Hydraulic and Hydrological Urban Catchment Modeling

(Case Study: Shashemene Town)

By
Mohammed Hasen

Addis Ababa, Ethiopia
June, 2017

ADDIS ABABA SCIENCE AND TECHNOLOGY UNIVERSITY

COLLEGE OF ARCHITECTURE AND CIVIL ENGINEERING

Hydrological and Hydraulic modeling of urban catchment

(Case of Shashemene Town)

A Thesis Submitted to the College of Architecture and Civil Engineering of Addis Ababa Science and Technology University in partial Fulfilment of the Degree of Master of Science (M.Sc) in Civil Engineering (Hydraulic Engineering)

By Mohammed Hasen

Advisor Dr. Eng. Habtamu Itefa

CERTIFICATION

The undersigned certify that he has read the Thesis **Hydraulic and Hydrological Modelling Urban Catchment** and hereby recommend for acceptance by the Addis Ababa Science and Technology University in partial fulfilment of the requirements for the degree of Master of Science in Civil Engineering.

____

Dr. Ing. Habtamu Itefa
(Advisor) Date_

Abstract

In 2016(G C), most of the states of Ethiopia are suffered from a disastrous flood that caused extensive damage to homes, agricultural lands, commercial property, and public infrastructures. Shashemene town and its surrounding were among the affected area by the storm runoff. Good example was Shashemene-Aje -Arbaminch road that was cut by the landslide combined with high flooding from the upstream of the Shashemene city catchment area in June 2016(GC). Modeling of such Storm water has a major role in preventing of flash floods and urban water-quality problems issues. However, in-detail modeling of large urban areas is time-consuming as it typically involves model calibration based on highly detailed input data. Storm water models of a lowered spatial resolution would thus appear valuable if only their ability to provide realistic results could be proved.

This study proposes a methodology for rapid catchment delineation and stormwater management model (SWMM) parameterization in urban area. A catchment delineation and SWMM parameterization is conducted for an urban area, in the Shashemene city. GIS methodology is utilized for simultaneous processing of data representing large areas. Literature values are also of importance where no spatial data is available. To evaluate the parameterization results, the SWMM application is run using an hourly data series of meteorological observations covering a period of ten years.

The routines established in the study make the catchment delineation and subdivision process reasonably fast and accurate, although manual work cannot be fully avoided due to defects in the input data. In contrast, the SWMM parameterization of the low-resolution subcatchments is the more challenging part and involves larger uncertainties. Even so, the model application provides sufficient results compared to literature and other studies and measurements performed on the site. Overall, the methods developed in this study provide a feasible approach for SWMM parameterization in large urban areas with visible difference of actual drainage capacity of storm water runoff and determined canal capacity from existing drainage facilities in the city of Shashemene .This require additional canal to alleviate this drainage problems.

***Keywords** ,Shashemene; Urban Hydrology and Hydraulics; Stormwater; Modeling; SWMM; ArcGIS*

DECLARATION AND COPY RIGHT

Mohammed Hasen Kedir _____________

mamehk@gmail.com Signature

DEDICATION

TO MY FAMILY

ACKNOWLEDGEMENTS

I indebted deeply my thanks to Almighty WAKA, the Creature and the owner of the special place like Arsedi , the special peoples like Kofin, to Hora and from Hora.

My heartfelt gratitude goes to my promoter (Advisor), Dr. Ing. Habtamu Itefa. I have been amazingly fortunate to have an advisor who gave me the freedom to explore on my own and at the same time the guidance to recover when my steps faltered. I hope that one day I would become as good an advisor to my students as he has been to me.

I owe my gratitude to Dr. Geremew and Dr. Fistum without whom this thesis would not have been possible. The support, encouragement and professional advice that I received from my them was of much help in the writing of this final thesis.

I would like to say thank you, with those people who have made this thesis possible and because of whom my graduate experience has been one that I will cherish forever.

I am most greatly thankful to Birds of Hora Arsedi, they were with me while I was alone ,telling me the secrete of Arsedi secretly, with their encouraging melody and sound through the past Months of my work.

Table of Contents

List of Tables

List of Figures

List of Abbreviations

ARFI .... annual Rain fall intensity

EIA...Effective Impervious Area

HD-.. ....hi density

IDF...intensity-duration-frequency

IMD max...Maximum Initial Moisture Deficit

NDVI .... normalized difference vegetation index

NLS ....National survey of Ethiopia

NMA-..National Meteorologi cal Agency

ONRS..Oromia National Regional State

OUPI-..Oromia Urban Planning Institute

SWMM-...Storm Water Management Modeling

SWR... ....Storm Water Runoff

TIA -..Total Impervious Area

1. INTRODUCTION

In Ethiopian context, where watersheds of many urban centers receive significant amount of annual rainfall and where rainfall intensity is generally high, control of runoff at source, flood protection, and safe disposal of storm water/runoff through proper modelling facilities becomes essential. The importance of urban storm water modeling is constantly increasing due to global trends like: urbanization, population growth, and climate change. Urbanization and population induce a rapid growth of cities, making storm water management ever more challenging while at the same time a rising number of people is affected by the harmful effects of storm water on the environment. In many areas, these effects are expected to be louder in the future due to climate change and associated higher frequencies of extreme weather events.

Storm water is pure rainfall plus anything the rain carries along with it. The concept of storm water is strongly related to urban areas where conveyance systems like rooftops, streets, parking lots, yards, sidewalks and filed, carrying number of pollutant with it exist. There is storm water system in place that consists of storm drains or catch basins , pipes and out falls that are designed to carry rain water away from developed areas in order to prevent flooding. Urban floods are thus not uncommon, sometimes causing material damage worth up to tens of millions of properties.

Despite flooding, storm water also is interesting regarding the urban water balance. The expansion of impervious land-cover implies both larger storm water runoff volumes and peak flows and consequently reduces other components of the hydrologic cycle, infiltration and evapotranspiration. Moreover, storm water directly transports harmful substances from urban surfaces into downstream water systems, thus degrading the water quality.

Both stormwater quantity and quality issues can be analyzed and tackled with the aid of stormwater modeling. Modeling can be conducted at different spatial resolutions by aggregating similar features together or presenting them separately. "The accuracy of coarse-scale models is not yet clearly known so one cannot fully trust the results" (Ghosh and Hellweger, 2011). Consequently, high-resolution modeling remains the most exact method, but is often not feasible for large geographical areas. Urban catchments are mostly ungauged, preventing model calibration. And even if the catchments were gauged, the numerous calibration parameters would make calibration hard. In addition, spatial data of sufficient resolution and quality is in many cases unavailable. The parameterization of large-scale storm water models thus remains challenging, requiring the development of new methodologies.

1.2. Statement of the Problem

In 2016, most of the states of our country were suffered from a disastrous flood that caused extensive damage to homes, agricultural lands, commercial property, and public infrastructures. The Shashemene Town Watershed catchment was selected as one of the study areas because, this region has suffered frequent severe storm water floods every year .

This storm water causes the loss of life and high land slide in the direction of the storm water outlet from the city. This out let is located in between the center of lakes of Hawasa and Shala. The area was affected by the Lack of sufficient rainfall for more than one year and shortage of infiltrating to the ground water, that was resulted to high runoff in the area. Since the soil of the area silt sand with shallow loam, it was cracked during the dry season and when the state of saturation arrived with storm water from Shashemene city and high plateau of Abaro mountain ,it can easily submerged and cause land slide to occurred. Shashemene was also affected by non routed flood in both direction Abaro and high Plato of upper Awasho kebele every year.

1.3. Objective

1.3.1. General Objective

The General objective of the study was to develop a thorough GIS-based storm water simulation methodology to demonstrate the potential and effect of flooding in highly urbanized areas.

1.3.2. Specific Objectives

- To perform detailed catchment delineation and surface discretization for modeling
- To estimate the existing drainage capacity
- To analysis and simulate the estimated run off with existing drainage capacity of sub catchments.
- To check the feasibility and reality of SWMM modeling approach in a large urban area.

1.4. Description of the study area

1.4.1. Location of the Project

Shashemene is found in Oromia National Regional state, in West Arsi Administrative zones. Shashemene, the capital town of West Arsi Zone, is located 250km south of Addis Ababa, with a surface area of 12,868 ha. The town is located between $7^{o}\,8'51''$N to $7^{o}18'19''$N latitude and $38^{o}32'43''$E to $38^{o}41'07''$E longitude (Figure 1. below).

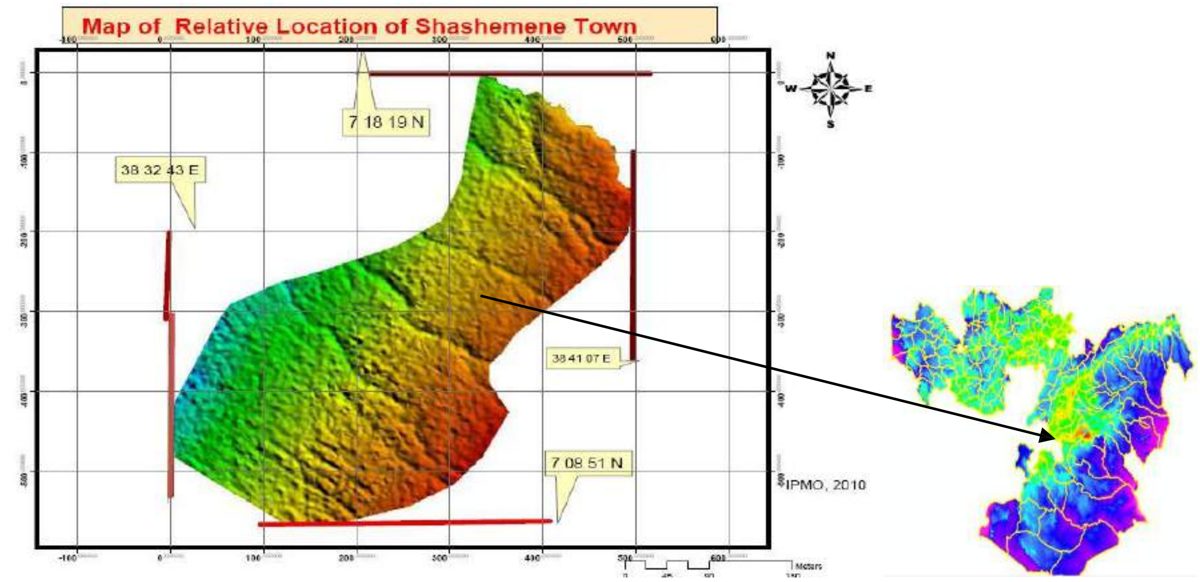


Figure 1. Location of Shashemene area(source, Oromia DEM January, 2009.)

The topography of Shashemene town is relatively regular and flat of mild slope. The dominant soil type within the town and even at a surrounding rural land is almost the same and covered by a loose gray-brown sandy soil and which is pyroclastic of porous texture.

There are four perennial rivers namely Esa(Alelu), Melka Oda, Dhadhaba Guda, and Dhadhaba Xiqa flowing across the Shashemene town. There are also some intermittent streams and gorges namely Gogeti, Tutu, Agamsa, Abiyu (Laftu) across the town. The rivers crossing town have a narrow and long stretching parallel catchment areas.

1.5. Thesis Structure

Chapter 2, Literature review on basic hydrological and hydraulic concepts as well as an introduction to rainfall-runoff modeling.

Chapter 3, It describes the development of an analytical model to represent and verify the proposed conceptual model.

Chapter 4, presents the methodologies and materials used for the study by visualizing the research process and showing which methods were applied for each task.

Chapter 5, presents results and discussions.

Chapter 6, draws the conclusions and remarks on the study and recommend further research on the areas and reference materials listed on this portion.

2. Literature Review

2.1. General Overview

In modeling and hydrological considerations catchment or drainage basin is the fundamental consideration. A catchment is the area contributing to the stream flows at a certain cross section. Catchments are delineated based on the topography of the area. The line from which water might drain to either one of two different catchments is called a divide. Moreover, the stream cross section through which all the runoff exits the catchment is called a pour point. This point can be located in any part of the stream network, depending on the size of the area of interest for the study at hand. (Dingman, 1994).

Traditionally, catchments have been delineated using topographic maps that show contours for the study area. In the last decades, however, digital elevation models (DEMs) have become the main data source used.

The catchment is often considered as a system , consisting of a control volume subject to the regional water-balance .The water balance is the backbone of the hydrologic and hydraulic modeling in a watershed.

$(P + I + Ar + Qi) - (R+Et + D + Qo + W) = \pm S$..1

P=precipitation

I=infiltration from surface water

Ar=artificial recharge

Qi=groundwater inflow

R= surface runoff

Et=evapotranspiration

D=drainage (including upward seepage)

Qo=groundwater outflow

W=withdrawal

$\pm S$ =change in storage

Depending on each particular situation one or more terms of the balance equation can be omitted. Mathematically the concept of a water balance seemed simple, but was not in fact. However, in practise, measuring all the fluxes accurately is not simple, because understanding of the various hydrological processes and the ability to measure the various water budget components are limited(Zekai Sen ,1995).

2.2. Effect of urbanization on water balance

Another important impact of urbanization on environment is that urbanization strongly affects surface water balance and then affects other hydrological and meteorological factors. The mean annual precipitation of these two periods is very close to each other. The study on effect of urbanization in Shijiazhuang, China show that the mean runoff is increased by around 32% due to the increased built-up area (27.4%) ET was decreased about 20%, and as a result, the surface temperature increased to compensate for the excessive energy due to ET reduction. This result implies that urbanization increases the risk of urban floods during extreme rainfall events and contributes to enhance the heat island (Shen et al., 2005).

In addition, fast urbanization can quickly increase the municipal water demand and consumption and lead to a higher pressure on the water supply, especially in semiarid or arid regions (Shen et al., 2005).

Urbanisation affects environmental factors of the area in many ways. Vegetation types and coverage are changed dramatically as land is cleared, while an increase in impervious surfaces occurs from the construction of roads and buildings . Landforms may be altered, as areas are flattened or built up to accommodate different constructions, and new water pathways are introduced. The inclusion of sewerage systems, water resource systems, irrigation and stormwater drains all provide new waterways for water to travel (Tang et al., 2003).

Water quality is another significant concern where urbanisation is concerned. The major source of groundwater contamination in developed countries involves saline intrusion, however rapidly developing cities are encountering more severe health issues (Rygaard, Binning & Albrechtsen 2011). Where resources and expertise are lacking, urban water supply systems are constructed with minimal long term design goals. Regulations concerning pollution and waste management may not be sufficient, and the culmination of these can lead to contamination resulting from placing abstraction bores too shallow or near sources of pollution.

2.3. Precipitation

2.3.1. General Overview

"Precipitation equals evaporation". All of the precipitation that falls originated as a water vapor that was originated from the surface of the earth. It is always raining somewhere on the earth , just as evaporation is always occurring over most of the earth's surface. At any given time, precipitation covers only about 2% to 5% of the surface of the earth , while evaporation is occurring over the remaining 95% to 98% of the earth. Thus as water vapor slowly evaporates over most of the earth, an approximately equal amount gets "concentrated„' into relatively small rain system that turn some of the vapor into precipitation.

So, averaged over the whole earth over a period of months, the amount of precipitation almost exactly balances the amount of evaporation. If this were not so, the atmosphere would either be filling up with water vapor , or be depleted in water vapor. (waetherquestions.com) .

Rainfall in Oromia is mainly Orographic, though local Conventional and Frontal types are also experienced. The duration, amount and spatial distribution of the rainfall of Oromia depends upon relative location of a place to Atlantic and Indian Ocean, and the Red sea, air pressure and air circulation systems and variation in elevations. In general, the main controls of the rainfall distribution of the Oromia region are South-Westerly Winds, South-Easterly Winds, North-Easterly Winds and High pressure cells over North-East Africa and the Arabian Peninsula. (ONRS, Program of plan on adaptation of climatic change).

Generally, the annual rainfall in the region ranges from 400-2400mm (the annual average is set between 450-1800mm when weighted throughout different localities), where the highest rainfall record is observed in the western regime of the region, while the lowest precipitation amount is recorded in the low lands of eastern and south eastern parts. This may cause heavy but temporally short rain with also potential thunder. (ONRS, Program of plan on adaptation of climatic change ,January, 2010).

2.3.2. Spatial and temporal variability of precipitation

Ethiopia is extremely vulnerable to the impacts of climate change due to social, economic and environmental factors. In particular, high levels of poverty, rapid population growth, a high level of reliance on rain-fed agriculture, high levels of environmental degradation, chronic food insecurity and frequent natural drought cycles increase climate change vulnerability in this country. Climate change will have a notable impact on Ethiopia"s temperature and precipitation: average annual temperatures nationwide are expected to rise by 3.1° C by 2060, and 5.1° C by 2090.

Oromia is characterized by a bimodal rainfall pattern, with the main rainy season (long rains) between June-September, covers most portion of the region. The average annual rainfall is between 450mm and 1800mm, with substantial variability in time and space between and within the areas. The changes of the seasonality, distribution and regularity of rain fall are becoming more of concern than the overall amount of rain fall. (ORA, 2008).

For instance, according to Oxfam International (2010), the rainfall data gathered from Batu (Rift Valley Area of Oromia) meteorological station showed, the total numbers of rainfall days were decreased from 73 rainy days in 1982 to 8 rainy days in 2007, whereas the magnitude of the total annual rain fall is more or less the same. As explained by same, in the last three to four decades rain fall has become highly variable and erratic in terms of amount and distribution in the area. The rainfall, when it occurs, is usually heavy and often causes floods, with hailstorms and/or windstorms. In the area, the same study, reported that the short rainy season (Arfassa) has failed repeatedly in the past 20 years; farmers have had to wait up to 10 months in recent years to see a drop of rain and also most months of a year are dry even during the main rainy Season, Ganna (Oxfam International, 2010). Similarly, the rainfall data extracted from Yabelo (Borena) meteorology station from 1987 to 2005 shows a high variation in number of rainy days and a slight downward trend. Under normal conditions, Ganna (long rains) season is from March to April, while also called Hagaya (also called Birra) (short rains) is from mid-September to mid-October south parts of the region (Borena). Now, no one knows when the rains come, and when they do, they are very short.

There are several factors affecting precipitation conditions on different regions around the globe ; include latitude, altitude, distance to areas of evaporation, dominating wind directions, position in relation to mountain ranges, and the temperature gradient between sea and continent (Kuusisto, 1986).

2.3.3. Precipitation measurement

Accurate precipitation measurements are crucial for successfully studying and modeling the processes that take place in a catchment. However, these input data always are subject to some degree of uncertainty. This uncertainty is induced by both the methods used for observing precipitation, and the methods used for generalizing the measurements to cover the whole area of the studied system, like the catchment.

The traditional way of measuring precipitation at a single point is simple. It is done by placing a vessel on an open field and measuring the amount of water caught. The results can be observed at certain intervals, say, once a day, or continuously with an automated metering system. The data gained is typically disturbed by errors from several sources. These include obstructions nearby, losses due to splash, evaporation or wetting, instrument errors, observer errors, errors due to varying observation intervals, and so on. Several different types of precipitation gages have been developed to minimize the impact of different error sources. In addition, the observed values are usually corrected to take into account any known systematic errors (Dingman, 1994; Kuusisto, 1986).

Discrete point measurements can be interpolated to obtain the areal precipitation as well as contours describing the spatial variability of precipitation over a region. There are several different mathematical approaches to solve this interpolation (e.g. the one presented by Thiessen (1911)), but they will not be discussed any further here. At present, an increasing share of precipitation measurements are conducted using radar or satellite observations. The new technology is useful as it can provide very detailed information on the areal precipitation and its spatial variability. Nevertheless, traditional observing still holds its place, as the remote sensing data still needs to be calibrated against ground measurements from rain gages. (Dingman, 1994)

2.3.4. The effect of urbanization on precipitation

On urban areas, precipitation is typically increased compared to natural conditions. Urbanisation affects environmental factors of the area in many ways. Vegetation types and coverage are changed dramatically as land is cleared, while an increase in impervious surfaces occurs from the construction of roads and buildings . Landforms may be altered, as areas are flattened or built up to accommodate different constructions, and new water pathways are introduced. The inclusion of sewerage systems, water resource systems, irrigation and stormwater drains all provide new pathways for water to travel (Lerner 1990).

Urbanisation has a significant effect on local water balances. In particular, the increase in impervious surfaces and change in soil quality affect runoff characteristics (Haase 2009).

Urbanising an area has a much greater impact than just increasing impervious surfaces and decreasing vegetation. There are more complex processes involved in these systems and total recharge can actually be increased due to changing water pathways (Lerner 1990). The addition of water supply and sewerage systems creates an opportunity for water leakage to occur and stormwater collection systems divert water from its natural course, potentially increasing local recharge. This is particularly relevant with West Arsi Zone sandy aquifers between Abas(Hawassa) and Shala lakes, where high infiltration rates may contribute to the potential for an overall increase in recharge after urbanisation has occurred in the downstream of Shashemene catchment.

2.4. Evapotranspiration

Evapotranspiration includes all the processes occurring in the proximity of land or water surfaces that result in evaporation of liquid water into atmospheric water vapor.

Evapotranspiration can be divided into two main components, caused by different physical phenomena. The first one, evaporation, refers to all the evaporation of water from the surface of ground, water, or a snowpack. This takes place when the water molecules on a surface acquire a sufficient amount of energy to escape the surface and enter the gas phase. (Vakkilainen, 1986).

One important point of view is the difference between the terms potential evapotranspiration and actual evapotranspiration. The first one refers to the possible evaporation rate if the amount of water in the soil was not limited, and if no advection or heat-storage effects took part. In other words, it assumes evapotranspiration to be energy-limited. In some regions, or during dry seasons, however, the process is limited by the soil water content. Thus, the amount of actual evapotranspiration differs from the potential value(Dingman, 1994).

The areal actual evapotranspiration can be measured or calculated in a number of ways. These may incorporate water-balance approaches, lysimeter or evaporation pan measurements, as well as empiric or semi-empiric formulas, etc. One of the commonly used methods is the Penman-Monteith equation (Dingman, 1994):

ET=($\Delta(K+L)+\rho_a c_a C_{at} e_a* (1-W_a/100))/(\rho_w \lambda_v[\Delta+\gamma(1+C_{at}C_{can}))$ .... (2)

where

ET = evapotranspiration rate from a vegetated surface [mm/d],

Δ = slope of the saturation-vapor-pressure vs. temperature curve at the air

temperature [mbar/°C],

K = net incoming shortwave radiation [kJ/m2/d],

L = net incoming long-wave radiation [kJ/m2/d],

ρ_a = density of air [kg/m3],

c_a = specific heat of the air [J/kg/°C],

C_{at} = atmospheric conductance for water vapor [mm/d],

e_a* = saturation vapor pressure at the air temperature [mbar],

W_a = relative humidity [%],

ρ_w = density of water [kg/m3],

λ_v = latent heat of vaporization [J/kg],

γ = psychrometric constant [mbar/°C],

C_{can} = canopy conductance [mm/d].

Another, more simple empirical method worth mentioning here is the Hargreaves" equation (Hargreaves and Allen, 2003):

ET_o=0.0023(TC+17.8)TR0.50 , ...(3)

where

ET_o = evapotranspiration rate [mm/d],

R_a = total incoming extraterrestrial radiation [mm/d],

T_C = temperature [°C],

T_R = daily temperature range [°C].

Unlike the Penman-Monteith equation, the Hargreaves" equation is used for calculating the evapotranspiration from meteorological observations data. But, in contrast, the only

input data required by the latter are the air temperatures (Hargreaves and Allen, 2003). This is the method utilized in the SWMM modeling software .

2.4.1. Evapotranspiration in urban areas

There is a documented relation between the degree of soil imperviousness and evapotranspiration. As the imperviousness increases, the rate of evapotranspiration linearly decreases (Haase, 2009). This is mainly due to reduction in vegetation (Fletcher et al., 2012). Thus, the impact of evapotranspiration is somewhat reduced when comparing the urban regional water balance with the rural areas. When considering stormwater modeling of short rainfall events on urban areas, the impact of evapotranspiration may remain almost negligible.

2.5. Infiltration

Infiltration is defined as „the movement of water from the soil surface into the soil" (Dingman, 1994). It is measured as the infiltration rate , the rate at which water is infiltrating into the soil. The maximum possible infiltration rate of a soil is called the infiltration capacity (or infiltrability). Moreover, the rate at which water arrives to the surface, through precipitation, is named the water-input rate.

The infiltration process can be limited in one of three different ways. (i) The process can be supply-controlled, meaning the water-input rate is less than or equal to the infiltration capacity, and all the incoming water is immediately infiltrated. (ii) The process may be limited by the infiltration capacity when the capacity is exceeded by the water-input rate. (iii)

The rising of the ground-water table to the ground surface level or above may completely stop the infiltration process, setting the infiltration rate to zero. (Dingman, 1994).

Infiltration takes place due to vertical differences in the hydraulic head (Vakkilainen et al., 1986):

H=h_g+h ,..(4)

where

H = hydraulic head [cm],

h_g = elevation head [cm],

h_t = pressure head [cm].

In case the hydraulic head is not constant, its difference over a certain distance is called the hydraulic gradient. The presence of the hydraulic gradient causes water in the soil to flow towards regions of lower hydraulic head. Particularly, if the vertical hydraulic gradient is zero, no vertical flow occurs. In that situation, the water content of the soil column is said to be at the field capacity. If additional water is now brought onto the surface of that soil column, the pressure head, and subsequently, the hydraulic head at the surface increases. This generates a flow downwards from the surface, and so infiltration occurs. (Vakkilainen et al., 1986; Dingman, 1994).

Infiltration rate at a point rarely remains constant during a single rainfall event. Typically, infiltration rates are high at the beginning of an event. Then they tend to promptly decline, asymptotically approaching a constant value. There are several factors influencing the infiltration rate and its temporal changes:

– the rate at which new water arrives to the surface, or, in case of ponding, the depth of ponds;

– hydraulic conductivity of the soil;

– the initial moisture state of the soil pores;

– soil surface inclination and roughness;

– the chemical characteristics of the soil surface;

– the physical and chemical properties of water(Dingman, 1994).

Infiltration over an area is hard to determine. This is because infiltration capacity shows great variations even within a range of few meters. In addition, not all the variations can be explained by soil properties, but they are also related to plant and animal activity as well as small-scale topographic features. (Dingman, 1994).

The hydraulic conductivity of soil can be measured empirically either in the laboratory or in the field (Vakkilainen et al., 1986). There are, also, plenty of literature values for conductivities of different soil types available for use. Either way, after the hydraulic conductivity is known, the water flow rate in the soil can be determined by applying the Darcy"s Law. For vertical unsaturated flow the law is expressed as (Dingman, 1994):

$qz=-K_h(\theta)[(1+d\psi(\theta))/dz]$, ...(5)

where

qz = Darcy flux in vertical direction [cm s^{-1}],

$K_h(\theta)$ = hydraulic conductivity [cm s^{-1}] as a function of soil-water content θ,

(θ) = pressure head [cm] as a function of soil-water content θ,

z = elevation [cm].

As can be noted from above Equation , the hydraulic conductivity and the pressure head are both functions of soil-water content.

A common physically-based theoretical approach for calculating infiltration is the Richards Equation. It is derived from the Darcy"s Law and the principle of conservation of mass. Nevertheless, its non-linear nature allows for only numerical solutions. Hence, the solution can be approximated by the Philip"s Equation, of which typically only the two first terms are considered (Dingman, 1994).

$f(t)=1/2S_pt^{1/2}+Kp$...
.(6)

where

$f(t)$ = infiltration rate [cm $^{s-1}$],

S_p = sorptivity [cm $s^{-1/2}$],

t = time [s],

K_p = hydraulic conductivity [cm $^{s-1}$].

However, the Philip"s Equation has some limitations. It lacks parameters for the characteristics of the rainfall(Dingman, 1994).

The Green-Ampt Model (Green and Ampt, 1911), based on the same principles as the Richards Equation but formulated differently, provides a „more holistic and informative view of the infiltration process" (Dingman, 1994).

The Green-Ampt Equation for infiltration capacity as a function of time is formulated as:

$$f(t) = k_h \times \left[\left(1 + |\psi_f|(\phi - \theta_0)/F(t)\right)\right], t_p < t < t_w \dots\dots\dots\dots\dots\dots..(7)$$

where

$f(t)$ = infiltration rate [cm s-1],

K_h* = hydraulic conductivity [cm s-1],

ψ_f = effective tension at the wetting front [cm],

$(\phi-\theta_0)$ = initial soil water deficit [-],

$F(t)$ = cumulative depth of the wetting front [cm],

t_p = time of ponding, or the instant of the surface layer becoming

saturated [s],

t_w = instant of the entire soil column becoming saturated [s].

The underlying assumptions of the Green-and-Ampt Model include the vertical soil water-content profile to be initially homogeneous, and the wetting front to be considered as a distinct discontinuity in that profile.

As stated above, infiltration capacity can vary greatly even over short distances. According to Kabat et al. (1997), using sufficiently detailed soil data as an input for areal modeling would in most cases result in too large an effort. This could be somewhat resolved by averaging these data over a larger area, thus compromising over the spatial resolution.

2.5.1. Effect of impervious surfaces on Infiltration

In urban areas, impervious surfaces eliminate infiltration and thus increase surface runoff (Fletcher et al., 2012). Reduced infiltration also affects groundwater recharge and results in lowered groundwater levels. The effect may be enhanced through groundwater seepage into drainage networks. Lowered groundwater levels may pose problems for groundwater use and reduce the base flows of urban streams.

2.6. Surface and subsurface flow

Surface runoff is a process that takes place on saturated sloping surfaces. Input of water to a saturated surface causes ponding. If the depth of ponding grows higher than the roughness of the ground , and the ability of surface tension to hold water motionless becomes exceeded, overland flow occurs(See figure-4 below) (Dingman, 1994)

Two main mechanisms can cause the saturation of a surface. First, it can be saturated from above (Horton, 1933). The resulting phenomenon is called Hortonian overland flow. It is considerable especially during intense rainfalls preceded by dry catchment conditions, or areas where the conductivity of the soil surface is low, the latter including urban impermeable areas as well as areas with soil frost. Second, saturation from below may result in saturation overland flow (Dunne, 1978). In that case, the ground-water table rises up to the level of the ground surface, constraining all additional water input to become overland flow(See figure 2).

During a rainfall event with a constant intensity, the overland flow keeps growing over time. Theoretically, the growth continues until the surface of the whole catchment is saturated. Due to irregularities in the small-scale surface topography, the surface runoff tends to form small streams already within minutes (Hyvärinen and Puupponen ,1986).

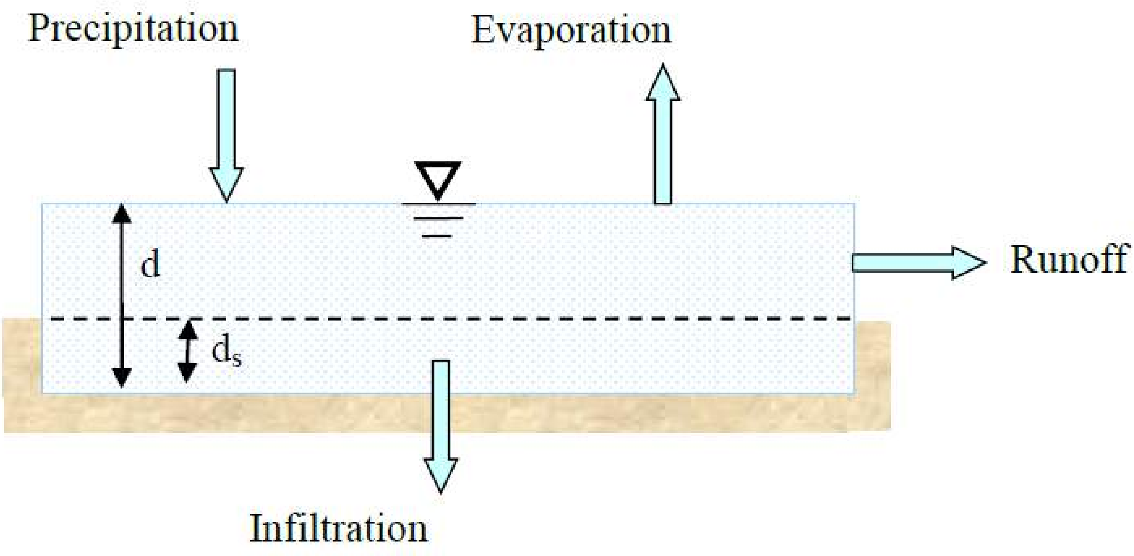


Figure-2. Conceptual view of surface runoff in SWMM, from Rossman (2010).

2.7. Channel flow

As noted above, the overland flow channelizes rather easily. These channels may appear in all kinds of depressions, including gutters, ditches, etc. On urban and sub-urban areas, some or all of the flow is usually collected into underground storm water or combined sewer network. To understand these conduit systems, one should be familiar with the basics of open-channel flow as well as closed-conduit flow. There is a wide variety of flow routing methods for modeling these flows. Some of these are briefly discussed below.

2.7.1. Open-channel flow

Despite of its name, open-channel not only takes place in ditches and streams, but also sewers not flowing full. This includes most stormwater conduits, as they are typically designed to operate well below their full depth, mainly to avoid flooding.

The nature of open-channel flow is in most real cases highly complex. Therefore, before modeling it, some assumptions are usually made (Durrans, 2003). These include one-dimensional flow, hydrostatic pressure distribution, constant water density, and channel length much greater than the flow depth. The Saint-Venant equations, based on mass continuity and the conservation of momentum, are partial differential equations accurately describing flow in these conditions.

The 1-dimensional Saint-Venant momentum equation can be arranged as follows:

$$\boldsymbol{Q = Qn(1 - {}^{1}/_{Sx}\frac{\partial y}{\partial x} - \frac{u}{Sog}\frac{\partial u}{\partial x} -}$$

$$\boldsymbol{\frac{1}{sog}\frac{\partial u}{\partial t})^{1/2}} \text{..(8)}$$

where

Q= actual unsteady flow [m^3/s],

Q_N = flow under normal conditions [m^3/s],

S_0 = slope of the channel bed in longitudinal direction [-],

u = flow velocity in the longitudinal direction [m/s]. (Durrans, 2003)

Various approximations of the Saint-Venant equations have been developed; kinematic, diffusion, and gravity waves.

In some applications, even the steady-flow routing can be sufficient. It directly translates and sums the inflow hydrographs to acquire an outflow hydrograph. The relation between discharge and flow depth can then be solved using the Manning equation (Durrans, 2003):

$v=(C_f R^{2/3} S_o^{1/2})/n$,...(9)

where

V= flow velocity [m/s],

Cf = unit conversion factor [m1/3/s]

n = friction factor [-],

R = hydraulic radius [m],

S_0 = channel slope [m/m].

The friction factor n in the above Equation is largely defined by the surface material of the streambed or the pipe, and plenty of n values for different materials appear in literature.

In storm water modeling, it is also important to understand overland flow. Overland flow refers to thin sheet-flow that occurs before the runoff gets channelized due to surface irregularities. The length of true overland flow is rarely more than 100 m. The open-channel flow routing principles can typically be applied to overland flow as well. Overland flow modeling may however need huge simplifications as the irregularities of the land surface are typically substantial compared to the thickness of the overland flow layer. For Shashemene there are some Channels (Drainage Networks) used to rout storm water from the city which are not sufficient to remove all storm drainage.

2.7.2. Closed-conduit flow

Opposed to open-channel flow, closed-conduit flow occurs in pipes that are full with water, i.e. there is no free water surface in the cross-section of the pipe.

The pipe flow is typically modeled based on conservation of energy. According to the energy equation the sum of pressure, elevation, and velocity heads must equal between two cross-sections of a pipe, excluding energy losses and inputs on the way. Thus, evaluating the losses is a central part of the modeling. These losses can be classified into either frictional or local losses, of which the first mentioned are, in most cases, of higher significance (Durrans, 2003).

The energy loss due to friction can be solved from the Darcy-Weisbach equation:

$h_L = f\, V^2/D2g$,..(10)

where

h_L = head loss due to friction [m],

f = friction factor [-],

L = pipe length [m],

D = pipe diameter [m],

V = cross-sectional averaged flow velocity [m/s],

g = acceleration of gravity [9.81 m/s2].

Several other methods of computing the frictional loss exist, too, including the Manning equation , the Chézy equation, and the Hazen-Williams equation. (Durrans, 2003)

2.8. Flooding hazards

Topographically, Oromia is the Region with a highland/mountainous and lowland . During the major rain, the major perennial rivers as well as their numerous tributaries carry their peak discharges (A, 2006). This causes flood; either flash (excess rains falling on upstream watersheds and gush downstream with massive concentration, speed and force suddenly) or riverside (happens usually due to increased water beyond the bank of the river). Usually, flash floods result in a considerable toll and devastation when they pass across or along human settlements and infrastructure concentration. The recent incident that the Dire Dawa City experienced in 2006 was typical of flash flood that collected from the upper catchments of East Hararghe highlands. On the other hand, much of the flood disasters in different area were attributed to rivers that overflow or burst their banks and inundate downstream plain lands.

This condition still observed in , Bale and Arsi ,in 2016 G.C that cause the loss of asset more than a millions , animals more than 6000 (six thousands) and loss of life of more than 20 human in 2016. Shashemene was one of the city affected by flood hazard this year with huge land slide due to this flood hazards in near villages to the downstream of the catchment outlet (Observed, 2016).

2.8. 1. Shashemene town storm water and drainage problem

The problem of ineffective land drainage occurs when inflow into the system exceeds outflow, so that there is a build-up of water over a period of time. This may occur rapidly over a few hours in response to heavy rainfall, or it may be a gradual rise in water table during wet periods. Flooding occurs when a channel has inadequate capacity to convey the amounts of water flowing into it, or when flood defense works fail. Thus, the solutions to land drainage problems invariably involve either control of inflow into the system or works to improve the capability of the drainage channels to carry flows through the system. The basic objective is to reduce the frequency and/or the intensity of inundation to acceptable levels, appropriate for the situation.

Shashemene town drainage problem is dominantly due to urbanization that change land surface characteristic without proper coverage of drainage line and blocking of natural waterway. At present time, an affecting storm water source of Shashemene town is totally rainwater runoff that flows over adjacent land of rural area and from an internal developed land of the town.

As the topography of the town is very flat of mild slope at a distant from the river bank, it hinders for a faster movement of runoff owing to the increased water volume over the surface of the town by taking long time to reach outlet or flow channel. These cause a crucial problem on the habitat"s life and properties by flooding on the town"s land surface and even cause traffic congestion by flowing over the road surface. e.g on the road of Alaba outlet around Maja Safar (See Figure 3a and 3b on Appendix).

Figure-3a. Land slide in Shashemene area (Captured in June,2016)

Gogeti stream is the most dominant channel by causing flood hazard. It originated from far place at Gumbicha cliff owning large watershed and accommodate storm of intense rainfall around the stream and flows through the town. The stream becomes shallow and narrow that almost diminishes loosing the capacity to convey the amounts of water flowing into it due to which flooding occurs starting at kebele 03, 04 to Maja area.

3. CONCEPTUAL MODELLING

3.1 Rainfall-runoff modeling

Modeling can be defined as simulating the natural world with a model representing a part of that world. Modeling is using models-physical, mathematical, or otherwise logical representation of a system , entity , phenomenon, or process as a basis for simulating the natural world with a model representing a part of that world. Methods for implementing a model (either statically or) over time to develop data as a basis for managerial or technical decision making. Mathematical models are „explicit sets of equations and numerical and logical steps", converting numerical inputs to numerical outputs (Dingman, 1994). It helps getting information about how something will behave without actually testing it in real life. The principal techniques of hydrological modelling make use of the two powerful facilities of the digital computer, the ability to carry out vast numbers of iterative calculations and the ability to answer yes or no to specifically designed interrogations. Applying these facilities, mathematical models are built up by careful logical programming to describe the land phase of the hydrological cycle in space and time.

Hydrological models are divided broadly into two groups; the deterministic models seek to simulate the physical processes in the catchment involved in the transformation of rainfall to stream flow, whereas stochastic models describe the hydrological time series of the several measured variables such as rainfall, evaporation and streamflow involving distributions in probability. In providing information for the design engineer, a combination of the deterministic and stochastic approaches is proving to be the most successful.(M. Shawl, 1994).

According to Karvonen and Kettunen (1986) and Dingman (1994), conceptual modeling consists of the following steps: (on Figure 4)

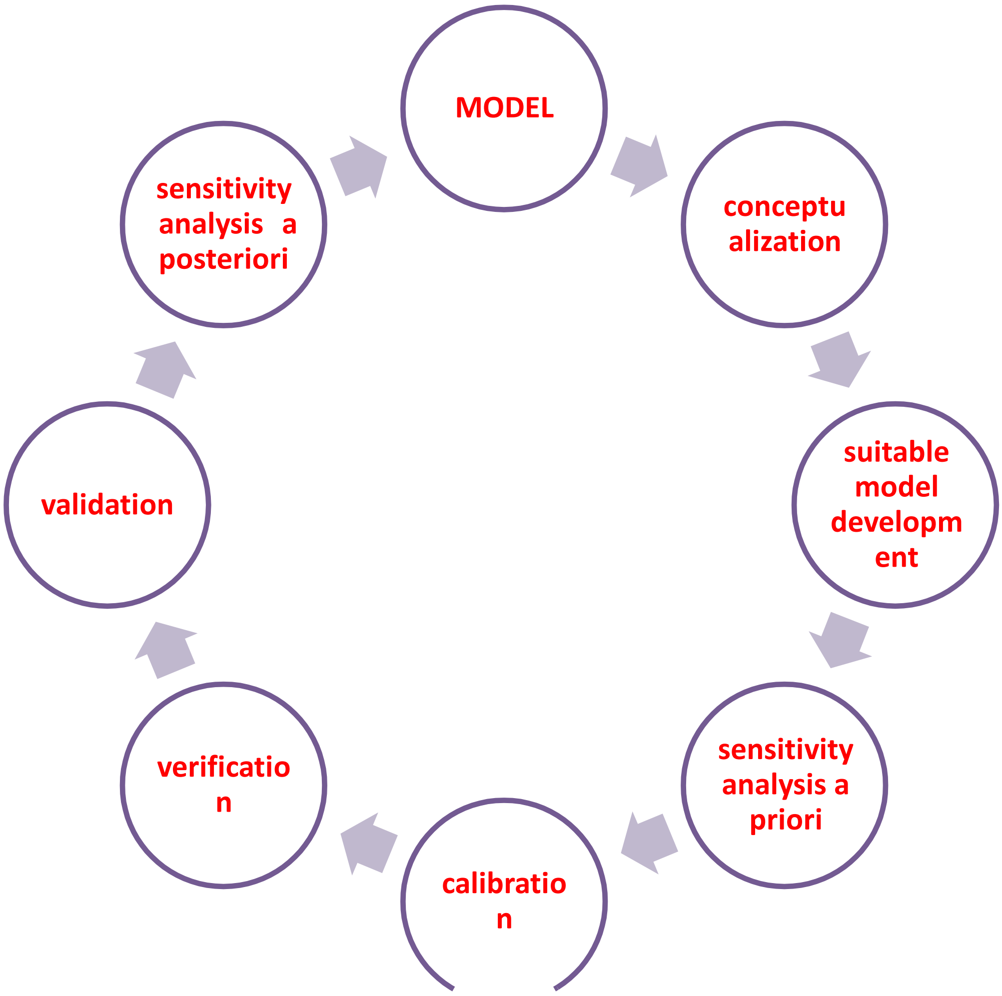


Figure 4. Physical Representation of Modeling

Rainfall-runoff modeling predicts the hydrological response (runoff) to a certain input (precipitation), usually as a function of time.

When applying the systems approach, the systems modeled can be classified into two categories: linear and non-linear.

Spatial boundaries of the system are typically defined according to catchment borders. Conceptually, the modeled system can either include all the hydrological processes in a catchment system, or be restricted to a surface runoff system. The difference is that the surface-runoff system uses only the fraction of precipitation that actually causes runoff as input, known as the effective precipitation. Another difference is that the catchment system, due to its complexity, is evidently non-linear. In contrast, the surface-runoff system can at times be reasonably approximated as being linear. (Diskin, 1981)

3.1.1. RATIONAL METHOD

For urban catchments that are not complex and are generally 50 hectares or less in size, it is acceptable that the design storm runoff be analyzed by the Rational Method. This method was introduced in 1889 and is still being used in most engineering offices in the United States. Even though this method has frequently come under academic criticism for its simplicity, no other practical drainage design method has evolved to such a level of general acceptance by the practicing engineer. The Rational Method properly understood and applied can produce satisfactory results for urban storm sewer, drainage capacity and small on-site detention design(Drainage Design Manual - 2002).

Rational Formula

The Rational Method is based on the Rational Formula:

$$Q = CIA \quad \text{...} (11)$$

in which:

Q = the maximum rate of runoff (SI units)

C = a runoff coefficient that is the ratio between the runoff volume from an area and the average rate of rainfall depth over a given duration for that area

I = average intensity of rainfall in inches per hour for a duration equal to the time of concentration, t_c

A = area

The time of concentration is typically defined as the time required for water to flow from the most remote point of the area to the point being investigated. The time of concentration should be based upon a flow length and path that results in a time of concentration for only a portion of the area if that portion of the catchment produces a higher rate of runoff.

Rational method has some major limitations

1. The rate of runoff resulting from any rainfall intensity is a maximum when the rainfall

intensity lasts as long or longer than the time of concentration. That is, the entire catchment area does not contribute to the peak discharge until the time of concentration has elapsed.

This assumption limits the size of the drainage basin that can be evaluated by the Rational Method. For large catchment areas, the time of concentration can be so large that constant rainfall intensities for such long periods do not occur and shorter more intense rainfalls can produce larger peak flows. Further, in semi-arid and arid regions, storm cells are relatively small with extreme intensity variations thus making the Rational Method inappropriate for catchment areas greater than 50 hectares.

(2) The frequency of peak discharges is the same as that of the rainfall intensity for the given time of concentration.

Frequencies of peak discharges depend on rainfall frequencies, antecedent moisture conditions in the catchment area, and the response characteristics of the drainage system. For small and largely impervious areas, rainfall frequency is the dominant factor. For larger drainage basins, the response characteristics control. For catchment areas with few impervious surfaces (little urban development), antecedent moisture conditions usually govern, especially for rainfall events with a return period of 10 years or less.

(3) The fraction of rainfall that becomes runoff (C) is independent of rainfall intensity or

volume.

This assumption is only reasonable for impervious areas, such as streets, rooftops, and parking lots. For pervious areas, the fraction of runoff does vary with rainfall intensity and the accumulated volume of rainfall. Thus, the application of the Rational Method requires the selection of a coefficient that is appropriate for the storm, soil, and land use conditions. Many guidelines and tables have been established, but seldom, if ever, have they been supported with empirical evidence.

(4) The peak rate of runoff is sufficient information for the design.

Modern drainage practice includes detention of urban storm runoff to reduce the peak rate of runoff downstream. Using only the peak rate of runoff, the Rational Method severely limits the evaluation of design alternatives available in urban and in some instances, rural drainage design (ERA, Drainage Design Manual - 2002)

3.2. SWMM

Rainfall-runoff models play an important role in urban water resource management. The EPA developed software for Hydraulic and Hydrologic modeling. The Storm Water Management Model (SWMM) developed by the US EPA is one of the most widely used dynamic rainfall-runoff model for analyzing quantity and quality problems associated with urban drainage system. SWMM was selected for; (1) SWMM is widely used in analysis and design of storm water drainage systems of urban areas. (2) Recently, USEPA released the version 5.1 of the SWMM software for Microsoft Windows, which has the capability of both single-event and continuous simulation for the prediction of flows and pollutant concentrations. The SWMM5.1 is a free software program, thus it is easily available to small municipalities and companies. Hence, SWMM5.1 use is expected to become widespread among end-users. It is one of the most successful models produced by the U.S. Environmental Protection Agency (EPA) for the water environment. Originally developed in 1969-71, it has withstood the test of time and continues to be widely used worldwide for analysis of quantity and quality problems related to storm water runoff, combined sewers, sanitary sewers, and other drainage systems in urban areas, with many applications in non-urban areas as well and has since then been upgraded several times. The current version (number 5.1) was completely re-written by the U.S. EPA and a consulting firm of CDM, Inc. (Rossman, 2010).

3.2.1. Environmental Compartments

SWMM is a full dynamic wave simulation model used for single event or long-term simulation of runoff quantity and quality, primarily from urban areas. Version 5.1 is a complete re-write of the previous release, running under Microsoft Windows and providing an integrated environment for editing data, running hydrologic, hydraulic and water quality simulations, and viewing the results. It conceptualises a drainage system as a series of water and material flows between several major environmental compartments.

3.2.2. The land surface compartment

The movement of water bodies occurring on the land surface are main concern of SWMM. For modeling, the land surface is divided into small, sufficiently homogeneous subcatchments, each catchment draining to a single discharge point. All of the sub catchments have their own sets of hydrological parameters such as imperviousness and depression storage. Based on these, several hydrological phenomena are simulated during every time step. (Rossman, 2016).

SWMM offers a selection of three different built-in infiltration models. These are (a) the Horton"s equation (Horton, 1933), (b) the Green-and-Ampt method (Green and Ampt, 1911), and (c) the Curve Number method. Infiltration only takes place on the pervious fraction of the subcatchment, defined by the imperviousness parameter(see figure-4 ,) input for SWMM Modeling).

Q=1.49W/$(d-dp)^{5/3}$⁄$S_0^{1/2}$, ...(12)

where

Q = subcatchment outflow [m^3/s],

W = subcatchment width [m],

n = Manning"s roughness coefficient

d = water depth [m],

dp = depth of depression (retention) [m],

S = slope [%].

Surface runoff generated at the source areas is defined to flow either into another subcatchment or into a drainage system entry point (Rossman, 2016).

3.2.3. The Conveyance compartment

The other major part of SWMM is the transport Conveyance. It describes the hydraulic routing of runoff and possible external inflows through a network of pipes and channels, also known as conduits. These conduits are the links of the drainage network, joined together at junction nodes, which can represent manholes, pipe connection fittings, etc. Typical conduit parameters include invert elevations at both ends, the conduit length, the Manning's roughness coefficient n, and cross-sectional geometry. Similarly, junction nodes have parameters such as invert elevation and depth from the ground surface. There are also other possible types of nodes like, flow dividers, storage units, pumps, and flow regulators. (Rossman, 2016).

SWMM offers three different options of flow routing: (i) steady flow routing, (ii) kinematic wave routing, and (iii) dynamic wave routing.

The choice of the routing method affects the accuracy of the results, as well as the time taken by running a simulation(Rossman, 2016).

3.3. Data Calibration

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 accurate spatial data and can be reasonably defined even without calibration. Those include sub catchment 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).

3.3.1. Subcatchment Discretization

Most study areas will require some level of discretization into multiple subcatchments in order to properly characterize the spatial variability in overland drainage pathways, surface properties, and connections into drainage pipes and channels. Discretization begins with the identification of drainage boundaries (drainage divides) using a topographic map, the location of major drainage canal inlets using a drainage system map, and the selection of channel/pipes to be simulated "downstream" in the model. In an urban area, drainage divides based strictly on topography might not apply, since the subsurface drainage network might transport water in a direction opposite to the surface gradient.

Hence, drainage boundaries must be determined with the aid of both a topographic map and sewer plans.

3.4. Previous studies

3.4.1. The Use of ArcGIS on rainfall-runoff modeling

This study mainly concentrates on the parameterization of a SWMM rainfall-runoff application. The first step in such a parameterization is the catchment delineation and subdivision. Chen and Tucker (2003) compared different approaches for catchment delineation, to burn a stream network into a digital elevation model (DEM) and to use the Watershed tool in ArcGIS. They concluded that GIS is a powerful tool for catchment delineation if there are comprehensive and accurate spatial data sources available and the details of the sewer system are correctly accounted for. They also add that manual fine-tuning is often necessary after the automated delineation process.

Based on the spatial resolution, GIS-based catchment modeling can be divided into distributed (or high-resolution) and aggregated (or low-resolution) approaches. A distributed model accounts for all minor spatial variations within the study area while an aggregated model excludes and generalizes details of the input data. Highly distributed models are typically used e.g. for modeling event peak-flows, whereas the more aggregated approaches are mainly suitable for studying large-scale processes like climate change as their limitations are less relevant in long-time-scale modeling.

Park et al. (2008) performed SWMM simulations with varying levels of sub catchment aggregation. They concluded that the simulated surface runoff was not affected by the spatial resolution of the model. On the other hand, accumulated pollution loads were reduced with an increasing level of aggregation. The peak flows appeared at slightly different time instants but otherwise the hydrograph was not affected by the model aggregation. Similarly, Ghosh and Hellweger (2011) did SWMM runs for 50 storm events. Their results indicated that the annual runoff is not dependent on the spatial resolution of the model. The effect of spatial resolution on simulated peak flows was altered. While for small storms the peak flows increased with an increasing level of aggregation a decrease was found for large storms.

On the contrary, Smith et al. (2005) found that the aggregation level of the catchment affected the difference between measured and simulated runoff volumes. They recommended minimizing those differences through calibration of model parameters such as the flow width.

There have also been several studies regarding the aggregation level of other rainfall-runoff models similar to SWMM. Thompson and Cleveland (2009) concluded that fully-distributed modeling with HEC-HMS is not feasible in cases where no calibration data is available. The possibilities of scaling the input data of the distributed KINEROS model to match different aggregation levels were studied by Thieken et al. (1999). The outcome was that the flow length could be used as a scaling factor for adapting the same model to catchment delineations of different scales. Zhang et al. (2013) summarized several previous studies and concluded that the flood volume is insensitive to the degree of catchment subdivision. They also noted that only a few studies have concentrated on the effect of catchment subdivision on water balance components like evapotranspiration and infiltration. Their results with a HEC-HMS model showed that overland flow length increases with increasing subcatchment area, infiltration parameters are independent of the model aggregation level, the quality of results decreases if the number of subcatchments is too large or too small, and unlike in previous studies, the aggregation level affects the components of the water balance. Zhang et al. (2013) also conclude that catchment subdivision is useful if detailed data on parameter variations between the subcatchments is available.

Besides the surface runoff processes, also the conveyance system is subject to generalization.

3.4.2. Estimating and calibrating SWMM parameters

A SWMM application includes numerous different parameters, of which several vary from subcatchment to subcatchment. These parameters can be classified to measured parameters (e.g. subcatchment area; Canal lengths, Canal shapes, bed slopes, and width; manhole type; soil types; land-use types; and rainfall depth) and inferred parameters (e.g. flow width; infiltration parameters; Manning"s n for pervious and impervious areas; depression storage for pervious and impervious areas; imperviousness; and Manning"s n for conduits). The measured parameters are typically easier to obtain, while the inferred parameters usually need to be calibrated. Nevertheless, also the first mentioned may sometimes involve large uncertainties arising from inaccuracies in sub catchment delineation.

However, if no runoff measurements required for calibration exist, literature values can be found for several of these parameters. Many parameter values are suggested in the SWMM Revised User"s Manual (Rossman, 2016). In contrast, other parameters such as flow width, hydrological slope, and imperviousness have to be obtained from spatial data.

Detailed spatial data on land cover types etc. is important for calibrating SWMM applications for urban catchments . Because such data is not always available, there have been attempts to reduce the complexity of the calibration process by concentrating only on the parameters the model is most sensitive to. The problem is, however, that SWMM is sensitive to different parameters in different catchments (e.g. Beling et al., 2011). This highlights the importance of always performing a sensitivity analysis before model calibration. In addition, the method of sensitivity analysis affects the obtained model sensitivity to different parameters (Jacobson, 2011).

Whether calibration could be left undone in some cases is an interesting question. According to Jang et al. (2007), even a non-calibrated SWMM model performs better than an ordinary hydrograph when modeling urbanizing (or urbanized) areas.

Overall, it seems there have been experiments with calibrated SWMM models of a low spatial resolution, as well as with non-calibrated high-resolution SWMM models. No studies were yet found where a low-resolution model was applied without calibration to a large urban area for continuous simulation. Thus, the application of this approach is definitely both interesting and challenging.

4. MATERIALS AND METHODs

4.1. MATERIALS

Materials(tools) that were used to carry out the modeling, were ArcGIS 10.1 and SWMM5.1.

4.2. METHODOLOGY

GIS was applied in the selection of appropriate study sites, drainage network preparation, delineation and catchments division into subdivide for the study(Fig.5).

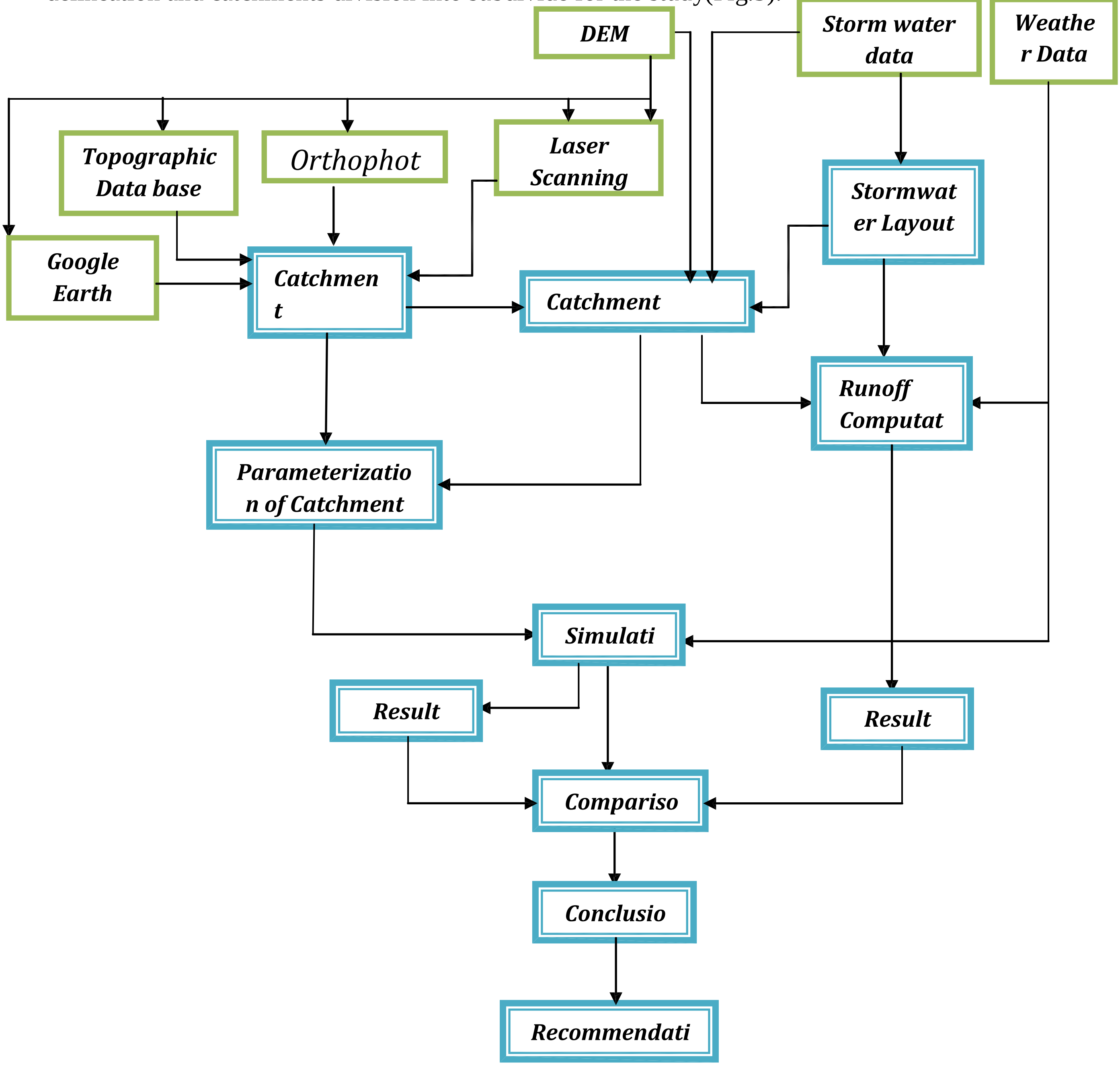


Figure 5. A flowchart of the methodology of the study

4.3. Some Hydrological condition in the Shashemene

Local hydrology is largely affected by the runoff upper part of the city from Abaro mountain and from the Awasho rural kebele in the direction of Kofale town in left to the study area. Also the geological and topographic conditions in the study area are dictated by According to OUPI(Oromia Urban Planning Institute), 2010, geologically, the largest part of Shashemene town is covered with volcanic material. The hill chain (Abaro) in the south-western part of the town is composed of basalts, and it is covered with volcanic topsoil materials of about one to two meters thick volcanic soil formations(See Table 1).

Table 1. Some Hydrologic condition in the Shashemene Town for 10 years period

Parameter		Value(MM)	Period
Mean Annual PPt	Min	25.58	2002
	Mean	59.35	1998-2007
	Max	95.88	1998
Mean Monthly PPt	Min	10.97	December, 1998
	Mean	59.35	
	Max	105.43	April 2000
Daily PPt	Min	0	
	Mean	3.4	
	Max	53.3	June 1998

Therefore the city is being affected by flood hazards from the upper parts to the rural area of the lower part. These research efforts have concentrated on storm water runoff measurements from different types of urban areas and detailed SWMM model parameterization and calibration for the Shashemene town.

4.4. Available spatial data

Publicly available spatial datasets that was used in this research were from different sources including from government office. Spatial data used were mainly acquired from the National land survey of Ethiopia(NLSE) through their open data files from different Sectors working in the city.

4.4.1. Orthophotos

The spatial data provides color orthophotos with a terrain resolution of 30 meters (NLSE, 2013). The orthophotos are aerial images that have been orthorectified to geometrically correspond with a map. These photos are a good reference for background data visualizations. More detailed aerial photos would have been provided by Shashemene City(see figure 6). Orthophotos proved to be adequate for this study.

Figure 6. Ortho photos of Shashemene Town.

4.4.2. DEM (Digital Elevation Modem)

The digital elevation model (DEM) provided by NLSE is a raster dataset with a 30m grid cell size. Each grid cell contains a value for the mean ground surface elevation of the cell. The height accuracy is declared to be less than 30 meters. The dataset has been computed from airborne laser scanning data (see figure 12) with a minimum point density of 0.5 points per square meter. Buildings are not depicted in the model. Instead, building cell values have been set according to a surface approximating the ground level at the site of the building. For this research, the digital elevation model 10 m was used in ASCII Grid format from the (National land survey of Ethiopia, 2013) (see Figure 7).

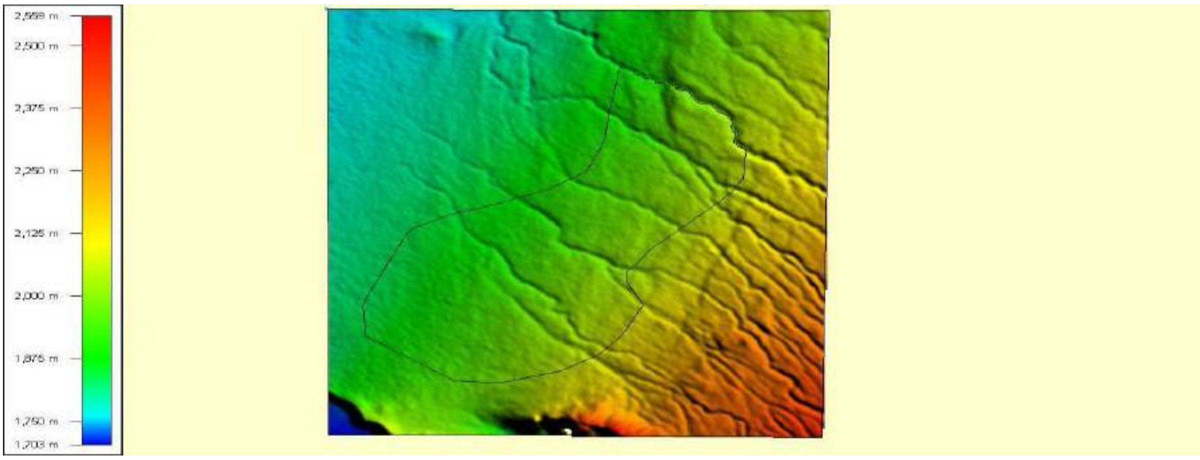


Figure 7. The digital elevation model (DEM).

4.4.3. Laser scanning data

The NLS laser scanning data is a three-dimensional (x, y, z) point dataset representing the ground surface as well as objects on top of that surface. Point categories include ground points, low vegetation points, water points, stream points, bridge points, etc. Points not suitable for any other classes are categorized as unclassified NLS (National survey of Ethiopia, 2013)(See Fig. 8).

Figure 8. Laser Scanning of Shashemene city

4.4.4. Topographic database

The topographic database of the NLS includes all types of objects that may appear on a typical base map. Each object belongs to a class such as traffic route networks, buildings & constructions, land use, water systems, elevations, and administrative borders. There are also sub-classes. Like , buildings are further classified according to the usage and the number of stories. Similarly, traffic route networks are classified as roads, streets, light traffic routes, railroads, etc. Streets and roads are stored in the database as linear features with a class number indicating width and the number of lanes. This is the most accurate nation-wide traffic route network dataset. (National land survey of Ethiopia, 2013) (See Fig. 9).

Figure 9. Topography of the Shashemene town

4.5. Stormwater system layout

Data describing the properties of the stormwater system was received from West Arsi Zone, the storm water runoff in the Shashemene area. The data had been imported into ESRI shape file format assumingly from the company"s database. The data received consisted of only feature classes, including:

- stormwater drains drainage ditch (as polyline features);
- manholes and junctions (as point features);
- attribute data belonging to nearby features (as point features) (See Fig. 10).

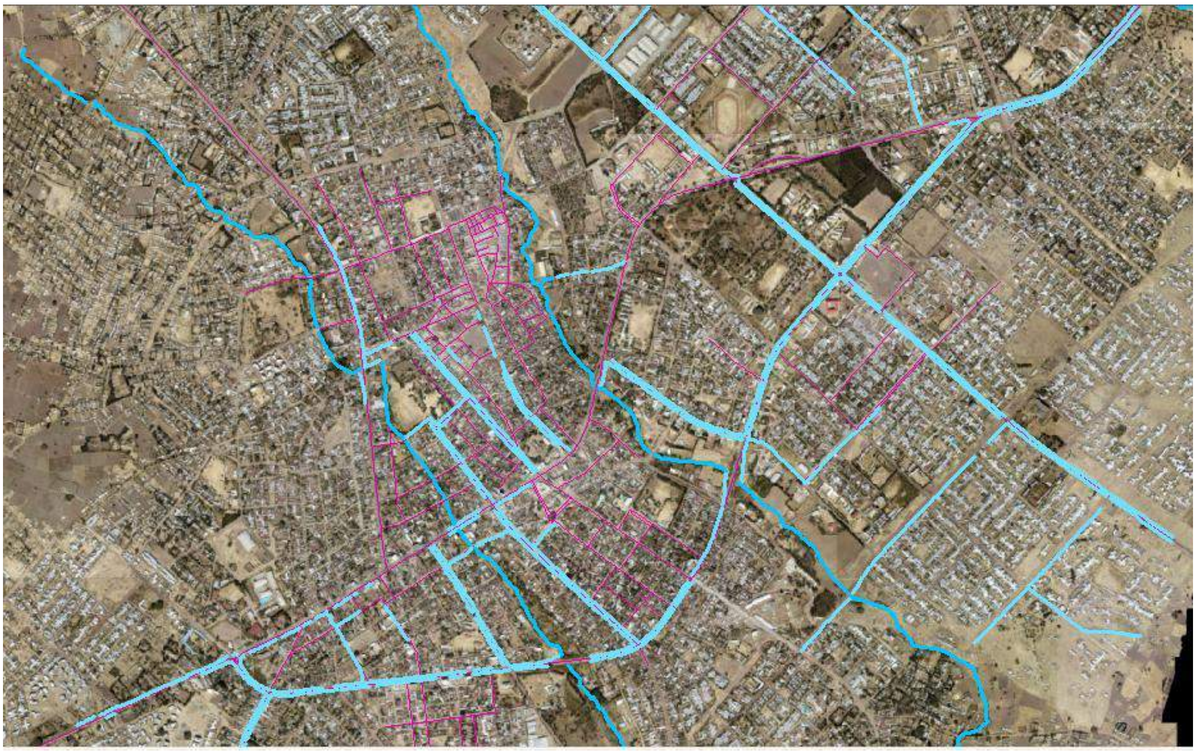


Figure 10 ,Part of the stormwater drainage network data(Ditch, with light blue)

In other words, most of the necessary information such as canal dimension, canal elevations, and manhole elevations, was not attributed to the objects forming the drainage network to separate point features in the proximity of the actual network objects. Several objects also completely lacked some or all of the basic attributes required for model parameterization, and there were occasional gaps in the geometrical continuity of the drainage network. Furthermore, many canals appeared to have imaginary duplicates in the data(see Figure 10). The features on the private properties and the ones on the street area had typically been stored in corresponding different feature classes. Overall, in terms of its quality the data was not well-suited for the purpose of this study.

4.6. Weather observations data

Weather data was obtained from the Ethiopia Meteorological Agency (EMA). Since Shashemene town has no its meteorological station, the data is adapted from the nearby towns Arsi Negele. These may be due to the fact that both towns have almost the same topography. According to the data obtained from Federal Meteorological Authority both towns do have similar rain fall distribution.

The data includes observed values for air temperature [°C], relative humidity [%], wind speed [m/s] and precipitation [mm] , at the Arsi Negele meteorological station in West Arsi Zone. The time period covered by the observations is nearly ten years, from 2005 to 2014 G.C. The data has been processed as described below graph.

Annual precipitation shows great variability during the observation period. The highest annual rain fall in the distribution is recorded in 2005, 2007, 2008 and 2014 comparatively , the year 2007(G.C) was record-breaking throughout Shashemene in these ten years.(Ethiopia Meteorological Agency, 2014), with an annual precipitation of 257 mm. On the contrary minimum annual rain fall within the past ten years was seen in 2009(mean ,25.58mm). In general the town has got rain fall throughout the year with significant amount of mean monthly range of rain fall. The range of mean monthly and total annual of rain fall in the town is about 94.4mm and 837.2mm respectively. (See Fig. 11).

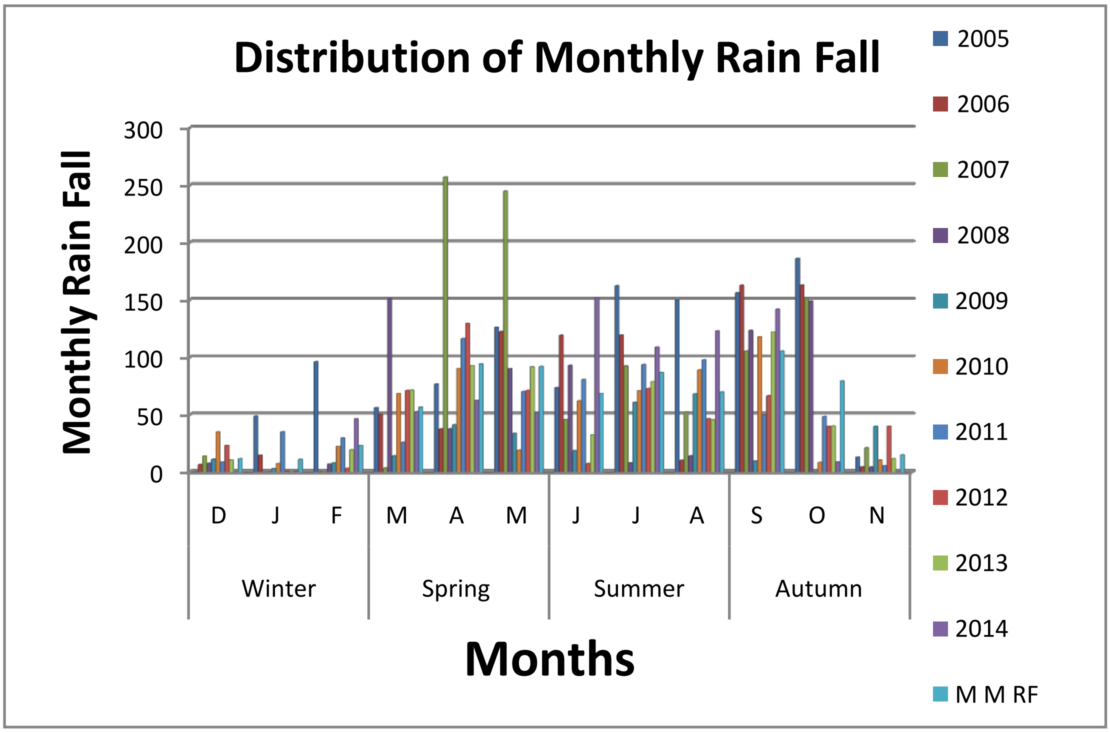


Figure11. Monthly precipitations of Shashemene , from 2005 to 2014.

Accordingly both monthly rain fall and ARFI of Shashemene town for the last ten years is not the same. Monthly rain fall intensity of the station was about 54.1mm within only 5 rainy days. This shows that for every one minute, about 10.8 mm rains were occurred. In the same manner in June 2005 about 8.5mm intensity of rain was recorded at every one minute. In general the possibility of high rain fall intensity in the town may be resulted from high annual rain fall. In here, the intensity of rain fall has direct and inverse relation with duration. Thus the rain fall of long duration in most case has low intensity. The rain fall of high intensity will be of short duration. With this condition, the possibility of runoff in 2008 was high as the rain fall duration in that year was low. The number of rainy days in that year was also smaller. According to observation under taken the maximum rainy days in the observation were 142 in 2005 and followed 134 days in 2006. Otherwise the numbers of rainy days in observation in all case are less than 125 days in all years. The duration of rainy time and the length of time in general are also short. As a result the town may expose to runoff and flooding condition. Off course with this line elements like temperature of the ground, soil characteristics, vegetation cover, geology, slope and volume of water in the soil are constant can be seen as major factors.

On the other hand the magnitude of the probability of occurrence of such rain fall in the next year in the town can be also calculated by using the formula of (M / n+1) 100 , Where "n" is number of observation, and "m" is rank order. In order to come up with the solution, first mean annual rain fall corresponding to the year and ranking the result from largest to smaller is the precondition to generate the probability of occurrences in the next year .

Table 2. Rainfall Intensity and Probability

Year	**Annul RF(mm)**	**Number of Rain Fall Days**	**Hourly Rain Fall (mm)**	**Rank**	**Annul RF intensity (mm/hr)**
2005	1144.1	142	234.6	2	8.06
2006	809.2	134	184	5	6.04
2007	986.3	123	188.6	3	8.02
2008	684.4	62	96.4	1	11.04
2009	306.9	63	71.9	10	4.87
2010	599.7	113	135.9	8	5.31
2011	660.1	119	186.6	7	5.55
2012	570.4	117	127.9	10	4.88
2013	614.6	121	155.3	9	5.08
2014	746.8	119	224.9	4	6.28

It is reasonable to slightly question the suitability of weather data from Kofale to describe the weather conditions in the study area. According to a local meteorologist at (Kofale, 2005), the Kofale neighborhood is characteristically colder than the city center of Shashemene. Kofale is less densely built and thus the urban heat island effect plays there a reduced role. Also, Kofale is located in the river valley at a level approximately 20 meters lower than the city center, and the Eessa ridge induces different microclimates on each side of the ridge. On a clear and calm weather, the measured air temperature in the city center may be even 3 to 4 °C warmer than at Kofale. A new meteorological station is actually planned to be installed in the city center of Shashemene to achieve weather forecasts better representing the city area. Currently the station is from Arsi Negele, neighboring town (see Table 2).

5. RESULTS AND DISCUSSION

5.1. Catchment and subcatchment delineation

5.1.1. Preliminary Catchment delineation

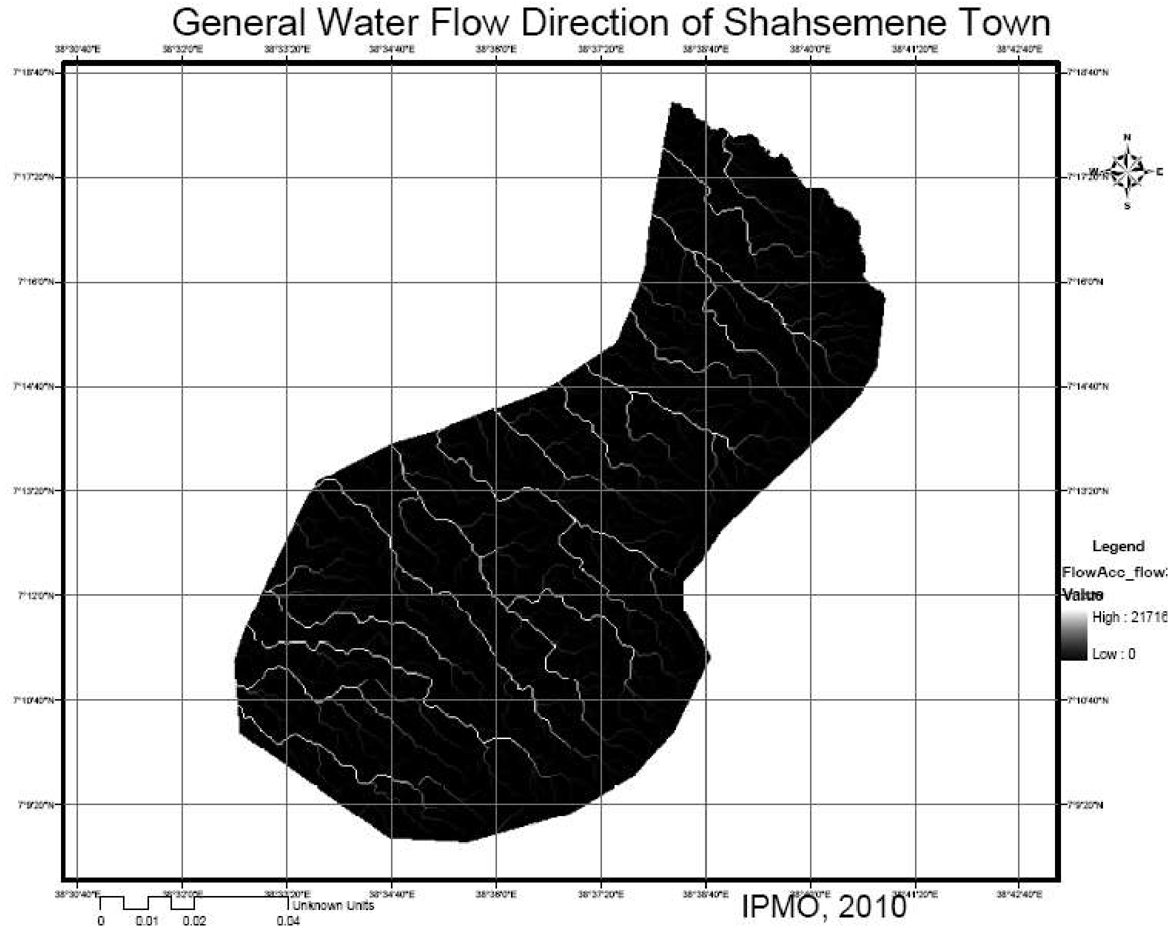


Figure-12. General Water flow directions(Source: Field Survey Result January, 2010, OUPI)

The visualization of the stream network in the FAC allowed for the determination of the streams draining into the River in to the Shashemene municipality area(see Figure-12).

Finally, the Watershed tool (Esri, 2012) was used to identify catchments contributing to the flow at each of the drainage points. The tool creates a raster where cell values indicate to which point cells are draining to.

Visual comparison was carried out to observe the similarities and differences of the catchment delineation and the stormwater drainage network layout . In a few places, the stormwater drains crossed the catchment borders. This is one signal that catchment delineation of an urban area should not only base on terrain topography but should also consider the stormwater system. (see Figure-13).

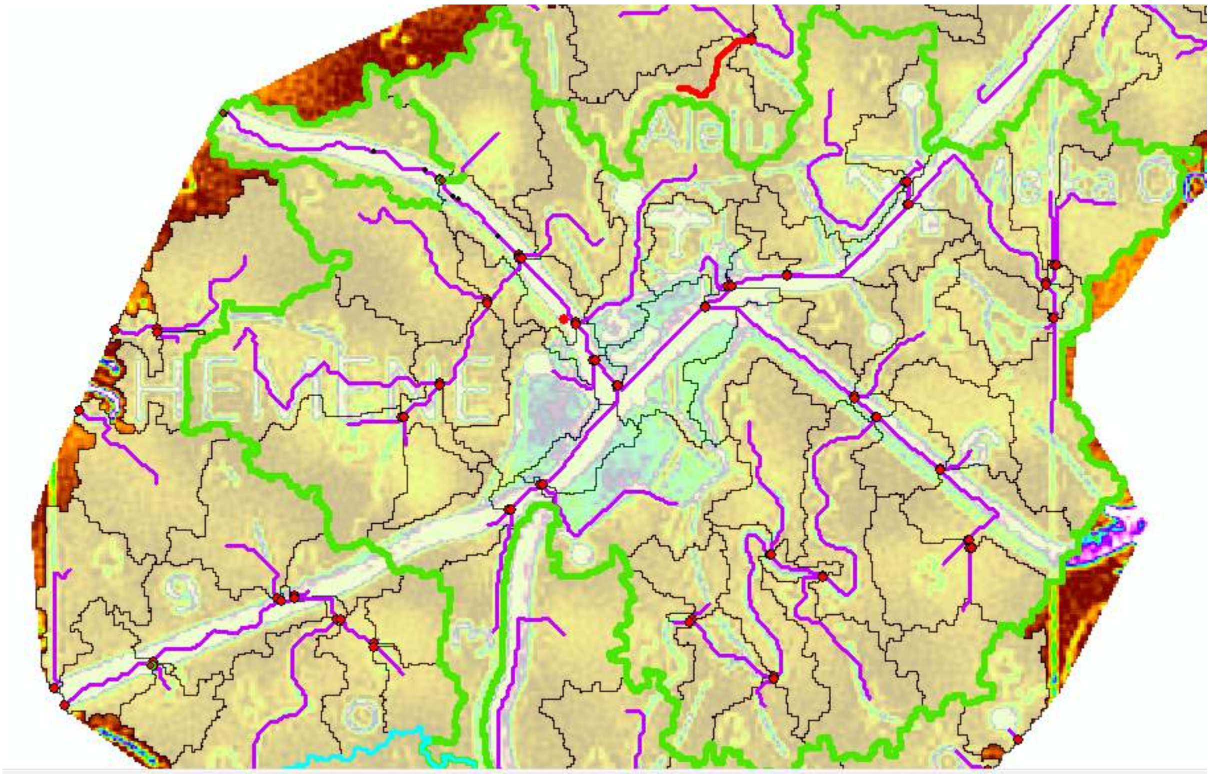


Figure-13. Preliminary catchment delineation (colored areas) overlaid with stormwater drainage network layout for visual comparison.

5.1.2. Detailed catchment delineation

The use of DEMs for watershed and stream delineation in urban areas results in stream networks not correspondent with reality due to the negligence of the stormwater drainage network and flow obstructions. This issue can be avoided by incorporating vector stream or drainage canal data to complement the original DEM.

To achieve a correct catchment delineation, all sinks noted in the coarse delineation phase needed to be drained by burning the DEM with culverts and stormwater drains. The only exceptions could have been depressions that also in reality are areas of internal drainage. No surface runoff is formed on such areas but all excess water is either evaporated or infiltrated.

Although different holes exist in Shashemene, the DEM showed there were none of them in the study area(see figure 20).

The stream network data was converted into raster format using the Polyline to Raster tool (Esri, 2012). Cell size was set to 30 m and the raster grid was snapped to the DEM.

The threshold value was set to 10620 cells , resulting in 52 catchments (see Figure 14).

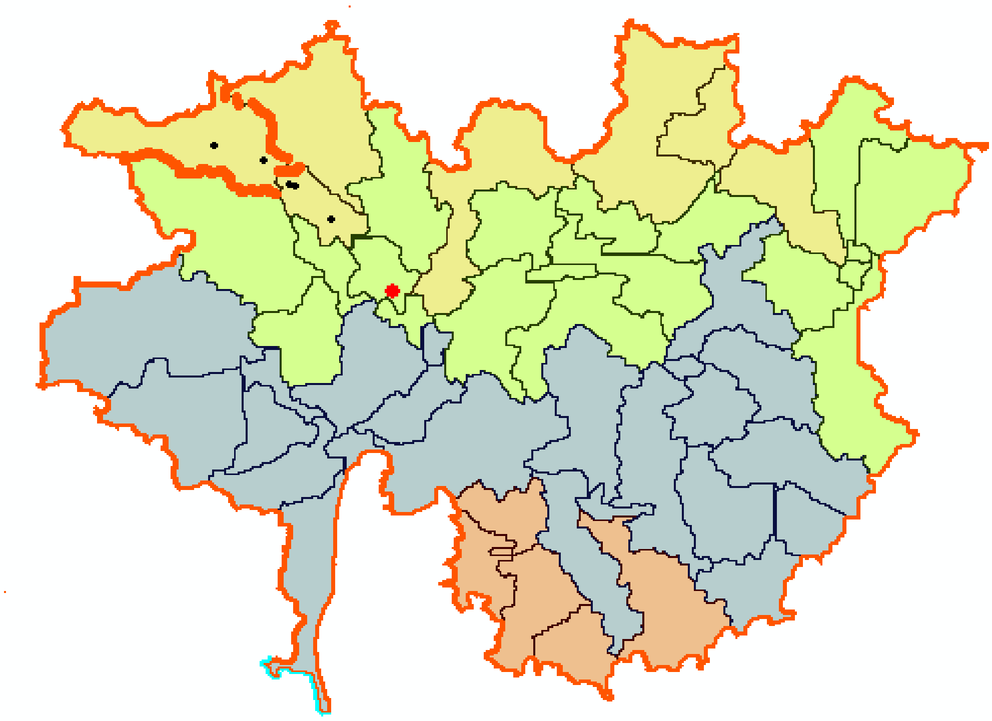


Figure-14. Detailed catchment delineation for the study area A. Red lines represent catchment borders.

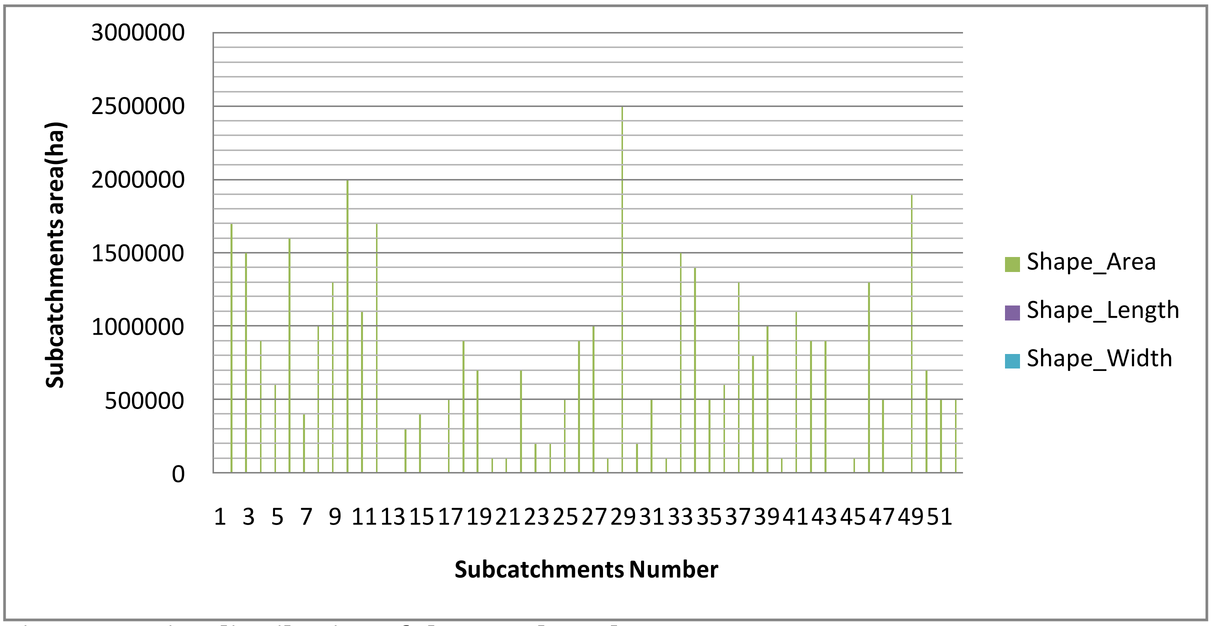


Figure 15. Size distribution of the 52 subcatchments.

5.1.3. Selecting the area for closer study

The next objective of the study was the subdivision of the catchments delineated above. For the purposes of testing SWMM in this work, the size of the study area should reduced compared to the previously defined on preliminary study area or the details delineated part can be used as selected area for SWMM . One of the reasons for this was the excessive amount of manual work required to make the stormwater network data usable in a large extent.

The new geographic scope was set to cover one catchment in the center of Shashemene. These particular catchments were chosen because in these areas:

– no combined drainage canal existed;

– stormwater drainage ditch line data that represented was of rather good quality .

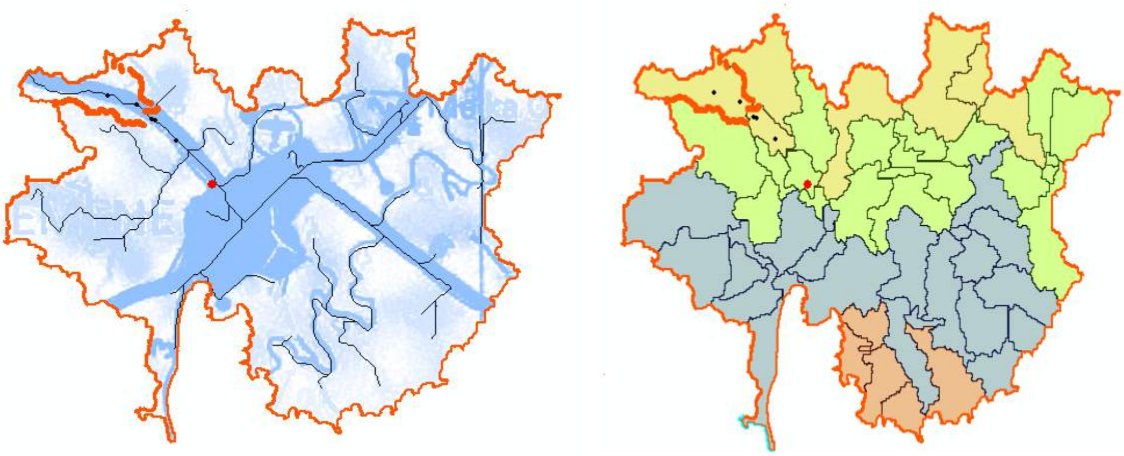


Figure -16. Sub division of Study area, consisting of catchments and drainage line

5.1.4. Catchment subdivision for study area (Specific area)

As some width were non-existent even in the point data, they needed to be interpolated from up- and downstream values. The network of drainage canals with a minimum width of 600 mm was then overlaid and visually compared with the streams of FAC using the threshold of 10.62 hectares (see Figure 17). In the majority of locations, the FAC stream continued further upstream than the 600 mm.

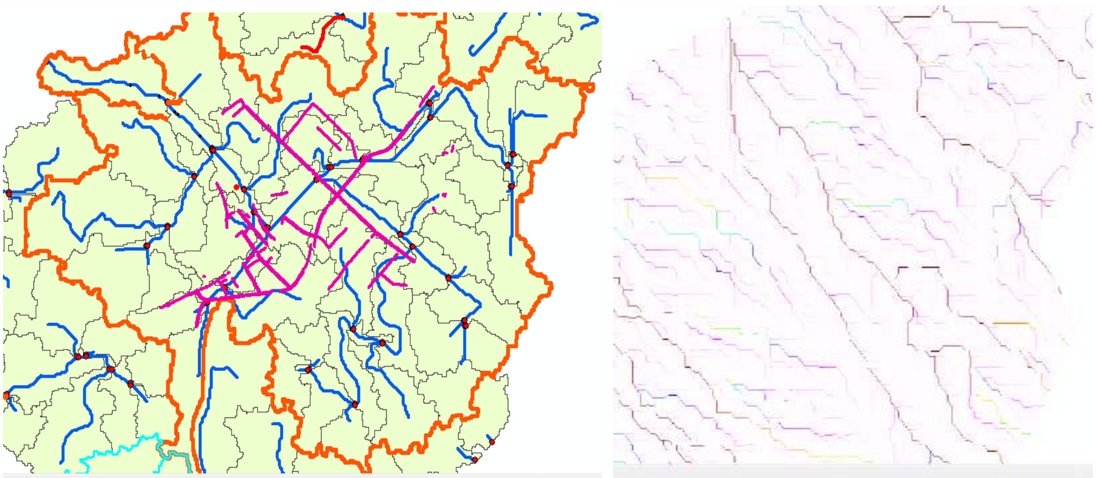


Figure-17. Flow accumulation grid (right) and stormwater drains with a minimum canal width of 600 mm (left).

Due to the poor quality of the drainage Canal data, the method based on the FAC values was judged to be a better starting point for the subdivision of large catchments.

After viewing the results, some subcatchments of the size of just a couple of cells were manually merged with their larger neighbors. The pour points and corresponding subcatchments are shown in (See figure 18).

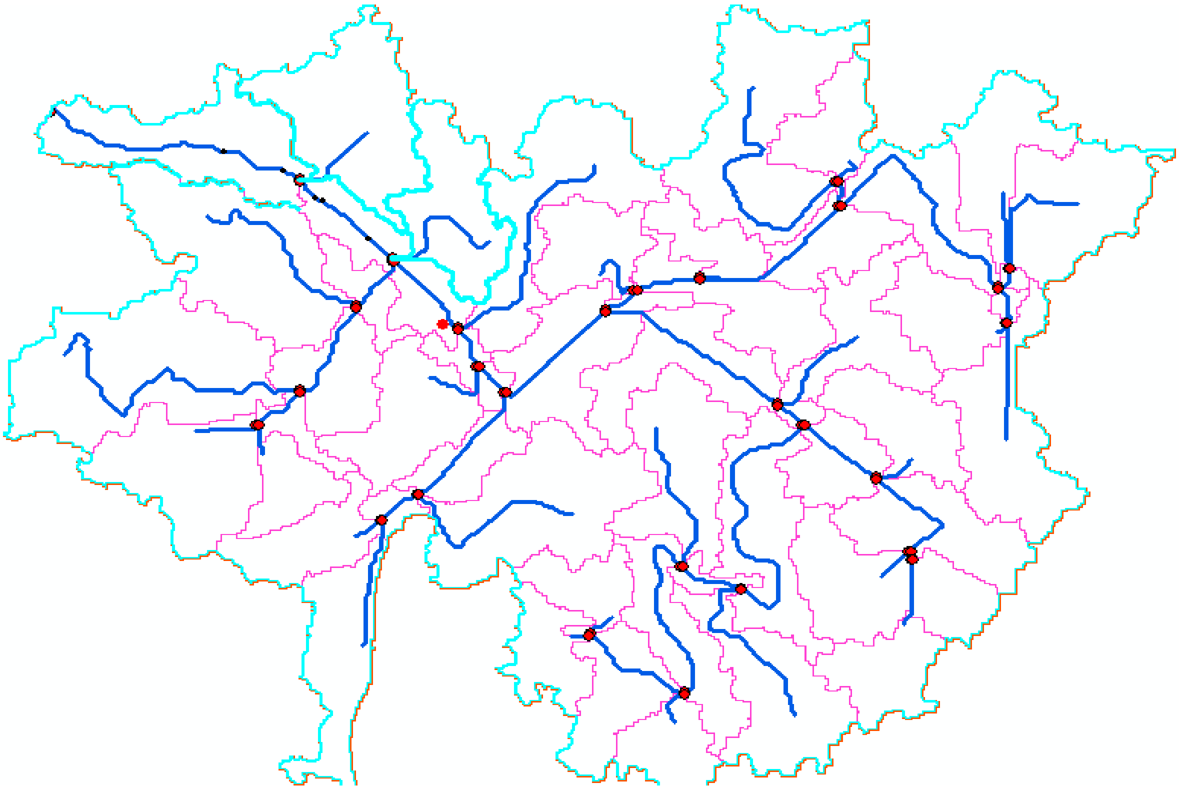


Figure-18.Pour points (red dots) and subcatchment delineation (black lines). Flow routes are presented blue with size distribution of all catchments.

The subcatchments in an SWMM model should be internally homogeneous in terms of e.g. land-use and surface materials. With the delineation method employed here one can be sure that that is not the case. One of the aims of the next subtitle (5.1.5.) is to find out whether it is possible to choose justified parameter values for such heterogeneous subcatchments.

5.1.5. Comparison of catchment delineation and subdivision

To validate the results of the catchment delineation and subdivision, comparison was made with the catchment formed observation on the Terrain Topography of the field area. I have performed detailed observation of location of pour points based on flow through the drainage ditch on one of the subcatchments of this study.

Overall, based on the results of this study, catchment delineation of an urban area should build both on (i) terrain topography as well as (ii) detailed and comprehensive data of the stormwater drainage system. Urban catchment delineation and subdivision is an iterative process that cannot be fully automated due to regular deficiencies in the data.

Manual work is always necessary to review the quality of the drainage network data and make corrections where needed. For a large area, data is likely to be insufficient to some extent. A typical cause of delineation error is that all culverts do not appear in the canal network data but need to be added manually.

Setting subcatchment pour point locations is a critical part of the subdivision process. The problem is that there is no one right way of doing it. The method for choosing pour points used in this study was straightforward to apply but is nonflexible in terms of creating subcatchments of a certain desired size class.

A couple more things must also be noted regarding the methods used: (i) Burning the stormwater drains in the DEM includes the assumption that water could enter the drain at any point along its course. Naturally this is untrue for all pipe flow, as the runoff may in reality enter only through inlets such as manholes etc. (ii) Some locations of drainage-flow bifurcation were omitted in the model. Existence of such details however implicates that the catchment delineation is ambiguous. In other words, runoff from certain areas could actually end up in two separate destinations depending on the state of the system. (iii) Due to sandy soil and hilly topography, it is likely that horizontal surface-layer and ground-water flow occurs at the study area. As flow within the soil might head in different directions than on the surface, the catchment delineation performed here applies only for the surface flow. To conclude, the above three aspects prove that the catchment delineation performed in this study is only a mere approximation.

5.2. Subcatchment parameterization

Subcatchments require a wide range of parameters until they can be modeled in SWMM. Some of these parameters are; subcatchment area, impervious surface, Catchment width, Inverted elevation and etc. easier to obtain, although uncertainties may be involved. In contrast, other parameters like flow width require complicated GIS processing to reach even rough estimates. One of the objectives of this study was to develop parameter estimation methods and to evaluate their applicability to large areas with the data sources typically available.

Before going into detail on the subcatchment-specific parameters, some general parameter settings are mentioned: (a) each subcatchment was manually assigned to a correct outlet node in the drainage network, (b) subcatchments were named with numbers corresponding to the outlet node numbering, (c) runoff from both the pervious and impervious fraction of a subcatchment was set to be routed directly to the outlet, and (d) all subcatchments were linked to the same rain gage station at Arsi Negele.

5.2.1. Imperviousness

The imperviousness parameter describes the percentage of impervious surfaces in relation to the total area of a subcatchment. It is often used as a calibration parameter as it is not quite straightforward to physically define, due to the fact that many surfaces are in reality partially impervious. For this research, no flow measurements were available, and therefore calibration was very difficult. Other ways to define the values of imperviousness are to estimate them based on land use data, or by automated or manual image processing of aerial or satellite orthophotos.

In this research, the manual land inventory of Shashemene was used to show Imperviousness layers for subcatchment parameterization(see Figure-19).

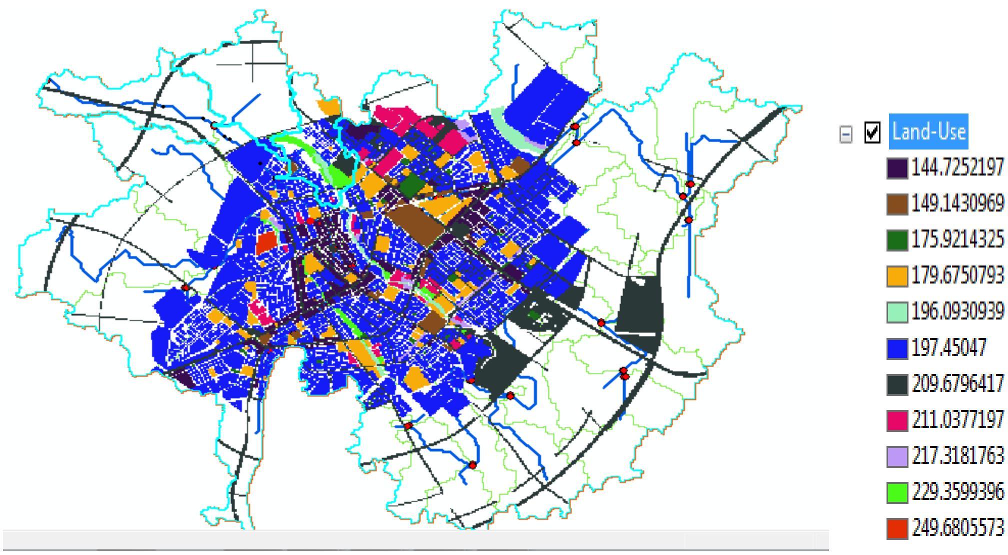


Figure-19. Subcatchment imperviousness values used in model parameterization.

Table 3.Land use of Shashemene

Land use	Hectare	Percentage
Residence	1050	45
Commerce	87	3.72
Services	357	15.3
Manufacturing and storage	74	3.2
Administration	17.5	0.77
Open space and greenery	226	9.68
Urban agriculture	67	2.87
Transport and street network	401	17
Special function	26	1.1
Water body	27	1.2
		100

Comparison with aerial orthophotos instantly showed that imperviousness values gained were in the appropriate order of magnitude.

Before using the land inventory data, the approach in this work had been trying to identify imperviousness through combining land use information from the topographic database and some other sources. The aim was to sum up all the impervious and pervious features in the subcatchments and used in SWMM.

5.2.2. Depression storage

SWMM treats the pervious and impervious parts of a subcatchment separately, and thus both may be given independent values of depth of depression storage. One can also define a portion of the impervious area to have no depression storage at all. This could be realistic on steep roofs, for example. The SWMM User"s Manual (Rossman, 2010) suggests some literature values for the depression storage. For impervious areas, the values range from 1.3 to 2.5 mm, and for lawns from 2.5 to 5.1 mm. The highest value is given for forest litter (7.6

mm). These values are not very exact, and depression storage actually is one of the common calibration parameters used for SWMM parameterization (Choi and Ball, 2002).

Based on the values above, the depression storage for all subcatchments were set to 1.9 mm for impervious subcatchment fraction and to 5.1 mm for the pervious fraction. The percent of impervious area without depression storage was set to zero. The values are compared with calibrated values by measurement (Rossman, 2010).

5.2.3. Infiltration

The Green-and-Ampt model used by SWMM to account for infiltration involves two soil-dependent parameters: (a) capillary suction head ψ, and (b) saturated hydraulic conductivity K. In addition, the initial state of the infiltration model is defined by a third parameter, the initial moisture deficit IMDmax.

Infiltration parameters depend on the soil type. On the map of our country, one can assume those areas to resemble their surroundings, indicating sand and till would be the dominant soil types within the specific study area. Typical infiltration parameter values for sandy soils are presented in Table 4 below.

Table 4. Literature values for infiltration parameters (Rossman, 2010).

Symbol	Variables	Unit	Value for San	Value for loamy sand	Value for sandy loam
k	Saturated Hydraulic Conductivity	Mm/hr	120	30	10.9
Ψ	Soil Suction Head	Mm	49.0	61	1100.453
ϕ	Porosity	-	0.437	0.437	

For the initial moisture deficit parameter Rawls et al. (1992) give maximum values from 0.35 for sand to 0.25 for sandy loam. Using these values as such depicts the soil as efficiently drained, implying that ground water would not limit the infiltration. This is assumingly not always true, but in the absence of better knowledge IMD max was still set to the value of 0.25.

5.2.4. Slope

In SWMM subcatchments are conceptually represented as rectangular planes. These planes are inclined so that all surface flow is directed perpendicularly towards one of the edges of the rectangle. The slope parameter tells the amount of inclination.

In reality, the subcatchment shape and slope vary within the subcatchment. This is the case especially with large heterogeneous subcatchments like those of this study area. Thus, the most feasible way to derive subcatchment slopes would be to calculate them from DEM.

Considering the model conceptualization, the composite slope of a subcatchment should be based on the slope along the flow paths in the subcatchment. This could be achieved by calculating the slope at each cell to the D8-derived flow direction from that cell. Unlike in the conceptual model, flow is concentrated in the stream cells while numerous upstream cells transfer only small amounts of flow. For this reason, the cell-by-cell slope should be weighted by flow accumulation before averaging over a subcatchment.

Here, the original DEM was used as a starting point since it is more suitable for the slope assessment. From the perspective of slope there is though one major flaw in this data the buildings have been erased from the terrain. Omitting buildings probably unrealistically reduces the average slope, as many rooftops are areas with large slopes.

The slopes obtained were then imported back into ArcGIS and are shown in (See Figure 20).

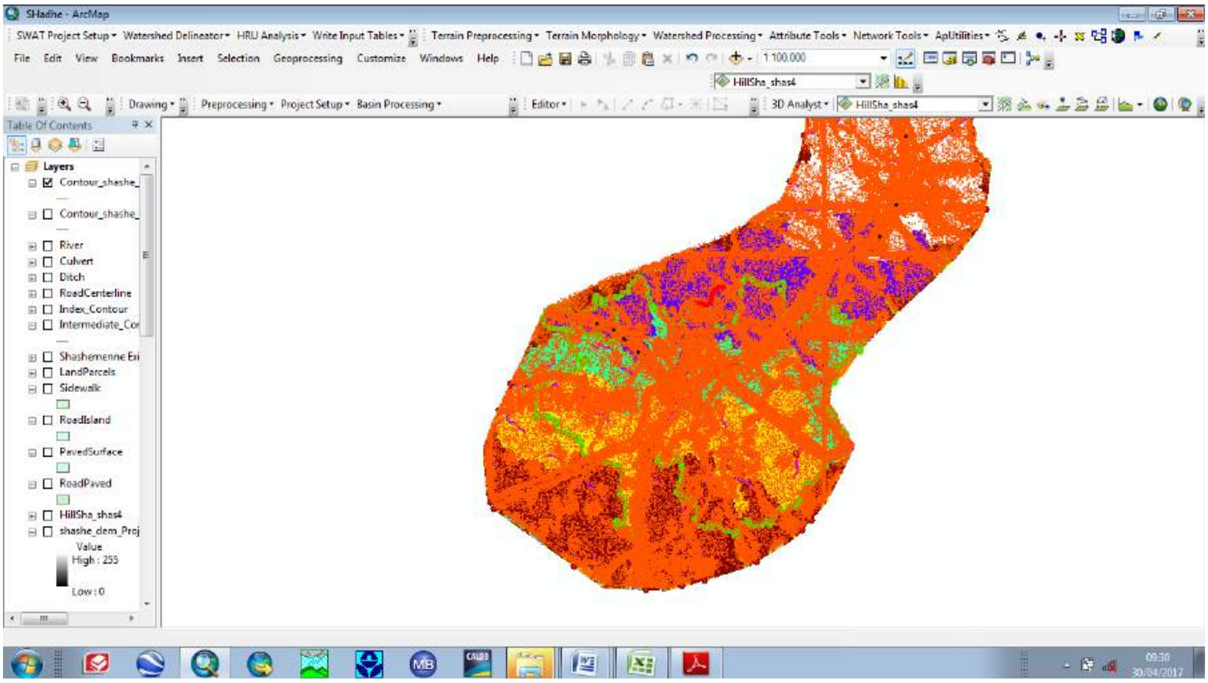


Figure -20. Slope raster showing extremely high slope values at rooftop edges.

Using those extremely high slopes in SWMM would not be conceptually realistic, as increasing the local slope after a certain point mainly affects the energy loss due to turbulence, thus not shortening the response time any more.

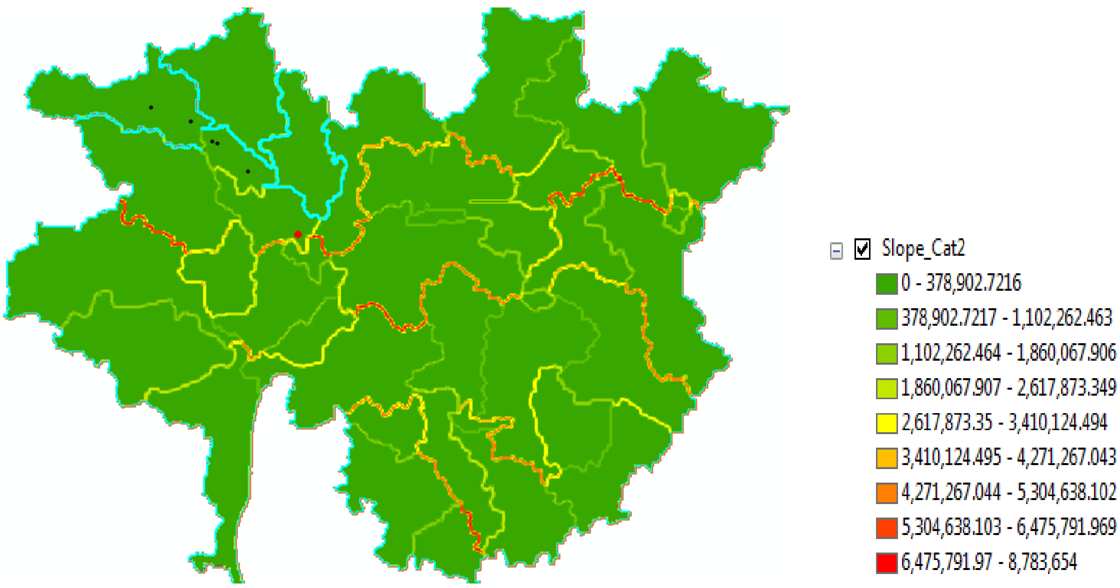


Figure21. Subcatchment mean hydrologic slopes that were used as model parameters.

In the SWMM conceptualization, flow paths from upstream cells to drainage points are straight parallel lines. In reality, these lines lie superimposed in the stream cells. Thus, to get results consistent with the SWMM conceptualization, the areal mean slope had to be weighted by the flow rate at each cell(See figure 21). As the slope parameter should characterize only overland flow, the cells where flow happened in stormwater drains had been set null in the FAC before the multiplication.

5.2.5. Manning's roughness coefficient n for overland flow

For impervious areas (old drainage ditch) the roughness coefficient n was set to the value of 0.025, which is the literature value for smooth asphalt (Rossman, 2010). For non-asphalt surfaces, this was considered a good compromise between smoother materials like rooftops, and slightly rougher materials like concrete.

For pervious areas n value of 0.3 was used. This was a compromise between the values for short grass (0.15), dense grass (0.24) and woods with light underbrush (0.40) (Rossman, 2010).

5.2.6. Flow width

Flow width is one of the least tangible SWMM parameters. It is defined as the characteristic width of the overland flow path for sheet flow runoff. Very typically it is used as a calibration parameter (Gironas et al., 2009), although there are ways to deduce an initial estimate even without calibration. According to Gironas et al. (2009), the width parameter can be calculated by dividing the subcatchment area by the length of the longest overland flow path in the area. In case several flow paths exist, their maximum lengths should be averaged.

The subcatchment flow length appeared to be 116 m on average, with individual values ranging from 10 to 232 m(See Table 9, in Annex).

The above number of source cells did sound rather high, as in the SWMM conceptual model source cells would only occupy one row of cells at the upstream edge of the rectangular subcatchment. Then, theoretically, the portion of source cells of the subcatchment area should be:

$(WFLC)/(WFLF)=LC/LF$ --(13)

where

WF = flow width parameter (m),

LF = flow length parameter (m),

LC = raster cell size (m).

To test how fulfilling the above equation would affect the flow length, the raster of the source cell downstream flow lengths was converted into point features and imported into Excel. The idea was to select only a certain share of the highest cell flow lengths so that the above equation would be true. Through manual iterations on several subcatchments it was found that counting only the highest percentile of the flow lengths on each subcatchment would fit the equation reasonably well.

Flow widths were finally calculated by dividing the subcatchment areas by the flow lengths acquired through both of the above approaches.

Results for the approach B are presented in (See Figure 22).

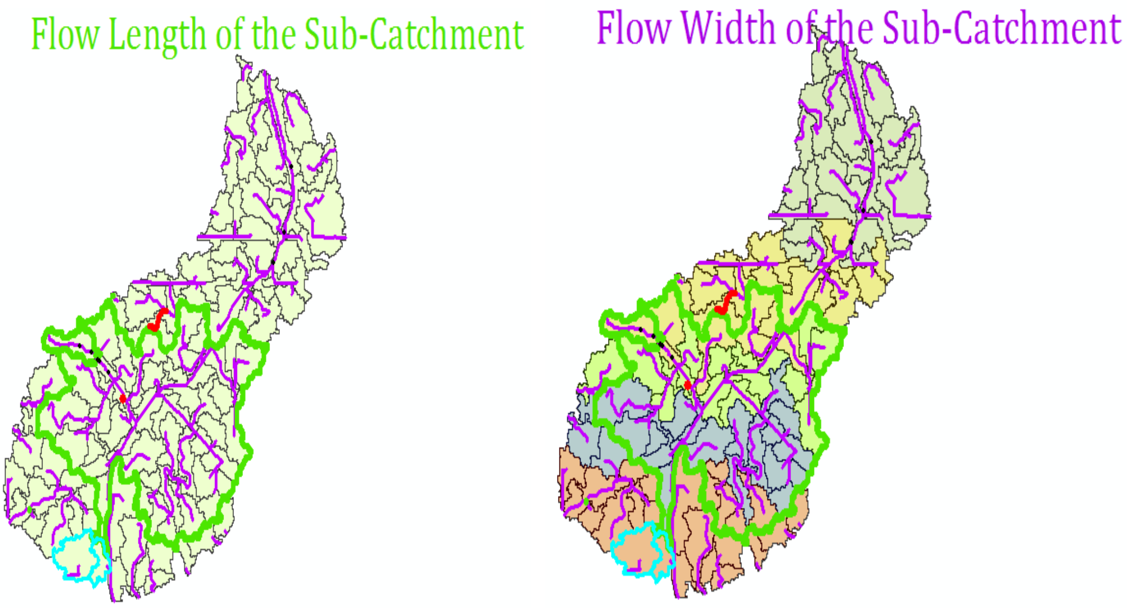


Figure-22. Subcatchment flow lengths (left) and flow widths (right) obtained using the approach 2.

The results were compared with the results of measured and calculated data for parameter calibrations in two of the subcatchments in question.

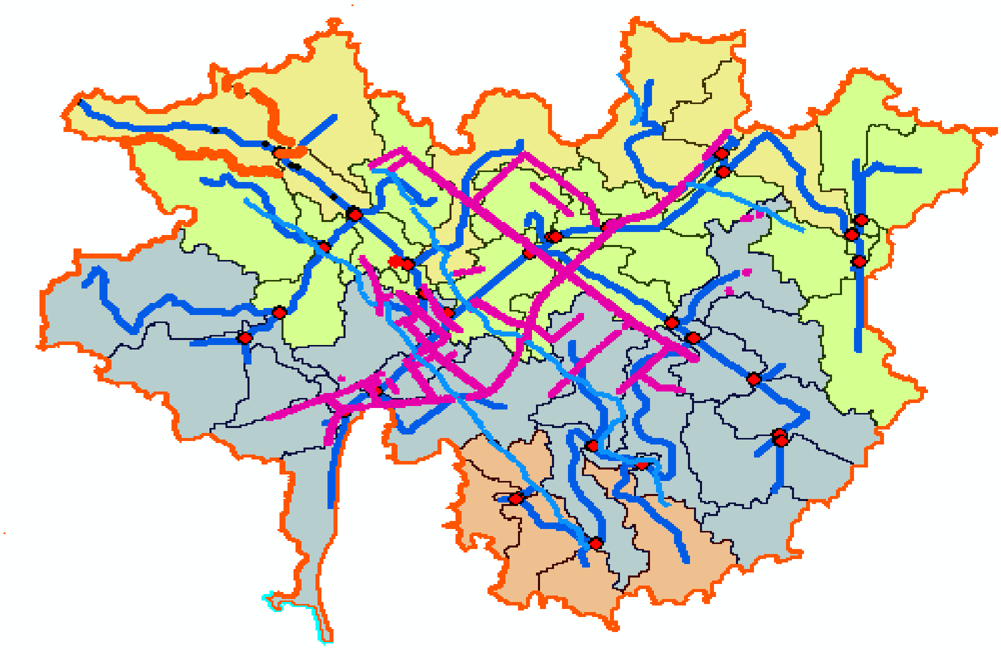


Figure-23. For comparison of storm runoff with existing drainage capacity

After the completion of Catchment delineation for the existing situation of study area, it is possible to calculate storm water from the available data and existing drainage capacity of the canal by (using Manning Formula Type is Rectangular)(See Figure 24).

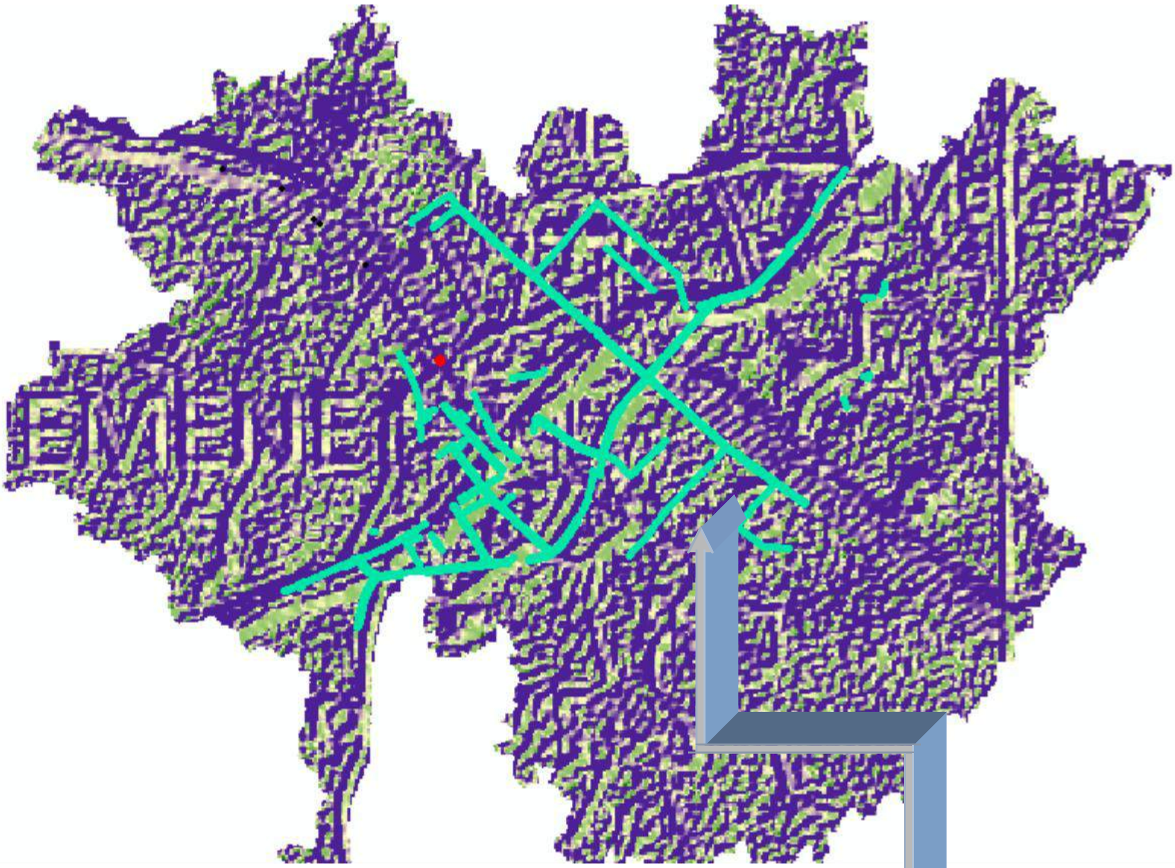


Figure 24. Existing drainage ditch(Light Green)

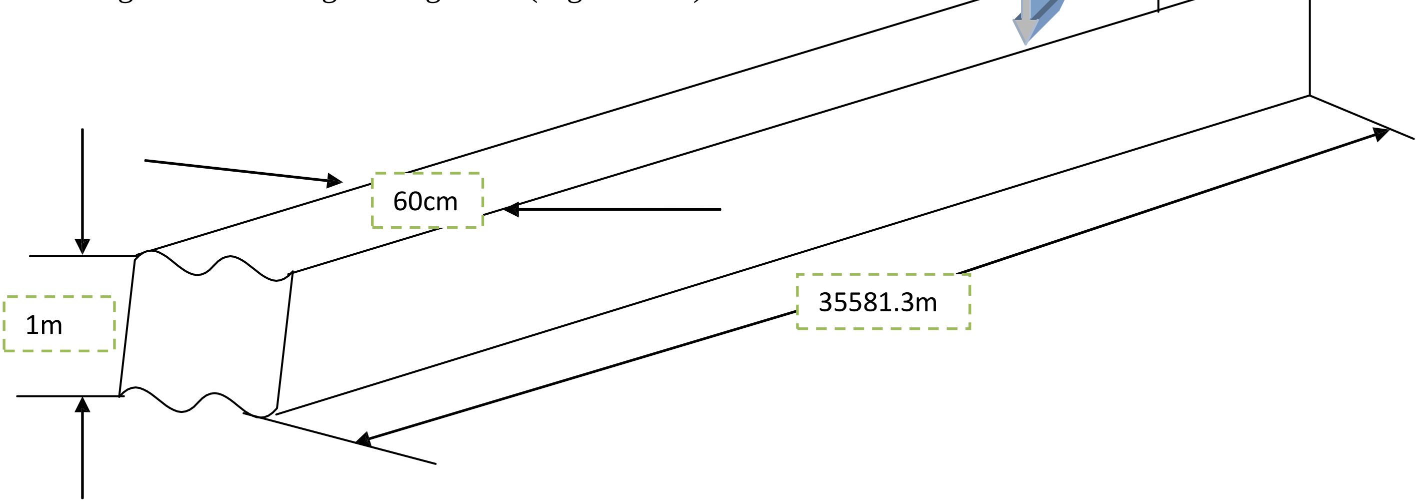


Figure 25. Drainage ditch dimension(The whole from figure 24)(Light green)

Drainage Ditch Width(B)= 0.6m

Ditch Depth(m)=1m

Manning (n)=0.015 (for old ditches)

Drainage Ditch slope(so)=0.0025

Length of Drainage Ditch (m)=355813

Area(m2)=Bd=0.6*1=0.6m2

Wetted Perimeter (P)=2d+B=2*1+0.6=2.6m

Hydraulic Radius(R)=A/P=0.6/2.6=0.23m

Velocity(V)=1/n($R^{2/3}$)($So^{1/2}$)=1.25m/s

Discharge (Q)=AV=0.6*0.75=0.75m3/s

Using Rational Method for Hydrologic Calculations of the area in the feature

Calculation of peak storm water runoff rate from a drainage area is often done with the Rational Method equation (Q = CiA). Provides peak runoff rates for small urban and rural catchment areas, less than 50 hectares, but is best suited to urban storm drain systems and rural ditches. It shall be used with caution if the time of concentration exceeds 30 minutes. Rainfall is a necessary input. This method, while first introduced in 1889, is still widely used. Even though it has come under frequent criticism for its simplistic approach, no other drainage design method has achieved such widespread use.

The Rational Method equation actually used to calculate peak storm water runoff rate is: **Q = CiA** (U.S. units), or **Q = 0.0028 CiA** (S.I. units) where:

- **A** = the area of the watershed (drainage area) that drains to the point for which the peak runoff rate is needed (**ha**)

- **C** = runoff coefficient for drainage area A.

i = the intensity of the design storm for peak runoff calculation (**mm/hr**

Q = the peak storm water runoff rate from the drainage area, A, (**m^3/s)**.

Step 1. Runoff Coefficients :-Since the physical interpretation of the runoff coefficient is the fraction of the rainfall on the watershed that becomes surface runoff, it"s value must be between one and zero. The value of the runoff coefficient for a given drainage area depends primarily on three factors: i) the soil type, ii) the land use, and iii) the slope of the watershed. Each of those factors will now be discussed briefly.

Step 2. Identifying Hydrologic Soil Type:- Sandy soils allow a high infiltration rate, so they have a relatively low storm water runoff rate and a relatively low runoff coefficient. Soils with a large clay content, however, have a low infiltration rate. As a result, they have a relatively high storm water runoff rate and a relatively high runoff coefficient.

The United States Soil Conservation Service (SCS) has identified four soil group classifications (A, B, C, or D) that can be used to help in determining values for drainage area runoff coefficients. Determination of which SCS soil group fits a particular soil may be on the basis of a measured minimum infiltration rate for the soil or on the basis of a description of the soil.

The minimum infiltration for each of the SCS soil groups are as follows:

Group A 0.30 – 0.45 in/hr, Group A Deep sand; deep loess; aggregated soils

Group B 0.15 – 0.30 in/hr, Group B Shallow loess; sandy loam (Andosols) for Shashemene area

Group C 0.05 – 0.15 in/hr , Group C Clay loams; shallow sandy loam; soils low in organic content; soils usually high in clay

Group D 0 – 0.05 in/hr , Group D Soils that swell significantly when wet; heavy plastic clays; certain saline soils

Step3. Land Use land cover Property:- Factors related to land use that affect the runoff coefficient are the fraction of the area with impervious cover, like streets, parking lots, or buildings and the extent of vegetative cover, which intercepts surface runoff(Table 9, Annex).

Step4. Slope of the area: In general, a watershed with a greater slope will have a higher runoff coefficient than one with a lesser slope.

The two tables below provide runoff coefficient values in terms of land use and watershed slope for each of the four SCS soil groups.

Table 5.Recommended Runoff Coefficient C for Pervious Surfaces by Selected Hydrologic Soil Groupings and Slope Ranges

Terrain Type	Soil Type			
	A	B	C	D
Flat, <2%	0.04-0.09	0.07-0.12	0.11-0.16	0.15-0.20
Rolling, 2-6%	0.09-0.14	0.12-0.17	0.16-0.21	0.20-0.25
Mountain, 6-15%	0.13-0.18	0.18-0.24	0.23-0.31	0.28-0.38
Escarpment, >15%	0.18-0.22	0.24-0.30	0.30-0.40	0.38-0.48

Table 6. Land use and run off coefficient

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
Multi units, detached	0.40-0.60
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
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)

Step 5. Watershed Time of Concentration estimation

The time of concentration doesn"t appear directly in the Rational Method equation. It is needed, however, for determination of the design rainfall intensity to use in the Rational Method equation. For a given watershed, the time of concentration is the time required for rainfall landing on the farthest point of the watershed to reach the watershed outlet.

Estimating Time of Concentration: Many empirical equations are available for calculating time of concentration for a watershed.

The **Manning equation** can be used for the open channel flow portion of the storm water runoff that typically occurs at the end of the runoff path. This equation is recommended for calculating open channel flow travel time.

The Manning equation for rectangular drainage ditch as follow:

S.I. units: $\boldsymbol{Q} = \left(\frac{\mathbf{1}}{\boldsymbol{n}}\right)\boldsymbol{A}(\boldsymbol{R})^{\frac{2}{3}}(\boldsymbol{S})^{\mathbf{1/2}}$ **...(14)**

Other equations (in addition to the Manning equation) that are used in calculating travel time for the open channel flow portion of the storm water runoff are:

Velocity of flow = **V = Q/A ....(15)**

Travel time = $\mathbf{t_3}$ **= L/(60V) ....(16)**

For designing a drainage system, the overland flow path is not necessarily perpendicular to the contours shown on available mapping. Especially in urban areas, the land will be graded and swales will intercept the natural contour and conduct the water to the streets, which reduces the time of concentration. Care shall be exercised in selecting overland flow paths in excess of 100 meters in urban areas and 200 meters in rural areas.

For my Case, Let L=100m, Flow path length=60m from the catchment flow path Generation, Channel slope=2.5%,manning coefficient=0.011, Residential Area=80% and under developed =20% (from Shashemene Land use data),Hydraulic soil group is 100% B (Andosol ,sandy Loamy, 2.5% Slope), Overland flow runoff coefficient=0.14 (0.12-0.17 for this case),So=0.00025

A=Bd=0.6*1=0.6m2

P=2d+B=2.6m

R=A/p=0.6/2.6=0.23m

$V=1/n(R)^{2/3}(So)^{1/5}=1/0.015(0.23)^{2/3}(0.0025)^{0.5}=1.25m/s$

Travel time = $\mathbf{t_3 = L/(60V)}$

=100/(1.25m/s*60s/min)=1.3min, Say 1min.

For overland flow length=60m, C=0.14 and slope of 2.5% , Inlet time is 29minutes (tabulated ,annex).

Tc=Inlet time +Flow time=29+0.98=**29.98minutes**

Step 6. Rainfall Intensity determination

The rainfall intensity, i, for use in the Rational Method equation is the intensity of a constant intensity storm with return period equal to a specified value for the purpose of the peak runoff rate being calculated, and duration equal to the time of concentration of the watershed. The return period to be used is typically specified by a state or local government agency.

Table 7. Rainfall intensity

Year	**Annul RF(mm)**	**Number of Rain Fall Days**	**Hourly Rain Fall (mm)**	**Rank**	**Annul RF intensity (mm/min)**
2005	1144.1	142	234.6	1	8.06
2006	809.2	134	184	3	6.04
2007	986.3	123	188.6	2	8.02
2008	684.4	62	96.4	5	11.04
2009	306.9	63	71.9	10	4.87
2010	599.7	113	135.9	8	5.31
2011	660.1	119	186.6	6	5.55
2012	570.4	117	127.9	9	4.88
2013	614.6	121	155.3	7	5.08
2014	746.8	119	224.9	4	6.28
					65.11

In order to determine the storm intensity for known duration and return period, some type of intensity-duration-frequency (IDF) data for the location of interest is needed. In general, for a given return period, a shorter duration storm will be of greater intensity than a longer duration storm.

From (Figure.47 in Annex) Shashemene is in Region A3. From Figure 45-10 for Region A2 with a duration equal to 30 minutes,

i_{10} (10-yr return period) = 67 mm/hr

i_{25} (25-yr return period) = 80 mm/hr

i_{50} (50-yr return period) = 90 mm/hr

Depending on the type of IDF data available, the design rainfall intensity, i, can typically be obtained for a given return period and storm duration by reading from a graph or interpolating from a table. When using an Excel spreadsheet for calculations, however, it is more convenient to have the IDF data in the form of an equation. When the data is fit to an equation, the typical form for the equation is: i = a/(d + b) for each return period of interest, where i is the storm intensity, d is the storm duration, and a & b are constants. The equation i = a/(d + b) can be rearranged into the form:

1/i = (1/a)d + b/a, which is a linear equation for **1/i** vs **d.**

The first step is entry of the drainage area=47ha, design return period(25 years) coefficient(C=0.4), and design storm duration (time of concentration) (29.98 say 30Minutes), Then it is also necessary to enter three pairs of values for storm duration and precipitation depth in mm, from the IDF data for the design location. For this case, the following pairs of values were read from the IDF diagram for A3 above and Graph from ERA drainage Design manual 2002 **(page 5-39).**

Table 8.Storm duration in region A3

Duration d in (mm)	Precipitation
15	102
30	67
60	42

Note that the rainfall duration values were chosen so that the design rainfall duration (30 min) falls with the range of those data points.

After entry of the above three pairs of values in the blue cells, the spreadsheet will calculate rainfall intensity, i, and 1/i for each storm duration. From the spreadsheet linear regression was carried out to find the values for the constants a and b in the equation $i = a/(d + b)$.

The resulting equation for rainfall intensity, **i**, as a function of storm duration, **d**, (for a 25 year storm) for this example is: **i = 3235/(d + 17.3)**

The spreadsheet calculates the design rainfall intensity by substituting the specified design storm duration (30 min) into the equation, giving:

i = 68.39mm/hr

Note that this is close to the value of **i = 68.39mm/hr** that was obtained from the Rain fall intensity calculation on above**(65.11mm/hr).** Both methods are limited by the accuracy to which the values can be read from the IDF diagram.

Peak Storm Water Runoff Rate can be calculated from either spreadsheet by the graph or rainfall intensity from given precipitation data. Rainfall intensity**(68.39mm/hr**) **i**, **specified watershed drainage area(47 ha)A**, and **runoff coefficient(0.4), C**, in the Rational Method equation:

Q = CiA. **=0.00278*0.4*68.39*47=3.60m3/s**

This is the peak storm water runoff currently (10 years from 2005) ,but for the same area in this case after another ten years (almost 25 years from now), by the intensity of 80mm/hr from the graph, Q will be increased by **14.45%** ,which is **4.21m3/hr**. When it is compared with the existing situation of the study area, peak runoff of the area cannot tolerate the existing capacity of the canals. For 50 years, discharge will be 4.70m3/hr.

Therefore additional canal dimension is very important to reduce risk of storm water runoff in the study area to rout the water to the outlet of the delineated catchment or to the Gogeti river stream .

Figure 26.Photo taken during drainage ditch over flow (Left Side or besides of bus station Entrance)

To overcome the peak storm water runoff problems, the study area needs canal dimension adjustment.

Velocity (V)=1.7m/s and area (A)=47ha

Existing Canal length (measured)(m)------------ 35813.344m(measured from topo map)

Volume of water that can flow through this length (m3)

V= B*D*L= 0.6*1*35813.344=21,488.01 m3,

From Equation Q=V/t, where ,V-Volume and t, second

t=V/Q=21,488.01/0.75m3/s=28,650.68s

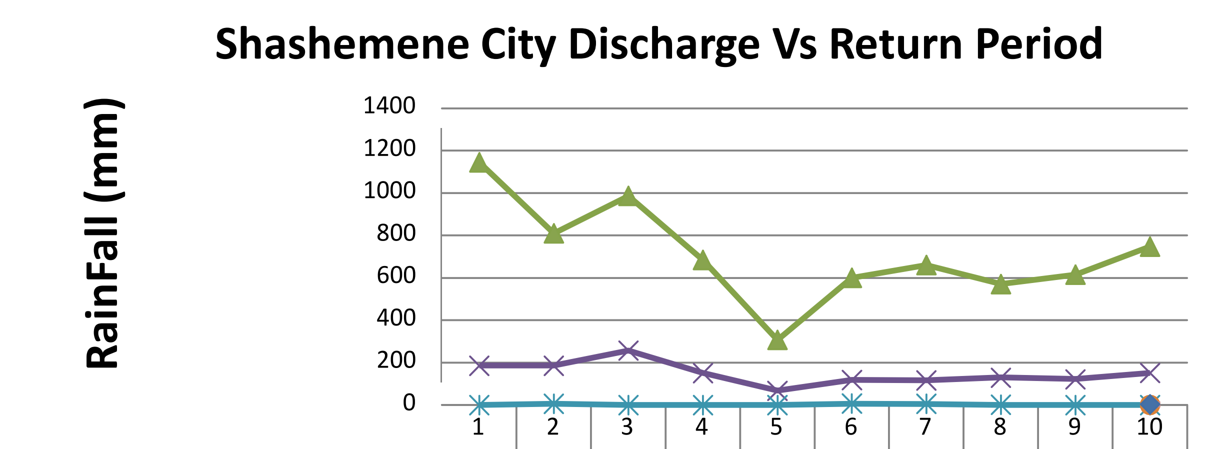
$Q=B*d*L/t$--------2 and $Q=1/n*(R^{2/3})*(S_o^{1/2})$--------------3

Equating 2 and 3

B*d*L/t=1/n*($R^{2/3}$)*($S_o^{1/2}$)

Substituting B=0.6m and d=1m with calculated value of R, The additional Canal Length

L=7.2t=7.2*28650.68=206,284.9 which is about 5times that of the existing canal.



	1	2	3	4	5	6	7	8	9	10
Annul RF(mm)	1144.1	809.2	986.3	684.4	306.9	599.7	660.1	570.4	614.6	746.8
Annual Max.Q	186	186	257	151.1	67.9	117.7	116.2	129.3	121.9	151.8
Annual Min.Q	0	6.5	0	0	0	7.3	5.3	0.8	0	0
SWR Q existing ditch										0.75
SWR Q of 10 yrs Return Period (Currently)										3.6
SWR Q of 25 years Return period										4.21
Modeled by SWMM										3.532

Figure 27. Discharge Vs Return period in Shashemene city

5.3. Subcatchment parameterization

This study showed that subcatchments can be rapidly parameterized by combining GIS methods with literature values. DEM-based subcatchment delineation was considered to be successful. The inaccuracy in the subcatchment areas was largely explained by the different locations of the subcatchment drainage points. Differences in other parameters, too, were partly induced by averaging the parameters over different geographical areas.

Imperviousness percentages calculated from the high density Imperviousness Layer showed larger values than those of hinterland study of west Arsi, Shashemene. This was believed to be due to some type of systematic error in the production of the imperviousness Layer. Our country's-wide dataset was probably not capable of taking into account all the local factors such as differences in surface materials used in each geographical area. It could also be that not all of the total impervious area (TIA) indicated by the imperviousness Layer actually behaved as effective impervious area (EIA). For SWMM parameterization, the difference could, if one wanted to be compensated by calibration based on a few subcatchments of different land use types. Nevertheless, even with some additional calibration effort this method would have required only a moderate amount work. The method seemed thus highly promising for imperviousness parameterization of a large number of SWMM subcatchments.

The slope of the overland flow was one of the parameters with values highly resembling the aggregated values by measurement.

Flow width and flow length showed notable difference to the values of Hinterland study and measured value. This was no surprise like told above, flow width is one of the least physically-based SWMM parameters, typically used as a calibration parameter. The method included some manual work, yet the results were not really applicable. An interesting remark is though that the flow lengths that was measured are lower than the maximum possible overland flow lengths suggested after actual discharge has been computed . The reason is that in a low-resolution modeling approach the flow length also needs to account for some parts , otherwise ignored gutter or small-canal flow in addition to the true overland flow. This explains the uncommonly very low flow lengths.

For the flow width, it was challenging to find any physically-based aggregated values. Such aggregated approaches are in fact against the conceptual basis of the SWMM software. This study presented some attempts to do so, but the results are not very successful on the top map of the study area. The approach used poorly took into account the channelized overland flow which has not yet entered the drainage network. It would be probably preferable that flow width and was kept as a calibration parameter, if possible.

The national datasets used were not detailed enough as they had no coverage for urban areas. On the other hand, even detailed maps would not have helped because infiltration in urban areas may be more dependent on the compaction of the soil surface than the actual soil type.

5.4. Stormwater conveyance system parameterization

As described in Chapter 4.5, the stormwater system data was of poor quality, and the attributes had been stored inconveniently. Time-consuming pre-processing was thus inevitable to make the data suitable for stormwater modeling.

5.4.1. System links (conduits)

Data validation revealed that many of the drainage ditch features had identical duplicates in the data. That was obviously incorrect. Hence, the features were dissolved so that the duplicates with the exactly same location and attributes were eliminated. There is yet a small risk that some real parallel drainage ditchs were deleted in the process. However, any real parallel drainage canals had typically been stored at slightly different locations, resulting in none of them being lost.

The ditch width were only available as attributes of point features in the proximity of the drainage ditch. The points also had one attribute telling the angle of the drainage ditch canals in relation to North. This was first considered potentially helpful in automating the process of attributing width to drainage ditch. However, a more practical routine was to use the Join by Location tool (Esri, 2012) to assign the attributes of nearby points to drainage ditch. This was done for all points indicating a width of 600 cm or more. For a number of drainage ditches, no nearby points were available. In all those cases, the depth had to be inserted manually based on the measured upstream and downstream canals dimensions. There were also some points indicating width of less than 600 cm even if the next upstream ditch section was 600 cm or more. Such contractions were not interpreted as errors as they can exist also in reality; especially on steep hills slopes where a smaller ditch canal will be capable of conveying the same discharge as the larger canal at the top of the hill.

It must be noted that the above described dissolving technique reduces the accuracy of the model. Some manholes and lesser drainage canals junctions were ignored if the incoming and outgoing ditches of the same width. As a result, the canal slopes were averaged, always assuming a constant slope between two major junctions(See Figure 28).

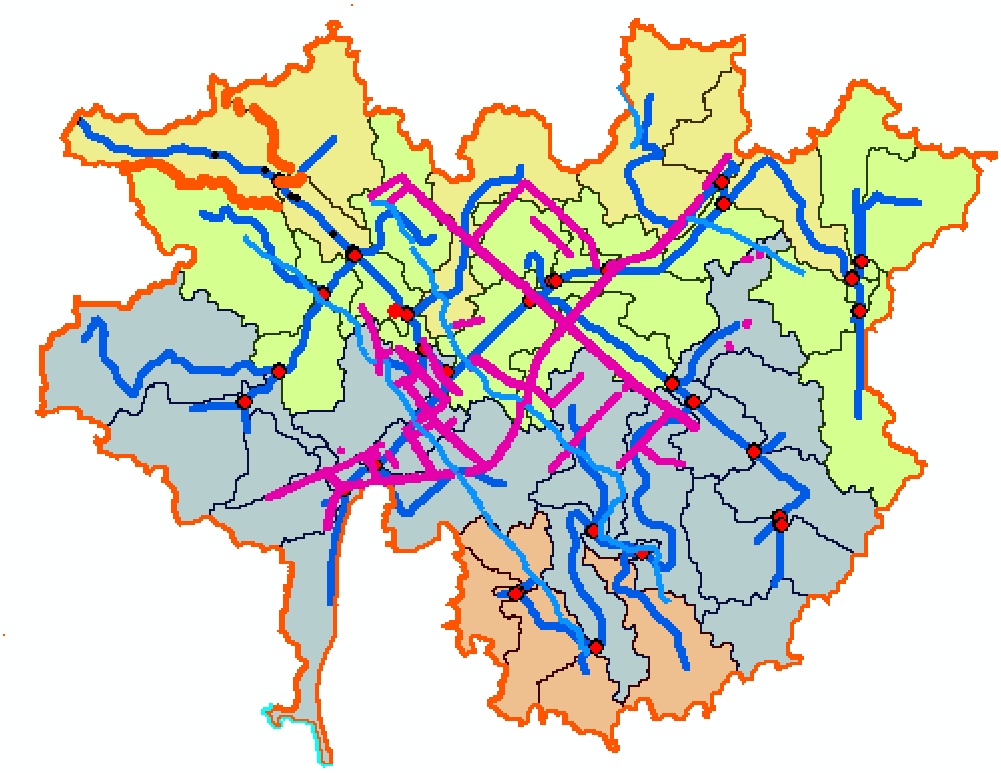


Figure 28.Stormwater drainage network used in modeling. System nodes are shown as dots and system links as blue lines

Drainage ditches length and the ditches end elevations are necessary parameters for hydraulic modeling in SWMM. Lengths were obtained as ArcGIS geodatabase format has a standard field indicating the length of the feature. Elevations needed more work, as those were presented in the data as attributes of separate points which were geometrically connected to the pipe ends by another layer of polyline features. Attempt was first made to develop an automatic procedure. The complexity of the data structure unfortunately prevented most approaches to correctly join the elevation points to the pipes and thus transfer the attributes. For a larger study area developing such a method could have been feasible, although probably not of any general use on other sites.

The most feasible option was to label the points with the attributes and manually attribute the elevations to the drainage canals. But, there were a definite risk of human error in manually typing over a hundred elevation records.

After Canals lengths and elevations were determined, the Manning"s roughness coefficient n had to be defined. No data was available on the pipe materials in the study area. According to the SWMM User"s Manual (Rossman, 2010), n value of 0.011 to 0.015 applies to both of these materials. Many of the ditches in the city center area are relatively old, and thus probably not as smooth as new ditch. Thus, roughness coefficient of 0.011 was used for all ditches.

5.4.2. System nodes

The point feature class containing all the junctions, inlets, and outfalls was created by the Feature Vertices to Points tool (Esri, 2012) as described above. Overall, 27 points were included. The one outfall was first moved to a new feature class. Each of the remaining points had to be then given attributes for invert elevation and maximum depth. Invert elevation tells the elevation of the bottom of the manhole (Adindan_UTM_Zone_37N reference system). As no data on this was available, the elevation was set by hand at 10 cm below the level of the lowest drainage canal connected to the manhole.

Maximum depth tells the elevation difference between the manhole invert and the ground surface. Add Surface Information tool (Esri, 2012) was used to add to the points a new field for ground level based on the DEM. Maximum depth could then be computed by subtracting the invert elevation from the ground level.

For the outfall point, invert elevations were set at the elevation of the incoming drainage canal ends.

5.4.3. Stormwater system parameterization summary

Parameterization of the transport compartment of SWMM was straightforward. However, work load was unnecessarily increased by the problematic structure of the input data. More detailed input data could considerably reduce the work needed here, thus enabling the inclusion of a less-skeletonized high-detail drainage network in the SWMM application. Such data might also prove valuable in the catchment delineation and subdivision process, allocation of drainage points could be based on an increased amount of drainage canal information. In addition, this would also allow for a higher degree of detail for the catchment surface discretization. If large areas were to be modeled, input data of a consistent structure with detailed attributes would be a necessity to maintain feasibility.

The non-random selection of the study area resulted in a drainage system less complex than what could have been the case in an arbitrarily chosen area. Even though There were natural streams in the study area, and all the stormwater was drained via separate natural drainage . The need to model combined drainage canals or open channels would have increased the complexity of the model and, accordingly, the challenge experienced.

The structure and parameters of the drainage network grow important only when modeling single runoff events, trying to estimate peak flows, like this studying area. Calibrating the transport system model would in such a case be highly necessary as the way of using literature values like in this study seems to be generalizing. In addition, the methodology of this study led to full reproduction of the hydraulic properties of the drainage network.

The manholes along a drainage canals with a constant width were for example omitted through the dissolving of the canal data. Neither were possible stormwater pumping stations in the study area modeled, as no data about their operation was not available.

One major cause of uncertainty is the fact that the smallest drainage canals were not being modeled at all. Practically, the modeled flow „jumps" from the canal inlets straight into the main drainage canal. This discontinuity should have been taken into account in the flow width calculations. Another option might have been to create some kind of imaginary conduit between the center point of each subcatchment and the drainage point related.

5.5. SWMM simulations

To put the above subcatchment and conveyance system parameterization to the test, selected model runs were performed and their results analyzed. In the absence of adequate runoff measurements, no actual validation of the model could be made. Hence, the emphasis was on **'normality checking'** the results against literature.

The actual SWMM model was developed from the ArcGIS data. All the features parameterized above had been stored in four separate Esri Shape files (subcatchments, junctions, conduits, and outfalls). For modeling, the geometry and attributes of all features were then converted into a SWMM project file using a custom-made Perl script. This worked well and the model created was usable right away.

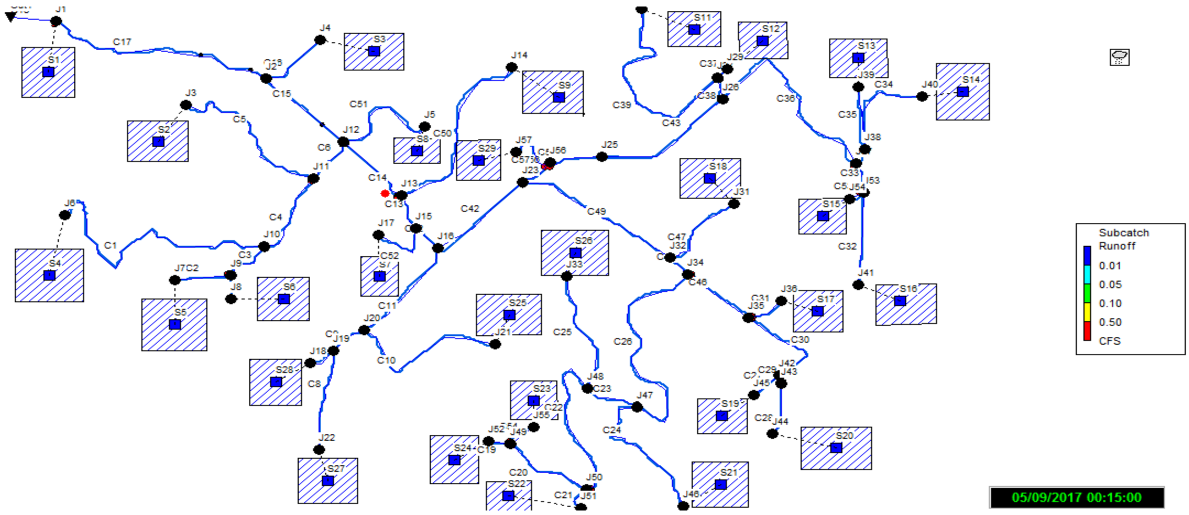


Figure 29. Structure of the SWMM model. Flow direction in the conduits is presented by arrows.

Before aiming for any actual results from the model, proper simulation time steps were to be chosen. The reporting time step and the dry-weather hydrologic time step were set to one hour. The hydraulic routing time step was set to 30 s, which was expected to be sufficiently short for dynamic wave routing (Rossman, 2010). The wet-weather time step was also set to 30 s.

The model was run with hourly precipitation data for the entire time period covered by the weather observations, from December 1998 until Nov 2007 (later referred to as the „long-term simulation"). Additional model runs were performed individually for each year during that period(See Figure 28).

Preliminary results for the long-term water balance showed that practically no runoff was generated on the pervious parts of the catchments due to excessive infiltration. This is probably due to uncertainties in the selection of infiltration parameters. The real soil type might not be sandy loam but silt loam, for example. Also, the actual value of maximum initial moisture deficit may be reduced due to groundwater interaction.

5.6. Uncertainties related to the selection of time steps

The impact of the wet-weather modeling time step on the long-term water balance was studied by varying the time step between long-term simulations otherwise identical. Lengthening the time step in the range of 10 seconds to 1 hour resulted in a growing continuity error caused by a faulty decline in infiltration. With a wet-weather time step of 30 s the continuity error for runoff was still at a rather tolerable level below 10 %, but with time steps of several minutes the error rapidly increased to over 15 %.(See Figure 28).

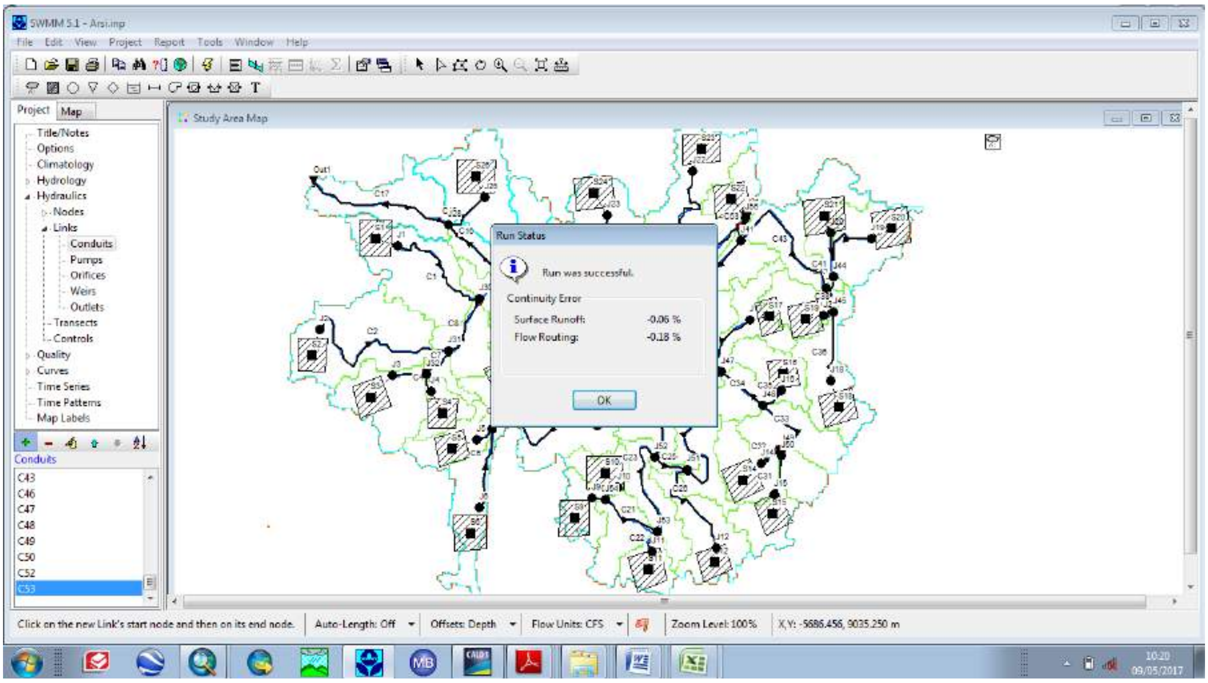


Figure 30. Uncertainty during data calibration

5.7. SWMM simulations Summary

The SWMM simulations performed to test the reasonability of the model were disturbed by excessive runoff continuity error. The error was evidently attributed to the Storm water flow accumulation and over land flow processes. Partly the error was caused by a publicly known flaw in the model algorithm, which could be quite simply counteracted by numerically altering the water balance results. However, for some reason this correction did not completely remove the run off-related continuity error. Maybe the logic of the correction method used was somehow faulty, or maybe there is also some other mechanism increasing the continuity error for long-term over land flow process simulations. Either way, the behaviour of the modelled water-balance components could not be properly assessed. The results suggest that there may be some error regarding the runoff processes in the current SWMM version.

The 1-hour simulation results for runoff water equivalent showed good accuracy compared to the measured and computed storm water flows. A thorough comparison was not possible as the runoff storm water content relating the two properties was unknown. These simulations were solely based on literature values, indicating that reasonable results for run off (over land flow) processes may be obtained with SWMM even without model calibration. (See Figure 31).

All catchments was considered to have similar land use, since calibration of all catchment was very difficult.

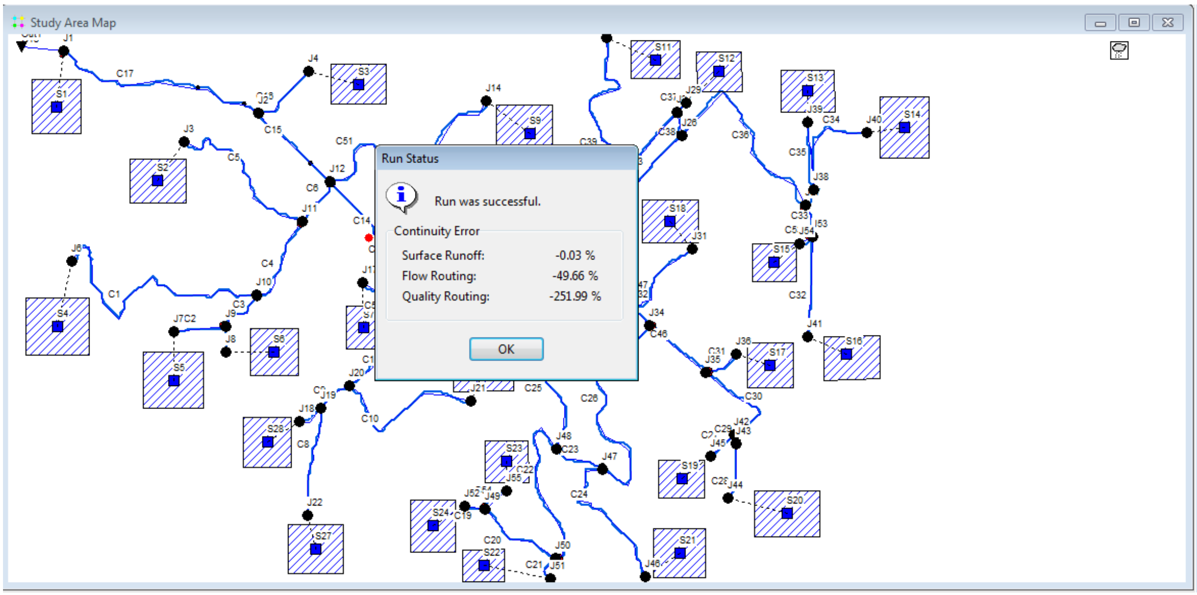


Figure 31. Simulation of rainfall data with continuity error

6. CONCLUSIONS AND RECOMMENDATIONS

6.1. Conclusions

Detailed catchment delineation and subdivision was successfully performed in two similar urban catchments with a total area of 47.23 km2. This resulted in 52 subcatchments with an average area of 1.7 hectares. The process involved substantial manual work but can henceforth be sped up by the routines established in this study. A complete automation would however be impossible due to the typical defects in input data. The results showed sufficient spatial accuracy for the intended use. Altogether, a catchment delineation and subdivision for use in SWMM modeling was found feasible to perform even for a large urban area using a GIS-based approach. Publicly available spatial data and data on stormwater system layout are all that is needed as process input.

Existing drainage capacity of the canal has been computed and comparison was made both with peak storm water runoff with 10 and 25 years return period and modeled storm water runoff . The effect was identified as it can continue harming the town with flooding. So, some remedy shall be taken at the place to control flooding and water stagnation problem by improving the capacity of channel. A possible remedy is to dig land to make an artificial channel and construct retaining wall on stream bank where is necessary. Storm water runs over roads, rooftops, and compacted land has also aggravate drainage problem and poses a physical hazard to a habitats life and property owing to the increase in water velocity and volume for surface runoff water.

This is highly occurs due to;

- ☞ Negligence to construct ditch and flood protection structure;
- ☞ Disposing of solid waste in open ditch and natural waterway;
- ☞ Overflow of ditch and flow channels during high rainy time;
- ☞ Negligence to maintain a deteriorated ditch and culvert;
- ☞ Dumping construction material on the road and open ditch;
- ☞ Constructing improperly designed ditch that cannot accommodate storm of intense rainfall and roads are eroded and deteriorate due this over flow runoff water;
- ☞ Formation of gorge and soil erosion

Parameterization of heterogeneous subcatchments of low spatial resolution turned out to be challenging and inaccurate. No clear procedures have been presented in literature on how to choose certain parameter values in an aggregated SWMM approach without calibration. A combination of GIS methods and literature values was used for the purpose, but the results were found partly inaccurate with respect to calibrated values. To reliably use the results in modeling, either calibration should be performed or the model sensitivity for the most hard-to-define parameters such as flow width or depression storage should be proved minor.

The effort needed for drainage network parameterization proved to be highly dependent on the quality of the input data. If the drainage canals and junction data has many gaps or is stored in the database in a cumbersome manner, detailed modeling of large systems may easily grow non-feasible.

The SWMM model runs conducted were troubled with excessive continuity errors. This is a sign that the selection of appropriate simulation time-steps for long-term modeling with low spatial resolution is not simple and should be further studied. Continuity errors cause the results for urban water balance to be slightly biased in our country since there is shortage of daily weather record data.

Overall, the methods developed in this study provide a feasible approach for SWMM parameterization for large urban areas. Despite the further research still needed, non-calibrated SWMM applications of low spatial resolution seem promising for certain tasks in stormwater modeling. The approach would suit especially for rapid stormwater modeling when studying large-scale processes, such as the effects of the climate change on urban water balance

In this study area, the additional drainage discharge canal is very critical, since the area is affected by storm water runoff that is behind the capacity of existing drainage canals.

6.2. Recommendations

There are several lines of research that ought to be further worked upon based on the findings of this study. The above discussed results seem to provoke a great number of related questions to which the answers could be found by conducting more simulations with the model built for this study.

Regarding subcatchment delineation for use in SWMM modeling, the best method of choosing drainage points to get catchments subdivided to a certain degree should be sought for. These methods could also be developed to aim for land-use homogeneity within the subcatchments. Related to that, the general effect of the subdivision level on the model performance would be useful to explore. It could definitely be worth trying to experiment with an even finer subdivision scheme if only drainage network data of a better quality was available.

It would be interesting to know, whether also the dry-weather time step could affect the errors. What might also be useful is to judge if the most appropriate time steps. Moreover, it would be interesting to study the outcome of using weather observations data of different time-resolutions. It may be that the partly-hourly-partly-daily data used here was not accurate enough for the task at hand.

Further analysis should also address in more detail the precision of the long-term water balance components given by the simulations. If those results were found to be far from reality, it would be valuable to see how the methods proposed here could be refined for a better outcome. Proving the results on water balance components well-reasoned would also open possibilities to use coarse-scale long-term SWMM modeling for simulating the effect of climate change scenarios in large urban areas.

Finally, additional drainage discharge ditches are very critical to mitigate flooding problem caused by Storm water runoff water which is behind the capacity of existing drainage canals. The Additional drainage ditch Length 5times that of the existing Should be needed and Stormwater runoff Should be routed to the outlet /to the Gogeti river stream/ .

7. REFERENCES

AMAGUCHI, H., KAWAMURA, A., OLSSON, J. and TAKASAKI, T., 2012. Development and testing of a distributed urban storm runoff event model with a vector based Catchment delineation. Journal of Hydrology,

BARCO, J., WONG, K.M. and STENSTROM, M.K., 2008. Automatic calibration of the US EPA SWMM model for a large urban catchment. Journal of Hydraulic Engineering.

BELING, F., GARCIA, J., PAIVA, E., BASTOS, G. and PAIVA, J., 2011. Analysis of the SWMM Model Parameters for Runoff Evaluation.

CALLOW, J.N., VAN NIEL, K.P. and BOGGS, G.S., 2007. How does modifying a DEM to reflect known hydrology affect subsequent terrain analysis.

Chanson H., (2004), The Hydraulics of Open Channel Flow, Butterworth-Heinemann, Oxford, UK.

CHEN, M. and TUCKER, C., 2003. Comparing Different Approaches Of Catchment Delineation. Cite seer.

CIWEM, 2009, Integrated Urban Drainage Modelling Guide, Hills city

CHOI, K. and BALL, J.E., 2002. Parameter estimation for urban runoff modeling. Urban Water.

Chow V.T., Maidment D., Mays L.,(1988), Applied Hydrology, McGraw-Hill Science/Engineering/Math, New York

Creek Solutions, Inc., 2010, WWHM4 SWMM element modeling, Designing for Multiple Municipal Storm Water Criteria, Creek.

David B. Thompson, Draft: 20 September 2006, , The Rational Method, Texas Tech University.

DINGMAN, S.L., 1994. Physical hydrology. Prentice Hall Upper Saddle River, NJ.

Dooge J-C.I., (1957), The Rational Method For Estimating Flood Peaks, Engineering, No.184.

DUNNE, T., 1978. Field studies of hill slope flow processes. Hill slope hydrology ,Mc Hill.

DURRANS, S.R., 2003. Storm water Conveyance Modeling and Design. Waterbury, CT: Haestad Methods, Inc.

ESRI, 2012. ArcGIS 10.1. Redlands, CA, USA: Esri Inc.

Fairfield J., Leymarie P., (1991), Drainage Networks from Grid Digital Elevation Models,

FDREMW, Federal Democratic Republic of Ethiopia, Ministry of Water Resources. (2002). Water sector development programme. Addis Ababa.

FLETCHER, T., ANDRIEU, H. and HAMEL, P., 2012. Understanding, management and modeling of urban hydrology and its consequences for receiving waters; a state of the art. Advances in Water Resources, .

GHOSH, I., 2010. Characterizing and understanding the effects of spatial resolution in urban hydrologic simulations.

GHOSH, I. and HELLWEGER, F.L., 2011. Effects of spatial resolution in urban hydrologic simulations. Journal of Hydrologic Engineering, **17**(1), pp. 129-137.

Glenn E. Moglen, CEE 5734 – Urban Hydrology and Stormwater Management, Calibrating SWMM.

GREEN, W.H. and AMPT, G., 1911. Studies on soil physics, The flow of air and water through soils. J. Agric .

HAASE, D., 2009. Effects of urbanization on the water balance – A long-term trajectory. Environmental Impact Assessment Review.

HAMILL, L., 2001. Understanding Hydraulics. Second Edition edn. Bristol: Palgrave.

HARGREAVES, G. and ALLEN, R., 2003. History and Evaluation of Hargreaves Evapotranspiration Equation. Journal of Irrigation and Drainage Engineering.

HORTON, R.E., 1933. The role of infiltration in the hydrologic cycle. Transactions, American Geophysical Union, **14**.

JACOBSON, C.R., 2011. Identification and quantification of the hydrological impacts of imperviousness in urban catchments: A review. Journal of environmental management, 92.

James B. Campbell, Virginia 24060, delineating drainage networks in urban areas

Journal of Hydraulic Engineering , 2012,EPA SWMM model for a large urban catchment.

KABAT, P., HUTJES, R.W.A. and FEDDES, R.A., 1997. The scaling characteristics of soil parameters: From plot scale heterogeneity to sub grid parameterization. Journal of Hydrology, 190.

Kent County Council . 2015. Drainage and Planning Policy Statement, Local flood risk management strategy guidance.

KREBS, G., KOKKONEN, T., VALTANEN, M., SETÄLÄ, H. and KOIVUSALO, H., 2013b. Spatial resolution considerations for urban hydrological modeling. Journal of Hydrology (under review), .

KREBS, G., KOKKONEN, T., VALTANEN, M., KOIVUSALO, H. and SETÄLÄ, H., 2013a. A high resolution application of a stormwater management model (SWMM) using genetic parameter optimization. Urban Water Journal.

LIONG, S., CHAN, W. and LUM, L., 1991. Knowledge-based system for SWMM runoff component calibration. Journal of Water Resources Planning and Management.

Luis F. Leon ,Version 2,October 2011, Step by Step Geo-Processing.

Oromia Urban Planning Institute .(OUPI) May, 2010, Urban Planning Preparation, Addis Ababa

PARK, S.Y., LEE, K.W., PARK, I.H. and HA, S.R., 2008. Effect of the aggregation level of surface runoff fields and sewer network for a SWMM simulation , Desalination. Prof. Dr. Peter Krebs (2011), Rainfall, Runoff and Time coefficient method, Industries wasserwirtschaft.

RAWLS, W., AHUJA, L., BRAKENSIEK, D. and SHIRMOHAMMADI, A., 1992. Infiltration and Soil Water Movement. In: D. MAIDMENT, ed, Handbook of hydrology. McGraw-Hill Inc., .

RODRIGUEZ, F., CUDENNEC, C. and ANDRIEU, H., 2005. Application of morphological approaches to determine unit hydrographs of urban catchments. Hydrological Processes.

ROSSMAN, L.A., 2010. Storm Water Management Model User's Manual - Version 5.1 U.S. Environmental Protection Agency.

SAUNDERS, W., 1999. Preparation of DEMs for Use in Environmental Modeling Analysis, 1999 Esri User Conference.

SCALENGHE, R. and MARSAN, F.A., 2009. The anthropogenic sealing of soils in urban areas. Landscape and Urban Planning.

Schade, T., P. Boulos, C.W. Baxter and M. Muleta. 2007. "Comprehensive ArcGIS-Based Urban Drainage

Shen et al., 2005, Urbanization affects environmental factors, Utah.

SMITH, D., LI, J. and BANTING, D., 2005. A PCSWMM/GIS-based water balance model for the Reesor Creek watershed. Atmospheric Research.

Tang et al., 2003, sewerage systems and stormwater drains ,USA.

Texas Department Of Transportation, 2011. Hydraulic Design Manual. Texas, USA: Texas Department of Transportation.

THIEKEN, A.H., LUECKE, A., DIEKKRUEGER, B. and RICHTER, O., 1999. Scaling input data by GIS for hydrological modelling. Hydrological Processes.

THIESSEN, A.H., 1911. Precipitation averages for large areas. Monthly Weather Review.

THOMPSON, D.B. and CLEVELAND, T.G., 2009. Subdivision of Texas Watersheds for Hydrologic Modeling. FHWA/TX -0−5822−01−2. Lubbock, Texas: Texas Tech

University College of Engineering.

Tyler Baumbach, Suzette R. Burckhard, Watershed Modeling Using Arc Hydro Tools. Geo HMS, and HEC-HMS

VALTANEN, M., SILLANPÄÄ, N. AND SETÄLÄ, H., 2013. Effects of land use intensity on stormwater runoff and its temporal occurrence in cold climates. Hydrological Processes, , pp. n/a-n/a.

Venkatesh Merwade , August 2012, School of Civil Engineering, Purdue University

Walter McDonald, , 2010, Using ArcGIS, HEC-GeoHMS and HEC-HMS to Storm Water Modeling.

WOOLHISER, D.A., 1981. Physically Based Models of Watershed Runoff, V.P. SINGH, ed. In: Rainfall-Runoff Relationship, May 18-21 1981, Water Resources Publications, pp. 189--202.

ZHANG, H., WANG, Y., WANG, Y., LI, D. and WANG, X., 2013. The effect of watershed scale on HEC-HMS calibrated parameters: a case study in the Clear Creek watershed in Iowa, USA. Hydrology and Earth System Sciences Discussions, **10**, pp. 965-998.

Appendix

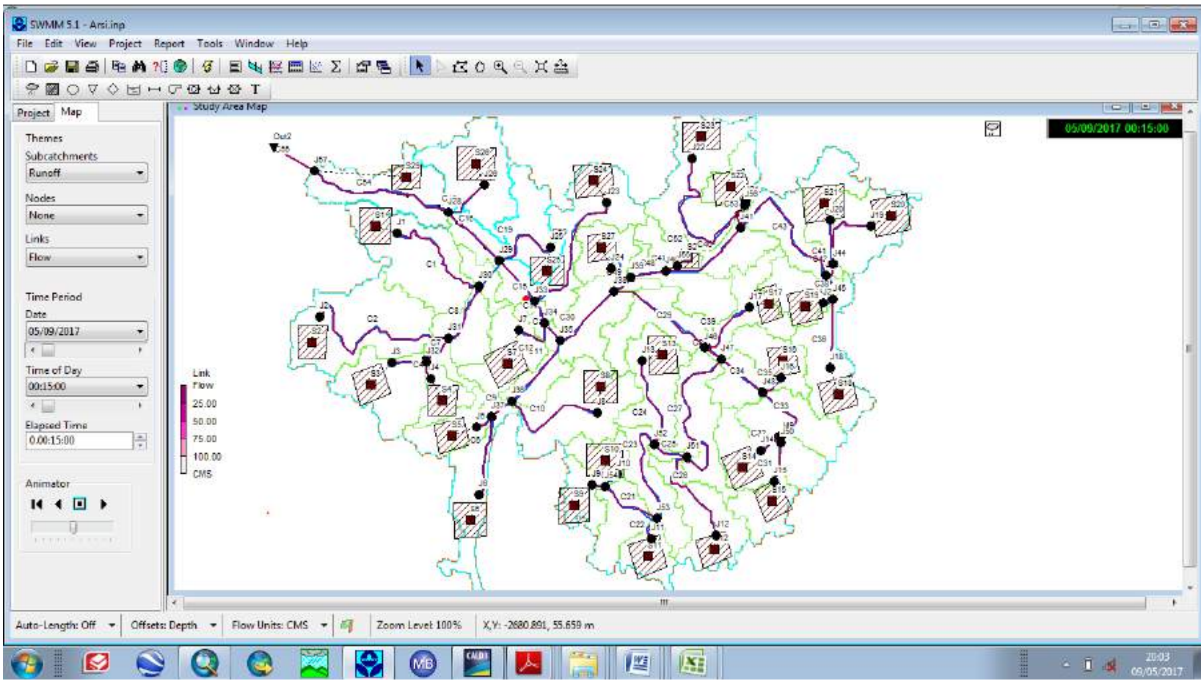


Figure 32. flow direction by SWMM

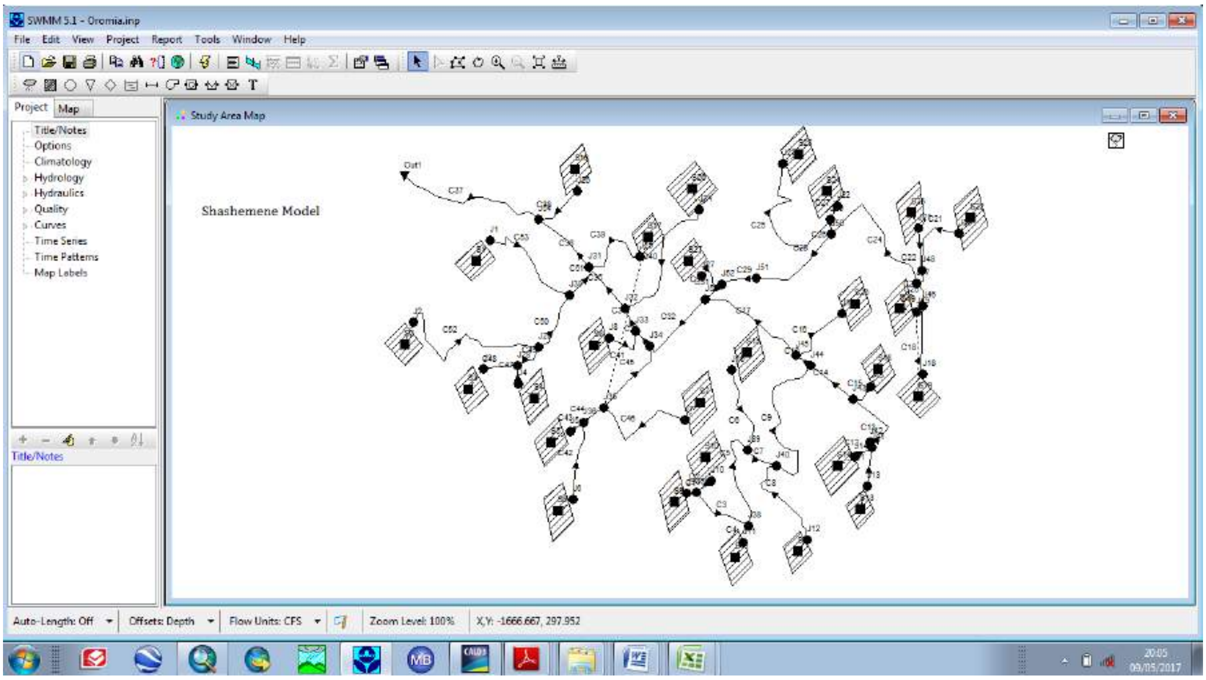


Figure 33. Catchment representation in SWMM

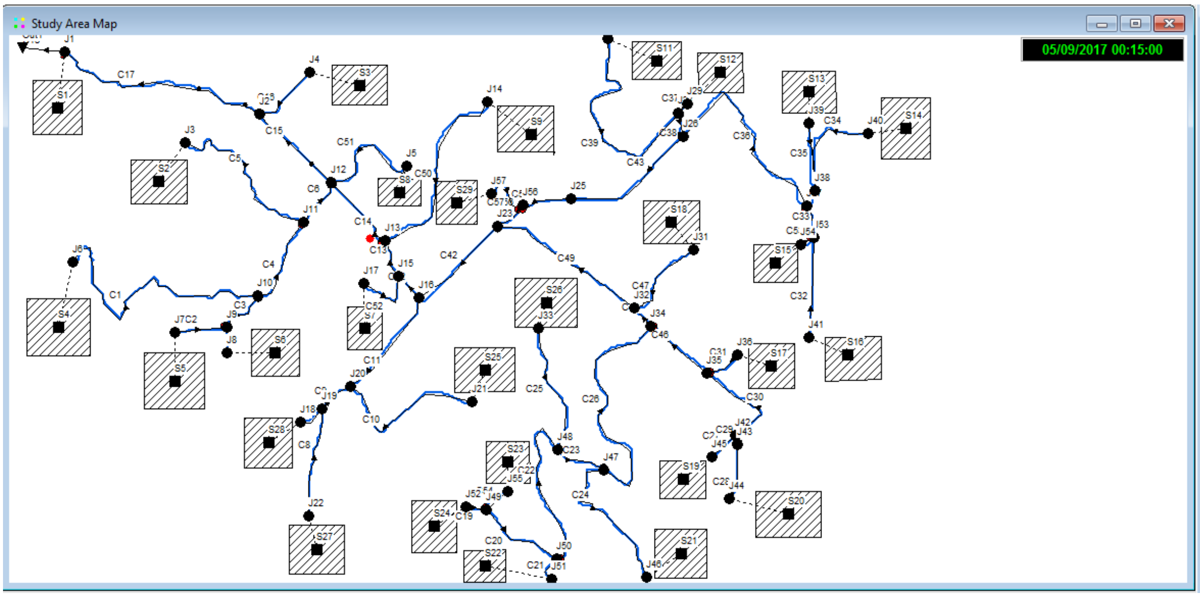


Figure 34.Flow outfall on SWMM

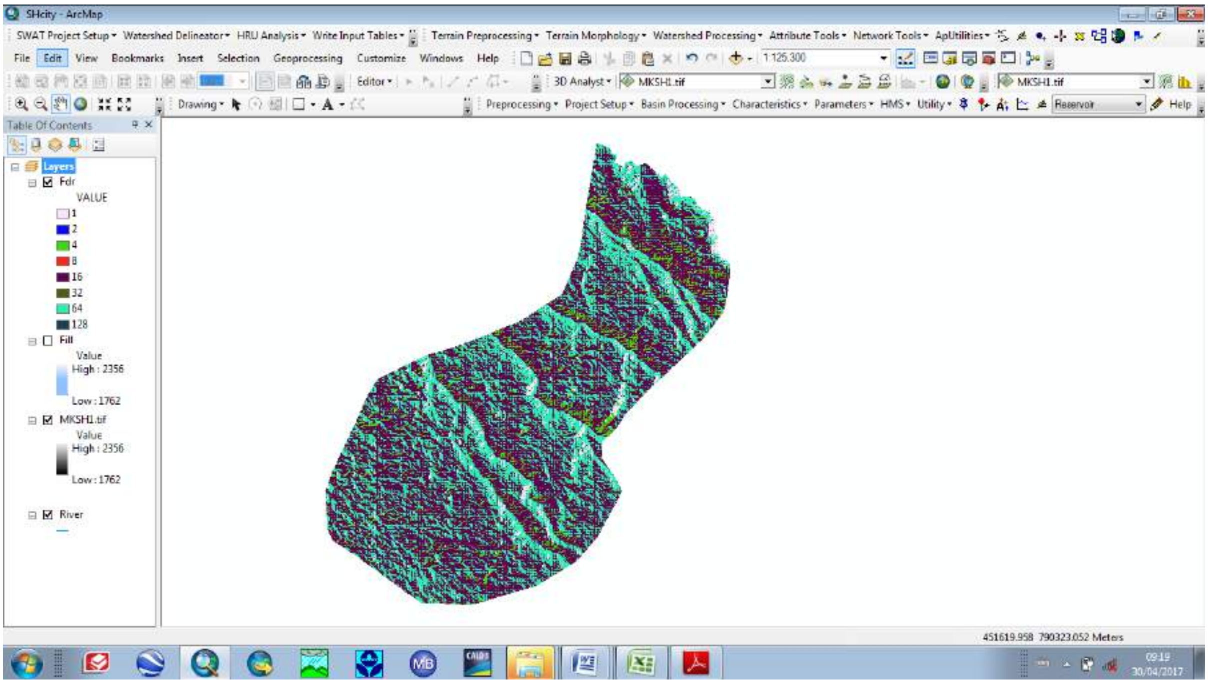


Figure 35. Flow direction of Shashemene Catchment

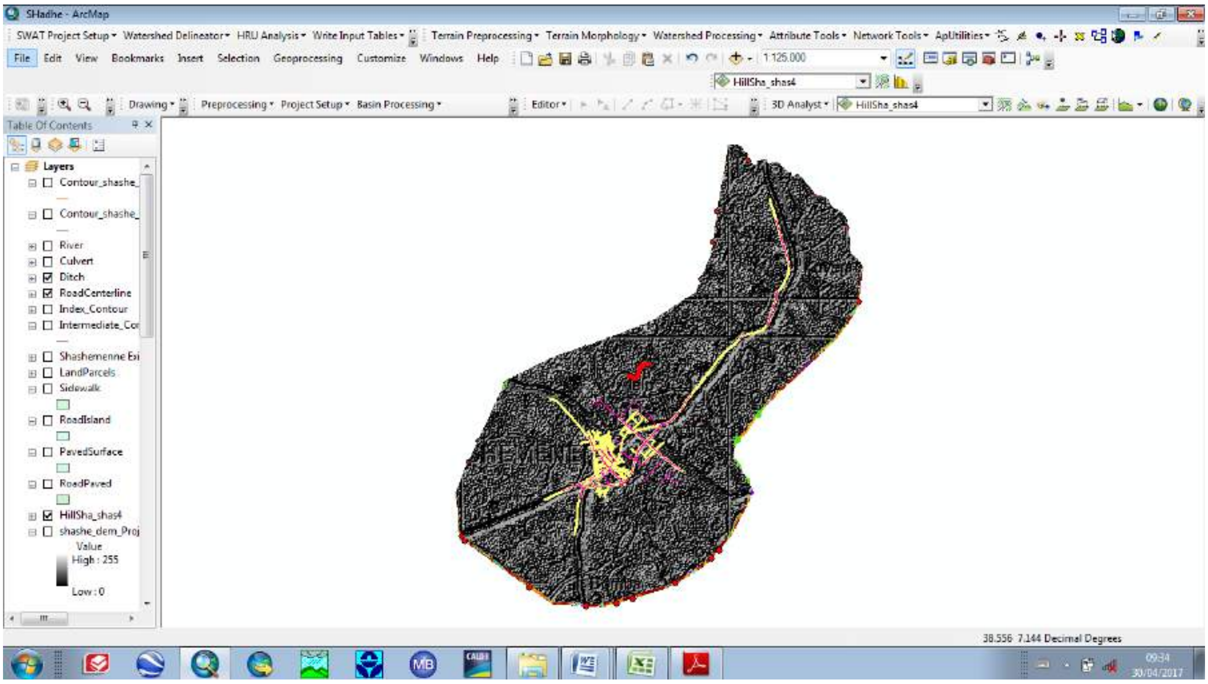


Figure 36. Hill shed of Shashemene

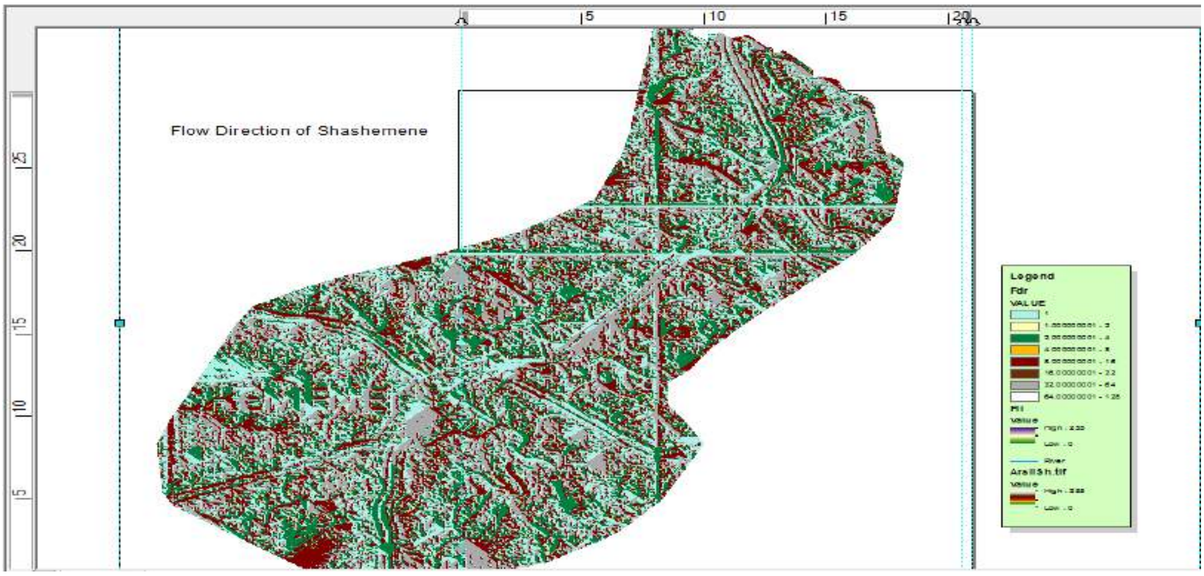
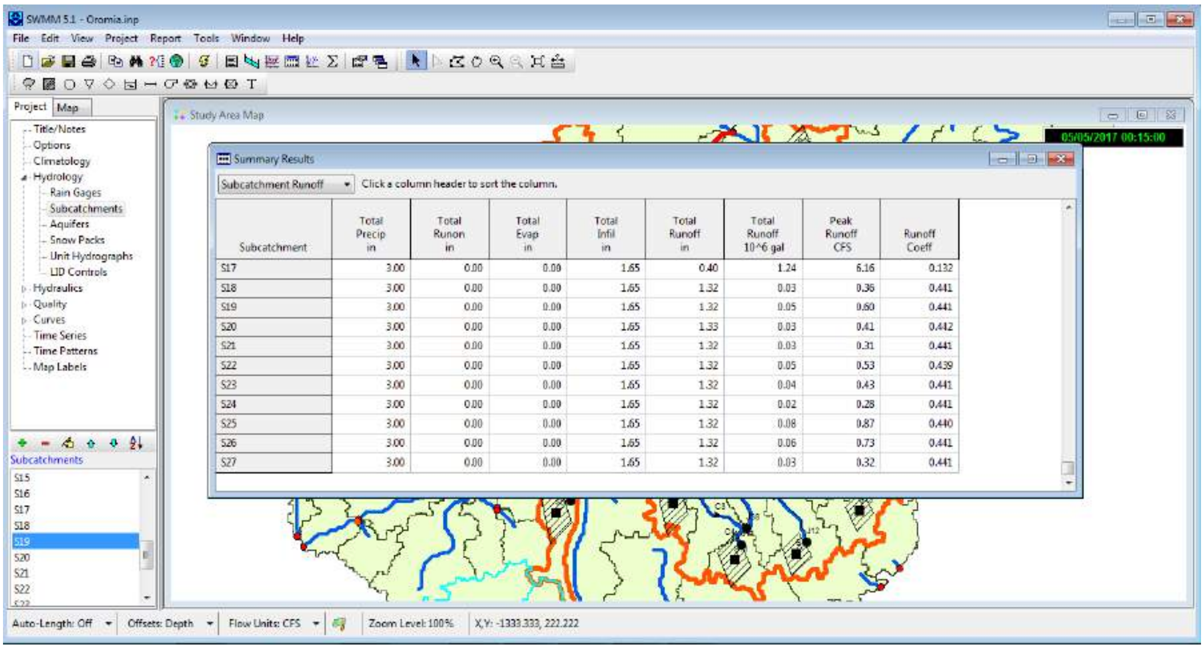


Figure 37.Flow direction and slope of the catchment



Subcatchment	Total Precip in	Total Runon in	Total Evap in	Total Infil in	Total Runoff in	Total Runoff 10^6 gal	Peak Runoff CFS	Runoff Coeff
S17	3.00	0.00	0.00	1.65	0.40	1.24	6.16	0.132
S18	3.00	0.00	0.00	1.65	1.32	0.03	0.36	0.441
S19	3.00	0.00	0.00	1.65	1.32	0.05	0.60	0.441
S20	3.00	0.00	0.00	1.65	1.33	0.03	0.41	0.442
S21	3.00	0.00	0.00	1.65	1.32	0.03	0.31	0.441
S22	3.00	0.00	0.00	1.65	1.32	0.05	0.53	0.439
S23	3.00	0.00	0.00	1.65	1.32	0.04	0.43	0.441
S24	3.00	0.00	0.00	1.65	1.32	0.02	0.28	0.441
S25	3.00	0.00	0.00	1.65	1.32	0.08	0.87	0.440
S26	3.00	0.00	0.00	1.65	1.32	0.06	0.73	0.441
S27	3.00	0.00	0.00	1.65	1.32	0.03	0.32	0.441

Figure 38. Summary of model result

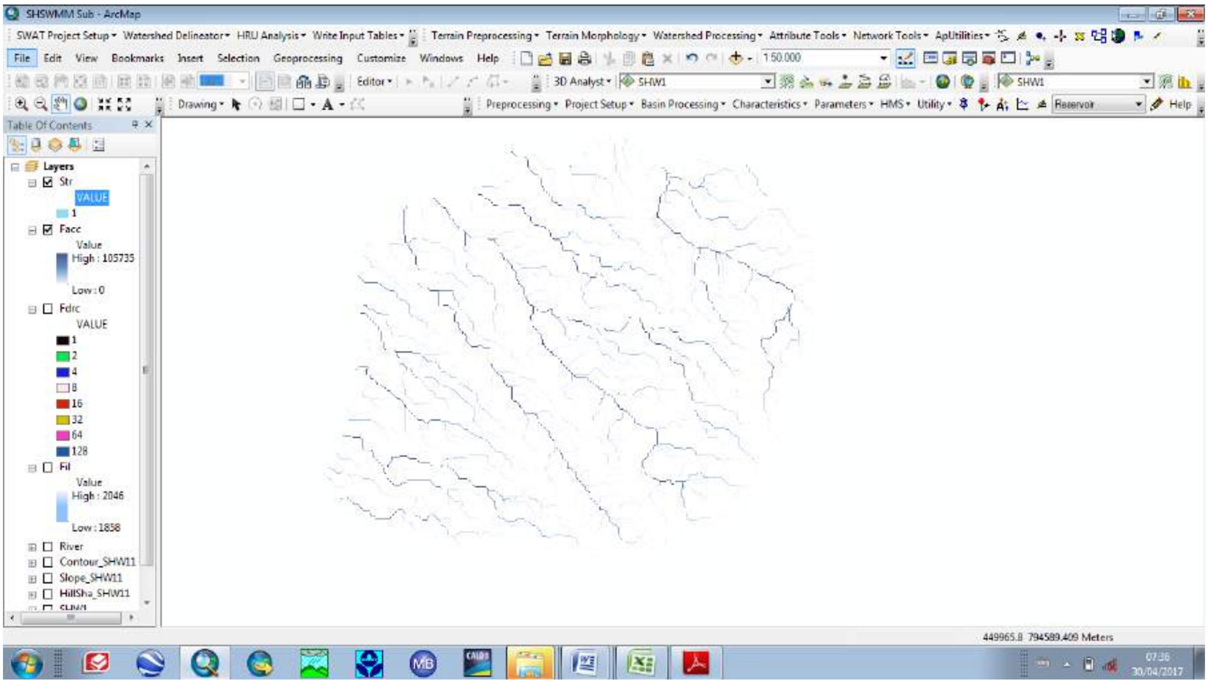


Figure 39. Flow direction as backdrop image on SWMM

Calculation of Design Rainfall Intensity
and Peak Storm Water Runoff Rate - S.I. units

(for specified return period and storm duration)

Inputs

Drainage Area, **A** =	47	ha	Runoff Coefficient, C =	0.4	
Design Return Period =	10	years	Design Storm Duration, **d** =	30	min
			(= time of concentration)		

Data from IDF (intensity-duration-frequency) graph or table for the design location:

Input i, mm/hr	Input d, min	Calculated 1/i, hr/mm
102	15	0.01
67	30	0.01
42	60	0.02

Calculation of equation constants (a & b) using linear regression:

slope =	0.000309	= 1/a	
	a = 1/slope =		3235
intercept =	0.0054	= b/a	
	b = a*intercept =		17.3

Calculation of Design Rainfall Intensity, i, using the equation: i = a/(d + b):

(using the value for storm duratin, **d**, specified above)

Design Rainfall Intensity, **i** = **68.4** mm/hr

Calculation of Design Peak Storm Water Runoff Rate, Q, using the equation: Q = CiA

Design Peak Storm Water Runoff Rate, **Q** = **3.60** m^3/s

Table 40. Peak Storm water run of for 10years return period

Calculation of Design Rainfall Intensity
and Peak Storm Water Runoff Rate - S.I. units

(for specified return period and storm duration)

Inputs

Drainage Area, **A** =	47	ha	Runoff Coefficient, C =	0.4	
Design Return Period =	25	years	Design Storm Duration, **d** =	30	min
			(= time of concentration)		

Data from IDF (intensity-duration-frequency) graph or table for the design location:

Input	Input	Calculated
i, mm/hr	d, min	1/i, hr/mm
102	15	0.01
67	30	0.01
42	60	0.02

Calculation of equation constants (a & b) using linear regression:

slope = 0.000309 = 1/a

a = 1/slope = **3235**

intercept = 0.0054 = b/a

102	15	0.01
67	30	0.01
42	60	0.02

a = 1/slope = **3235**

intercept = 0.0054 = b/a

b = a*intercept = **17.3**

Calculation of Design Rainfall Intensity, i, using the equation: i = a/(d + b):

(using the value for storm duratin, **d**, specified above)

Design Rainfall Intensity, **i** = **80.0** mm/hr

Calculation of Design Peak Storm Water Runoff Rate, Q, using the equation: Q = CiA

Design Peak Storm Water Runoff Rate, **Q** = **4.21** m^3/s

Table 41. Peak Storm water run of for 25years return period

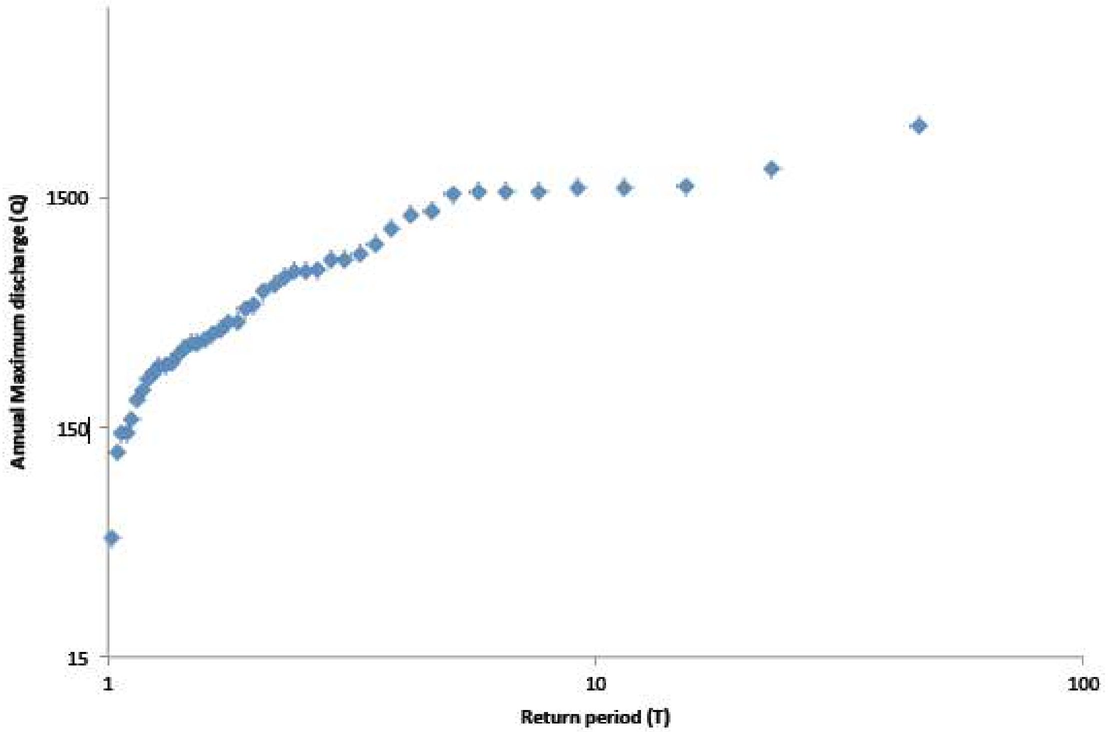


Figure 42.Graph of Discharge Vs Return Period

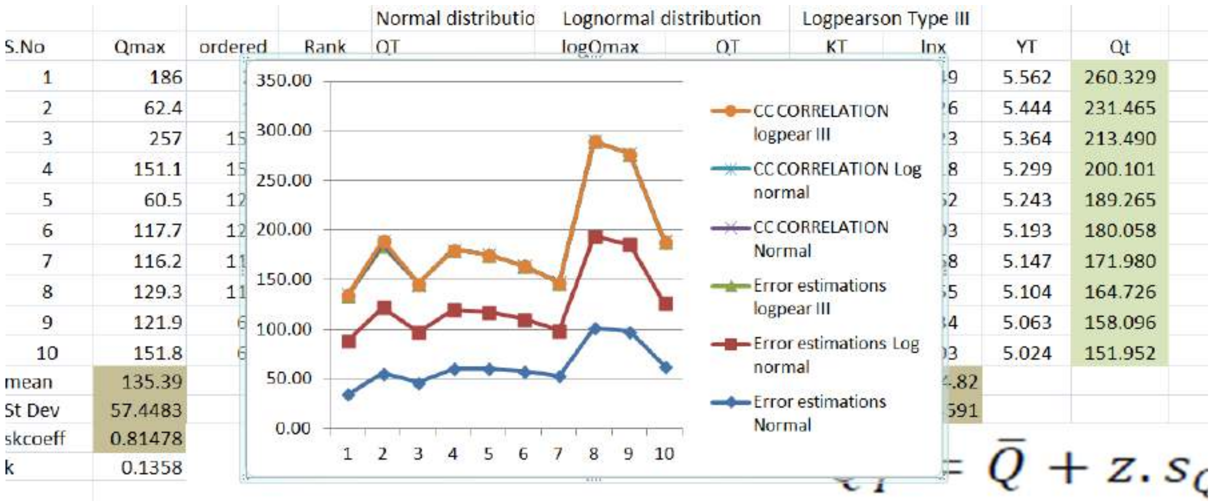


Figure 43. Graphical representation of Simulations

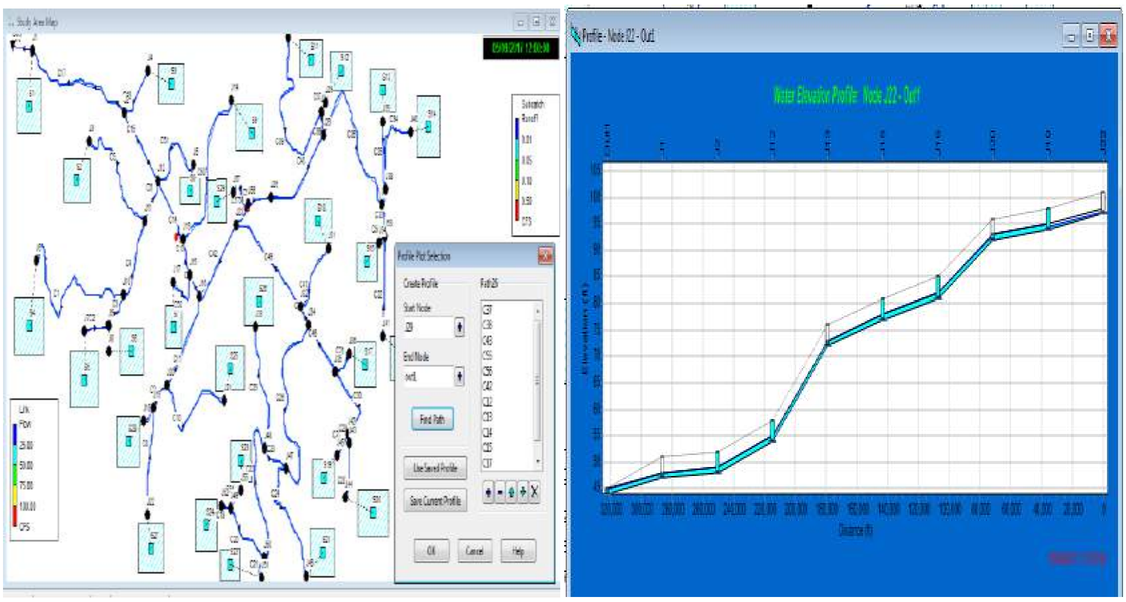


Figure 44.Flow path, graph of flow path and dynamic simulation with respect to Canal capacity.

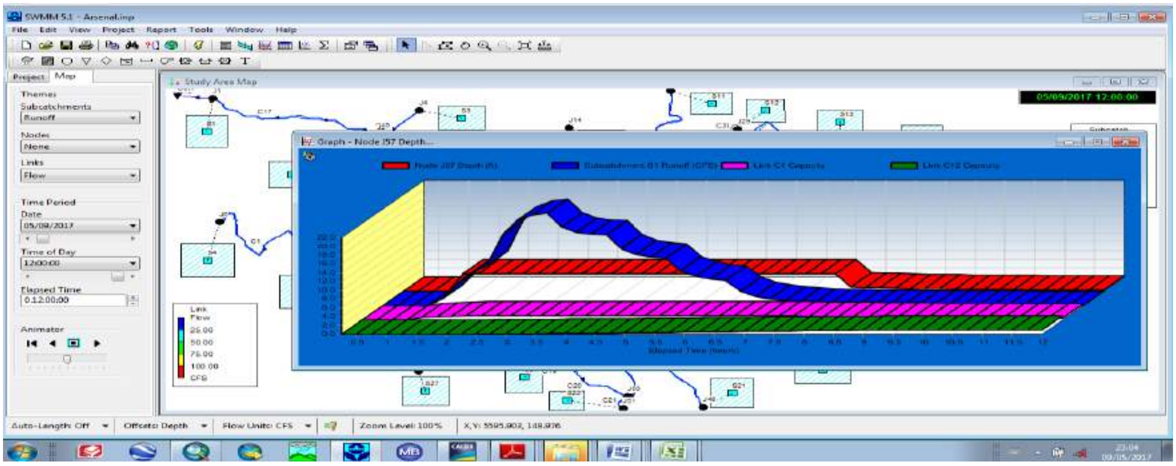


Figure .45. Graphical representation of Simulations of subcatchment flow

Table 9, Width ,Determination from Sub catchments

Number of Catchments	Shape_Area	Shape_Length	Shape_Width
1	2079363.106	11853.65218	175
2	1712347.551	10082.15799	170
3	1568219.512	9870.725955	159
4	969847.463	6538.997449	148
5	640624.3492	5315.375925	121
6	1660317.478	12191.62758	136
7	470249.1289	4429.487033	106
8	984072.3014	7720.794855	127
9	1298167.694	6960.82464	186
10	2020707.989	9871.039076	205
11	1145586.214	8142.094848	141
12	1773801.673	8521.040912	208
13	4448.936464	463.803588	10
14	358586.0108	5231.365749	69
15	414643.2452	3881.184758	107
16	49382.72848	1139.01576	43
17	492491.4772	5188.44457	95
18	933832.6814	8309.600661	112
19	748305.2722	5568.4523	134
20	76966.23358	1476.446167	52
21	122790.9042	2361.80047	52
22	744750.9693	5104.407466	146
23	179294.8321	3037.596304	59
24	189974.0049	3669.858043	52
25	533444.2037	6117.296597	87
26	936059.3044	9238.76041	101
27	1042414.742	7214.165511	144
28	107221.5399	1645.408143	65
29	2500363.747	10798.89091	232
30	238025.9166	4092.190417	58
31	490734.8792	5104.55011	96
32	82751.86404	1476.314909	56
33	1500257.341	8985.948468	167
34	1409928.894	7171.854055	197
35	510760.0885	4640.751356	110

36	598399.0444	4682.429392	128
37	1292901.545	8606.331713	150
38	838641.2334	6369.777507	132
39	1066006.913	6960.19486	153
40	168622.3576	2573.319342	66
41	1142129.122	9746.967511	117
42	936116.3345	8100.358217	116
43	940552.8505	5484.675449	171
44	2669.477147	253.164583	11
45	140593.7649	2783.893792	51
46	1314311.474	9618.666351	137
47	568610.3178	4302.716877	132
48	17352.04915	759.308239	23
49	1894090.927	10715.1615	177
50	685639.7259	6876.563375	100
51	563275.3195	4303.070922	131
52	554385.3512	4640.645556	119
Sum	42715032.08		6039
Mean			116

Table 10. Descriptions of the four SCS soil groups Of Ethiopia

Soil Types Hydrologic Soil Group

Oe--- Eutric Histosols --------------------D
Qc ---Cambric Arenosols -----------------A
Rc--- Calcaric Regosols -------------------A
Re--- Eutric Regosols--------------------- A
Th--- Humic Andosols ---------------------B
Tm--- Mollic Andosols --------------------B
Tv Vitric Andosols -----------------------B
Vc --Chromic Vertisols ------------------D
Vp-- Pellic Vertisols ---------------------.D
Xh --Haplic Xerosols ---------------------B
Xk-- Caloic Xerosols ---------------------B
Xl-- Luvic Xerosols -----------------------C
Yy-- Gypsic Yermosols------------------ B
Zg-- Gleyic Solonchaks ------------------D
Zo --Orthic Solonchaks ------------------B

Source: Ministry of Agriculture

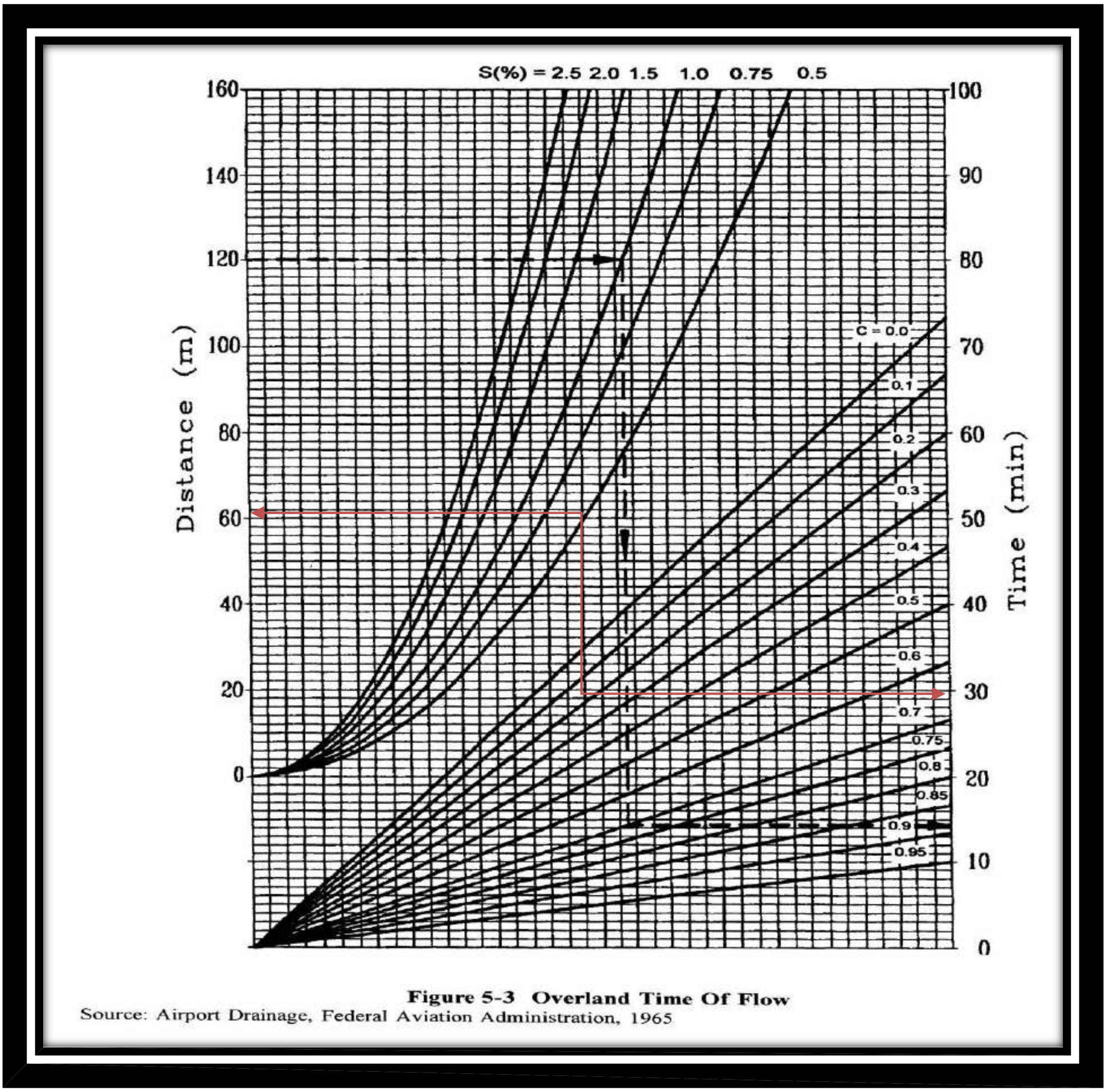
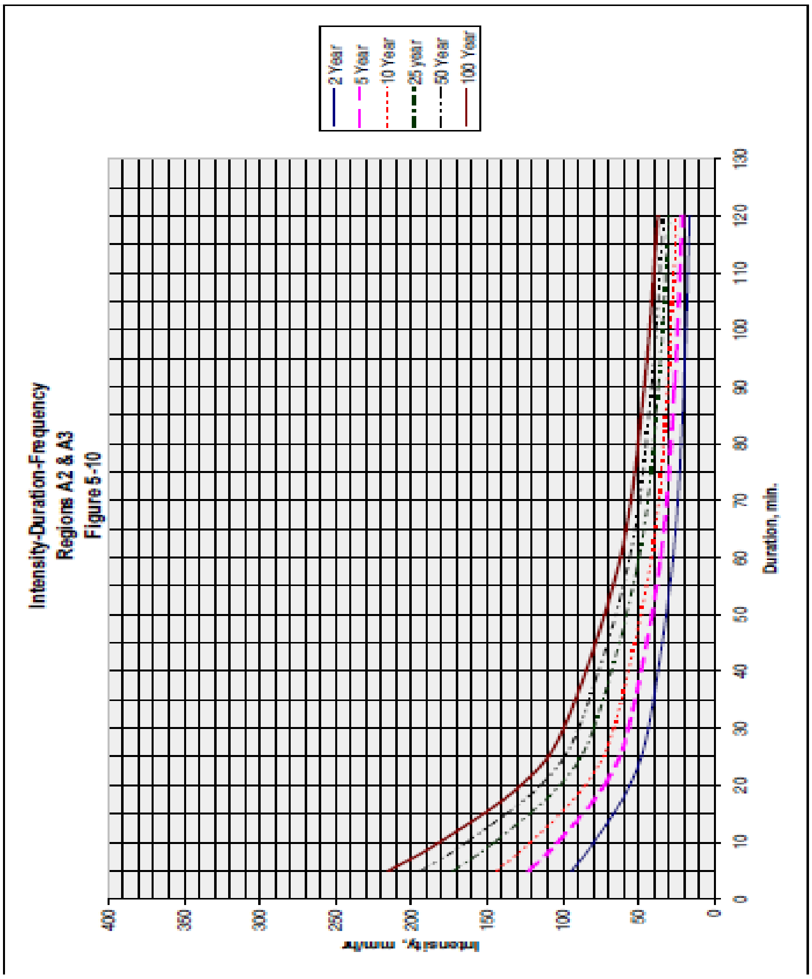
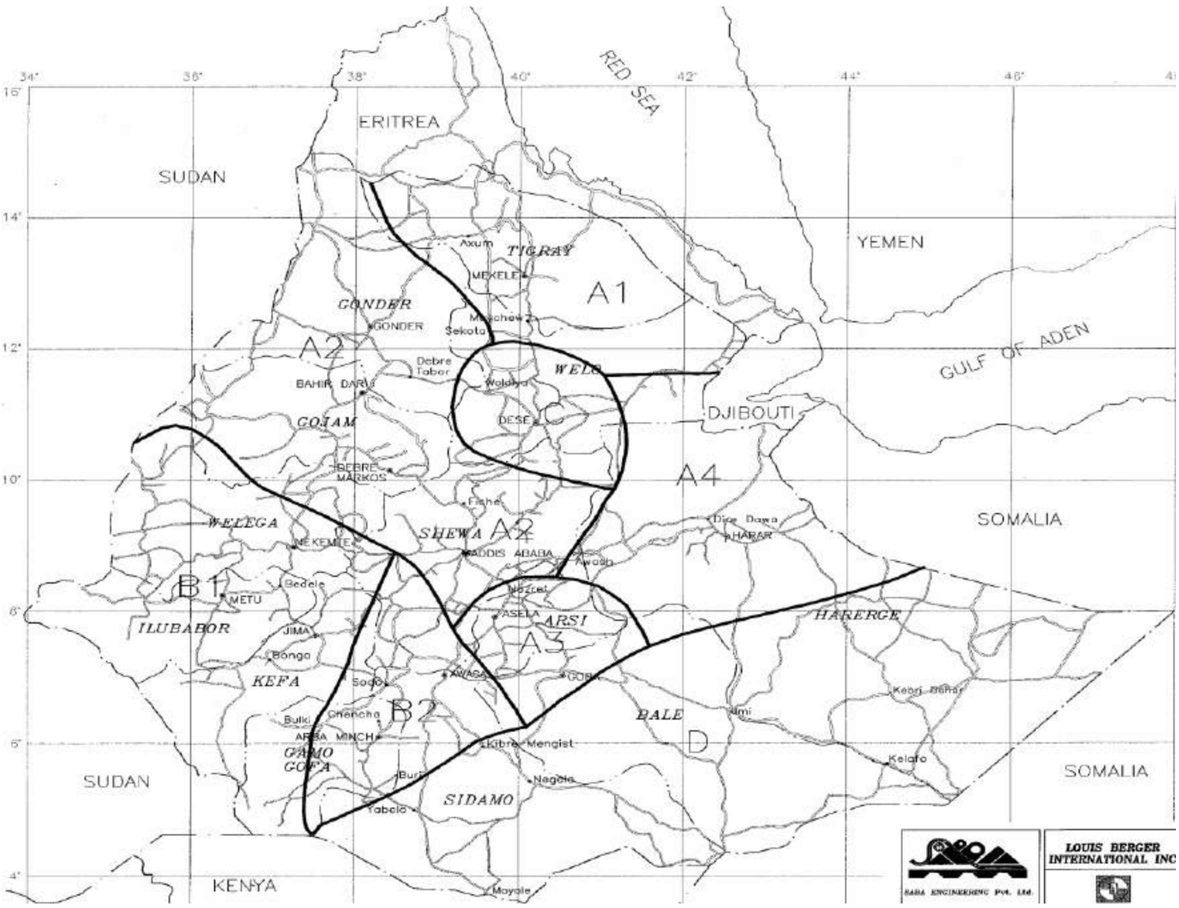


Figure 46, Overland time of Flow determination



Figure, 47, Intensity, Duration and frequency (Source, ERA drainage manual, 2002)



(Source, ERA drainage Manual, Figure 48.Precipitation in Ethiopia by rainfall region)

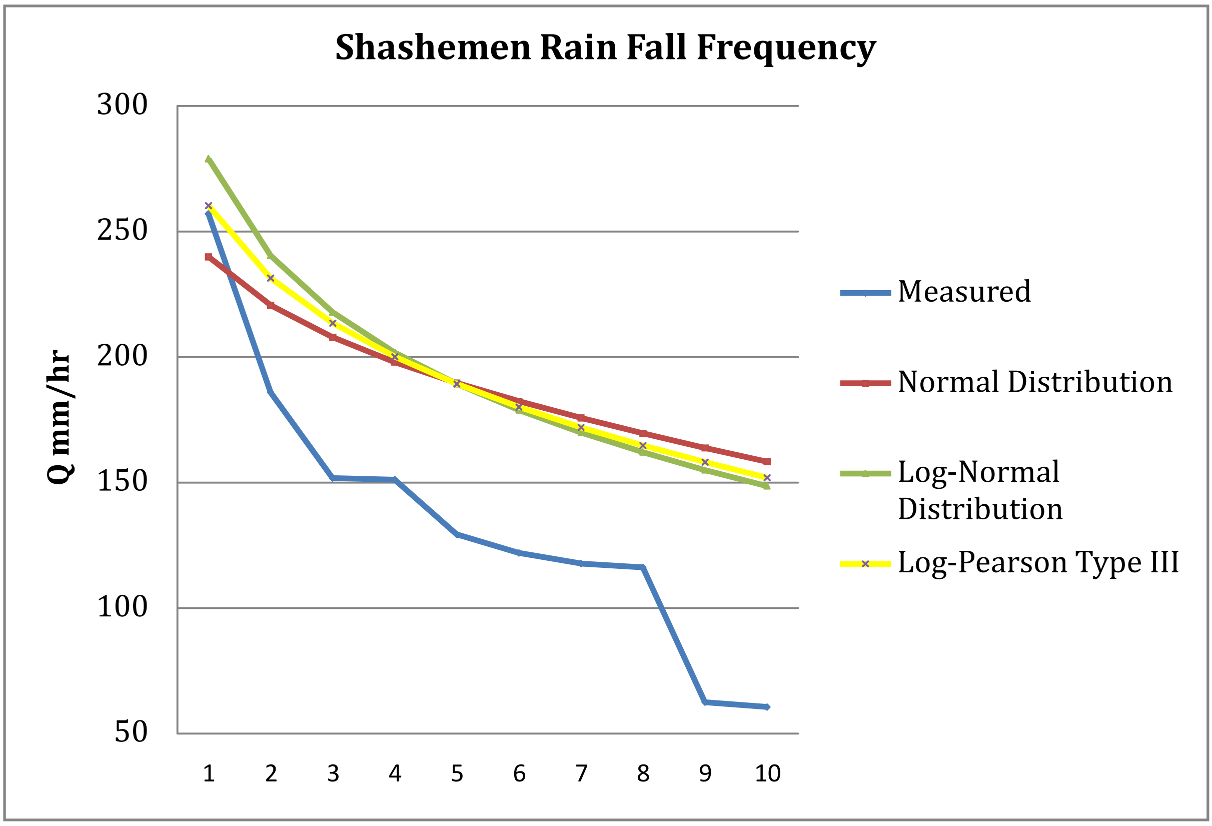


Figure49a. Max. Rain Fall Frequency of Shashemene

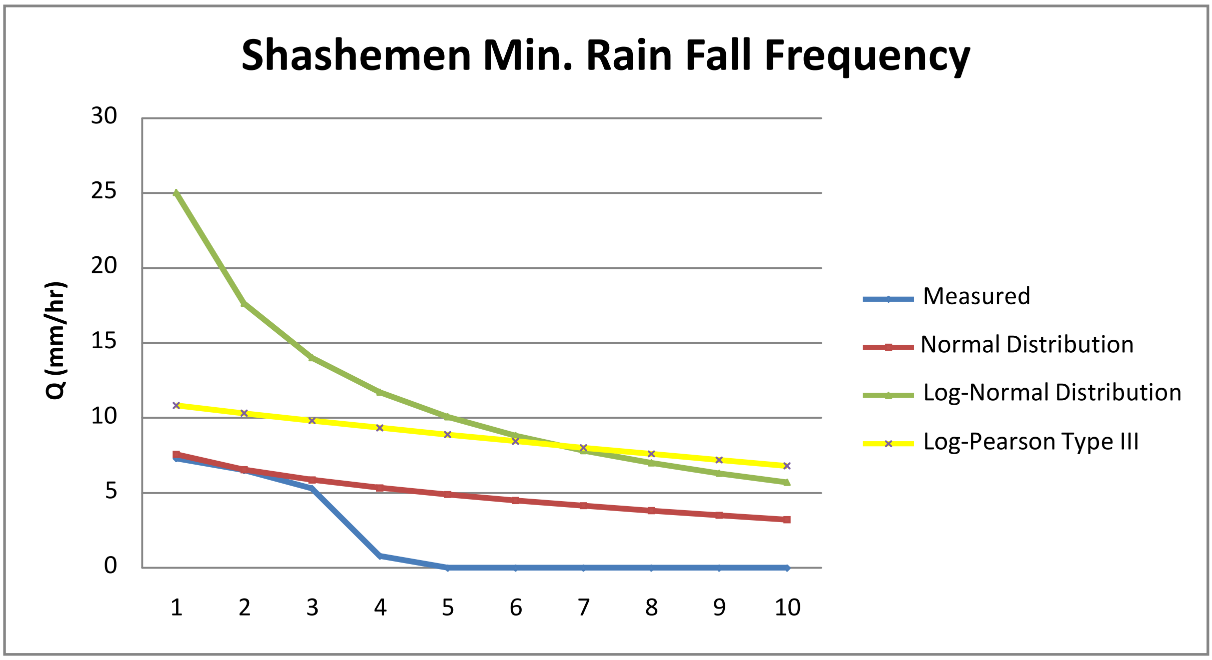


Figure 49b. Min. Rain Fall Frequency of Shashemene

Figure-3b. Land slide in Shashemene area (Captured in June,2016)