ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

ADDIS ABABA INSTITUTE OF TECHNOLOGY

Improving Urban Runoff Drainage Systems In Shire Indaselassie Town

A thesis submitted and presented to the school of graduate studies of Addis Ababa University in partial fulfillment of the degree of Masters of Science Civil Engineering (Hydraulic Engineering Stream)

By

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Addis Ababa University

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Declaration

The author declares that this thesis work is his authentic work and that all sources of materials used for this thesis have been duly acknowledged. This Thesis have been submitted in partial fulfillment of the requirement for MSc degree at Addis Ababa Institute of Technology and deposited at the university library to be made available to borrowers under rules of the library. The author declares that this thesis is not submitted to any other institution anywhere for the award of any academic degree, diploma or certificate.

Mearg Belachew

Sign.

Dedication

This Thesis is dedicated to my father Belachew Mekuria and to my mother Beriha Tesfay.

Acknowledgement

Prior to any thing, I am very glad to give my honor and glory to the almighty of God. I would like to express my sincerely gratitude to my advisor Dr. Daneal Fekresilassie (Ass. Professor in Addis Ababa University) for his guidance.

Here also my praise to the organizations: Ethiopian Metrology Agency, Ministry of Water Resource and Municipal city of Shire those are gives me necessary materials. Last but not least, I would like to express thank to the Ministry of Education those who helped me financially for successful completion of this thesis.

Finally, we are very much happy to express my heartfelt thanks to my Families and friends who have supported throughout my study in AAiT.

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Abstract

Storm water discharges are produced when the capacity of the land to retain precipitation is exceeded and run-off occurs. Run-off will be influenced by rainfall and intensity and duration, antecedent storms and a number of watersheds, and land use characteristics such as slope, soil type, and impervious surfaces.

In Shire Enda Selassie city rainfall induced flooding during rainy season is a regular phenomenon. This may be due to either small size of the drainage canal, an incremental of rainfall, improper design of self-cleansing velocity or increasing of impervious surfaces of the catchment. Identifying the main problem associated with the drainage system feasibility and making the system sustainable is important.

To Simulate the flooding problem in the city using SWMM model which shows the over loaded conduits as well as junctions then select the preferable depth of flow in conduit or junction without overflow. Delineation of the area creates by depending on site visiting bounds the region by flow contributes to the junction or drainage. For one control point the total area contributes flow is delineated .The total catchment area modeled by SWMM is 56.8 ha, which have 131 junctions, 130 conduits (links) and two out fall considers. For this catchment the total runoff computed using model is 9.28m3/s whereas calculated by rational method is 7.4m^3/s. The network link simulation a result implies around 22.9 % of links is over pressured due to flood.

The total simulation accuracy of the runoff and network systems as assessed by statistical methods, were RNS, =0.62 and R2 = 0.97. The proposed mitigation measures are modifying the depth of links for those links. Another case the model cannot simulate the flood problem exists due to partially blocked the links, so for this problem continuous maintenance is mandatory. Therefore, the effectiveness of different sustainable drainage systems; Trench filter, swale and green roof and enlargement of pipe diameters were evaluated for this case study.

Key Words: Ethiopia, Shire Endaslasse, SWMM model, Sustainable drainage system, Mitigation measures.

List of Abbreviations

Catch-A1, Cach-A2... Catch-An – Sub-Catchments

CSA - Central Statistical Agency

CN - Curve Number

DEM - Digital Elevation Model

DMS - Degrees Minutes Seconds

EMA- Ethiopian Mapping Agency (Current nomenclature)

EP - Environmental protection

EPA - Environmental Protection Agency

ERA - Ethiopian Roads Authority

FHWA - Federal Highway administration

GIS - Geographic Information System

GPS – Geographical positioning system

HSG- Hydrological Soil Group

IDF- Intensity-Duration-Frequency

SWMM - Storm Water Management Model

USGS- United States Geological Survey

1. Introduction

1.1 Background

Storm water discharges are produced when the capacity of the land to retain precipitation is exceeded and run-off occurs. Run-off will be influenced by rainfall and intensity (millimeter of rain fall per hour) and duration, antecedent storms and a number of watersheds, and land use characteristics such as slope, soil type, and impervious surfaces.

Floods are natural and seasonal phenomena, which play an important environmental role, but when they take place at the built environments, many losses of different kinds occur. By its side, urban growth is one of the main causes of urban floods aggravation. Changes in land use occupation, with vegetation removal and increasing of impervious rates lead to greater run-off volumes flowing faster. Intense urbanization is a relatively recent process; however, floods and drainage concerns are related to city development since ancient times. Drainage systems are part of a city infrastructure and they are an important key in urban life. If the drainage system fails, cities become subjected to floods, to possible environmental degradation, to sanitation and health problems and to city services disruption. On the other hand, urban rivers, in different moments of cities development history, have been considered as important sources of water supply, as possible defenses for urban areas, as a way of transporting goods, and as a means of waste conveying.

1.2 Background of Shire Endaslasse

Shire-Endaslasse is a hub that connects part of Amhara region, Western Tigray the whole zone and Central zone of Tigray. Besides, as the area is potential for agriculture and other investments it is attracting a lot of investment activities in the area. The influence area for the town is, therefore, very vast which cannot be restricted to only geographical accessibility like to any other town; its location, its economic activity and the existing transport network (Airline and road) diversified the interaction of the town beyond geographical linkages.
Data regarding the population size of the town taken from the May 2007 census result and the 2012 CSA projected population based on 2007CSA which was 47,284 and 59, 887respectively. However, the study after looking different scenarios and based on the existing fast growing town

and potential of the town, the growth rate is computed on the high growth rate (4.5%) has put the projected figure to reach a total of 104,426 by 2024/2025.

The Shire Endaslasse city usually suffers from urban storm water drainage management problems. Because of poor urban drainage management and poor urban storm water design the majority of the drainage facilities got silted-up or blocked by silts and various solid waste materials. Besides, along with the high rate of urbanization these conditions worsened by unmanaged construction materials and bare terrains which surrounds the city and drains an external flood in to the city. The figure below shows as the silted up conduit and the flooded access road.

Figure 1-0.1 Silted Drainage and flooded the access road.

The increase in population density and building density exert the most obvious influence on hydrological processes in an urban area. Modification of the land surface during urbanization alters the stormwater runoff characteristics. The major modification which alters the runoff process is the impervious surfaces of the catchment such as roofs, sidewalks, roadways and parking lots, which were previously pervious.

Another factor is the natural channels, which were in existence before urbanization, are often straightened, deepened and lined to make them hydraulically smoother. Gutters drains and storm drainage pipes are laid in the urbanized area to convey runoff rapidly to stream channels. These increase flow velocities, which directly affect the timing of the runoff hydrograph. The combined effect of all these changes is to reduce the lag time of runoff. Since a larger volume of runoff (Due to urbanization) is discharged within a shorter time interval, the peak discharge inevitably increases.

As cities starts to grow, especially after the industrial Era, urbanization problems become greater and urban floods increased in magnitude and frequency. The traditional approach for the drainage systems, which were important as a sanitation measure in the first times of the city's development, conveying stormwater and wastewater, turned unsustainable. Flow generation increased and end of pipe solutions tended to just transfer problems to downstream. The water in the city needs to be considered in in an integrated way and sustainable solutions for drainage systems have to account for urban revitalization and river rehabilitation, better quality of communities' life, participatory processes and institutional arrangements to allow the acceptance, support and continuity of these proposed solutions.

Flooding is a natural occurrence caused by heavy rain. If it rains hard enough or long enough, low-lying areas are at risk of flooding. A flash flood occurs when the amount of rainfall overwhelms our natural and man-made drainage systems or low-lying areas. Generally, one inch of rain per hour will cause flash flooding.

Urbanization is the changing of land use from forest or agricultural uses to suburban and urban areas. The creation of impervious surfaces that accompanies urbanization profoundly affects how water moves both above and below ground during storm events, the quality of that stormwater, and the ultimate condition of nearby rivers, and lakes.

1.3 Problem Statement

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

In Shire Endaslasse city rainfall induced flooding during rainy season is a regular phenomenon. Almost every year a significant part of the city suffers badly by the flooding. The situation is likely to be worsening in the future due to climate change, which may lead to more frequent and intense precipitation. The Existing Drainage Structures most of them were designed under inadequate hydrological data condition and hydraulic analysis. The urban drainage of these structures failed to deliver the design yields due to sediments, less capability and failure-stability problems. Presently some of the problems that have been observed in the Shire Endasselase town are such as, base failure, Depressions, Shoving, Edge crack, Shoulder erosion, Silted drainage ditches, flooding etc. Major necessity to conduct this research work is frequent happening of floods in the area.

1.4 Objective

1.4.1 General Objective

The general objective of the study is to assess the applicability of urban storm water drainage system in solving the existing drainage problem and minimizing flood risk of Shire Endaslasse town.

1.4.2 Specific Objective

- ✓ To develop DDC (Depth duration curve) and IDF curve for Shire Indaselassie town.
- ✓ To identify the critical parts of drainage system flooding risk
- ✓ To Check the performance of existing drainage system
- ✓ To propose remedial measures

1.3 The Scope and limitation of the thesis

This research covers about 56.8 ha area which have 131 nodes and 130 conduits. The limitation of this model is it is tedious to delineate for large catchments. The software model needs primary data with the high quality to minimizing the error, but to collect the primary data there was financial limitation. For calibration and validation of this model needs observed flow rate and depth of flow at different time interval, but for this thesis for calibration and validation by using depth of flow. The SWMM model needs continuous record precipitation but in the Ethiopian metrology Agency record of data is daily precipitation.

1.4 The Outline of the thesis

This thesis organized into five chapters from introduction to conclusion and recommendation. The first chapter includes the introduction, background, Statement of problem, Objectives and specific objective, the scope and limitation of the research. Chapter two covers background related research reviews. Chapter three explains the material needs for the research and methodology how to analysis, method of analysis.

The fourth chapter holds all the results of statement of the problem, computation of flow by using rational formula and SWMM, network of links simulation. Comparison their result determined by those methods.

Finally, the last chapter covers conclusion and recommendation that gives the final judgment and mitigation for the existing problem.

2. Literature review

Urban drainage was firmly established as a vital public works system in the early parts of the twentieth century. Engineers continued to improve design concepts and methods. During the second half of the twentieth century regulatory elements were spread in the United States, Europe, and other locations addressing urban drainage issues. 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 concerns for ecosystem protection and urban sustainability. (Anteneh, 2015)

As pointed out by Silveira (2001), the practice of urban drainage in developing countries is typically more complex than in developed countries as it is subject to more difficult socio-economic, technological and climatic conditions. At the heart of the problem is an urban development without planned drainage, with its typical associated increase on flood frequency and degradation due to erosion and sedimentation. Some of the reasons explaining the lack or absence of adequate planning are:

- urban development occurs too fast and unpredictably;
- peri - urban areas are often urbanized without taking into account the city regulations (unregulated developments, invasion of public areas);
- peri - urban and risk areas (flood plains and hill side slope areas) are occupied by low income population;
- lack of funds for planning activities; and
- lack of proper institutional organization (lack of information, guidelines, regulations and law enforcement)

The urban drainage system was first challenged due to the interactions between human activities and the natural water 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 rain water diversion from natural drainage system and generating a considerable runoff. Consequently, both types of water are requesting 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 rain water, 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),p - 28)

2.1 Related studies on Drainage

Studies on Drainage issues in Ethiopia, those researchers post graduate Students of Addis Ababa University their case study in addis Ababa and Benishangule-Gumuz. The first thesis is titled, "Study of the Urban Drainage System in Addis Ababa, Yeka Sub city" (Dagnachew, 2009). The second thesis is titled, "Investigation on Storm Drainage Problem of Addis Ababa - Case Study at Gotera – Wollo Sefer, Saris - Gotera and Ring Road" (Desalegn, 2011). The thesis third title is Integrated Urban Drainage System; the Case of Ayat to Megenagna LRT Route (Anteneh 2015), the fourth title Performance Assessment of Road Drainage Structures and Proposed Mitigation Measures: The case of Daleti-Odagodere Gravel Road in Benishangul-Gumuz Region (Yifred, 2015) and the fifth title Modeling and analyses of urban flooding, Addis Ababa Bole Sub city Case study (Nejib 2016).

Related Studies on Drainage by using SWMM model reviews; the title Modeling and analyses of urban flooding, Addis Ababa Bole Sub city Case study (Nejib 2016) and the research journal (M. L. Waikar* and Undegaonkar Namita U, January, 2015) Urban Flood Modeling by using EPA SWMM.

2.2 The Impact of Urbanization on the Drainage System

Urbanization is the changing of land use from forest or agricultural uses to suburban and urban areas. The creation of impervious surfaces that accompanies urbanization profoundly affects how water moves both above and below ground during storm events, the quality of that stormwater, and the ultimate condition of nearby rivers, and lakes.

There are significant differences between the natural water cycle and the urban water cycle there are a number of connected processes including evaporation, condensation, precipitation and formation of ground water (see figure (a) below). However, the natural water cycle is included into the urban water cycle, but it is interrupted due to less natural infiltration and less ground water formation due to increase of the impermeable surfaces (see figure (b)).

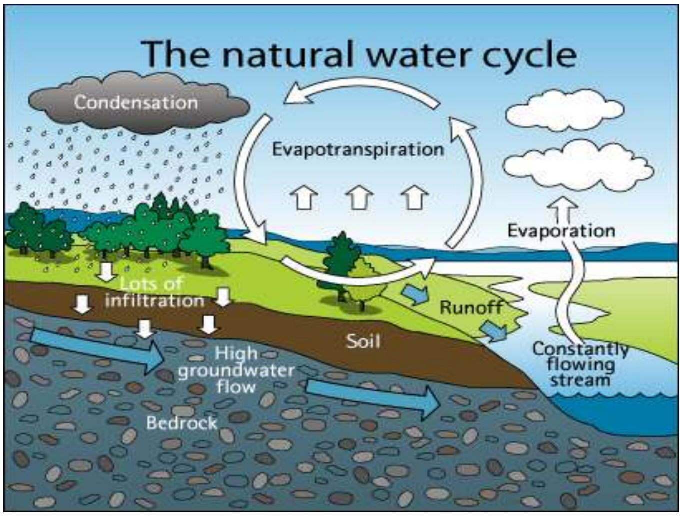


Figure 2-0.1 The Natural water cycle (Moheseen, 2015)

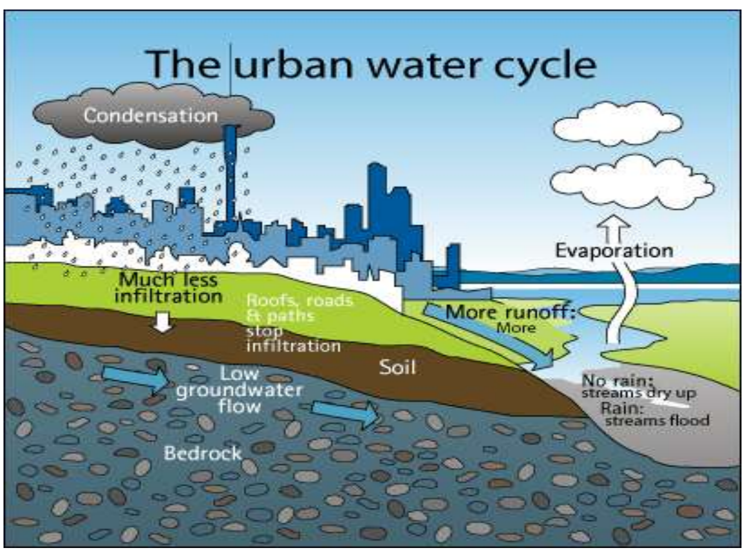


Figure 2-0.2 Urban Water cycle (Moheseen, 2015)

The main difference between the natural and urban water cycle is the degree of natural infiltration, where in both cycles the rainwater is subjected to losses due to the evaporation to the air and transpiration by the plants. But in the natural areas, the surplus of the rainwater infiltrates in to form or/and recharge the groundwater. Still, a proportion of runoff can be formed (overland flow), but it is relatively less than the infiltration and depends on the surface permeability and soil type dominated, also is changeable under the storm event. In the urban areas, the same cycle is taking place, but the permeability degree of the surfaces influencing the ration between groundwater proportion and runoff proportion; Where the hard and impervious surfaces increases the surface runoff proportion comparing to the groundwater one. Further the hard and impervious surfaces influencing the runoff to move faster on the urban surfaces than the natural surfaces; causing flooding and surcharge on the sewers and recipient. (Bulter and Davies, 2011).

Further, both of the groundwater and surface runoff end up in the recipient such as river and both contribute to the river but in different way. While the groundwater contributes to the base flow of the river, the surface runoff increases the flow under the storm event and thereafter increases the volume in river (Bulter and Davies, 2011)

2.3 Components of Urban Drainage system

A stormwater drainage system describing processes and components are shown in Figure

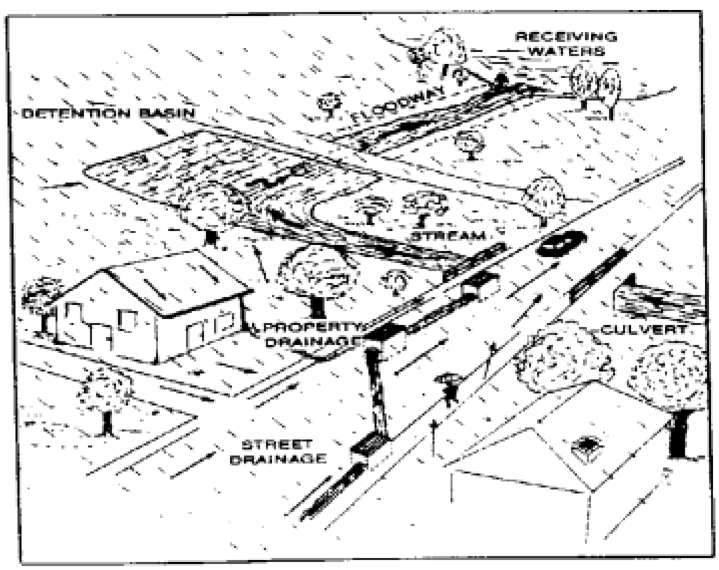


Figure 2-0.3 the Urban Drainage System (The Institution of Engineers, 1987)

2.3.1 The main parts of an urban drainage system

- Property Drainage

Properties consists of two types of hydrologically important surfaces namely Impervious and pervious areas. Impervious areas consist of surfaces such as roofs, backyard sheds, garages, driveways, access roads, parking places, tennis courts etc. pervious areas in urban catchments consist of areas such as residential backyards, parks, playgrounds etc. the stormwater from both impervious and pervious surfaces is connected to the drainage system. The roof is the main impervious portion of a property.

The roof drainage system consists of gutters, down pipes, receiver boxes and runoff inlets. Roof gutters have different shapes such as rectangular and trapezoidal gutters. The gutter generally discharges freely into a receiver box, the depth of which can be selected so as to match the use of a downpipe of convenient size. The receiver box is then connected by downpipes. It should be noted that the receiver boxes are used only for large buildings such as office building, schools, factories etc., and not for small houses.

- Street Drainage

Streets are normally drained by a network of gutters, pits and pipes. The street drainage system collects runoff from road surfaces as well as land adjoining streets, and discharged to trunk drains. In addition, in some cases runoff from properties is also disposed to the street drainage system.

The stormwater from the street gutter system enters the underground drainage system through inlets located in street gutters. There are types of inlets in use namely Kerb inlets, gutter inlets and combined inlets. The kerb inlet has a vertical opening to catch the gutter flow. Although the gutter may be depressed slightly in front of the kerb inlet, it offers no obstruction to traffic. The gutter inlet is an opening covered by a horizontal grate through which stormwater enters. The disadvantage of the gutter inlet is that debris collecting on the gate may block the inlet (Hammer, 1977). Combined inlets, composed of both kerb and gutter openings, are also common in drainage systems.

Street grade, kerb design and gutter depression define the best type of inlet for a particular situation. Nevertheless, minimizing traffic interference and reducing inlet blockage often take precedence over hydraulic efficiency in selecting the type of inlet.

- Trunk Drainage

The trunk drainage system consists of large or open channels (concrete or earth) which carry runoff from the local street drainage system to receiving waters. Trunk drains serve several sub-areas, which are physically large, and therefore; the overflows are likely to cause direct damage and prolonged inconvenience (The Institution of Engineers, 1987).

- Retention and Detention Basins

Retention basins, often known as water quality control ponds at least in recent times, are small lakes located in stream or off-stream along urban waterways. They can be extremely effective in removing pollutants, since they allow a range of physical, chemical and biological processes to take place, which improve water quality. Retention basins hold stormwater for considerable periods, which cause stormwater to be in the hydrologic cycle via infiltration, percolation and evapotranspiration (The Institution of Engineers, 1987).

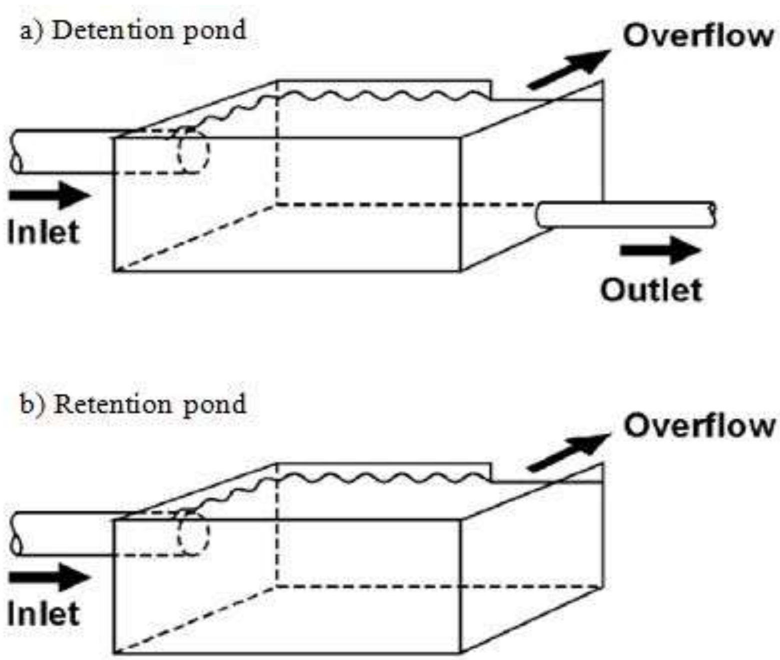


Figure 2-0.4 Flow retention and detention structures

Detention basins are commonly known as retarding basins. The detention basins hold runoff effort short time periods to reduce peak flow rates and later release into natural or artificial watercourses. Therefore, the volume of stormwater runoff is relatively unchanged from the original volume. They may be sport fields of specific sizes and shapes (e.g. football and cricket grounds). The sides of basins are usually sloping earth embankments, suitable for spectator use if they are sport fields (The Institution of Engineers, 1987).

- Receiving Water Bodies

The major receiving water bodies that are considered in urban drainage include rivers, lakes, and the sea and ground water storages.

2.4 Storm water management

An assessment of the project will have on existing peak flows and watercources shall be made by the designer during the initial phase. The assessment shall identify the need for storm water management and non-point source pollution control (SWM & NPSC) facilities and potential locations for these facilities, when runoff flows along the ground, discharge or non-point source pollution. Mitigating measures can include, but are not limited to, detention/recharge basins, grassed swales, channel stabilization measures, and easements.

Storm water management, whether structural or non-structural, on or off site, must fit into the natural environment, be functional, safe, and aesthetically acceptable.

Several alternatives to manage storm water and provide water quality may be possible for any location. Careful modeling and produce optimum results.

The natural water cycle is altered by human activity in two main forms: water is extracted for water supply for human activity and the natural drainage is altered by the shift in land use with more impervious area (Butler & Davies, 2000). Hence, it raises the need to drainage of wastewater and storm water. However, storm water becomes more important when it comes to flooding since the quantity of water is much higher.

The storm water results from all kind of precipitation (snow melt, rainfall, etc.) and comprises the water flowing in the surface (Butler & Davies, 2000). Therefore, the characteristics of both the rainfall and the catchment area represent important factors in the storm water properties. Indeed, part of the water of the rainfall goes to initial losses as interception, depression storage, infiltration and evapotranspiration. The remaining water is the runoff (Durrans & Haestad methods, 2003).

Storm water is water generated on the land surface, originating from rainfall or melting snow or ice (Durrance, 2003). The concept of stormwater is strongly related to urban areas where conveyance systems exist.

2.5 Green Roof

Vegetated roofs should preferably be constructed on flat or gently sloped surfaces. In a publication by Belan & Otto (2004), some innovative European techniques have managed to grow vegetation on roofs with a 45 degrees of tilting are discussed. The best results are achieved when the slope of the roof is between 5 and 20 degrees, since the water then can be drained by gravity (Hinman, 2005). For flat roofs, a drainage layer able to remove the water from the root zone and the impermeable membrane must be installed (Belan & Otto, 2004). Green roofs are suitable on a number of buildings, ranging from industries, warehouses, office complexes, hospitals, schools, and garages to residential buildings (GVSDD, 2005).

Results by Miller, (2001b) and FLL (2002) found in GVSDD (2005) indicate that green roofs with about 75 mm of growing media have the capacity of removing around 50% of the annual rainfall volume through retention and evapotranspiration. Best results were achieved during storm events when the soil was not fully saturated (GVSDD, 2005). TRCA et al.,(2010) states that the reduction percentage is a function of the depth of the soil, the roof slope and climatic conditions of the area. The review presents a number of monitored results on extensive green roofs, all varying between 50- 85% reductions of stormwater (TRCA et al., 2010). LIDC (2007b) highlights similar results with removal rates of water varying from 58% to 71%, depending on the thickness of the soil and the characteristics of the vegetation.

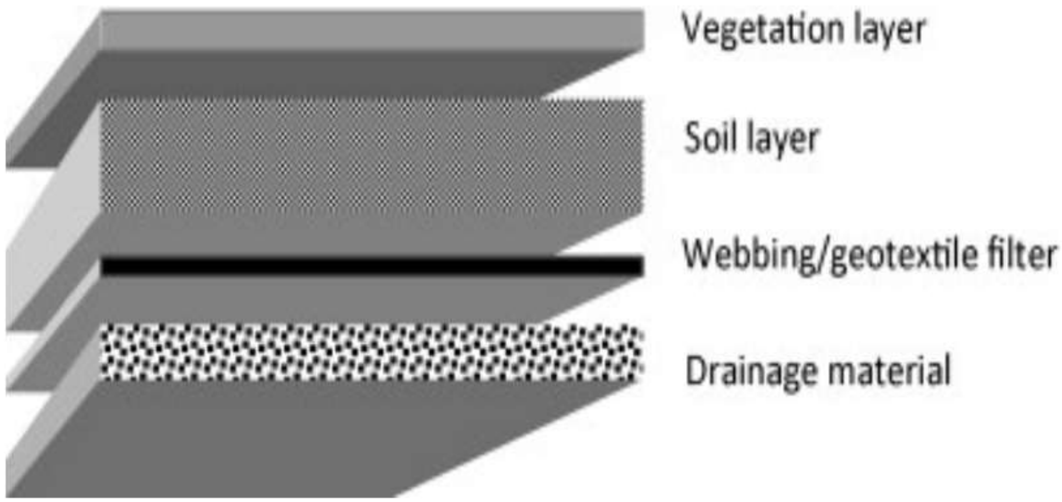


Figure 2-5 Typical Cross-Section of Green-roof

Green roofs can be implemented on new buildings as well as a retrofitted on existing structures (Stahre, 2006). In the article by Becks et al., (2010), the potential for retrofitting is investigated. The article enlightens the importance of retrofitting for green roofs. The United Kingdom is taken as an example. As for today, over 80% of the building stock for 2025 is already constructed, and if there is to be an increase in the usage of vegetated roods, retrofitting is an important aspect.

2.6 Swales

Swales are successfully applicable adjacent to parking areas, commercial and light industrial facilities, roads and highways, residential developments, and are often implemented as an alternative to curbs/gutters and storm sewers (Cahill Associates Inc., 2006). Normally, swales are located along the impervious area it is to drain, and runoff may enter the swale all along its length (Stahre, 2006). To increase the storage capacity of the swale, a feasible solution is to design a soak away under the bottom of the swale (ibid.). Brown et al., (2001b) indicates that the side slopes should be maximum 3:1 to ensure as large a wetted perimeter as possible, thereby contributing to a high efficiency of the solution.

Normally, swales take a parabolic or a trapezoidal design (Cahill Associates Inc., 2006). The longitudinal slope should not exceed 4%, and should preferably be around 1-2%, according to TRCA et al., (2010). If the slope is too steep, water velocities become too high, which may result in erosion and inadequate infiltration to the soil (Stormwater Center, 2011c). If dimensioning for the biggest estimated flow with a recurrence time of two years, VADCR (1999) writes that the maximum flow should be 1.2m/s, and if dimensioning for a rain with an estimated return period of ten years, the largest flow should be 2.1m/s.

When designing swales, size can be decided through the usage of Manning's Equation, but a rule of thumb to facilitate the procedure is to install swales on at least 1% of the drainage area (USEPA, 1999b). TRCA et al., (2010) on the other hand writes that a suitable size for the swale is about 5-15% of the contributing drainage area. This area for swales is normally smaller than 2 hectares (Stormwater Center, 2011c). The efficiency in performance is often directly proportional to the maintenance frequency. Maintenance includes keeping a thick grass cover, removing excess sediments, repairing damaged areas and handling possible puddles created (USEPA, 1999b).

2.7 Overview of Runoff estimation methods

Catchments runoff, and its characteristics such as volume, peak flow and return period can be determined using one of four methods: Statistical analysis of observed flow records; Regional methods; Transfer methods and Rainfall-Runoff methods (manual or computerized). These are briefly described below.

2.7.1 Statistical analysis

Statistical Analysis of observed records makes probabilistic statements about flows (annual discharge, frequency of occurrence are a representative sample of the population of flow events. The technique has an advantage over all other runoff estimation methods; in that it is the only technique which makes use of measured flows for an area of interest. However , it is rarely applicable to urban areas as there are few urban gauging stations, those records that do exist are short in comparison to rural sites, and suffer from a likelihood of significant changes to the catchment during the period of measurement, making them unsuited to probabilistic analysis. In addition, an observed flow technique cannot be used to make inferences about changes to runoff following changes to catchment characteristics.

2.7.2 Regional Methods

Regional Methods relate a dependent variable, such as annual discharge, with physically related independent variables, such as catchment area. Given this relationship (correlation or regression) for a gauged site, and information on the independent variable, the dependent variable can be determined for ungagged site. These methods are specifically regional as they are intended only for use in regions that supplied the data underpinning the original relationship (e.g. UK planning regions). These methods are easy to apply, and have the advantage that effects of urbanization can be investigated, for example by manipulating an impervious area variable. However, they do require a large number of flow records from which the appropriate relationships can be determined. Thus regional methods are rare in urban drainage studies due to the lack of flow records. Greater in urban flow monitoring, and methods to extend observed flow records using rainfall records mean that regional methods are in future likely to become more popular for urban areas.

2.7.3 Transfer methods

Transfer methods are used to determine hydrologic characteristics, say a flood return period, for a smaller or larger part of catchment with similar runoff characteristics the discharge transfer is made in proportion to the ratio of tributary area, and an exponent n determined from regional methods or from the slope of a regional area-flood graph. It is assumed that the only significant difference between the two areas is the catchment area. Another transfer method interpolates results from unknown data from two or more points on the same catchment. These methods require relatively little data, but suffer where catchments are not homogeneous in land use and land cover and also have a complexity of stormwater structure that modify runoff.

2.7.4 Rainfall-runoff methods

Rainfall-runoff methods are the only methods that incorporate precipitation, the causal process behind runoff estimates of discharge. They do by considering rainfall as intensity (mm/hr) or as a hyetograph (a graph of time distribution of rainfall over a catchment). There are several approaches (coefficient methods, soil-complex-cover methods, unit hydrograph methods), described in more detail below, all of which have been applied to the estimation of urban runoff characteristics, including, for example, hourly.

Stormwater

Water is a finite and vulnerable resource, essential to sustain life, development and environment. Stormwater is the water draining off a site from the rain that falls on the roof and land, and everything it carries with it. The soil, organic matter, litter, fertilizers from gardens and oil residues from driveways it carries can pollute downstream waterways. Rainwater refers only to the rain that falls on the roof, which is usually cleaner. However, stormwater can be a valuable resource. Reusing stormwater can save potable water and reduce downstream environmental impacts. In urban areas stormwater is generated by rain runoff from roofs, roads, driveways, footpaths and other impervious or hard surfaces. In Australia the stormwater system is separate from the sewer system. Unlike sewage, stormwater is generally not treated before being discharged to waterways and the sea. Poorly managed stormwater can cause problems on and off site through erosion and the transportation of nutrients, chemical pollutants, litter and sediments to waterways. Well-managed stormwater can replace imported water for uses where high quality water is not required, such as garden watering. (Hatt, B, et al 2004).

Runoff

New developments have a direct impact on existing drainage infrastructure and the surrounding environment (Butler and Maksimovic, 2001). They increase the area of paved surfaces, thus reducing infiltration, while causing surface runoff to exhibit higher peak flows, larger volumes, and shorter times to peak and accelerated transport of pollutants and sediment from urban areas. This results in pollution of the receiving watercources and increased flood risk within the development. Controlling surface runoff thus becomes a key element in working towards urban sustainability. (Markopoulos,et al., 1999).

2.8 Urban Drainage System Modeling

The main objective of modeling in urban watershed is to understand how the system responds for a single or series of storm events specified by the modeller and take appropriate action accordingly (Akan, 1993). 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 modelling. 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.

Arch SWAT Software is also used to model and analyze flood especially for rural areas with a large catchments. SWMM is used in urban areas flood modelling and analyses to fix the drainage size by considering the pervious and impervious areas. So this thesis is flood modeling in urban areas and the SWMM software is more comfortable, because its calibrated and validated software in our country Cities of Jimma and so on. SWMM was used, adapted and calibrated for the Ballona Creek Watershed, a catchment in Southern California.

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, Thunder storms (convective rainfall) may be highly localized, and nearby gages may have very dissimilar readings.

2.9 Calibration Against data

SWMM parameters should typically be calibrated and validated against measurements to reach reliable results. However, some of the model parameters are quite straightforward to deduct from e.g. accurate spatial data and can be reasonably defined even without calibration. Those include subcatchment areas and slopes, for instance. On the other hand, parameters such as the flow width, the depression storage, the roughness coefficients, and the infiltration parameters involve larger uncertainties and are commonly used as calibration parameters. Nevertheless, also the first mentioned 'straightforward' parameters involve uncertainties and are often calibrated for a better fit. (Liong et al., 1991)

2.10 Stormwater pipe Materials

Following the guidelines from the International Standards Organization for Testing and Materials (ASTM), in order to reduce flooding, promote adequate drainage, and reduce maintenance, certain pipe materials depending upon usage and location of the pipes. Material for pipe used for conveyance of stormwater shall be in accordance with the following (ASTM 2008):

- Reinforced concrete pipe, double-walled high-density polyethylene pipe, corrugated double-walled polyvinyl chloride pipe, corrugated metal pipe and non-reinforced concrete pipe may be used to convey onsite stormwater runoff (stormwater runoff generated by the subject property including areas such as buildings, parking lots, etc.).
- Reinforced concrete pipe is required for all detention basin outlet structures and outlet pipes.
- Reinforced concrete pipe is required when stormwater systems carry water from offsite areas (stormwater runoff generated by areas other than the subject property, where the runoff drains across the subject property) or when there is a potential to cause flooding or damage to adjacent properties. Reinforced concrete pipe is also required for all stormwater systems located within new residential developments (includes residential condominium developments).
- Stormwater pipes installed under city streets shall be reinforced concrete pipe. Stormwater pipes within the roadway prism of city streets and JPEs, but not under the pavement, shall also be of reinforced concrete pipe
- Any stormwater pipe/system that is installed with intent of dedicating to the City, whether inside or outside of the right-of-way, shall be constructed with reinforced concrete pipe.

- Existing residential subdivisions that have a roadside ditch and driveway pipe stormwater system may use reinforced concrete pipe , double-walled high density polyethylene pipe, corrugated double-walled polyvinyl chloride pipe ,corrugated metal pipe , and non-reinforced concrete pipe as desired by the responsible agency, corporation, or individual. Reinforced concrete pipe is required underneath any driveways or entrances that are heavily traveled.
- For the situations referred to above where reinforced concrete pipe is required, ductile iron pipe might be allowed to be substituted for reinforced concrete pipe when site limitations exist. (ASTM, 2008)

3. Research Method and Material

3.1 Description of the Area

3.1.1 Location

The city of Shire Endaslasse is located in northern part of Ethiopia. Its administrative location is Tigray National Regional State, North western zone. While, its absolute geographic coordinates $14^0 6$'N $38^0 17$'E, the average elevation of the town is approximately 1923 m.a.s.l. The town is found about 1106kms faraway from Addis Ababa via Mekelle-Adigrat way and about 300kms from Mekelle, regional capital, to Northwest of the region.

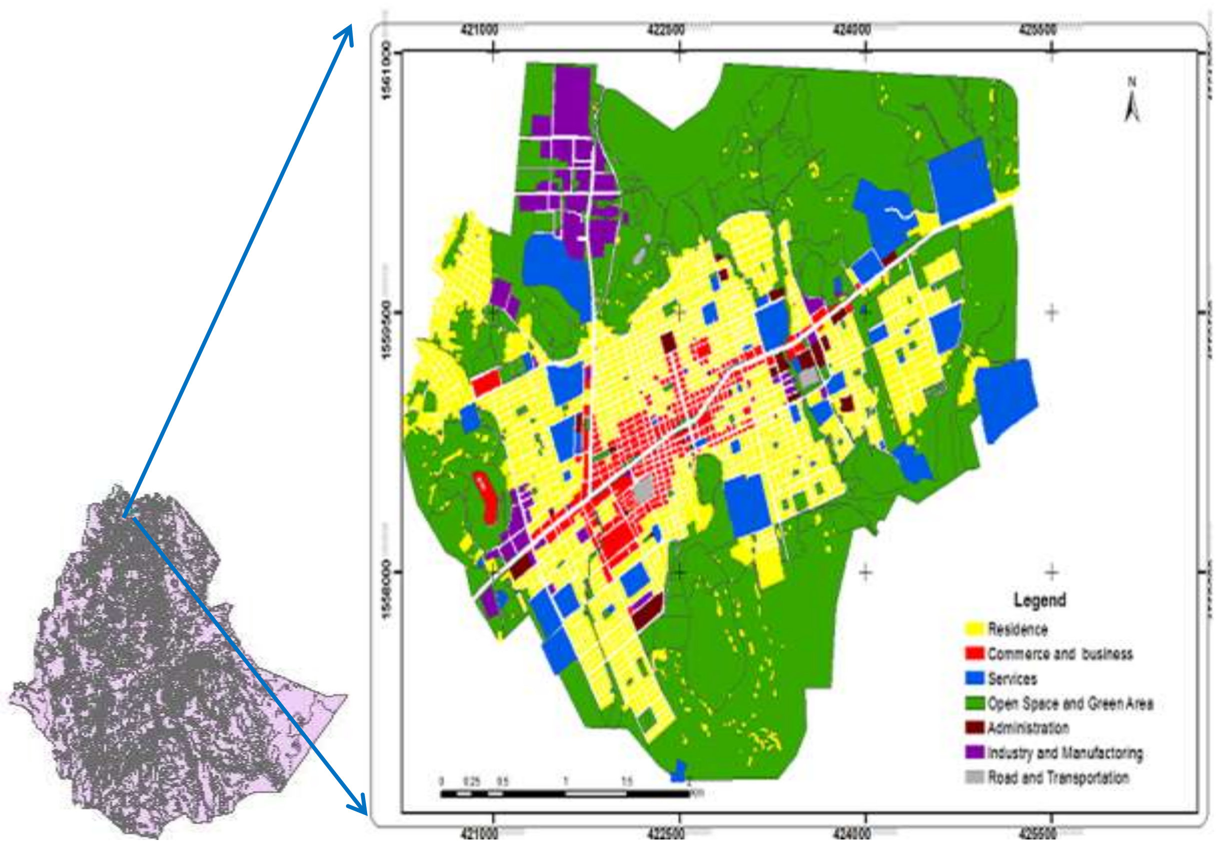


Figure 3-0.1 the location of Shire Endaslasse

3.1.2 Climate

The climatic condition is Tropical savannah Climate with average rainfall reaching 905mm.The mean minimum monthly temperature is expressed between 5.1^0c in September and $5.7\ ^0$c in June and the mean maximum monthly temperature range from 24^0c to 28^0c in May.

Table 3-0-1 Temperature Maximum, Minimum and average

Months	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Average min.	4.1	3.9	2.7	5.5	3.5	5.7	5.0	5.1	4.1	4.8	4.6	5.8
Average max.	24.1	24.5	26.1	27.0	27.0	27.7	28.2	26.9	25.0	25.3	25.4	25.3
Mean	14.1	14.2	14.4	14.3	15.2	16.7	16.6	16	14.0	14.1	15.0	15.5

3.1.3 Demography

The 2007 national census report by CSA indicated that the population size of Shire Endaslasse town was about 47,284 (21,867 males and 25,417 females). As shown in the statistical abstract of 2012 by the National Population and Housing Census of Ethiopia, population size of the town was 59,887 (27,691 males & 32,196 females) by June 2012. The two figures show that Shire Endaslasse has been growing by 2521 persons every year between 2007-2012E.C according CSA population projections. Using the CSA data and other regional sources, projected population of Shire Endaslasse for 2015 by the study team indicates that population size of the town was 67,243 (31,097 male & 36146 female).

Accordingly, Shire Endaslasse town is the largest urban population in North Western Zone.

Structurally, this population figure in the town is organized in to five major Kebeles: Kebele

1 (Dedebit), Kebele 2 (Suhul), Kebele 3 (Hibret), Kebele 4 (Adi-Kentibay) and Kebele 5 (Lekatit).

3.1.4 Expansion History of Shire Endaslasse Town

Shire Endaslasse town settlement development is mainly a reflection of three peculiar historical planning trends and development effects that has resulted to the current socio economic transformation. The first is from its birth till Italian war times, secondly from victory till 1994 and the Last trends the current scenario after the National Urban Planning Institute Plan.
Using historical Satellite data, an attempt has been made to examine the historical expansion trend of Shire Endaslasse town starting from the year 1973, since it is the earliest time where historical spatial data has been available. Accordantly, in the year 1973, the total area of Shire Endaslasse town was 48.7 hectare. Increasing by an average rate of 3.8 hectare per year, this area has raised to 91.3 hectare in 1984. Afterwards, the expansion rate of the town has risen to 14.8 hectare per year and the total built up of the town has reached to 552.8 hectare in January 2013. In this regard, the contribution of the 1994 NUPI plan was significant to reshape the different spatial expansion trend of the town. Therefore, Shire Endaslasse has expanded its settlement

areas about 2 fold within two decades and it is also evident that the urban growth direction has been pulled to North West Direction future expanding the town linearly.

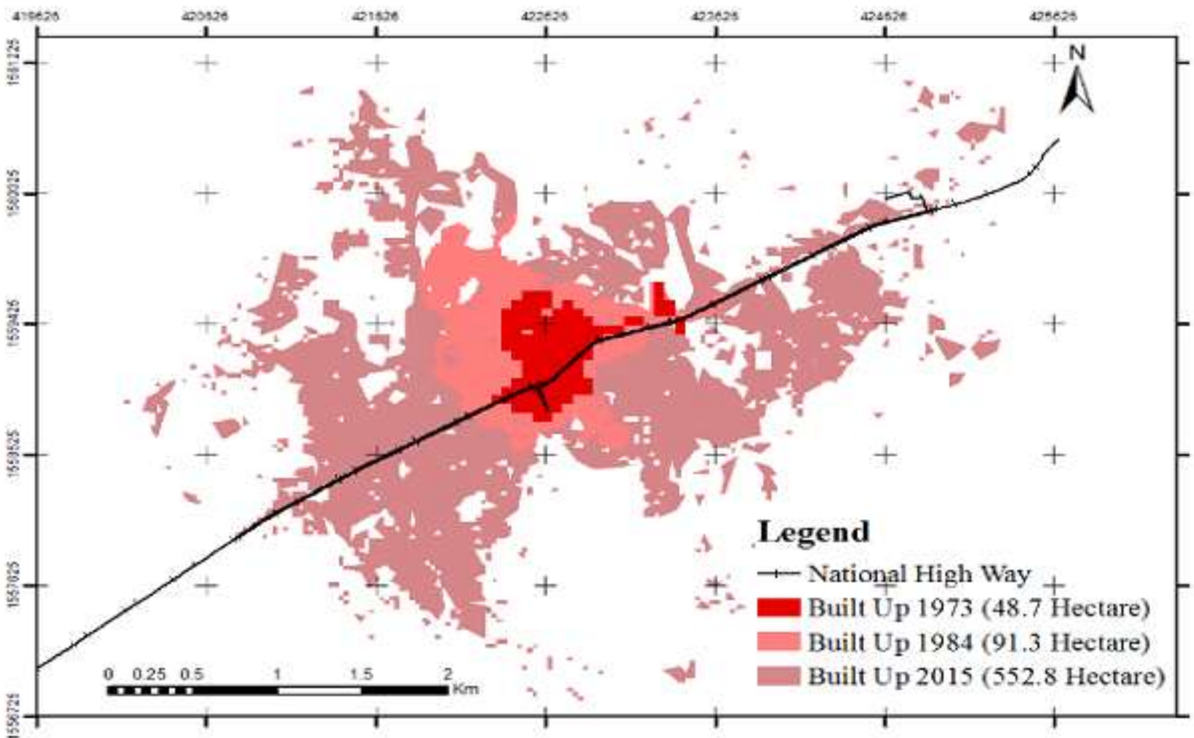


Figure 3-0.2 Urban Growth of Shire Endaslasse

Source: Shire Endaslasse Municipality

3.1.5 Existing Drainage System in Shire Endaslasse

In Shire city there are two types of urban drainage lines. Namely: closed (buried) and open drains.

- Closed urban drainage lines

These are part of the drainage system which has been constructed by the municipality and Ethiopian Roads Authority to safely discharge the flood generated within the city in the process of urbanization. But, as observed during data collection through field survey with the help of road net-work/Base map and check list, these drains becomes causes of flooding by collecting rain water from various parts of the drainage system. These drains totally concentrated on asphalt roads. The majority of these drains have been blocked by solid wastes of various types and silts that washed and joined from the highly eroded terrains, quarrying activities, unmanaged construction sites, dumping of excavated materials and others. Since, all the closed/buried drains have difficulty to easily control or inspect, they have easily been silted-up and blocked. In addition to this, the manholes have been closed with no reason. Some of these manholes are non-functional and do not allow inspection to take place.

- Open urban drainage lines

Similar to the closed urban drainage line types the function of open drains is to safely remove the storm water generated in the built-up area of the city to the final receiving system. According to physical observation during field survey (data collection) these drain types are concentrated/ limited only on non-asphalt roads and are very limited in length.

3.1.5.1 Existing drainage dimension

The existing drains are too limited and about 65.2% of them is closed pipe, 25.2% of them is Gutter with cover and the rest 9.6% is Gutter with no cover or open.

Table existing drainage dimension

No	Type of Drain	Code	DIMENSION (cm)			Length (m)	Road Surface
			Height, H	Width, B	Diameter, D		
1	Circular Concrete Pipe	A			90	26800	Asphalt
		B			90	1300	Asphalt
		C			90	405	Gravel
		D			90	2040	Asphalt
		E			90	310	Gravel
					Subtotal	**30855**	
2	Rectangular Masonary with Cover	H	160	140		3560	Asphalt
		I	90	120		2110	Gravel
		J	100	130		880	Gravel
		K	100	90		5400	Gravel
					Subtotal	**11950**	**0**
3	Rectangular Masonry with no Cover	L	70	70		475	Gravel
		M	130	120		775	Asphalt
		N	120	120		700	Asphalt
		O	130	90		335	Asphalt
		P	170	450		1265	Gravel
		Q	120	100		1010	Gravel
					Subtotal	**4560**	

The existing drainage system doesn’t meet the city’s storm drainage requirement. This is mainly due to the reason that the existing & still the recently constructed urban storm drainage systems has laid with little or with no detailed hydrological and hydraulic investigations. Apart from this there is no well-organized regular or routine clearing and maintenance operation plugged along with the existing storm drain system. These conditions necessitate the idea of making detailed hydrological and hydraulic studies on the area of storm drainage management affaires in relation to the road development program of the city.

Apart from these problem as there is no interim drainage system laid at feeder roads and at other appropriate drainage areas, there is a strong concentrated flow directing towards the main road leading to the bridge site from right side existing road pavement and adjacent catchment area s there by creating a considerable depth of water on the road surface specially in the low lying areas during rainy seasons. This results in street damages and economic losses by enhancing traffic interruption and troubles for the city life. Quite a lot of these problems are intensified with the increase of the population and traffic load on the area.

An important cause of flooding is the insufficient capacity of current drainage systems. The combination of the higher rates of storm water and increasing illegal connection of sewerage system conveyed result in a high-risk of overloading the capacity of the system (EEA, 2001). Consequently, overburden networks are exposed to the emergence of surcharge that may cause flooding.

Figure 3.0.3Flooding and silted drainage nodes

3.2 Necessary Data

3.2.1 Rainfall Data

The rainfall data was gathered from the National Metrological Agency. At the study area there is one station, this station is suitable for the study. The data includes the record of daily rainfall precipitation throughout the year. These rainfall data are records of rainfall depth in 24h duration of 22 years of Shire station. The daily maximum rainfall of the year as follows which helps to develop the IDF curve.

Table 3-0-2Annual Daily Maximum Rainfalls (mm)

S.N	Year	Daily Max. Rainfall (mm)
1	1992	45
2	1993	49
3	1994	46
4	1995	63
5	1996	51.7
6	1997	50
7	1998	85
8	1999	60
9	2000	72.2
10	2001	49.9
11	2002	40.2
12	2003	48.9
13	2004	46.2
14	2005	64.1
15	2006	51.6
16	2007	56
17	2008	55.7
18	2009	46.5
19	2010	101.8
20	2011	45.6
21	2012	94.3
22	2013	73.2

3.2.1.1 Missing value estimation

Rainfall data having significant portion of the historic record missing, needs to be estimated. Accordingly, the historical daily rainfall data of each considered station was checked for its missing value of the considered record time. To estimate the missed rainfall values, the Inverse Distance Weighting method (Simanton and Osborn, 1980) which is the most commonly used for estimation of missing data was used. This method was chosen over the other methods because, it takes in to account the distance of neighboring stations to the target station (i.e. the closer the distance, the higher the similarity and vice versa). It is widely used and recommended by the United States Army Corps of Engineers (USACE, 2000). The formula for estimation of missing value of an observation, θm, using the observed values at other stations is given by:

$$Z_j = \frac{\sum_{i=1}^{r} Z_i(\,d_{ij})^{-n}}{\sum_{i=1}^{r} d_{ij}{}^{-n}} \qquad \text{Equation (3.1)}$$

where, Z_j is the observation at the base station j, r is the number of neighboring stations, Z_i is the observation at station i, d_{ij}, is the distance from the location of station i to station j, and n is referred to as the friction distance (Vieux, 2001) that ranges from (1–6), the common value 2 (Clark, 2000), was employed for this study.

3.2.1.2 Consistency analysis of the data set

The consistency of the data set of the given stations was cheeked by the double mass-curve method in-reference to their neighborhood stations. The double mass curve was plotted by using the cumulative annual rainfall of the base station as ordinate and the average cumulative annual rainfall of the neighboring stations as abscissa.

3.2.1.3 Homogeneity test

To determine the homogeneity and/or independency of the rainfall data series, the Mann-Kendall non-parametric trend test was used at 5% significance level (Mann, 1945 and Kendall, 1975). This test was chosen over other trend detection tests due to:

1) The Mann-Kendall test is a rank based non parametric test. When compare to parametric test like t-test the Mann-Kendall test has a higher power for non-normally distributed data which are frequently encountered in hydrological records like precipitation (Onoz and Bayazit, 2003; Yue and Pilon, 2004).
2) In comparison to other non-parametric tests, like Spearman's rho test, the power of the Mann-Kendall test is similar to the point where both give indistinguishable results in practice

3) The Mann Kendall test has been extensively used to determine trends in similar hydrologic studies done in the past (Hirsch et al, 1982; Lins and Slack, 1999; Burn et al, 2004; Abdul Aziz and Burn, 2006).

The Mann-Kendall statistic defined by variable S was computed which is the sum of the difference between the data points for a series (x1...xn) come from a population where the random variables are independent and identically distributed is given by:

$$S = \sum_{i=1}^{n-1} \sum_{j=i+1}^{n} sgn(X_j - X_i), \text{ where } sgn(X_j - X_i) = \begin{Bmatrix} +1(X_j - X_i) > 0 \\ 0\ if(X_j - X_i) = 0 \\ -1(X_j - X_i) < 0 \end{Bmatrix}$$

………… Equation (3.2)

Rainfall Variability

The knowledge of year to year rainfall variation is useful for planning and management of water resource. Variability of rainfall can be expressed by the coefficient of Variability *(CV)* which is defined as: the ratio of standard deviation to the mean in percent. The Value of this coefficient of variability provides year to year variation of rainfall on a given location According to Hare (1983), *CV* is used to classify the degree of variability of rainfall events as: Less variability, moderate variability and high variability those depends on the percent of variability. When *CV* < 20% it is less variable, *CV* from 20% to 30% is Moderately variable, and *CV* > 30% is highly variable. Areas with *CV* > 30% are said to be Vulnerable to drought.

3.2.2 Topography Data

The Digital Elevation Model is a digital topographic map, which contains the elevation of all the points located at the region. A 30 ×30 DEM data was obtained from Ethiopian Ministry of Water and Energy GIS Department, and the river basin extraction was operated in Arc GIS using the Arc Hydro extension. DEM preprocessing was executed through the following consecutive Arc Hydro process: Fill sink, flow accumulation, flow direction, stream definition, Stream Segmentation, Catchment Grid Delineation, Catchment Polygon processing, Drainage line processing, Adjoint Catchment processing, Longest flow path for Adjoint Catchment and Point Delineation.

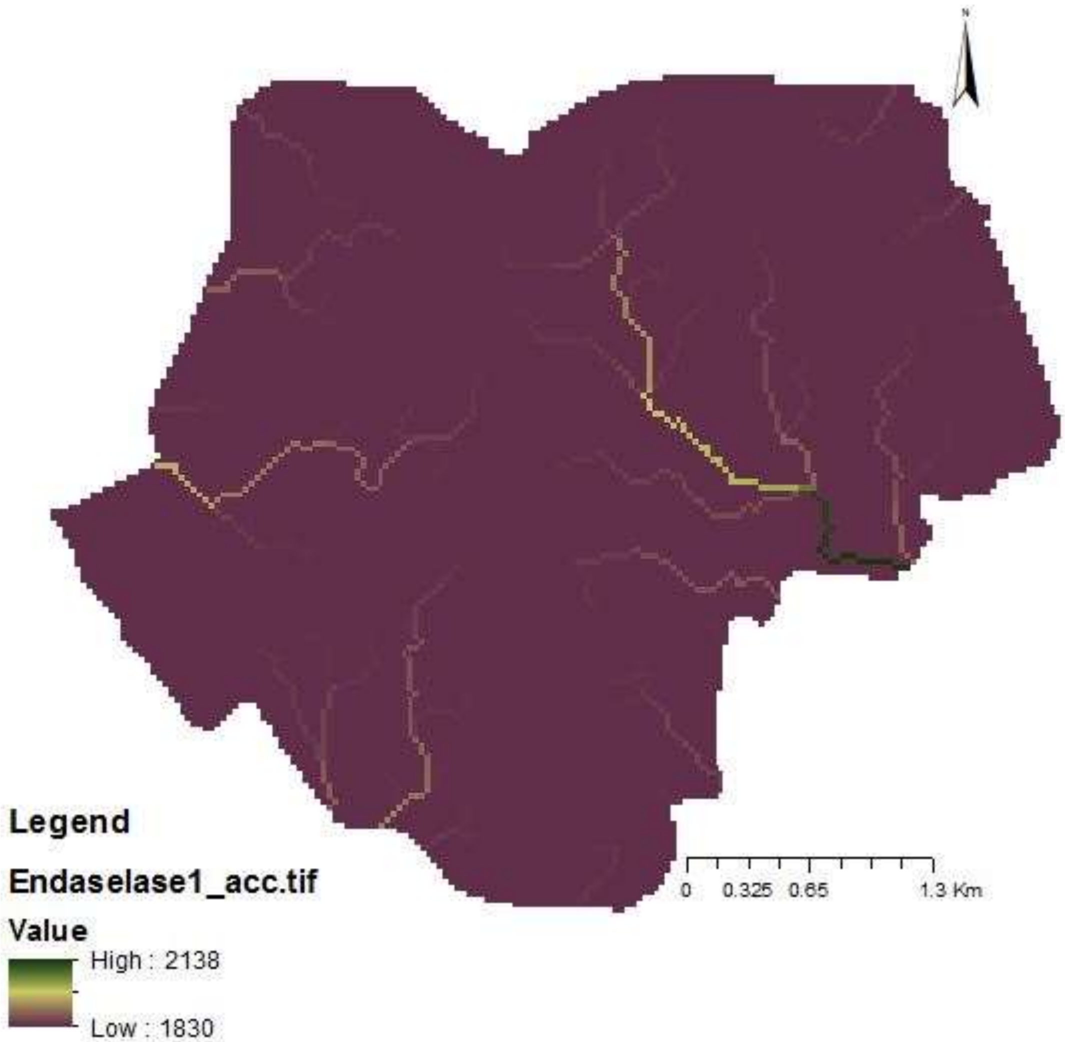


Figure 3-0.4 DEM 30 by 30 Resolution of Shire Endaslasse

3.2.3 Land Use Land Cover Data

Another type of data required is the land use land cover of the study area. The land use land cover data essential input for the calculation method runoff coefficient determination. The runoff coefficient of a surface is dependent on the land use and cover of the area. The map is obtained from Regional master plan office.

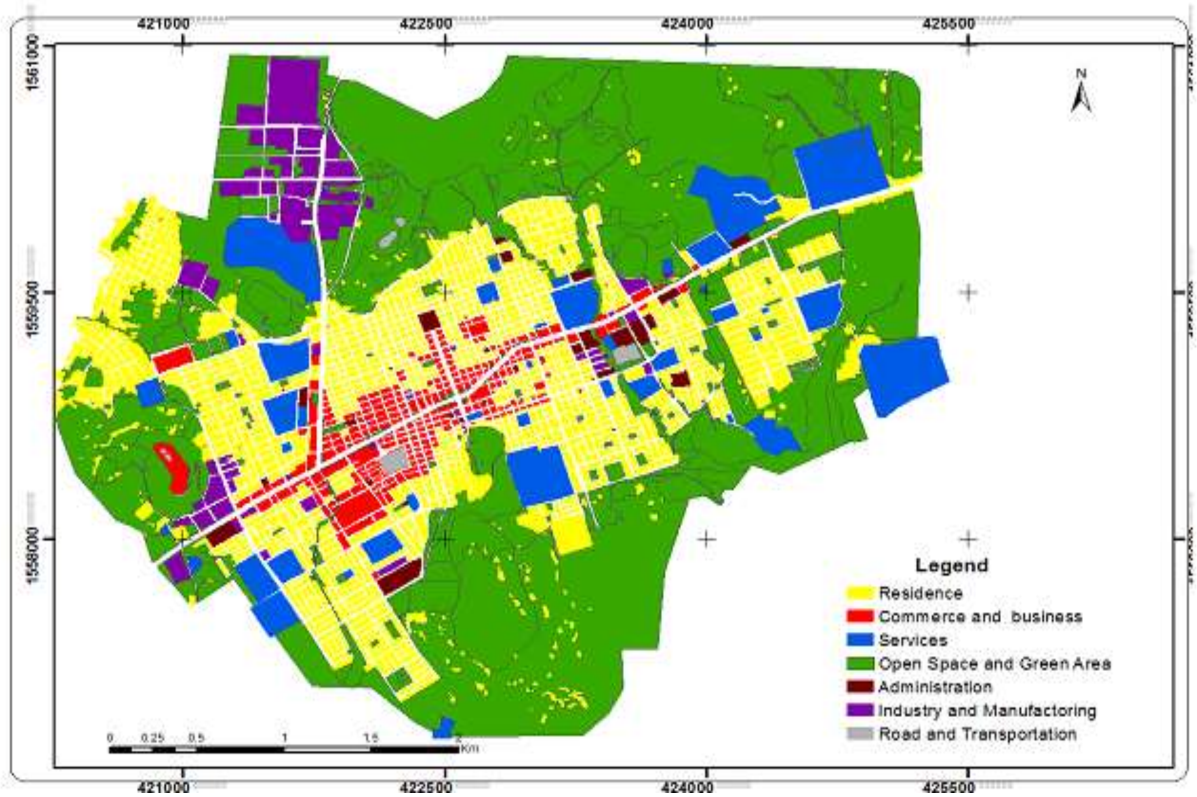


Figure 3-0.5 Land Use and Land cover map of Shire Endaslasse

3.2.4 Soil Type Data

The soil type of Shire Endaslasse is Chromic Cambsoil and Chromic Vertosoil. Their hydrological soil group represents B and D respectively according ERA manual 2013. Another type of data required is the soil type of the study area. It is an essential input for calculation of runoff coefficient. The runoff coefficient of a surface will vary with soil type of the area, due to variation of infiltration rate. Rather it is take in to consideration when proposing an integrated sustainable drainage system than urban areas.

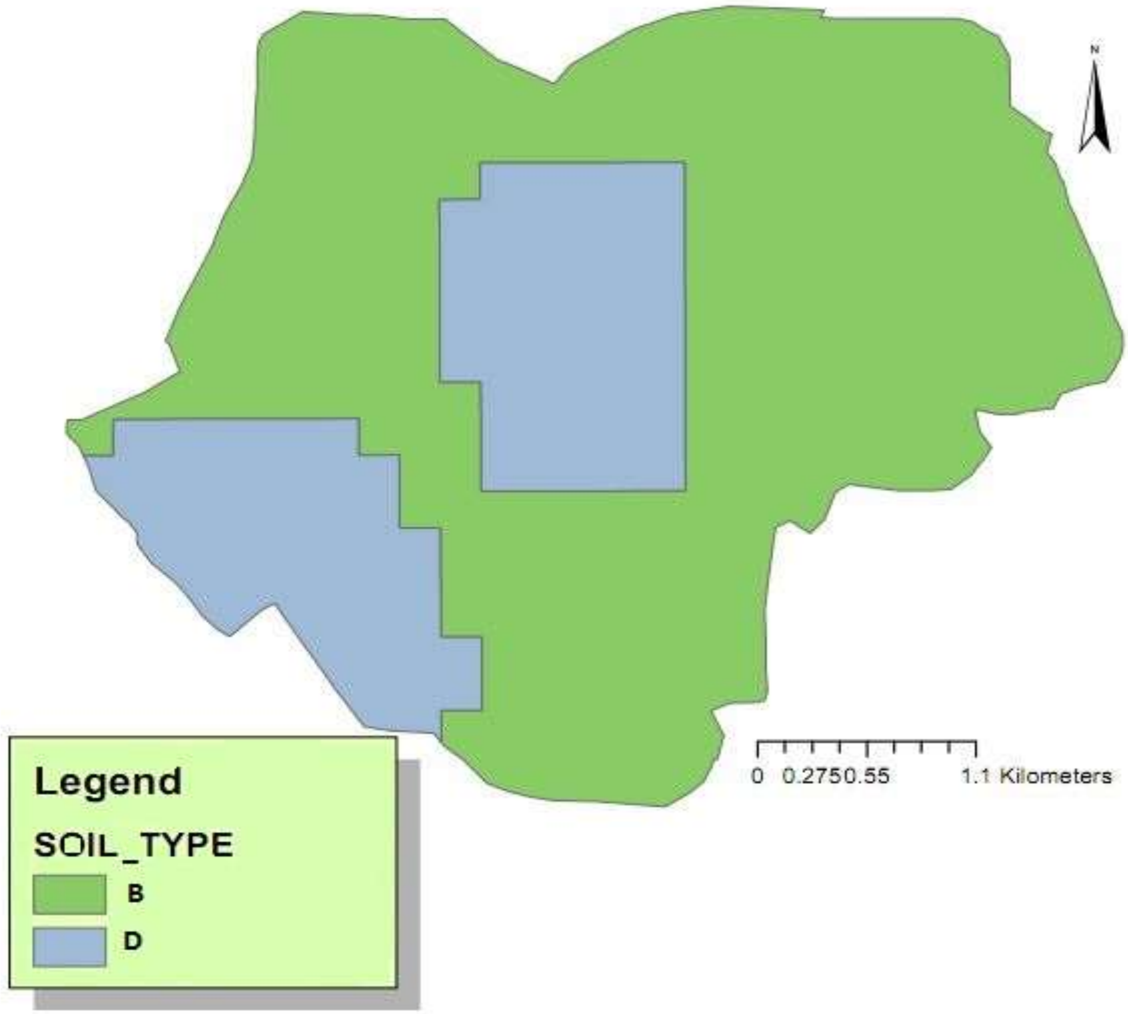


Figure 3-0.6 Soil Type of Shire Endaslasse (Source: Ethiopian Ministry of Water and Energy)

3.3 Hydraulic and Hydrological Modeling

3.3.1 The SCS Method

A relationship between accumulated rainfall and accumulated runoff was derived by SCS from experimental plots for numerous hydrologic and vegetative cover conditions. Data for land-treatment measures, such as contouring and terracing, from experimental catchment areas were included. The equation was developed mainly for small catchment areas for which daily rainfall and catchment area data are ordinarily available. SCS developed the method for small and midsized watersheds (< 1000 sq. km) (NRCS, 2004) the method also considered the initial rainfall losses to interception, depression storage and infiltration rate that decreases during the course of storm and Converts basin storage into something simpler and more manageable (a "curve number" CN). It was developed from recorded storm data that included total amount of rainfall in a calendar day but not its distribution with respect to time. The SCS runoff equation is therefore a method of estimating direct runoff from 24-hour or 1-day storm rainfall. The equation is:

$$Q = \frac{(P-Ia)2}{(P-Ia)+S} \qquad \text{..........Equation (3.3)}$$

Where:
Q = accumulated direct runoff, mm
P = accumulated rainfall (potential maximum runoff), mm
Ia = initial abstraction including surface storage, interception, and infiltration prior to runoff, mm
S = potential maximum retention, mm.

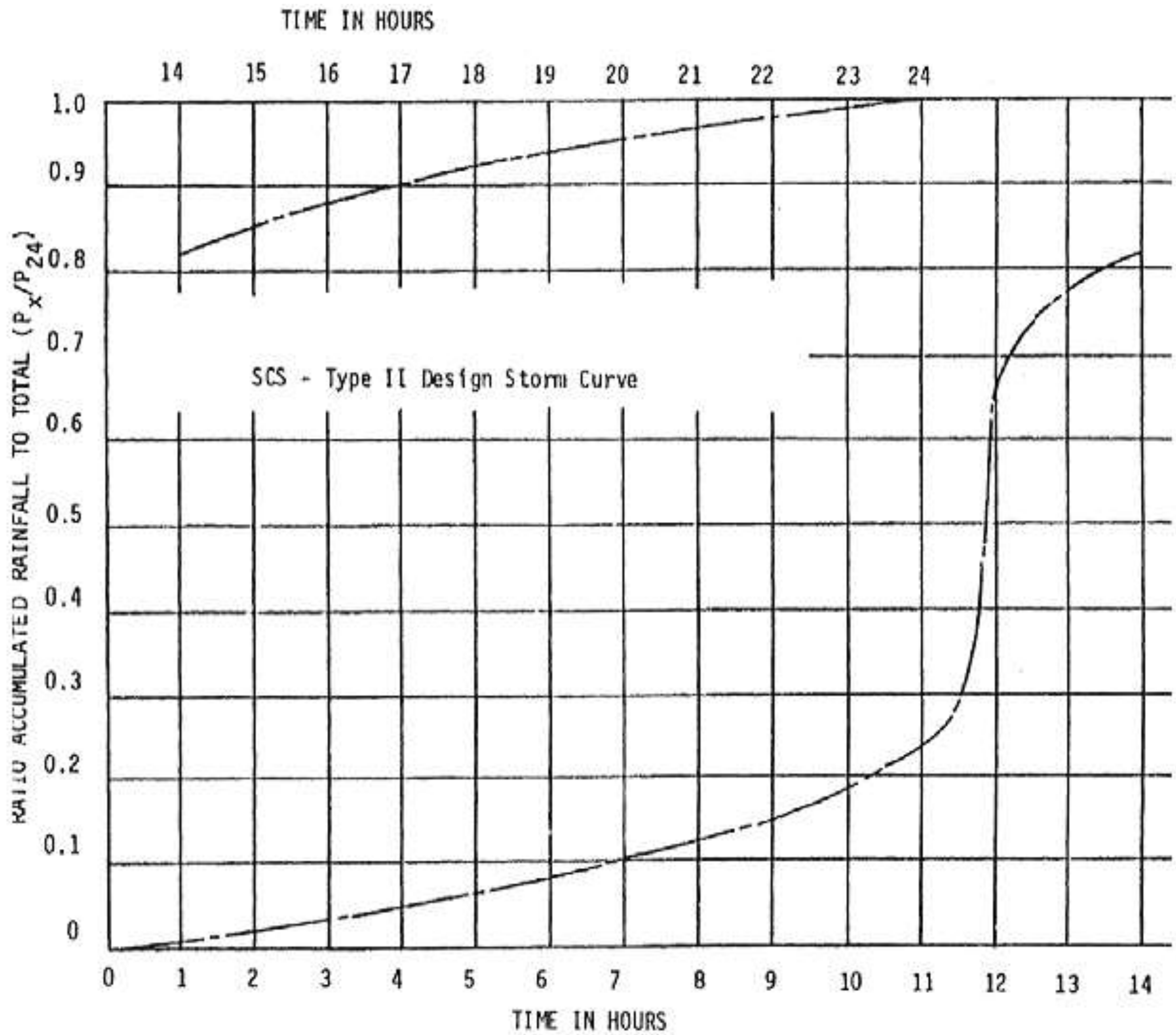


Figure 3-0.7 Type II Design storm Curve

The relationship between I_a and S was developed from experimental catchment area data. It removes the necessity for estimating I_a for common usage. The empirical relationship used in the SCS runoff equation is:

$$Ia = 0.2S \qquad \text{……………………………………………….Equation (3.4)}$$

Substituting 0.2S for I_a in the above equation the SCS rainfall-runoff equation becomes:

$$Q = \frac{(P-0.2S)2}{(P+0.8S)} \qquad \text{………………………………………………..Equation (3.5)}$$

S is the soil and cover conditions of the catchment area through the CN. CN has a range of 0 to 100, and S is related to CN by:

$$S = \frac{25400}{CN} - 254 \qquad \text{………………………………………….Equation (3.6)}$$

Ia/p Parameter

Ia/p is a parameter that is necessary to estimate peak discharge rates. Ia denotes the initial abstraction and p is the 24 hour rainfall depth for a selected return period. The 24 rainfall depth is taken from the frequency analysis result or from the ERA DDM rainfall region rainfall depth recommendations. For a given 24 hour rainfall distribution Ia/P represents the fraction of rainfall that must occur before runoff begins.

Peak Discharge Estimation

The following equation were used for the estimation of the peak discharge in SCS method

$$Qp = QuAQ \qquad \text{……………………………………………Equation (3.7)}$$

Where

q_p = peak discharge, m3/s

q_u = unit peak discharge, m3/s/km2/mm

A = drainage area, Km2

Q = depth of runoff, mm

The unit peak discharge is obtained from the following equation, which requires the time of concentration (tc) in hours and the initial abstraction rainfall (Ia/p) ration as input:

$$Qu = \propto 10^{Co + C1 \log tc + C2 \log tc} \qquad \text{…………………………………Equation (3.8)}$$

Where C0, C1 and C2 = regression coefficients given in table appendix for various Ia/p ratios: $\propto$ = unit conversion factor equal to 0.000431 in SI unit.

3.3.2 Rational Method

The Rational Formula is the most commonly used method of determining peak discharge from small drainage areas:

> In the design of stormwater drainage system, the main purpose of hydrologic analysis is to determine the maximum amount of run-off (peak discharge) that can be accumulated at certain storm drainage outlet (usually a ditch) along a highway/access road alignment section.
>
> The Rational method, one of the most commonly used simplified models for road storm drainage, is primarily based on the concept that the peak discharge from a watershed will always occur when the rain lasts long enough at its maximum intensity to enable all portions of the basin to contribute to the flow.

For this thesis Rational Method is appropriate for comparison with the model result; because of area for each catchment is less than 50 hectare (0.5sqkm) according to ERA manual 2013. The peak runoff is given by the following expression:

$$Q = \frac{C_f CIA}{360} \qquad \text{……Equation (3.9)}$$

Where: Q – Discharge at outlet (m3/s)

C_f – Climatic change factor (it is available for Design period > = 25 years)

C – Rainfall-Runoff Coefficient

I – Maximum probable rainfall Intensity (mm/hr)

A– Catchment Area (hectares)

Components of the Rational Formula

Area (A): The area draining to any point under consideration in a stormwater management system must be determined accurately. Drainage area information should include:

a. Land use: present and predicted future as it affects degree of protection to be provided and percentage of imperviousness.
b. Character of soil and ground cover as they may affect the runoff coefficient.
c. General magnitude of ground slopes which, with previous items above and shape drainage area, will affect the time of concentration. This includes information about individual lot grading and the flow pattern of runoff along a wales, streets and gutters.

The Runoff Coefficient (C): is expressed as a dimensionless decimal that represents the ratio of runoff to rainfall. Except for precipitation, which is accounted for in the formula by using the average rainfall intensity over some time period, all other portions of the hydrologic cycle are contained in the runoff coefficient. Therefore, C includes interception, infiltration, evaporation, depression storage, and groundwater flow. The variables needed to estimate C should include soil type, land use, degree of imperviousness, watershed slope, surface roughness, antecedent moisture condition, duration and intensity of rainfall, recurrence interval of the rainfall, interception and surface storage. The fewer of these variables used to estimate C, the less accurately the rational formula will reflect the actual hydrologic cycle.

Table 3-0-3Runoff coefficients for different land cover

Description of Area	Runoff Coefficients
Business: Downtown areas	0.70-0.95
Neighborhood areas	0.50-0.70
Residential: Single-family areas	0.30-0.50
Residential:Multi units, detached	0.40-0.60
Residential: Multi units, attached	0.60-0.75
Suburban	0.25-0.40
Residential (0.5 hectare lots or more)	0.30-0.45
Apartment dwelling areas	0.50-0.70
Industrial: Light areas	0.50-0.80
Industrial: Heavy areas	0.60-0.90
Parks, cemeteries	0.10-0.25
Playgrounds	0.20-0.40
Railroad yard areas	0.20-0.40
Unimproved areas	0.10-0.30

Source: ERA manual

Rainfall intensity (i): the determination of rainfall intensity, i, for use in the Rational Formula involves consideration of three factors:

a. Average frequency of occurrence
b. Intensity-duration characteristics for a selected rainfall frequency.
c. The rainfall intensity averaging time, Tc.

The critical storm duration that will produce the peak discharge of runoff is the duration equal to the rainfall intensity averaging time. The average frequency of rainfall occurrence used in the design of the stormwater system theoretically determines how often the structure will fail to serve the protective purpose for which it was designed.

The rainfall intensity average time (Tc) is usually referred to as the time of concentration. However, rainfall intensity averaging time more accurately defines the reason for and the use of this variable. Tc is not the total duration of a storm, but is a period of time within some total storm duration during which the maximum average rainfall intensity occurs.

Travel time (Tt) is the time it takes water to travel from one location to another in a watershed. The rainfall intensity averaging time (Tc) is computed by summing all the travel time for consecutive components of the storm water conveyance system. Several factors will affect the time of concentration and the travel time. These include:

SURFACE ROUGHNESS: one of the most important effects of urbanization on stormwater runoff is increased flow velocity. Undeveloped areas have very slow and shallow overland flow through vegetation which becomes modified by development. The flow is then delivered to streets, gutters and storm sewers that transport runoff downstream more rapidly due to the decreased resistance of the ground cover. Thus, reducing travel time through the watershed.

CHANNEL SHAPE AND FLOW PATTERNS: in small rural watersheds, much of the travel time results from overland flow in upstream areas. Typically, urbanization reduces overland flow lengths by conveying stormwater into a channel as soon as possible. Since channel designs have efficient hydraulic characteristics, runoff flow velocity increases and travel time decreases.

I. Sheet Flow Time

Sheet flow is flow over plan surfaces. It usually occurs in the headwater of the streams (usually for the first 100-130m run). With sheet flow, the friction value (Manning‟s roughness coefficient ,n) which take into account the effect of raindrop impact, drag over the plan and other ground cover barriers has a significant impact on the overall sheet flow travel time determination. Manning‟s kinematic solution (Overton and Meadows 1976) is used to compute sheet flow travel time.

$$Tt = \left[{0.091(nL)^{0.8}} \Big/ {(P_2)^{0.5} S^{0.4}} \right]$$ ....Equation (3.10)

Where: Tt = travel time, hr: n = Manning's roughness coefficient: L = flow length, m

P2 = 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)

$For\ Unpaved \quad V = 4.9178(S)^{0.5}$ ....Equation (3.11)

$For\ paved \quad V = 6.1961(S)^{0.5}$ ....Equation (3.12)

Where: V = average velocity, m/s

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

According to ERA, 2002 the above two equations are based on the solution of Manning's equation with different assumptions for n (Manning's roughness coefficient) and r (hydraulic radius, meters). For unpaved areas, n is 0.05 and r is 0.12; for paved areas, n is 0.025 and r is 0.06.

III. Open Channels Flow

Open channels are assumed to begin where surveyed cross section information has been obtained, where channels length width are obtained from the AutoCAD master plan of the study area. Average flow velocity is usually determined for bank-full elevation. Manning's equation or water surface profile information can be used to estimate average flow velocity. When the channel section and roughness coefficient (Manning's n) are available, then the velocity can be computed using the Manning Equation:

$V = \frac{1}{n}(R^{2/3} S^{1/2})$ ………………...Equation (3.13)

Where: V= average velocity, m/s

R= hydraulic radius, m (equal to a/pw)

A = cross sectional flow area, m2

Pw = wetted perimeter, m

S = slope of the hydraulic grade line, m/m

n = Manning‟s roughness coefficient

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

Tc = Tt1 + Tt2 + … Ttm ....Equation (3.14)

Where: Tc = time of concentration, hr

m = number of flow segments

Hydrologic Soil groups

Soils are classified by natural resources conservation service into four hydrologic soil groups based on the soil's runoff potential. The four hydrologic soil groups are A, B, C and D. whereas generally have the smallest runoff potential (A) and D's the greatest.

Details of this classification can be found in urban Hydrology for small water sheds published by the engineering Division of the natural resources Conservation Service, United States department of Agriculture, Technical release-55.

Group A is sand, loamy sand or sandy loam types of soils. It has low runoff potential and high infiltration rates even when thoroughly wetted. They consist chiefly of deep, well to excessively drained sands or gravels and have a high rate of water transmission.

Group B is silt loam or loam. It has a moderate infiltration when thoroughly wetted and consist chiefly or moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures.

Group C soils are sandy clay loam. They have low infiltration rates when thoroughly wetted and consist chiefly of soils with a layer that impedes downward movement of water and soils with moderately fine to fine structure.

Group D soils are clay loam, silt clay loam, sandy clay, silt clay or clay. This HSG has the highest runoff potential. They have very low infiltration rates when thoroughly wetted and consist chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a clay pan or clay layer at or near the surface and shallow soils over nearly impervious material.

Table 3-0-4Curve Number

Land Description	Hydrologic soil group			
	A	B	C	D
Cultivated land: with out conservation treatment	72	81	88	91
Cultivated land: with conservation treatment	62	71	78	81
Pasture or range land: poor condition	68	79	86	89
Pasture or range land: good condition	39	61	74	80
Meadow: good condition	30	58	71	78
Wood or forest land: thin stand, poor cover, no mulch	45	66	77	83
Wood or forest land: good cover	25	55	70	77
Open space, lawn, parks, golf courses, commentaries, etc				
Good condition: grass cover on 75 % or more of the area	39	61	74	80
Fair condition: grass cover on 50% to 75% of the area	49	69	79	84
Commercial and business area (85 % impervious)	89	92	94	95
Industrial districts (72 % impervious)	81	88	91	93
Residential				
Average lot size Average % impervious				
1/8 acre or less 65	77	85	90	92
1/4 acre 38	61	75	83	87
1/3 acre 30	57	72	81	86
½ acre 25	54	70	80	85
1 acre 20	51	68	79	84
Paved parking lots, roofs, driveways, etc	98	98	98	98
Streets and roads:				
Paved with kerbs and storm sewers	98	98	98	98
Gravel	76	85	89	91
Dirt	72	82	87	89

Although cities are in contact with water from various origins such as ground water, streams flowing through or near the city as the sea being receiving water, the major concern of urban drainage systems is water originating in the city itself, i.e. water from local rainfall and its interaction with the water originating from other water bodies (Maksimovic and prodanovic, 2001).

Rainfall infiltration losses depend primarily on soil characteristics and landuse (surface cover). The NRCS method uses a combination of soil conditions and land use to assign runoff CNs. Suggested runoff curve numbers are as follows.

For a watershed that has variability in land cover and soil type, a composite CN is calculated and weighted by area. The Value of curve number for different types f land cover as follows in table below.

3.3.3 Hydraulic and Hydrological Modeling by Using Rational Method

Modeling of hydrologic processes for estimating runoff generated from the catchment for the purpose of stormwater drainage design is discussed here. Stormwater runoff generated by a rainfall event depends on the catchment characteristics and rainfall characteristics. Design Requirements for urban stormwater drainage are usually specified in terms of a rainfall of certain return period.

3.3.4 Determination of short Duration Rainfall

For estimation of runoff especially for urban areas short duration rainfall are necessary. However especially in developing countries like Ethiopia availability of short duration rainfalls is scarce and data available is mostly for daily rainfall data. In such cases determination of design rainfall is becoming an approximation and thus leading to frequent failure of drainage network and subsequent floods. In the absence of short duration rainfall data, data is generated for short durations like 1hr,2hr,3hr,6hr and 12hr rainfall values were obtained using Ethiopian road authority manual empirical reduction formula is used in the absence of observed data (t-hour rainfall). Frequency analysis was then carried out to establish Intensity Duration Frequency (IDF) relationships.

$$Rt = \frac{t(b+24)^n}{24(b+t)^n} * R_{24} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{Equation (3.15)}$$

Where: Rt - rainfall at t, t-duration, b = 0.3 and n = 0.92 are constants. (Source: ERA manual)

Flooding in the cities and the town is a recent phenomenon caused by increasing incidence of heavy rainfall in a short period of time, inadequate capacity of drains and lack of maintenance of the drainage infrastructure. The annual disasters from urban flooding are now much greater than the annual economic losses due to other disasters. This demanding re consideration of design of drainage system which in turn requires intensity of rainfall calculated depending upon short duration of rainfall.

3.3.5 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 (Ven Te Chow, 1988). The rainfall data of study area are available, for determining of the design rainfall by using frequency analysis.

Rainfall frequency analysis derives functions that relate quintile's of rainfall depth with its recurrence intervals (return periods) from the maximum rainfall depth series at particular duration. Rainfall frequency analysis typically provides rainfall accumulation values at a point for a specified exceedence probability and various durations (Maidment, 1993). Rainfall frequency analysis is used extensively for design of engineering works that control storm runoff. These include municipal storm sewer systems, high way and railway culverts, and agricultural drainage systems. Rainfall frequency analysis also plays an important role in a diverse range of non-structural problems involving natural hazards associated with extreme rainfall events. (Maidment, 1993)

Using the Log Pearson type III distribution, the frequency factors, K_T, corresponding to the annual maximum rainfall magnitude were determined. Hazen plotting position was used in the computation for this frequency distribution. The calculated frequency factors, K_T of the stations are shown in Appendix. Plots were made between the values of maximum annual daily rainfall as ordinate and the corresponding frequency factor (K_T) as shown below.

3.3.6 Determination of Extreme Rainfall Values (XT)

The selected the log-Pearson type III probability distribution was used to calculate the extreme rainfall values (X_T) at different rainfall durations and return periods to form the historical IDF curves. The computed extreme rainfall values for the durations of interest; 5, 10, 15, 30, 60, 120 and 180 minutes and return periods of 2, 5, 10, 25, 50 and 100 years computed for station is presented.

The X_T values show that, X_T was directly related to both return period and duration of rainfall event; i.e. for each return period the values of extreme rainfall events increases with increasing the rainfall durations and for each duration, it increase with increasing of return period. The highest rainfall value was obtained for rainfall duration of 24hr and 100 years return period, whereas the lowest rainfall value was obtained for duration of 5 minute and 2 years of return period. In general, within certain rainfall duration, the values of calculated as follows.

Probability Plotting Position

Probability plotting position refers to the probability value assigned to each pieces of data to be plotted. The main purpose of probability frequency analysis is to obtain a relationship between the magnitude of the flood or storm and its probability of exceedence. The simplest empirical technique is to arrange the given annual peak event data in the descending order, and to assign the ranking number m to each event. The severest event is, thus, placed first with its rank as 1. The second severest event can be placed second position, its ranking number will be 2 and the least severity level is placed last at the end of the rank number. (Garg, 2005)

Numerous methods have been proposed for the determination of plotting positions. Weibull formula is used for Gumbel probability distribution function, Blom formula is used in lognormal probability distribution, and Hazen formula is used in log Pearson probability distribution function. According to Al-Weshah (2000) and Chow (1964), the choice of method depends on the acceptance of certain statistical principles and on the aim of the analysis to be undertaken.

3.3.7 IDF Curve Development

The IDF analysis starts by gathering time series records of different durations. After time series data is gathered, annual extremes are extracted from the record duration.

The annual extreme data is then fit to a probability distribution, in order to estimate rainfall quantities.

Rainfall intensity

Rainfall intensity (I) is defined as the ratio of the total amount of rainfall (R) (rainfall depth) falling during a given period to the duration of the period (D) and is usually expressed in depth per unit time, usually as mm per hour (mm/h). The non-recording type rain gauge records the rainfall depth only but in the case of recording type rain gauge, the duration and depth of rainfall are recorded simultaneously. Hence, it is calculated from the information on the recording type rain gauge chart (Bhattacharya, 2003).The average intensity can be expressed as:

$$\boldsymbol{I = \frac{R}{D}} \quad \text{……………………………………………………Equation (3.16)}$$

Where: R is rainfall (mm) and D is duration (hr.)

The rainfall data obtained over a long period in the past is required to construct rainfall intensity duration and frequency curves. The maximum mean rainfall intensity experienced during each storm can be determined for the given duration using the above equation.

Rainfall intensity is the inverse function of rainfall duration; that is longer rainfall duration, the rainfall intensity is less and vice versa. It is a general phenomenon that the rainfall intensity is not the same throughout the storm period but varies from time to time. (Suresh, 2005)

For accurate hydrologic analysis, 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.

Reliable rainfall intensity estimates are necessary for hydrologic analyses, planning and design problems. The rainfall intensity duration frequency (IDF) curve is one of the most common tool for urban drainage design and any water resource management structures. Information from IDF curves are used to describe the frequency of extreme rainfall events of various intensity and durations.

The typical establishment of rainfall IDF curves involves three steps, such as: extraction of maximum time series for several rainfall durations, fitting of a distribution in each maximum time series and finally estimation of the IDF curve parameters. The probability distribution function (PDF) or Cumulative Distribution Function (CDF) is fitted to each group comprised of the data value for any specific duration. The maximum rainfall intensity for each time interval is related with the corresponding return period from the cumulative distribution function. Probability of occurrences of maximum rainfall generated for any specific duration can be presented in the form of cumulative distribution function for different return periods. If the cumulative frequency is known, the maximum rainfall intensity can be determined using an appropriate theoretical distribution function (such as Generalized extreme value GEV, Gumbel and Pearson type III). In the presence of climate change, the theoretical distribution based on historical observations would be different for the future condition.

3.3.8 Parameter Estimation Method

Parameter estimation is an activity which requires fitting the IDF and defining the relationship of IDF frequency regime. There are two methods of parameter estimation suggested by Koutsoyiannis *et al.* (1998). These are the robust estimation and the one-step least squares methods based on optimization technique. The second method estimates all parameters (θ, η and ω) in one step, by minimizing the total square error of the fitted IDF relationship to the data. The minimization of the total error can be performed using the embedded solver tools of the MS-Excel spreadsheet by a trial and error procedure.

There are also various IDF Curve-Fitting software's developed to estimate the parameters based on the general IDF relationship. This software enables one to define and solve parameters (a, b, and c) from a set of pairs of data values of an intensity-duration equation of the general form (EXACT, 2006):

$$I = \frac{a}{[Td+b]c} \qquad \text{……………………………………………Equation (3.17)}$$

Where, I is rainfall intensity, mm/h, Td is time duration, min, '*a'* is coefficient with the same unit as I, b is time constant, minute and c is an exponent usually less than 1.

Catchment and sub catchment delineation

The catchment area to be drained is required to be defined based on the topography of the area. In some cases, there may be discharges into the catchment other than through gravity flow from areas outside the normal catchment. For example, there may be pumping of urban drainage flow from an adjacent catchment. The effective catchment area contributing to the catchment drainage flow includes all such contributory catchments.

Drainage network in a catchment whether pre-urban or urban catchment, consists of distribution of drainage paths (drains, streams) that converge to form the main drain/stream at the outlet of the catchment concerned. Urban catchments are usually modeled not as a single catchment but as collection of sub catchments. This practice has several advantages. Distributed properties in a heterogeneous urban catchment can be taken into consideration. In relation to the design of components of urban stormwater drainage system, the design flows are required not only at the outlet of the catchment but also at the outlets of all drainage paths converging to the main stream. Also, the effect of attenuation of peak flow within the drainage network of the catchment by routing the flows discharged into the main stream (and sub-mains) at different points with time lags provides realistic distributed discharges in the network. Overland flow routing may be not necessary if a subcatchment is very small.

Infiltration Analysis

The process of penetrating of water from the ground surface under gravity is called infiltration. Many factors influence the infiltration rate. It mainly depends on condition of soil surface, existing land cover, properties of soil beneath. Properties of soil include porosity, hydraulic conductivity and existing moisture content of the soil. These properties significantly vary with time and space, and therefore the infiltration process is a complex process to describe mathematically.

If a rainfall with a constant intensity begins on a dry soil, the rainfall intensity is less than the potential infiltration. As the rainfall continues, the wetting front penetrates deeper and deeper. Thus saturated soil depth increases and the potential infiltration rate reduce. Water will start to pond on the surface and overland flow is generated when the rainfall intensity is greater than the infiltration capacity of the soil.

The potential infiltration rate from a surface is the infiltration rate *(f)* if water is pounded on the Surface at a given time. The infiltration rate is less than the potential infiltration rate when the rate of supply of water on to the surface is less than the potential infiltration rate. The cumulative infiltration *(F)* is the accumulated depth of water infiltrated during a given time period is the integral of the infiltration rate over the period. Therefore, $f(t)=dF(t)/dt$. For all soils, the potential infiltration rate decreases as soil moisture content increases and approaches to a constant rate as more soil becomes saturated.

3.4 Hydraulic and Hydrological Modeling by Using Software Package

Software packages are available to simulate the behavior of a given system so that proper measures will be taken based on the simulation results. However proper care should be taken as simulation model by itself is not smart to solve the problem and the output of the model depends on many factors like how the modeller schematized the system within the model, the quality of input data used and on the calibration of the model using observed data.

The main objective of modelling in urban watershed is to understand how the system responds for a single or series of storm events specified by the modeller and take appropriate action accordingly (Akan, 1993). 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 modelling. 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.

Arch SWAT Software is also used to model and analyze flood especially for rural areas with a large catchments. SWMM is used in urban areas flood modelling and analyses to fix the drainage size by considering the pervious and impervious areas. So this thesis is flood modeling in urban areas and the SWMM software is more comfortable, because it is calibrated and validated software in our country Cities of Jimma and so on. SWMM was used, adapted and calibrated for the Ballona Creek Watershed, a catchment in Southern California.

3.4.1 Storm Water Management Model (SWMM)

This comprehensive model was developed by a consortium of American engineers for the US Environmental Protection Agency (EPA). SWMM takes the runoff and catchment characteristics, determines quantity and quality of runoff, routes the runoff through a combined or separate sewer system and identifies the effluent impaction receiving waters. Thus it is a mathematical model capable of representing urban storm runoff including sewage storage and treatment and combined sewer overflow phenomena. (Torno, 1975)

The computer package contains five blocks of subroutines.

1. Executive block contains service routine controlling the computational blocks and has a subroutine with the ability to combine data sets and separate out puts into a single routing, thereby enabling of large areas.

2. Runoff block computes the storm water runoff and pollution loadings for a given event for each sub-catchment and the results from the input into the relevant inlet to the main sewer system. Infiltration on previous areas (a loss to the urban system) is calculated by Horton's equation. Overland flow following the filling of depression storages and gutter flows are simulated using manning's equation and the continuity equation. The runoff competitions require hyetograph data, numerate characteristics for each subcatchment, Horton coefficients and pipe dimensions and roughness. Land use and other surfaces features and street frequency are needed for quantifying pollution.

3. Transport block contains the center core off the model with routines for routing flows through the main sewer system. It receives the calculated inputs from runoff, the dry weather flow assessed from population statistics, etc. and any infiltration into the sewers. The routing procedure uses a finite difference solution of the St. Venant equations. When a pipe is surcharged, the excess is stored at the upstream manhole. Flows may be routed a maximum of five outlet points. The particulars of sewer sizes, slopes, types of manholes and any internal control structure must be entered into the block.

4. Storage block contains modules to simulate the treatment of the sewage and receives the hydrograph and pollution sequences at one of the outlet points of the Transport block. This block can also calculate costs of selected treatment processes.

5. Receiving water block receives the output from Transport or Storage and computes the effect of the discharges on the quality of receiving water which are simulated by a network of nodes connected by a channels of specified constant surface and cross-sectional areas. Control weirs or tidal conditions can also modeled.

3.4.1.1 Parameter in SWMM

Subcatchment

SWMM required subdividing the catchment area into a number of sub-catchment. Each subcatchment is considered as an independent hydrologic unit with predefined topography and drainage system. The runoff can be routed through nodes in the drainage system, or to another subcatchment.

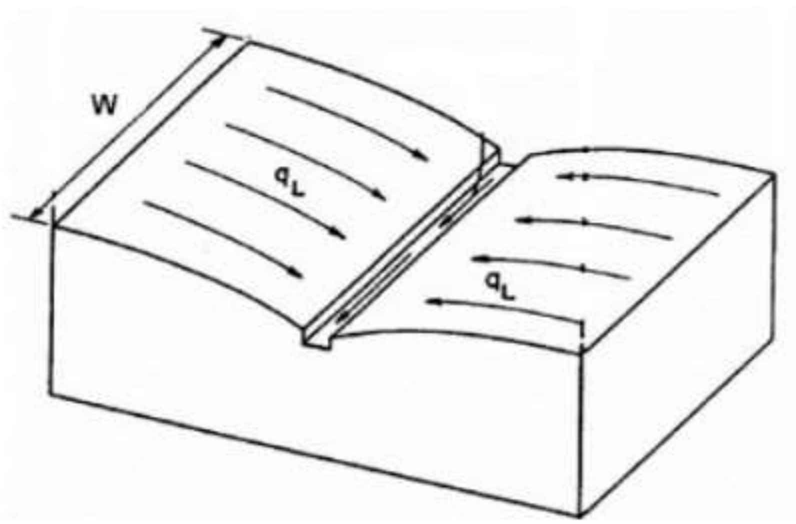


Figure 3-0.8Idealized Sub-catchment (Rossman, 2010)

The above figure shows how the overland flow can be estimated at SWMM, where the idealized subcatchment is illustrated as a rectangular and with a uniformed slope along the sub-catchment and the flow drains into a single outlet channel.

The features of each subcatchment must be also assigned (Rossman, 2010):

- ✓ Assigned Rain gage
- ✓ Subcatchment
- ✓ Assigned land use
- ✓ Imperviousness
- ✓ Slope
- ✓ Characteristic width of overland flow,
- ✓ Manning's for overland flow *n* both pervious and impervious areas

- ✓ Percentage of impervious area with no depression storage

***SLOPE*:** Using the GIS tools and the available online mapping software helps to localize the catchment area on the map, thus defining the location X and Y – coordinates as well as the average percentage slope (head loss per unit length). Defining the catchment slope is important to evaluate the natural drainage pathways (Rossman, 2010)

$$\%Slope = \frac{altitude\ 2 - altitude\ 1}{Length} * 100\% \quad \text{……………………Equation (3.18)}$$

***The imperviousness of the subcatchment*:** it is important to divide each subcatchment into pervious and impervious portions depends on the surfaces permeability. The pervious portion is the most permeable; means the runoff is subjected to losses to the unsaturated upper soil zone due to infiltration, but the impervious portion has lower infiltration capacity and can relatively produce 100% runoff after a rainfall event.

The Manning's roughness coefficient n : reflects the degree of resistance that the overland flow might meet while running on the surfaces, therefore it is important to be defined for impervious and pervious portion.

Depression storage: the depression capacity varies according to the surfaces, in another word the ability of the surfaces to fill up the pores before the runoff take place. Is about 1.2 mm for impervious surfaces and 7.62 mm for green areas (Rossman, 2010).

Subcatchment width: SWMM manual is defining the subcatchment width as:

$$Width = \frac{Subcatchment\ area}{Length\ of\ the\ longest\ overland\ flow\ path} \quad \text{………………………Equation (3.19)}$$

Where the overland flow length, is defined as the longest flow path that the water can approach (Rossman, 2010, p17) the maximum lengths from few possible pathways must be averaged and can be used to determine the subcatchment width. These pathways must reflect slow flow rather than rapid flow for attenuation purposes (Rossman, 2010).

The runoff infiltration in pervious portion can be modeled through Horton, Green-Ampt or Curve Number infiltration models. However, the snow accumulation, re-distribution and melting can be also modeled but first after Snow Pack object is assigned. Also one can model the groundwater flow but a set of Groundwater parameters must be assigned first. The Land Uses associated pollutants and wash-off from subcatchment can be also modeled as well as different types of LIDs can be modeled by assigning a set of predesigned LID controls to the subcatchment (Rossman, 2010).

3.4.1.2 Conduit

The runoff can be transported into conduits/ pipes or channels from one node to another in the drainage system, where the natural flow is considered. SWMM provides with a variety of open and closed pipes depend on cross-sectional shapes of each. (Rossman, 2010)

SWMM used the manning Equation to estimate the flow rate in each conduit, where the relationship between the runoff flow-rate (Q), crass sectional area (A), hydraulic radius and slope (S) are expressed in the following equation. Hence, the conduit roughness coefficient is variable value and depends on the material of the conduit. The Manning's roughness coefficient (n) expressed the conduit's roughness.

$$Q = \frac{1}{n} * A * R^{2/3} * S^{1/2} \quad \text{.............................Equation (3.20)}$$

where: Q- Flow rate (m^3/s)

A-Cross-sectional area of the conduit

R- Hydraulics radius

S- Slope of the conduit

n – Manning's roughness coefficient

in order to enable SWMM to follow the flow rate in conduits, there is few other input parameters must be predefined.

- ✓ Name of the inlet and outlet node
- ✓ Offset height or elevation above the inlet and outlet node inverts
- ✓ Conduit length
- ✓ Manning's roughness
- ✓ Cross-sectional geometry

3.4.2 Calibration and Validation of model

An important part of any modeling exercise is the model calibration. Calibration is a process where in certain parameters of the model are altered in a systematic fashion and the model is repeatedly run until the simulated results match field observed values within an acceptable level of accuracy. The process of model calibration is quite complex and limited by the model itself, input, and output data. Imperfect knowledge of watershed characteristics, mathematical structures of the hydrological processes and model limitations can cause error in calibration process. Before starting model calibration, field conditions at the site should be properly characterized. Lack of proper site characterization may lead to a wrong representation of the simulated system. Model calibration can be performed either by trial and error or by automated techniques. Automated calibration can be performed by means of specifying an objective or a set of objective functions (Schaake, 2003). Uncertainty in models and data leads to uncertainty in model parameters and model predictions. Automated parameter estimation techniques for model calibration are accurate and rapid. Validation of hydrologic models is a process of matching the simulated results with observed values without altering the calibrated parameters.

3.4.3 Evaluation of Model Performance

The performance and behavior evaluation of hydrological models is commonly made through comparison of different efficiency criteria. To achieve adequate reliability of the simulation models, it is important that they are rigorously calibrated and validated before any analysis and/or management scenario analysis are conducted. It is highly recommended to do the sensitivity analysis of model parameters before starting the calibration process. Model calibration and evaluation efforts are performed to achieve a reasonable correspondence between measured field data and the output of the model. The model could be evaluated in order to determine the performance that how the model, simulated value fitted with the observed value. Statistics techniques like the coefficient of determination is one of the method to assess the model performance and also estimate that at which level simulate value fitted with the observed value. It shows the best fitness and efficiency of the model. R square describes the proportion of the total variance in the measured data that can be explained by the model. It ranges from 0.0 to 1.0. High values indicating better agreement. The goodness-of-fit measures used were the coefficient of determination (R^2) and the Nash-Sutcliffe efficiency (ENS) value. The Nash and Sutcliffe simulation efficiency indicates the degree of fitness of the observed and simulated plots (Santhi et al.2002).

$$E_{NS} = 1 - \frac{\sum_{i=1}^{n}(q_{oi}-q_{si})^2}{\sum_{i=1}^{n}(q_{oi}-q_{oAv})^2} \quad \text{………………………………Equation (3.21)}$$

Where: q_o - observed discharge

q_{si} – simulated discharge

q_{oav} - observed average discharge

q_{siav} – simulated average discharge

$$R^2 = \frac{[\sum_{i=1}^{n}(q_{si}-q_{siAv})\sum_{i=1}^{n}(q_{oi}-q_{oiAv})]^2}{\sum_{i=1}^{n}(q_{si}-q_{siAv})^2\sum_{i=1}^{n}(q_{oi}-q_{oiAv})^2} \quad \text{…………………… Equation (3.22)}$$

The regression coefficient (R2), and the Nash-Suttcliffe (1970) simulation efficiency (ENS) was checked in accordance to Santhi et al. (2001) recommendation (R^2 >0.6 and ENS> 0.5).The Nash-Sutcliffe coefficient (ENS) was used to evaluate the measure of efficiency that relates the goodness-of- fit of the model to the variance of measured data.

4. Result and Discussion

4.1 Consistency and Homogeneity of the Rainfall Data Series

The double mass curve which is the plot of the cumulative annual rainfall data of the base station with the cumulative average annual rainfall data of neighborhood stations of the stations considered are presented below. Accordingly, the graph of the double mass curve plot was found to be almost linear for the considered station with a coefficient of determination (R2) 0.9961. This implies that the rainfall data was consistent over the considered period of time.

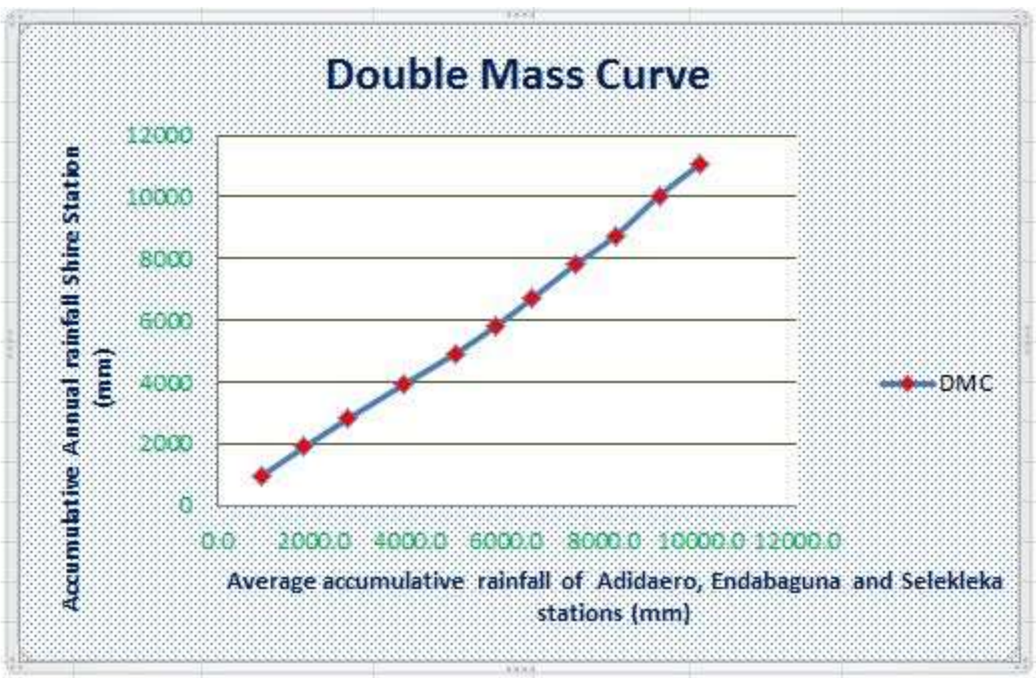


Figure 4.0.1 Double Mass Curve

To determine the homogeneity of the rainfall data series of each station, a trend analysis was made by a non-parametric Mann-Kendall rank test method at 5% significance. The basic statistics associated with this test are summarized in Table 2 for all the stations considered.

Summery statistics of the Mann-Kendall trend test for the stations considered

Parameter	Shire	Selekleka	Endabaguna	Adidaero
Kendall's tau	0.0182	-0.3455	0.0182	-0.1636
S	1.0000	-19.0000	1.0000	-9.0000
p-value (Two-tailed)	1.0000	0.1646	1.0000	0.5423
alpha	0.05	0.05	0.05	0.05

Based on the hypothesis made, the computed p-values for stations of Shire, Selekleka, Endabaguna, and Adidaero were found to be greater than the significance level (alpha = 0.05), thus one can detect that there is no trend in the rainfall data series of these stations. This depicts that the data is homogenous, independent and identically distributed.

Using the daily maximum rainfall from metrological agency, 24 hour design rainfall was calculated using Log Pearson type III distribution methods, because it is the greater R^2 value than Gumbel Distribution. The parameter of IDF determines by using Miduss Software. The values are compared with Ethiopian Roads Authority recommended values and the maximum was taken, as it is recommended by ERA. The rainfall of ERA is attached on the appendix part.

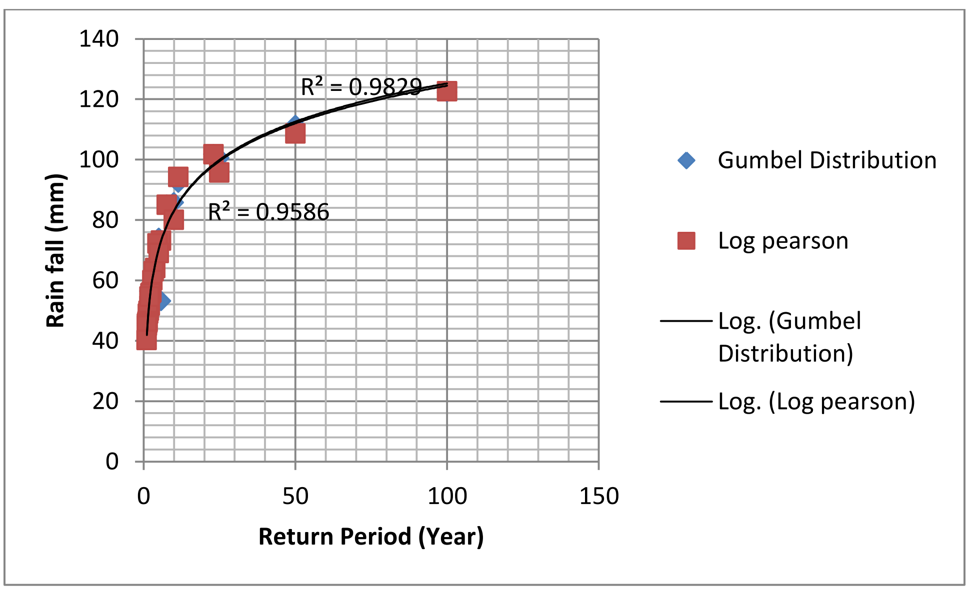


Figure 4.0.2 Frequency Distribution

Table 4-0-1Rain fall in mm corresponding to Return Period

Time Duration	Return Periods (Tr) and Corresponding Rain fall in mm					
	2	5	10	25	50	100
5 min	8.64	10.91	12.65	15.12	17.15	19.37
10 min	14.41	18.21	21.11	25.23	28.63	32.33
15 min	18.58	23.49	27.22	32.53	36.92	41.69
30 min	26.33	33.28	38.57	46.10	52.31	59.07
1 hr	33.69	42.58	49.35	58.98	66.93	75.58
2 hr	39.86	50.38	58.40	69.79	79.19	89.43
3 hr	42.90	54.21	62.84	75.10	85.22	96.24
6 hr	47.33	59.81	69.33	82.85	94.02	106.17
12hr	51.15	64.64	74.92	89.54	101.61	114.74
24 hr	54.68	69.10	80.09	95.72	108.62	122.66

In this thesis takes the MIDUSS 2 software application for the determination of IDF parameters with minimum errors. The value of parameters of the study area as follows in table below for different return periods and duration.

Table 4-0-2Coefficient of Intensity develops

Coeff.	Return periods					
	2 yr	5 yr	10 yr	25 yr	50 yr	100 yr
A	1854.62	2338.74	2716.58	3246.93	3637.86	4160.03
B	18	17.966	18	18	17.785	18
C	0.92	0.9197	0.92	0.92	0.918	0.92

4.2 IDF Curve of Shire Endaslasse

The IDF curve develops for Shire Indaselassie by using 22 years (From 1992-2013 G.C) of annual Daily maximum rainfall as shown below.

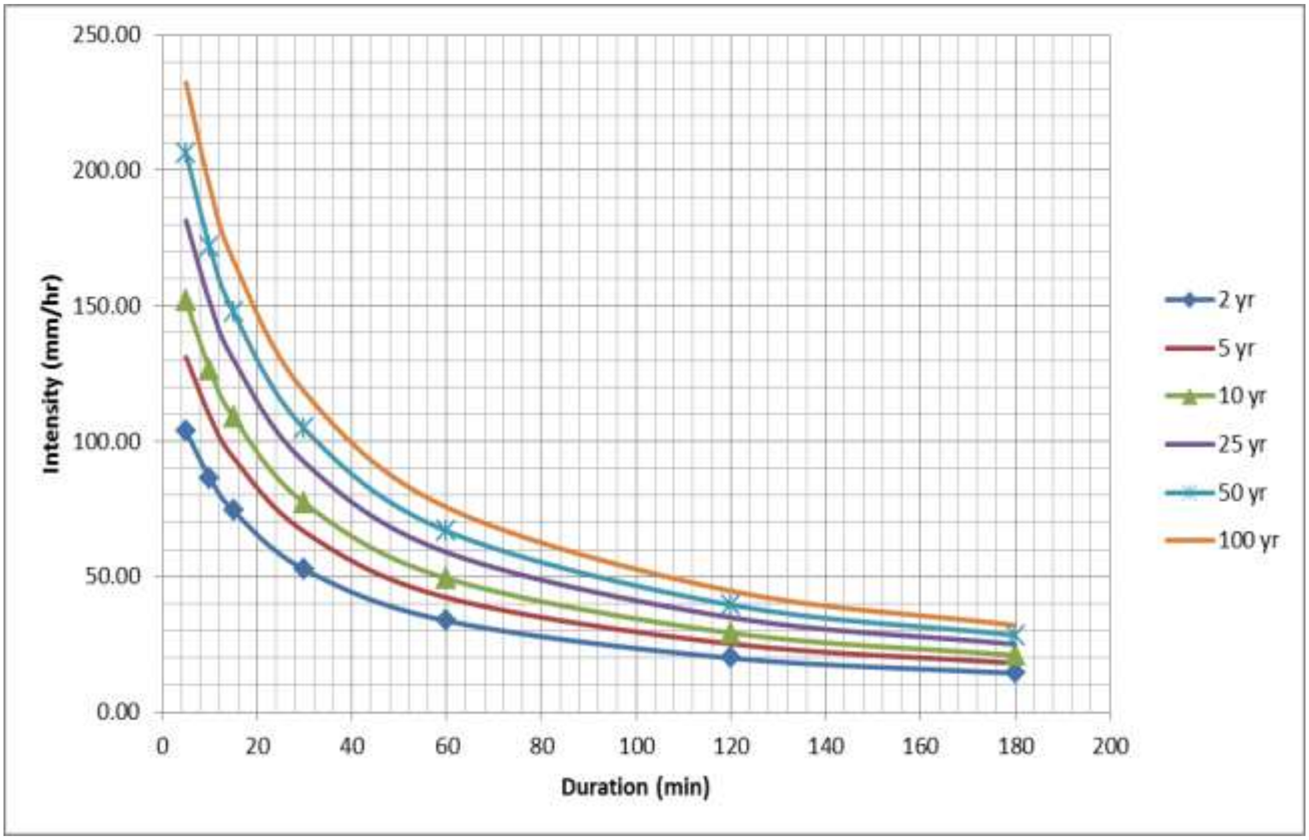


Figure 4.0.3 IDF Curve of Shire Endaslasse

Suitable equation for the IDF curve of Shire Endaslasse rain gauge station for 2 yr., 5 yr., 10 yr.,25 yr., 50 yr., and 100 yr Return Periods as follows below in table.

Table 4-0-3IDF curve equation for different return periods

Tr	Equation	R2
2	y=-25.85ln(x)+144	0.9972
5	y=-32.7ln(x)+182.03	0.9976
10	y=-37.87ln(x)+210.92	0.9976
25	y=-45.261ln(x)+252.09	0.9976
50	y=-51.47ln(x)+286.49	0.9977
100	y=-57.99ln(x)+323.01	0.9976

The depth duration curve for return periods of 2 yr., 5 yr., 10 yr.,25 yr., 50 yr., and 100 yr. as follows below in figure.

4.3 Runoff Determination by Using Rational Method

For this thesis Rational Method is appropriate for comparison with the model result; because of area for each catchment is less than 50 hectare (0.5sqkm) according to ERA manual 2013. By Using Equation (3.7) the peak discharge for each catchment determined.

The main purpose of determination of runoff using rational method helps for comparison with the model result of SWMM model runoff values.

Catchment area delineation by depending field survey on the study area and AutoCAD master plan determined the catchment area values. The runoff coefficient determined from characteristics of different land and land cover obtained from Table 3.3. Time of concentration determined using equation (3.12) as elaborates in the methodology. For the corresponding time of concentration IDF determined 10 years of design period. Finally using Equation (3.7) peak flow calculated.

Since the rational method equation and their characteristics have explained in the methodology part. Runoff coefficient Determination for the different catchment characteristics as follows below in table (4.5).

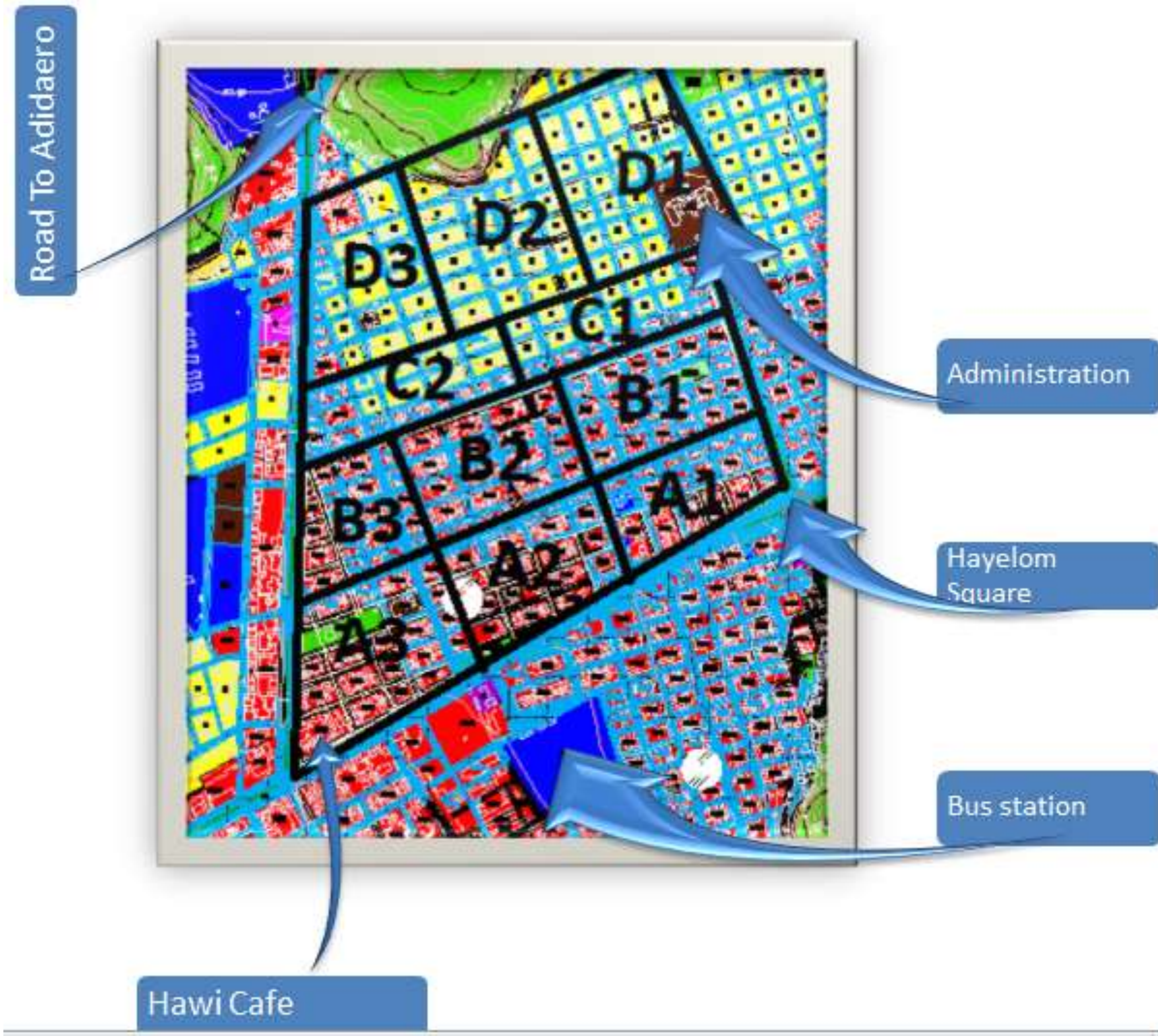


Figure 4.0.4The study area

Table 4-0-4Runoff coefficient values for the catchment part 1

Catch	Sub.catch	Area Sub	Land use	Area (m2)	C (Run off Coeff.	Area Prop	Cw
Hayelom Square To Hawi Café (A)	A1	26880	Multi-Family	17000	0.45	0.28	0.54
			Green Area		0.3	0.00	
			Brick	420	0.75	0.01	
			Cobble	7360	0.65	0.18	
			Asphalt	2100	0.8	0.06	
	A2	51247.5	Multi-Family	34861.5	0.45	0.31	0.52
			Green Area	896	0.3	0.01	
			Brick	650	0.75	0.01	
			Cobble	11590	0.65	0.15	
			Asphalt	3250	0.8	0.05	
	A3	61490.25	Multi-Family	32553.48	0.45	0.24	0.53
			Green Area	4730.77	0.3	0.02	
			Brick	1126	0.75	0.01	
			Cobble	17450	0.65	0.18	
			Asphalt	5630	0.8	0.07	
Road to Adminstration To Midregenet Hotel (B)	B1	42853.5	Multi-Family	21357	0.45	0.22	0.55
			Green Area	1696	0.3	0.01	
			Brick	366	0.75	0.01	
			Cobble	16562	0.65	0.25	
			Asphalt	2872.5	0.8	0.05	
	B2	63112	Multi-Family	37502	0.45	0.27	0.51
			Green Area		0.3	0.00	
			Brick		0.75	0.00	
			Cobble	25610	0.65	0.26	
			Asphalt		0.8	0.00	
	B3	40321.5	Multi-Family	21913.5	0.45	0.24	0.33
			Green Area		0.3	0.00	
			Brick	399	0.75	0.01	
			Cobble	16014	0.65	0.26	
			Asphalt	1995	0.8	0.04	

Table 4-0-5Runoff coefficient values for the catchment part 2

Catch	Sub.catch	Area Sub	Land use	Area (m2)	C (Run off	Area Prop.*C	Cw
Road to Adminstration To Adis Ababa Hotel	C1	46138	Multi-Family	28135	0.45	0.27	0.54
			Green Area		0.3	0.00	
			Brick	782	0.75	0.01	
			Cobble	15266	0.65	0.22	
			Asphalt	1955	0.8	0.03	
	C2	31898	Multi-Family	20071.5	0.5	0.31	0.56
			Green Area		0.3	0.00	
			Brick	499	0.75	0.01	
			Cobble	10080	0.65	0.21	
			Asphalt	1247.5	0.8	0.03	
Adminstration to Ethiopia Hotel (D)	D1	111290	Multi-Family	75500	0.45	0.31	0.52
			Green Area		0.3	0.00	
			Brick	620	0.75	0.00	
			Cobble	33620	0.65	0.20	
			Asphalt	1550	0.8	0.01	
	D2	68019.5	Multi-Family	41348.5	0.45	0.27	0.54
			Green Area		0.3	0.00	
			Brick	1176	0.75	0.01	
			Cobble	21480	0.65	0.21	
			Asphalt	4015	0.8	0.05	
	D3	18636	Multi-Family	12326	0.45	0.30	0.54
			Green Area		0.3	0.00	
			Brick	400	0.75	0.02	
			Cobble	3910	0.65	0.14	
			Asphalt	2000	0.8	0.09	

Runoff values for each catchment and time of concentration with respect the Intensity. The governing value of time concentration for the catchment is minimum 15min. The calculated value of time of concentration which satisfies the limitation then with the respect this value Read from IDF curve for 10 yrs. return period the Intensity (mm/hr.).

Table 4-0-6Hayelom Square to Hawi Café

Sub Catch	Area (ha)	Tc (min)	IDF(Tr = 10 yr)	Cw	Q (CMS)
Catch-A1	2.688	18.51	99.68	0.537	0.4
Catch-A2	5.125	28.22	83.70	0.519	0.62
Catch-A3	6.149	22.43	92.46	0.533	0.85

Table 4-0-7Road to Administration to Mdregenet Hotel

Sub Catch	Area (ha)	Tc (min)	IDF(Tr = 10 yr)	Cw	Q (CMS)
Catch-B1	4.285	20.73	96.11	0.547	0.63
Catch-B2	6.311	28.22	84.44	0.531	0.79
Catch-B3	4.32	22.43	93.13	0.55	0.573

Table 4-0-8Road to Administration to Addis Ababa hotel

Sub Catch	Area (ha)	Tc (min)	IDF(Tr = 10 yr)	Cw	Q (CMS)
Catch-C1	4.614	23.42	91.49	0.536	0.63
Catch-C2	3.19	24.85	89.24	0.563	0.445

Table 4-0-9Administration to Ethiopia hotel

Sub Catch	Area (ha)	Tc (min)	IDF(Tr = 1(	Cw	Q (CMS)
Catch-D1	11.128	25.43	88.37	0.517	1.41
Catch-D2	6.802	26.87	86.29	0.539	0.878
Catch-D3	1.864	26.87	86.29	0.536	0.239

4.4 Runoff Determination Using SWMM

This model helps to determine the runoff, the network simulation of the links (conduit) and which indicates the depth of flow through conduit.

The input data for this model is the invert elevation of the nodes, the maximum depth of flow for the conduit, length of links, catchment area, N-Impervious and N-Pervious. For this research the depth of the conduit takes the depth of the existing depth of conduit in the study area.

The total Catchment area is 56.478 ha which divides in to 11 sub catchments. The number of node 131, tow outfalls and 131 links. According to Rossman (2010), This Model uses the Continuous rainfall data or the IDF for corresponding time of concentration of sub-catchments. The time of concentration varies from 18.87min- 28.77min for all catchments. The Intensity rainfall for the respected values of Time of concentration which varies 99.7mm/hr. – 83.7mm/hr. for the 10yr design period.

Pervious and impervious percentage determined from the land use and land cover characteristics. Rossman (2010) recommended the percentage of Imperviousness for Asphalt 100%, for residential area 50% and for forests and green areas 2%.

The depression Storage for different types of Land covers there is in table form in the appendix part. The manning roughness for conduit and over land flow there is in the appendix.

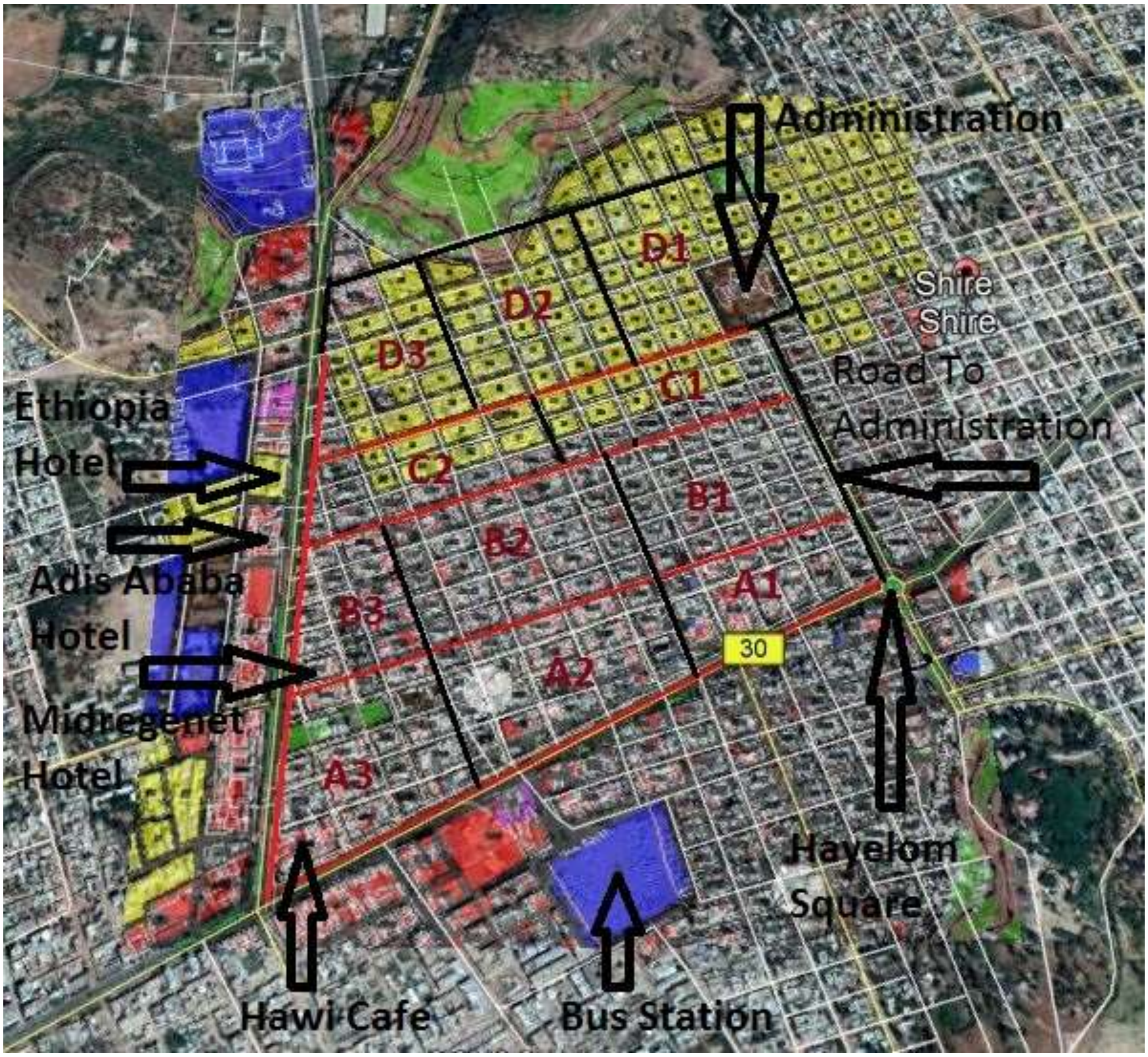


Figure 4.0.5 The image over lay AutoCAD design map with the Google earth map Shire

The Detail study area map as follows below in figure. This map shows the depth of flow in the links, depth of flow for Nodes at each individual segment, and the direction of flow.

Detail Study area map Part 1

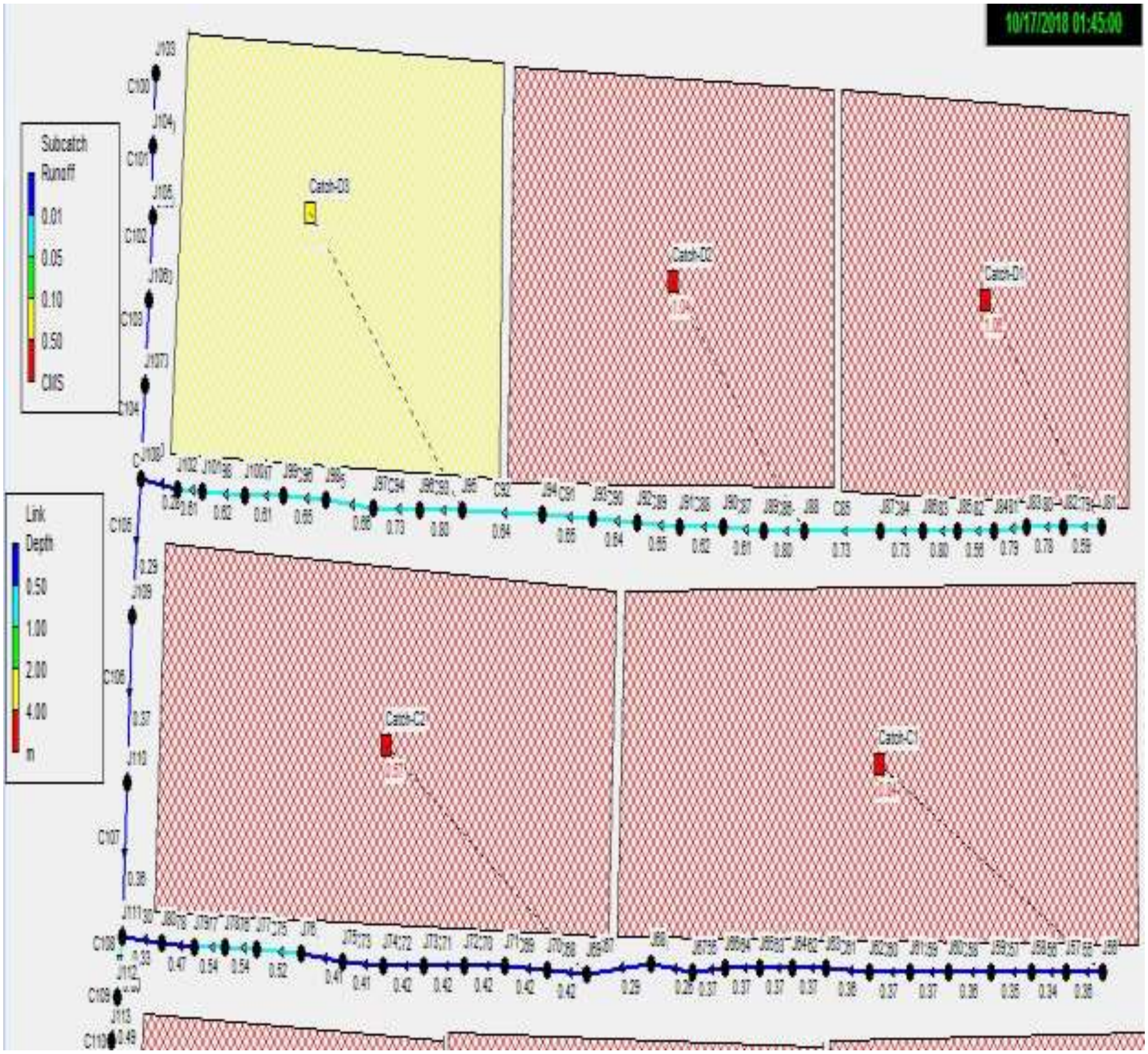


Figure 4.0.6Detail Study area map part 1

Detail Study area map Part 2

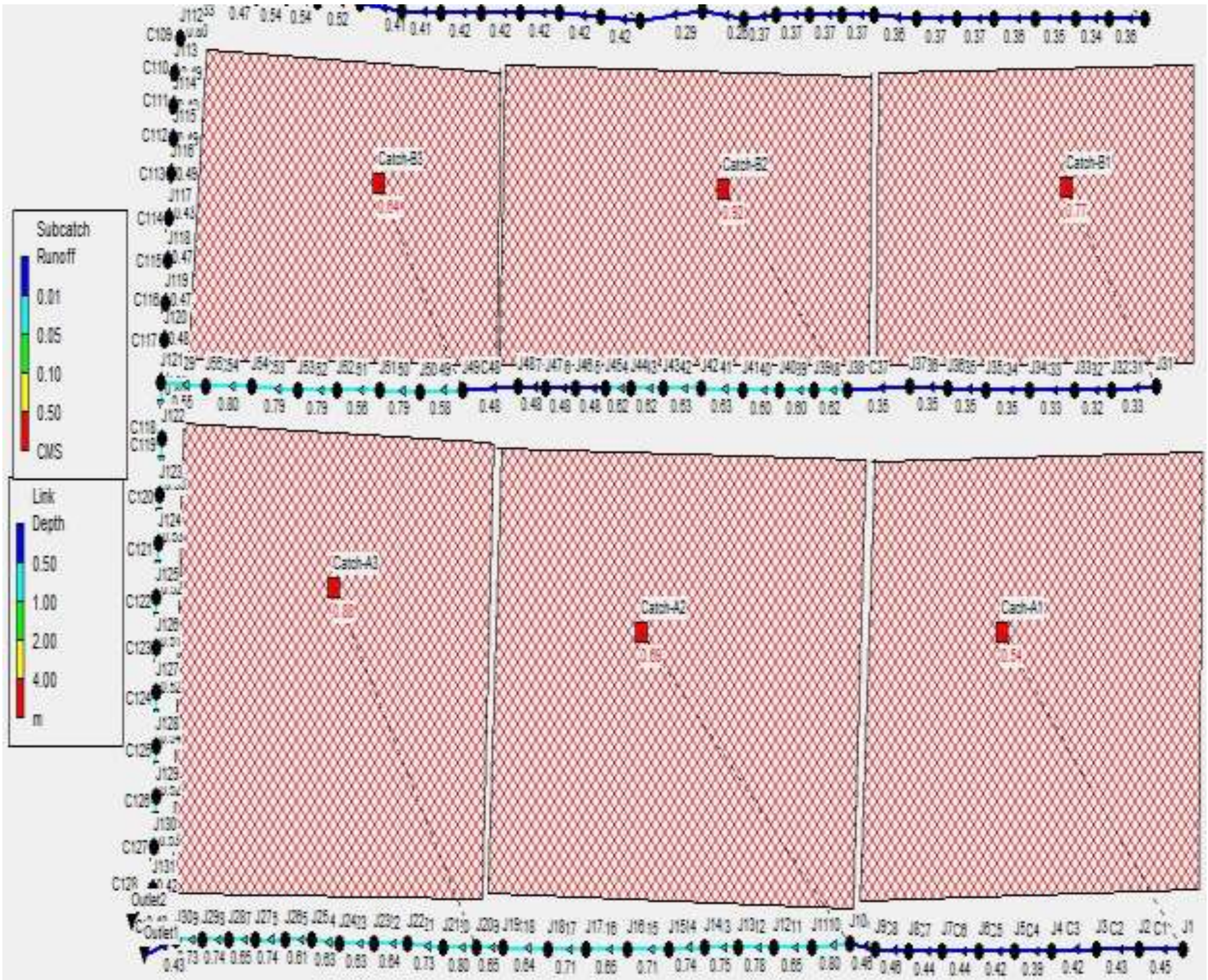


Figure 4.0.7 Detail Study area map part 2

4.4.1 Runoff results of SWMM Model

This model needs continues rainfall data or can take the maximum hour precipitation (Intensity) for the design period computed takes as input precipitation according to EPA SWMM manual. For the 10 yrs. return period the Intensity as input with to Time of concentration of each Sub catchment . The maximum flow rate exists in each catchment at critical moment; as follows in table below.

Table 4-0-10Sub-catchment Peak Runoff

Summary Results

Subcatchment Runoff — Click a column header to sort the column.

Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Total Runoff mm	Total Runoff 10^6 ltr	Peak Runoff CMS
Cach-A1	100.40	0.00	0.00	1.58	93.30	2.51	0.60
Catch-A2	83.70	0.00	0.00	1.58	69.50	3.56	0.77
Catch-A3	92.46	0.00	0.00	1.58	75.44	4.64	1.00
Catch-B1	95.46	0.00	0.00	1.58	86.68	3.71	0.86
Catch-B2	83.70	0.00	0.00	1.58	73.09	4.61	1.03
Catch-B3	92.46	0.00	0.00	1.58	80.34	3.24	0.72
Catch-C1	88.49	0.00	0.00	1.58	82.21	2.62	0.63
Catch-C2	90.43	0.00	0.00	1.58	84.14	3.88	0.93
Catch-D1	85.54	0.00	0.00	1.58	81.17	1.51	0.38
Catch-D2	85.54	0.00	0.00	1.58	75.51	5.13	1.15
Catch-D3	87.59	0.00	0.00	2.18	59.01	6.57	1.21

The first column shows the hourly precipitation values. The last column the peak Runoff of for all sub-catchments.

The runoff hydrograph for six sub-catchment as you shown in figure below.

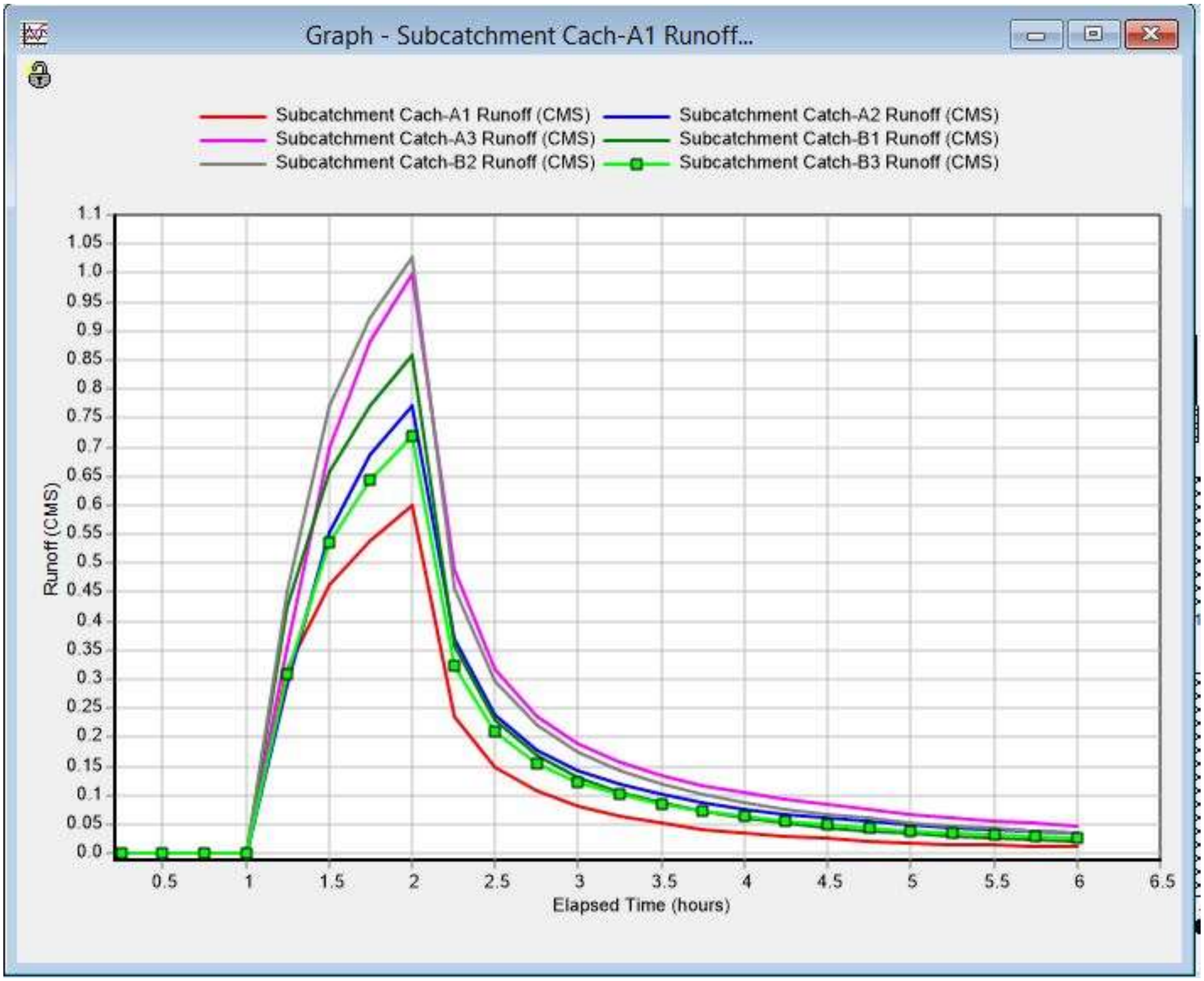


Figure4.0.8 Subcatchment Runoff Hydrograph

The outfall Runoff

Table 4-0-11Outfall Loading Summary result

Summary Results

Outfall Loading — Click a column header to sort the column.

Outfall Node	Flow Freq. Pcnt.	Avg. Flow CMS	Max. Flow CMS	Total Volume 10^6 ltr
Outlet1	82.11	0.445	0.829	7.909
Outlet2	81.55	1.156	1.684	20.386

4.4.2 Network of Links Simulation

In the Network of link simulation shows the depth of flow in conduit, in the junction (node), flooded nodes or links, and water profile of the flow. The simulation result for each links shown below in table.

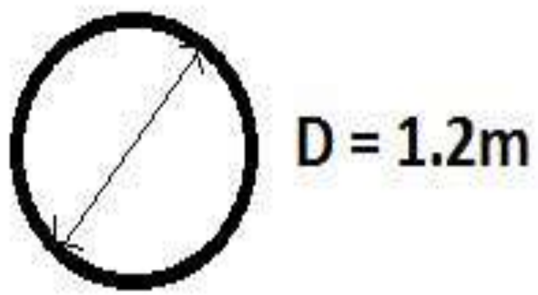
Table 4-0-12Network of link Simulation part 1

Summary Results

Link Flow — Click a column header to sort the column.

Link	Type	Maximum |Flow| CMS	Day of Maximum Flow	Hour of Maximum	Maximum |Velocity|	Max / Full Flow	Max / Full Depth
C1	CONDUIT	0.598	0	02:00	1.90	0.67	0.60
C2	CONDUIT	0.596	0	02:00	2.02	0.62	0.57
C3	CONDUIT	0.596	0	02:00	2.06	0.60	0.56
C4	CONDUIT	0.595	0	02:00	2.28	0.52	0.52
C5	CONDUIT	0.595	0	02:00	2.06	0.60	0.56
C6	CONDUIT	0.595	0	02:00	1.92	0.66	0.59
C7	CONDUIT	0.595	0	02:00	1.92	0.66	0.59
C8	CONDUIT	0.594	0	02:00	1.84	0.69	0.61
C9	CONDUIT	0.594	0	02:01	1.84	0.69	0.61
C10	CONDUIT	0.938	0	02:06	2.04	1.06	1.00
C11	CONDUIT	0.916	0	01:27	2.08	1.02	0.92
C12	CONDUIT	0.897	0	01:35	1.93	1.07	1.00
C13	CONDUIT	0.887	0	01:39	1.92	1.07	1.00
C14	CONDUIT	0.852	0	01:42	1.89	1.05	1.00
C15	CONDUIT	0.867	0	02:07	1.95	1.03	0.91
C16	CONDUIT	0.867	0	01:51	1.96	1.03	0.92
C17	CONDUIT	0.861	0	01:51	1.94	1.03	0.92
C18	CONDUIT	0.863	0	02:10	1.97	1.03	0.92
C19	CONDUIT	0.873	0	02:10	2.01	1.02	0.89
C20	CONDUIT	0.895	0	02:23	1.94	1.06	1.00
C21	CONDUIT	0.876	0	02:23	1.94	1.04	0.93

The result shows the velocity in the links which varies 1.9m/s to 2.04m/s are safe to scour and the depth of flow maximum 1.07m from J9 – Outlet 1 (Along the Hayelom Square to Hawi cafe). The recommended Diameter of the conduit is 1.2m. The existing drainage depth maximum is 0.9m.



.

The result shows the flow velocity through those conduits is efficient for the conduits. The depth of flow indicates there are pipes affect to flood because the maximum depth of flow is 0.9 m in the existing drainage for the above drainage segments. For the rest conduits simulation result there is at the appendix.

The Node (junction) flooding as shown below in summary result the remained there is at the appendix part.

Table 4-0-13Node Flooding

Summary Results

Node Flooding — Click a column header to sort the column.

Node	Hours Flooded	Maximum Rate CMS	Day of Maximum Flooding	Hour of Maximum Flooding	Total Flood Volume 10^6 ltr
J10	0.72	0.473	0	02:00	0.699
J11	0.07	0.027	0	02:07	0.004
J12	0.78	0.080	0	01:27	0.148
J13	0.78	0.050	0	02:08	0.104
J14	0.80	0.053	0	01:41	0.089
J15	0.21	0.007	0	01:42	0.002
J16	0.21	0.013	0	02:07	0.008
J17	0.47	0.018	0	01:44	0.010
J18	0.23	0.010	0	01:51	0.004
J19	0.05	0.004	0	02:10	0.000
J20	1.17	0.989	0	02:00	2.609
J21	0.07	0.043	0	02:23	0.004
J22	0.05	0.011	0	02:23	0.001
J23	0.02	0.002	0	02:24	0.000
J24	0.02	0.003	0	02:24	0.000
J26	1.19	0.047	0	02:25	0.118
J27	0.03	0.002	0	02:25	0.000
J28	1.20	0.049	0	02:26	0.126
J29	1.20	0.032	0	02:27	0.076

The above table shows the volume flood and the point of the flood exists, for this problem mitigation problem increasing of the depth of the link necessary because if we consider the climatic change the risk worst.

The water elevation profile between Junctions (J14-J29)

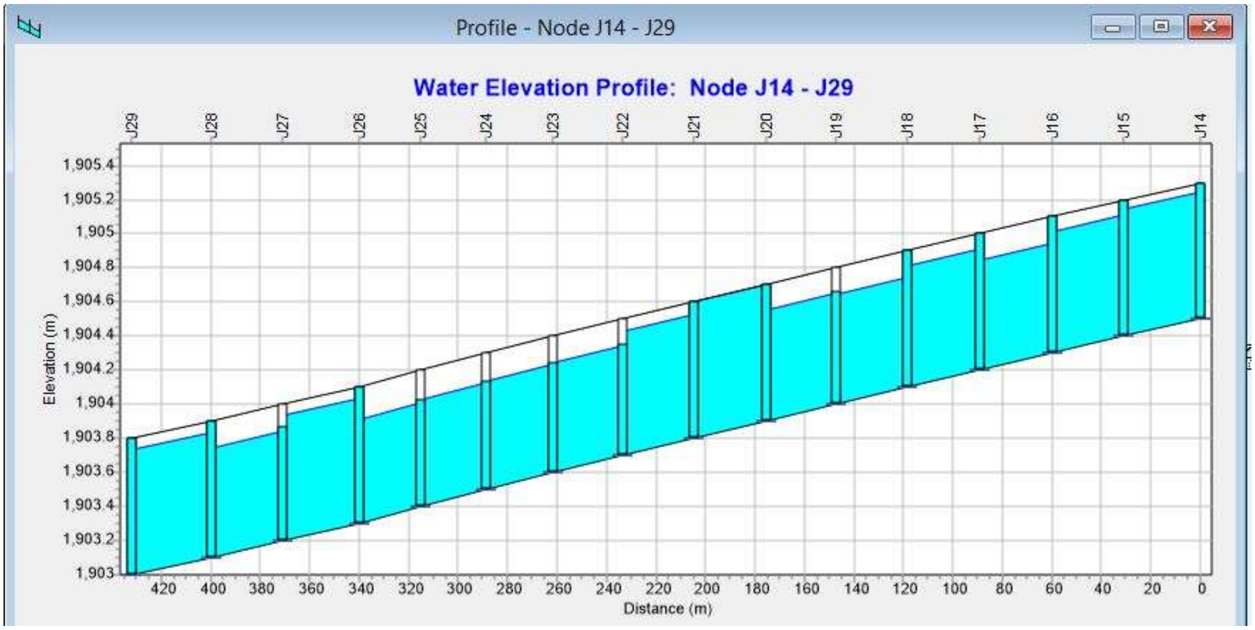


Figure4.0.9 Water Elevation Profile

The water elevation profile showed the flooded nodes and the depth of flow.

4.4.3 Model Calibration and validation

The calibration performs by using changing (adjusting) the sensitive parameters of the model simulated with the Observed depth values. The value of sensitive parameters takes for the model as shown below. The performance of the model is excellent results according to the coefficient of determination and Nash cliff were RNS, =0.62 and R2 = 0.97. So this model is suitable for the design of the drainage in the Shire Endaselassie by using those adjusted sensitive parameters.

Sensitive parameter used values

Calibration Parameters For SWMM model					
Parameters	Meaning		Value Range	Initial Values	Used Values
N - Imperv	Manning's rougness Coefficient For impervios		0.011 - 0.015	0.012	0.019
N - perv	Manning's rougness Coefficient For impervios		0.05-0.8	0.025	0.265
Destrore - Imperv	Depth of Depression Storage on impervious area		0 - 3	1	1.5
Destrore - perv	Depth of Depression Storage on impervious area		3. - 9	4	6
Infiltration Method	Green Ampt	Suction head	3.5	3.5	3.5
		Conductivity	0.5	0.5	0.5
		Initial deficit	0.25 - 0.26	0.25	0.26

. The Subcatchment simulated flow as shown below for comparison with the rational equation determined result.

Table 4-14Model Result Comparison with Rational method

Subcatchment	Peak Runoff using Rational method (CMS)	Peak Run Using SWMM model (CMS)	Variation
Catch - A1	0.4	0.6	0.2
Catch - A2	0.62	0.77	0.15
Catch - A3	0.85	1.0	0.15
Catch - B1	0.63	0.86	0.23
Catch - B2	0.79	1.03	0.24
Catch - B3	0.573	0.72	0.147
Catch - C1	0.445	0.63	0.185
Catch - C2	0.63	0.93	0.3
Catch - D1	0.239	0.38	0.141
Catch - D2	0.872	1.15	0.278
Catch - D3	1.41	1.21	0.21
Qtotal =	7.459	9.28	1.821

The SWMM model was applied to an urban catchment in Shire Endaslasse city and rational method is used for comparison. Calculations are presented; the total runoff from whole sub-catchments by SWMM is 9.28m3/sec, whereas by rational method is 7.459 m3/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 2.177m^3/sec, whereas by Rational Method is1.109m^3/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)

4.5 Mitigation Measures

- Pipe diameters of the drainage should be completely updated according to new ERA hydrology manual. The system includes some pipes whose dimensions do not correspond to the drainage system model, so that a review of the system model is recommended to increase the model reliability. Developing the man power skill of SWMM software for planning, analysis and modeling of storm water runoff drainage systems in urban areas and monitoring the infrastructures new proposed drainage.

- The enlargement of diameters is considered as a possible solution when simulations proved enough capacity at downstream pipes, whereas upstream pipes cannot handle a certain rain. In such case, the line is undersized and an enlargement of the pipes diameter is the best option. Pipe enlargement recommended drainage Hayelom square to Hawi café as result shows increase to 1.2m diameter of concrete pipe.

- Existing drainage system administration road to Mdregenet hotel not functioning properly at this time. Hence, it is advisable to undertake a comprehensive clearing and maintenance work on the existing storm drain system along with the approach road storm drain system development otherwise there may be a danger of traffic interruption due to concentrated flow condition on the roadway.

- Grass swales
  These are depressions in the grassed terrain designed to function as small unlined channels in which storm water runoff is slowed down and partially infiltrated along their course. The flow left after infiltration is conveyed to the storm drain system at the downstream. Grass swales also perform similar to grass filter strips when the slopes are small, less than 5%. The grass swales provided along the parking lots are suitable.

- Provide pervious pavements facilitate peak flow reduction, ground recharge and pollution filtering. Then place of pervious pavements Suitable in parking lots and walkway. Un lining the adjacent ditches to increase infiltration rate.

5. Conclusion and Recommendation

5.1 Conclusion

According to the results in this research the major problem I have seen for the highly over flooded in the nodes. So if the continues rainfall exists dangerous to the society to perform any activity in time.

The simulation result of SWMM model shown as total flow rate for all catchments 56.478 ha have 9.28 m^3/s which determined by SWMM model were as 7.459 m^3/s is by rational formula. Related researches have study to solve the drainage problem in different areas. The total runoff SWMM model result for 18 ha 3.47 m^3/s and, whereas by rational method is 3.14 m^3/s. (Nejib, 2016). The total runoff from seven sub catchments by SWMM is 2.177m3/sec, whereas by Rational Method is1.109m3/sec. (M. L. Waikar* and Undegaonkar Namita U, January, 2015).

Mainly replacements of pipes are a suitable approach to solve local flooding problems. However, this kind of system modifications should be carefully replaced since it could move the problem to other parts of the system instead of solving the flooding risk.

As the model result shows the conduit inadequate to carry the peak flow discharge enlargement of the pipe is available at this current condition. The access road in the study area is denser for every 100m there is access road to any direction so the imperviousness is high.

The model result Shows only the pipes of inadequate to carry the capacity of the conduit 10yrs Designed period. That means the depth of flow reduction on the conduit by the debris and siltation data is available. The other problems exists partial blocked of the drainage system which causes local flooded at that area of minimum cross-section flow.

For the proportional delineated sub catchments for the minimum time of concentration which have maximum Intensity so the maximum value of intensity is the input for the model. In this research the minimum time of concentration is 18.51 min for this Tc which has 100.4 mm/hr. Intensity.

Generally it can be 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 damages to road surfacing material and flooding problems in the area.

5.2 Recommendation

I recommend the existing pipe are modified before high risk is exists. Generally when we see the digital elevation map of Shire Indaselassie the urbanization develops in the gentle area so it is hard to control flooding exists due continues rainfall.

The flow contribution area to the some nodes is high so another drainage system proposed is suitable to eliminate the flooding problem. Drainage clearing habits develops by the administration level. The risk comes due to decreasing its cross-sectional flow is gradual.

When they proposed new drainage system in the city without detail analysis aligned the drainage this habits causes losses of life, financial crisis's to modify again and material wastage. Developing skill of SWMM model is suitable to control flood risk as well as design efficient network of links.

Pitching of roofs by grasses is the best mitigation for decreasing of runoff but it is not common in our country. I appreciated the current situation on the city for any design of building they must to put setback 2m (space provided for green area) but still this setback takes as for the external verandah.

Providing porous concrete mix and minimizing the area of impermeable in any building construction by developing green areas.

The new urban development projects can be enforced with conditions to maintain natural drainage, minimize the impervious areas to be created, provide sufficient areas for infiltration and storm water storage. By providing Swales a long parking lots and pervious walkways.

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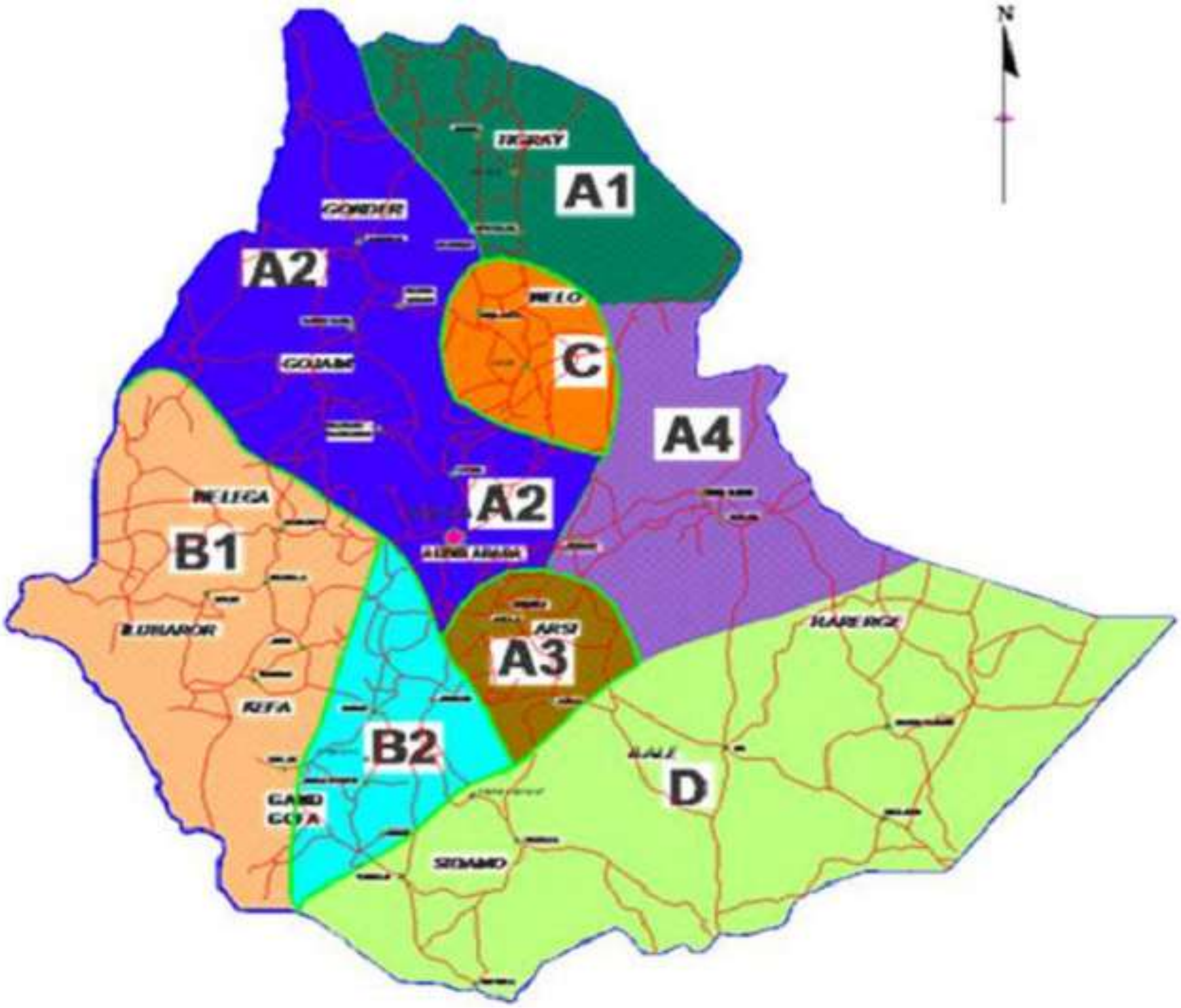
Appendix

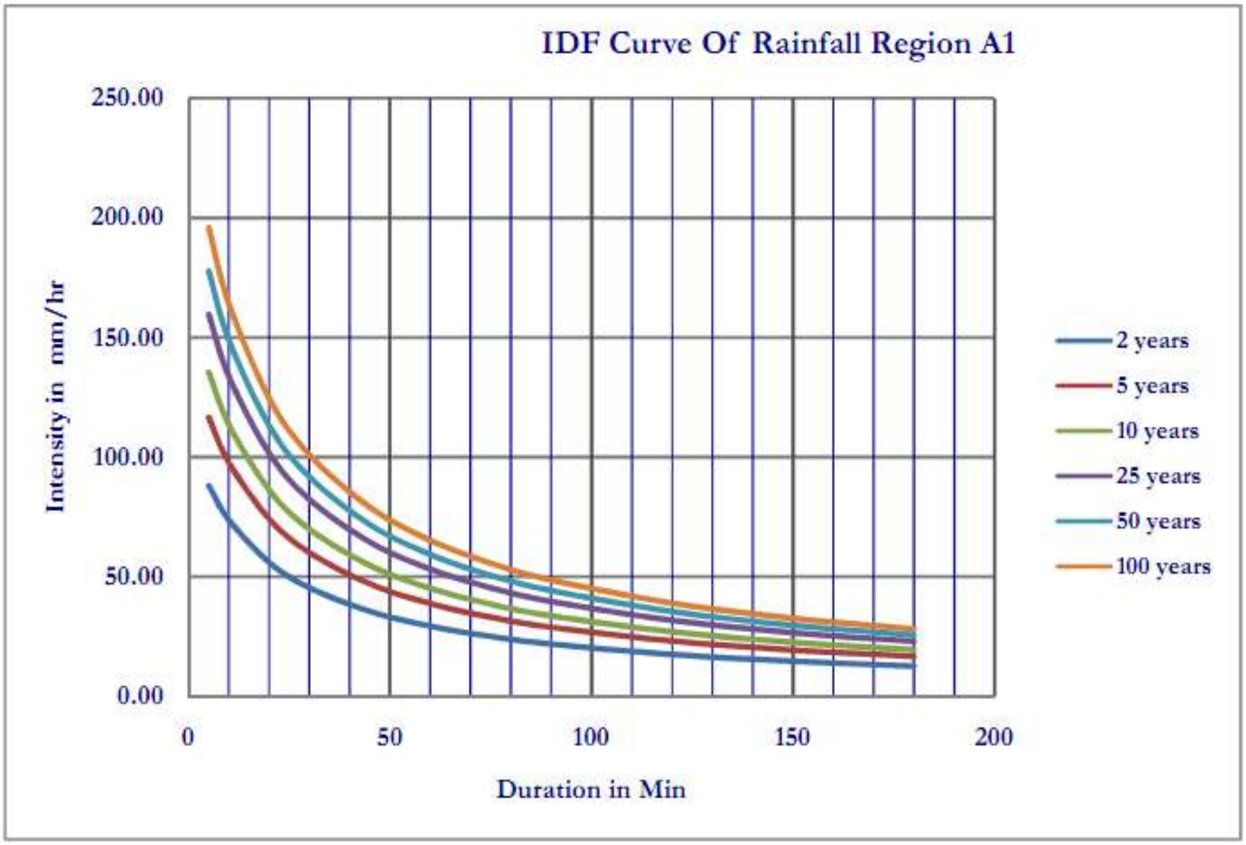
Appendix 1

Rainfall region classification in Ethiopia, Shire Indaselassie includes in region 1(A1)



Appendix 2

The IDF curve developed by ERA for region 1 (A1).



Appendix 3

Comparison of IDF curve results of Shire station with IDF curve from ERA for Region A1

Tr (yr)	2yr		5yr		10yr		25yr		50yr		100yr	
Duration (min)	ERA	Self	ERA	Self	ERA	Self	ERA	Self	ERA	Self	ERA	Self
5	89.5	103.6	118	131.0	136.0	151.8	160.0	181.4	179.0	206.3	198.0	232.4
10	76	86.5	99	109.3	115.0	126.7	134.0	151.4	148.0	172.0	165.0	194.0
15	67	74.3	86	93.9	100.0	108.9	118.0	130.1	129.5	147.7	144.0	166.7
20	58	65.3	75	82.5	89.0	95.6	104.0	114.3	115.0	129.7	125.0	146.5
25	50	58.3	68	73.6	79.0	85.4	91.0	102.0	101.0	115.7	111.0	130.7
30	45	52.7	60	66.5	71.0	77.1	82.0	92.2	90.5	104.5	100.0	118.1
35	42	48.1	55	60.7	65.0	70.4	75.0	84.2	85.0	95.4	94.0	107.8
40	39	44.2	51	55.9	60.0	64.8	70.0	77.5	78.0	87.8	85.0	99.3
45	37	41.0	48	51.8	54.5	60.1	65.0	71.8	72.0	81.4	79.5	92.0
50	34	38.2	45	48.3	52.0	56.0	60.0	66.9	68.0	75.8	75.0	85.7
55	31	35.8	42	45.2	49.0	52.5	58.0	62.7	63.0	71.0	70.0	80.3
60	30	33.7	40	42.6	46.0	49.4	55.0	59.0	60.0	66.8	67.0	75.6
65	28	31.8	37	40.2	42.5	46.6	50.0	55.7	57.0	63.1	62.0	71.4
70	26	30.2	35	38.1	40.0	44.2	48.0	52.8	54.0	59.8	59.0	67.6
75	25	28.7	33	36.2	38.0	42.0	46.0	50.2	50.5	56.8	57.0	64.3
80	24	27.3	31	34.5	36.0	40.0	45.0	47.8	49.0	54.2	54.5	61.3
85	23	26.1	30	33.0	34.0	38.2	42.0	45.7	47.0	51.7	50.0	58.5
90	22	25.0	29	31.5	33.0	36.6	40.5	43.7	45.0	49.5	49.0	56.0
95	21	24.0	28	30.3	32.0	35.1	39.0	41.9	44.0	47.5	48.0	53.7
100	20	23.0	27	29.1	31.0	33.7	38.0	40.3	42.0	45.7	46.0	51.6
105	19.5	22.2	26	28.0	30.0	32.5	36.5	38.8	41.0	44.0	44.0	49.7
110	19.3	21.4	25	27.0	29.0	31.3	34.0	37.4	40.0	42.4	42.0	47.9
115	19.1	20.6	24.5	26.0	28.0	30.2	33.0	36.1	39.0	40.9	40.0	46.3
120	18.9	19.9	23.5	25.2	27.0	29.2	32.0	34.9	38.0	39.5	39.0	44.7

Appendix 4

The depth of flow in the links, maximum velocity and maximum flow as shown below in table forms.

Summary Results

Link Flow — Click a column header to sort the column.

Link	Type	Maximum |Flow| CMS	Day of Maximum Flow	Hour of Maximum Flow	Maximum |Velocity| m/sec	Max / Full Flow	Max / Full Depth
C21	CONDUIT	0.876	0	02:23	1.94	1.04	0.93
C22	CONDUIT	0.876	0	02:24	1.99	1.02	0.91
C23	CONDUIT	0.887	0	02:24	2.02	1.02	0.90
C24	CONDUIT	0.885	0	02:25	2.06	1.01	0.89
C25	CONDUIT	0.865	0	02:25	2.08	0.95	0.79
C26	CONDUIT	0.844	0	02:25	1.88	1.04	0.93
C27	CONDUIT	0.869	0	02:25	1.95	1.03	0.90
C28	CONDUIT	0.839	0	02:27	1.85	1.05	1.00
C29	CONDUIT	0.824	0	02:29	1.85	1.03	0.92
C30	CONDUIT	0.822	0	02:29	2.95	0.57	0.54
C31	CONDUIT	0.856	0	02:00	2.10	0.93	0.76
C32	CONDUIT	0.856	0	02:00	2.21	0.87	0.72
C33	CONDUIT	0.855	0	02:00	2.13	0.90	0.75
C34	CONDUIT	0.853	0	02:00	1.95	1.00	0.82
C35	CONDUIT	0.854	0	02:00	1.99	0.98	0.80
C36	CONDUIT	0.852	0	02:00	1.95	0.99	0.82
C37	CONDUIT	0.851	0	02:01	1.97	0.99	0.81
C38	CONDUIT	0.920	0	02:13	1.97	1.07	1.00
C39	CONDUIT	0.914	0	02:13	2.05	1.03	0.90
C40	CONDUIT	0.906	0	02:14	2.05	1.02	0.90
C41	CONDUIT	0.879	0	02:15	1.95	1.04	1.00

Summary Results

Link Flow — Click a column header to sort the column.

Link	Type	Maximum |Flow| CMS	Day of Maximum Flow	Hour of Maximum	Maximum |Velocity|	Max / Full Flow	Max / Full Depth
C41	CONDUIT	0.878	0	02:15	1.95	1.04	1.00
C42	CONDUIT	0.865	0	02:15	1.95	1.03	0.92
C43	CONDUIT	0.882	0	02:16	1.98	1.03	0.89
C44	CONDUIT	0.880	0	02:16	1.99	1.03	0.90
C45	CONDUIT	0.860	0	02:17	2.61	0.72	0.63
C47	CONDUIT	0.853	0	02:17	2.58	0.72	0.63
C48	CONDUIT	0.852	0	02:17	2.57	0.72	0.63
C49	CONDUIT	1.381	0	01:38	2.93	1.08	1.00
C50	CONDUIT	0.926	0	02:23	2.01	1.06	1.00
C51	CONDUIT	0.916	0	02:24	2.74	0.73	0.63
C52	CONDUIT	0.893	0	02:24	1.97	1.04	0.93
C53	CONDUIT	0.872	0	02:24	1.94	1.04	0.93
C46	CONDUIT	0.857	0	02:17	2.62	0.71	0.62
C54	CONDUIT	0.865	0	02:27	1.95	1.03	0.92
C55	CONDUIT	0.924	0	02:00	2.08	1.02	0.92
C56	CONDUIT	0.924	0	02:00	2.20	0.96	0.78
C57	CONDUIT	0.922	0	02:00	2.11	1.00	0.82
C58	CONDUIT	0.909	0	02:00	2.05	1.02	0.92
C59	CONDUIT	0.894	0	02:00	2.02	1.03	0.92
C60	CONDUIT	0.879	0	02:00	1.98	1.03	0.92
C61	CONDUIT	0.879	0	02:00	2.04	0.99	0.81

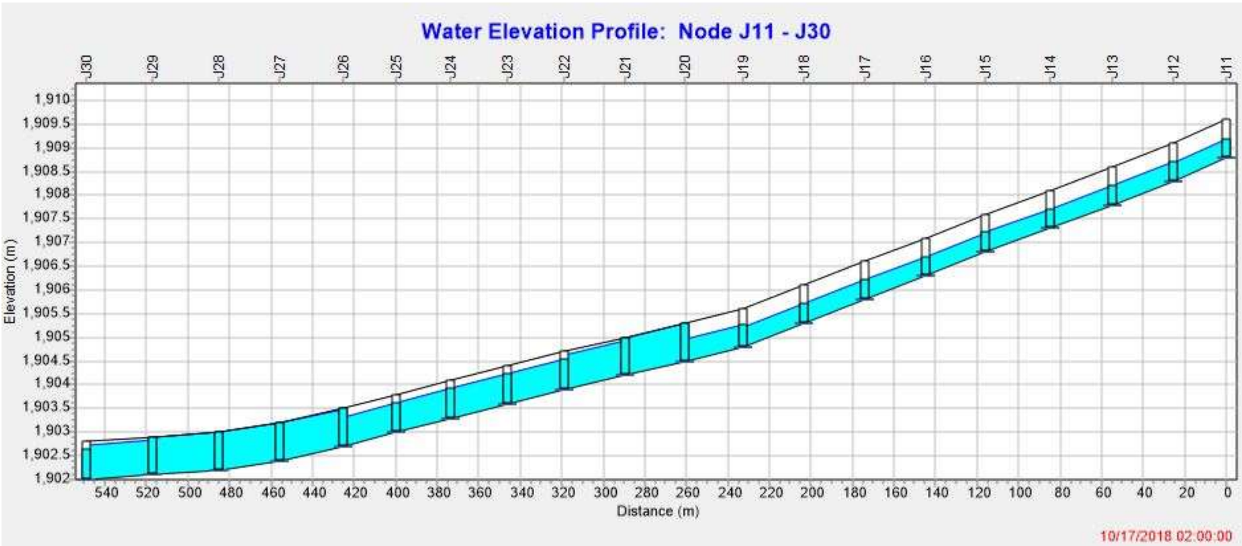
Summary Results

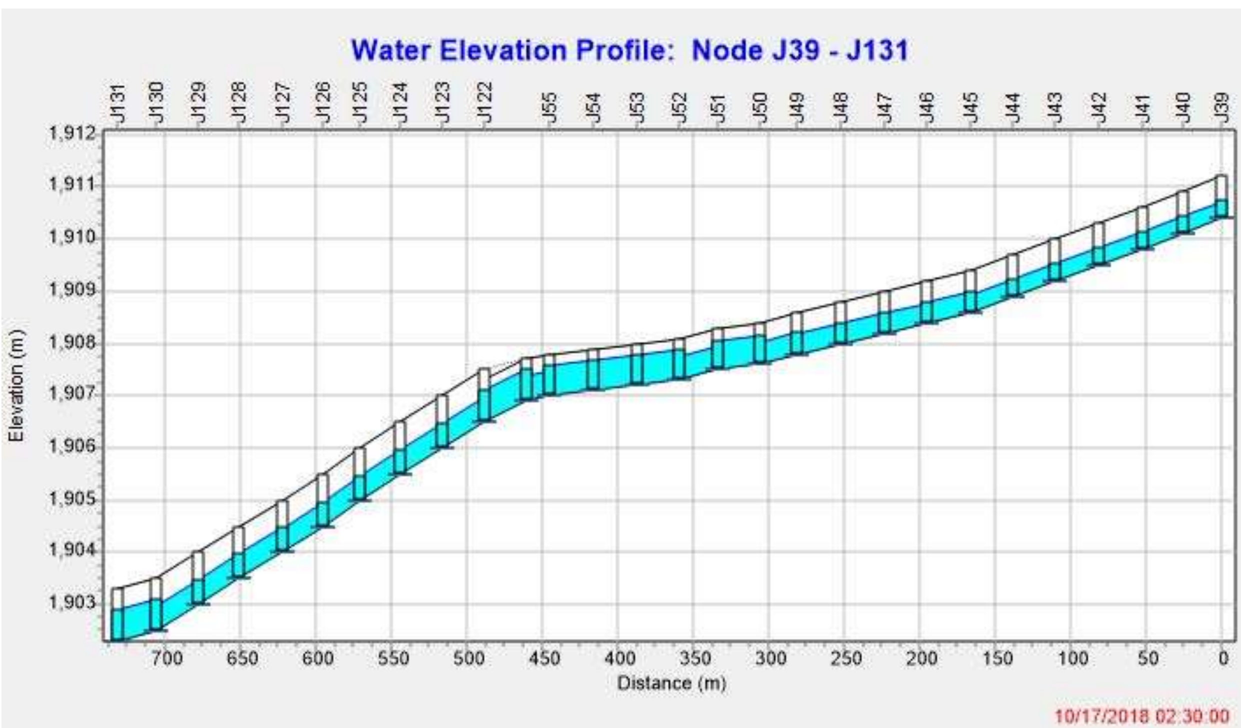
Link Flow — Click a column header to sort the column.

Link	Type	Maximum |Flow| CMS	Day of Maximum Flow	Hour of Maximum	Maximum |Velocity|	Max / Full Flow	Max / Full Depth
C61	CONDUIT	0.879	0	02:00	2.04	0.99	0.81
C62	CONDUIT	0.879	0	01:52	1.98	1.03	0.92
C63	CONDUIT	0.885	0	01:52	2.03	1.01	0.90
C64	CONDUIT	0.900	0	01:54	2.02	1.03	0.89
C65	CONDUIT	0.883	0	01:54	1.99	1.03	0.92
C66	CONDUIT	0.880	0	01:52	3.04	0.60	0.56
C67	CONDUIT	0.872	0	01:52	2.63	0.72	0.63
C68	CONDUIT	1.301	0	01:40	2.82	1.07	1.00
C69	CONDUIT	1.280	0	01:41	2.75	1.08	1.00
C70	CONDUIT	1.253	0	01:39	2.73	1.05	1.00
C71	CONDUIT	1.250	0	01:34	2.80	1.03	0.92
C72	CONDUIT	1.241	0	01:35	2.81	1.02	0.92
C73	CONDUIT	1.234	0	01:42	2.89	0.98	0.80
C74	CONDUIT	1.247	0	01:43	2.85	1.01	0.90
C75	CONDUIT	0.948	0	02:09	2.04	1.07	1.00
C76	CONDUIT	0.903	0	01:21	1.97	1.05	1.00
C77	CONDUIT	0.899	0	01:22	1.94	1.07	1.00
C78	CONDUIT	0.886	0	01:22	2.29	0.87	0.73
C79	CONDUIT	1.208	0	02:00	2.73	1.01	0.91
C80	CONDUIT	1.041	0	01:47	2.21	1.08	1.00
C81	CONDUIT	1.007	0	01:44	2.18	1.07	1.00

Appendix 5

Water surface elevation profile for the following nodes.





Appendix 6

Description of Area	Runoff Coefficients
Business: Downtown areas	0.70-0.95
Neighborhood areas	0.50-0.70
Residential: Single-family areas	0.30-0.50
Residential:Multi units, detached	0.40-0.60
Residential: Multi units, attached	0.60-0.75
Suburban	0.25-0.40
Residential (0.5 hectare lots or more)	0.30-0.45
Apartment dwelling areas	0.50-0.70
Industrial: Light areas	0.50-0.80
Industrial: Heavy areas	0.60-0.90
Parks, cemeteries	0.10-0.25
Playgrounds	0.20-0.40
Railroad yard areas	0.20-0.40
Unimproved areas	0.10-0.30

(Source: Hydrology, Federal Highway Administration, HEC No. 19, 1984)

Roughness coefficient for different catchment characterstics.

Type of Drainage Area	Runoff Coefficient, C*
Business:	
Downtown areas	0.70 - 0.95
Neighborhood areas	0.50 - 0.70
Residential:	
Single-family areas	0.30 - 0.50
Multi-units, detached	0.40 - 0.60
Multi-units, attached	0.60 - 0.75
Suburban	0.25 - 0.40
Apartment dwelling areas	0.50 - 0.70
Industrial:	
Light areas	0.50 - 0.80
Heavy areas	0.60 - 0.90
Parks, cemeteries	0.10 - 0.25
Playgrounds	0.20 - 0.40
Railroad yard areas	0.20 - 0.40
Unimproved areas	0.10 - 0.30
Lawns:	
Sandy soil, flat, 2%	0.05 - 0.10
Sandy soil, average, 2 - 7%	0.10 - 0.15
Sandy soil, steep, 7%	0.15 - 0.20
Heavy soil, flat, 2%	0.13 - 0.17
Heavy soil, average, 2 - 7%	0.18 - 0.22
Heavy soil, steep, 7%	0.25 - 0.35
Streets:	
Asphaltic	0.70 - 0.95
Concrete	0.80 - 0.95
Brick	0.70 - 0.85

Appendix 7

Input parameters of SWMM

Table Depression Storage (Rosman, 2010)

Impervious surfaces	1.27- 2.54 mm
Lawns	2.54 – 5.08
Pasture	5.08 mm
Forest litter	7.62

Table manning roughness coefficient for closed conduit (Rossman, 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

Table Manning's roughness coefficient (n) over land flow (Rosman 2010)

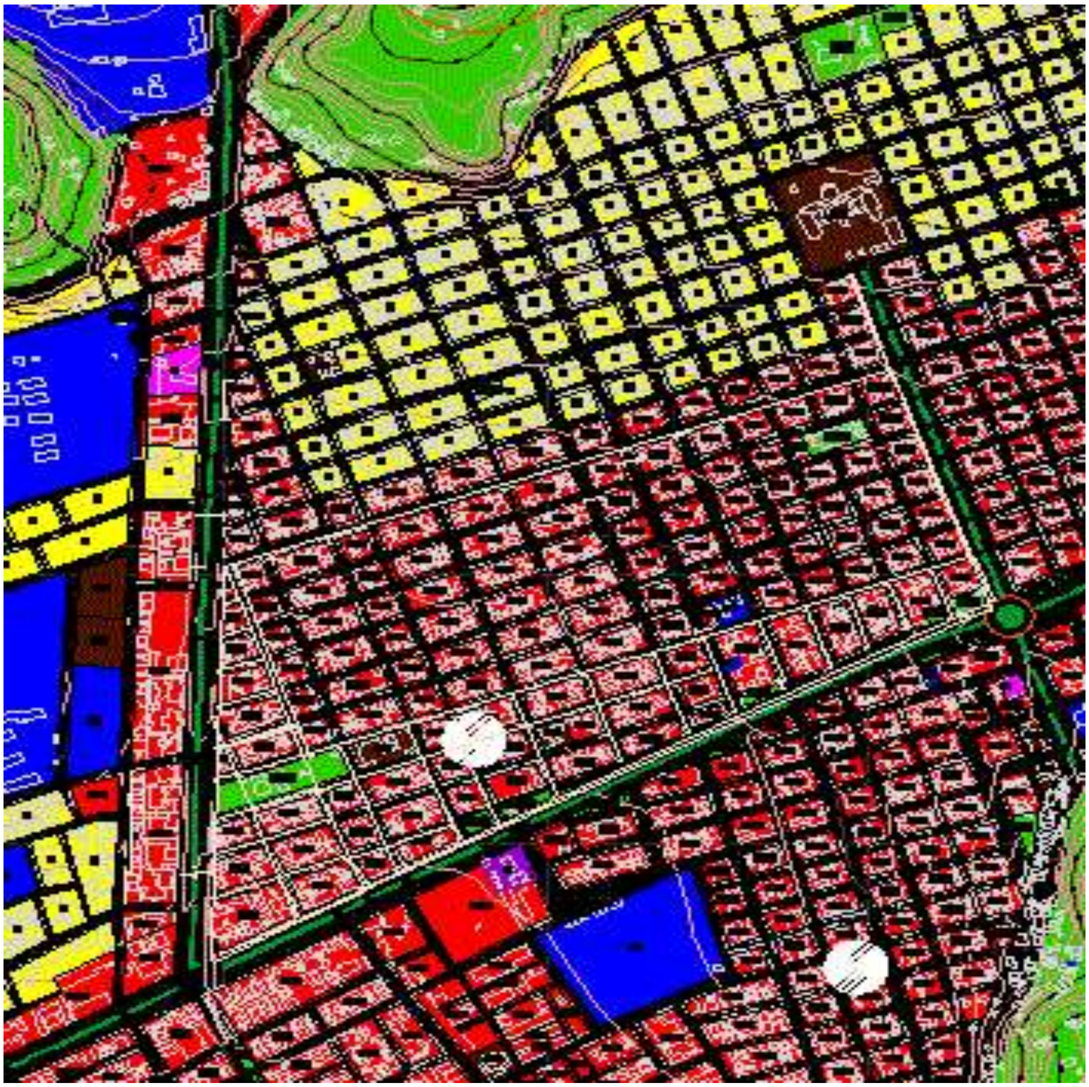
Surface	n
Smooth asphalt	0.011
Smooth concrete	0.012
Ordinary concrete lining	0.013
Good wood	0.014
Brick with cement mortar	0.014
Vitrified clay	0.015
Cast iron	0.015
Corrugated metal pipes	0.024
Cement rubble surface	0.024
Fallow soils (no residue)	0.05
Cultivated soils	
Residue cover < 20%	0.06
Residue cover > 20%	0.17
Range (natural)	0.13
Grass	
Short, prarie	0.15
Dense	0.24
Bermuda grass	0.41
Woods	
Light underbrush	0.40
Dense underbrush	0.80

Table Manning's roughness coefficient (n) Open channels (Rosman 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 8

The map of the modeled Area Auto CAD figure.



Hayelom Square to Hawi Café			Road to AdmintrationTo Mdregenet Hotel		
Drainage Segment	Length (m)	Invert elivation	Drainage Segment	Length (m)	Invert elivation
J1 - J2	28	1914.8	J31 - J32	24	1915
J2 - J3	22	1914.2	J32 - J33	21	1914.3
J3 - J4	21	1913.6	J33 - J34	23	1913.6
J4 - J5	23	1913	J34 - J35	28	1912.9
J5 - J6	21	1912.4	J35 - J36	27	1912.2
J6 - J7	25	1911.8	J36 - J37	28	1911.6
J7 - J8	25	1911.2	J37 - J38	27.5	1911
J8 - J9	28	1910.6	J38 - J39	28	1910.7
J9 - J10	28	1909.8	J39 - J40	26	1910.4
J10 - J11	26	1909.3	J40 - J41	26	1910.1
J11 - J12	25.5	1908.8	J41 - J42	29	1909.8
J12 - J13	29.5	1908.3	J42 - J43	29	1909.5
J13 - J14	30	1907.8	J43 - J44	28	1909.2
J14 - J15	31	1907.3	J44 - J45	28	1908.9
J15 - J16	29	1906.8	J45 - J46	29	1908.6
J16 - J17	29	1906.3	J46 - J47	28	1908.4
J17 - J18	29.5	1905.8	J47 - J48	29	1908.2
J18 - J19	29	1905.3	J48 - J49	29	1908
J19 - J20	28	1904.8	J49 - J50	25	1907.8
J20 - J21	29	1904.5	J50 - J51	27	1907.6
J21 - J22	29	1904.2	J51 - J52	26	1907.4
J22 - J23	28	1903.9	J52 - J53	28	1907.2
J23 - J24	27	1903.6	J53 - J54	29	1907.1
J24 - J25	26.6	1903.3	J54 - J55	29	1907
J25 - J26	25	1903			
J26 - J27	31	1902.7			
J27 - J28	29	1902.4			
J28 - J29	32	1902.2			

J29 - J30	31	1902

Road to Administration to Front of Addis Ababa Hotel			Administration To Ethiopia Hotel		
Drainage Segment	Length (m)	Invert elivation	Drainage Segment	Length (m)	Invert elivation
J56 - J57	25	1914.7	J81 - J82	29	1918
J57 - J58	22	1914.4	J82 - J83	22	1917.6
J58 - J59	24	1914.1	J83 - J84	23	1917.2
J59 - J60	26	1913.8	J84 - J85	29	1916.8
J60 - J61	27	1913.5	J85 - J86	28	1916.4
J61 - J62	28	1913.2	J86 - J87	28.5	1916
J62 - J63	26	1912.9	J87 - J88	29	1915.6
J63 - J64	28	1912.6	J88 - J89	29.5	1915.2
J64 - J65	27	1912.3	J89 - J90	25	1914.9
J65 - J66	27	1912	J90 - J91	26	1914.6
J66 - J67	28	1911.7	J91 - J92	29	1914.3
J67 - J68	29	1911.4	J92 - J93	28	1914
J68 - J69	28	1911.1	J93 - J94	29	1913.7
J69 - J70	28	1910.9	J94 - J95	28	1913.4
J70 - J71	29	1910.7	J95 - J96	29	1913.1
J71 - J72	29	1910.5	J96 - J97	29	1912.7
J72 - J73	28	1910.3	J97 - J98	29	1912.3
J73 - J74	28	1910.1	J98 - J99	28.5	1911.9
J74 - J75	26	1909.9	J99 - J100	25	1911.5
J75 - J76	27	1909.7	J100 - J101	26	1911.2
J76 - J77	26	1909.5	J101 - J102	25	1911
J77 - J78	28	1909.3			
J78 - J79	29	1909.1			
J79 - J80	28	1909			