volume to outfall $87.75*10^3$ m^3 were occurred from all 17 sub catchment. Detail show on appendix I.

Table 17: total average peak flow and total volume at each outlet.

	Total average flow in (m^3/s)	Total volume to outfall $*10^3$ (m^3)
Outlet- 1	0.87	46.117
Outlet -2	0.315	17.057
Outlet -3	0.461	24.58

4.1.7. LID scenario simulations

Hydrologic results for scenarios bio retention Permeable pavement, infiltration trenches and vegetative swales are compared with the base case (current conditions) in Table 18 were 25 year design storm used.

Table 18: low impact developing (LID) results

Item	Base case	Bio-retention cell		Permeable pavement		Infiltration trench		Vegetative swales	
		value	%	Value	%	Value	%	Value	%
Peak runoff (m3/s)	1.64	1.224	25.4	1.4	14.2	1.26	23.7	1.59	5.1
Total volume to outfall (10^3) m^3	87.75	54.7	46.2	69.3.	21.2	63.5	34.4	78.1	11.2

In comparison to the current conditions, bio-retension scenario considered on sub catchment S8,S9,S10 , S11 ,S13,S14,S15,S16 and S17 were reduce total outfall volume by 46.2 % and peak flow 25.4%.The infiltration trench scenario as well on the same sub catchment reduced total volume to outfall by 34.4 % and peak flow by 23.7% as compared to current conditions as shown on table 18.

(Kamal Ahmed, 2017) was used to compare the simulation results before and after LID structure installation. The study area infiltration trench was indicates that total volume and peak flow are reduced to (20.95% and 17.5%) respectively for two sub-catchments, S1 (highest by area) and S6 (lowest by area).

Davis (2008) observed that bio-retention can reduce the peak flow from 44% to 66% depending on site conditions such as soil and basin slope with substantially delayed time-to-peak. Therefore, this two scenario what options may be Acceptable in Debre Berhan town drainage system.

4.1.8. Area were LID control applied

The best alternative LID control for Debre Berhan town depending on the total volume and peak flow rate occurrence at the area as SWMM5 model simulation indicates, bio retention were the most recommended .were governmental land at side of the road collected and identified to apply bio retention are about 3.6 ha. As shown in figure below.

Table 19: Area proposed to construct bioretention

Code of area	Area (m^2)
BR1	1300
BR2	1265
BR3	1210
BR4	1158
BR5	662
BR6	828
BR7	1540
BR8	2432
BR9	834
BR10	680
BR11	2620
BR12	590
BR13	1132
BR14	2210
BR15	1280
BR16	1250
BR17	2270
BR18	682
BR19	1350
BR20	2110
BR21	1380
BR22	1258
BR23	1370
BR24	2370
BR25	779
BR26	659
BR27	781

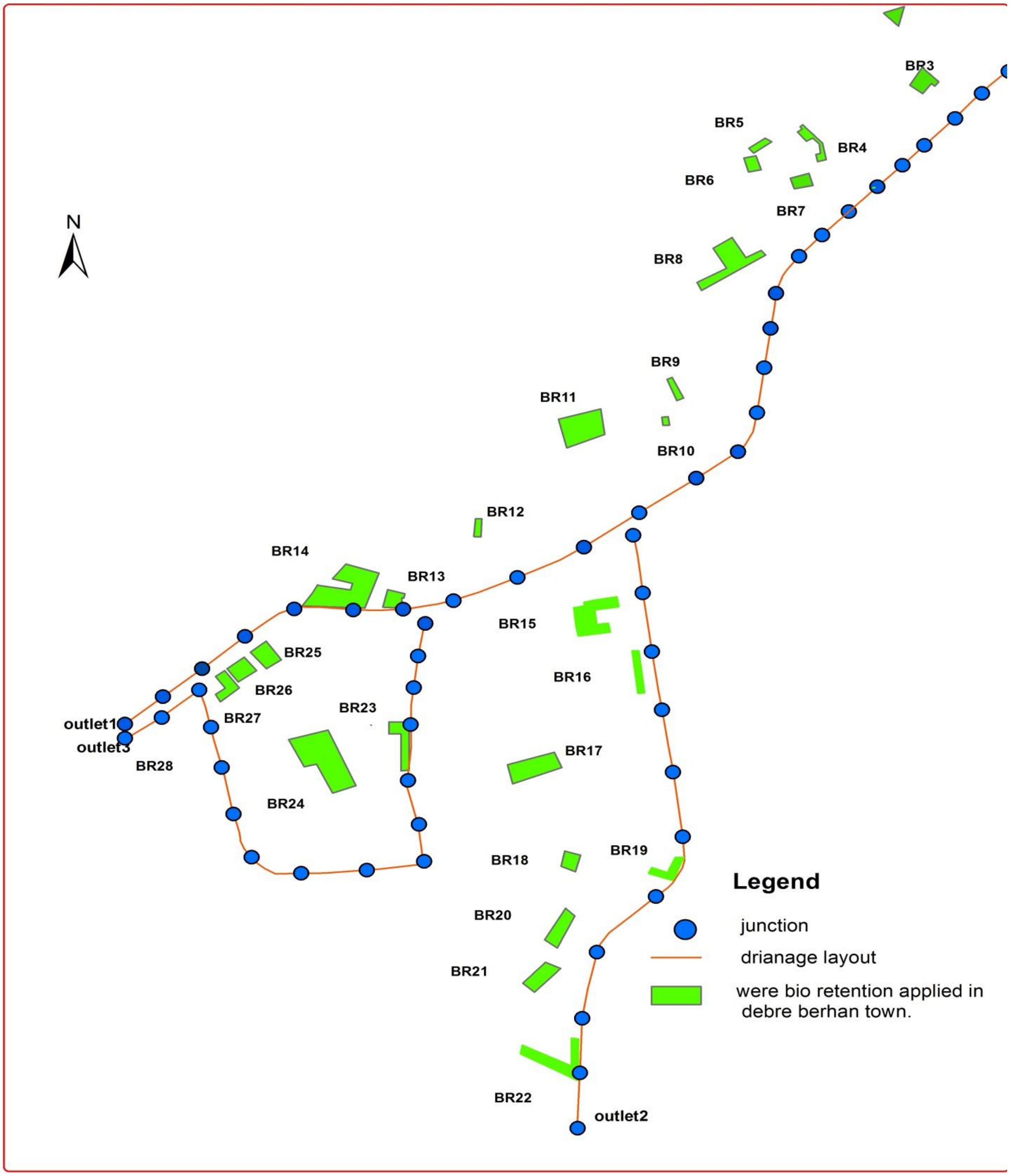

Table 20: layout of Identified bio retention site

Area In Debre Berhan town were bioretenstion proposed to apply

Figure 41:a) Area in Debre Berhan town proposed to apply bio retention

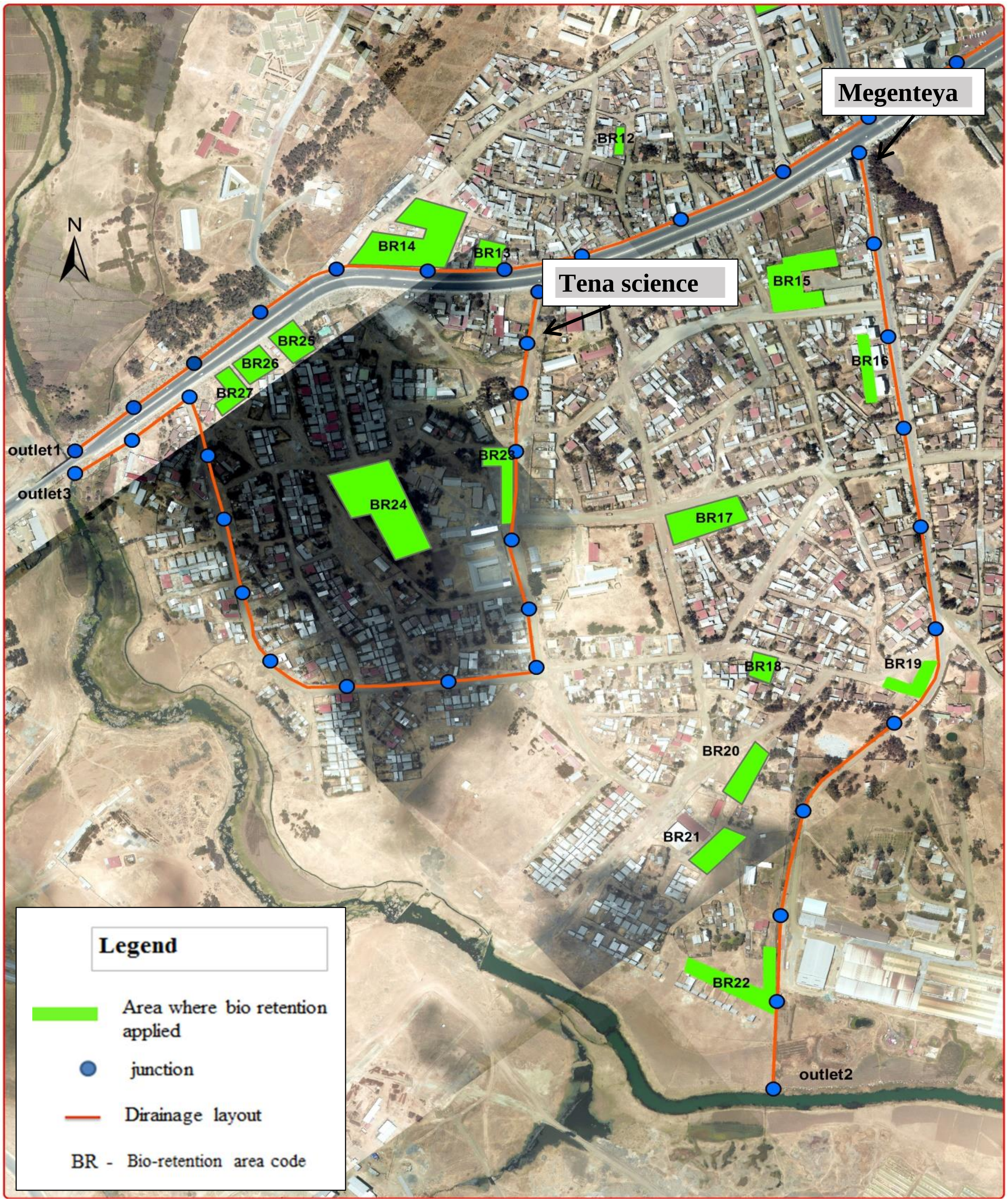

Figure 42:b) Area in Debre Berhan town proposed to apply bio retention

CHAPTER FIVE

5.1. CONCLUSIONS AND RECOMMENDATION

5.1.1. Conclusions

This study assessed the performance of storm water drainage system of Debre Berhan town and peak runoff in urban sub catchments especially on area from Debre Berhan bus station to Beressa River, megenteya to Beressa River and Debre Berhan Tena science to Beressa River. The hydrology was simulated using EPA SWMM5 with and without LID. The model calibration and validation were done by using 10 days recorded flow depth near out let 2 and rainfall data of 10 days parallel to the day that the flow depth recorded were used. 5 day rainfall data used to calibrate the model, where the left 5 day rainfall data used to validate the model with the flow calculated using the parallel recorded depth.

The performance of SWMM5 model for the area was carried out using Coefficient of Determination (R^2) ,The Nash-Sutcliffe coefficient (NSE) and Relative Error (RE) value and the results are in limitations $R^2 = 0.852$,NSE=0.932 and relative error (RE)=9.7 which is quite satisfactory. So the SWMM5 is the powerful tool for analyzing the effective urban flood water control & management for the study area, Debre Berhan town. According to simulation almost all of the modeled drainage systems are flooding for each event scenario rain. For T25 event 35% of closed rectangular channel and 32.5% of Open rectangular channel of the Study Area were flooding. Average flow1.64 m^3/s and total volume to outfall $87.7*10^3$ m^3 were occurred from all 17 sub catchments.

Results of study indicate that LID can be effective for reducing peak flow in Debre Berhan town. In comparison to the current conditions, bio-retension reduced total outfall volume by 46.2%, and average flow by 25.4%, the infiltration trench scenario as well reducing total volume to outfall 34.4% and average flow by 23.7%, Permeable pavement reduced total outfall volume by 21.2% and average flow by 14.2 % and Vegetative swales reduced total outfall volume by 11.2 % and average flow by 5.1% as compared to current conditions.

Therefore Bioretention cells which produced the good level of water management LID implementation attractive in area and increasing interests in LID technologies as a more sustainable approach to urban Stormwater management.

5.1.2. Recommendation

- Provision of proper connections or integrations between the road network and drainage network systems is required with regular maintenance.
- Existing drainage system lay on roads of Debre Berhan University road way not functioning properly at this time. Hence, it is advisable to undertake a comprehensive clearing and maintenance work on the existing storm drain system along with the approach road storm drain system development otherwise there may be a danger of traffic interruption due to concentrated flow condition on the roadway.
- This study properly considered the effect of LID locations and sizes in the modeling processes by introducing relative performance of each LID, the analysis of cost-to-benefit representing percent runoff volume reduction per unit cost could not be conducted due to lack of published cost information. To determine a more cost-effective LID controls, a preliminary cost-to-benefit analysis is suggested to be performed in addition to site characterization.
- Updating the skill of community regarding to LID control is necessary, Because Community participation in flood risk assessment, planning, implementation and care of LID control measures management plans.
- LID is more complicated to model, design, construct, and maintain than its conventional counterparts. Care should be taken when planning and designing LID systems and those considering the use of LID should first count the cost and decide whether it is a worthwhile investment.
- Future research is needed regarding modeling parameters of LID practices and the optimum combinations between the sustainable urban drainage solutions and the conventional measures. Moreover, cost-benefit studies must be included in the design process, in order to determine the feasibility of conventional and LID solutions regarding the achievement of sustainability goals.
- It is batter to document soft copy and hard copy of design document of each drainage system. Because it is difficult to get design document for drainage system constructed before 10 years. (For, ERA).

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Appendix

Appendix A: Rainfall data of Debre Berhan station

Debre berhan Metrological data for Debre Derhan Study area

Name	Elevation	Geogr1	Geogr2	Element	Year	Month	Time	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
Debre Berhan	2750	39.5	9.63	PRECIP	1997	01	09:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1.8	6.1	5.5	14.6	1.5	0	0	0	0	0	0	0	
Debre Berhan	2750	39.5	9.63	PRECIP	1997	02	09:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	0	0	0	0	0	0	0			
Debre Berhan	2750	39.5	9.63	PRECIP	1997	03	09:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	18	7.4	3	0	0	12	0	0	1.1	0
Debre Berhan	2750	39.5	9.63	PRECIP	1997	04	09:00	0	12	0	0	0	0	23.3	4.5	0	0	0	0	0	0	0	0	0	0	0.4	14.7	6.7	18.9	0	1.3	0	0.4	0	0	0.2	0	
Debre Berhan	2750	39.5	9.63	PRECIP	1997	05	09:00	0	0	0	0.6	0.3	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	19
Debre Berhan	2750	39.5	9.63	PRECIP	1997	06	09:00	1.3	0	0	0	0	3.8	1.5	0	1.8	0	0	0	1.8	1.7	1.3	0	0	0	0	5.9	3.2	0.6	8.3	0	20	2	8.2	2.4	15	18	
Debre Berhan	2750	39.5	9.63	PRECIP	1997	07	09:00	5.8	21.2	0	11.8	8.5	19.2	1.2	26.6	7.7	12.4	11.5	21	0	2.8	20.5	14.3	0	2.2	10	1.6	0.8	17.4	32	0	4.8	0	2.4	0	0	15	1.8
Debre Berhan	2750	39.5	9.63	PRECIP	1997	08	09:00	22.1	1.6	1.8	0	0.1	2.6	0	2	0.7	8.3	2.5	0	0.3	7.8	25.8	24.5	9.2	29.8	1.2	25.2	1.8	1.5	13	3.3	0.2	7.7	3.1	4.7	0	0.2	0
Debre Berhan	2750	39.5	9.63	PRECIP	1997	09	09:00	0	0	5.8	0	1.5	6.2	0	0	1.3	0	0	0	0	0.9	7.6	0.9	0.3	0.6	0	0	3.8	1.3	0	0	0	2.6	0.2	1.8	0	0	
Debre Berhan	2750	39.5	9.63	PRECIP	1997	10	09:00	1.6	0	0	0	0	0	0	0	3.7	0	0	0	0	0	0.5	0	14	23.1	0	0.6	1.7	7.3	11	2.3	8.1	3.4	3.5	4.3	3.3	1.9	0
Debre Berhan	2750	39.5	9.63	PRECIP	1997	11	09:00	23.6	23.8	30.7	37.3	7.55	12.2	2.15	3.9	2.8	3.2	35.6	29.9	24.4	18.3	37	12.2	18	4.9	18	9.05	19.7	35.5	30	6.65	25	14.7	23	12	1.8	20	
Debre Berhan	2750	39.5	9.63	PRECIP	1997	12	09:00	1.5	0	15.9	8.1	3.95	0	0	0.7	1.6	2.7	3.35	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Debre Berhan	2750	39.5	9.63	PRECIP	1998	01	09:00	0	2.4	2.1	0	0	0	0	0	0	9	3.5	4	0	0	2.1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Debre Berhan	2750	39.5	9.63	PRECIP	1998	02	09:00	0	0	0	0	0	0	0	0	0	0	0	0.1	0	0	0	0	10	0	0.8	0	0	0	0	0	0	0	0	2.2			
Debre Berhan	2750	39.5	9.63	PRECIP	1998	03	09:00	1.1	0	0	0.1	0.3	1.2	9.9	2.3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Debre Berhan	2750	39.5	9.63	PRECIP	1998	04	09:00	0	0	0	0	0	0	0	0	0	5.2	4.5	0	1.3	6.5	17.7	1.3	7.2	0.1	0	0	0.3	1.5	1.4	0	1.3	0	0	1	0	0	
Debre Berhan	2750	39.5	9.63	PRECIP	1998	05	09:00	0	6.4	14.2	3.1	1.5	0	0	0	0	0	0.4	0.5	7.4	0.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.1	8.9
Debre Berhan	2750	39.5	9.63	PRECIP	1998	06	09:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3.7	0	0	0	7.8	0	0	0	0	2	
Debre Berhan	2750	39.5	9.63	PRECIP	1998	07	09:00	17.5	0.3	0.6	1.8	0	0	0.1	27.2	14	0	10.3	4.3	6.3	3	3.5	13	5.5	17.4	1.2	1.6	4.7	33	13	26	27	24.5	0.5	30	3.4	7.5	40
Debre Berhan	2750	39.5	9.63	PRECIP	1998	08	09:00	2.8	5.9	12.6	1	9.5	12.6	12.2	3.6	19.9	10.2	6.2	4.8	15.9	14.9	12.5	6.1	2.4	0	0.9	2	0	3.3	26	22.9	15	2.5	12	9.2	8	16	18
Debre Berhan	2750	39.5	9.63	PRECIP	1998	09	09:00	0	1.9	13	0	0	9.2	1.5	6.4	0	0	0	0	0	4.3	0	0	0	2.6	0	0	8.5	0	6.5	13	3.7	0	0	0	0	0	
Debre Berhan	2750	39.5	9.63	PRECIP	1998	10	09:00	0	0	0	0	0	0	0	0	3.2	0	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Debre Berhan	2750	39.5	9.63	PRECIP	1998	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	
Debre Berhan	2750	39.5	9.63	PRECIP	1998	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

Debre Berhan	2750	39.5	9.63	PRECIP	1999	01	09:00	0	0	0	0	0	0	0	0	0	0	0	0	0	6.9	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Debre Berhan	2750	39.5	9.63	PRECIP	1999	02	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			
Debre Berhan	2750	39.5	9.63	PRECIP	1999	03	09:00	2	1.8	0	0	10.8	0	2.4	3.2	0	1.2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4.2	0.9	0	0
Debre Berhan	2750	39.5	9.63	PRECIP	1999	04	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	2.8	
Debre Berhan	2750	39.5	9.63	PRECIP	1999	05	09:00	6.5	0	0	0	0	0	0	0	0	0	2.9	0	0	0	0	0	0	0	0	0	0	1.6	0	0	0	0	0	0	0	0	0
Debre Berhan	2750	39.5	9.63	PRECIP	1999	06	09:00	0	2.2	0	0	0	0	0	0	0	0	0	0	0	2	0	0	0	0	0	0	8	7.8	0	0	3.6	1.8	13	11	0	0	
Debre Berhan	2750	39.5	9.63	PRECIP	1999	07	09:00	8.8	5	0	4.3	6.8	20.5	23.2	3.2	8	7.4	28.7	10.5	9.8	11.7	42.4	14	13	4.5	13	2.8	28.8	21.6	4.7	23.2	8.4	9.5	0.4	0.3	13	11	3.5
Debre Berhan	2750	39.5	9.63	PRECIP	1999	08	09:00	0.7	15.7	5.9	45.7	0.9	24.4	3.1	8.4	11.5	9.9	34.2	23.4	4.3	0	7.2	2.1	9.2	0	18	9	2.2	19.6	3.1	13.2	12	26.3	0	19	19	17	1.1
Debre Berhan	2750	39.5	9.63	PRECIP	1999	09	09:00	2.1	0	11.8	5.8	0	0.3	10.1	2.3	0	4.5	0	0.7	3.7	4.1	0	0	0.5	2.4	0	0	2.6	0	0	0.6	0.7	0	0	0	0.2	0	
Debre Berhan	2750	39.5	9.63	PRECIP	1999	10	09:00	0	0	0	0.6	3.6	9.6	11.4	4.2	3	13.4	0	0	0	0	6.6	4.1	0.2	1.3	0.4	1.2	0	0	0	0	0	0	0	0	0	0	0
Debre Berhan	2750	39.5	9.63	PRECIP	1999	11	09:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1.4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	
Debre Berhan	2750	39.5	9.63	PRECIP	1999	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
Debre Berhan	2750	39.5	9.63	PRECIP	2000	01	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
Debre Berhan	2750	39.5	9.63	PRECIP	2000	02	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		
Debre Berhan	2750	39.5	9.63	PRECIP	2000	03	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	4	0	0	0	0	0	22	0
Debre Berhan	2750	39.5	9.63	PRECIP	2000	04	09:00	2.4	0	0	0	0	0	0	0	0	0	0	0	0	0	0.6	15.9	2.8	0	10	0	0.5	0	0	0	0	0	5.8	3.5	1	4.4	
Debre Berhan	2750	39.5	9.63	PRECIP	2000	05	09:00	1.2	0	4.1	1	15.7	7.4	0	7.7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Debre Berhan	2750	39.5	9.63	PRECIP	2000	06	09:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2.4	7.1	13	0	13	5.9	0	4.3	
Debre Berhan	2750	39.5	9.63	PRECIP	2000	07	09:00	8.2	0	2	1.7	14.4	11	1.7	41.1	30.3	0	10.2	33.6	21	15.8	1.5	2.7	6.7	8.8	36	9	1.1	18.1	32	13.4	9.3	16.4	0	2.3	2.6	2	0
Debre Berhan	2750	39.5	9.63	PRECIP	2000	08	09:00	0	5	0.7	2.6	0	17.1	5.5	17.6	2	13.6	30.1	43.8	31	0.8	5	8.5	20	6.8	20	5.1	10	2.3	0	17.4	14	0	0	0	8	31	0
Debre Berhan	2750	39.5	9.63	PRECIP	2000	09	09:00	13.4	0.6	3.1	1.2	14.1	23	0.5	0	1	0	0.3	7.2	5.1	0	3.5	0	0	0	0	0	0	0.8	0	0	0	0	0	5.4	10	16	
Debre Berhan	2750	39.5	9.63	PRECIP	2000	10	09:00	2.1	0	13.1	0	0	0	0	12.7	0	0.2	0	0	0	0	0	0	0	0	0.4	0	0	0	0	0	0	0	0	0	0	0	0
Debre Berhan	2750	39.5	9.63	PRECIP	2000	11	09:00	0	0	0	0	0	0	0.2	0.4	0.3	16.9	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	
Debre Berhan	2750	39.5	9.63	PRECIP	2000	12	09:00	0	0	0	0	0	0	0	0	0	4.8	0.4	0.3	0	0	1.3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Debre Berhan	2750	39.5	9.63	PRECIP	2001	01	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
Debre Berhan	2750	39.5	9.63	PRECIP	2001	02	09:00	0	0	0	0	0	0	0	19.3	4.4	10.1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0			
Debre Berhan	2750	39.5	9.63	PRECIP	2001	03	09:00	0	0	0	0	0	2.6	3.1	3.5	6	8.1	10.9	2.8	0	0	0	2.2	3	4.2	0	0	1	0	4.5	3.7	8.1	2.2	0	0	4.6	0.7	0
Debre Berhan	2750	39.5	9.63	PRECIP	2001	04	09:00	0	0	0	0	0	0	0	0	0	0	0	0	0	1.3	0	0	0	0	0	2	3.1	2.5	0	0	0	0	0.1	6.3	3.5		
Debre Berhan	2750	39.5	9.63	PRECIP	2001	05	09:00	0	2.3	0	0	8.2	0	0	6	0	18.7	1	0	0	0	28	0	0	0	0	0	0	0	0	0	0.4	0	0	0	0	0	0
Debre Berhan	2750	39.5	9.63	PRECIP	2001	06	09:00	0	0	0	0	0	0	0	0	0	0.1	1.6	0	0	0	0	0.8	0.2	11.6	0	0	0.4	0.3	0	0	14	4.9	1.2	0	0	0	
Debre Berhan	2750	39.5	9.63	PRECIP	2001	07	09:00	14.3	4.9	15.2	0	12.2	14.4	9.2	11.7	7.3	1.6	0	10.2	30.1	20	13.6	24.7	8	16.6	24	18.8	15.9	9	5.1	22.7	7.2	2.8	20	38	2.3	3.3	24
Debre Berhan	2750	39.5	9.63	PRECIP	2001	08	09:00	25.4	0	24	6.1	5.9	1.3	8	31.3	31	4.8	0	0.4	1	5.3	13.4	2.2	0.3	0	0	1.4	0.7	8.6	11	0	18	0.5	9.8	14	1	18	17
Debre Berhan	2750	39.5	9.63	PRECIP	2001	09	09:00	6.4	0	0	1.2	0.5	0	0.5	12.8	0.4	0	5.2	3	0.2	0.5	0	0	0	0	0	0	0	0	0.3	1.2	0	0	0	0	0	0	
Debre Berhan	2750	39.5	9.63	PRECIP	2001	10	09:00	0.3	0.5	0	0	0	0	3	0	0	0	0	0	0	0	0.3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Debre Berhan	2750	39.5	9.63	PRECIP	2001	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	
Debre Berhan	2750	39.5	9.63	PRECIP	2001	12	09:00	0	0	0	0	0	0	2.3	1.1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Debre Berhan	2750	39.5	9.63	PRECIP	2002	01	09:00	2.8	12.3	3	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
Debre Berhan	2750	39.5	9.63	PRECIP	2002	02	09:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2.9	22	3.6	0	0	0	0	0	0	0	0	0	0			
Debre Berhan	2750	39.5	9.63	PRECIP	2002	03	09:00	1.2	21.8	5.7	5.6	1.6	0	0	0	0	0	0	0	0	0	3.4	4.7	0	8.5	0	0	0	0	0	0	0	0	8.1	0	0	0	0
Debre Berhan	2750	39.5	9.63	PRECIP	2002	04	09:00	0	0.3	0	5.7	2.6	0	0	0.4	14.9	0.5	0	0	0	0	0	0	0	15.4	6.3	0	0	0	0	0	0	0	0	0	0	0	
Debre Berhan	2750	39.5	9.63	PRECIP	2002	05	09:00	0	0	2.1	0	5.3	0	0.5	0	0	0	0	0	0	0	0	5	0	0	4.1	1	0	0.4	0	0	0	0	0	0	0	0	0
Debre Berhan	2750	39.5	9.63	PRECIP	2002	06	09:00	0	0	0	7.3	0	0	0	0	2.8	0	0	0	0	3.1	0.7	0	0	0	0	0	14.5	0.7	0	0	0	0	0	0	0	0	
Debre Berhan	2750	39.5	9.63	PRECIP	2002	07	09:00	0	0	0	0	0	0	0	0	0	2.5	8.2	0	18	2.8	5.4	0	1	3	0.5	1.2	0	0.2	20	34.3	1.8	18.1	19	21	22	27	8.1
Debre Berhan	2750	39.5	9.63	PRECIP	2002	08	09:00	0.6	31.3	0.2	13.5	21.8	23.2	0.1	11.8	2.1	18.5	21.2	2.1	28.4	19.8	0	16.6	17	20.6	4.2	0.7	4.4	3.2	3.5	5	1.2	0	9.9	1.2	0	2.5	10
Debre Berhan	2750	39.5	9.63	PRECIP	2002	09	09:00	14.5	2.6	4.9	1.5	0.6	25	0.3	40.6	0	0	0	0	0	2.5	5.8	1.9	0.4	4.8	0	0	0	0	3	0	0	0	0	0.7	0	0	
Debre Berhan	2750	39.5	9.63	PRECIP	2002	10	09:00	0	0	0	0	0.8	2.3	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
Debre Berhan	2750	39.5	9.63	PRECIP	2002	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	
Debre Berhan	2750	39.5	9.63	PRECIP	2002	12	09:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1.5	0	0	0	0	0	0	0	0.4	0.5	0.2	0.2	0	5.4	0.2	0	0
Debre Berhan	2750	39.5	9.63	PRECIP	2003	01	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	2.6	11.5	0	1.5	0	0	0
Debre Berhan	2750	39.5	9.63	PRECIP	2003	02	09:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	7.2	15.6	0.7	0.2	13	0			
Debre Berhan	2750	39.5	9.63	PRECIP	2003	03	09:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.8	0	1.9	0	1.2	36.2	0	0	3.4	0	0	0	0	12	4.7
Debre Berhan	2750	39.5	9.63	PRECIP	2003	04	09:00	0	0	0	0	0	0	0	0	0	0	0	6.6	0.2	9.3	15.1	0	0.6	6	0.2	0	32.5	0.3	1.7	1.1	10	0	0.7	1	0	0	
Debre Berhan	2750	39.5	9.63	PRECIP	2003	05	09:00	0	0	0	0	0	0	0	0	1.4	0	0	0	0	2.4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Debre Berhan	2750	39.5	9.63	PRECIP	2003	06	09:00	0	6	0	0	0	7.4	0	0	0	0	0	0	0	0	0	0	0	0	0	1.9	20.4	20	2.4	0	0.4	5.3	0	8.7	0	27	
Debre Berhan	2750	39.5	9.63	PRECIP	2003	07	09:00	0.2	1.8	22	4.8	6	3.1	16.2	0.2	0	26.1	18.7	1.5	36.7	5.2	2.4	0	12	0.5	1	4.4	23.6	8.2	29	9.4	12	15.6	20	21	10	5	17
Debre Berhan	2750	39.5	9.63	PRECIP	2003	08	09:00	25.3	5.4	4.8	4.5	7.8	3.6	47.8	15.3	19.7	1.9	0.5	1.2	3.9	2.8	1.7	17	0.2	0.1	13	7.1	45.2	3.1	22	0.4	3	7.3	0.6	14	1.8	0.2	8.2
Debre Berhan	2750	39.5	9.63	PRECIP	2003	09	09:00	6.4	4.6	0.2	1	0.7	4.5	1.4	2.7	3.1	0.2	3.5	4.4	0.5	8.2	1.4	0	0	0	0	3.6	5.2	0.5	6.4	3.9	0	0.9	0.7	6.4	0	3.8	
Debre Berhan	2750	39.5	9.63	PRECIP	2003	10	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
Debre Berhan	2750	39.5	9.63	PRECIP	2003	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	
Debre Berhan	2750	39.5	9.63	PRECIP	2003	12	09:00	0	0	0	0	0	6.6	0.8	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Debre Berhan	2750	39.5	9.63	PRECIP	2004	01	09:00	0	0	0	0	0	0	0	0	0	0	0	0	8.3	0.2	0	0	0	0	12	0	1	0	0	1.8	0	0	0	0	0.8	0	0
Debre Berhan	2750	39.5	9.63	PRECIP	2004	02	09:00	6.5	3.2	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		
Debre Berhan	2750	39.5	9.63	PRECIP	2004	03	09:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	6.6	14.1	0	0	0	0	0	0	0	1.2	4.9	0	1.2	1.7	0
Debre Berhan	2750	39.5	9.63	PRECIP	2004	04	09:00	0	0	1.2	32.5	0	0.8	3.4	0.7	1	12.6	0	0	4.2	11.3	6.8	0	0.2	0.5	0	18.3	0	0	0	0	0	0	0	0	0	20	
Debre Berhan	2750	39.5	9.63	PRECIP	2004	05	09:00	0	5.6	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
Debre Berhan	2750	39.5	9.63	PRECIP	2004	06	09:00	0.7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	23.6	2.5	0	3.8	7	6.2	0	17	30.4	0.2	5.7	1.2	1	
Debre Berhan	2750	39.5	9.63	PRECIP	2004	07	09:00	13	0	0.9	3.6	15.6	2.5	0.4	0	0	0	45.6	6.5	12.2	29.8	6.6	13.7	15	14.5	14	8.3	8.8	6.6	19	31.7	17	17.4	3.3	5.6	17	2.5	2.8
Debre Berhan	2750	39.5	9.63	PRECIP	2004	08	09:00	27.6	7.8	14.3	3.2	10.3	7	31.8	3	21.8	15.8	0	41.3	23.2	3	31.2	10.2	0.5	0	0.2	22.6	10.5	2.2	0	0	1.5	1.7	7.3	2	0.4	0	0.9
Debre Berhan	2750	39.5	9.63	PRECIP	2004	09	09:00	8.5	0	1.7	0.2	10	0	0	0	0	0	0.2	12	0	11.4	0.3	2.5	7.7	12	6.6	3.8	0	0	0	0	2	0	0	0	0	0	
Debre Berhan	2750	39.5	9.63	PRECIP	2004	10	09:00	0	0	3.6	0	0.3	5.5	0.4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4.3	0	0	0	0	0	0
Debre Berhan	2750	39.5	9.63	PRECIP	2004	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	12	0	0	0	0	0	0	0	
Debre Berhan	2750	39.5	9.63	PRECIP	2004	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

Debre Berhan	2750	39.5	9.63	PRECIP	2005	01	09:00	0	0	0	0	0	0	0	0	0	0	0	11.8	2.6	9.8	0.8	0	0	0	0	0	0	0	0	0	6.6	2.7	0	0	0	0	0
Debre Berhan	2750	39.5	9.63	PRECIP	2005	02	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	4.5	0	0			
Debre Berhan	2750	39.5	9.63	PRECIP	2005	03	09:00	0.5	0	2.8	0	0	0.3	8.6	0	0	0	0	0	0	0	0	12.7	1.7	0	0	0	0	2	0	0	0	0	0	0	0	0	0
Debre Berhan	2750	39.5	9.63	PRECIP	2005	04	09:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0.8	0	10.8	11	10	2	2.1	0.7	0	6.2	0	1.6	3.8	0.3	0	0	0	
Debre Berhan	2750	39.5	9.63	PRECIP	2005	05	09:00	2.7	0.5	8.5	1.2	0.2	0	0	22.8	0	0	12.2	0.2	0	0	0	0	0	8.7	0	11.1	0	0	4	0.2	0.3	0.5	0	0	3.3	0	0
Debre Berhan	2750	39.5	9.63	PRECIP	2005	06	09:00	0	0	0	3.5	0	0	4.2	7.3	0	0	0	0	0	0	0	0	0	0	1.5	0.4	2.9	10.7	5.2	1.5	0.6	8	4.6	38	0	2.7	
Debre Berhan	2750	39.5	9.63	PRECIP	2005	07	09:00	0.4	0	0.4	15.7	0	27.6	13	1.6	13.8	39.7	5.2	16.5	0.6	11.7	5.4	0	46	15.6	1.5	26.4	3.5	1.8	0.3	16	0.8	15	2	30	0.4	0	0
Debre Berhan	2750	39.5	9.63	PRECIP	2005	08	09:00	9.2	0.8	6.4	1.8	3	0.2	3.6	2.3	27.2	0	3.7	3.5	3.3	0	0	14	20	14	11	16	0.8	28.5	4.8	4.2	0	31.5	1.8	0.8	1.7	10	4.2
Debre Berhan	2750	39.5	9.63	PRECIP	2005	09	09:00	0.2	16.2	0.2	3.4	12.7	18.5	0	2.7	1	2.7	0.2	13.7	2.8	1	12	0	0	0.5	3	0	0	0	1.2	0.5	0	0.2	14	0	0	0	
Debre Berhan	2750	39.5	9.63	PRECIP	2005	10	09:00	0	0	0	0	0	0	0	0	0	0.2	0	0.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Debre Berhan	2750	39.5	9.63	PRECIP	2005	11	09:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	
Debre Berhan	2750	39.5	9.63	PRECIP	2005	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
Debre Berhan	2750	39.5	9.63	PRECIP	2006	01	09:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	17.3	0	0	0	0	0	0	0	0	0	0
Debre Berhan	2750	39.5	9.63	PRECIP	2006	02	09:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	7	2.4	0	0	12	0	0	0	0	0			
Debre Berhan	2750	39.5	9.63	PRECIP	2006	03	09:00	0	0	0	0	0	0	0	0	0	0	0.4	1.5	0	4.2	0	0	0	7	0	0	0	0.2	0	0.2	9.6	0	0	7.5	18	0.2	12
Debre Berhan	2750	39.5	9.63	PRECIP	2006	04	09:00	8	8.7	0.4	2.5	0	0.4	1	0	0	0	0	0	0	0	0	0	0	0	0	0.5	12.6	0	0	0	0	0	0	4.2	0	0	
Debre Berhan	2750	39.5	9.63	PRECIP	2006	05	09:00	0	0	5.1	6.8	0	0	0	0	0	0	0	0	0	0	0	1.2	2	0	0	0	0	0	0	2.2	2.5	0	0	0	0	0	0
Debre Berhan	2750	39.5	9.63	PRECIP	2006	06	09:00	0	0	0	0	0	0	0	0	0	0	11.8	5	0	0	0	0	3.8	0.5	0	0	4	1.8	0	0	0	0	0	1	7.3	0	
Debre Berhan	2750	39.5	9.63	PRECIP	2006	07	09:00	0.6	0.4	2.8	2.6	41.2	29.5	1.6	19.8	42.5	22.5	31.3	26.5	12	1.2	9.2	2.8	1.5	18.5	0	12.7	2.4	17.3	9	11	7.6	3	7.5	44	6.5	15	31
Debre Berhan	2750	39.5	9.63	PRECIP	2006	08	09:00	25.8	0.8	11.2	18	5.9	8.2	22.5	1.5	6.8	8.5	6	7.6	8.5	13.5	5.8	0	1.2	2.6	3.4	0.4	14.3	14	15	2	9.1	3	0.3	2.4	0	5.4	0.5
Debre Berhan	2750	39.5	9.63	PRECIP	2006	09	09:00	2.3	5	1.8	4.2	0	13.3	5	0	0.5	10.2	4.3	0	0	3.2	3	0	4.7	1.7	0	0	0	0.4	0	0	0.2	0	0	0	0	0	
Debre Berhan	2750	39.5	9.63	PRECIP	2006	10	09:00	0	2	0	0	0	0	0	0	0	0	0	0.8	0.8	0	0	0	0	0	0	0	0	0	0	0	0	0.2	0	4.8	0	0	0
Debre Berhan	2750	39.5	9.63	PRECIP	2006	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	0.6	
Debre Berhan	2750	39.5	9.63	PRECIP	2006	12	09:00	9	0.3	0	1	16	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
Debre Berhan	2750	39.5	9.63	PRECIP	2007	01	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	2	0	0	0	0	0	0	0
Debre Berhan	2750	39.5	9.63	PRECIP	2007	02	09:00	24.2	0	4.6	0	0	0.4	0	0	0	0	0	0	0	0	0	0	0	1.2	0	0	0	0	0	0	0	0	0	0			
Debre Berhan	2750	39.5	9.63	PRECIP	2007	03	09:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.5	0	0	1.6	3.2	0	0	0	0	0	0	0	2.6	1
Debre Berhan	2750	39.5	9.63	PRECIP	2007	04	09:00	0	0	0	3.2	0.3	4.5	0	0	10.1	17.3	20.7	1.5	1.6	7.4	0	0.7	0	0.4	2.2	0	0	0	0	1.7	0.2	0	0	0	0	0	
Debre Berhan	2750	39.5	9.63	PRECIP	2007	05	09:00	0	0	0	0	0	0	0	0	0	0	0	0	0	4.1	7.3	0.2	0	0	0.5	0	0	0	0	0	0	0	0	1.4	0	0	0.1
Debre Berhan	2750	39.5	9.63	PRECIP	2007	06	09:00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1.8	6.3	1.7	4.6	0.7	0.6	2	2.6	13	8.3	0	24.6	13	0	0	14	
Debre Berhan	2750	39.5	9.63	PRECIP	2007	07	09:00	19.6	1	5	3.8	1.2	0	16.2	20.8	0.8	5.4	9.8	0	5.8	5.2	10	0.2	0	1.3	30	26.2	10.8	12.5	18	14.6	4	16.3	6.2	21	9.4	7.2	28
Debre Berhan	2750	39.5	9.63	PRECIP	2007	08	09:00	6.6	9.2	15.4	3.2	33.7	15.7	6.8	1.4	15.5	2.6	4.6	1.2	8.5	15.7	20	20.5	30	0	16	34	0.8	25.5	28	11.3	0.8	0	5.2	57	12	8.8	4.8
Debre Berhan	2750	39.5	9.63	PRECIP	2007	09	09:00	5.3	12.7	17.5	6.2	5.2	23	0.4	13.5	0	22.4	0.3	0	0.2	1.4	6.8	9	0.2	0	4.4	0	0	0	0	0	0	0	0	0	0	0	
Debre Berhan	2750	39.5	9.63	PRECIP	2007	10	09:00	1.6	0.9	0	0	0	0	0	0	0	0	0	0	0	0	1.5	0	0	0.9	0	0	0	0	0	0	0	0	0	0	0	0	0
Debre Berhan	2750	39.5	9.63	PRECIP	2007	11	09:00	0	0	0	0	0.4	2.6	1.2	1.3	0.2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	
Debre Berhan	2750	39.5	9.63	PRECIP	2007	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

Appendix B: Different models used for storm water runoff simulation

Models	Continuous Or Event Model	Impervious And Pervious Area Lumped Or Separated	Loss Model Overland Flow	Routing Method	Pipe Routing Method	Water Quality Parameters simulated?	Output	Low Impact Development (LID)
CIVIL CAD	Event	Separate	Time-area	Horton	Manning's Equation	No	Hydrographs at each pit can be modeled	X
SWMM5	• Event or • Continuous	Separate	• Horton • Green-Ampt	Nonlinear reservoir	Kinematic wave Dynamic wave	Yes	Generated runoff hydrographs at defined points	√
MOUSE	• Event or • Continuous	Lumped	Horton	•Time-area •Kinematic wave	•Kinematic wave •Diffusive wave	Yes	generated runoff	X
RAFTS	Event or Continuous	Separate	Philip's equation or ARBM model	Laurenson's (1964) runoff routing procedure	Muskingum method or Time-lag	No	Generated runoff hydrographs at defined points	X
STORM	• Event or • Continuous	Lumped	• Runoff coefficient •SCS method	Not considered	Not considered	Yes	One hydrograph for single catchment	X

Appendix C: Accumulate annual RF for DB station and Avrg accumulate of three stations

Year	debre berehan annual	Accummulative of Debre Berhan station	debre sina	mendida	chacha	Avrg.3 station	Avrg.Acummulative of 3 station
1997	1935.8	1935.8	3592.7	763.1	763.1	1706	1706.30
1998	859.1	2794.9	2421.2	1202.9	1202.9	1609	3315.30
1999	937	3731.9	2140.9	657.5	657.5	1152	4467.27
2000	985.3	4717.2	2282.3	514.2	514.2	1104	5570.83
2001	930.1	5647.3	1603.2	1027.73333	1027.7	1220	6790.39
2002	830.1	6477.4	1286	921.6	921.6	1043	7833.46
2003	1005.5	7482.9	1737	1070.6	1070.6	1293	9126.19
2004	1023.2	8506.1	2028	1206.8	1206.8	1481	10606.72
2005	932.4	9438.5	1952.9	929.1	929.1	1270	11877.09
2006	948.1	10386.6	2151.6	1066.4	1066.4	1428	13305.22
2007	1083.5	11470.1	1879.8	1128.7	1128.7	1379	14684.29

Appendix D: Summary of the probability distribution

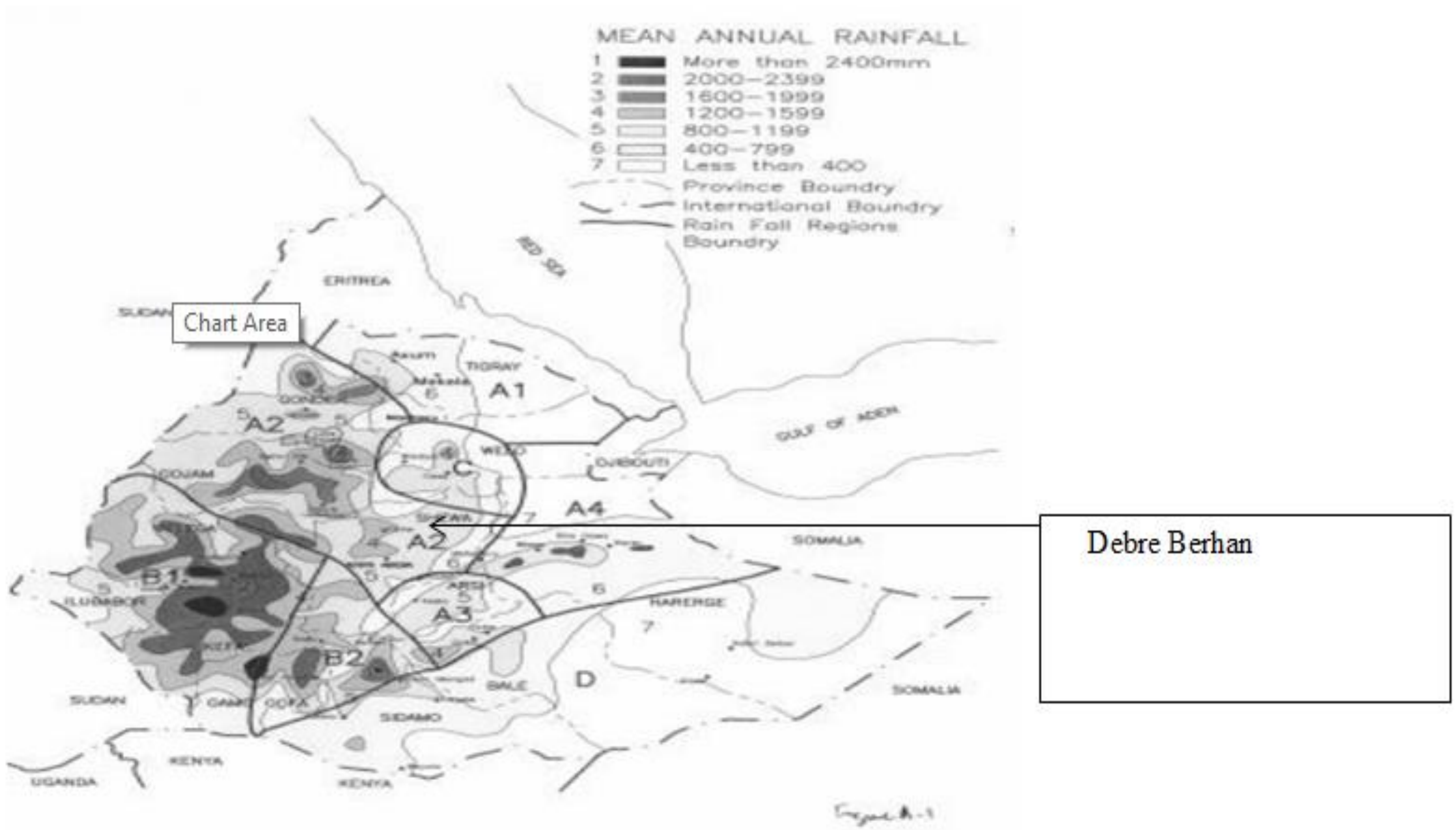
IDF Curve Developed from Debre berhan Meteorological Stations

Summary of the three Probability Distribution for Debre Berhan 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 value for A2
						K_T	Y_T	X_T	K_T	Y_T	X_T	K_T	X_T	
500	0.002	375.13	70.81	2.57	0.08	2.88	2.79	612.53	3.54	2.84	687.61	4.40	686.48	132.87
200	0.005	375.13	70.81	2.57	0.08	2.58	2.76	580.89	3.08	2.80	634.05	3.68	635.78	120.07
100	0.010	375.13	70.81	2.57	0.08	2.33	2.75	556.02	2.71	2.77	594.62	3.14	597.35	110.61
50	0.020	375.13	70.81	2.57	0.08	2.05	2.72	530.06	2.32	2.74	555.84	2.59	558.78	101.29
25	0.040	375.13	70.81	2.57	0.08	1.75	2.70	502.61	1.92	2.71	517.37	2.04	519.93	92.03
10	0.100	375.13	70.81	2.57	0.08	1.28	2.67	462.88	1.32	2.67	466.05	1.31	467.55	79.68
5	0.200	375.13	70.81	2.57	0.08	0.84	2.63	428.47	0.80	2.63	425.51	0.72	426.10	69.95
2	0.500	375.13	70.81	2.57	0.08	0.00	2.57	369.66	-0.09	2.56	364.03	-0.16	363.49	55.26

Appendix E: Intensity duration frequency curve development

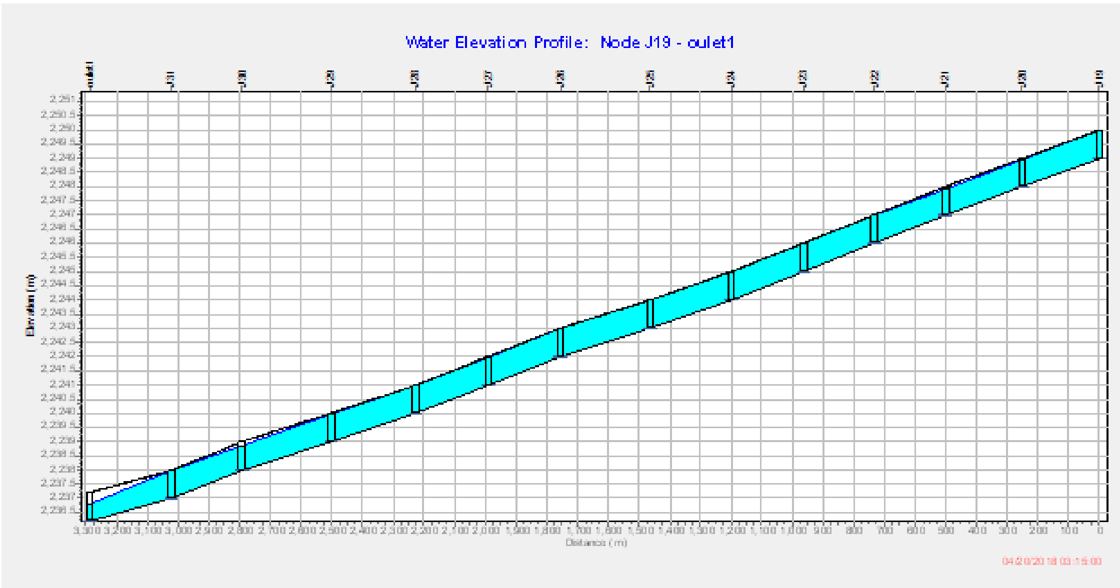
Duration (Minutes)	Duration (hr)	Rainfall Ratio(RRt)	Depth for given Return periods(mm)						Intensity for given return periods(mm/hr)					
			2 years	5 years	10 years	25 years	50 years	100 years	2 years	5 years	10 years	25 years	50 years	100 years
5	0.08	0.15	53.73	62.28	67.28	73.06	77.05	80.82	644.80	747.37	807.39	876.68	924.56	969.85
10	0.17	0.24	90.03	104.35	112.73	122.41	129.09	135.42	540.18	626.11	676.39	734.44	774.55	812.49
20	0.33	0.37	136.79	158.55	171.28	185.98	196.14	205.75	410.37	475.65	513.84	557.95	588.42	617.24
30	0.50	0.45	166.28	192.73	208.20	226.07	238.42	250.10	332.55	385.46	416.41	452.15	476.84	500.20
45	0.75	0.53	195.27	226.33	244.51	265.49	279.99	293.71	260.36	301.78	326.01	353.99	373.32	391.61
60	1.00	0.58	214.83	249.00	269.00	292.09	308.04	323.13	214.83	249.00	269.00	292.09	308.04	323.13
75	1.25	0.62	229.22	265.69	287.02	311.65	328.68	344.78	183.38	212.55	229.62	249.32	262.94	275.82
90	1.50	0.65	240.43	278.68	301.06	326.89	344.75	361.64	160.29	185.78	200.70	217.93	229.83	241.09
120	2.00	0.70	257.11	298.01	321.94	349.57	368.66	386.72	128.55	149.00	160.97	174.79	184.33	193.36
150	2.50	0.73	269.24	312.07	337.13	366.07	386.06	404.97	107.70	124.83	134.85	146.43	154.42	161.99
180	3.00	0.75	278.68	323.01	348.95	378.90	399.59	419.16	92.89	107.67	116.32	126.30	133.20	139.72
210	3.50	0.77	286.35	331.91	358.56	389.34	410.60	430.71	81.82	94.83	102.45	111.24	117.31	123.06
240	4	0.79	292.81	339.39	366.64	398.11	419.85	440.42	73.20	84.85	91.66	99.53	104.96	110.10
270	4.5	0.81	298.36	345.82	373.59	405.65	427.81	448.77	66.30	76.85	83.02	90.15	95.07	99.73
300	5	0.82	303.22	351.46	379.68	412.27	434.79	456.09	60.64	70.29	75.94	82.45	86.96	91.22
330	5.5	0.83	307.55	356.48	385.10	418.16	440.99	462.60	55.92	64.81	70.02	76.03	80.18	84.11
360	6	0.84	311.45	360.99	389.98	423.45	446.58	468.46	51.91	60.17	65.00	70.58	74.43	78.08
390	6.5	0.85	314.99	365.10	394.42	428.27	451.66	473.78	48.46	56.17	60.68	65.89	69.49	72.89
420	7	0.86	318.24	368.86	398.48	432.68	456.31	478.67	45.46	52.69	56.93	61.81	65.19	68.38
450	7.5	0.87	321.23	372.33	402.23	436.75	460.61	483.17	42.83	49.64	53.63	58.23	61.41	64.42
480	8	0.88	324.01	375.56	405.71	440.54	464.60	487.35	40.50	46.94	50.71	55.07	58.07	60.92
510	8.5	0.88	326.61	378.56	408.96	444.06	468.32	491.26	38.42	44.54	48.11	52.24	55.10	57.80
540	9	0.89	329.04	381.38	412.01	447.37	471.81	494.92	36.56	42.38	45.78	49.71	52.42	54.99
570	9.5	0.90	331.33	384.04	414.88	450.49	475.09	498.36	34.88	40.43	43.67	47.42	50.01	52.46
600	10	0.90	333.49	386.55	417.59	453.43	478.19	501.62	33.35	38.65	41.76	45.34	47.82	50.16
630	10.5	0.91	335.54	388.92	420.15	456.21	481.13	504.70	31.96	37.04	40.01	43.45	45.82	48.07
660	11	0.91	337.49	391.18	422.59	458.86	483.92	507.63	30.68	35.56	38.42	41.71	43.99	46.15
690	11.5	0.92	339.35	393.33	424.92	461.39	486.59	510.42	29.51	34.20	36.95	40.12	42.31	44.38
720	12	0.92	341.12	395.39	427.14	463.80	489.13	513.09	28.43	32.95	35.59	38.65	40.76	42.76



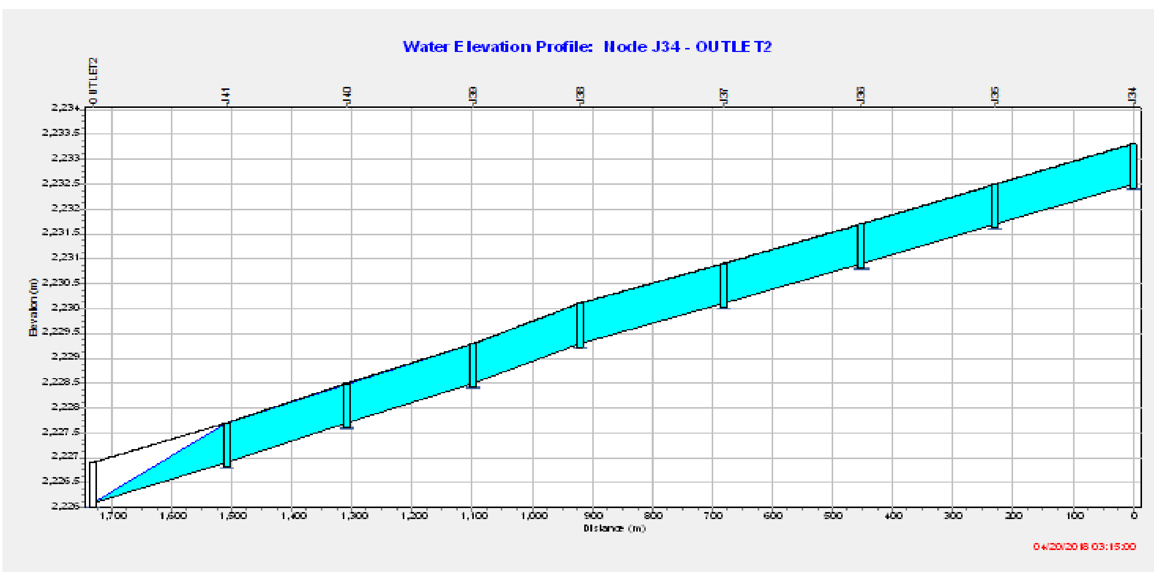
Mean annual rainfall for Ethiopia (ERA)

Appendix F: Water flow profile for flooding junction

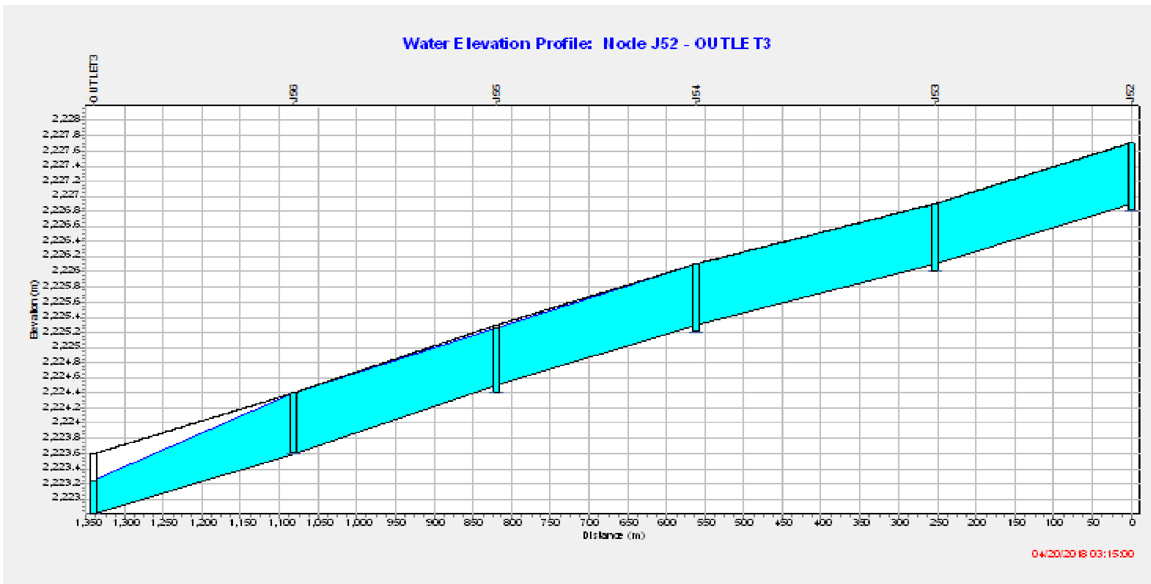
from bus station to berresa river



Megenteya to berresa river



from DB tena sceince to berresa river



Appendix G: Summary result sub catchment runoff

Summary Result for Bus station to Beressa river study area

Topic: Subcatchment Runoff   Click a column header to sort the column.

Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Total Runoff mm	Total Runoff 10^6 ltr	Peak Runoff CMS	Runoff Coeff
S1	407.65	0.00	2.60	23.03	338.93	10.85	0.58	0.831
S2	407.65	0.00	2.60	25.02	275.66	8.55	1.27	0.676
S3	407.65	0.00	2.58	16.71	372.84	8.58	1.27	0.915
S4	407.65	0.00	2.59	17.39	369.85	34.03	4.93	0.907
S5	407.65	0.00	2.59	17.38	369.80	22.93	3.37	0.907
S6	407.65	0.00	2.58	16.26	374.39	19.88	2.96	0.918
S7	407.65	0.00	2.59	17.08	374.29	12.16	1.52	0.918
S8	407.65	0.00	2.59	17.88	366.09	23.06	3.42	0.898
S9	407.65	0.00	2.58	16.31	375.41	15.88	2.34	0.921
S10	407.65	0.00	2.58	16.06	375.97	24.06	3.54	0.922
S11	407.65	0.00	2.59	17.97	366.39	13.92	1.99	0.899

Appendix H: Summary Result of link flow and node flow at each junction

Topic: Link Flow Click a column header to sort the column.

Link	Type	Maximum |Flow| CMS	Day of Maximum Flow	Hour of Maximum Flow	Maximum |Velocity| m/sec	Max / Full Flow	Max / Full Depth
C3	CONDUIT	0.000	0	00:00	0.00	0.00	0.27
C4	CONDUIT	0.576	0	01:36	2.07	0.51	0.46
C5	CONDUIT	0.575	0	01:38	2.10	0.37	0.49
C6	CONDUIT	1.315	0	00:15	2.49	0.85	0.89
C7	CONDUIT	1.098	0	00:15	1.86	1.07	1.00
C8	CONDUIT	1.127	0	02:04	2.19	1.05	0.88
C9	CONDUIT	1.113	0	01:54	2.10	0.84	0.88
C10	CONDUIT	1.223	0	04:02	2.23	1.09	0.95
C11	CONDUIT	1.207	0	04:02	2.13	1.00	0.95
C12	CONDUIT	1.014	0	07:11	1.79	1.12	0.98
C13	CONDUIT	1.004	0	00:23	1.72	1.08	0.98
C14	CONDUIT	1.048	0	08:12	2.21	1.02	0.80
C15	CONDUIT	1.044	0	08:13	2.98	0.62	0.79
C16	CONDUIT	2.241	0	00:24	3.79	1.10	1.00
C17	CONDUIT	0.853	0	10:12	2.03	1.03	0.86
C18	CONDUIT	0.847	0	10:14	1.81	0.78	0.86
C19	CONDUIT	1.145	0	06:03	1.99	1.11	0.98
C20	CONDUIT	1.130	0	07:40	2.07	1.09	0.94
C21	CONDUIT	1.125	0	05:53	1.95	1.04	0.96
C22	CONDUIT	1.181	0	00:15	2.03	1.09	1.00
C23	CONDUIT	1.151	0	00:16	1.97	1.08	1.00
C24	CONDUIT	1.108	0	00:15	1.90	1.10	1.00
C25	CONDUIT	1.063	0	11:33	1.85	1.11	1.00
C26	CONDUIT	1.154	0	00:15	1.98	1.08	1.00
C27	CONDUIT	1.162	0	00:15	2.06	1.09	1.00
C28	CONDUIT	1.093	0	00:18	2.09	1.11	0.99
C29	CONDUIT	1.043	0	12:29	2.08	1.08	0.93
C30	CONDUIT	1.042	0	12:30	2.18	0.97	0.93
31	CONDUIT	0.935	0	00:24	1.94	1.04	0.81

Node flow at each junction

Topic: Node Inflow Click a column header to sort the column.

Node	Type	Maximum Lateral Inflow CMS	Maximum Total Inflow CMS	Day of Maximum Inflow	Hour of Maximum Inflow	Lateral Inflow Volume 10^6 ltr	Total Inflow Volume 10^6 ltr	Flow Balance Error Percent
J3	JUNCTION	0.000	0.000	0	00:00	0	0	0.000
J4	JUNCTION	0.581	0.581	0	01:35	10.8	10.8	0.082
J5	JUNCTION	0.000	0.576	0	01:36	0	10.8	0.063
J6	JUNCTION	1.271	1.362	0	00:15	8.1	19.4	0.040
J7	JUNCTION	0.000	1.315	0	00:15	0	19.4	0.098
J8	JUNCTION	1.178	2.276	0	00:15	8.02	27.9	0.077
J9	JUNCTION	0.000	1.127	0	02:04	0	25.3	0.069
J10	JUNCTION	4.644	5.728	0	00:15	31.9	59.3	0.014
J11	JUNCTION	0.000	1.223	0	04:02	0	39.3	0.066
J12	JUNCTION	3.129	4.318	0	00:15	21.5	62.2	0.064
J13	JUNCTION	0.000	1.014	0	07:11	0	40.7	0.140
J14	JUNCTION	2.754	3.633	0	00:15	18.6	60.5	0.079
J15	JUNCTION	0.000	1.048	0	08:12	0	43.6	0.057
J16	JUNCTION	1.521	2.516	0	00:35	11.7	55.7	0.015
J17	JUNCTION	0.000	2.241	0	00:24	0	55.3	0.051
J18	JUNCTION	0.000	0.853	0	10:12	0	39.8	0.098
J19	JUNCTION	3.147	3.957	0	00:15	21.6	62.8	0.067
J20	JUNCTION	0.000	1.145	0	06:03	0	50.4	0.110
J21	JUNCTION	0.000	1.130	0	07:40	0	50.4	0.101
J22	JUNCTION	2.192	3.177	0	00:15	14.9	66.2	0.061
J23	JUNCTION	0.000	1.181	0	00:15	0	51.7	0.104
J24	JUNCTION	3.343	4.479	0	00:15	22.5	75.2	0.065
J25	JUNCTION	0.000	1.108	0	00:15	0	50.1	0.148
J26	JUNCTION	1.898	2.922	0	00:15	13	62.6	0.086
J27	JUNCTION	0.000	1.154	0	00:15	0	54.9	0.112
J28	JUNCTION	0.000	1.162	0	00:15	0	54.1	0.131
J29	JUNCTION	0.000	1.093	0	00:18	0	51.8	0.160
J30	JUNCTION	0.000	1.043	0	12:29	0	51.7	0.131
J31	JUNCTION	0.000	1.042	0	12:30	0	51.6	0.158
oulet1	OUTFALL	0.000	0.935	0	00:24	0	46.1	0.000

Appendix I: Summary result of outfall loading

Topic: Outfall Loading Click a column header to sort the column.

Outfall Node	Flow Freq. Pcnt.	Avg. Flow CMS	Max. Flow CMS	Total Volume 10^6 ltr
oulet1	98.19	0.870	0.935	46.117

Summary Result for megenteya to Beressa river study area

Summary result sub catchment runoff

Topic: Subcatchment Runoff Click a column header to sort the column.

Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Total Runoff mm	Total Runoff 10^6 ltr	Peak Runoff CMS	Runoff Coeff
S12	320.96	0.00	0.00	14.74	301.40	24.71	4.52	0.939
S13	320.96	0.00	0.00	12.18	310.44	34.77	7.19	0.967
S14	320.96	0.00	0.00	13.01	307.60	50.75	9.47	0.958
S15	320.96	0.00	0.00	11.82	311.49	135.81	28.16	0.970

Summary Result of link flow

Topic: Link Flow Click a column header to sort the column.

Link	Type	Maximum |Flow| CMS	Day of Maximum Flow	Hour of Maximum Flow	Maximum |Velocity| m/sec	Max / Full Flow	Max / Full Depth
C32	CONDUIT	0.000	0	00:00	0.00	0.00	0.00
C35	CONDUIT	0.650	0	00:43	1.64	0.98	0.99
C36	CONDUIT	0.652	0	00:15	1.63	1.00	1.00
C37	CONDUIT	0.639	0	00:26	1.60	1.00	1.00
C38	CONDUIT	0.738	0	00:15	1.84	1.00	1.00
C33	CONDUIT	0.000	0	00:15	0.00	0.00	0.50
C34	CONDUIT	0.650	0	00:49	1.64	1.00	0.99
C39	CONDUIT	0.678	0	00:17	1.73	1.00	0.99
C40	CONDUIT	0.678	0	00:43	1.72	0.97	0.99
C41	CONDUIT	0.613	0	00:15	1.84	0.93	0.83

Summary Result of Outfall loading

Topic: Outfall Loading Click a column header to sort the column.

Outfall Node	Flow Freq. Pcnt.	Avg. Flow CMS	Max. Flow CMS	Total Volume 10^6 ltr
OUTLET2	98.33	0.315	0.613	17.057

Summary result for Tena science collage to Beressa river study area

Summary result sub catchment runoff

Topic: Subcatchment Runoff Click a column header to sort the column.

Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Total Runoff mm	Total Runoff 10^6 ltr	Peak Runoff CMS	Runoff Coeff
S16	407.65	0.00	0.00	19.38	355.46	114.10	15.06	0.872
S17	407.65	0.00	0.00	20.30	345.65	194.25	24.72	0.848

Summary Result of link flow

Topic: Link Flow Click a column header to sort the column.

Link	Type	Maximum |Flow| CMS	Day of Maximum Flow	Hour of Maximum Flow	Maximum |Velocity| m/sec	Max / Full Flow	Max / Full Depth
C42	CONDUIT	0.000	0	00:00	0.00	0.00	0.00
C43	CONDUIT	0.000	0	00:00	0.00	0.00	0.00
C44	CONDUIT	0.000	0	00:00	0.00	0.00	0.00
C45	CONDUIT	0.000	0	00:00	0.00	0.00	0.00
C46	CONDUIT	0.000	0	00:00	0.00	0.00	0.00
C47	CONDUIT	0.000	0	00:00	0.00	0.00	0.00
C48	CONDUIT	0.000	0	00:00	0.00	0.00	0.00
C49	CONDUIT	0.000	0	00:00	0.00	0.00	0.00
C50	CONDUIT	0.000	0	00:00	0.00	0.00	0.00
C51	CONDUIT	0.000	0	00:15	0.00	0.00	0.50
C52	CONDUIT	0.532	0	00:15	1.33	1.00	1.00
C53	CONDUIT	0.481	0	15:00	1.20	1.00	1.00
C54	CONDUIT	0.525	0	00:15	1.34	1.00	1.00
C56	CONDUIT	0.461	0	00:17	1.49	0.88	0.78
C55	CONDUIT	0.542	0	00:15	1.43	0.98	0.98

Topic: Outfall Loading Click a column header to sort the column.

Outfall Node	Flow Freq. Pcnt.	Avg. Flow CMS	Max. Flow CMS	Total Volume 10^6 ltr
OUTLET3	98.33	0.461	0.461	24.583

Appendix J: Rainfall data of 10 day, used model Calibration and validation.

calibration		**validation**	
Date	daily ppt.(mm)	date	daily ppt.(mm)
29-05-2018	22.1	04-05-2018	30.65
12-06-2018	23.6	06-05-2018	3.95
14-06-2018	15.85	18-06-2018	37.25
21-07-2018	3.35	19-06-2018	2.7
27-07-2018	8.1	25-07-2018	15.85

Appendix K: Calculation of three error functions of model

A).Coefficient of Determination (R^2),

Record avrg.(Head) (m)	Simulated Flow rate (using SWMM5) CMS	Calc.flow rate (CMS) (using Manning's) (observed) CMS	$q_t^{obs} - q_t^{avg.obs}$	$q_t^{sim} - q_t^{avg.sim}$	$(q_t^{obs} - q_t^{avg.obs})*(q_t^{sim} - q_t^{avg.sim})$	$(q_t^{obs} - q_t^{avg.obs})^2$	$(q_t^{sim} - q_t^{avg.sim})^2$
0.71	0.286	0.253	-0.009	0.002	-0.00002	0.000073	0.00000
0.64	0.244	0.224	-0.038	-0.040	0.00151	0.001427	0.00160
0.72	0.288	0.258	-0.004	0.004	-0.00002	0.000019	0.00002
0.59	0.225	0.203	-0.059	-0.059	0.00345	0.003423	0.00348
0.66	0.281	0.233	-0.029	-0.003	0.00009	0.000867	0.00001
Total	**1.324**	**1.171**	**1.324**	**1.171**	**0.00502**	**0.005808**	**0.00511**
Avrage	**0.2648**	**0.2343**	**0.2648**	**0.234**			

$$\sum_1^5 (q_t^{obs} - q_t^{avg.obs}) * (q_t^{sim} - q_t^{avg.sim}) = \mathbf{0.00214}$$

$$\sqrt{(\sum_1^5 (q_t^{obs} - q_t^{avg.obs})^2} * \sqrt{\sum_1^5 (q_t^{sim} - q_t^{avg.sim})^2} = \mathbf{0.005447764}$$

$$R^2 = \mathbf{0.852}$$

B).The Nash-Sutcliffe coefficient (RNS)

Record avrg.(Head) (m)	Simulated Flow rate (using SWMM5) CMS	Calc.flow rate (CMS) (using Manning's) (observed) CMS	$(q_t^{obs} - q_t^{sim})^2$	$(q_t^{obs} - q_t^{avg.obs})^2$
0.71	0.286	0.253	0.00106	0.00007
0.64	0.244	0.224	0.00039	0.00142
0.72	0.288	0.258	0.00092	0.00002
0.59	0.225	0.203	0.00046	0.00341
0.66	0.281	0.233	0.00235	0.00086
Total	**1.324**	**1.171**	**0.00518**	**0.00578**
Avrage	**0.2648**	**0.2343**		

$$\sum_{t=1}^{5}(q_t^{obs} - q_t^{sim})^2 = 0.00518$$

$$\sqrt{\sum_{t=1}^{5} q_t^{obs} - q_t^{avg.obs})^2} = 0.076027508$$

$$\text{The Nash-Sutcliffe Coeff.(RNS)} = 1 - \frac{\sum_{t=1}^{5}(q_t^{obs} - q_t^{sim})^2}{\sqrt{\sum_{t=1}^{5} q_t^{obs} - q_t^{avg.obs})^2}} = 0.932$$

C).Relative Error (RE)

Avrg depth recorded) (m)	**Simulated Flow rate (using SWMM5) CMS**	**Calc.flow rate (CMS) (using Manning's) (observed) CMS**	**(q_t^{obs} - q_t^{sim})**
0.71	0.286	0.253	-0.022
0.64	0.244	0.224	-0.022
0.72	0.288	0.258	-0.030
0.59	0.225	0.203	-0.007
0.66	0.281	0.233	-0.029
Total	**1.42**	**1.309**	**-0.111**
Avrage	**0.284**	**0.2619**	

$$\sum_{t=1}^{5}\left|q_t^{\ obs} - q_t^{\ sim}\right| \quad = \quad \mathbf{0.111}$$

$$\sqrt{\sum_{t=1}^{5} q_t^{\ obs}} \quad = \quad \mathbf{1.144}$$

$$\text{relative Error(Re)} = \frac{\sum_{t=1}^{5}\left|q_t^{\ obs} - q_t^{\ sim}\right|}{\sqrt{\sum_{t=1}^{5} q_t^{\ obs}}} \quad 0.097$$

= 9.7%