Addis Ababa University

Addis Ababa Institute of Technology

School of Civil and Environmental Engineering

Assessment of Stormwater Drainage System in Assosa Town

A Thesis Submitted to the School of Graduate Studies, Addis Ababa Institute of Technology in Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil and Environmental Engineering (Water Supply and Environmental Engineering)

By

Abdu Biru Siragi

Advisor Dr. Fiseha Behulu

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Abdu Biru Siragi

Approved by Board of Examiners

Dr.Fiseha Behulu______ __________

Advisor Signature

Dr. Ing. Geremew Sahilu______ __________

Internal Examiner Signature

Dr. Yilma Sileshi_________ __________

External Examiner Signature

__________________________ __________

Chairman, Department of Graduate Committee Signature

DECLARATION

I, the undersigned declare that this Thesis is my original work performed under the supervision of research advisor Dr. Fiseha Behulu and has not been presented as a thesis for a degree in any other university in other University, All sources of materials used for this thesis have also been duly acknowledged.

Name: Abdu Biru

Signature: ____________

Place: Addis Ababa Institute of Technology, Addis Ababa University

Date: January 30, 2019

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ABSTRACT

Urbanization is one of the key factors that contribute to urban flooding, which has caused major destruction to the environment, public and private buildings and disrupts public life. In particular, the increase in population and building density influence the change in hydrological characteristics in urban areas. Conversion of pervious areas into impervious areas increases the stormwater runoff quantity dramatically.

One way of minimizing urban flooding is to convey stormwater to receiving waters through stormwater drainage systems, which has been practiced in some parts of Assosa town. In Assosa town, drainage problem become an issue during rainy season. This study deals with evaluate the existing stormwater drainage system of Assosa town considering hydraulic and hydrologic parameters in stormwater model. Despite there are many places in the city facing storm drainage problem; Stadium-Mazoria, Stadium-Hibret Bank, Sarbetoch-Anuar, Police commission-Ethiopia Hotel and Gebriel-Muluwongel church are areas selected for this study.

This stormwater drainage problem has necessitated use of simulation studies for understanding complexities related to the urban flood management; so, the simulation has done by Stormwater Management Model (SWMM). Daily rainfall data is obtained from nearby Assosa meteorology station. The selected extreme daily event was disaggregated into hourly events using reduction formula. Thereafter, the EPA SWMM is used to simulate the response of catchment to rainfall events in which runoff, water depth profile, and outflow hydrograph are obtained. Runoff is also obtained from rational formula for comparison purpose, and for obtaining design intensity a frequency analysis is carried out. LID measures (vegetative swales) also performed to evaluate how LID measures (vegetative swales) influenced urban runoff reduction.

The result of this study shows that the problem in the study area caused by insufficient drainage operation, minimum drainage pipe size, improper maintenance and clogging of drainage line by waste material. In addition the results showed that the LID measures are effective in controlling the surface runoff.

Keywords: Stormwater drainage, SWMM, runoff reduction, mitigation measure, Assosa

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ABBREVIATION

AASHTO	American Association of State Highway and Transportation Officials
Arc	GIS: Architectural Geographical Information System
BGRS	Benishangul-Gumuz Regional State
DEM	Digital Elevation Model
EPA	Environmental Protection Agency
EP	Environmental protection
ERA	Ethiopian Road Authority
FPP	Flood preparedness program
FUPI	Federal Urban Planning Institute
GoF	Goodness of Fit
GIS	Geographical Information System
GPS	Geographical positioning system
IDF	Intensity Duration Frequency
LID	Low Impact Development
NUPI	National Urban Planning Institute
NRCS	Natural Resources Conservation Service
SUDs	Sustainable Urban Drainage Systems
SWMM	Storm Water Management Model
SCS	Soil Conservation Service
Tc	Time of concentration
USGS	United State Geological Survey

1. INTRODUCTION

1.1. Background

Urbanization alters the natural process of stormwater runoff (Pazwash, 2011). Consequently, it increases stormwater runoff quantity due to the increase of impervious area, such as roads, parking lots, and rooftops. On the other hand, it has also been well acknowledged that processes which continuously take place in urban development can negatively affect stormwater runoff quality. Such issues are further reflected in the subsequent impact of water quality of natural receiving water bodies by altering physical, chemical and biological conditions of water (Konard and Booth, 2005).

Stormwater discharges are produced when the capacity of the land to retain precipitation is exceeded and runoff occurs. Runoff will be influenced by rainfall and intensity (millimeter of rainfall per hour) and duration, antecedent storms and a number of watersheds, and land use characteristics such as slope, soil type, and impervious surfaces (AASHTO, 1991 et al). Furthermore, the increases of peak flow, flow volume, flow velocity, as well as event frequency could deteriorate water quality in downstream areas (Burszta-Adamiak and Mrowiec, 2013).

The differences that can be expected between undeveloped-natural watersheds and developed-urban watersheds are associated with watershed characteristics that affect the speed and volume of runoff events. In disturbed areas (i.e. urbanized areas), the capacity to retain rainfall increases due to impervious surfaces like roads, gutters and a parking lot. Undisturbed areas have a greater capacity to retain storms/runoffs. This increased retention is associated with interception and infiltration of rainfall. In natural systems, the runoff process produces a network of channels that increase in size as the watershed area increases. That is, the final receiving system for all runoffs is a water way such as: stream or river.

Stormwater management with regard to urban drainage system depends on the development of a given area. In Ethiopian context, where watersheds of many urban centers receive significant amount of annual rainfall and where rainfall intensity is generally high, control of runoff at the source, flood protection, and safe disposal of the excess water/runoff through proper drainage facilities becomes significant (FUPI, 2008). Drainage problems in urban areas include flooding, deterioration of roads, land degradation, sedimentation, and blockage of drainage facilities, water logging and others. With urbanization, impermeability increases with the increase in impervious surfaces, drainage pattern changes, overland flow gets faster, flooding and environmental problems such as land degradation increases (AASHTO, 1991 et al). For example, in Assosa, all the stormwater is discharged in to the existing natural water ways/rivers during rainy seasons. That is why most of the rivers in Assosa, became polluted.

The absence of adequate integration between road and urban stormwater drainage network is also the other challenge in urban areas. This is due to the fact that the runoff generated with in a particular urban area will not safely be discharged in to the final receiving system. This will be the source of environmental problems like overtopping, erosion, pollution, barrier to traffic and other related problems. The expansion of Assosa town has been associated with the rapid conversion of land from rural to urban uses as other emerging town. For the last few decades it has been noticed that there is an intensive conversion of rural land to urban development like buildings, transportation networks, recreation areas and other manmade structures where most of them are impermeable structures.

The study area within Assosa town is also the drainage system where all the above mentioned challenges have been facing. In order to cope with those types of extreme events, advanced computer models are a solution as they can facilitate the stakeholders in developing the appropriate management strategies and mitigation measures (Barco et al. 2008). However, the availability of the appropriate data is limited, especially for urban areas, and as a consequence the model parameterization is impeded.

Computer based urban drainage models cover hydrologic and hydraulic aspects starting from precipitation input for each sub catchments to water depth and flow rates in pipe networks (Olofsson, 2007). SWMM (Storm Water Management Software), MOUSE (Modelling of Urban sewers), SOBEK Urban and MIKE Urban are some of the main software packages used for urban drainage system modeling. For this research SWMM is used because it is open source software, its input file is a single text file which is simple to manipulate in automated system.

1.2. Statement of the problem

In urban areas, impermeability increases with the increase of impervious surface, this in turn increases the overland flow resulting in flooding and related environmental problems. The pattern of urbanization and modernization in Ethiopia has increased urban infrastructure development (Dagnachew, 2011) which in turn affecting the stormwater management. In addition to increased impermeability of the urban landscape, the planning as well as implementation of stormwater infrastructures is presumed to be insufficient.

In the last twenty years, quite tremendous efforts have been taken to construct significant number of hydraulic structures in Assosa to be used for drainage system. The urban drainage of these structures failed to serve for the intended purpose due to number of observed challenges. Some structures were blocked by sediments that emanate from the upland areas. On the other hand the existing drainage channels were found to be inadequate in their size to discharge the incoming flow. The problem associated with foundation conditions within the vicinity of stormwater drainage channels is also one of these issues. Presently, some of the problems that have been observed in the Assosa town are either under the category of engineering (i.e. hydraulic, hydrologic or structural) and management matters. Mainly, the management related problems are the results of lack of integration among stakeholders in the solid waste disposal mechanisms.

One of the problems associated with hydraulic issues is the spatial distribution of the system in which the drainage channels are only limited to the asphalt roads; whereas the gravel roads have earthen channels that are not even constructed properly. Frequent flood during the rainy season has damaged the streets, drainage canals, residential and institutional houses, market area and so on. The flood has also created depressions and gullies in some parts of the town.

Under the hydrological associated issues, the town is facing the problem of improper urban drainage design systems. The road side ditches already constructed along the asphalt road are poorly designed only considering discharging storm runoff from the asphalt road, however the fact is most of catchment stormwater is conveyed by those existing ditches at study area. This made most ditches to be clogged with silt and urban effluent.

The other crucial problem in the town is the absence of proper Solid Waste Management System. Currently the existing stormwater drainage systems in the town are not properly functioning because they are filled with solid wastes and silt from the runoff. Moreover the movement of cattle in the town and garbage thrown from the residential areas to the streets is also damaging the existing stormwater drainage system. As a result of these, drainage channel bank destruction is quite common issue in the town.

To overcome this problem particular study regarding challenges that have been observed in Assosa town is essential. It is believed to be evaluated by modeling the existing system both from hydraulic and hydrologic perspectives.

1.3. Research Objective

1.3.1. General objective of the study

The general objective of this study is to evaluate the existing stormwater drainage system of Assosa town considering hydraulic and hydrologic parameters in stormwater model.

1.3.2. Specific objective

- To assess the hydraulic performance of existing drainage systems of the town.
- To identify the major challenges in stormwater drainage management system.
- To map the most affected drainage areas of the town with mitigation measures.

1.4. Research Questions

- What is the hydraulic performance of existing drainage systems of the town?
- What are the major challenges in managing the drainage system in the study area?
- What are the mitigation measures for runoff problem in the study area?

1.5. Scope and Limitation of the Thesis

Since Assosa town has covered large catchment area, assessing the whole town is not possible from the finance and time view of point to come up with solution for the current stormwater drainage problem. Therefore, some representative major flood prone areas have been selected based on administrator interview and field observation (Stadium-Mazoria, Stadium-Hibret Bank, Sarbetoch-Anuar, Police commission-Ethiopia Hotel and Gebriel-Muluwongel church).

Generally; it will address issues related to urban stormwater drainage by considering hydraulic and hydrologic parameters in stormwater modeling. The specific focus of this study includes: assessing hydraulic performance of the existing drainage system, identify some of challenges and mitigation measures in managing the existing stormwater drainage system.

Following rainfall event, the amounts of generated runoff are of high concern when assessing urban areas, this in term of both quantity and quality. Hence, in this thesis there will be focus only on the quantity of the runoff.

1.6. Significance of the Study

Ethiopia is at the eve of developing into a middle income economy with its development being shown by the level of road network and urban drainage under construction. Roads are being either constructed or being improved throughout the country. It is therefore important to have good drainage systems in such roads if they are to be sustainable and economically viable as far as maintenance is concerned.

Inadequate drainage has far reaching consequences on the roads because poor drainage cause clogging of water on road surfaces and hence deterioration.

Generally, managing urban stormwater drainage system has a significant role for viable environmental management by keeping the service life of urban infrastructures such as roads, buildings, telephone lines, water supply lines and the existing rivers

This research is aimed at coming up with finding out reasons for inadequate provision of drainage system and the hydraulics performance of the drainage systems.

It is supposed the concerned body of Assosa town administrator may use it as a reference while they are preparing their annual plans for urban drainage system. And also it will be an alternative means of ensuring sustainable development in Assosa by strengthening the environmental and socioeconomic activities regarding to urban drainage system.

2. LITERATURE REVIEW

2.1. Introduction

The Millennium Development Goals and the development agenda of most developing countries focus on issues around health, sanitation and water provision (https://sustainabledevelopment.un.org). However, urban flooding due to lack of adequate drainage systems poses one of the greatest threats to human settlements today (Parkinson, 2003); a threat that will exponentially increase as the climate changes and weather patterns become more erratic and intense (Sakijege, Lupala & Sheuya, 2012). Such issues are more pronounced when it comes to the urban hydrologic processes which are affected by the interaction of human beings' day to day activities. Among the main issues, proper disposal of stormwater from urban areas remains one of the challenging tasks.

Stormwater drainage is the process of draining excess water from streets, sidewalks, roofs, buildings and other areas. It can be any precipitation, such as rain, snow and sleet that falls on the surface of the earth. In general, areas with natural, unaltered groundwater, about 10% of the precipitation become runoff and about 50% infiltrates into the soil to form or replenish groundwater and flows into streams. Evaporation and uptake by plants accounts to the remaining 40%. When natural conditions change due to development, land use and other activities, this water cycle becomes altered .As the land becomes more covered with impervious surfaces, more precipitation converts as runoff. This runoff carries the dust, other loads, and pollutants. When the development is more as much as 55% may become runoff (JNNURM Project, 2006).

2.2. Historical Perspectives of Urban Drainage

To understand the concepts and need for implementation of drainage Systems, it is necessary to contextualize, in a historical manner, how the urban drainage systems were developed. Through time, and up to the Modern Age, drainage works, as a rule, were not considered as necessary infrastructures providing conditions for the development and ordering of urban centers (Matos, 2003). Nevertheless, rainwater drainage systems have been found in much more ancient cities or city ruins.

In the period prior to the Christian era, systems implemented by the Persians and Greeks are worthy of note. Drainage networks constructed by the Romans (8^{th} Century B.C. ~ 3^{rd} Century A.D.) may be observed yet today, with small sections still in operation. The same holds true for ruins of cities built by Pre-Columbian peoples in various countries of Latin America (TIM, 2008).

Before giving an account of the important transformation which occurred in the 19^{th} Century related to sanitary waste systems and rainwater drainage, it is worthwhile to go back some years in history when, according to Silveira (2002), a period of elimination of flooded areas began, the burying of septic tanks and then their replacement by underground channeling systems. This process for removal of waste and rainwater began in Italy as a result of observing a correlation between mortality rates of people and animals and the sanitation system; after that, it was adopted in innumerable European cities as a public health measure.

According to Burian and Edwards (2002), drainage systems were used to collect rainwater, dispose of wastewater and prevent flooding as early as 3000 BC. The systems were designed through a process of trial and error, which has shaped the drainage systems of today. Although there is not enough evidence to present a picture of the drainage systems in totality, some ideas can be deduced from archaeological evidence from those times (Burian & Edwards, 2002). The issue of storm water drainage was clearly recognized as a cause for concern because it resulted in the flooding of cities (Scullard, 1967; Strong, 1968 in Burian & Edwards, 2002).

Between 1850 and the end of the 19th Century, many important cities of the world, principally European capitals, were provided with large single underground networks for waste materials (rainwater drainage and sewage conducted by the same conduits) (Silveira, 2002). According to Jones & MacDonald (2007), in terms of urban drainage, one of the most famous examples of this period was the reconstruction of Paris in the Second Empire of Haussmann and Napoleon III, who used a system designed to rapidly remove waters from the city.

The sanitary hygiene concept, as already mentioned, foresees rapid expulsion of water from the city for the purpose of preserving the health of the population and eliminating any type of discomfort the water could cause. Nevertheless, what was not foreseen in this effort for channeling is the impact caused downstream, since the sanitary hygiene concept acts only in a local manner in the sense of transferring the problem to other regions. Thus, this concept, associated with rapid growth of the urban population, and by the latter, understanding all the characteristics brought about by urbanization such as increasing imperviousness of the soil and removal of plant cover, is responsible for constant flooding, landslides, and problems created in relation to recharging of aquifers.

Such a concept was adopted worldwide up to the 1960s of the past century (20th Century). In this period, developed countries began to perceive the conflict generated between the existing rainwater drainage system and the environment. Thus began a new drainage concept, an evolution of the former; concerned not only with public health, but also with the environmental question.

The drainage systems of the Roman Empire were the first documented systems that were seemingly planned and organized (Burian & Edwards, 2002) and then combined with other systems, despite the reduction of capacity within each. Due to the capacity problem experienced with combined sewers, drainage for wastewater and stormwater was separated as the treatment of the wastewater in the combined system was too costly (Wentzel, 2013). These systems were "…neither sustainable nor resilient to large storms and hurricanes" (Novotny, 2008). This separated system is what has been used in the modern world and the one implemented in Assosa town.

2.2.1. Basics of Urban Drainage System

Urban drainage is concerned with the collection and conveyance of wastewater and stormwater from urban areas (Butler and Davies, 2004). An important social aspect is to maintain public health and safety; hence an efficient drainage of stormwater and wastewater is essential to avoid impact of flooding on life and property. In addition, the current environmental awareness involves the protection of the receiving waters from the pollutants that may be dragged by water flowing in the surface during heavy rain events (Viessman et al., 2009).

The urban drainage system was first challenged due to the interactions between human activities and the natural water cycle, where this cycle was interrupted due to either (a) abstraction of water for drinking purposes and generating a wastewater also (b) increasing the impervious surfaces that causing rainwater diversion from natural drainage system and generating a considerable runoff. Consequently, both types of water need immediate drainage (Bulter and Davies, 2011). In this thesis, the rainfall-generated runoff is only of concern and the urban generated wastewater won't be discussed further.

The runoff is a rainwater (can be also resulted from other forms of precipitation), which fallen on impermeable surfaces and caused distinguished damages, flooding and also further health risks due to the pollutants from air or the catchment itself (Bulter and Davies, 2011) The impacts of urbanization on the urban drainage are discussed further in the following sections, in terms of quantity and to certain extend runoff quality also.

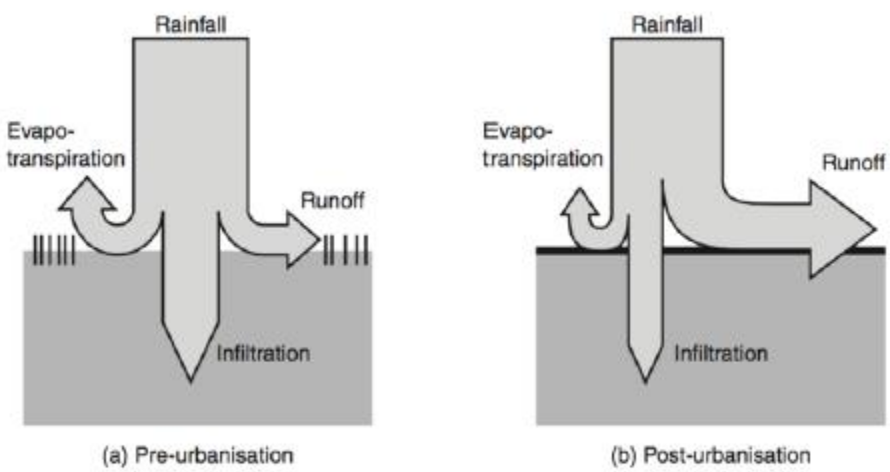

Figure 2.1: The impact of urbanization on the rainwater-generated runoff (Butler & Davies, 2011)

2.2.2. Development of Modern Urban Drainage Practices

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

The public perspective of urban drainage changed during the nineteenth century from a neglected afterthought to a vital public works system. The public also shifted their stance regarding funding the construction and maintenance of sewer systems. The shift in public perspective was driven by many factors, but the most important was probably the scientific evidence accumulated during the second half of the century linking sanitary wastes and disease transmission (Scullard 1967).

The perspective of urban drainage also changed from a design standpoint during the nineteenth century. Most sewers constructed before the nineteenth century were not planned or designed by an engineer using numerical calculations. Instead a trial-and-error process was executed, which in some cases eventually produced well-functioning systems (Reynolds, 1946).

Currently, Engineers continued to improve design concepts and methods. During the second half of the twentieth century regulatory elements were promulgated in the United States, Europe, and other locations addressing urban drainage issues (Burain & Edwards, 2002). Extensive monitoring efforts vastly improved the understanding of urban drainage quantity and quality characteristics. Computer modeling tools advanced the methods used to design and analyze urban drainage systems. Regulations, monitoring, computer modeling, and environmental concerns have altered the perspective of urban drainage from a public health and nuisance flooding concern during the first half of the twentieth century into a public health and nuisance flooding with additional concerns for ecosystem protection and urban sustainability (Burain & Edwards, 2002).

Methods to design and construct sustainable urban drainage systems are currently being researched and tested. Alternative development concepts (e.g., low-impact development) are influencing development practices to minimize the impacts of development on stormwater drainage. In addition, alternative on-site wastewater management strategies are being touted as more sustainable than centralized wastewater management for some situations. Communities are searching for innovative techniques to capture, detain, and use rainwater within the watershed instead of constructing massive drainage structures. Many communities are developing watershed-wide stormwater quality management plans to meet the dual objectives of flood prevention and water quality control. Urban drainage has indeed expanded significantly during the past few decades beyond a technical challenge to drain the urban area expeditiously to include the consideration of social, economic, political, environmental, and regulatory factors.

In recent decades, new focus approaching sustainability have been studied, with various names: Low Impact Development (LID), in the USA and Canada; Sustainable Urban Drainage Systems (SUDS), in the United Kingdom; Water Sensitive Urban Design (WSUD), in Australia; and Low Impact Urban Design and Development (LIUDD), in New Zealand. Regardless of the name, the ideas and concepts of the sustainable systems presented are very similar and all make reference to balance among the variables of the hydrologic cycle and their effects on watersheds.

Landscaping, environmental, and economic gains reinforce the advantages presented by this conception of urban drainage treatment, controlling not only the peak flows, but also the volume, the frequency, the duration, and the quality of runoff and drainage (Souza, 2005).

In contrast, developing countries are relatively behind the times, since quantitative control of urban drainage is still limited and quality control of the water drained is still far from accomplished. This reinforces the need for researching means for encouraging the use of techniques, like charging for water use, with the goal of control at the source and maintenance of characteristics of the pre-development hydrologic cycle.

2.2.3. The Impact of Urbanization on the Drainage

One of the most direct and noticeable environmental consequences of urbanization is the change in land use and land cover, in the form of increased impermeable surfaces, affecting the hydrological cycle. This is made apparent through changes in several hydrological features, including those in surface and groundwater levels, surface runoff patterns, and water pollution (Arup, et al., 2006).

According to the USGS (2016a), land-use change due to urbanization impacts water systems in a variety of ways. In the initial phase of large scale urbanization, an increase in storm runoff and erosion can be noted. This is due to the decrease in vegetation acting as a hinder for storm runoff, allowing an amplification on its pace. Consequently, the quantity of sediment being washed into streams is increased. Additionally, alterations to water-drainage patterns – common in this initial phase – can cause flooding. Alterations can include the introduction of drainage pipes, and the channelization of rivers.

During continued urbanization, more roads, houses, and buildings of both commercial and industrial kind are added to the area. More people can now live together on a closer area, consequently increasing wastewater amounts being discharged into local water bodies. The impermeable surfaces in form of pavements and constructions hinder the infiltration of water into ground water reserves, giving less water for groundwater recharge. Furthermore, an increase in stormwater runoff can also be noted. Water that was to be infiltrated, will instead reach water bodies with the risk of causing flooding (USGS, 2016a).

According to a study made in North Virginia (Anderson, 1970) on the effects of urban development on floods in that area, the general conclusions that could be drawn were that the installation of sewer systems, independent of impervious development, led to an increase in flood-peak magnitudes. Another conclusion was that completely impervious surface areas increase average-sized floods. However, no significant impacts could be shown on larger floods, such as 100-year floods. Other findings include that the lag time, defined as *the time from the centroid of rainfall excess to the centroid of direct runoff*, seems to be the basin characteristic mostly affected by urbanization. Supported by Shuster et al. (2005), it is concluded that the lag time decreases with urbanization, giving higher runoff peaks and thus higher volumes of water added to receiving water bodies.

Kent et al. (2010) emphasize the effect that urbanization has on flow velocities, as vegetation is replaced by impermeable surfaces, such as streets and gutters. This has a generally decreasing effect on the travel time through the watershed, as runoff is being transported downstream much faster. Similarly, overland flow lengths are decreased due to runoff being conveyed to channels. These are constructed with high hydraulic efficiency, once again increasing flow velocities. It is also mentioned that urbanization can have either an increasing or decreasing effect on slopes. Supported by USGS (2003), peak discharges of floods are strongly influenced by changes in the natural landscape, as storage capacities are decreased with land use change, and the movement of water is increased, as described above. This results in higher peak discharges in urban streams, and also larger volumes of discharged water during floods. An example of this has been presented by the USGS (2003), where the stream flow in two creeks – one urban, one rural – have been compared during the same flood. The urban creek saw earlier and faster increase in stream flow, together with higher peak discharge and overall volume during the same flood.

Finally, it has been shown that urbanization has a great impact on the quality of water. Litter, lawn and garden chemicals, paints and oils, and detergents high in phosphorus, are some acknowledged releases made into storm and wastewater sewers, harming water sources and the surrounding environment (USGS, 2016b).

A typical impact of urban development on recipient waters is increased amounts of water entering them, and a lower time of concentration. The combination of these two events can result in altered stream paths due to erosion of streambeds and stream banks, and more rapid runoff. Rapid runoff and impermeable surfaces also prevent infiltration to soil and aquifer recharge, which can lead to lower sustained stream flows, especially during dry periods (Coles, et al., 2012).

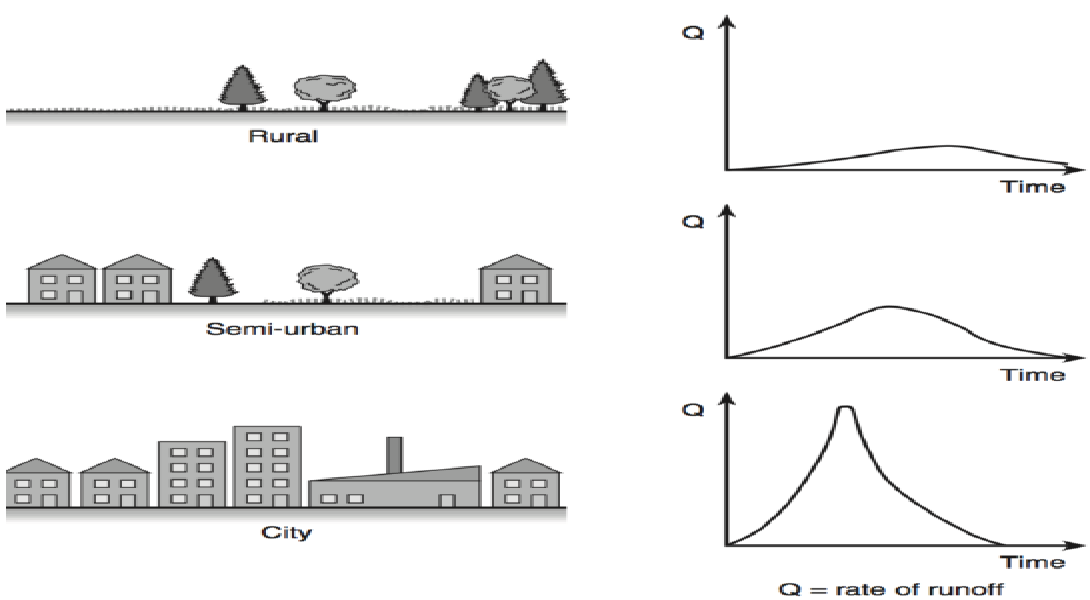

Figure 2.2: The impact of the urbanization on the peak flow of the runoff (Butler & Davies, 2011)

2.3. The Stormwater Runoff Generation and its Management

The urban runoff generation can be described in different ways. However, *Figure 2.3* explains the different processes that lead to the surface runoff formation, where stormwater (A) runs over the impermeable surfaces and form the surface runoff (B) also the overland flow ((C) the surplus from infiltration) join together to the surface runoff and flow into the sewers (D). These different processes are mainly depends on the rainfall intensity and duration as well as on the nature of the catchment, nevertheless the nature of the surfaces (Bulter and Davies, 2011).

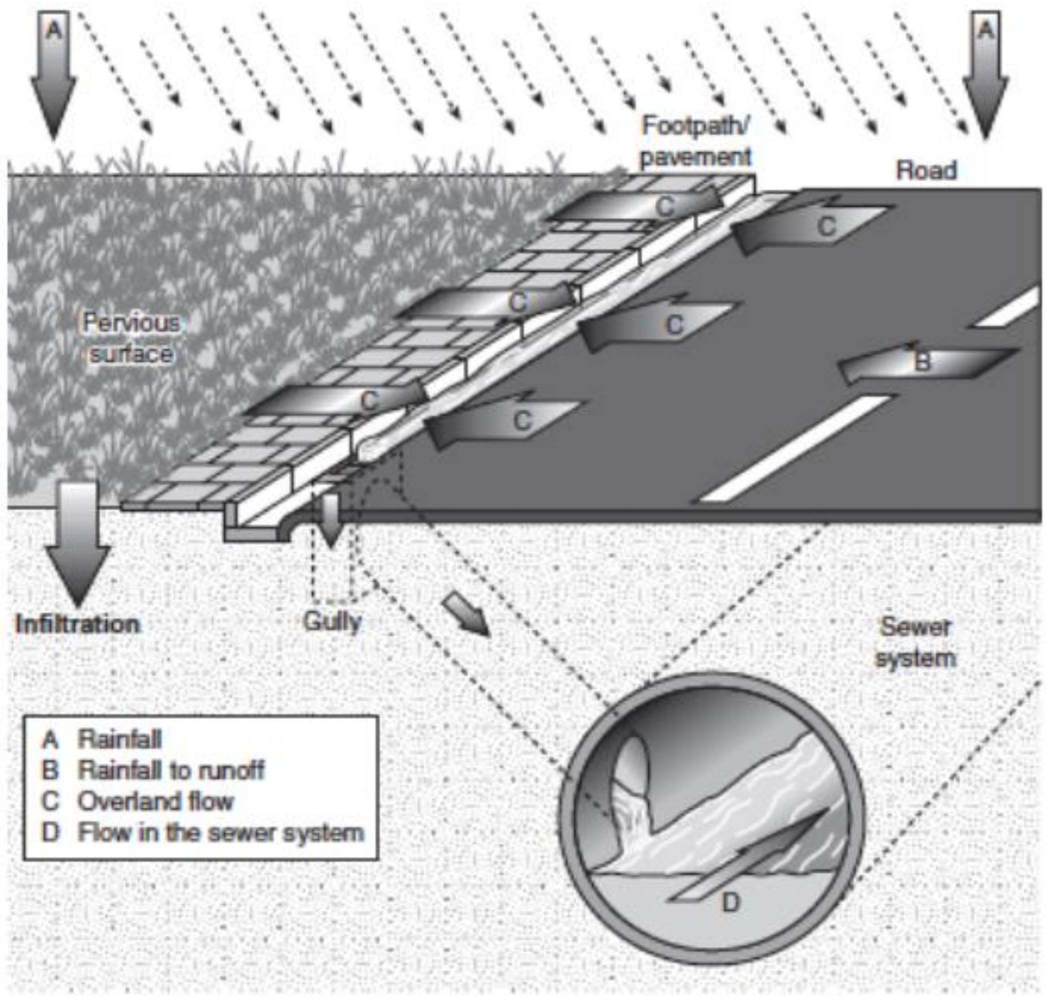

Figure 2.3: Runoff generation process (Butler & Davies, 2011)

As mentioned above, the urban development is producing more runoff comparing to natural areas, which in return is leaving a further negative impact on the sewer system in these areas. Hence, more runoff is generated under heavy rainfall event and overloads the system designed capacity, thereafter encouraging surface flooding and more frequent manholes surcharge.

According to urban Stormwater drainage design manual, stormwater management is concerned with the collection, conveyance, storage, treatment and disposal of stormwater runoff in a way that minimize accelerated channel erosion, increased flood damage, and/or degradation of water quality and in a manner to enhance and ensure the public health, safety & general welfare, which shall include a system of vegetative or structural measures, or both that control the increased volume and rate of stormwater runoff caused by manmade changes to the land (MoWUD, 2008).

While major problems associated with stormwater include flash flooding, erosion and sedimentation, water quality degradation as well as surface pollution, the traditional concept for urban drainage design has been associated with stormwater disposal, with the basic philosophy of transferring any amount of stormwater as quickly as possible out of built-up areas. While this temporarily served to eliminate the local flooding problems, it only relayed the problem further downstream, where not only is flooding a concern but pollution and erosion of natural water courses are as well (Verworn, 2002).

Similarly stormwater management in the urban areas of Ethiopia has continues to predominantly focus on collecting runoff and channeling it to the nearest watercourse. This means that stormwater drainage currently prioritizes quantity (flow) management with little or no emphasis on the preservation of the environment. The result has been a significant impact on the environment through the resulting erosion, siltation and pollution. An alternative approach is to consider stormwater as part of the urban water cycle, a strategy which is being increasingly known as Water Sensitive Urban Design (WSUD) with the stormwater management component being known as Sustainable Drainage Systems (SuDS).

Stormwater management and their associated issues have been studied by many researchers. Most of them argued that integrated approaches are more realistic and worthy in urban developments. For instance, Verworn (2002), indicated concept of stormwater management at its source, in which stormwater runoff is not immediately discharged to natural rivers but is stored and treated or re-used locally, at or close to its point of generation with the aim of making use of infiltration capacity wherever possible. He also noted that such management approach is found to be an ideal rehabilitation method, as it has the benefits of recharging soil moisture and groundwater; reducing peak runoffs as well as reducing total runoff volume to drainage systems. Armitage (2010) has also noted that a major weakness of urban drainage management is the lack of holistic considerations of other integral parts. Similarly, Mohd Nor et al, (2011) indicated that the problems associated with stormwater shall be dealt in an integrated approach so that the effects of individual problems can be reduced in downstream areas.

2.4. Urban Drainage system Modelling

Models are just simplifications of the real condition that is going to be simulated or analyzed. Modeling in urban drainage system serves various purposes such as the overall assessment of drainage area response as a part of strategic and master planning to the detailed network and providing necessary support to primary activities such as elements design, assessment of pollution, operational management, real time control and analysis of interactions among sub-systems.

The type of model applied depends on the goal of Modeling, spatial coverage, data and technology availability. There are a number of empirical hydrologic methods that can be used to estimate runoff characteristics for the drainage areas. The most commonly used stormwater models can generally be classified as either hydrologic, hydraulic, or water quality models (Giudice, et al. 2013) and, the general description of those models are as follows:-

Hydrologic models: - are models used to simulate runoff volumes, peak flows, and the temporal distribution of runoff at a particular location resulting from a given precipitation of an event. Hydrologic models are also used to simulate how the drainage area parameters will cause runoff either to flow relatively unhindered through the system to a point of interest, or to design a detention or retention system to route runoff hydrographs through storage areas or channels (Looper, et al. 2012).

Hydraulic models:- are models used to simulate the water surface elevations (HGL), energy grade lines, flow rates, velocities, pipe size and other flow characteristics throughout a drainage network that result from a given runoff hydrograph or steady flow input. The hydraulic model also used for various computational routines such as to route the runoff through the drainage network, which may include channels, pipes, control structures, and storage areas (Mannina & Viviani, 2010, Urbonas, 2007).

Water quality models: - are models used to evaluate the effectiveness of an agency recommended best management practices (BMPs), simulate water quality conditions in a lake, stream, or wetland, and to estimate the loadings to water bodies. Often the goal is to evaluate how some external factor (such as a change in land use or land cover, the use of best management practices (BMPs), or a change in lake internal loading) will affect water quality. Parameters that are frequently modeled include total phosphorus, total suspended solids and dissolved oxygen (Mailhot, et al. 1997, Mannina, & Viviani, 2010).

The selections of models are range from very basic tools with minimal data input requirement, to complex tools that require expertise. In general, the selection of appropriate urban drainage models are depends on a number of factors (Mailhot, et al. 1997, Urbonas, 2007). The main factors for the selection of urban drainage models including:

Desired output (outflow hydrograph, peak runoff rate and volume, pollutant removal and infiltration loss):- some models can be used to estimates peak runoff rates, but cannot be used to simulate total runoff volumes (Rational Method). In the contrary, other methods can only estimates total runoff volumes. While others, such as the natural resources conservation service (NRCS) model for example, can be used to simulate both total runoff volume and peak rate, and runoff hydrographs.

Scale of project and Drainage Area Size: -because of their assumptions and/or theoretical basis, some models can only use to simulate runoff volumes or rates for drainage areas less than 20 acres, while other methods can be applied for a larger drainage area of 20 square miles or more 9 (Vaze, & Chiew 2003).

The availability of various model input parameters (soil type, topographic): - Simple models, such as the modified rational methods, require basic data such as rainfall intensity, runoff coefficient and drainage area, while other, more sophisticated methods have extensive data requirement, including long-term rainfall and temperature data.

Level of professional expertise required to perform modeling: - the level of expertise required to perform modeling is the most important factor for both theoretical and practical reasons, compare to less trained professionals in knowledge, model output analysis, decision-making and a range of other capabilities. For this research SWMM is used because it is open source software, and simplicity to simulate the main stormwater drainage hydrological and hydraulic processes.

2.5. The Stormwater Management Model (SWMM)

SWMM was developed in 1971, with the support of the U.S. Environmental Protection Agency first. It’s widely used in urban storm flood, combined sewers, drains and other drainage systems analysis and design since its initial development in the world (Rossman, 2010).

The EPA Storm Water Management Model (SWMM) is a dynamic rainfall-runoff simulation model used for single event or long-term (continuous) simulation of runoff quantity and quality from primarily urban areas. This model is mainly used in small cities’ runoff and water quality forecasting, river drainage simulation and compute and drainage pipe network system verification and management. It consists of runoff yield model, infiltration model, surface runoff concentration model and pipe hydraulic dynamic model (Wang and Xu, 2008).

Runoff yield model: The process of rainfall turning into net rain is called runoff yield. Based on the land use status and surface drainage directions, study area is discredited into several sub catchments. According to the hydrology differences, each sub catchment can be divided into pervious and impervious sub areas. Surface runoff can infiltrate into the upper soil zone of the pervious subarea, but not through the impervious subarea. Impervious areas are themselves divided into two subareas - one that contains depression storage and another that does not. Those three subareas’ runoff yield is computed separately and the sub catchments’ runoff is the sum of subareas.

Infiltration model: Infiltration is the process of rainfall penetrating the ground surface into the unsaturated soil zone of pervious sub catchments areas. SWMM offers three choices for modeling infiltration: Horton's Equation, Green-Ampt Method and SCS Curve Number Method. Horton's Equation mainly describes the relationship that infiltration intensity changes with rainfall duration and does not reflect the underlying surface status of saturated and unsaturated zone. Green-Ampt Method assumes that a sharp wetting front exists in the soil column, separating soil with some initial moisture content below from saturated soil above. A sufficient rainfall and infiltration process can make the soil saturated. SCS Curve Number Method assumes that the total infiltration capacity of a soil can be found from the soil's tabulated Curve Number. During a rain event this capacity is depleted as a function of cumulative rainfall and remaining capacity.

Surface runoff concentration model: The task of surface runoff concentration is to convert the subareas' net rainfall process into sub catchments' discharge process. In SWMM, this task is realized by making three subareas approximated as nonlinear reservoir. Namely, solving continuity equation and manning formula simultaneously.

Pipe hydraulic dynamic model: The pipe flow routing in SWMM is calculated by conservation of mass and conservation of energy formula. And three solving methods are supplied: Steady Flow, Kinematic Wave and Dynamic Wave (Rui, 2004):

Steady flow: Steady Flow routing represents the simplest type of routing possible (actually no routing) by assuming that within each computational time step flow is uniform and steady. Thus it simply translates inflow hydrographs at the upstream end of the conduit to the downstream end, with no delay or change in shape. The normal flow equation is used to relate flow rate to flow area (or depth).

Kinematic wave: This routing method solves the continuity equation along with a simplified form of the momentum equation in each conduit. The latter requires that the slope of the water surface equal the slope of the conduit. The maximum flow that can be conveyed through a conduit is the full normal flow value. Any flow in excess of this entering the inlet node is either lost from the system or can pond atop the inlet node and be re-introduced into the conduit as capacity becomes available. Kinematic wave routing allows flow and area to vary both spatially and temporally within a conduit. This can result in attenuated and delayed outflow hydrographs as inflow is routed through the channel. However this form of routing cannot account for backwater effects, entrance/exit losses, flow reversal, or pressurized flow and is also restricted to dendritic network layouts. It can usually maintain numerical stability with moderately large time steps, on the order of 1 to 5 min. If the aforementioned effects are not expected to be significant then this alternative can be an accurate and efficient routing method, especially for long-term simulations.

Dynamic wave: Dynamic Wave routing solves the complete one-dimensional Saint Venant flow equations and therefore produces the most theoretically accurate results. These equations consist of the continuity and momentum equations for conduits and a volume continuity equation at nodes. With this form of routing it is possible to represent pressurized flow when a closed conduit becomes full, such that flows can exceed the full normal flow value. Flooding occurs when the water depth at a node exceeds the maximum available depth and the excess flow is either lost from the system or can pond atop the node and re-enter the drainage system. Dynamic wave routing can account for channel storage, backwater, entrance/exit losses, flow reversal and pressurized flow. Because it couples together the solution for both water levels at nodes and flow in conduits it can be applied to any general network layout, even those containing multiple downstream diversions and loops. It is the method of choice for systems subjected to significant backwater effects due to downstream flow restrictions and with flow regulation via weirs and orifices. This generality comes at a price of having to use much smaller time steps, on the order of a minute or less (SWMM can automatically reduce the user-defined maximum time step as needed to maintain numerical stability).

2.6. Urban Stormwater Drainage Experience in Ethiopia

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

In order to establish the fact that drainage problem exists in the Ethiopian towns and to understand the works that are done, literatures are reviewed. The literatures showed no doubt on the existence of drainage problem in the Ethiopian towns. The presentations of the problems in the literature are presented either in the form of malfunctioning of specific component of the urban transportation or broader problems on urban drainage systems themselves. Some studies on drainage problems that exist in Ethiopian towns are reviewed in this section.

Table 2.1: Some stormwater drainage related studies in Ethiopia

No	Topic	Study area	Author(s) (Year)	Scientific contribution	Model used
1.	Road and urban storm water drainage network integration in Addis Ababa: Addis Ketema Sub-city	Addis Ababa	Dagnachew (2011)	Published (*Journal of Engineering and Technology)*	-
2.	Assessment of the Effect of Urban Road Surface Drainage: A Case Study at Ginjo Guduru Kebele	Jimma	Getachew and Tamene(2015)	Published (*International Journal of Science, Technology and Society)*	SWMM
3.	An Approach to Drainage System Sustainability	Wolaita Soddo	Mitiku and Mekdes (2017)	Published (*International Journal of Waste Resources*)	-
4.	Urban stormwater drainage system in the central part of Addis Ababa	Addis Ababa	D. Muschalla and M.Ostrowski (2002)	Published (*Ninth International Conference on Urban Drainage (9ICUD*)	-
5.	Modeling and Analyses of Urban flooding in Bole Subcity System Performance and Evaluation of Possible Improvements	Addis Ababa	Nejib (2016)	Msc Thesis (*AAU-AAiT*)	SWMM
6.	Highway drainage facilities problems (Case Study – Assessment of Drainage Problems in Adama)	Adama	Meraf (2015)	Msc Thesis *(AAU-AAiT)*	HEC-RAS
7.	Assessment of Stormwater Drainage Systems in Kemise Towm	Kemise	Biniyam (2016)	Msc Thesis (*AAU-AAiT)*	-

8.	Evaluation of Drainage system in Kebena stream catchment	Addis Ababa	Eskedar (2013)	Msc Thesis *(AAU-AAiT)*	-
9.	Integrated urban drainage system; the case of Ayat to Megenagna light rail transit system route	Addis Ababa	Anteneh (2015	Msc Thesis *(AAu-AAiT)*	-
10.	Investigation of Flooding Problems in Urban Drainage System: the case at Zenebe Werk in Addis Abeba, Ethiopia	Addis Ababa	Habtamu (2017)	Msc Thesis *(AAu-AAiT)*	-
11.	Investigation on storm drainage problem of Addis Ababa (case study at Gotera – Wollo Sefer, Saris - Gotera and Ring Road)	Addis Ababa	Desalegn (2011)	Msc Thesis *(AAu-AAiT)*	-

As we seen from the above table most of the studies were conducted on the assessment of urban drainage problems, and some of them have been studied by the concept of modeling of urban drainage. From the above mentioned studies some of them are summarized as follows. Dagnachew, (2011) has studied the Road and urban stormwater drainage network integration in Addis Ketema Sub-city of Addis Ababa. In this study, the main challenge is the lake of adequate integration between road and urban stormwater drain lines followed by blockage of existing channels by solid wastes. The undersized capacity of drainage channels are also found to be the major causes for overtopping during the rainy seasons. Similarly, with the objectives of surface water drainage problems and its network integration systems assessment, Getachew and Tamene, (2015) have conducted a study in Ginjo Guduru Kebele of Jimma town. Getachew and Tamene were concluded that road surface drainage of the study area found to be inadequate due to insufficient road profile, insufficient drainage structures provision, improper maintenance and lack of proper interconnection between the road and drainage infrastructures thereby resulting damages to road surfacing material and flooding problems in the area. The other study was conducted by Mitiku and Mekdes, (2017), with objective of assessing the approach to drainage system sustainability in Wolaita Soddo town. Inadequate coverage, poor quality and inappropriate provision of drainage infrastructure were problems identified in this study.

From above mentioned studies some of them were conducted by using model and summarized as follows. Nejib (2016) has studied Modeling and Analyses of Urban flooding in Bole Sub city System Performance and Evaluation of Possible Improvements by using SWMM. Nejib was concluded that road surface drainage of the study area found to be inadequate due insufficient road profile, insufficient drainage structures provision, improper maintenance and lack of proper interconnection between the road and drainage infra-structures thereby resulting flooding problems in the area. Similarly with the objective of investigating of the drainage problems of Adama town along the Nazareth-Assela road and propose remedial measures, Meraf (2015) has conducted a study in Adama town by using HEC RAS.

After its inception, Federal Urban Planning Institute (FUPI) (the then NUPI) has been involving in planning and design of urban storm water drainage facilities as part of the Master/Development Plan of a city/town with the objective of keeping the life of urban infrastructure and to protect the urban environment like water pollution from non-point sources of storm water, Air pollution from stagnated water and Soil from erosion and degradation.

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

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

2.7. Urban Stormwater Drainage Conditions in Assosa

The absence of adequate stormwater drainage network in the town that feed the existing main channels resulted in underutilization of the channels discharge capacity thereby enhancing the problems of over flooding of residential, commercial, institutional areas and overland flow to create inconvenience on the traffic movement and urban sanitation.

The existing conditions of drainage system at the study area were investigated in detail, and photos are taken to show the existing conditions of each and every drain. Most of the existing urban drainage systems of Assosa town are not maintained and managed properly. As shown in figure 2.4, due to poorly defined inlets & outlets stagnant water is formed in the open drains.

Figure 2.4: Rectangular open drains with poor outlet (July, 2018)

According to the Benishangul-Gumuz Regional State (BGRS) asset management plan report (2017), total roads length in the Assosa town is about 169.24 kilometers by considering all types of roads (primary, secondary and tertiary). Approximately, 26.5 % of the total length is covered by drainage facilities and remaining 73.5 % of the roads are not having drainage facilities. From this analysis the special integration between all roads and urban stormwater drainage network is found to be 26.5 %. The implication of this is that for a kilometer of road about 265 meter of it has drainage line. Whereas 73.5% of the existing road does not have a drainage system, which implies that considering a kilometer of road 735 meter of it is without a drainage line. The main point that should be underlined here is that even if 26.5% of the road network is integrated with drainage line; a significant part of the network is integrated with one side drainage structure.

2.7.1. Solid Waste Management in Assosa

Dumping solid waste materials in to drainage is the major challenge of stormwater drainage system in study area. After each occurrence of flooding and storm, wastes are dumped in ditches and drainage channels. Theses drainage channels remains unattended to and thereby get clogged. This causes blockage of channels for the subsequent runoffs and other contents. Urban litter (alternatively called trash, debris, junk, floatables, gross pollutants, rubbish or solid waste) has become a major problem in the study area it typically consists of manufactured materials such as bottles, plastic and paper wrappings, newspapers, shopping bags, cigarette packets and remains of chat. As a result of damping these solid wastes in to drains the drainage system has been clogged and causes flooding over streets and walk ways. Figure 2.5 shows the deterioration of the functionality of the drains in these areas of study. Also as this blockage exists, the road pavement attached to these drains is also under threat. Water builds up on the pavement (flood) thereby causing a wear and tear, with washing of bitumen and other road components into drains thereby causing further damage and leading to drain failures.

Figure 2.5: Excess sediment and dumping of solid waste in to drainage system (July, 2018)

Similarly, some of the manholes or catch pits in the study area have been clogged with waste and blocked due to lack of clearance. As a result the runoff that is generated in that sub basin over flows with higher velocity which erodes the ditches as well as the road and walk ways. Figure 2.6 shows flow over the manholes.

Figure 2.6: Failed drainage manhole (July, 2018)

2.7.2. Liquid Waste Management in Assosa

Like solid waste management, liquid waste management has its own setback. Assosa city, as most of urban centers in Region, can hardly manage the liquid waste. In such away inadequacy and loose organization towards such program is seen clearly within the realm of its administration; hence liquid waste management should be one of the issues for which the city administration has to give due attention.

The main sources of liquid wastes are households (residential units), commercial establishments and hotels. As there is no adequate provision for garbage container in the city, residents dispose wastes into the drainage channels, bridge/ culvert liquid open spaces. The conditions are more sever at flat areas.

The water that is running directly into the streams is often picking up pollutants along the way. These pollutants can include motor oils and gasoline that leak from vehicles, waste from sewer lines and anything else that will float or dissolve in water. Most of the drainage lines in study area oblige as waste disposal and clogged by liquid and solid wastes. Aside its' challenge to the drainage system, it could also cause a health problem and also it degrades the aesthetic value of the environment. Figure 2.7 shows the release of liquid waste in the study area.

Figure 2.7: Liquid and solid waste in the drainage system (July, 2018)

2.8. Urban Flood Mitigation Measures

Flood mitigation options are measures that are aimed at reducing the flood risk on a floodplain. This can be achieved with a combination of structural and non-structural measures.

Structural measures are those that change the flood behavior to reduce flood risk. This may be through lowering flood levels (example creek diversions, capacity improvements) or by changing the direction of flows or areas of inundation (example levees, flood gates). Alternatively, the magnitude of flows on the floodplain can be reduced through upstream retardation measures (example dams, basins).

Non-structural measures are those that do not change the flooding behavior but alter how the people and property on the floodplain are affected by floodwaters. These measures include removing people from the floodplain (example land-use zoning, voluntary purchase) or reducing the damage caused by floodwaters (building resilience). As well, non-structural measures include changing the way that people prepare for a flood (flood warnings, readiness) and act during (evacuations) and after a flood event (recovery).

In view with the local actual situation of study area, effective, rational, economic and feasible corrective measures are to be developed to tackle the problems. In general, the following main points should be considered:

A) ***Improved Facility Management Service***: facility maintenance service is the integrated management to enhance the performance of the infrastructure. Facility maintenance is often seen as an annoyance or as a "necessary evil." This is partially due to the assumption that facility maintenance generates costs but does not give much in return. What is not understood is that high-quality maintenance has many positive, mostly indirect, effects on the infrastructure performance. In most towns of Ethiopia, asset management practice is poor; most infrastructures are maintained if there are some ceremonies. The situation here in Assosa is not different from this, which needs improvement by the concerned bodies.

B) ***Maintenance of Existing Drains***: As observed in the study area, cleaning up and maintenance before the rainy season should be done in all the channels to avoid sedimentation and siltation which are one of the main causes of poor drainage. It is believed that if this is done in place it will mitigate the problem of stagnant water and flooding problems significantly in the town.

C) ***Construction of Additional Drains***: As there are areas without stormwater drains, the construction of additional drainage infrastructure is a must at the study area. Considering the slope of the area and runoff condition, it is recommended that provide drainage system the victim area. There are also local earth roads which have no drainage system at all. On these roads are difficult to walk during the rainy season because of mud accumulation. So these unpaved roads should be reconsidered with its proper drainage system.

D) ***Sustainable Urban Drainage Systems (SUDS):*** The current drainage system is conventional drainage type at the study area. But it is better to develop sustainable urban drainage systems (SUDS) or Low Impact Development (LID) to improve drainage and reduce the volume of surface at the study area. The use of green space in the design of

SUDS allows water to be controlled using trees and vegetation, green roofs, ponds and wetlands. Green roofs can especially be implemented in order to increase interception, stormwater storage and evaporation in highly urbanized areas where the space to introduce green infrastructure is restricted.

E) ***Awareness Creation at Community Level***: It is clearly observed that the attitude of some urban communities is poor towards drainage system. A paradigm shift of the behavioral pattern of the urban community (with respect to indiscriminate dumping of refuse, building without leaving appropriate setbacks) is very crucial to the mitigation of flooding in the study area. This can be done by formulating a wide campaign on public awareness and sensitization through seminars and workshops concerning environmental protection.

F) ***Dust Bins:*** the other mitigation measure is that the garbage containers should be provided at central and selected place. Therefore, by providing garbage containers on main and collector roads or by encouraging small enterprises it is better to reduce the dumping of solid waste in the open drains, manholes and culverts.

3. MATERIALS AND METHODS

3.1. Description of the Study Area

This study is conducted in Assosa town which is the capital of Benishangul-Gumuz Regional State (BGRS) of Ethiopian. It is located at the north western part of the country, 678 km away from the nation’s capital, Addis Ababa. The total area of the town is estimated to be 1345ha. The town is divided in to four Kebeles with a total population of 30,800 as per the estimate given in 2011.

The town could be considered as one of the border towns in the country. It is located at 90 km away from the Ethio-Sudanese border. It is situated on a relatively flat plane at an average altitude of 1525 m asl. Geographically, the town is located between $10^0$00’ and $10^0$03’ north and between $34^0$35’ to $34^0$39’ east. It lies on an area of approximately 982.5 ha. It is surrounded by resettlement villages: in the north by Amba 8 and Amba 3, in the East by Amba 4 and in the South by Amba 38 (National Urban Planning Institute, 1995).

Assosa town is located in the ‘kola’ climate zone. It has mean maximum and minimum temperature of 33.7^0c and 11.6^0c respectively. The maximum temperature varies between 23.8^0c to 33.7^0c. The mean annual rainfall is about 991.5 mm. the rainy season extends from April to November, but the maximum rainfall occurs in summer season (i.e. between June and August).

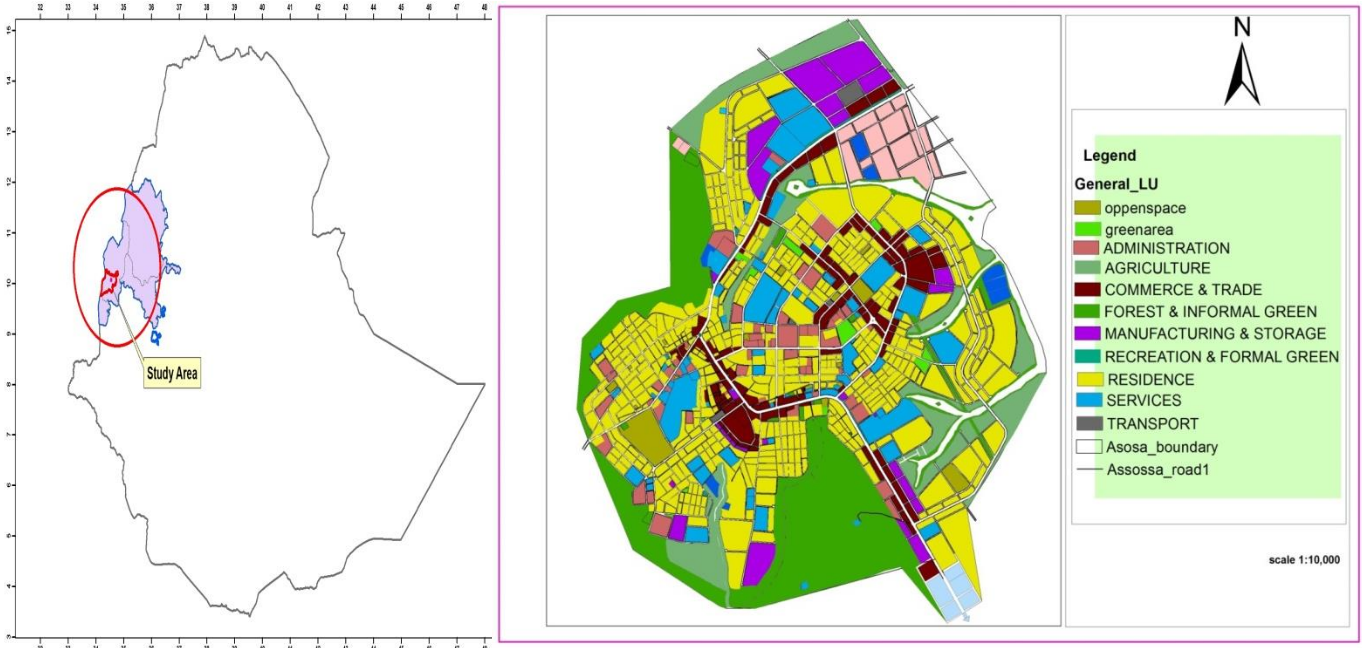

Figure 3.1: Location map of Assosa town (Assosa town Municipality, 2014)

3.2. Materials

The following materials have been used to conduct this research:

1. Contour Map (Topo Map): In order to successfully delineate a watershed boundary, it is needed to visualize the landscape as represented by a topographic map. This map helps to examine the elevation, determine flow direction and flow length of the catchment areas.

2. Master Plan: to look into the overall conditions of land use land cover of the study area.

3. Tape meter: to measure the existing stormwater drainage lines depth, width and diameter which helps to evaluate the capacity of the drainage system.

4. GPS: to measure the elevation of the existing drainage system.

3.2.1. Data Types and Sources

This part contains of the types and sources of data which were used in this study. Consequently, the qualitative as well as quantitative type of data has been used for this research. Rainfall data were used to develop IDF curve and data sources for this research were both primary and secondary sources.

3.2.1.1 Primary data sources

Field survey/observation, interview of professionals and results were the primary data sources which were engaged in this study.

3.2.1.2. Secondary data sources

Meteorological data (rainfall data) from National Meteorological Agency of Ethiopia, master plan, other findings/ literatures and reports were secondary data sources which that were used for this particular research.

3.3. Methodology

3.3.1. Research Development

Evaluating urban stormwater drainage system is challenging and hence needs an ample methodology. Two types of methodologies were used to perform this research.

The descriptive type was used to describe challenges and factors which impaired the performance of stormwater drainage system and the condition of the study area. Whereas, the exploratory type was particularly used to explore the existing condition and coverage of urban stormwater drainage facilities which have been used by the town and challenges of stormwater management practices for the existing drainage problem.

3.3.2. Data Collection Techniques

Field Survey

Field survey was employed to measure the dimensions of drainage lines located in the study area, to gather information about the current condition of the drainage system with the help of master plan and check list as per the objective of this study.

Interview

Generally, this was employed for the professionals in the study area to collect data related to flooding and major flood prone areas, major challenges to stormwater drainage management, impacts of drainage system in the town and possible suggestions from expert point of view so as to handle the challenges of the drainage system in the study area.

3.3.3. Data Analysis

The collected data were analyzed with the help of known computer software including:

- ARC-GIS: to analyze the spatial data and delineate the catchments of the study area.
- Auto CAD: to integrate it with contour map and determine the flow direction for each drainage line.

- SWMM: is a dynamic rainfall-runoff simulation model used for single event or long-term (continuous) simulation of runoff quantity and quality from primarily urban areas.
- Microsoft excels: to calculate the runoff on the excel spreadsheet and prepare a comparison graph between the calculated (rational formula) output and the model output runoff.

3.3.4. Data presentation

The analyzed data have been presented with Statistical tools such as tables and graphs. SWMM figures and digital photos, which were captured during field survey, have also been incorporated as evidence to summarize the findings and field surveys.

3.4. Research Framework

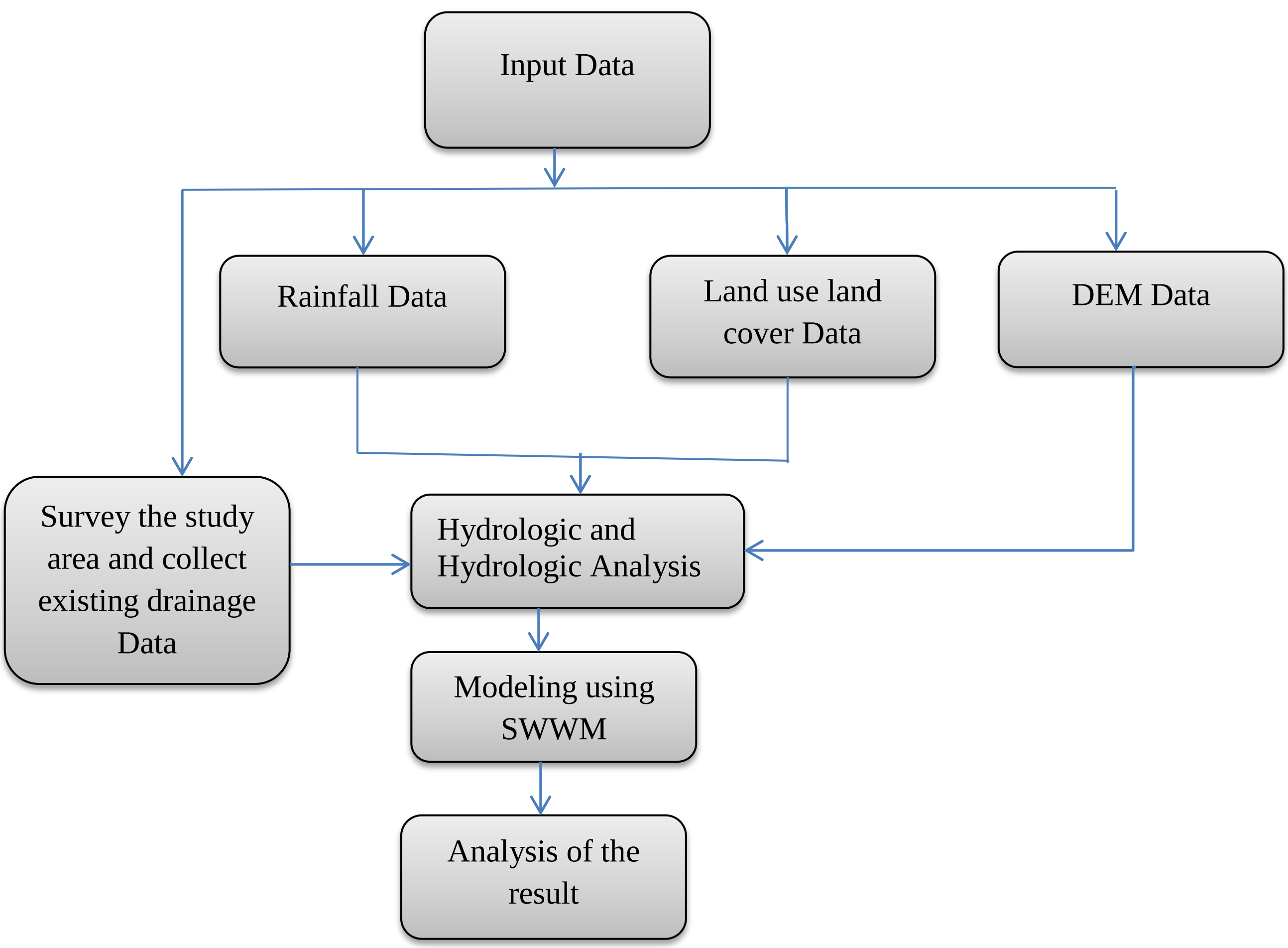

Figure 3.2: Conceptual frame work

3.5. **Hydro Climatic Data Availability and Its Quality**

3.5.1. Rainfall Data

The main data required for this thesis is rainfall data. Rainfall data for Assosa town can be obtained from only one station, namely Assosa meteorology Station. The data includes the record of daily rainfall precipitation throughout the year ranging from 1986 to 2017 for 32 consecutive years.

The collected data from the Ethiopian meteorology agency has missing data for the consecutive years starting from 1990 up to 1994 and from 1997 up to 1999. But the extreme rainfall events no need to fill the missing data.

Check the Quality of Data

An outlier is an observation that deviates significantly from the bulk of the data, which may be due to errors in data collection, or recording, or due to natural causes. The presence of outliers in the data causes difficulties when fitting a distribution to the data. Low and high outliers are both possible and have different effects on the analysis. Testing of high and low outliers was done using the equation

$Y_H = Y_m + K_n * S_y$ ……………………………………………………………Equation (3.1)

$Y_L = Y_m - K_n * S_y$ …………………………………………………………..….Equation (3.2)

where Y_H, Y_L are high and low outlier thresholds in log and Y_m is the mean, n is the sample size, S_y is the standard deviation and K_n is the parameter given in Chow et al.(1988), for sample sizes varying from 10 to 140.

According to Chow et al. (1988), if the station skew is greater than +0.4, tests for high outliers are considered first; if the station skew is less than -0.4, tests for low outliers are considered first. Where the station skew is between ±0.4, tests for both high and low outliers should be applied before eliminating any outliers from the data set.

3.5.2. Return Period –Reoccurrence of Flood

Subsequently, Rational Method is used to relate or compare the results. Return period, also called recurrence interval is a term commonly used in hydrology. It is the average time interval between the occurrence of storms and floods of a given magnitude. The selection of the modeling return period depends on economic balance between the costs of periodic repair or replacement of the structure, potential flood hazard to property, expected level of service, budgetary constraints as well as the magnitude and risk associated with damage from larger flood events. For structures where a potential damage or functional operational requirement warrants a more severe criterion a greater modeling recurrence interval should be used.

3.5.3. Rainfall Frequency Analysis

The objective of frequency analysis of hydrologic data is to relate the magnitude of extreme events to their frequency of occurrence through the use of probability distributions (Chow, et al., 1988). Since rainfall data of previous years are available for the project area, the design rainfall can be determined by frequency analysis. Furthermore, the IDF curve can be developed using the frequency analysis.

The historical rainfall data available is a 24hr duration rainfall hence an appropriate IDF reduction method needs to be used to obtain rainfall intensities of shorter duration. The IDF reduction method suggested in Ethiopian Road Authority Drainage Design Manual 2013 is used in this study.

For accurate hydrologic analyses, reliable rainfall intensity estimates are necessary. The IDF relationship comprises the estimates of rainfall intensities of different durations and recurrence intervals. There are commonly used theoretical distribution functions that were applied in different regions all over the world; (e.g. Generalized Extreme Value Distribution (GEV), Gumbel, Pearson type III distributions) (AlHassoun, 2011). Two common and suggested by Ethiopian Drainage Design Manual (ERA, 2013) Gumble theory of distribution and log person type-II frequency analysis techniques were used to develop the relationship between rainfall intensity, storm duration, and return periods from rainfall data for this study.

Gumble theory of distribution

Gumble distribution methodology was selected to perform the flood probability analysis. The Gumble theory of distribution is the most widely used distribution for IDF analysis owing to its suitability for modeling maxima. It is relatively simple and uses only extreme events (maximum values or peak rainfalls). The Gumble method calculates the 2, 5, 10, 25, 50 and 100- year return intervals for each duration period and requires several calculations. Frequency precipitation P_T

(mm) for each duration with a specified return period T (in year) is given by the following equation.

$P_T = P_{ave} + KS$ ……………………………………………………………………Equation (3.3)

Where K is Gumbel frequency factor given by:

$$K = -\frac{\sqrt{6}}{\pi}\left[0.5772 + \ln\left[\ln\left[\frac{T}{T-1}\right]\right]\right]$$ ………………………………………Equation (3.4)

Where P_{ave} is the average of the maximum precipitation corresponding to a specific duration.

$$P_{ave} = \frac{1}{n}\sum_{i=1}^{n} P_i$$ ……………………………………………………………..Equation (3.5)

Where P_i is the individual extreme value of rainfall and n is the number of events or years of record. The standard deviation is computed using the following relation:

$$S = \left[\frac{1}{n-1}\sum_{i=1}^{n}(P_i - P_{ave})^2\right]^{1/2}$$ ………………………………………………Equation (3.6)

where S is the standard deviation of P data. The frequency factor (K), which is a function of the return period and sample size, when multiplied by the standard deviation gives the departure of a desired return period rainfall from the average.

Log Pearson type-III

The Log Pearson type-III probability model is used to calculate the rainfall intensity at different rainfall durations and return periods to form the historical IDF curves for each station. Log person type-III distribution involves logarithms of the measured values. The mean and the standard deviation are determined using the logarithmically transformed data.

In the same manner as with Gumble method, the frequency precipitation is obtained using log person type-III method. The simplified expression for this latter distribution is given as follows:

$P^* = \log(P_i)$……………………………………………………………Equation (3.7)

$P^*_T = P^*_{ave} + K_T S^*$…………………………………………………..Equation (3.8)

$$P^*_{ave} = \frac{1}{n}\sum_{i=1}^{n} P^*$$ ………………………………………………………...Equation (3.9)

$$S^* = \left[\frac{1}{n-1}\sum_{i=1}^{n}(P^* - P^*_{ave})^2\right]^{1/2}$$ …………………………………Equation (3.10)

Where P^*_T, P*avg and S^* are frequency precipitation, average of the maximum precipitation corresponding to a specific duration and the standard deviation of P^* data based on the logarithmically transformed P_i values respectively. K_T is the Pearson frequency factor which depends on return period (T) and skewness coefficient (C_s).

The skewness coefficient, C_s, is required to compute the frequency factor for this distribution. The skewness coefficient is computed by;

$$C_s = \frac{n\sum_i^{ni}(P_i^* - P_{ave}^*)^3}{(n-1)(n-2)(S^*)^3}$$ ……………………………………………Equation (3.11)

K_T values can be obtained from tables in many hydrology references; for example (reference Chow (1988)). By knowing the skewness coefficient and the recurrence interval, the frequency factor, K_T for the log person type-III distribution can be extracted. The antilog of the solution in P^*_T will provide the estimated extreme value for the given return period.

Goodness of Fit Test

The goodness of fit (GOF) tests measures the compatibility of a random sample with a theoretical probability distribution function. These tests show how well the selected distribution fits to data. There are three most commonly used GOF tests. These tests are the Anderson-Darling, the Kolmogorov-Smirnov, and the Chi-Squared tests. In all three tests a parameter or statistic unique to each method is calculated for the required distribution types and these distributions are ranked based on their parameter values.

Kolmogorov-Smirnov Test

This test is used to decide if a sample comes from a hypothesized continuous distribution. It is based on the empirical cumulative distribution function (ECDF). Assume that we have a random sample x_1, x_2..., x_n from some continuous distribution with CDF F(x). The empirical CDF is denoted by

$$F_n(x) = \frac{1}{n} \cdot \left[\text{Number of observations} \leq x\right]$$

The Kolmogorov-Smirnov statistic (D) is based on the largest vertical difference between *F(x)* and *Fn(x)*. It is defined as

$$D_n = \sup_x | F_n(x) - F(x) |$$

When comparing different distribution lower statistics means better fit.

Anderson-Darling Test

The Anderson-Darling procedure is a general test to compare the fit of an observed cumulative distribution function to an expected cumulative distribution function. This test gives more weight to the tails than the Kolmogorov-Smirnov test.

The Anderson-Darling statistic (A^2) is defined as

$$A^2 = -n - \frac{1}{n} \sum_{i=1}^{n} (2i-1) \cdot [\ln F(X_i) + \ln(1 - F(X_{n-i+1}))]$$

The hypothesis regarding the distributional form is rejected at the chosen significance level (alpha) if the test statistic, A^2, is greater than the critical value obtained from a table. When comparing different distribution lower statistics means better fit.

Chi-Squared Test

The Chi-Squared test is used to determine if a sample comes from a population with a specific distribution. This test is applied to binned data, so the value of the test statistic depends on how the data is binned. Although there is no optimal choice for the number of bins (k), there are several formulas which can be used to calculate this number based on the sample size (N). For example, EasyFit employs the following empirical formula:

$$k = 1 + \log_2 N$$

The data can be grouped into intervals of *equal probability* or *equal width*. The first approach is generally more acceptable since it handles peaked data much better. The Chi-Squared statistic is defined as,

$$\chi^2 = \sum_{i=1}^{k} \frac{(O_i - E_i)^2}{E_i}$$

Where Oi is the observed frequency for bin i, and Ei is the expected frequency for bin i calculated by

$$E_i = F(x_2) - F(x_1)$$

Where F is the CDF of the probability distribution being tested, and x_1, x_2 are the limits for bin i. When comparing different distribution lower statistics means better fit.

Easy Fit 5.6 Professional software is used for testing goodness of the recommended Log Pearson type-III and Gumble Methods.

3.5.4. Development of Intensity- Duration- Frequency Curve (IDF Curve)

Rainfall of a place can be completely defined if the intensities, durations and frequencies of the various storms occurring at that place are known. Whenever an intense rain occurs, its magnitude and duration is generally known from meteorological readings. This available data can be used to determine the frequencies of the various rains. Such frequency data for storms of various durations can be represented by Intensity-Duration- frequency (IDF) curves. An IDF curve is a plot of average rainfall intensity (rainfall depth is averaged over the duration i.e. $\frac{Rainfall\ depth}{Duration}$=average rainfall intensity in that duration).

The rainfall depths obtained from gauging station are of 24hr duration depth. Design and analysis of drainage structures require rainfall intensity duration relationship of shorter duration. Because rainfall data of shorter duration is unavailable, appropriate IDF derivation for shorter duration is required. Ethiopian Road Authority (ERA) Drainage Design Manual of 2013 suggests the following equation for calculation of shorter duration rainfall from 24hour duration rainfall.

$$R_{Rt}=\frac{t(b+24)^n}{24(b+t)^n}$$ ………………………………………………..…Equation (3.12)

Where: R_{Rt} = Rainfall depth ratio Rt: R_{24} Rt = Rainfall depth in a given duration t

R_{24} = 24hr rainfall depth Coefficients; b=0.3 and n= 0.78 - 1.09 (Fiddes et al., 1974)

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

- Using the trend line equation obtained Log Pearson-III distribution method of frequency analysis, i.e. y = -37.76ln(x)+205.92 where y is 24-hour rainfall depth (R_{24}) of a return period x under consideration, R_{24} is calculated for 2, 5, 10, 25,50 and 100 year return period.

  Rearranging the above equation gives

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

  Substituting Intensity (mm/hr) It= $\frac{R_t}{t}$ in the above equation

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

- Using b = 0.3 and n = 0.92 as suggested by ERA manual results are tabulated for rainfall durations 10, 20, 30 … 180 minutes. The resulting table is graphed for each return period.
- The resulting table is graphed for each return period. That is IDF curve is developed using reduction formula.

3.5.5. The Rational Formula Method

This method is widely used for peak discharge determination from smaller catchments (less than 0.5 km^2 areas) as recommended by Ethiopian Road Authority drainage modeling manual. Here on this thesis, the peak discharge from fifty (50) sub catchment areas are determined by rational method. The idea behind the rational method is that if a rainfall of intensity (I) ,begins instantaneously and continues indefinitely, the rate of runoff will increase until the time of concentration t_c, when all of the watershed is contributing to flow at the outlet. The product of rainfall intensity I and watershed area A is the inflow rate for the system, IA, and the ratio of the peak discharge Q to this inflow rate expresses the coefficient of runoff C. The discharge is expressed in the rational formula as the following form:

Q =0.00278 CIA ……………………………………………………..Equation (3.13)

Where:

Q: Peak runoff rate, runoff flow (m^3/s)

I: is rainfall intensity (mm/hour)

A: is catchment area (hectares)

C: runoff coefficient

The formula follows the following assumptions:

- The peak probability to happen (return period) is equal to the rainfall intensity
- The runoff coefficient C is constant during the rain storm
- The concentration time is approached

Runoff Coefficient: C-Value

The runoff coefficient (C value) is a dimensionless empirical- constant value that represents the percentage of the rainfall that becomes runoff (Rossman, 2010). It assumed to vary according to time and rainfall intensity.

The C-value varies and depends on permeability of the surfaces, where the areas with low infiltration capacity (impervious surfaces, urban areas, steeped gradient) compromise high C-value comparing permeable surfaces (forest, cultivated land, flat surfaces and pervious surfaces). In another word, impermeable surfaces produce more runoff than the permeable one (Lindholm, 2014).

The high C-value means low infiltration capacity of the surfaces and increases the risk for urban flash/surface flooding.

$$C_{weigted}=\frac{\sum A_i * C_i}{A_T} \quad \text{...Equation} \quad (3.14)$$

Where; C i- runoff coefficient for a given hydrologic soil group area

Ai -area under each hydrologic soil group and

A_T -total catchment area considered

Table 3.1: Runoff coefficient C-value (Norskyyann, 2012)

Type of surface	Runoff coefficient
Impervious surfaces (rooftop, concrete t/asphalt, mountain	0.85-0.95
Urban centre- dens inhabited areas	0.7-0.9
Apartment/townhouse	0,6-0,8
Detached/family houses area	0.5-0.7
Gravel/ unpaved road	0.5-0.8
Lawns, cultivated land, parks, cemeteries	0.3-0.5
Industrial areas	0.3-0.9
For flat area and permeable surfaces low values are considered	

Time of Concentration

Time of concentration or what also called Travel- time is defined as the time between the occurrences of rainfall event until excess water leaves the catchment at the very most downstream outlet (Laurenson, 1964). In spite of the importance of time of concentration, it is sometimes very difficult to determine. There are several different methodologies for calculating time of concentration; however, the most common form, and the method used for this study, is the NRCS Velocity Method. In this method, time of concentration is the summation of the respective travel times (T_t) for the different methods of flow: sheet flow, shallow concentrated flow, and open channel flow (USDA, 1986). The detail computation of time of concentration is presented at Appendix 3.

i. Sheet Flow

Sheet flow occurs as water flows across plane surfaces, typically near the beginning of stream formation. Studies have shown that the maximum length of sheet flow is limited to 100 to 130 meters. According to the NRCS Velocity Method, the travel time for sheet flow is calculated as

$T_t = [0.091 * (nL)^{0.8}]/(P_2)S^{0.4}$ ……………………………………..Equation (3.15)

Where: T_t = travel time, hr: n = Manning's roughness coefficient: L = flow length, m P_2 = 2 year, 24 hour rainfall, mm S = slope of hydraulic grade line (land slope), m/m.

ii. Shallow Concentrated Flow

After a maximum of 100 meters, sheet flow usually becomes shallow concentrated flow. And the average velocity for this flow can be determined by the following formula, in which average velocity is a function of watercourse slope and type of channel. (ERA, 2002)

For Unpaved $V=4.9178 * (S)^{0.5}$ ………………………………………….Equation (3.16)

For paved $V=6.1961 * (S)^{0.5}$ ………………………………...…….Equation (3.17)

Then, the velocity is divided to the flow distance using equation below to determine the total overland flow time.

$T_{travel}=\frac{L}{60*V}$ ………………………………………………………...…Equation (3.18)

Where:

Tt= ravel-time of concentration (minutes)

L = flow length, m

V=Average velocity over the surface (m/s)

S = slope of hydraulic grade line (watercourse slope), m/m

Channel Flow Time

Open channel flow occurs where shallow concentrated flow enters a storm conveyance system (i.e. pipes, culverts, ditches and canals,). The travel time during open channel flow is calculated using with the average velocity calculation following Manning’s equation

$$V=\frac{R^{\frac{2}{3}}S^{\frac{1}{2}}}{n} \quad \text{……………………………………………………..Equation (3.19)}$$

Where *V* is the average velocity in meter per second, *R* is the hydraulic radius in meter, *S* is the slope of the hydraulic grade line in m/m, and *n* is Manning’s roughness coefficient.

The time of concentration is the sum of T_t values for the various consecutive flow segments:

$$T_c = T_{t1} + T_{t2} + \ldots + T_{tn} \quad \text{………………………………………Equation (3.20)}$$

Where:

T_c = time of concentration, hr

n = number of flow segments

3.6. Modelling Rainfall Using EP SWMM5

3.6.1. EP SWMM 5

SWMM is a dynamic rainfall-runoff simulation model used for single event or long-term (continuous) simulation of runoff quantity and quality from primarily urban areas. The runoff component of SWMM operates on a collection of sub catchment areas that receive precipitation and generate runoff and pollutant loads. The routing portion of SWMM transports this runoff through a system of pipes, channels, storage/treatment devices, pumps, and regulators. SWMM tracks the quantity and quality of runoff generated within each sub catchment, and the flow rate, flow depth, and quality of water in each pipe and channel during a simulation period comprised of multiple time steps. Even for small catchments, runoff and consequent model predictions (and prototype measurements) may be very sensitive to spatial variations of the rainfall. For instance, thunderstorms (convective rainfall) may be highly localized, and nearby gages may have very dissimilar readings.

For modeling accuracy (or even more specifically, for a successful calibration of SWMM), it is essential that rain gages be located within and adjacent to the catchment.

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

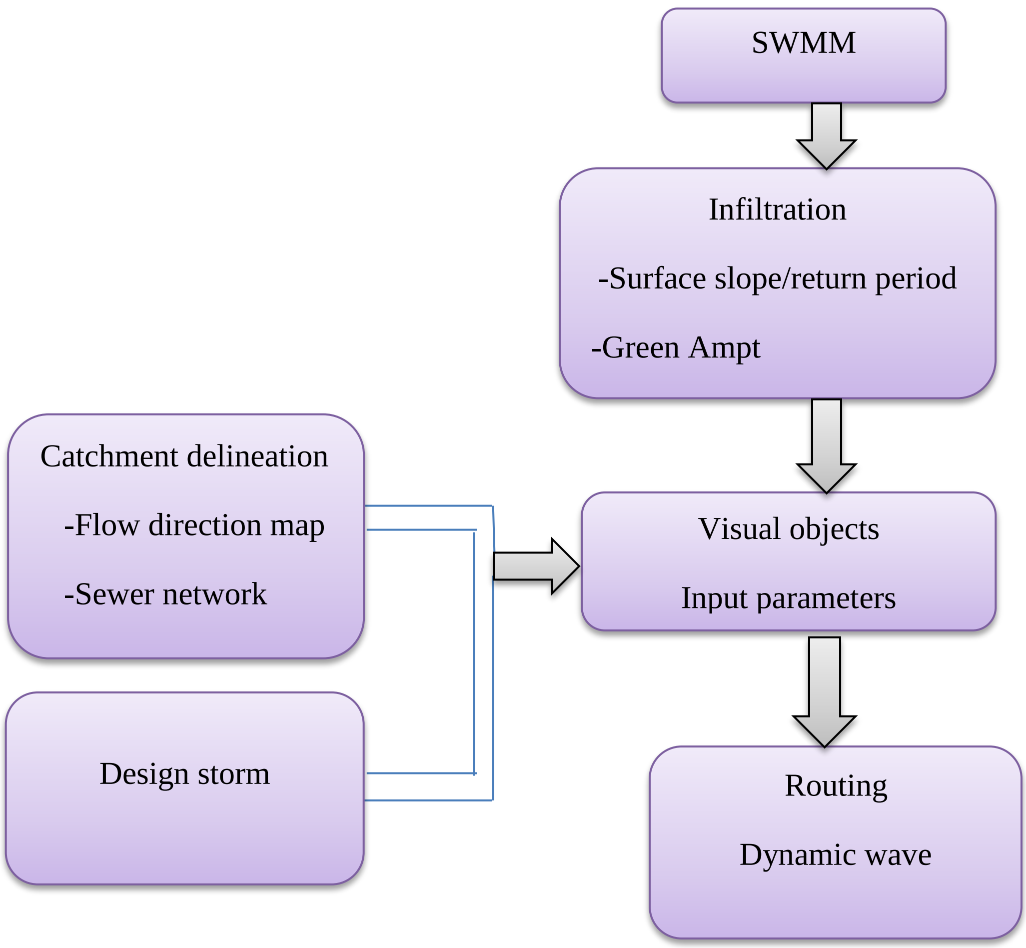

Figure 3.3: Graphical summery of the steps forming the proposed methodology in SWMM

3.6.2. Model parameterization

SWMM requires three major information for runoff quantity modeling; (1) physical catchment characteristics, (2) rainfall and (3) infiltration. The physical catchment data are total catchment area (A), percentage of impervious area (%Imp), flow width (W), average slope (So), surface depression storage and surface roughness. Most of this information was derived from Assosa town mater plan. Field survey was also carried out to confirm the surface and subsurface drainage patterns in order to accurately discretize the sub catchment areas. The area-weighted percent imperviousness was determined by summing the amount of impervious area of each sub catchment and dividing this sum by the total catchment area.

The sub catchment slope was assumed equal with the flow path slope and was estimated as the elevation difference divided by the flow path length on map. Flow width is one of the least tangible SWMM parameters. It is defined as the 'characteristic width of the overland flow path for sheet flow runoff' (Rossman, 2010). According to Rossman (2010) and Gironás et al. (2009), the width parameter can be calculated by dividing the sub catchment area by the length of the longest overland flow path in the area. Impervious depression storage (Dimp) was estimated from literatures. Within SWMM, the potential for runoff to commence immediately after rainfall is considered by allowing a percentage of the impervious area to have a zero depression storage. The default value of zero depression storage used is 25 %. The surface roughness values were adopted from the SWMM User's manual (Rossman, 2010).

The infiltration parameters required in the model are suction head (ψ_s), conductivity (Ks), and maximum moisture deficity (θ_{max}). Because it was not feasible to obtain this detailed information through field samples, the infiltration parameters adopted from SWMM manual (Rossman, 2010) based on the soil types were used (silty clay).

For each of the sub catchments documented within the model, there must be a linked rain gage to convey information concerning wet weather from either time series input or *.DAT data files. For the four outfall points, invert elevations were set at the elevation of the incoming pipe ends. As all outfalls are below the water level of sub catchments, a fixed type boundary condition with the appropriate water elevation at the mean water level of outfall points were provided.

Table 3.2: Parameters for stormwater runoff quantity modeling reported in the litrature and used in the model developed in this study

Name of parameter	Meaning		Literature range	Used values/Sensitivity to peak flow
N-Imperv	Manning's roughness coefficient for impervious area		0.01-0.014[2]	0.01
N-Perv	Manning's roughness coefficient for pervious		0.02-0.05[2]	0.02
Dstore-Perv	Depth of depression on pervious area		2.0-12.5[1]	2
Dstore-Imperv	Depth of depression on impervious area		1.3-3.8[1]	1.3
Conduit roughness	Manning's roughness coefficient for conduit		0.011-0.024[3]	0.013
Infiltration method	Green Ampt	Suction head	4.09-140.208[3]	24
		Conductivity	0.02[3]	0.02
		Initial deficity	0.26[3]	0.26

[1]City of Calgary (2000), [2]Chow V.T (1986), [3]Rossman and Huber (2016)

3.6.3. Governing Equation

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 below. Overland flow is generated by modeling the sub catchment as a nonlinear reservoir, as sketched in Figure 3.4.

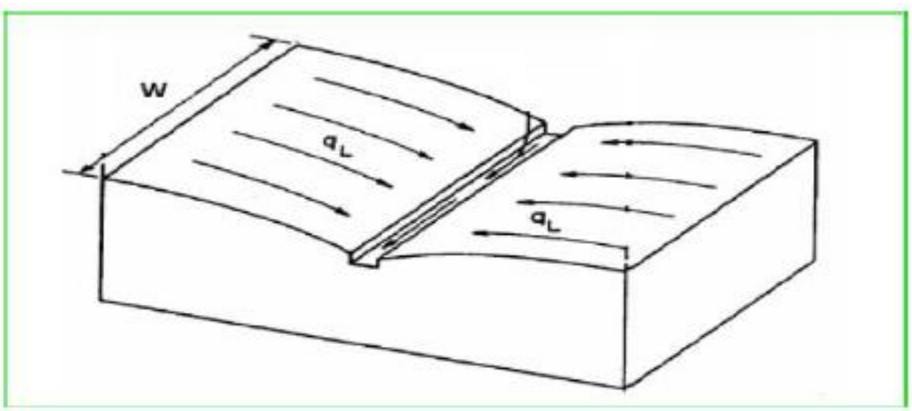

Figure 3.4: Idealized representation of a sub catchment

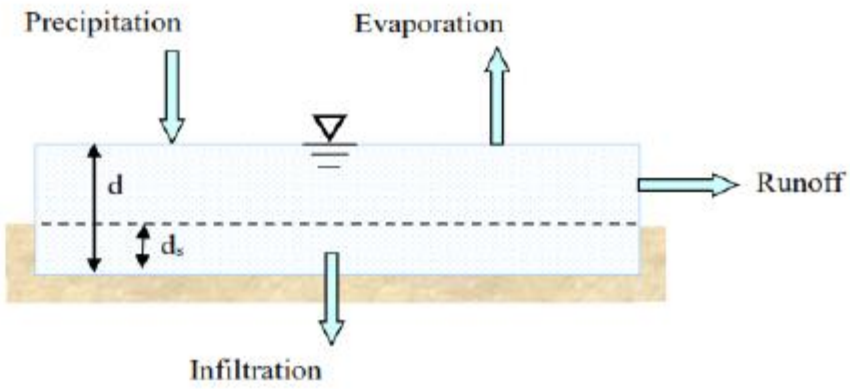

Figure 3.5 Nonlinear reservoir model of a sub catchment

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:

$$\frac{\partial d}{\partial t} = i - e - f - q$$

Where: i = rate of rainfall + snowmelt (ft/s); e = surface evaporation rate (ft/s) f = infiltration rate (ft/s); q = runoff rate (ft/s). Note that the fluxes i, e, f, and q are expressed as flow rates per unit area (cfs/ft2 = ft/s). The required rain input data in urban drainage applications depends on the nature of the engineering task. The rain data is commonly measured as intensity (mm/h) or depth (mm) and it is related to a statistical concept: frequency. The frequency is normally represented as the return period, which is the probability that a rainfall with certain intensity will be exceed or equaled in any year (Durrans & Haestad Methods, 2003).

3.6.4. Runoff Evaluation

According to Durrans & Haestad Methods (2003), the runoff is the amount of water in a rainfall that is not lost to interception, evapotranspiration or infiltration, so that it ends on water bodies or stormwater collection structures after running through the surface. Hence, the amount and characteristics of runoff not only depends on the rainfall pattern, but also on the catchment properties.

The catchments could be described as hydrological units where storm runoff and infiltration are generated in the basis of a single set of model parameters and input data. They represent the level of spatial discretization of the hydrological model (DHIa, 2011).

To simulate the generation of runoff, some parameters must be determined in the program to define the sub catchments properties. The catchment was divided in 50 smaller sub catchments connected to one node. This involves that 50 nodes are linked to sub catchments and collect the runoff generated within the area. The required input data to define the properties of the sub catchments is set by the choice of one hydrological model.

The imperviousness was calculated for each sub catchment according to the percentage of different surfaces, see the following equation.

$$\varphi = (A_1 \cdot \varphi_1 + A_2 \cdot \varphi_2 + \cdots + A_n \cdot \varphi_n)/(A_1 + A_2 + \cdots + A_n)$$ ……Equation (3.21)

Where;

φ= imperviousness of the hole sub catchment, φi=imperviousness of each type of surface, Ai=area of each surface. Table 3.3 includes the coefficients of imperviousness recommended in the Percentage for each type of surface:

Table 3.3: Impervious area as a percentage of land use (Rossman and Huber, 2016)

Land Use	Percent Impervious Area
Commercial	56
Industrial	76
High density residential	51
Medium density residential	38
Low density residential	19
Institutional	34
Agricultural	2
Forest	1.9
Open Urban Land	11

4. RESULTS AND DISCUSSIONS

4.1. Rainfall Analysis Result

Rainfall data of the area has been collected from the Assosa gauging station which is controlled by the National Meteorology Service Agency (NMSA). This is the only station available and representative for the area. Figures 4.1 below shows Assosa time series rainfall data.

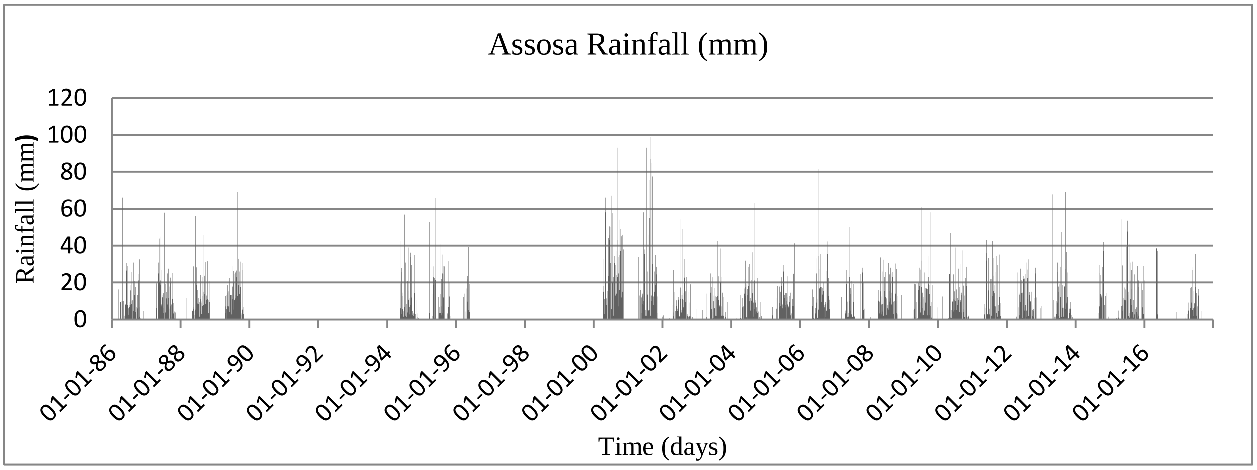

Figure 4.1: Assosa rainfall data

4.1.1. Rainfall Data Quality Result

Test for Outliers

This test helps to avoid those data lie out of the range in between the lowest and the highest values .The result of lowest datum and the highest datum are shown in the following table 4.1 and figure 4.3. The maximum daily rainfall and the full analysis of outlier test are presented at appendix 2.

Table 4.1: Tabular comparison of daily maximum rainfall prior to test for outliers

Station	Assosa
Mean	56.7
Maximum	102.5
Minimum	27.6
Higher outlier	130.9
Lower outlier	27.4
Skewness coefficient	-0.1

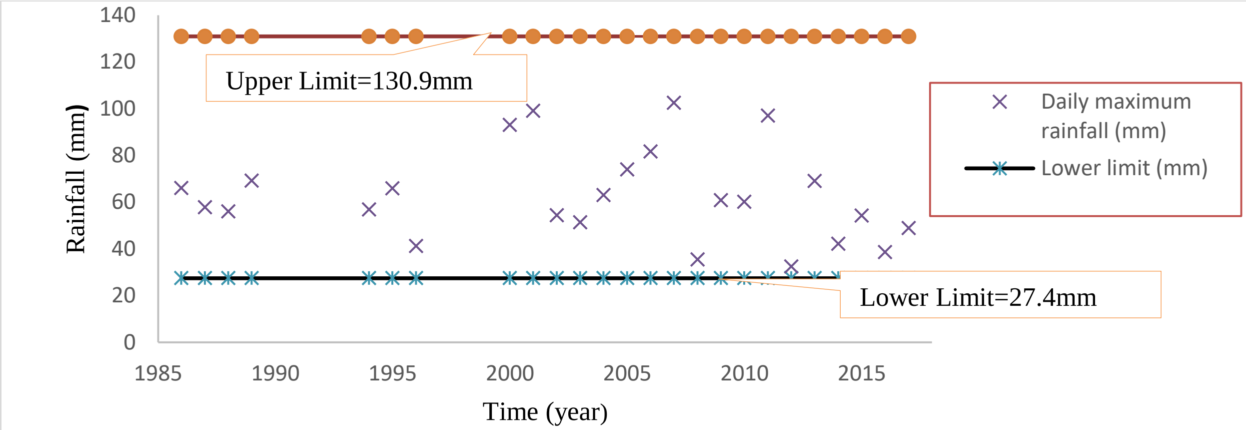

Figure 4.2 Graphical comparison of maximum rainfall prior to test for outlier

As can be observed from above table and graph there is no any data either greater than upper limit or less than lower limit. So all the available rainfall data can satisfy our condition (there is no rejection of data).

4.1.2. Rainfall Frequency Analysis

The rainfall frequency analysis is done using both Gumble and log Pearson type III methods as recommended by ERA manual 2013. The result obtained is tabulated in the following table.

Table 4.2: Yearly Extreme series frequency analysis

Return Period (Tr)	Extreme Rainfall depth (mm)	
	Gumble	Log Person Type III
2	59.58	60.25
5	76.96	78.19
10	88.47	89.33
25	103.01	102.77
50	113.80	112.37
100	124.51	121.63

In order to identify which distribution fits to the theoretical probability distribution, goodness of fit test conducted using Easy Fit 5.6 professional software and the log Pearson Type-III distribution fits for the statistical value for all the three different test methods is lesser than that of the Gumble values as tabulated below. That is, Log Pearson Type-III method have proved to be good fit in all the three tests compared to the Gumble's Method. Accordingly, the Log Pearson-III is chosen for further analysis. The statistics for both methods are calculated and the ranking is given below table 4.3.

Table 4.3: Goodness of Log-person-III and Gumbel methods

Distribution	Kolmogorov-Smirnov		Anderson-Darling		Chi-squired	
	Statistic	Rank	Statistic	Rank	Statistic	Rank
Log-person type III	0.10875	1	0.37234	1	3.0471	1
Gumble	0.11951	2	0.41768	2	4.6178	2

Table 4.4: Log pearson type-III daily heaviest rainfall analysis for Assosa

Return period(T)	Exceedance Probability	Skew Coefficient(Cs)	Frequency factor, K(T,n)	Standard deviation, Sy	Y_T	X_T(mm)
2	0.5	-0.1	0.017	0.136	1.78	60.3
5	0.2		0.846	0.136	1.89	78.2
10	0.1		1.27	0.136	1.95	89.3
25	0.04		1.716	0.136	2.01	102.8
50	0.02		2	0.136	2.05	112.4
100	0.01		2.252	0.136	2.09	121.6

4.1.3. Intensity Duration Frequency Curves

The IDF curve is developed from a 24-hour rainfall data of 26 years duration obtained from Ethiopian Meteorological Agency - gauge located Assosa town. Reduction equation as depicted in the methodology section has been applied. Consequently, the following IDF curve has been produced. The data obtained for production of IDF curve is the result of calculations using reduction formula and it is tabulated below. Then, using the data in the table the IDF curve has been produced as shown below.

Table 4.5: Duration of rainfall against their corresponding average intensities

Duration (Minutes)	Intensity for given return periods(mm/hr)					
	2 years	5 years	10 years	25 years	50 years	100 years
5	105.1	136.4	155.8	179.3	196.0	212.2
10	88.0	114.3	130.5	150.2	164.2	177.7
15	75.9	98.5	112.6	129.5	141.6	153.3
20	66.9	86.8	99.2	114.1	124.7	135.0
25	59.8	77.7	88.7	102.1	111.6	120.8
30	54.2	70.3	80.4	92.5	101.1	109.4
35	49.6	64.3	73.5	84.6	92.5	100.1
40	45.7	59.3	67.8	78.0	85.3	92.3
45	42.4	55.1	62.9	72.4	79.1	85.7
50	39.6	51.4	58.7	67.6	73.9	80.0
55	37.2	48.2	55.1	63.4	69.3	75.0
60	35.0	45.4	51.9	59.7	65.3	70.7
65	33.1	43.0	49.1	56.5	61.8	66.8
70	31.4	40.8	46.6	53.6	58.6	63.4
75	29.9	38.8	44.3	51.0	55.7	60.3
80	28.5	37.0	42.3	48.6	53.2	57.6
85	27.3	35.4	40.4	46.5	50.8	55.0
90	26.1	33.9	38.7	44.6	48.7	52.7
95	25.1	32.5	37.2	42.8	46.8	50.6
100	24.1	31.3	35.8	41.1	45.0	48.7
105	23.2	30.2	34.5	39.6	43.3	46.9

110	22.4	29.1	33.2	38.2	41.8	45.3
115	21.7	28.1	32.1	36.9	40.4	43.7
120	21.0	27.2	31.1	35.7	39.1	42.3

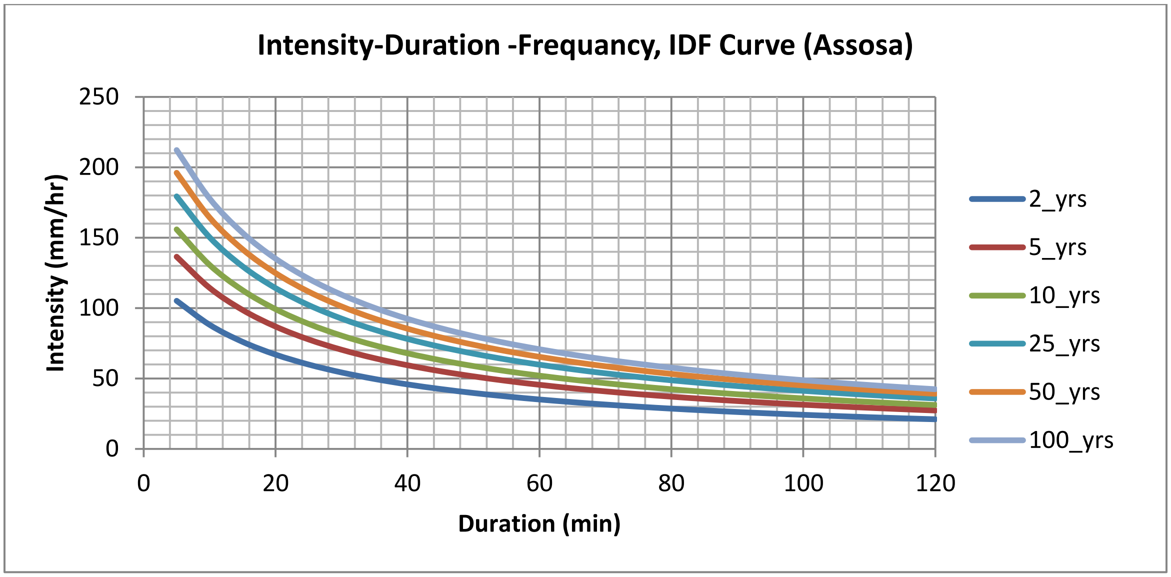

Figure 4.3: IDF Curve for Assosa from Assosa Observatory

For this study calculated IDF curve for specific study area (Assosa) is applicable, but for comparison of the result found from this study and IDF curve developed by ERA for region B1 is presented. Assosa is also found in the region B1 (Welega), but the date ranges of the data used by the authority is not mentioned, because the original release year of the manual is, 2002 it is clear that latest record they had used 2002 or earlier. Even though data ranges of this analysis and the authorities are not the same, the IDF curve developed by ERA is presented here for comparison.

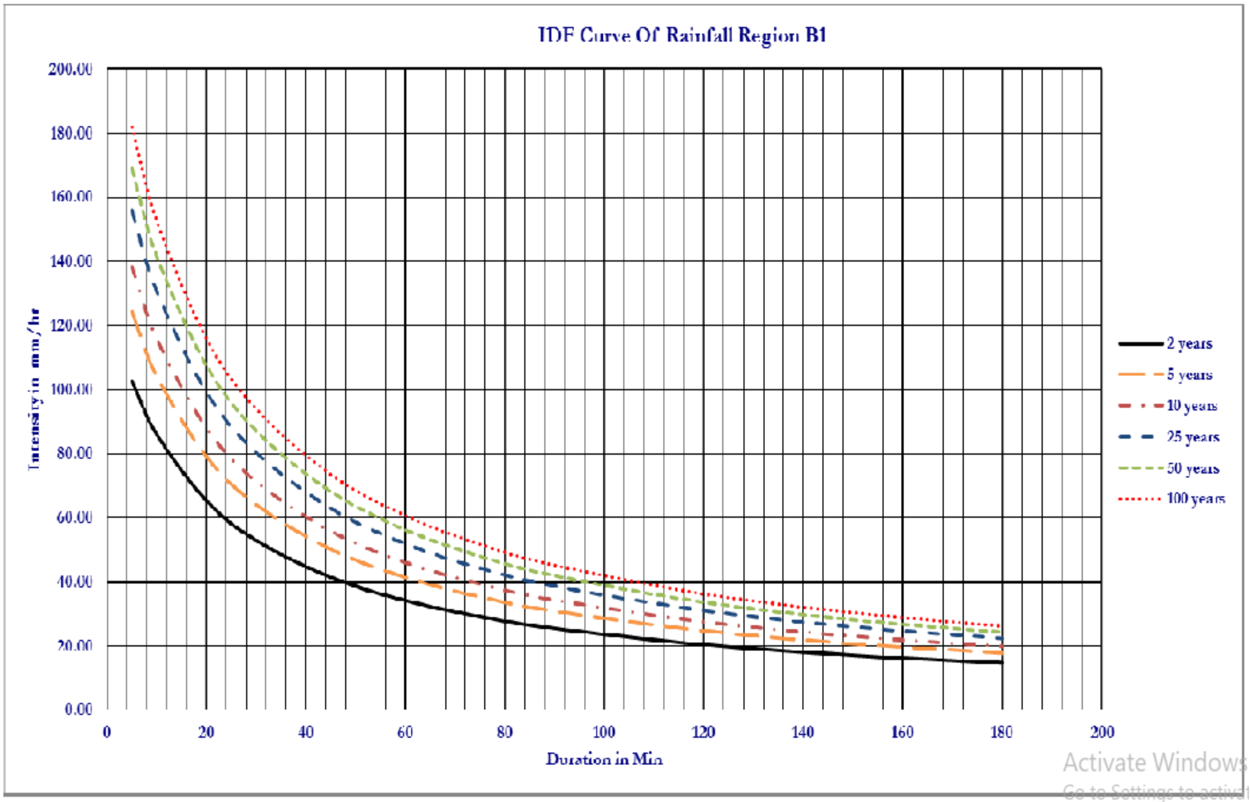

Figure 4.4: IDF curve of rainfall region B1

Table 4.6: Comparison of IDF curve results with IDF from ERA

	Present study	ERA	Present study	ERA	Present study	ERA	Present study	ERA	Present study	ERA	Present study	ERA
Duration	T-2	T-2	T-5	T-5	T-10	T-10	T-25	T-25	T-50	T-50	T-100	T-100
5	**105.1**	117.3	**136.4**	152.3	**155.8**	178.7	**179.3**	213.7	**196.0**	240.1	**212.2**	266.6
10	**88.0**	98.9	**114.3**	128.4	**130.5**	150.7	**150.2**	180.2	**164.2**	202.5	**177.7**	224.8
20	**66.9**	69.6	**86.8**	90.3	**99.2**	106.0	**114.1**	126.8	**124.7**	142.5	**135.0**	158.2
30	**54.2**	54.3	**70.3**	70.4	**80.4**	82.7	**92.5**	98.8	**101.1**	111.1	**109.4**	123.3
60	**35.0**	33.8	**45.4**	43.9	**51.9**	51.6	**59.7**	61.6	**65.3**	69.3	**70.7**	76.9
90	**26.1**	25.2	**33.9**	32.7	**38.7**	38.4	**44.6**	45.9	**48.7**	51.5	**52.7**	57.2
120	**21.0**	20.3	**27.2**	26.3	**31.1**	30.9	**35.7**	36.9	**39.1**	41.5	**42.3**	46.1

It can be seen from the two IDF curves (the one developed in this study and that of IDF curve given in the ERA drainage design manual) and the above table that the IDF curve of Assosa developed by ERA have difference with the one developed in this study. Notice that each value in the above Table 4.6 for self-study result is smaller than that of the correspondent values of ERA IDF. It would be said that the use of ERA's IDF curve is safe with regard to design purposes but it could be uneconomical.

The following are possible causes of IDF curve change.

- The recorded rainfall data length of duration can cause the changes. The rain fall data duration for this study is 26 years (Daily rainfall). Whereas that of the ERA's data range is not known. The longer the duration is the better is the result.
- The climatic trend change due to global warming produce the change as the rainfall intensity, duration and time of occurrence is highly affected. (Raul Rodriguez, *et al*, 2013).
- The other possible reason may be the ERA's IDF developed based on regional rainfall data; however the developed IDF for this study is based on site rainfall data.

4.2. Drainage Network Mapping

As chapter three mentioned, the modeled area is divided into 50 sub catchments and flows from roof top, asphaltic road and walkway. The network consists of 50 nodes means 50 junctions and 4 outfalls, 50 conduit links and 1 rain gage station. The sub catchments are denoted as S and node as a J. Whereas, conduits are donated by C, then the direction of flow in conduit can be seen after simulation, this direction is shown in filled arrows in Fig. 4.6. Green Ampt method is selected for infiltration model and Dynamic Wave is selected as routing model.

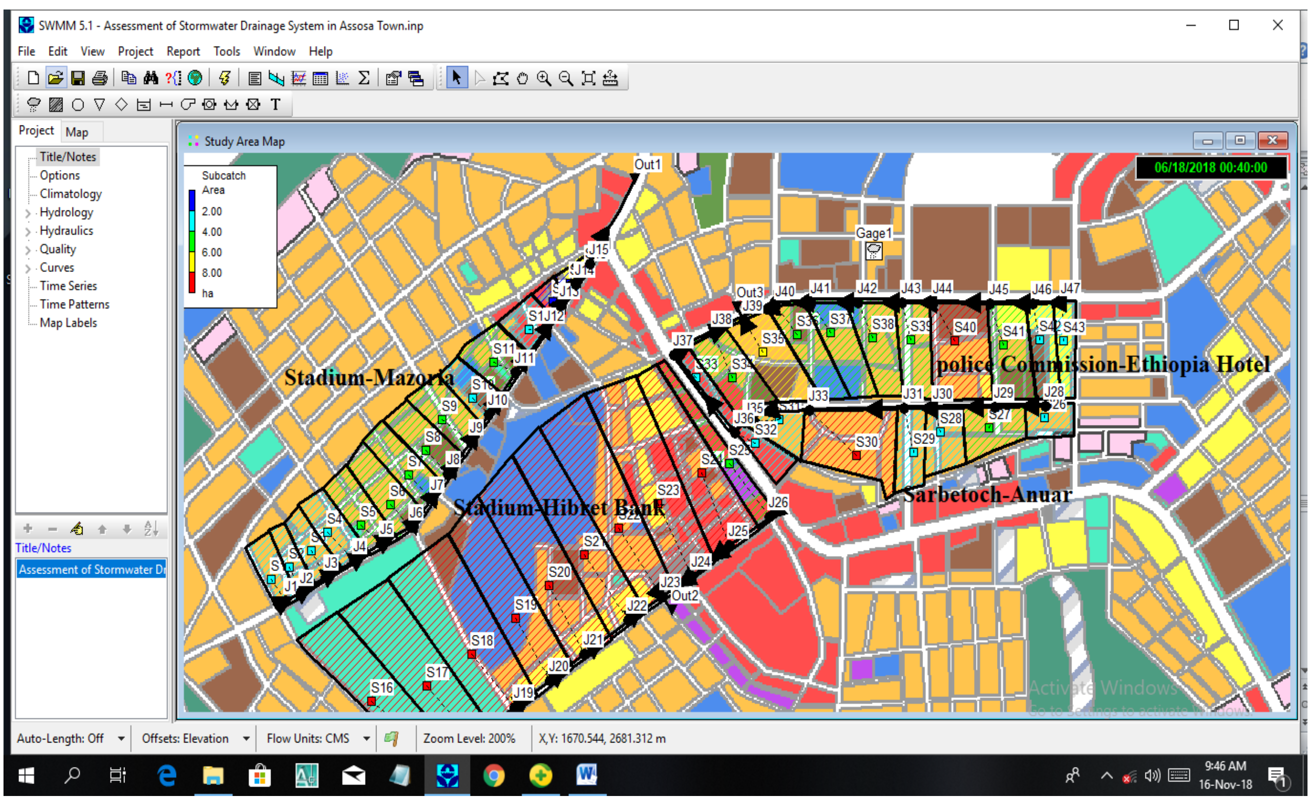

Figure 4.5: Sketch of the model map

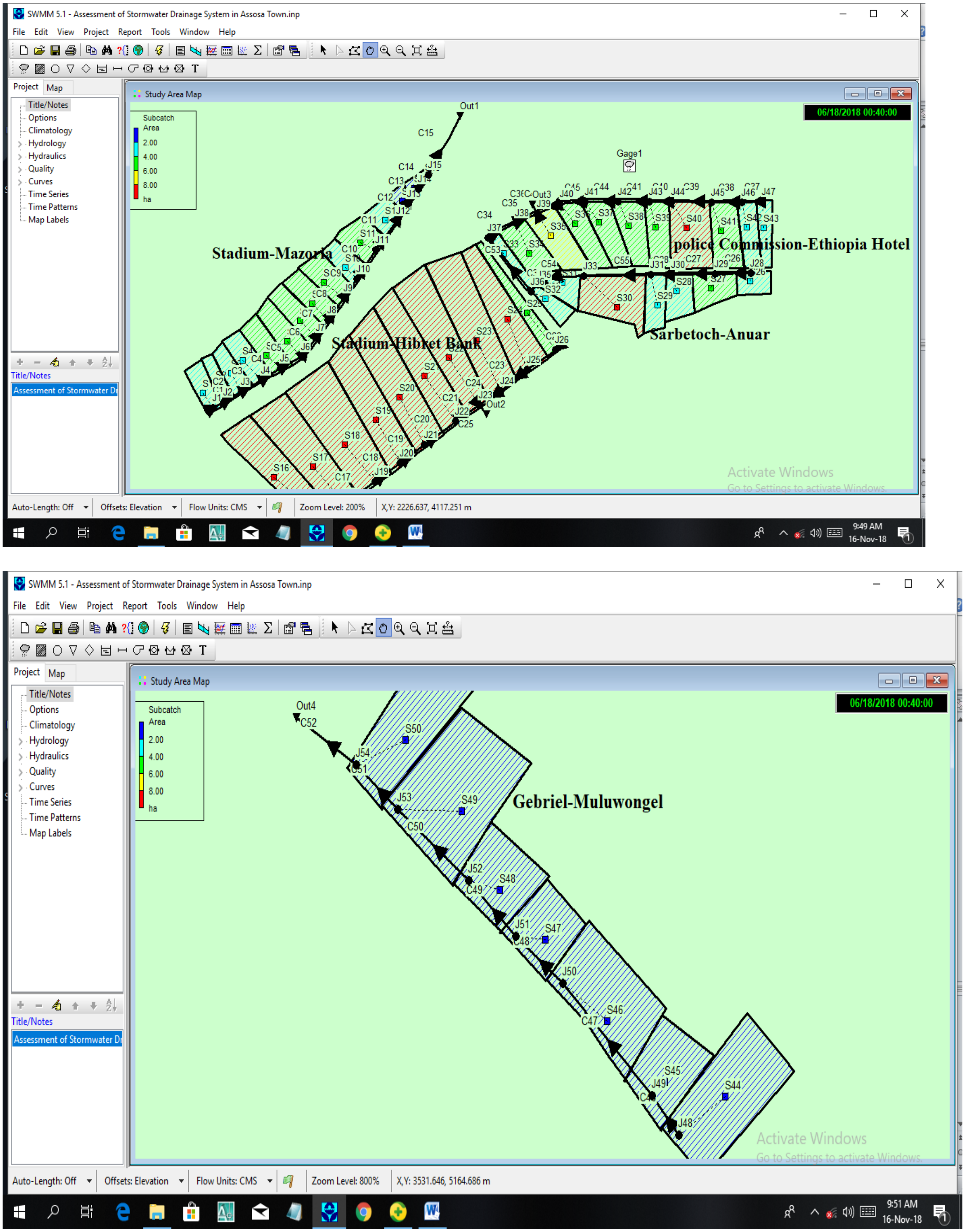

Figure 4.6: Network of the modeled areas

The manholes/junctions are all modeled as circular manholes; but from Sarbetoch to Anuar modeled with rectangular shape. It has been assumed that there are no energy losses in the manholes. Moreover, the model includes boundary conditions to represent various types of water loads, as infiltration or fixed water levels. The precipitation is introduced into the model by associating each sub catchment to the rainfall time-series. The general network performance is determined by infiltration rates and the average water flow production.

4.3. Drainage Discharge Estimation Result from Empirical Formula

Weighted run off coefficient for all five catchments are calculated and tabulated below. Run off coefficient values for different land use land covers are referenced from Assosa town Master plan. As it can be seen from the land use land cover composition table below, the weighted runoff coefficient for sub catchment Stadium-Mazoria, Stadium-Hibret Bank, Sarbetoch-Anuar, Police Commission-Ethiopia Hotel and Gebriel-Muluwongel Church are 0.78, 0.75, 0.75, 0.767 and 0.75 respectively. This implies that 78.8%, 75%, 75%, 76.7% & 75% of the rainfall rate shall be measured as peak runoff rate when spatially and temporally continuous rainfalls for at least duration equal to the time of concentration from each corresponding catchment. These values are within the expected value for weighted runoff coefficient of urban areas (Ven Te Chow *et al*, 2012).

Table 4.7: Runoff coefficient for different catchments

		Surface condition				
S. No.	Catchment	Description	Area proportion (%)	Runoff coefficient,C	C*A	$C_{wighted}$
1	From Stadium to Mazoria	Residential	75.01	0.85	0.638	0.788
		Commercial	5.70	0.725	0.041	
		Recreational	1.61	0.3	0.005	
		Services	3.27	0.2	0.007	
		Administration	14.42	0.675	0.097	
2	From Stadium to Hibret Bank	Residential	29.06	0.9	0.262	0.75
		Commercial	11.44	0.95	0.109	
		Recreational	26.29	0.5	0.131	
		Services	24.39	0.25	0.061	
		Administration	7.73	0.95	0.073	
3	From Sarbetoch to Anuar	Residential	100	0.75	0.750	0.75
4	From Police Commission to Ethiopia Hotel	Residential	61.76	0.8	0.494	0.767
		Commercial	7.58	0.9	0.068	
		Services	9.38	0.25	0.023	
		Administration	21.28	0.85	0.181	
5	From Gebriel to Mulu-wongel	Residential	5.65	0.75	0.042	0.750
		Commercial	94.35	0.75	0.708	

Time of concentration for the sub catchments is calculated using NRCS Velocity Method and presented in the table below. But as ERA (2013) recommended use a minimum tc value of 7 minutes for asphaltic and developed urban areas and a minimum tc value of 15 minutes for areas that are not developed and intercepting catchments. Therefore, the bigger duration shall be taken as a value for time of concentration of the study area for the reason that within this period all the catchments contribute flow to the outlet of point of interest of this study. Therefore, 15 minutes shall be considered as time of concentration of the study area. Based on this, the design intensity shall be read from IDF curve for corresponding recurrence intervals. Length of flow as well as average slope for calculation of time of concentration was obtained by field survey.

Table 4.8: Runoff Stadium-mazoria

Sub catchment label	Area(ha)	Total tc (min)	Minimum tc (min)/ERA	IDF for T_10yrs	Weighted runoff coef. C	Discharge(m^3/s)
S1	2.760	11.82	15	105.0	0.700	0.56
S2	2.110	11.82	15	105.0	0.600	0.37
S3	2.900	12.10	15	105.0	0.600	0.51
S4	3.695	13.78	15	105.0	0.600	0.65
S5	4.090	14.76	15	105.0	0.850	1.01
S6	4.520	16.36	15	105.0	0.850	1.12
S7	4.240	14.83	15	105.0	0.850	1.05
S8	4.120	13.03	15	105.0	0.850	1.02
S9	5.250	11.59	15	105.0	0.750	1.15
S10	3.030	10.66	15	105.0	0.750	0.66
S11	4.149	8.98	15	105.0	0.750	0.91
S12	2.700	6.22	15	105.0	0.600	0.47
S13	1.200	3.91	15	105.0	0.600	0.21
S14	0.780	3.04	15	105.0	0.600	0.14
S15	0.380	2.01	15	105.0	0.600	0.07

Table 4.9: Runoff Stadium-Hibret Bank

Sub catchment label	Area(ha)	Total tc,min	Minimum tc,min/ERA	IDF for T_10yrs	Weighted runoff Coefficient, C	Discharge(m^3/s)
S16	11.68	8.84	15	105.0	0.8	2.727
S17	13.86	9.42	15	105.0	0.8	3.235
S18	17.88	8.19	15	105.0	0.75	3.913
S19	12.60	12.18	15	105.0	0.75	2.757
S20	14.73	12.45	15	105.0	0.75	3.223
S21	12.48	13.93	15	105.0	0.75	2.731
S22	15.80	14.17	15	105.0	0.75	3.458
S23	16.14	16.28	15	105.0	0.75	3.533
S24	11.37	14.80	15	105.0	0.75	2.488
S25	4.60	11.93	15	105.0	0.7	0.940

Table 4.10: Runoff Sarbetoch –Anuar

Sub catchment label	Area(ha)	Total tc,min	Minimum tc,min/ERA	IDF for T_10yrs	Weighted runoff coef. C	Discharge(m^3/s)
S26	2.38	5.57	15	105.0	0.75	0.521
S27	4.22	7.57	15	105.0	0.76	0.936
S28	3.73	10.11	15	105.0	0.75	0.816
S29	3.59	13.08	15	105.0	0.74	0.775
S30	9.15	10.53	15	105.0	0.75	2.003
S31	2.73	9.98	15	105.0	0.75	0.598
S32	2.17	10.29	15	105.0	0.76	0.481

Table 4.11: Runoff police commission- Ethiopia Hotel

Sub catchment label	Area(ha)	Total tc,min	Minimum tc,min/ERA	IDF for T_10yrs	Weighted runoff coef. C	Discharge(m^3/s)
S33	2.32	8.97	15	105.0	0.7	0.475
S34	4.66	16.79	15	105.0	0.68	0.924
S35	5.29	12.12	15	105.0	0.82	1.267
S36	4.40	13.15	15	105.0	0.8	1.028
S37	5.02	11.93	15	105.0	0.78	1.143
S38	4.89	13.91	15	105.0	0.79	1.127
S39	4.63	12.80	15	105.0	0.8	1.081
S40	8.34	14.54	15	105.0	0.75	1.825
S41	5.63	14.61	15	105.0	0.8	1.314
S42	3.50	14.89	15	105.0	0.78	0.796
S43	2.62	14.95	15	105.0	0.8	0.612

Table 4.12: Runoff Gebriel - Muluwongel church

Sub catchment label	Area(ha)	Total tc,min	Minimum tc,min/ERA	IDF for T_10yrs	Weighted runoff coef. C	Discharge(m^3/s)
S44	1.45	6.68	15	105.0	0.75	0.317
S45	0.77	5.02	15	105.0	0.666	0.150
S46	1.30	5.19	15	105.0	0.75	0.285
S47	0.65	4.32	15	105.0	0.666	0.126
S48	0.79	5.12	15	105.0	0.666	0.154
S49	1.70	6.08	15	105.0	0.75	0.371
S50	1.39	6.69	15	105.0	0.666	0.270

4.4. Drainage Network Modelling Result

The model covers a total area of 264 ha of sub catchments which consist of impervious surfaces as roofs or roads and total of 16 km length of network simulation.

4.5. Simulation Results

Figure 4.7 shows the hydrographs for each outlet for the design storm (10-years rainfall), where each hydrograph represent the total inflow at each of the outlets. Six hour duration of rainfall is considered for simulation.

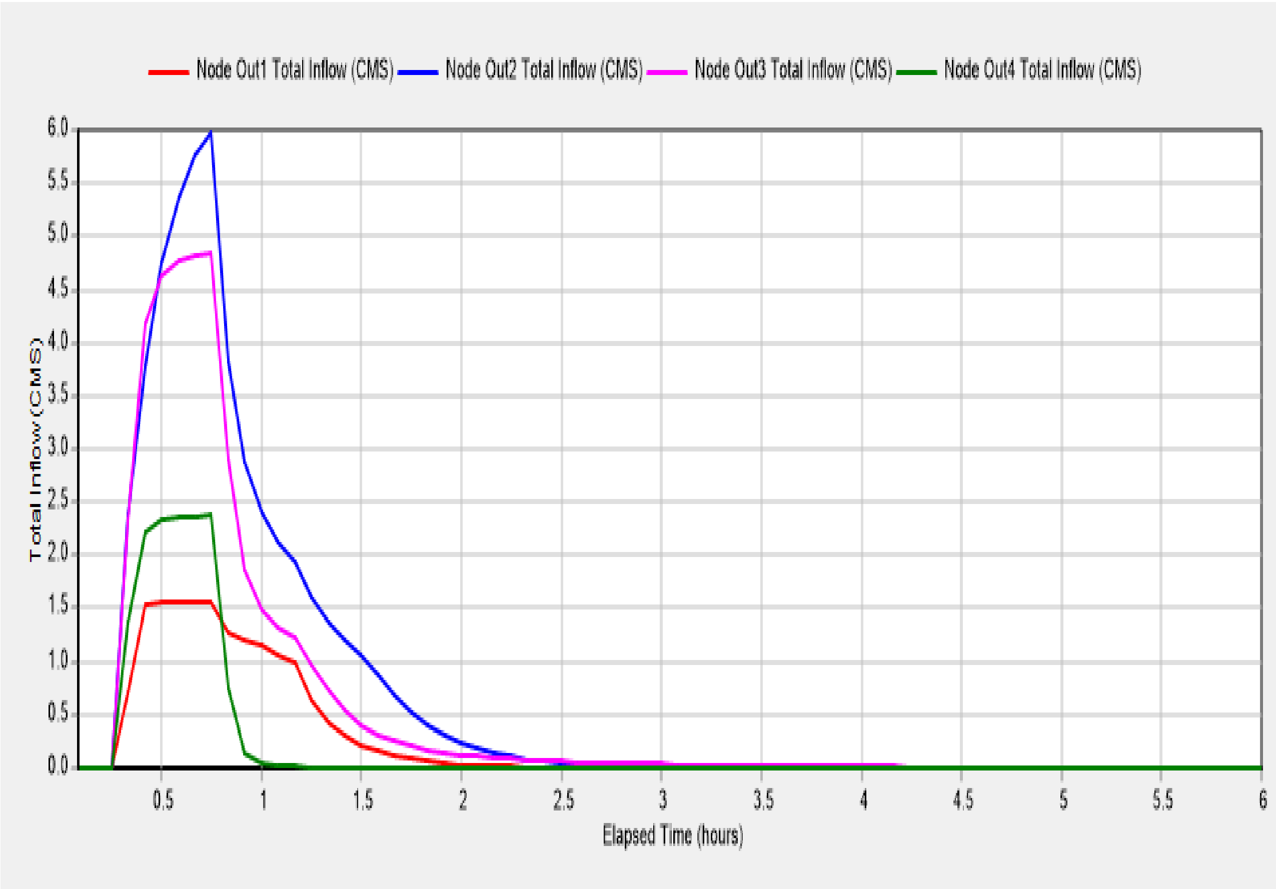

Figure 4.7: Runoff hydrographs for drainage system outlets

Figure 4.8 illustrates the hydraulic response of nodes from police commission to Ethiopia hotel at 00:40. In case of design storm of current rainfall data, the designed capacity was not able to meet the runoff inflow without significant risk for surcharge or flooding. *Figure 4.9* illustrates a map of study area where there was flooding or surcharges atop the nodes in the entire drainage system. This flooding occurred as the flow exceeded the designed capacity and the manholes surcharge was likely to occur. The simulations results show a considerable flooding a tope most of the nodes in each of the selected area (see Appendix 7). The water profile plot is obtained for condiuts from node J47 to J39 is as shown in Fig. 4.8. the simulation status report shows that sections between these nodes are surchared (flooded). The depth of surcharged at node J42 and at node J43 is 0.2m above crest level, whereas J44 is 0.4m.

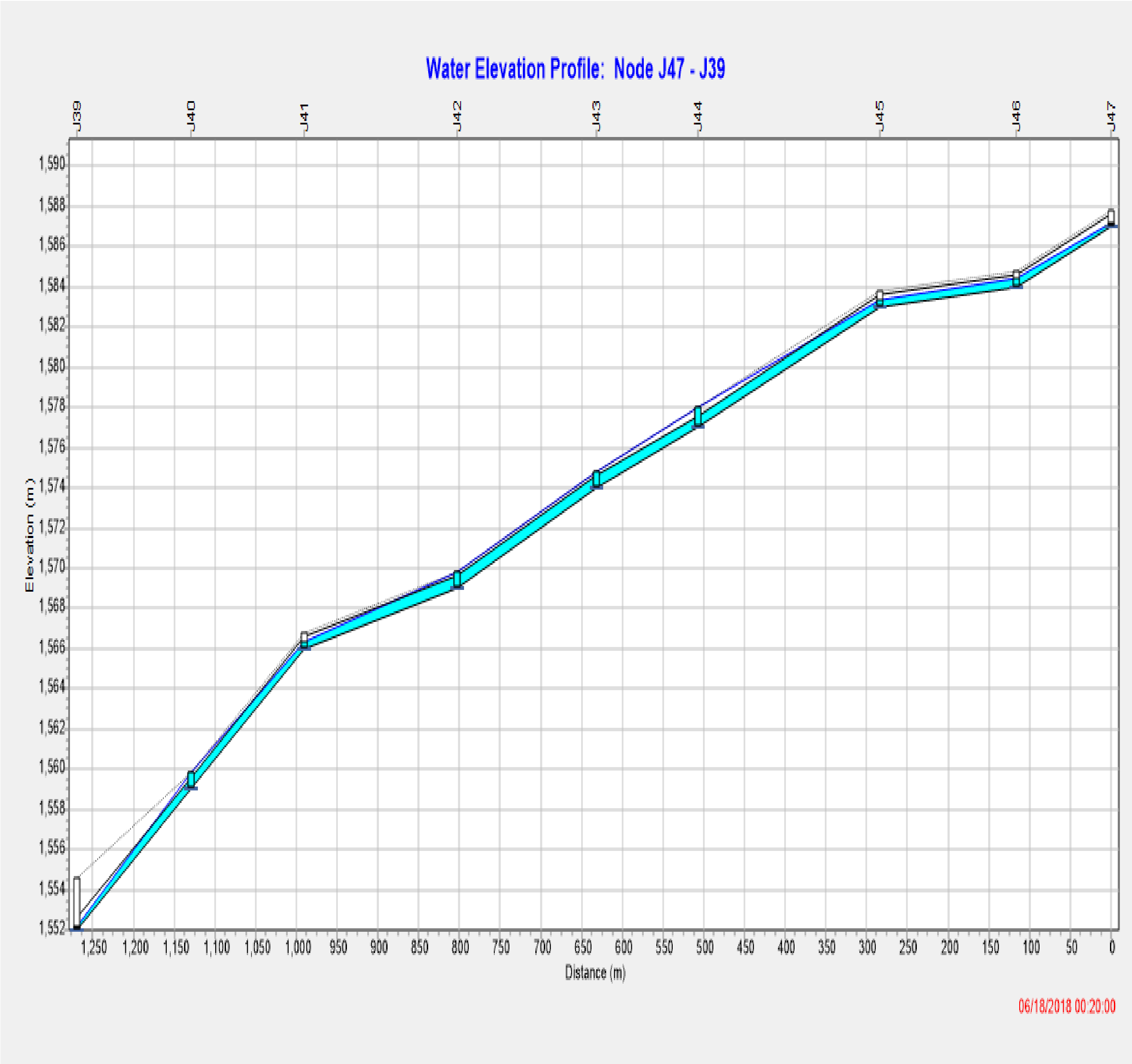

Figure 4.8: profile plot from police commission to Ethiopia Hotel to the design storm

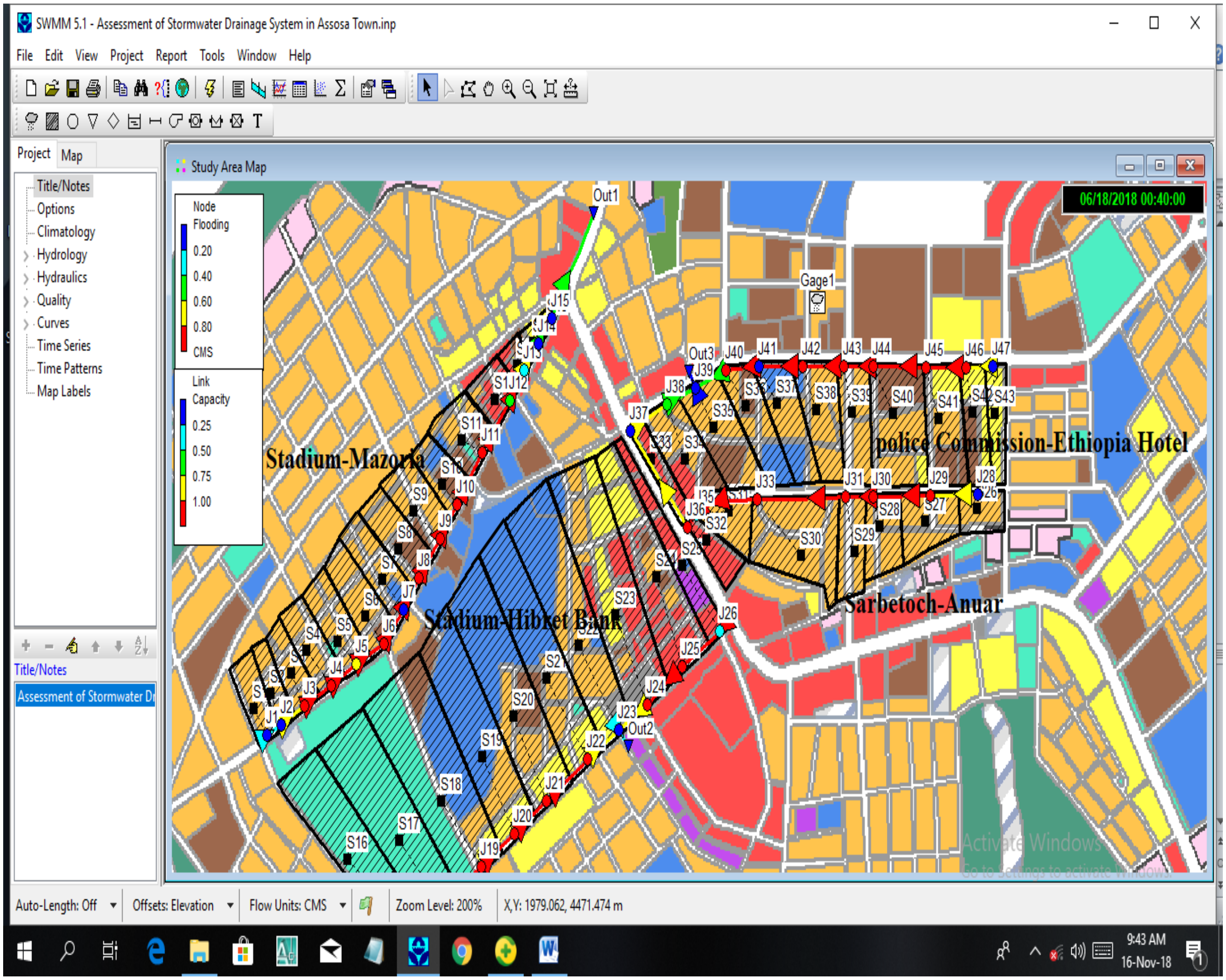

Figure 4.9: Study map with flooding atop most nodes the design storm

Figure 4.10 illustrates the hydraulic response of nodes from Gebriel to Muluwongel church at 00:40 minutes. In case of design storm of current rainfall data, the designed capacity was able to meet the runoff inflow without significant risk for surcharge or flooding. But there was flooding problem due to clogging of the drainage line by different waste material.

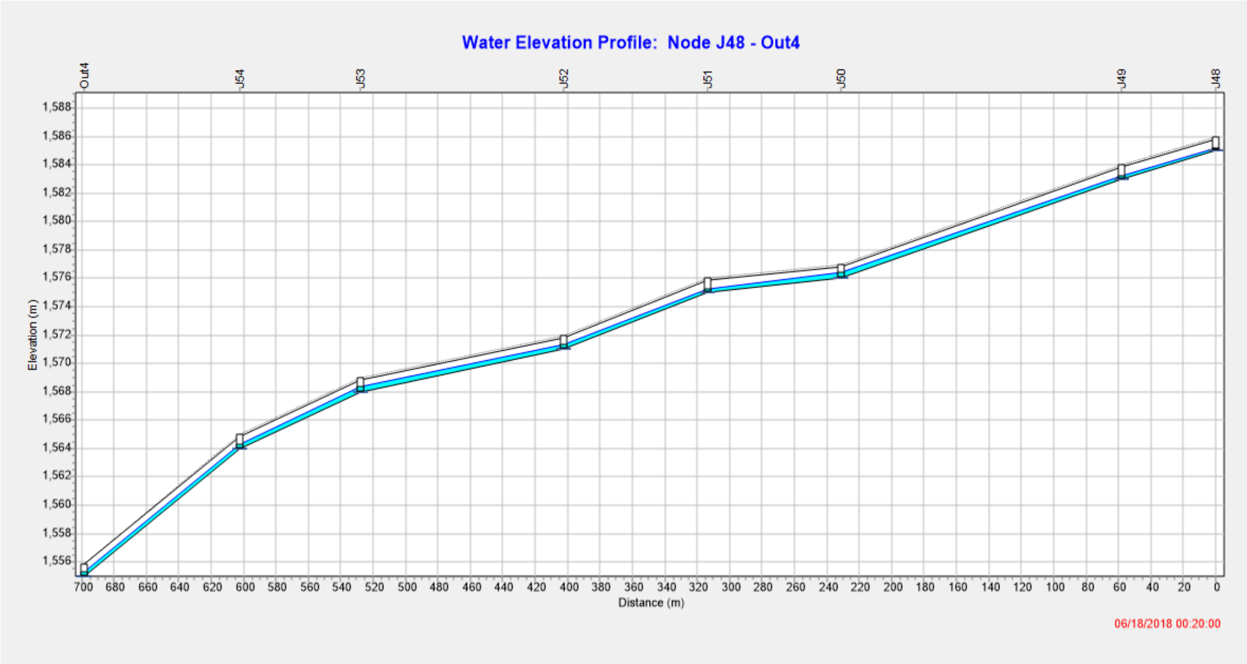

Figure 4.10: Profile plot from Gebriel to Mulu-wongel church response to the design storm

Status report can also give the information of flooding nodes, and the graph for node flooding is shown in Figure 4.12. The rate of inflow (from status report) at node, J19 is 4.43 m^3/sec, at J20 is 3.90 m^3/sec, and at node J22 is 2.82 m^3/sec.

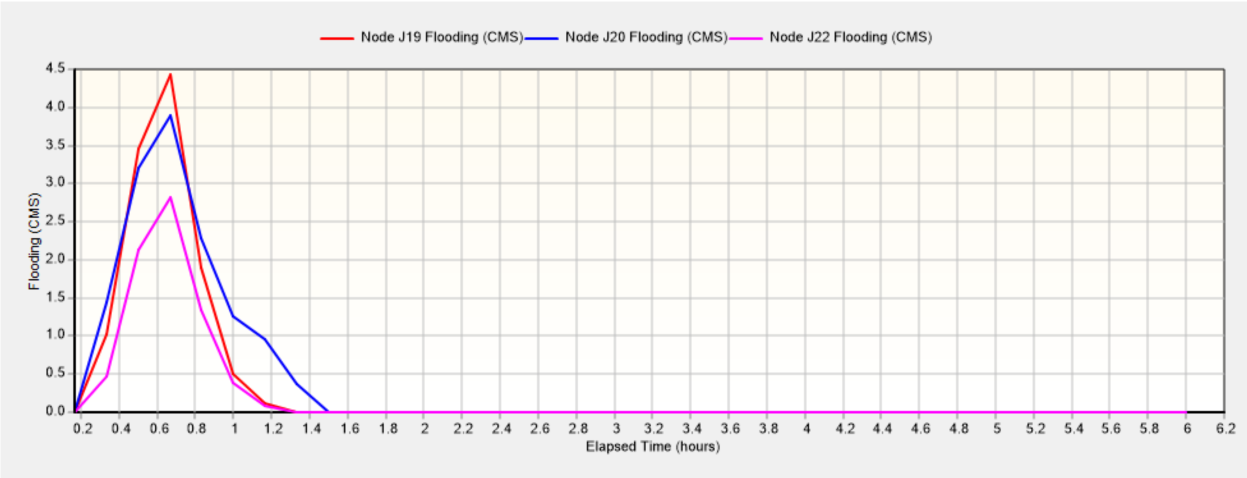

Figure 4.11: Node flooding graph

4.6. Comparison of Rational Formula and SWMM result

During comparison, the simulated peak flow show good relationships with the rational formula values in the catchments. The goodness of fit was also assessed by plotting the simulated versus calculated values of peak flow as shown in Figure 4.13.

Table 4.13:The entire catchment peak runoff result from both models

Sub catchments label	Rational formula, peak output ,m^3/sec	SWMM peak output, m^3/sec	Difference
S1	0.56	0.78	-0.22
S2	0.37	0.6	-0.23
S3	0.51	0.82	-0.31
S4	0.65	1.04	-0.39
S5	1.01	1.14	-0.13
S6	1.12	1.25	-0.13
S7	1.05	1.18	-0.13
S8	1.02	1.15	-0.13
S9	1.15	1.2	-0.05
S10	0.66	0.86	-0.20
S11	0.91	1.18	-0.27
S12	0.47	0.77	-0.30
S13	0.21	0.34	-0.13
S14	0.14	0.22	-0.08
S15	0.07	0.11	-0.04
S16	2.73	3.03	-0.30
S17	3.24	3.61	-0.37
S18	3.91	4.87	-0.96
S19	2.76	3.39	-0.63
S20	3.22	3.95	-0.73
S21	2.73	3.34	-0.61
S22	3.46	4.19	-0.73
S23	3.53	4.48	-0.95
S24	2.49	3.18	-0.69
S25	0.94	1.29	-0.35
S26	0.52	0.68	-0.16
S27	0.94	1.21	-0.27
S28	0.82	1.07	-0.25
S29	0.78	1.02	-0.24
S30	2.00	2.62	-0.62
S31	0.60	0.78	-0.18
S32	0.48	0.62	-0.14
S33	0.47	0.68	-0.21
S34	0.92	1.61	-0.69
S35	1.27	1.78	-0.51
S36	1.03	1.41	-0.38
S37	1.14	0.87	0.27
S38	1.13	1.52	-0.39
S39	1.08	1.35	-0.27
S40	1.83	2.44	-0.61
S41	1.31	1.66	-0.35
S42	0.80	1.03	-0.23
S43	0.61	0.74	-0.13
S44	0.32	0.42	-0.10
S45	0.15	0.22	-0.07
S46	0.28	0.37	-0.09
S47	0.13	0.19	-0.06
S48	0.15	0.23	-0.08
S49	0.37	0.56	-0.19
S50	0.27	0.4	-0.13
Total Runoff,m^3/s	58.30	73.45	-15.15

The simulated and calculated values for runoff were correlated, and the R^2 values of 0.984 were considered acceptable. The comparison results indicated that the model structure and parameters matched the runoff producing pattern and that the compared model was suitable for simulating storm runoff in the study area.

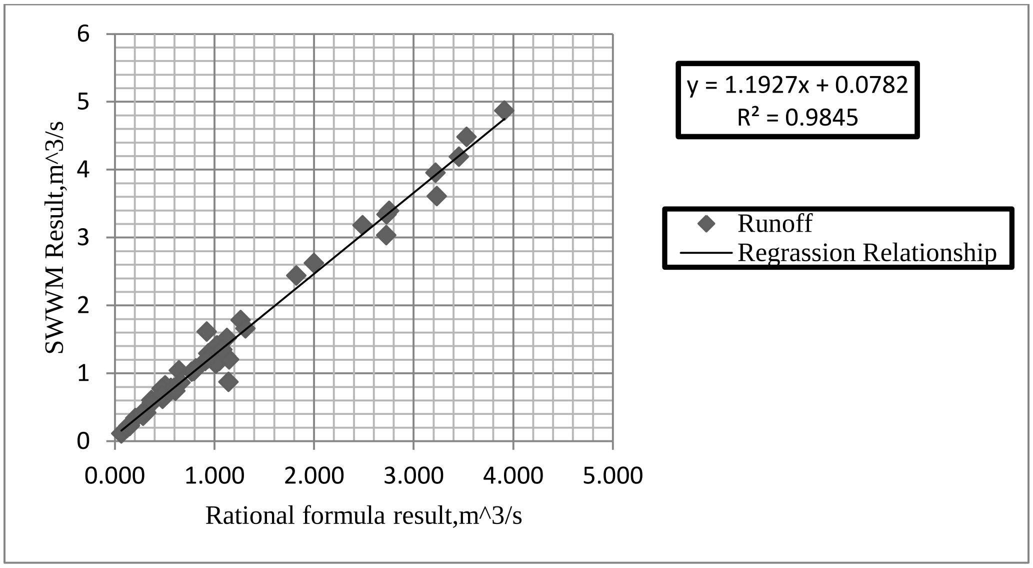

Figure 4.12: Correlated plot of calculated by rational formula and simulated flow from entire catchment

During model comparison, agreements between calculated and simulated values can be evaluated using graphical method. It is done by making comparisons between calculated and simulated peak discharge and rising and falling limb.

The maximum values obtained from model study and rational methods are given in table 4.14 above. These values are plotted in graphical format for comparison purpose and are shown in figure 4.13 shown below.

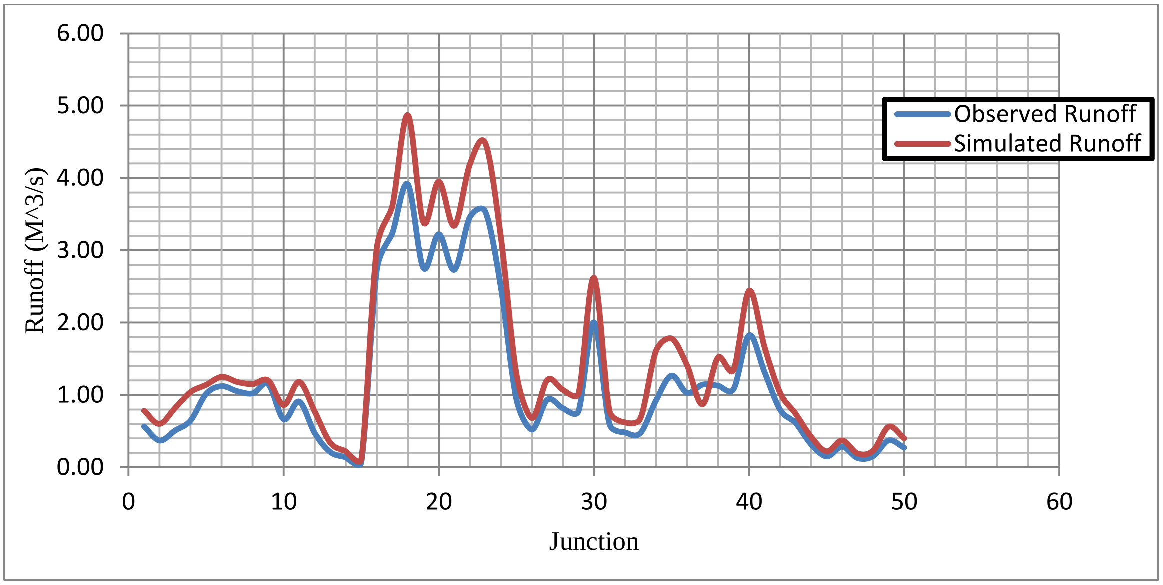

Figure 4.13: Graphical plot for runoff obtained using SWMM and Rational method from entire catchment

The US EPA SWMM 5 model was applied to an urban catchment in Assosa town and rational method is used for comparison. Calculations are presented, the total runoff from whole sub catchments by SWMM is 73.45m3/sec, whereas by rational method is 58.303m3/sec. The runoff obtained in the simulation is then used as input data at each node connected to a catchment. The same study was done in India and the total runoff from seven sub catchments by SWMM is 2.177m3/sec, whereas by Rational Method is 1.109m3/sec. (M. L. Waikar* and Undegaonkar Namita U, January, 2015) Urban Flood Modeling by using EPA SWMM 5 SRTM University‟s Research Journal of Science April 2015 /Spl. Vol., no 1; (ISSN : 2277 - 8594 Print).

4.7. Adequacy of Existing Drainage System

Urban stormwater drainages are designed based on different criteria so that they can give better services regarding to safely removing (neither siltation nor scouring) the urban runoff in to the water ways. Flooding over asphalts, walkways and near the residences has been such a big problem in the town. Therefore, an effort has been done here to evaluate the capacity and performance of these drainage systems.

Since there is no recorded data about the dimensions of these drainage systems a field survey was made to measure their dimensions so that the amount of discharge conveyed in the existing drainage system and its velocity could be determined. Table 4.14 shows a sample calculation of the discharge that is conveyed through the existing drainage system and its velocity using the manning's formula. The rest of the calculation is given in the Appendix 1.

Table 4.14: Computation of the existing drainage system

No.	Flow Direction		Drainage Types			Length	Slope	n	Area	Perimeter	Runoff	Velocity
			Rectangular		Circular							
	From	To	B(m)	Y(m)	D(m)	L(m)			(m^2)	(m)	(m^3/s)	(m/s)
1	J1	J2			0.8	30	0.01	0.02	0.502	2.512	0.86	1.71
2	J2	J3			0.8	60	0.01	0.02	0.502	2.512	0.86	1.71
3	J3	J4			0.8	60	0.02	0.02	0.502	2.512	1.21	2.42
4	J4	J5			0.8	60	0.01	0.02	0.502	2.512	0.86	1.71
5	J5	J6			0.8	60	0.01	0.02	0.502	2.512	0.86	1.71
6	J6	J7			0.8	60	0.02	0.02	0.502	2.512	1.05	2.09
7	J7	J8			0.8	60	0.01	0.02	0.502	2.512	0.86	1.71
8	J8	J9			0.8	60	0.01	0.02	0.502	2.512	0.86	1.71
9	J9	J10			0.8	60	0.01	0.02	0.502	2.512	0.86	1.71
10	J10	J11			0.8	60	0.01	0.02	0.502	2.512	0.86	1.71
11	J11	J12			0.8	60	0.01	0.02	0.502	2.512	0.86	1.71
12	J12	J13			0.8	60	0.02	0.02	0.502	2.512	1.21	2.42
13	J13	J14			0.8	60	0.02	0.02	0.502	2.512	1.11	2.21
14	J14	J15			0.8	60	0.03	0.02	0.502	2.512	1.49	2.96
15	J15	Out 1			0.8	60	0.02	0.02	0.502	2.512	1.05	2.09

The values of velocity obtained from Manning's formula and SWMM are given in table 4.15 below. These values are plotted in graphical format for comparison purpose and are shown in figure 4.14 shown.

Table 4.15: The velocity result from manning's formula and SWMM

Link or Conduit	Calculated velocity from Measured (m/s)	Simulated Velocity (m/s)	Difference
C1	1.71	1.87	-0.16
C2	1.71	1.7	0.01
C3	2.42	2.19	0.23
C4	1.71	1.66	0.05
C5	1.71	1.9	-0.19
C6	2.09	2.16	-0.07
C7	1.71	1.76	-0.05
C8	1.71	1.68	0.03
C9	1.71	1.8	-0.09
C10	1.71	1.49	0.22
C11	1.71	1.65	0.06
C12	2.42	2.38	0.04
C13	2.21	2.14	0.07
C14	2.96	2.92	0.04
C15	2.09	1.89	0.20
C16	1.73	1.76	-0.03
C17	1.41	1.45	-0.04
C18	2.00	1.89	0.11
C19	1.41	1.3	0.11
C20	1.41	1.51	-0.10
C21	2.00	1.85	0.15
C22	1.73	1.76	-0.03
C23	1.73	1.58	0.15
C24	2.44	2.49	-0.05
C25	3.46	3.79	-0.33
C26	1.97	2.19	-0.22

C27	1.39	1.26	0.13
C28	1.97	1.87	0.10
C29	1.97	2.1	-0.13
C30	1.94	1.82	0.12
C31	3.12	3.05	0.07
C32	2.59	2.94	-0.35
C33	1.85	1.91	-0.06
C34	1.85	1.6	0.25
C35	1.68	1.6	0.08
C36	2.00	1.87	0.13
C37	1.41	1.4	0.01
C38	2.44	2.46	-0.02
C39	1.41	1.59	-0.18
C40	1.58	1.76	-0.18
C41	1.58	1.9	-0.32
C42	1.41	1.6	-0.19
C43	2.23	2	0.23
C44	2.52	1.9	0.62
C45	1.78	1.81	-0.03
C46	1.78	2.44	-0.66
C47	2.30	2.95	-0.65
C48	1.78	1.3	0.48
C49	1.78	1.34	0.44
C50	1.78	1.61	0.17

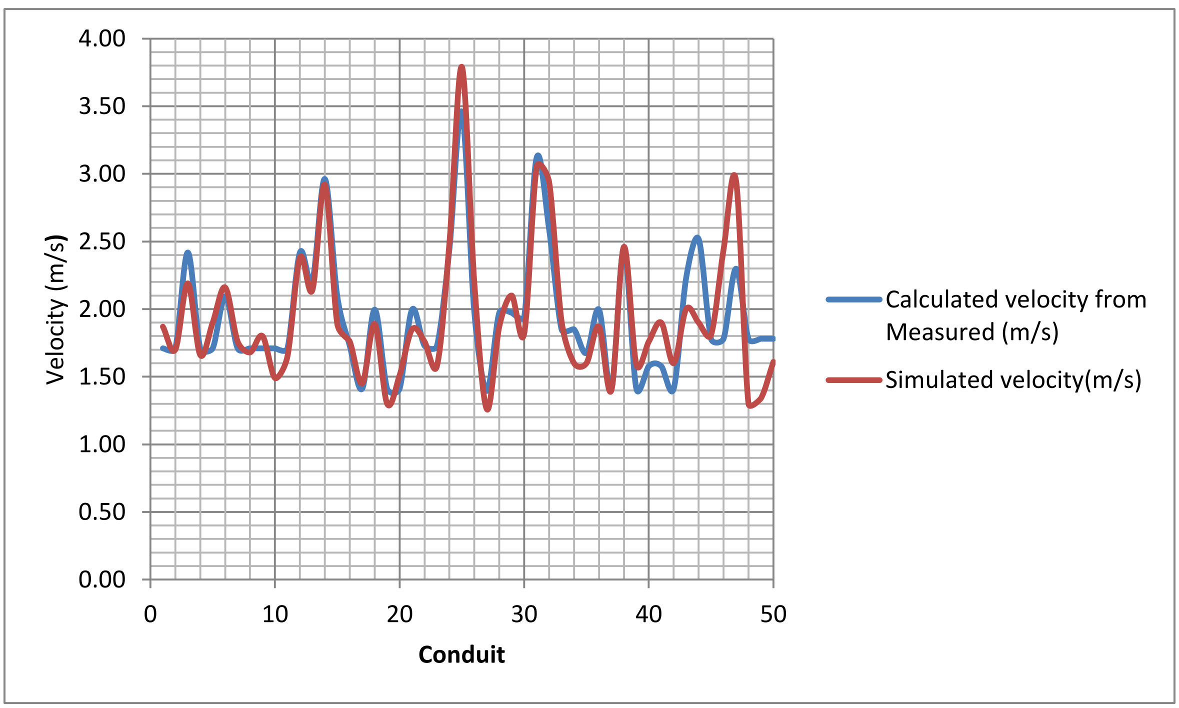

Figure 4.14: Graphical plot for velocity obtained using manning's formula and SWMM from entire conduit

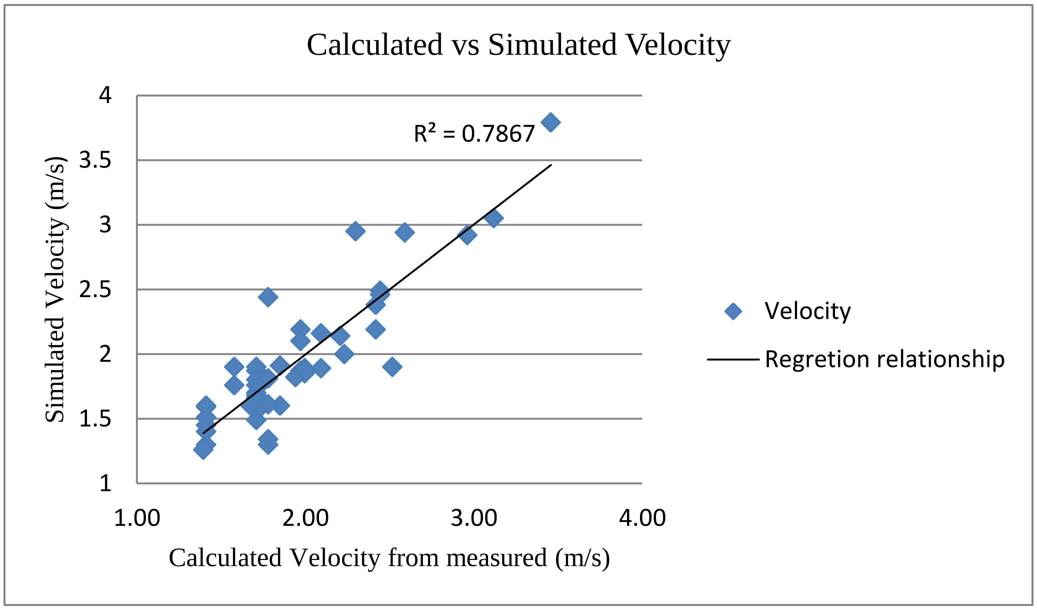

Figure 4.15: Correlated plot of calculated and simulated velocity from entire conduit

4.8. Peak Runoff Reduction

The percentages of peak runoff reduction by LID scenarios in SWWM were calculated and analyze for evaluating their peak runoff reduction effectiveness (Table 4.17). The runoff reduction percentages of vegetative swale ranged from approximately 7.19% to 64.29% in all sub catchments and the reduction percentages are much higher (61.45–64.29%) in the sub catchments S30, S31 and S49 depends up on land use land cover. The lower runoff reduction percentages are 7.19% and 7.55% in the sub catchments S21 and S24 respectively because they are highly commercial center. Overall, the LID measures can reduce the peak runoff to a certain magnitude.

Table 4.16: Peak runoff before and after introduce LID and percentage of peak runoff reduction under vegetative swale scenario

Sub catchments	SWMM Output (CMS) Before Introduce LID	SWMM Output (CMS) After Introduce LID	Percentage of peak runoff reduction
S1	0.78	0.60	23.08
S2	0.6	0.55	8.33
S3	0.82	0.60	26.83
S4	1.04	0.90	13.46
S5	1.14	0.92	19.30
S6	1.25	1.01	19.20
S7	1.18	0.93	21.19
S8	1.15	1	13.04
S9	1.2	0.87	27.50
S10	0.86	0.7	18.60
S11	1.18	1.02	13.56
S12	0.77	0.5	35.06
S13	0.34	0.2	41.18
S14	0.22	0.12	45.45
S15	0.11	0.07	36.36
S16	3.03	2.01	33.66
S17	3.61	3.2	11.36
S18	4.87	3.4	30.18
S19	3.39	2.48	26.84
S20	3.95	2.8	29.11
S21	3.34	3.1	7.19
S22	4.19	3.39	19.09
S23	4.48	3.54	20.98
S24	3.18	2.94	7.55
S25	1.29	1.1	14.73
S26	0.68	0.3	55.88
S27	1.21	0.98	19.01
S28	1.07	0.82	23.36
S29	1.02	0.8	21.57
S30	2.62	1.01	61.45
S31	0.78	0.3	61.54
S32	0.62	0.4	35.48
S33	0.68	0.5	26.47
S34	1.61	1	37.89
S35	1.78	1.1	38.20
S36	1.41	1.2	14.89
S37	0.87	0.6	31.03
S38	1.52	1.2	21.05
S39	1.35	0.99	26.67
S40	2.44	1.5	38.52
S41	1.66	1.1	33.73
S42	1.03	0.9	12.62
S43	0.74	0.5	32.43
S44	0.42	0.3	28.57
S45	0.22	0.11	50.00
S46	0.37	0.2	45.95
S47	0.19	0.1	47.37
S48	0.23	0.13	43.48
S49	0.56	0.2	64.29
S50	0.4	0.2	50.00
Total Q (MCS)	73.45	54.39	26

6. CONCLUSION AND RECOMMENDATION

6.1. Conclusion

In the study area it is observed that drainage problem is a cause of flooding on pavement, congested traffic flow and difficulty on day to day activity of people. To investigate the cause of the problem, we try to assess the drainage system in the flood prone areas and site investigation was done by collecting direct field data to assess the storm drainage condition and operation management problem.

The hydrologic performance of the drainage systems was simulated using SWMM and Rational formula. The total runoff from whole catchments by SWMM is 73.45 m^3/sec, whereas by rational method is 58.30 m^3/sec. Results of simulation show that SWMM gives relatively excess runoff values as compared to Rational Method. However, the SWMM Parameters may need more calibration for more reliable results. The same study was done in India and the total runoff from seven sub catchments by SWMM is 2.177 m^3/sec, whereas by Rational Method is 1.109 m^3/sec. (M. L. Waikar* and Undegaonkar Namita U, January, 2015).

Under the current rainfall conditions the system responded with serious problems and was not able to drain the generated runoff. So, the systems were vulnerable for flooding and surcharges atop the nodes were considerable in the most drainage systems.

So this drainage problem in the study area is as a result of:

- Inadequate structure provision, which is the hydraulic capacity of the drainage structures, is less than the design discharge.
- Siltation and blockage with sand and waste material respectively of the drainage structures.

6.2. Recommendation

Drainage problem become major challenge for recently constructed road as it is observed in Assosa town. As a number of road projects are constructed with huge investments, for drainage related issues emphasis shall be given. The following recommendations have been drawn from this study.

- Design of the structures
  - ✓ All consideration, such as appropriate design method which depends on the catchment area, variability of climate, future settlement of people, expansion of urbanization and other factors shall be taken into account during the detail design of the drainage facilities so as the structures capacity shall accommodate the design flood.
  - ✓ In case if the problem occurs and the town administration shall to take action to keep the serviceability of the road, the rehabilitation needs to be supplemented by the detail design to alleviate the problem permanently with low cost.
  - ✓ There is a need to introduce LID to reduce the runoff production and to minimize the receiving water pollution.
  - ✓ Since the velocity of runoff is high in existing drainage channel, it is need to provide drop structure to decrease flow velocity.
- Continuous Monitoring of the drainage facilities
  - ✓ Continuous monitoring of the drainage facilities is required to take timely action where unexpected problem encounter that may create risk on the people, road and surrounding environment.
  - ✓ Periodically, cleaning of the drainage facilities is also required to prevent of clogging of the drainage system.

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8. APPENDICES

Appendix 1: Computation of the existing drainage system

From Stadium to Mazoria												
No.	Flow Direction		Drainage Types			Length	Slope	n	Area (m^2)	Perimeter (m)	Runoff (m^3/s)	Velocity(m/s)
			Rectangular		Circular							
	From	To	B(m)	Y(m)	D(m)	L(m)						
1	J1	J2			0.8	30	0.01	0.02	0.502	2.512	0.86	1.71
2	J2	J3			0.8	60	0.01	0.02	0.502	2.512	0.86	1.71
3	J3	J4			0.8	60	0.02	0.02	0.502	2.512	1.21	2.42
4	J4	J5			0.8	60	0.01	0.02	0.502	2.512	0.86	1.71
5	J5	J6			0.8	60	0.01	0.02	0.502	2.512	0.86	1.71
6	J6	J7			0.8	60	0.02	0.02	0.502	2.512	1.05	2.09
7	J7	J8			0.8	60	0.01	0.02	0.502	2.512	0.86	1.71
8	J8	J9			0.8	60	0.01	0.02	0.502	2.512	0.86	1.71
9	J9	J10			0.8	60	0.01	0.02	0.502	2.512	0.86	1.71
10	J10	J11			0.8	60	0.01	0.02	0.502	2.512	0.86	1.71
11	J11	J12			0.8	60	0.01	0.02	0.502	2.512	0.86	1.71
12	J12	J13			0.8	60	0.02	0.02	0.502	2.512	1.21	2.42
13	J13	J14			0.8	60	0.02	0.02	0.502	2.512	1.11	2.21
14	J14	J15			0.8	60	0.03	0.02	0.502	2.512	1.49	2.96
15	J15	Out 1			0.8	60	0.02	0.02	0.502	2.512	1.05	2.09
From Stadium to Hibret Bank												
16	J17	J18			0.6	40	0.02	0.02	0.283	1.884	0.49	1.73
17	J18	J19			0.6	40	0.01	0.02	0.283	1.884	0.40	1.41
18	J19	J20			0.6	40	0.02	0.02	0.283	1.884	0.56	2.00
19	J20	J21			0.6	69	0.01	0.02	0.283	1.884	0.40	1.41
20	J21	J22			0.6	40	0.01	0.02	0.283	1.884	0.40	1.41
21	J22	J23			0.6	40	0.02	0.02	0.283	1.884	0.56	2.00
22	J23	Out 2			0.6	40	0.02	0.02	0.283	1.884	0.49	1.73
23	J24	J23			0.6	40	0.02	0.02	0.283	1.884	0.49	1.73
24	J25	J24			0.6	40	0.03	0.02	0.283	1.884	0.69	2.44
25	J26	J25			0.6	20	0.06	0.02	0.283	1.884	0.98	3.46
From Sarbetoch to Anuar												
26	J28	J29	0.4	0.56		310	0.02	0.02	0.224	1.52	0.44	1.97
27	J29	J30	0.4	0.56		66	0.01	0.02	0.224	1.52	0.31	1.39
28	J30	J31	0.4	0.56		45	0.02	0.02	0.224	1.52	0.44	1.97
29	J31	J33	0.4	0.56		185	0.02	0.02	0.224	1.52	0.44	1.97
30	J33	J35	0.4	0.56		129	0.02	0.02	0.224	1.52	0.44	1.94
31	J35	J36	0.4	0.56		50	0.05	0.02	0.224	1.52	0.70	3.12
32	J36	J37	0.4	0.56		29	0.03	0.02	0.224	1.52	0.58	2.59
From Ethiopia hotel to police commission												
33	J37	J38			0.9	40	0.01	0.02	0.636	2.826	1.18	1.85
34	J38	J39			0.9	40	0.01	0.02	0.636	2.826	1.18	1.85
35	J39	Out 3	3.5	2.9		40	0.001	0.02	10.150	9.3	17.01	1.68
36	J40	J39			0.6	40	0.02	0.02	0.283	1.884	0.56	2.00
37	J41	J40			0.6	35	0.01	0.02	0.283	1.884	0.40	1.41
38	J42	J41			0.6	40	0.03	0.02	0.283	1.884	0.69	2.44
39	J43	J42			0.6	40	0.01	0.02	0.283	1.884	0.40	1.41
40	J44	J43			0.6	40	0.01	0.02	0.283	1.884	0.45	1.58
41	J45	J44			0.6	40	0.01	0.02	0.283	1.884	0.45	1.58
42	J46	J45			0.6	40	0.01	0.02	0.283	1.884	0.40	1.41
43	J47	J46			0.6	40	0.03	0.02	0.283	1.884	0.63	2.23

From Gebriel to Muluwongel church												
No.	Flow Direction		Drainage Types			Length	Slope	n	Area	Perimeter	Runoff	Velocity(m/s)
			Rectangular		Circular							
	From	To	B(m)	Y(m)	D(m)	L(m)			(m^2)	(m)	(m^3/s)	
44	J48	J49			0.85	40	0.02	0.02	0.567	2.669	1.43	2.52
45	J49	J50			0.85	40	0.01	0.02	0.567	2.669	1.01	1.78
46	J50	J51			0.85	60	0.01	0.02	0.567	2.669	1.01	1.78
47	J51	J52			0.85	60	0.02	0.02	0.567	2.669	1.30	2.30
48	J52	J53			0.85	60	0.01	0.02	0.567	2.669	1.01	1.78
49	J53	J54			0.85	60	0.01	0.02	0.567	2.669	1.01	1.78
50	J54	Out 4			0.85	60	0.01	0.02	0.567	2.669	1.01	1.78

Appendix 2: Test for outlier

Year	Annual daily Maximum rainfall (mm)	Y=logX	(Y-Ymean)^2	(Y-Ymean)^3
1986	66	1.8	0.002	0.000
1987	57.8	1.8	0.000	0.000
1988	56	1.7	0.001	0.000
1989	69.2	1.8	0.004	0.000
1994	56.8	1.8	0.001	0.000
1995	65.9	1.8	0.002	0.000
1996	41.2	1.6	0.026	-0.004
2000	93	2.0	0.036	0.007
2001	99	2.0	0.048	0.010
2002	54.3	1.7	0.002	0.000
2003	51.3	1.7	0.005	0.000
2004	63	1.8	0.000	0.000
2005	74	1.9	0.008	0.001
2006	81.6	1.9	0.018	0.002
2007	102.5	2.0	0.054	0.013
2008	35.4	1.5	0.052	-0.012
2009	60.8	1.8	0.000	0.000
2010	60.1	1.8	0.000	0.000
2011	97	2.0	0.044	0.009
2012	32.5	1.5	0.071	-0.019
2013	69	1.8	0.004	0.000
2014	42.1	1.6	0.024	-0.004
2015	54.2	1.7	0.002	0.000
2016	38.6	1.6	0.037	-0.007
2017	48.9	1.7	0.008	-0.001
Sum	1570.2	44.4	0.4	-0.004
Mean	62.8	1.8		
Sy	0.159			
Kn	2.486			
Skewness	-0.1			
Upper limit	130.9			
Lower limit	27.4			

Appendix 3: Computation of time of concentration

Sub Catchment label	Time of Concentration														
	n	Slope	Sheet Flow		Shallow Concentrated Flow			Channel Flow Time							Total Tt(min)
From Stadium to Mazoria			Length, L (m)	Tt(min)	L(m)	V(m/s)	Tt(min)	Elevetion	L(m)	Slope	n	Hydraulic radius, R (m)	V,m/s	Tt (min)	
S1	0.016	0.01	130	8.4	130	0.6	3.3	1586			0.1	0.2			
S2	0.016	0.01	130	8.3	135	0.7	3.4	1580	67.0	0.1	0.1	0.2	0.0	0.0	11.8
S3	0.016	0.01	127	8.2	140	0.7	3.6	1578	114.0	0.0	0.1	0.2	0.1	0.2	11.8
S4	0.016	0.01	128	8.5	190	0.6	5.0	1577	128.0	0.0	0.1	0.2	0.2	0.3	12.1
S5	0.016	0.01	130	8.9	200	0.6	5.5	1576	125.0	0.0	0.1	0.2	0.1	0.3	13.8
S6	0.016	0.01	128	9.0	250	0.6	7.1	1575	130.0	0.0	0.1	0.2	0.2	0.3	14.8
S7	0.016	0.01	129	9.2	188	0.6	5.4	1573	133.0	0.0	0.1	0.2	0.1	0.2	16.4
S8	0.016	0.01	130	8.9	143	0.6	3.9	1572	115.0	0.0	0.1	0.2	0.1	0.3	14.8
S9	0.016	0.01	130	8.4	116	0.7	3.0	1568	147.0	0.0	0.1	0.2	0.1	0.2	13.0
S10	0.016	0.01	127	7.7	102	0.7	2.4	1566	128.0	0.0	0.1	0.2	0.1	0.2	11.6
S11	0.016	0.02	108	6.4	89	0.8	2.0	1565	189.0	0.0	0.1	0.2	0.2	0.6	10.7
S12	0.016	0.02	96	5.2	40	0.9	0.8	1564	194.0	0.0	0.1	0.2	0.2	0.6	9.0
S13	0.016	0.03	71	3.4	28	1.1	0.4	1563	109.0	0.0	0.1	0.2	0.1	0.3	6.2
S14	0.016	0.05	65	2.6	21	1.4	0.3	1558	102.0	0.0	0.1	0.2	0.1	0.1	3.9
S15	0.016	0.06	46	1.8	16	1.5	0.2	1556	94.0	0.0	0.1	0.2	0.1	0.1	3.0
From Stadium to Hibret Bank															2.0
S16	0.016	0.04	128	5.2	250	1.2	3.5	1572			0.1	0.2			
S17	0.016	0.04	130	5.2	280	1.2	4.0	1568	137.0	0.0	0.1	0.2	0.0	0.1	8.8
S18	0.016	0.04	128	5.1	210	1.2	2.9	1564	204.0	0.0	0.1	0.2	0.1	0.2	9.4
S19	0.016	0.01	126	7.4	202	0.7	4.5	1557	176.0	0.0	0.1	0.2	0.0	0.1	8.2
S20	0.016	0.01	130	7.7	200	0.7	4.5	1555	214.0	0.0	0.1	0.2	0.1	0.3	12.2
S21	0.016	0.01	130	7.9	250	0.7	5.9	1552	210.0	0.0	0.1	0.2	0.1	0.2	12.4
S22	0.016	0.01	130	7.9	260	0.7	6.1	1549	150.0	0.0	0.1	0.2	0.1	0.1	13.9
S23	0.016	0.01	130	7.9	350	0.7	8.3	1558	214.0	0.0	0.1	0.2	0.0	0.1	14.2
S24	0.016	0.01	130	7.7	310	0.7	7.0	1563	171.0	0.0	0.1	0.2	0.0	0.1	16.3
S25	0.016	0.01	129	7.6	190	0.7	4.3	1568	132.0	0.0	0.1	0.2	0.0	0.1	14.8
From Sarbetoch To Anuar															11.9
S26	0.016	0.03	100	4.8	34	1.0	0.6	1575			0.1	0.1			
S27	0.016	0.02	112	5.9	71	0.9	1.4	1570	202.0	0.0	0.1	0.1	0.0	0.2	5.6
S28	0.016	0.01	125	7.6	100	0.7	2.4	1568	244.0	0.0	0.1	0.1	0.1	0.3	7.6
S29	0.016	0.01	130	8.8	146	0.6	3.9	1566	117.0	0.0	0.1	0.1	0.1	0.1	10.1
S30	0.016	0.01	130	7.9	96.0	0.7	2.3	1561	375.0	0.0	0.1	0.1	0.1	0.4	13.1
S31	0.016	0.02	129	7.2	129	0.8	2.7	1559	269.0	0.0	0.1	0.1	0.1	0.4	10.5
S32	0.016	0.01	125	7.6	112	0.7	2.6	1556	44.0	0.1	0.1	0.1	0.0	0.0	10.0
From Police Commission to Ethiopia hotel															10.3
S33	0.016	0.02	110	6	150	0.8	3.0	1558			0.1	0.2			
S34	0.016	0.02	110	5.8	15	0.9	2.9	1554	34.0	0.1	0.1	0.2	0.0	0.0	9.0
S35	0.016	0.02	130	7.3	225	0.8	4.8	1550	182.0	0.0	0.1	1.1	2.7	8.1	16.8
S36	0.016	0.02	126	7.3	264	0.8	5.8	1558	140.0	0.1	0.1	0.2	0.0	0.1	12.1
S37	0.016	0.01	128	8.3	133	0.6	3.4	1564	138.0	0.0	0.1	0.2	0.0	0.1	13.1
S38	0.016	0.01	130	8.4	206	0.6	5.3	1567	187.0	0.0	0.1	0.2	0.1	0.2	11.9
S39	0.016	0.01	120	7.6	207	0.7	5.1	1569	170.0	0.0	0.1	0.2	0.1	0.2	13.9
S40	0.016	0.01	117	8.4	208	0.6	5.9	1574	125.0	0.0	0.1	0.2	0.0	0.1	12.8
S41	0.016	0.01	120	8.6	209	0.6	5.9	1577	223.0	0.0	0.1	0.2	0.1	0.2	14.5
S42	0.016	0.01	126	8.9	208	0.6	5.9	1583	166.0	0.0	0.1	0.2	0.0	0.1	14.6
S43	0.016	0.01	129	9.1	207	0.6	5.9	1587	116.0	0.0	0.1	0.2	0.0	0.1	14.9

From Gebriel to Muluwongel															15.0
S44	0.016	0.03	120	5.4	48	1.1	0.8	1585			0.1	0.6			
S45	0.016	0.06	100	3.4	8	1.6	0.1	1583	58.0	0.0	0.1	0.6	0.6	0.6	6.7
S46	0.016	0.07	110	3.5	32	1.6	0.3	1576	173.0	0.0	0.1	0.6	0.5	1.5	5.0
S47	0.016	0.07	105	3.5	11	1.6	0.1	1575	82.0	0.0	0.1	0.6	1.0	1.3	5.2
S48	0.016	0.07	110	3.5	16	1.6	0.2	1571	89.0	0.0	0.1	0.6	0.5	0.8	4.3
S49	0.016	0.04	120	5.0	37	1.2	0.5	1568	125.0	0.0	0.1	0.6	0.7	1.4	5.1
S50	0.016	0.03	125	5.6	66	1.0	1.1	1564	74.0	0.1	0.1	0.6	0.5	0.6	6.1
															6.7

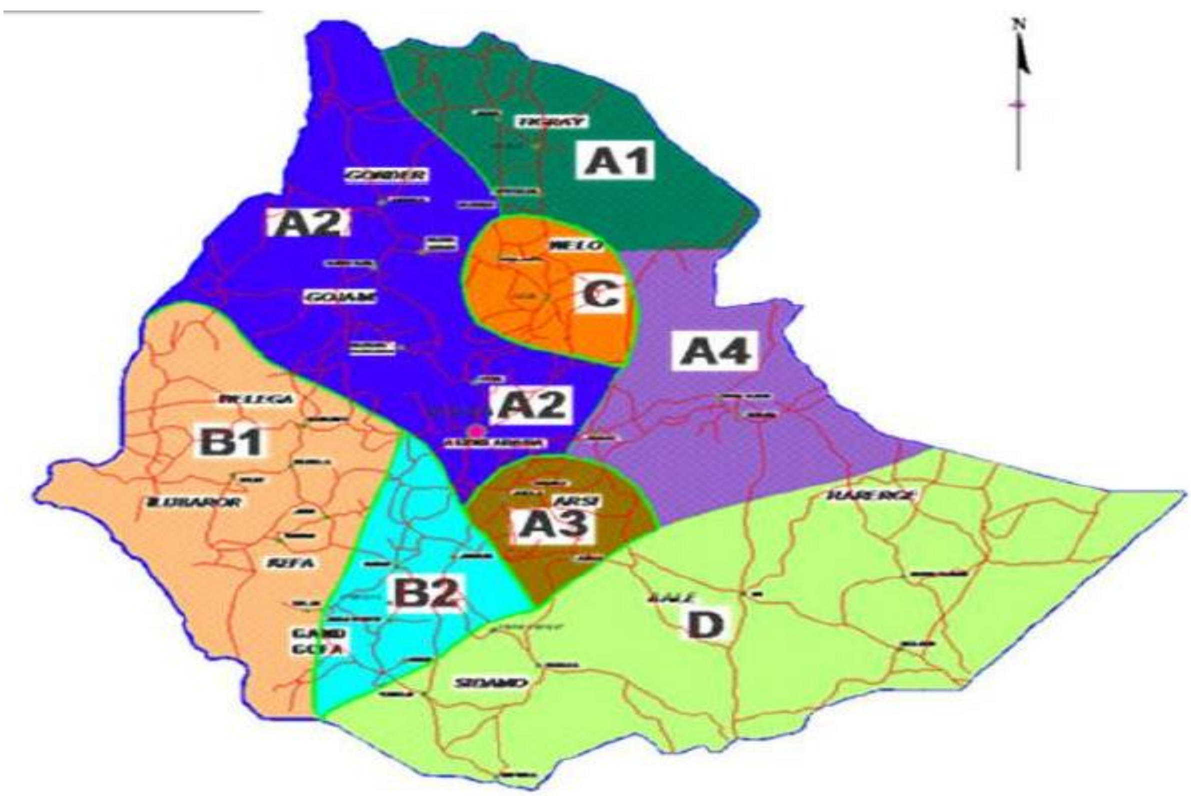
Appendix 4: Manning's roughness coefficient (n) - close conduits (Rossmann, 2010)

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

Appendix 5: Manning's roughness coefficient (n) - open channels (Rossman, 2010)

Channel Type	Manning n
Lined Channels	
- Asphalt	0.013 - 0.017
- Brick	0.012 - 0.018
- Concrete	0.011 - 0.020
- Rubble or riprap	0.020 - 0.035
- Vegetal	0.030 - 0.40
Excavated or dredged	
- Earth, straight and uniform	0.020 - 0.030
- Earth, winding, fairly uniform	0.025 - 0.040
- Rock	0.030 - 0.045
- Unmaintained	0.050 - 0.140
Natural channels (minor streams, top width at flood stage < 100 ft)	
- Fairly regular section	0.030 - 0.070
- Irregular section with pools	0.040 - 0.100

Appendix 6: Rainfall regions in Ethiopia



Appendix 7: Model output of profile plot

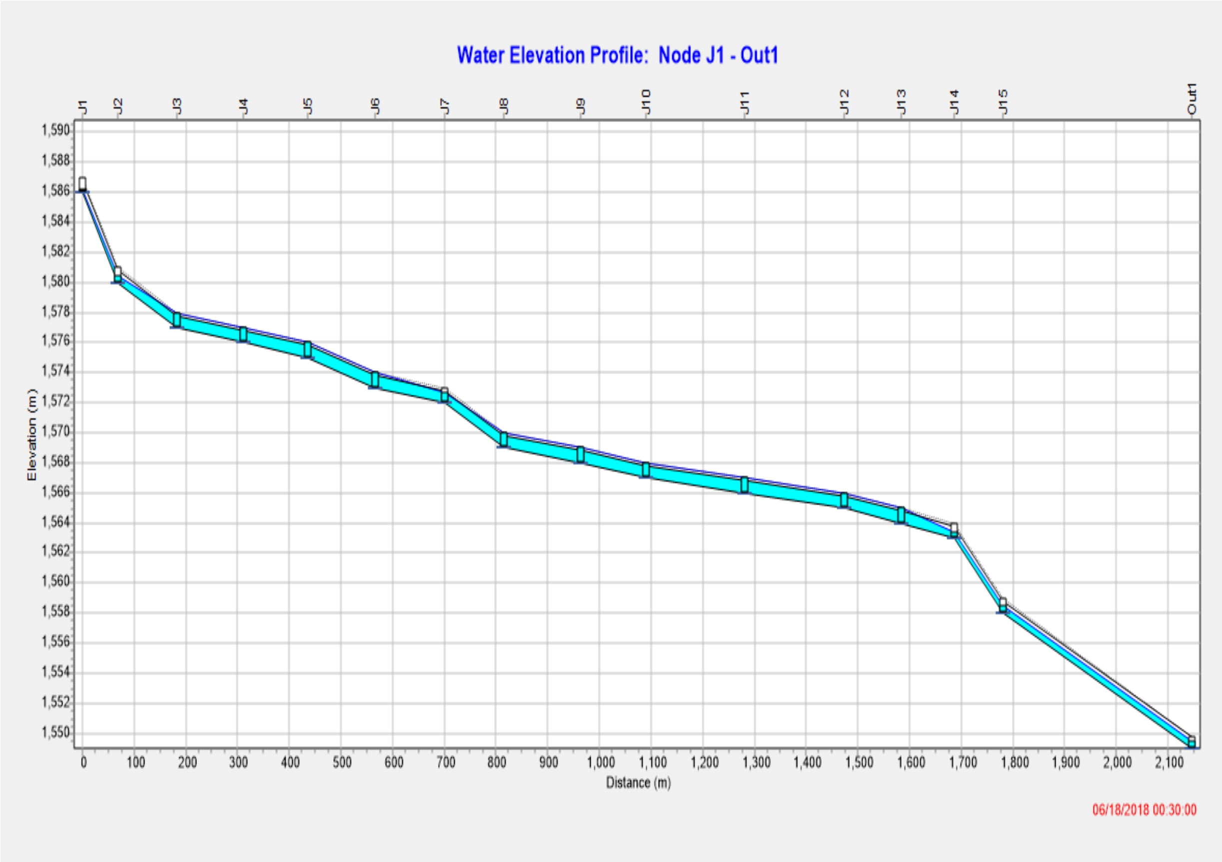


Figure 1: profile plot from stadium to mazorial to the design storm

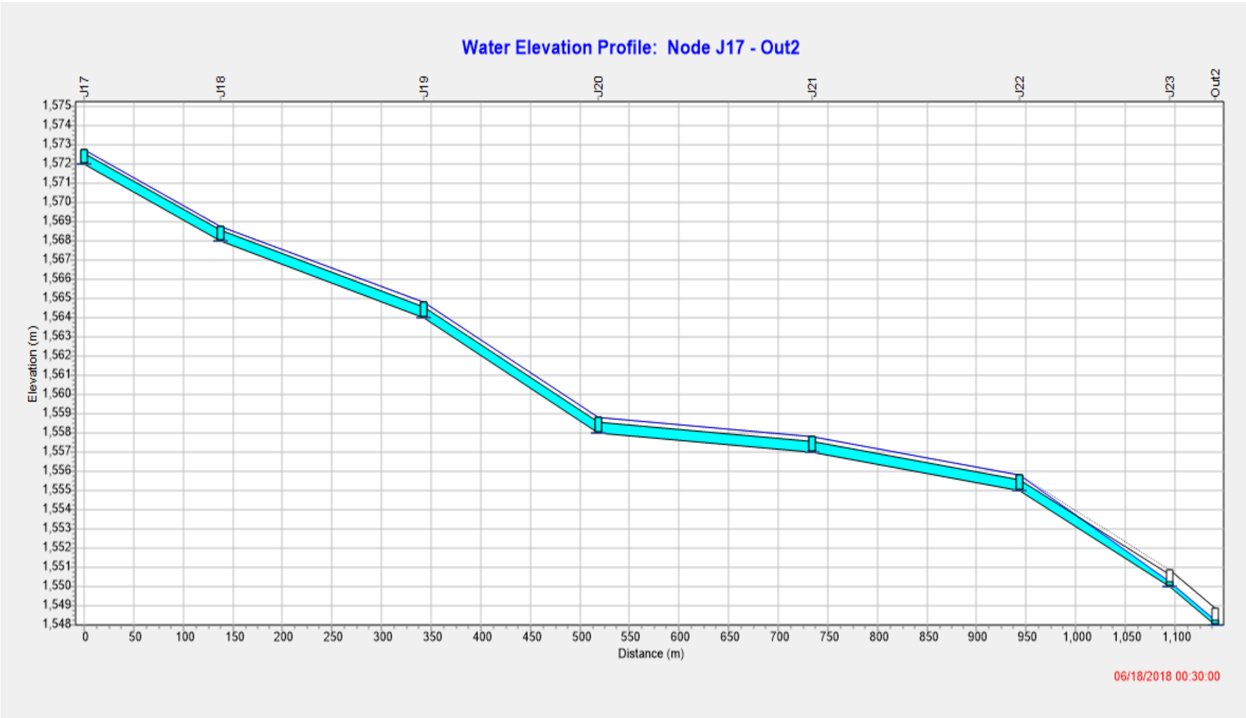


Figure 2: Profile plot from stadium to Hibret bank to the design storm

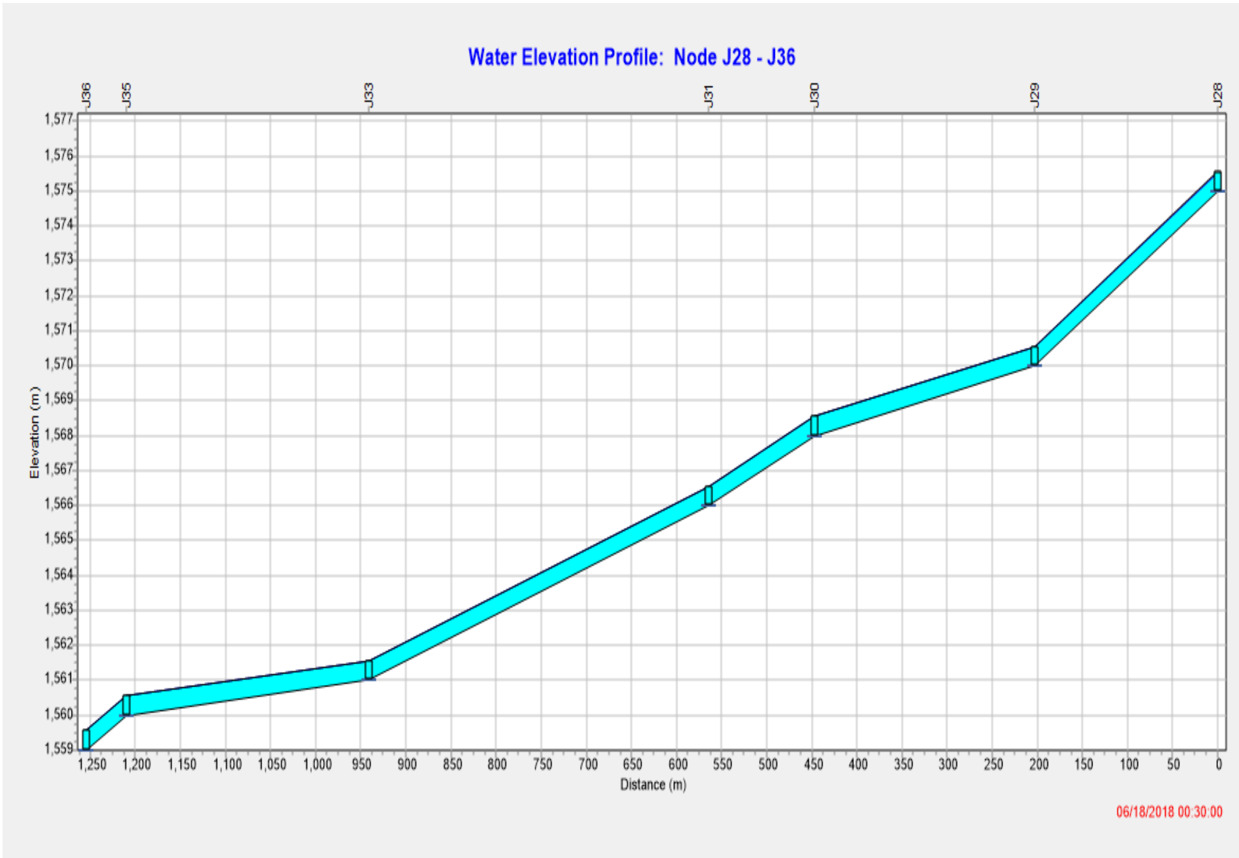


Figure 3: profile plot from Sarbetoch to Anuar to the design storm