



ADDIS ABABA UNIVERSITY  
INSTITUTE OF TECHNOLOGY  
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

MODELING AND ANALYSES OF URBAN FLOODING IN BOLE SUBCITY  
SYSTEM PERFORMANCE AND EVALUATION OF POSSIBLE  
IMPROVEMENTS USING EP SWMM5

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

By

Nejib Hassen Abdullahi

Addis Ababa  
Ethiopia  
November, 2016



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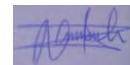
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## DECLARATION

I declare that this thesis, which I submit to School of Civil and Environmental Engineering of Addis Ababa University in partial fulfillment of the requirement of degree of Master of Science in Hydraulics Engineering, is my own personal effort. The thesis has not been submitted previously, in whole or in part, to qualify for any other academic award. Furthermore, I took reasonable care to ensure that the work is original, and, to the best of my knowledge, does not breach copyright law, and has not been taken from other sources except where such work has been cited and acknowledged within the text.



Nejb Hassen Abdullahi

November, 2016

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**ABSTRACT**

The runoff generation in Bole area Addis Ababa is increasing due to a combination of larger rainfall intensity and more impervious areas. As a consequence several properties have been affected by basement flooding during heavy rainfall events when the network was severely overburdened. This thesis aims to study the management of storm water excess in order to evaluate how drainage networks could be improved to avoid basement flooding events. The investigation is focused on the identification of critical parts of the drainage system under risk of flooding, along with the feasibility analysis of different potential solutions to handle the excess of storm water.

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

The simulations confirm the insufficient capacity of the system to handle heavy rainfall with one third of the nodes flooded during a 10-year return period rain. With 67% of concrete pipes running two third of pipe during 5-year return period rain and a 32.6% of nodes flooded during a 10-year return period rain can be affirmed that the drainage system situation is critical. The total runoff from whole sub-catchments by SWMM is  $3.47\text{m}^3/\text{sec}$ , whereas by rational method is  $3.14\text{m}^3/\text{sec}$ . The total simulation accuracy of the runoff and network systems as assessed by statistical methods, were  $R_{NS} = 0.622$ ,  $RE = 0.098$ , and  $R^2 = 0.935$ . Therefore, the effectiveness of different sustainable drainage systems (grass, ditches, detention ponds and gravel tanks) and enlargement of pipe diameters were evaluated for this case study. Continuous rains have shown to be the worst case rainfall scenario in this drainage system.

Generally it can be concluded that road surface drainage of the study area found to be inadequate due insufficient road profile, insufficient drainage structures provision, improper maintenance and lack of proper interconnection between the road and drainage infra-structures thereby resulting flooding problems in the area.

**ABBREVIATIONS**

AACRA-----	Addis Ababa City Road Authority
AASHTO-----	American Association of State Highway and Transportation Officials
AAU-----	Addis Ababa University
A2-----	Rainfall Region Classification of Ethiopian Road Authority
CRS-----	Circular, Rectangular, O-Shaped
DHI-----	Danish Hydrology Institute
EEA-----	European Environmental Agency
EGL-----	Energy Grade Line
EP-----	Environmental protection
EPA-----	Environmental Protection Agency
ERA-----	Ethiopian Road Authority
FFG-----	Flash Flood Guidance
FHWA-----	Federal Highway administration
GDP-----	Gross Domestic Product
GIS-----	Geographical Information System
HGL-----	Hydraulic Grade Line
RTC-----	Real Time Control
SWMM-----	Storm Water Management Model
UH-----	Unit Hydrograph
WMO-----	World Meteorological Organization

## CHAPTER ONE

### 1 INTRODUCTION

#### 1.1 General Background

Among all natural disasters that affect humanity, floods are the most common phenomenon that causes severe damage (EEA, 2001). This kind of events threatens infrastructures, properties and human lives due to an excess of runoff flowing in the surface. The term flood refers to a flow of water over areas which are habitually dry. Flood hazard is generally assessed through the evaluation of its impact parameters, such as water depth and velocity, and its associated probability of occurrence. Floods interfere with efficient drainage and economic use of lands for agricultural or industrial purposes. Floods also damages drainage channel, bridges, sewer outfalls and other structures. Human influence is an important factor that many artificial changes in the river system may induce morphological changes and subsequent rising of the water or bed level.

Flooding in urban areas is not just related to heavy rainfall and extreme climatic events; it is also related to changes in the built-up areas themselves. Urbanization restricts where floodwaters can go by covering large parts of the ground with roofs, roads and pavements, thus obstructing natural channels. Large-scale urbanization and population increases have led to large numbers of people, especially the poor, settling and living in floodplains in and around urban areas (Daniel p. Loucks and Eelco van Beek, 2005).

As the topography of the country is rather rugged with distinctly defined watercourses, large scale flooding is limited to the lowland flat parts of the country. However, intense rainfall in the highlands causes flooding of settlements in a number of river basins and torrential floods are also produced in Addis Ababa and in another main city. Several small streams originating in the mountain range at the foot of which the city lies traverse metropolitan Addis Ababa. Torrential rains, common during the rainy season, cause sudden rise in the flow of these streams, which bring about flood damages to settlements along their banks. A similar situation affects the town of Dire Dawa. (WMO/GWP Associated Programme on Flood Management ETHIOPIA: INTEGRATED FLOOD MANAGEMENT Kefyalew Achamyeh)

As regards urban and infrastructure flooding, the city administration of Addis Ababa has prepared a flood protection scheme including structural and non-structural activities to be

implemented over a 15-year period. The structural intervention covers construction of retaining walls and dykes and improvement of river channels. The non-structural plans include reforestation and proper zoning concerning settlements close to the streams and adequate early warning. Similar plans have been developed for the city of Dire Dawa. (WMO/GWP Associated Programme on Flood Management ETHIOPIA: INTEGRATED FLOOD MANAGEMENT Kefyalew Achamyeleh)

With urbanization, impermeability increases because of the increase in impervious surfaces. This in turn changes the drainage pattern, increases overland flow resulting in flooding and related environmental problems. The impact of this is severe on spatial structures like road. This is because, flooding and its related environmental problems like sheet and gully erosion, surface inundation tends to affect road services and its life span. (Journal of Engineering and Technology Research Vol. 3(7), pp. 217-225, July 2011, Road and urban storm water drainage network, Dagnachew Adugna Belete)

Drainage problems in urban areas include flooding, deterioration of roads, land degradation, sedimentation, blockage of drainage facilities, water logging, etc.

With urbanization, impermeability increases with the increase in impervious surfaces (i.e. residential houses, commercial buildings, paved roads, parking lots, etc.), drainage pattern changes, overland flow gets faster, flooding and environmental problems such as land degradation increases. It is a crucial problem facing the existing and future road infrastructure.

In spite of these problems, drainage facilities in most urban centers of the country are nearly absent or at a lower coverage. Planning and modeling rarely guide construction or provision of such facilities and management. (URBAN STORM WATER DRAINAGE MODELING MANUAL FOR ETHIOPIA)

In General, there is a need of studying the storm water management in Bole sub-city in order to modeling the required improvements in the drainage system consequently, decrease the risk of flooding.

## 1.2 Statement of the Problem

There is a problem of drainage such as overtopping and Flooding of the area during intensive rainfall and this may be due to either small drainage canal diameter, an incremental of rainfall, the increasing of pavements in the overall catchment or urbanization. Identifying the main problem with drainage system feasibility and making the system sustainable.

Due to improper management of the sewage system the waste is entering in drainage System and some of the top element of the manhole has been broken this may causes problem of aesthetic and healthy at large it may increase flood risk due to the entrance of the sewerage to the drainage system. To overcome these problems the management has to reduce the amount of sewerage entering into the drainage system, as well as to improve the current network performance and network covering the overall sub-city.

## 1.3 Objectives

General objectives of these Thesis is to identify the critical parts of the drainage system in Bole sub-city with risk of flooding and analyze the different possible solutions.

Specific Objectives are:

- To construct DDF and IDF curve for the Bole sub city.
- To identify critical parts of the drainage system with basement flooding risk in today's situation, as well as in potential future scenarios.
- To simulate the performance of the system taking into account the upcoming changes in precipitation and land use.
- To analyze the current drainage system performance regarding storm water excess management.

## 1.4 Scope and Limitation of the Thesis

This thesis includes the model of the drainage utility using rational formula and SWMM to compare the results, for calibrating the two results. These software has a limitation on the large catchment delineation, and for the large catchment it is very tedious and the aim of this thesis was to model the drainage for Bole sub city but due to the limitation of software the model only covers around 6km pipe length modeling starting from Emperial hotel to Bole international airport and fourteen sub catchments. In this fourteen sub catchments the runoff from each sub-

catchments were modeled and flood routing was done. Also the drainage networks were simulated by considering each sub catchments.

The software needs primary data with high quality to minimize the errors within the data, but to collect the primary data there was a financial limitation. The other limitation was problem of secondary data from AACRA there was no recorded data in AACRA so that, this thesis models overall drainage utility and compare the results.

### **1.5 Outline of the Thesis**

This report is divided into 5 Chapters. Chapter one contains the introduction and background, Chapter two contains Literature review information regarding storm water management and flooding, as well as guidelines in the field, some hydraulic theory, description of the current drainage system and description of different models.

Chapter three presents a general description of the study area, modeling of drainage utility using rational formula and a description of SWMM software the base tool of this study.

In Chapter four the obtained results from the simulations are presented and discussed, both the runoff and the network results. The results using rational formula and the results of the SWMM are compared and the other theoretical flood reducing methods are included in this Chapter.

Chapter five finally, presents the conclusion and recommendation about the obtained results.

## **CHAPTER TWO**

### **2 LITRATURE REVIEW**

#### **2.1 Storm Water and Drainage System**

##### **2.1.1 Storm water management**

Storm water management is an increasingly important consideration in the modeling of urban drainage systems. Storm water management practices, when properly selected, modeled, and implemented, can be utilized to mitigate the adverse hydrologic and hydraulic impacts caused by drainage facilities, thereby protecting downstream areas from increased flooding, erosion, and water quality degradation. Existing downstream conveyance constraints, particularly in cases where the roadway drainage system connects to existing drainage systems, may warrant installation of detention/recharge basins to limit the peak discharge to the capacity of the downstream system.

Poor management storm water increases total flow, flow rate, flow velocity and depth of water in downstream channels. In addition to storm water peak discharge and volume impacts, roadway construction or modification usually increases non-point source pollution primarily due to the increased impervious area. Properly modeling storm water management facilities, particularly detention/recharge basins, can also be used to mitigate non-point source pollution impacts by providing extended containment durations, thereby allowing settlement of suspended solids.

An assessment of the impacts the project will have on existing peak flows and watercourses shall be made by the designer during the initial phase. The assessment shall identify the need for storm water management and non-point source pollution control (SWM&NPSPC) facilities and potential locations for these facilities - When runoff flows along the ground, it can pick up soil contaminants such as petroleum, pesticides, or fertilizers that become discharge or non-point source pollution. Mitigating measures can include, but are not limited to, detention/recharge basins, grassed swales, channel stabilization measures, and easements.

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

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

Revegetation with native, non-invasive grasses, shrubs and possibly trees may be required to achieve compatibility with the surrounding environment. (Urban Storm Water Drainage Modeling Manual for Ethiopia)

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

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

### **2.1.2 Drainage system**

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

The negative aspect is the spill of untreated water to the watercourses (UNEP, 2004). The separated system comprises two separate pipelines for waste and storm water protecting from flooding in the basement and floors of houses in low-lying during extreme rainfalls, as well as avoiding the release of pollutants into the environment (EPA, 1999). Storm water is normally less polluted than sewage water, so that it can be led to detention basins or watercourses saving energy and cost, whereas wastewater requires a deeper treatment.

In our country the sewerage and the storm water or drainage system is isolated.

Even if the two systems are isolated there is illegal peoples those who connects there sewer line with the drainage or storm water channels. These brings the flooding of the drainage system due to the over loading of the drainage canal. The storm water is conveyed with open channel and it joins the river near to the area without any treatment.

## **2.2 An Overview on Flooding**

A wide range of natural hazards are present in Ethiopia, including drought, floods, landslides, human and animal diseases, pests, earthquakes, and urban and forest fires. Recurrent drought and floods in particular have the most severe impacts on people's lives in Ethiopia (refer to Table below). The country's vulnerability to natural disasters is due to a number of inter-linked factors. These include dependence on rain-fed agriculture, under-development of water resources, land degradation, low economic development, and weak institutions. Furthermore, with a population of 80 million people, Ethiopia is the second most populous country in sub-Saharan Africa, and has a relatively rapid annual population growth rate of 3.2%. With a GDP of us\$200 per capita, Ethiopia is also one of the world's poorest countries.

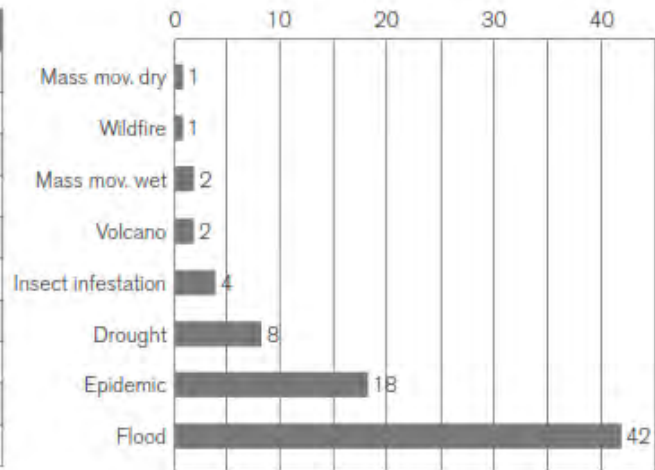
Drought is the most significant and recurrent climate-related hazard affecting the country. Ethiopia has mainly dry sub-humid, semi-arid and arid regions, all of which are prone to desertification and drought. Ethiopia has a long history of recurring drought; however, since the 1970s, the magnitude, frequency, and impacts of droughts have become more severe. Moreover, due to climate change and human-induced factors, the areas affected by drought and desertification are expanding in Ethiopia.

Flash floods and seasonal river floods are becoming increasingly common due to deforestation, land degradation, increasing climate variability, and settlement patterns.

Table 1: Top ten disasters in Ethiopia

Top 10 Disasters in Ethiopia,  
1999-2009

Hazard	People Affected	Date
Drought	12,600,000	2003
Drought	4,500,000	2008
Drought	2,600,000	2005
Flood	361,600	Oct 2006
Flood	239,586	July 2007
Flood	235,418	Apr 2005
Flood	110,000	Apr 2003
Flood	79,000	Oct 1999
Flood	45,000	Oct 1999
Flood	38,000	Aug 2006

Natural Disaster Occurrence Reported,  
1980-2008

Source: EM-DAT: The OFDA/CRED International Disaster Database, Université catholique de Louvain, Brussels, Belgium.

During the past two decades, major floods in 1999, 2003, 2005, 2006, and 2007 have caused significant loss of life and property. Large-scale flooding is limited to the lowland areas of the country; however, intense rainfall in the Highlands causes flooding of settlements in a number of river basins, particularly the Awash river Basin in the rift Valley. Annual flooding in urban areas, especially in Addis Ababa, causes property damage along streams descending from the nearby hills. Flash floods are common in most parts of the country, especially when rains occur following prolonged dry spells.

A flood can be described as an event with extreme runoff water (EEA, 2001). General and fairly common classification is based on the geographical area (rural or urban flooding) in combination with the water body which is responsible of the flood (coastal, river, flash precipitation, groundwater or sewer flooding) (Ashley et al., 2007). Enormous damages can be caused by floods, but they can be worst in the case of urban floods. Human and material damages, breaks down on the drinking water or electricity supplies are some of the undesirable consequences.

Particularly, Bole sub-city area is affected by a combination of flashflood and drainage flooding. Flash flooding is caused by heavy, short and intense rainfalls which result in high speed flowing storm water and rising flood waters.

Sewer flooding is the consequence of either the system failure or the insufficient capacity to convey high quantity of storm water during heavy rainfall events (EEA, 2001).

The risk of flooding is determined by natural factors, along with human intervention (EEA, 2001). In the Table 2 below can be seen some driving forces leading to more frequent and severe floods.

Table 2: Factors contributing to flood (WMO, 2008)

<b>Meteorological Factors</b>	<b>Hydrological Factors</b>	<b>Human factors aggravating natural flood hazards</b>
Rainfall	Soil moisture level	Land-use changes (e.g. surface sealing due to urbanization, deforestation)
Cyclonic storms	Groundwater level prior to storm	Occupation of the flood plain obstructing flows
Small-scale storms	Natural surface infiltration rate	Inefficiency or non-maintenance of infrastructure
Temperature	Presence of impervious cover	Too efficient drainage of upstream areas increases flood peaks
Snowfall and snowmelt	Chanel cross-sectional shape and roughness	Climate change affects magnitude and frequency of precipitations and floods
	Presence or absence of over bank flow, channel network	Urban microclimate may enforce precipitation events
	Synchronization of runoffs from various parts of watershed	
	High tide impending drainage	

In the following subchapters the drainage system with regards to storm water conveyance and the surcharge on it will be further described, along with the effects of climate change and the urbanization process in the runoff generation.

### 2.2.1 Effects of climate change in flooding events

The climate in Ethiopia is classified as temperate moist, characterized by abundant precipitation during the summer season annually. However, the climate has changed over the last decades as a result of the global warming, especially in the recent years, leading to a rise of both temperatures and precipitation (CLUVA climate change and vulnerability of African cities Addis Ababa).

Addis Ababa has a pronounced rainfall peak during the boreal summer (July to August) and exhibits a rainfall minimum during the boreal winter (December to February). The city has a temperate climate due to its high-altitude location in the subtropics. Average monthly temperatures vary between 10 and 20 °C, and are the lowest during summer, due to the prevailing cloud cover experienced during this season.

The results of climate simulations for the period 2010-2050 suggest that:

- ✓ Mean annual rainfall could increase by 35% - 50% with respect to the current situation;
- ✓ An increase of at least 1.5 °C in mean annual temperature is expected.

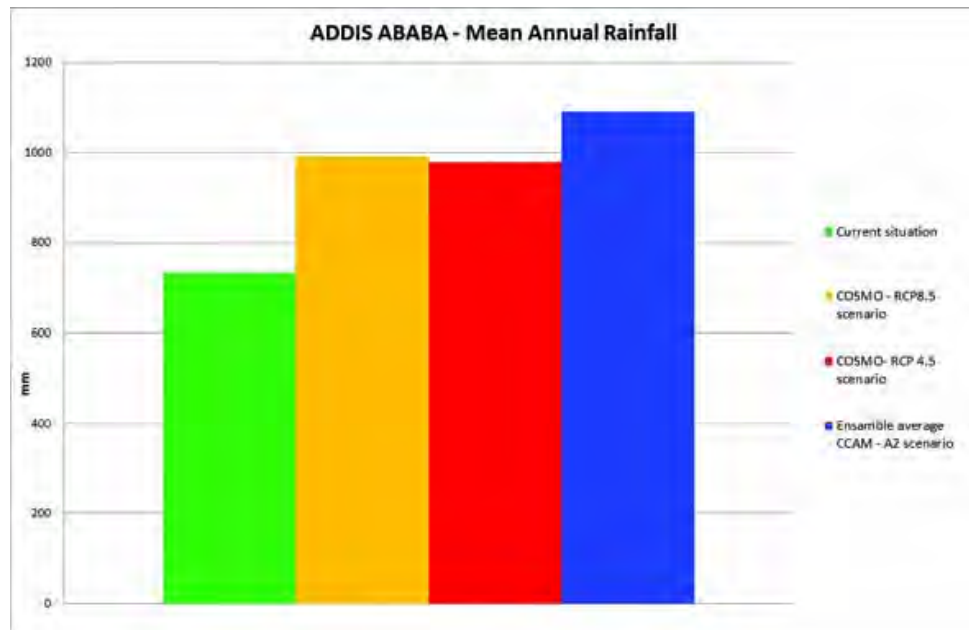


Figure 1: Addis Ababa –mean annual rainfall

Climate change is induced by greenhouse gases, in particular CO emissions. Even if the concentration of greenhouse gases decrease in the atmosphere or, at least remains the same generation rate, climate change is likely to continue affecting the global warming for many decades (Sweden Commission on Climate and Vulnerability, 2007).

Moreover, precipitation is the major parameter within climate change causing the greatest flood impacts (Semadeni-Davies et al., 2008). The annual precipitation in Europe is estimated to increase in a 1% to 2% per decade (IPCC, 2001). The analysis of Extreme Rainfall Events, based on climate projections data until 2050, suggests that the intensity of Extreme Rainfall Events is expected to decrease, although an increase of the frequency of Extreme Rainfall Events is envisioned. However, assuming a business as usual scenario of population growth, runoff of rain waters is expected to increase due to the decrease of the permeability of the urban environment. (CLUVA climate change and vulnerability of African cities Addis Ababa)

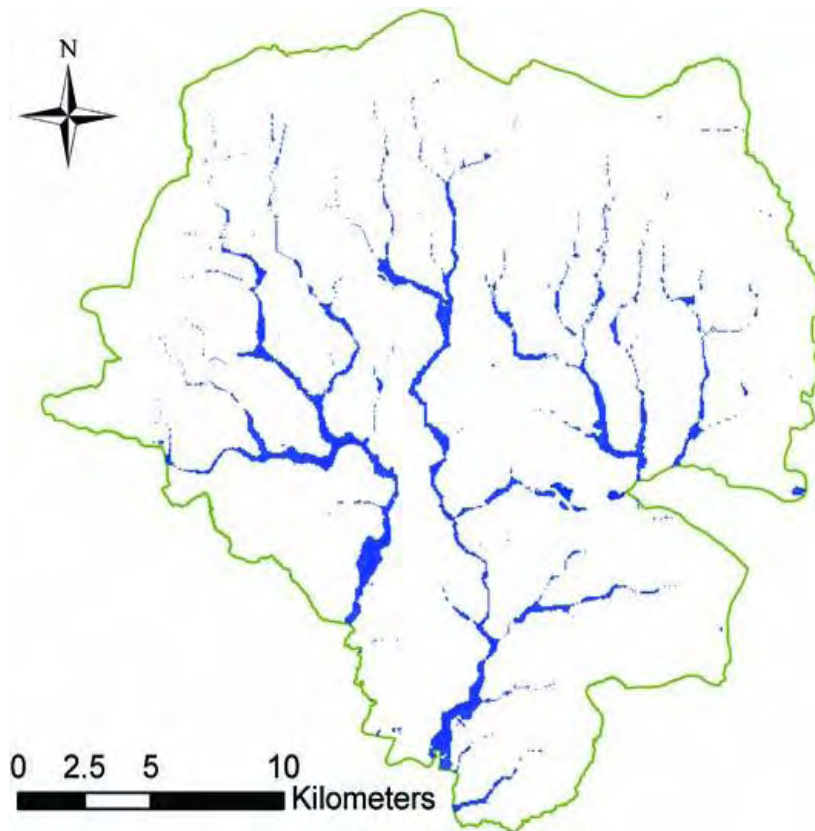


Figure 2: Map of topographic wetness index for Addis Ababa (areas more susceptible to flooding phenomena based on a geomorphologic criterion.)

The analysis of the monthly average rainfall clearly shows that the current condition is extremely dry. Analysis based on climate projections reveals that this condition is expected to continue in the next 40 years with an increase of the duration of dry periods. (CLUVA climate change and vulnerability of African cities Addis Ababa, Urban Characterization and Green Structure Mapping) The 67% of the population in the flood prone residential area lives in mud and wood constructions that are particularly vulnerable to flood action.

The “mud and wood” residential type constitutes around 51% of the flood prone residential buildings.

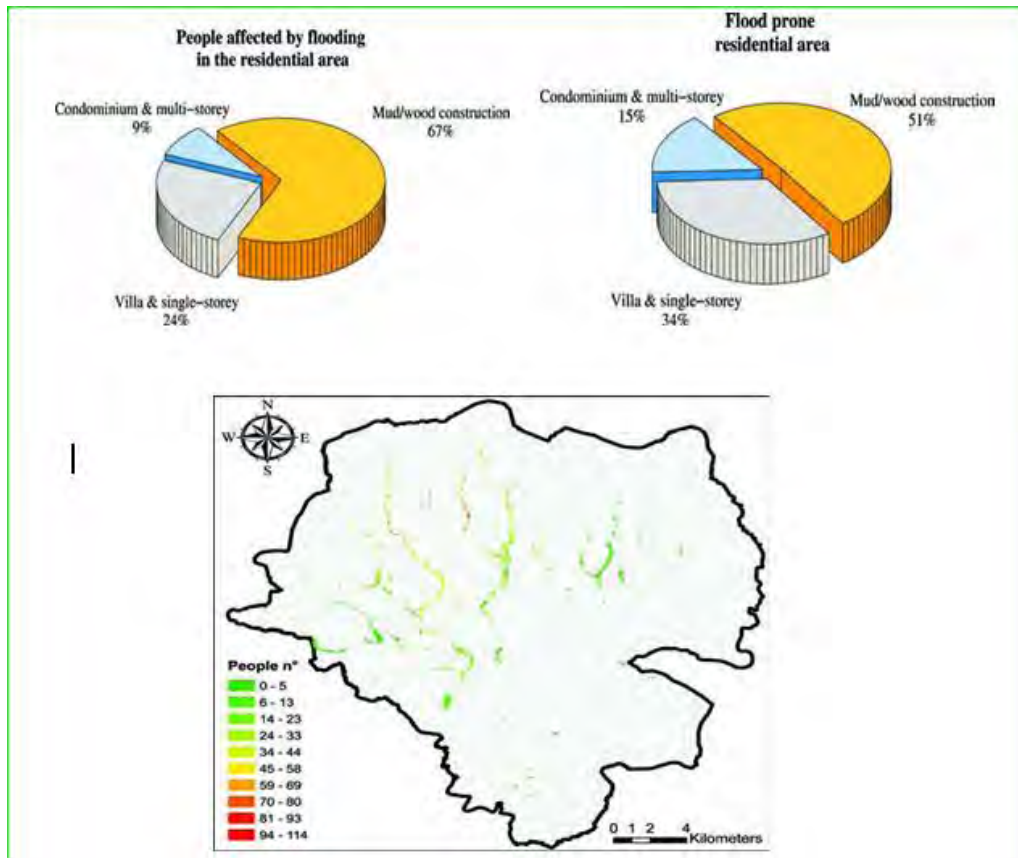


Figure 3: The urban residential hot spots for flooding in Addis Ababa.

Source: (CLUVA climate change and vulnerability of African cities Addis Ababa)

Losses of green structures in the urban core and peri-urban areas in Addis Ababa are evident; in particular a dramatic reduction in agricultural land is estimated.

A business as usual scenario (i.e. continued low density development including flood prone areas) modelled to 2025 suggests that the spatial extent of the urban zone could increase by around half again from 2011 and around a third of the city's agricultural land and almost a quarter of its other vegetated areas would be lost.

It would be also expected that a further 31% of the riverine corridor of 2011 would be lost, thus exacerbating problems of flooding, e.g. through the introduction of impervious cover and as a result of the increased exposure of the population within flood prone areas. Both climate change and urban development are expected to change surface temperatures.

However, land surface cover differences areas associated with land surface temperature ranges of more than 25°C, where as the range for climate change projections is less than 1.5°C. (CLUVA climate change and vulnerability of African cities Addis Ababa)

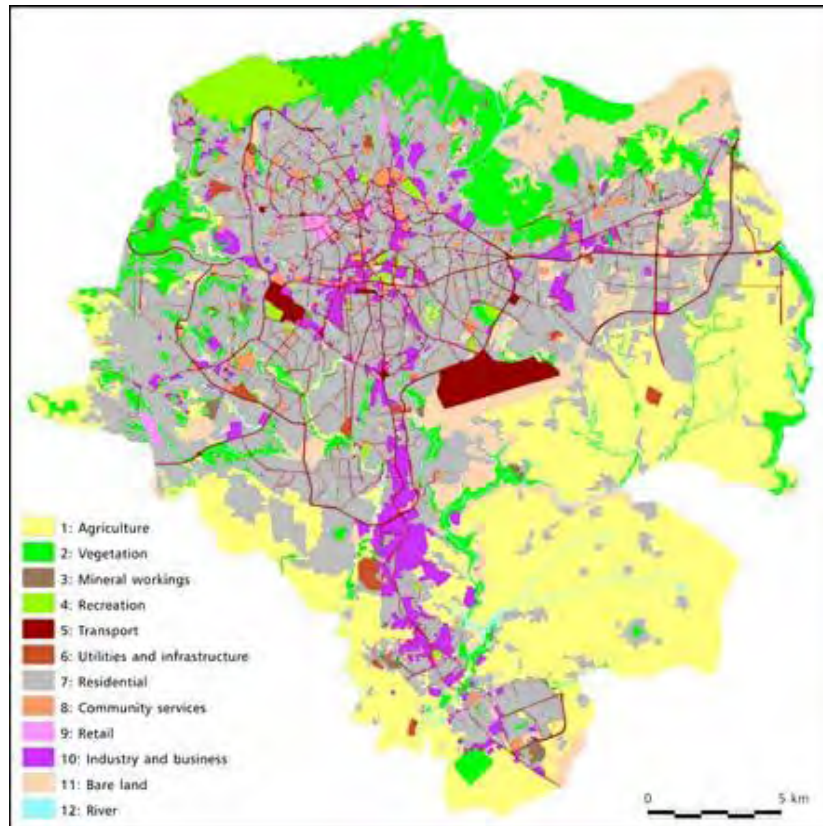


Figure 4: Urban morphology types in Addis Ababa.

Source: (CLUVA climate change and vulnerability of African cities Addis Ababa)

Moreover, the runoff will increase as a direct consequent of the precipitation modifications. The 100 year flow (probability of flow reaching this flow value once every 100 years) will suffer and important increment. Drainage system will be heavily burdened due to the inflow increments across the weak seasons: autumn, winter and spring when the ground is saturated and the evaporation rates are low. In addition, enlargements of the base flows have knock-on effects such as sediment transport (Semadeni-Davies et al., 2008).

### 2.2.2 Effects of urbanization in flooding events

The evolution of the land use is much related with urban development and the increment of floods derived from it. In the undeveloped areas the water coming from precipitation infiltrate in the soil filling the holes between particles until the storage capacity (saturation) is fulfill. After that, the runoff generation starts on the surface.

However, within urbanized areas the paved and other impervious surfaces hinders the capacity of the soil to absorb water. Consequently, the velocity of the runoff is increased leading to sharp peak discharges and greater amount of water in the surface (EPA, 2003). Figure 4 show the influence of urbanization in the runoff generation. As can be seen, the water cycle balance is modified since the groundwater table level decreases and the runoff is raised instead.

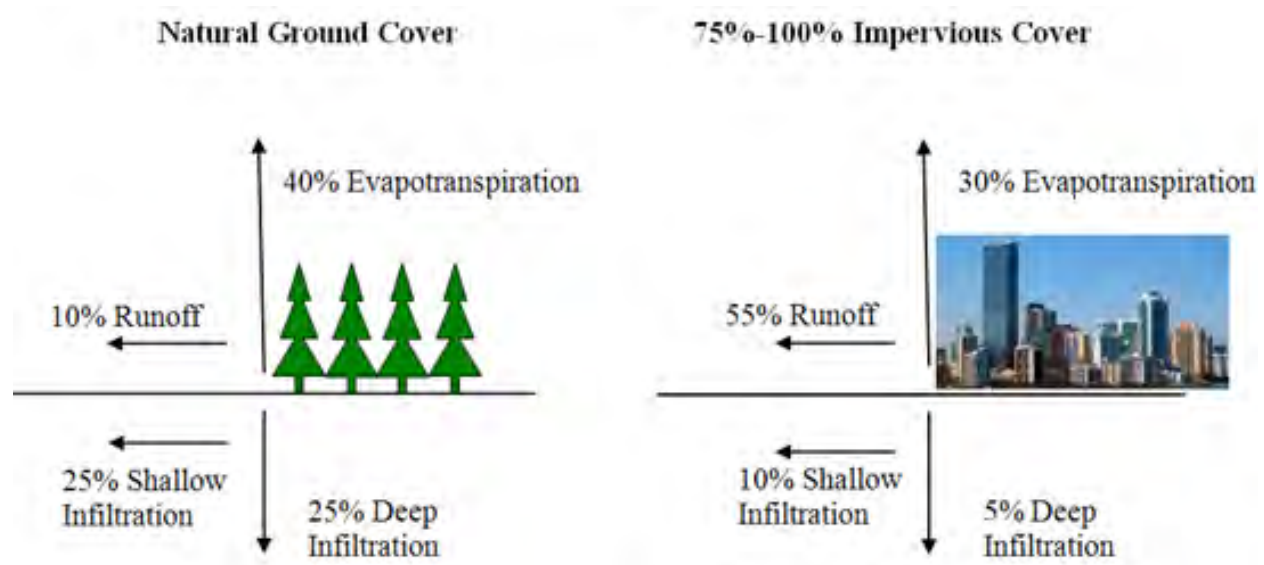


Figure 5: Influence of urbanization in runoff generation

(Source: Modified from EPA, 2003).

But not only the quantity of water is modified, water quality is also affected. Organic and nutrient pollutant concentration in the water increases considerably if the water flows above the surface. As a result, the increasing pollutant concentration decreases the possible water uses (Choi, 2004).

### 2.2.3 Existing urban drainage system

Addis Ababa the capital of Ethiopia is one of the major cities in Africa where the main head quarter of the African Union and other international organizations are found.

Though the very intensive political, economic and social activities taking place in Addis Ababa, the infrastructural development specially on the area of surface and sub surface drainage system is not compatible with the ever-growing activities of the city. The existing ill effected drainage system doesn't meet the city's storm drainage requirement. This is mainly due to the reason that the existing & still the recently constructed urban storm drainage systems has laid with little or with no detailed hydrological and hydraulic investigations. Apart from this there is no well organized regular or routine clearing and maintenance operation plugged along with the existing storm drain system. These conditions necessitate the idea of making detailed hydrological and hydraulic studies on the area of storm drainage management affaires in relation to the road development program of the city.

New Stadium Bridge approach existing road found in Bole sub city among the major storm water drainage problem areas where the situation is highly magnified during flood times of the year. The approach road to the New Stadium Bridge which in turn connected to feeder roads consists of previously constructed underground drains, gutters and inlet storm drainage structures. Some are not functioning today (underground drain pipe with all its auxiliary structures like inlets and access holes coming from both side and join the approach road storm drain system at junction near the bridge, some running from Gerji direction at both sides of the existing road to the bridge approach pavement partially functioning but the pipes are actually in bad condition, the street inlets are clogged with garbage and solid waste for a number of years and others are under sized where most sections are broken due to over capacity operation in its life time.

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

An important cause of flooding is the insufficient capacity of current drainage systems. The combination of the higher rates of storm water and increasing illegal connection of sewerage system conveyed result in a high-risk of overloading the capacity of the system (EEA, 2001).

Consequently, overburden networks are exposed to the emergence of surcharge that may cause flooding.



Figure 6: Current flood at the new stadium bridge the flood stops transportation



Figure 7: Photo showing under sized inlet in front of Bole International Airport

### 2.3 Guidelines

The main current legislation regarding the assessment and management of flood risk in Europe is the Directive 2007/60/CE (European Parliament, 2007), where the following statement is included: “Floods have the potential to cause fatalities, displacement of people and damage to the environment, to severely compromise economic development and to undermine the economic activities of the Community”.

This directive is mandatory for every EU country, and should be able to assess and reduce flood risk in order to protect human health, environment, properties and economic activities. To fulfill with succeed this document three main measures must be considered: complete a preliminary flood risk assessment, flood hazard maps along with flood risk maps and flood risk management plan. This directive affects every kind of floods, from those referred to river banks and coastal to urban floods caused by surface runoff or the overload of the drainage system. These steps will be followed in this thesis, starting with the situation of floods in the area, subsequently the determination of hazard and risk points within the network and finally the measure to be adopted in order to manage them.

Several measures can be adopted, but most of them will consider the Directive 2000/60/CE (European Parliament, 2000). This document has the target of maintain and improve the aquatic environment. Storm water, in its path until the inlet points, drag an important load of pollutants that either finish into the drainage system and finally toward the river obviously.

The following guidelines should be used to select the hydrology method for computing the modeling peak flow:

Table 3: Size of drainage area and hydrologic method

<b>Size of Drainage Area</b>	<b>Hydrologic Method</b>
1 - 50 Hectares	Rational Formula
50 - 1300 Hectares	NRCS* TR-55 / SCS Method and Other Unit Hydrograph Methods
Greater than 1300 Hectares	NRCS TR-20 or HEC-1 Method

\* U.S. Natural Resources Conservation Service (NRCS), formerly the U.S. Soil Conservation Service (SCS)

Alternatively, Modified Rational Method could be used for size of drainage area greater than 50 hectares.

The peak flow from a drainage basin is a function of the basin's physiographic properties such as size, shape, slope, soil type, land use, as well as climatologically factors such as selected rainfall intensities. The methods presented should give acceptable predictions.

## 2.4 Theory: Hydraulic Background

The engineering calculations used in the modeling and evaluation of drainage systems are, to a large extent, applications of the fundamental physical laws of conservation of mass, energy and momentum (Hager, 2010).

Flow in drainage systems can be classified in mainly two types: pipe flow running under pressure and open channel flow, characterized by water conveyed by gravity with a free surface at atmospheric pressure. However, when it comes to sewer systems the flow is a combination of both of them, resulting in a part-full pipe flow. Water runs by gravity with a free surface and only fills the pipe area when the capacity of the pipe is exceeded (Butler & Davies, 2000).

The most basic principle of the hydraulics of pipelines is the conservation of mass (Larock, Jeppson & Watters, 1999). This principle is expressed by means of the continuity equation for steady incompressible flows in a conduit, see Equation below.

$$Q=A_1.V_1=A_2.V_2$$

Where: Q=discharge (m<sup>3</sup>/s); A=cross sectional area (m<sup>2</sup>); V=mean cross-sectional velocity

The linear momentum principle is governed by the impulse-momentum equation. Equation (2) is the momentum equation for steady incompressible flow in a pipe (Larock, Jeppson & Watters, 1999).

$$\sum F=\rho \cdot Q \cdot v$$

Where F= External forces acting in the pipe surface (N)

$\rho$ =density of water (kg/m<sup>3</sup>); Q=flow (m<sup>3</sup>/s); v=velocity (m/s)

The flow energy, in terms of head depth (m), consists on the sum of three terms (pressure head, velocity head and datum head), see Equation below. This equation is also referred as to Bernoulli's Equation (Butler & Davies, 2000).

$$H=P/\rho g + v^2/2g + z$$

Where H=hydraulic head (m), P=Pressure (Pa),  $\rho$ =density of water (kg/m<sup>3</sup>), g= gravity (m/s<sup>2</sup>), v= velocity (m/s), z= potential head (m)

Water always flows from regions with higher energy level to regions with lower energy level (Butler & Davies, 2000). The energy grade line (EGL or EL) represents the total amount of energy available in the system, i.e. it is drawn a vertical distance from the datum equal to the sum of the potential head, velocity head and pressure head.

Therefore, to calculate the energy grade line it is necessary to determine the losses through the system. The hydraulic grade line (HGL) represents the water level in an open channel, whereas it is the elevation to which water would rise in a pressurized conduit. It is calculated by subtracting the velocity head term from the energy grade line (Brown, Sten & Warner, 2009).

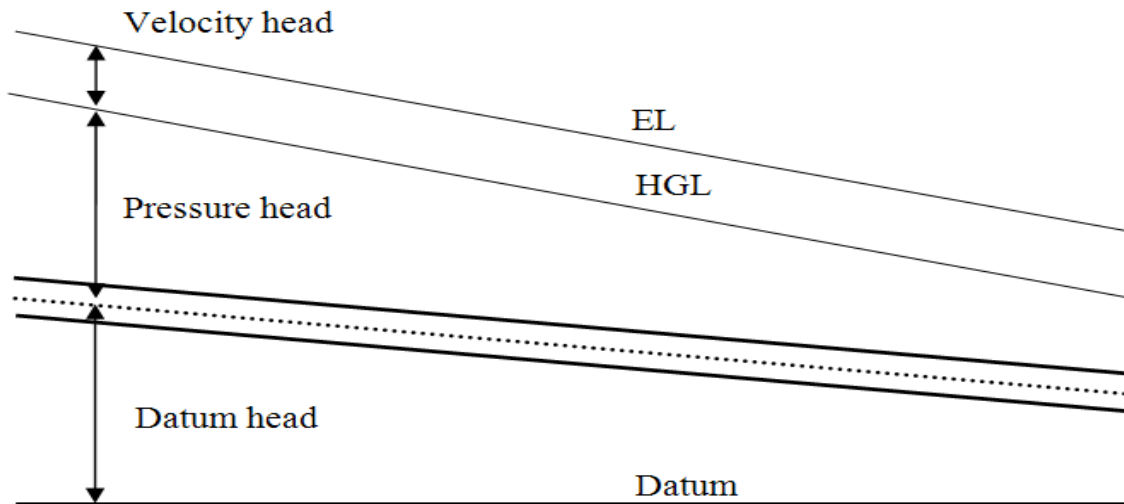


Figure 8: Hydraulic and energy grade lines in pipe flow (modified from Butler & Davies, 2000).

The energy losses in a pipe consist of friction losses and local losses. Friction losses involve forces between the liquid and the solid boundary, while local losses account for disruptions to the flow at local features. The friction losses are represented by the Darcy-Weisbach Equation, valid for both laminar and turbulent flow (Durrans & Haestad Methods, 2003).

$$h_f = \frac{\lambda L}{D} * \frac{v^2}{2g}$$

Where  $h_f$  = head loss due to friction (m),  $\lambda$  = friction factor,  $L$  = pipe length (m),  $D$  = pipe diameter

When it comes to urban flooding, two important hydraulic concepts arise: surcharge and backflow. A drainage system is surcharged when its capacity is exceeded, i.e. it receives greater volume of water than the system can convey. As a result, the water level rises upstream due to the network overloading (Butler & Davies, 2000). If the energy line reaches higher points downstream than upstream, water may change the regular flow direction leading to backflow.

Flood frequency Analysis, the determination of flood flows at different recurrence interval, is a common problem in hydrology. The standard procedure to determine probabilities of flood flows consists of fitting the observed stream flow record to specific probability distributions.

However, this procedure only works for basins:

- ✓ That have „long enough „stream flow records to warrant statistical analysis;
- ✓ Where flood flows are not appreciably altered by reservoir regulation, channel improvements (levees) or land use change.

Continuous hydrologic simulation is a valuable tool to determine flood frequencies in ungauged watershed and in gauged watersheds that have short stream flow records or are heavily regulated. Since hydrologic simulation models the rainfall-runoff relationship in the basin, it can also be used to check the validity of the probabilistic distribution selected for gauged unregulated watersheds with long stream flow records. Meteorological data in the study watersheds extend from 15 to 30 years. Stream flow records, if available at all, are often much shorter and most of the watersheds are ungauged. Continuous hydrologic simulations can use the observed meteorological data available to extend the existing stream flow record from a few years to 20 or 25 years.

The extended record can then be fitted to a probabilistic distribution. Hydrologic simulation can also be used to determine flood frequencies in ungauged stream. In this case, the model is calibrated to a near-by, hydrological similar gauged stream. The model parameters are then adjusted to reflect the physical changes between the calibrated watershed and the ungauged watershed. Finally, all the available observed meteorological data is used to create a long stream flow record for the ungauged stream, which can be fitted to a statistical distribution. This method produces much better results than alternative simplified approaches such as comparison to similar watersheds, or indirect approaches that equate runoff frequency to precipitation frequency (such as unit hydrograph of rational formula). Land use changes can have a significant effect on flood flow frequencies. Historic stream flow records may be non-stationary for basins in which widespread urbanization is taking place. Hydrologic simulation uses historic flow records to calibrate to the historic conditions and it then incorporates the effects of future urbanization and land management.

In addition to creating or extending stream flow records, hydrologic simulation can be used to study the validity of assumed probabilistic distribution for peak flows.

The calculated run off intensity above certain return periods greatly exceeds the precipitation for the same return period. Since continuous hydraulic simulation maintains a continuous accounting of soil moisture, it provides a unique tool to analyze the complex relationship between frequencies of precipitation, soil moisture and runoff. The basic requirement for any watershed hydrology model is its capability to estimate surface runoff adequately because it influences the development of hydraulic structures. An accurate simulation of yield from a watershed is quite essential, as it is often required to determine the magnitude of flood from the watershed. During the past two decades there has been a dramatic increase in the development and use of hydrological models to evaluate the complex environmental processes and to assess non-point source of water resources.

However, in Ethiopia little use has been made of hydrological models to develop management plan for small agricultural watersheds using a systematic modeling approach. It is obvious that rainfall data for several years are required for developing the long term management plan of a watershed. Many process-based hydrological models, including SWMM, have the capability to generate rainfall and, thereafter, surface runoff, on daily, monthly and annual bases. An adequate procedure to calibrate and validate the SWMM modeling is an important issue. A model should be tested adequately before using it to develop effective watershed management plans, specifically if generated rainfall is the basic input.

Weather data are frequently needed to evaluate the long-term effects of proposed anthropogenic hydrological changes. These evaluations are often undertaken using deterministic mathematical models of hydrological process. In addressing hydrological response to weather inputs it is seldom sufficient to examine only the response to observed weather events. Use of observed sequences gives a solution based on only one realization of the weather process. What would be the result if another series with the same properties as the observed series were used? To answer these questions, it is desirable to generate synthetic sequences of weather data based on the stochastic structure of the metrological process with an appropriate model developed for that purpose.

The weather variable needed for most of the hydrological models include precipitation, maximum and minimum temperatures, solar radiation and other (Knisel, 1980). These variables are usually recorded daily, and most deterministic models required daily values.

Rainfall predication plays an important role in modeling the water harvesting structures, erosion control measures and for developing the management plan for the critical erosion-prone areas of a watershed. Several research workers across the world have developed rainfall predication models to solve the foresaid problems (Buishand, 1978; Richardson, 1981; Georgica and Bars, 1984).

In developing countries, like Ethiopia rainfall generally is measured only in a few watersheds. In Ethiopia most of the watersheds selected for development and management purpose under various projects have only one rain gauge for the entire watershed. Sometimes it is almost impossible to gauge each and every sub watershed of all watersheds of the country. The ability to generate rainfall for predication of surface runoff and thereafter developing the management plan is essential keeping the above mentioned importance of the generated rainfall in estimation of runoff in mind. This study was undertaken to estimate runoff of a watershed and Modeling and analyses of Urban Flooding in Addis Ababa bole Sub city flood using Arch GIS and Global Mapper.

#### Management Plan:

The general goal of the management plan is to reduce the amount of water entering into the system, as well as to improve the current network performance. Environmental-friendly practices are the major solutions to offset the impacts caused by urbanization by returning to nature developed spaces (Durrans & Haestad Methods, 2003). Therefore, the study was firstly focused on the implementation of sustainable drainage systems to avoid the overloading of the network. Nevertheless, other approaches based on conventional infrastructural modifications are also considered due to the critical situation of this area.

#### Management plan considerations:

Several ideas should be kept in mind in order to modeling suitable solutions to avoid flooding, as well as to adjust them to the specific area. Some of these factors are presented below (Durrans & Haestad Methods, 2003):

- Ground conditions: Clay and bedrock are the main materials under the soil. The high imperviousness of these materials hinders the infiltration of water into depth ground.
- Spaces: the location of the solution should consider the topographic conditions (i.e. slopes and lands height, among others), as well as free surface space circumstances due to the high concentration of urbanized areas.

- Aesthetic: it is necessary to avoid negative aspects as eutrophication, the possibility of odours, mosquitoes and so forth since the actions will be located in an urban area.

## 2.4 An Overview on Models

A GIS is a system of hardware, software and procedures to facilitate the management, manipulation, analysis, modeling, representation and display of geo-referenced data to solve complex problems regarding planning and management of resources (NCGIA, US 1990). The heart of GIS is the analytical capabilities of the system. What distinguishes the GIS from other information systems are its spatial analysis functions. Although the data input is, in general, the most time consuming part, it is for data analysis that GIS is used. The analysis functions use the spatial and non-spatial attributes in the database to answer questions about the real world. Geographic analysis facilitates the study of real-world processes by developing and applying models. Such models illuminate the underlying trends in geographic data and thus make new information available.

Results of geographic analysis can be communicated with the help of maps, or both. The organization of database into map layers is not simply for reasons of organizational clarity; rather it is to provide rapid access to data elements required for geographic analysis. The objective of geographic analysis is to transform data into useful information to satisfy the requirements or objectives of decision-makers at all levels in terms of detail. An important use of the analysis is the possibility of predicting events in another location or at another point in time.

A working Geographic Information System seamlessly integrates five key components: hardware, software, data, people, and methods.



Figure 9: Components of Geographic Information System

## Data models in GIS

There are two types of data models in GIS. There are two conceptual representations used in GIS: grid (sometimes called 'raster') and vector. These are very different ways of thinking about geography, which lead to very different methods of analysis.

The grid or 'raster' representation of a map assumes that the map area is divided into cells (sometimes erroneously called pixels), normally square or at least rectangular, on a regular grid. Each cell is supposedly homogeneous, in that the map is incapable of providing information at any resolution finer than the individual cell. The map shows exactly one value (land use, elevation, political division, etc.) for each cell.

This is a very simple representation in the computer: conceptually, a 2-D matrix of values which correspond to a grid placed over the paper map. The vector representation of a map is points on a map are stored in the computer with their 'exact' (to the precision of the original map and the storage capacity of the computer) coordinates. Points can be connected to form lines (straight or described by some other parametric function) or chains; can be connected back to the starting point to enclose polygons or areas.

Global mapper is a software use to generate contours using DEM digital elevation model (30mx30m) and also to generate the watershed of the modeling area.

Arch SWAT Software is also used to model and analyze flood specially for rural areas with a large catchments. EP SWMM5 is used in urban areas flood modelling and analyses to fix the drainage size by considering the pervious and impervious areas. So this thesis is flood modeling in urban areas and the SWMM software is more comfortable, because it's calibrated and validated software in our country Cities of Jimma and so on.

SWMM was used, adapted and calibrated for the Ballona Creek Watershed, a catchment in Southern California. A geographic information system (GIS) was used to process the input data and generate the spatial distribution of precipitation. Catchment was delineated by 1579 catchments, 2648 channels and over 263 km long pipes. ARC/INFO GIS was used to compute the sub catchment and channels/pipes slopes. The slopes were then used to compute the impervious depression storage coefficients.

SWMM was used on watershed of Cascina Scala, Pavia in Italy, to produce the time varying hydrograph. Study also compared SWMM and fuzzy logic.

The study revealed for the events with total rainfall less than 25 mm, the correlation trend produced from either the SWMM or the fuzzy logic model fits well with measured data. However, for rainfall greater than 25 mm, from fuzzy logic's predictions fit better than the SWMM results.

SWMM accounts for the spatial variability of rainfall by allowing the user to define any number of Rain Gage objects along with their individual data sources, and assign any rain gage to a particular SWMM Sub catchment object (i.e., land parcel) from which runoff is computed. If multiple gages are available, this is a much better procedure than is the use of spatially averaged (e.g., Thiessen weighted) data, because averaged data tend to have short-term time variations removed (i.e., rainfall pulses are “lowered” and “spread out”). In general, if the rainfall is uniform spatially, as might be expected from cyclonic (e.g., frontal) systems, these spatial considerations are not as important. In making this judgment, the storm size and speed in relation to the total study area size must be considered.

Storm movement can significantly affect hydrographs computed at the catchment outlet (Yen and Chow, 1968; Surkan, 1974; James and Drake, 1980; James and Shtifter, 1981). When more than one gage is available to apply to the simulation, it is possible to simulate moving storms, as rainfall in one part of the basin may be different from rainfall in another part of the basin. Movement of a storm in the downstream direction increases the hydrograph peak, while movement upstream tends to level out the hydrograph (Surkan, 1974; James and Drake, 1980; James and Shtifter, 1981).

## CHAPTER THREE

### 3 MATERIAL AND METHODS

#### 3.1 General Description of the Study Area (Bole Sub-City)

Addis Ababa is located in the central highlands of Ethiopia. Geographically, it is located at  $9^{\circ} 38' 0''\text{N}$  between  $38^{\circ} 42' 0''\text{E}$ , with the elevation of 2326m above sea level at Bole International Airport, in the southern periphery, and the highest over 3700 m at Yeka Mountains, north of the city. This means, an average elevation of 2408 meters above sea level having an average minimum temperature of  $5\text{C}^{\circ}$ , maximum temperature  $27\text{C}^{\circ}$ , and average annual rainfall of 1188.27 millimeter. (CSA, 2004 and Addis Ababa City Administration, 2004)

Addis Ababa is constituted as a City Government. The city covers a total area of 530 square kilometers (53000 ha) (CSA, 2007). Out of Ethiopia's estimated urban population of nearly 12 million, about 2.738248 or 23 per cent of the total lives in Addis Ababa, of which 1304518 (47.6%) are male and 1433730 (52.4%) female. The average number of persons per household is 4.1, and the total number of households in the city is 651970. The rate of growth of the population of the city is estimated to be 2.1% (CSA, 2007). The City has gained international status by being the seat of the African Union, several international organizations and numerous embassies. Addis Ababa is located in the central part Ethiopia and belongs to the western highlands. The metropolis located in the highlands is also close to the Rift Valley. Addis Ababa is bordered by all sides by the Regional State of Oromiya Regional State.

Addis Ababa is divided into ten Sub-Cities stemming from the 2003 reforms onwards and every sub-city has its own administrative autonomy. The spatial organization shows that Lideta, Kirkos, Arada and Addis-Ketema represent the core or central area where as Akaki, Nefas-Silk-Lafto, Kolfe Keraniyo, Gulele, Yeka and Bole correspond partly to the expansion areas at their peripheries. In general the case of Akaki Kaliti sub-city, the large sub-cities have more dwellers than the small central part sub-cities.

The study area, Bole Sub-city, is one of ten districts in Addis Ababa. It is situated in the southeastern part of Addis with a total Area  $120.93\text{ km}^2$  and is accessible to all parts of the city, Addis Ababa (Bole Sub City eco-city project). It is relatively well developed and planned.

In the previous administration, the present Bole Sub-City was incorporated in Zone 3 that incorporated four Woredas. According to the recent administration the previous Wereda 17 is approximately equal in area to Bole Sub-city. It was composed of 12 Kebeles. Now, Bole Sub-City is organized into 11 Kebeles.

According to the population unit of the Finance and Economy Development Bureau, the population of Bole Sub-City is estimated to be 325,022 in year 2006/07 G.C (1999E.c)

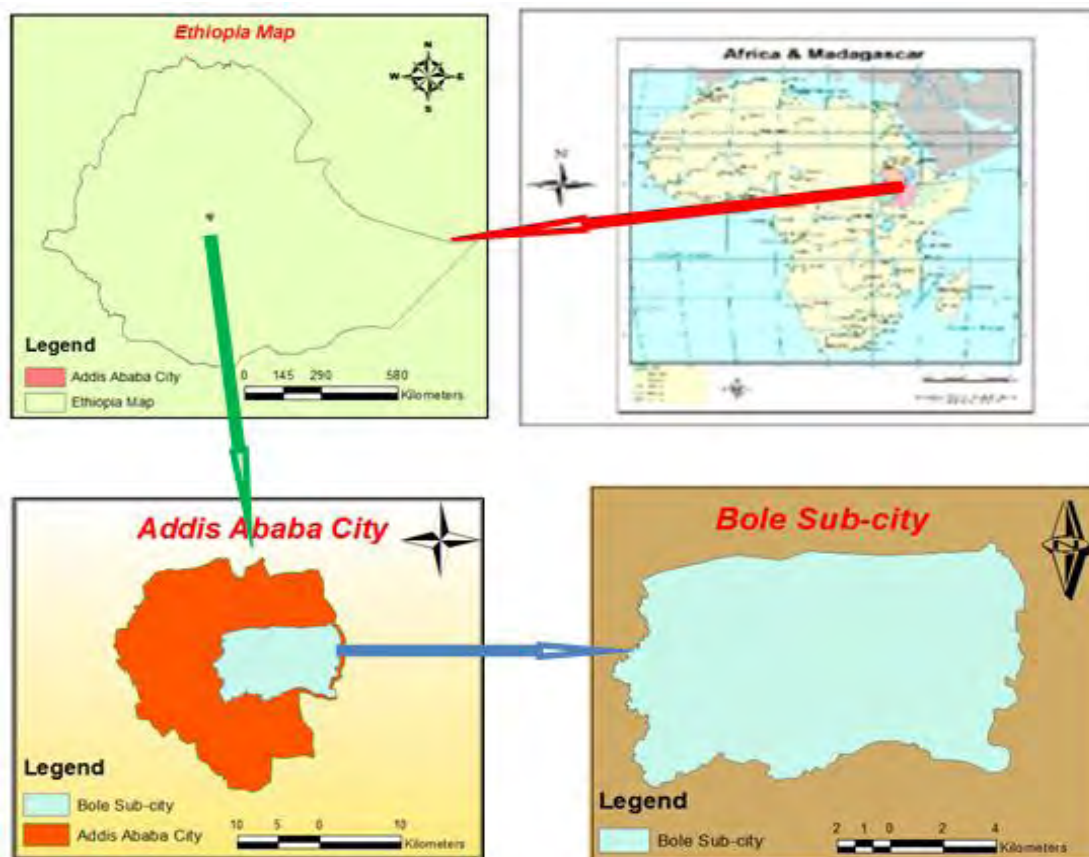


Figure 10: Location of the study area Bole sub city (Google earth map)

### 3.1.1 Hydrogeology and topography description

Bole sub city district is characterized by a highly flat relief with small hills, see Figure below. The general slope is about 0.9%, dipping from the Northern or upper catchment part towards the lower catchment. The highest elevation in Bole sub city is 3500 meters, while the lowest point in the study area is 1000 meters (DEM). The steeper areas are the south edge and the western part, which consist mainly on small valleys bordering the urbanized area.

The geology in the area is quite uniform. The bedrock consists mainly on a combination of granodiorite and granite, which both are acidic intrusive rocks. (Ethiopian geological service) However, the bedrock tends to be porphyritic, eutric nitosols, pellic vertisols or augen-bearing in the most northeaster part. The soil is basically black cotton soil with some bedrock outcrops in the highest parts. In addition, wave washed sediment as gravel or sandy till are present in small areas in the middle-southern part. The groundwater flows from north towards kality catchment due to elevation difference. Depending on the geology, the exploitation capacity varies from 200-600 l/h in bedrock to 1-5 l/s in gravel and sandy soils located under impermeable sediments. (Ethiopian geological service)

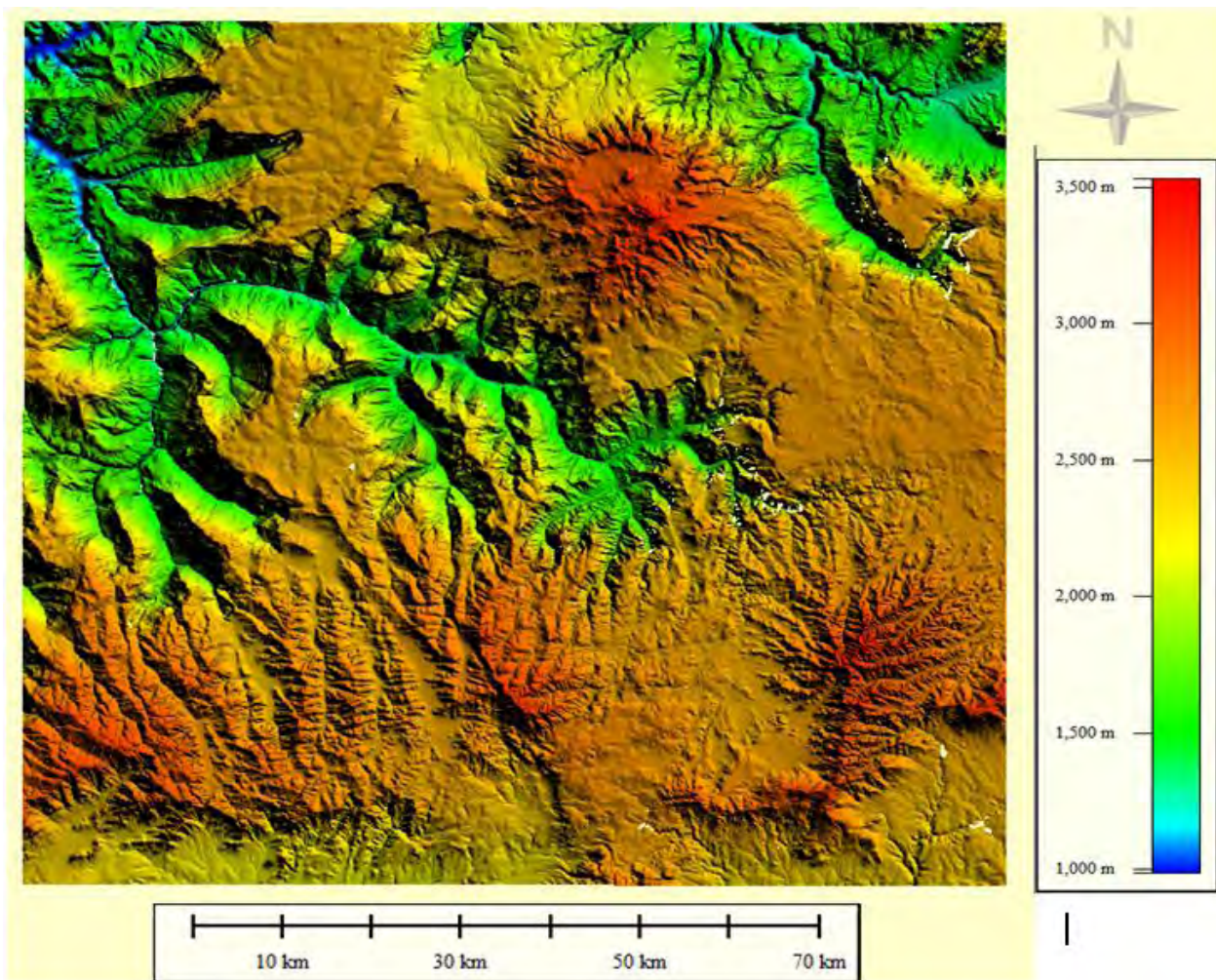


Figure 11: DEM of the study area, Bole sub city (30mx30m)

(Source: ministry of water resource)

### 3.1.2 Land use, land cover and soil type

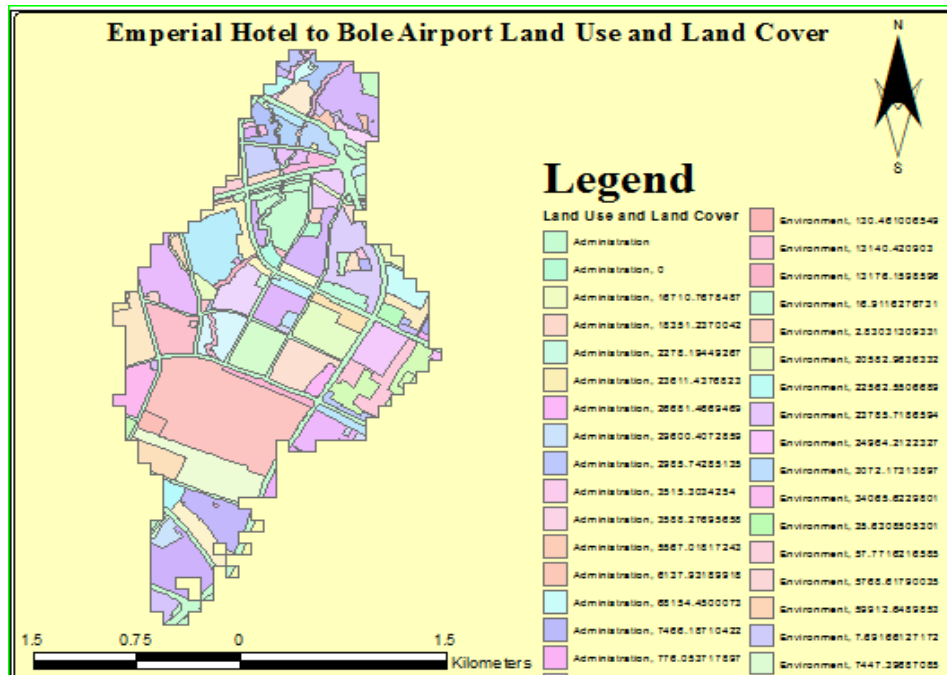


Figure 12: Land use and land cover of the modeled area

(Source: ministry of water resource Addis Ababa land use, May 2015)

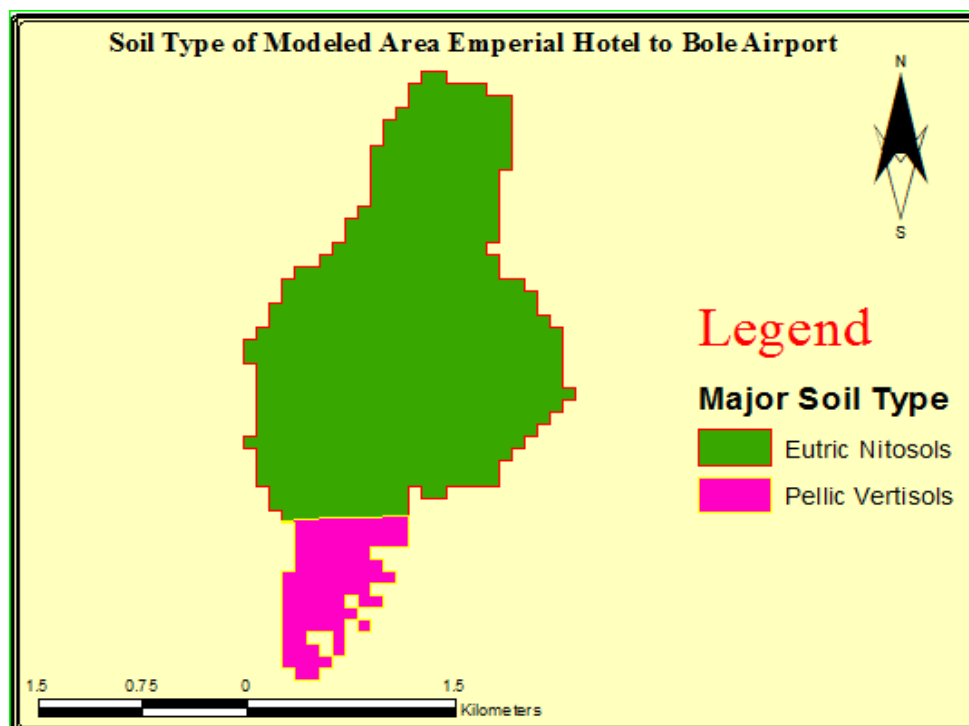


Figure 13: Soil type of the modeled area

(Source: ministry of water resource, Addis Ababa land use, May 2015)

Table 4: Soil type of the modeled area

Soil Types	Hydrologic Soil Group
Eutric Nitosols	B
Pellic Vertisols	D

The modeled area contains all types of urban area means, there is commercial, administration, and business area and also these modeled areas contain areas that are not developed.

The modeled area has only two types of soils these are eutric nitosols and Pellic Vertisols so according to typical hydrologic soils groups for Ethiopia these soils are tabulated as follows.



Figure 14: Modeled study area top view and present flooding condition

(Source: Google Earth & Camera)

## 3.2 Hydraulic and Hydrological Modeling Using Rational Method

### 3.2.1 Return period

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

The modeling storm frequency (return periods) provided by Low Volume Road Modeling Manual for different structures corresponding to the road modeling standard which is reproduced as Table 5 below adopted accordingly on drainage structure modeling return period selection on this road project. To impart a sustainable transportation system, to promote and enhance the overall socio economic development along the project road corridor, the SNNP National Regional State Rural Roads Authority planned to construct The Road Project with a modeling Standard of DC1. Hence the modeling storm frequency stated under DC1 used to compute the modeling discharge and to modeling the respective drainage crossing structures. But for those Structures which have not stated or considered in DC1 standard like short, medium, and long span Bridges we use the next Geometric Modeling Standard which is DC2.

Table 5: Design Storm Frequency (yrs) by Geometric Design Criteria

Structure type	Geometric design standard			
	DC4	DC3	DC2	DC1
Gutters and inlets	5	5	5	2
Side ditches	15	10	10	5
Ford	15	10	10	5
Drift	15	10	10	5
Culvert diameter <2m	25	20	20	10
Large culvert diameter >2m	50	25	20	10
Gabion abutment bridge	50	25	20	-
Short span bridge (<10m)	50	50	25	-
Masonry arch bridge	50	50	25	-
Medium span bridge (10 – 50 m)	100	100	50	-
Long span bridge >50m	100	100	100	-

### 3.2.2 Rainfall analysis (modelling rainfall computation)

In order to apply flood estimation models for peak discharge computation using available rainfall data, the rainfall depth-duration-frequency relationship is required. Rainfall records were obtained from the National Meteorological Services Agency located in Bole sub city.

Available rainfall data on these stations has been collected and analyzed in order to prepare the necessary depth or intensity input data for peak discharges computation. The analysis and processing is aimed at determination of appropriate intensity-duration relationship applicable for the thesis.

Estimates of maximum rainfall depths for different return periods (T) are obtained by statistical technique of frequency analysis. Extreme value type I, Gumbel and Log Pearson Type III distributions can be used for modeling storm determination of desired return periods in areas where appropriate IDF curves are not available. Thus, the analysis consists of determining maximum rainfall depths associated with T value of interest.

In the absence extreme rainfall values for periods less than 24 hours (12, 6, 3 or less than these hours) ,it is difficult to apply regression analysis to drive appropriate IDF curve for a given area, hence rainfall ratio method is used to estimate the rainfall depth to be distributed on a given duration based on a 24 hour rainfall. With this condition, the following the relationship adopted for IDF development at a given station.

$$R_t/R_{24} = (t / 24) [(b + 24)^n / (b + t)^n ]$$

where:  $R_t$ :  $R_{24}$  -Rainfall ratio,  $R_t$ : Rainfall in a given durations<sup>\*\*\*</sup> (hr)

$R_{24}$ : Rainfall in 24 hours, n: constant, b: constant, t: time (hr)

Based on studies of a large number of rainfall gauges in East Africa, the average values of b and n are found to be 0.3 and 0.9 respectively. These values have been adopted for this thesis project IDF development. (M. L. Waikar\* and Undegaonkar Namita U, January, 2015)

Extreme rainfall depth at Bole sub city station for different return periods was determined using Log Pearson Type III distributions analysis.

$$Y_T = Y_{avg} + K_T * S_y$$

Where:  $Y_T = \text{Log } X_T$  –logarithm of Rainfall depth ( $X_T$ ) at return period T years [mm]

$Y_{avg}$ = Mean value of logarithmic rainfall data (daily) [mm]

$S_y$  = Standard deviation [mm]

$K_T$  = Log Pearson Type III distribution frequency factor (taken from readily available table)

$$Y_{avg} = \frac{\sum Y}{n}$$

$$\sigma = \sqrt{\frac{\sum Y^2 - \frac{1}{n}(\sum Y)^2}{n - 1}}$$

Y = logarithm of Rainfall depth (X)

n = Total number of X (individual data)

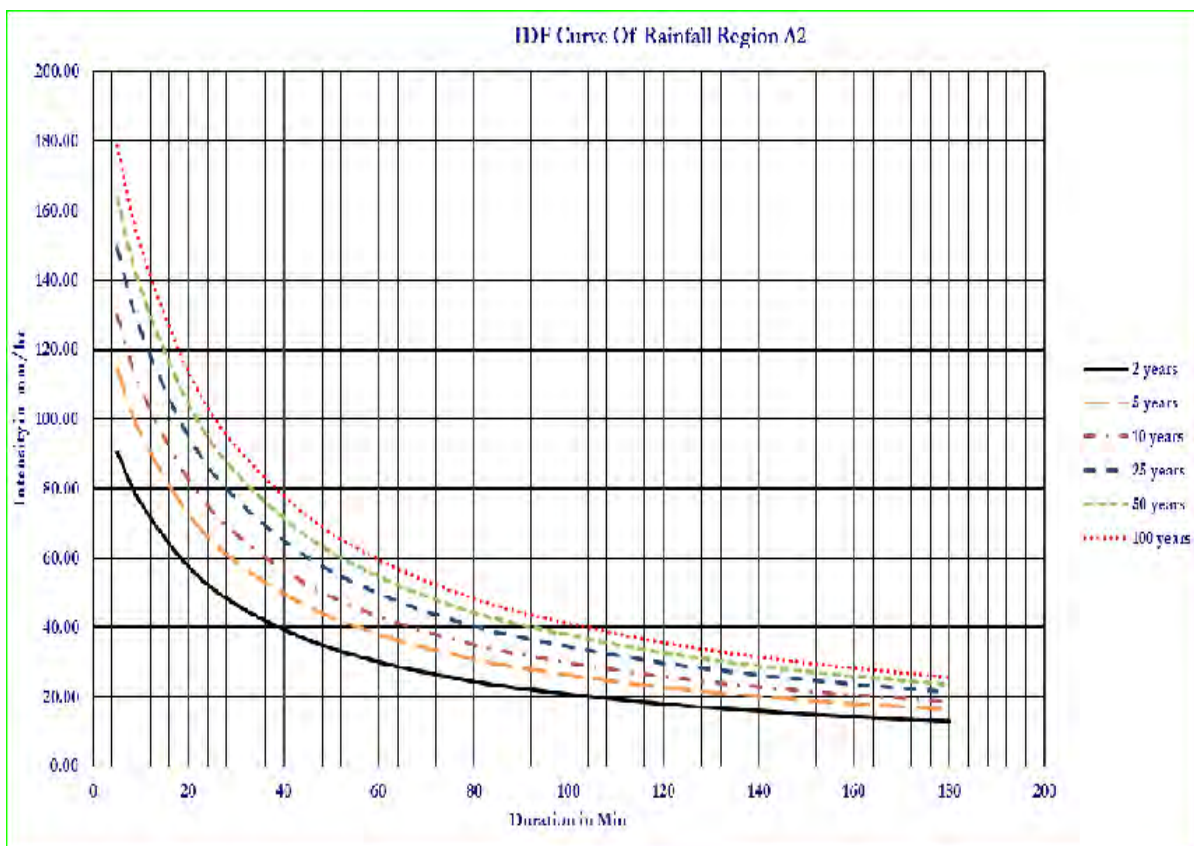


Figure 15: IDF curve of rainfall region A 2

Source: (Ethiopian Road Authority, 2013 G.C)

### 3.2.3 Rational Formula

This method is widely used for peak discharge determination from smaller catchments (less than 0.5Km<sup>2</sup> area) as recommended by Ethiopian Road Authority drainage modeling manual. Here on this Thesis, the peak discharge from fourteen (14) sub catchment areas are determined by rational method.

The idea behind the rational method is that if a rainfall of intensity (I) ,begins instantaneously and continues indefinitely, the rate of runoff will increase until the time of concentration  $t_C$ , when all of the watershed is contributing to flow at the outlet. The product of rainfall intensity I and watershed area A is the inflow rate for the system, IA, and the ratio of the peak discharge Q to this inflow rate expresses the coefficient of runoff C. The discharge is expressed in the rational formula as the following form:

$$Q = 0.00278 CIA C_f$$

Where: Q = is in m<sup>3</sup>/s,

I = is rainfall intensity in mm per hour

A = is catchment area in hectares

$C_f$  = frequency factor

C = runoff coefficient

The duration used for the determination of the modeling precipitation intensity I is the time of concentration ERA provided.

#### Runoff Coefficient

The runoff coefficient C is estimated on the basis of factors such as slopes, soil texture, land use, vegetation coverage and terrain type. Runoff coefficients are theoretically restricted to the range of 0 to 1.0. Runoff coefficient values for pervious surfaces by selected hydrologic soil groups and slope range given in the Ethiopia Road Authority drainage modeling manual 2013 reproduced as Table below is used here in this project accordingly. Equation below is used to determine weighted average runoff coefficient values for each catchment area under rational method.

$$C_{weighted} = \frac{\sum(A_i * C_i)}{A_T}$$

Where  $C_i$ - runoff coefficient for a given hydrologic soil group area

$A_i$  -area under each hydrologic soil group and  $A_T$  -total catchment area considered

Table 6: Runoff coefficient based on hydrologic soil group

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

(Source: FHWA, Urban drainage modeling manual, Circular No.22, 2nd Edition).

Table 7: Selected runoff coefficient values under different surface conditions

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

(Source: FHWA, Urban drainage modeling manual, Circular No.22, 2nd Edition).

\*Higher values are usually appropriate for steeply sloped areas and longer return periods because infiltration and other losses have a proportionally smaller effect on runoff in these cases. (Source: FHWA, Urban drainage modeling manual, Hydraulic Engineering Circular No. 22, Second Edition).

Average runoff coefficient values are taken from the above table for both main drive pavement (Cav. =0.83) and side walk brick surfaces (Cav. =0.78). However, weighted average runoff coefficient values are taken for discharge estimation from adjacent catchment areas as shown in table above.

### **Time of Concentration ( $t_c$ )**

Time of concentration is the time taken for runoff to travel from the most remote point of the catchment to the drainage structure. For specific drainage basin the time of concentration consists of sheet flow, over land flow and channel flow time.

The time of concentration calculated as the sum of the travel times within the various consecutive flow segments. The minimum time of concentration value of 15 minutes & 7 minutes are used to read rainfall intensity values from IDF curve for peak discharge estimation from catchment and pavement surfaces respectively as recommended by ERA and FHWA drainage modeling manuals. The values of intensity for  $T_{10}$  of 15 minutes and 7 minutes from Ethiopian Road Authority IDF curve is 95 and 108 respectively.

### **Sheet Flow Time**

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

$$T_t = \left[ \frac{0.091(nL)^{0.8}}{(P_2)^{0.5} S^{0.4}} \right]$$

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

$P_2$  = 2-year, 24-hour rainfall, mm

$S$  = slope of hydraulic grade line (land slope), m/m.

### Over Land Flow Time

The time required for runoff to flow over the surface from the end of sheet flow to the nearest channel inlet is primarily a function of the length of overland flow, slope of the drainage basin, and surface cover.

The overland flow velocity for the overland flow distance is estimated by plugging equation below developed for pervious surface flow velocity estimation purpose.

$$V = 4.9178(S)^{0.5}$$

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

$$T_{travel} = \frac{D}{60 \times V}$$

Where:  $T_{travel}$ -time of concentration (minutes): D –Overland flow Distance (m)

V -approximate flow velocity over the surface (m/s) based on catchment Characteristics (land use of the area)

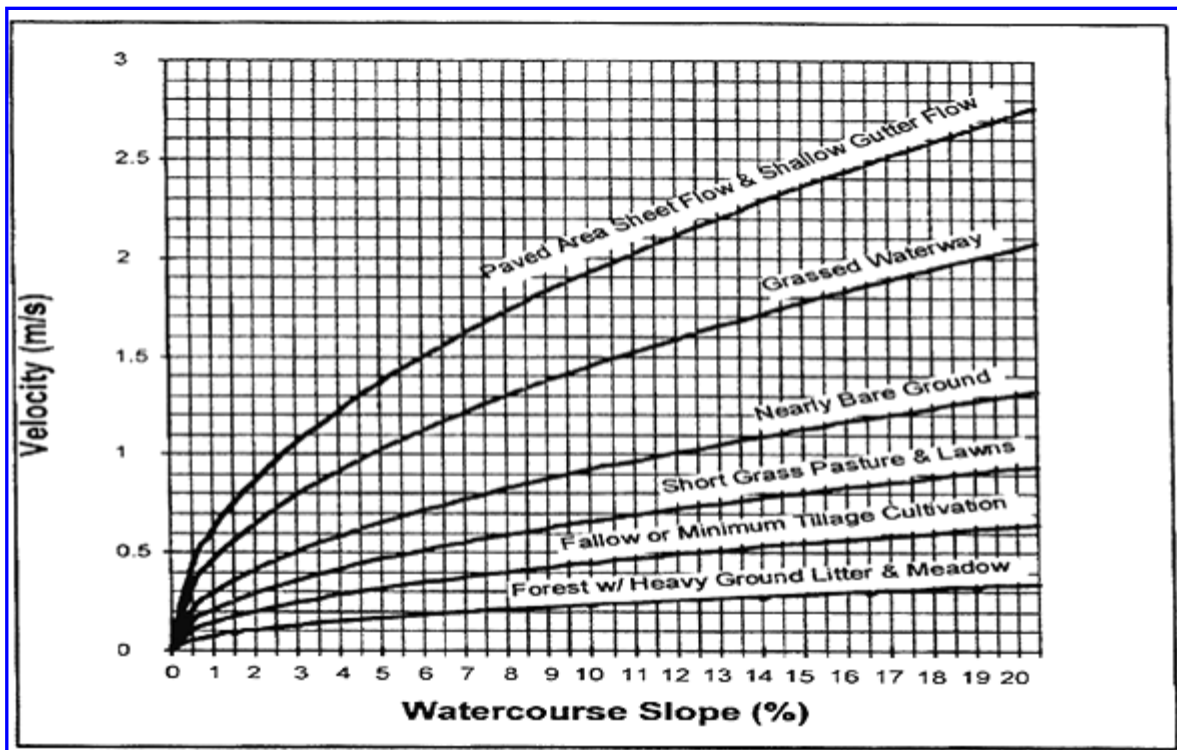


Figure 16: Velocity for estimating overland flow time

(Source: FHWA, Urban drainage modeling manual, Circular No.22, 2nd Edition).

### Channel Flow Time

Refers the time required for the runoff to flow from the channel inlet point to the outlet point.

The time of concentration calculated by using Kirpich formula:

$$T_{travel} = 0.02 L^{0.77} S^{0.385}$$

Where:  $T_{travel}$  = time of concentration (min):  $L$  = maximum length of travel (m)

$S$  = slope, equal to  $H/L$  where  $H$  is the difference in elevation between the most remote point in the basin and the outlet

In order to minimize error in calculating time of concentration particularly when the average basin slope varies significantly from the mean channel slope, time of concentration is calculated for two sections. The first one is from the outlet point to 0.7 of the channel flow length and the second one is from 0.7L to the end of the channel. Hence the channel flow time is added to the overland flow time to obtain the total time of concentration.

Hence, the total travel time which is considered to be time of concentration for the overall catchment flow system will be determined by summation of all travel times under different flow system. Time of concentration, catchment average slope and Computed time of concentration values is present on appendix part of this thesis.

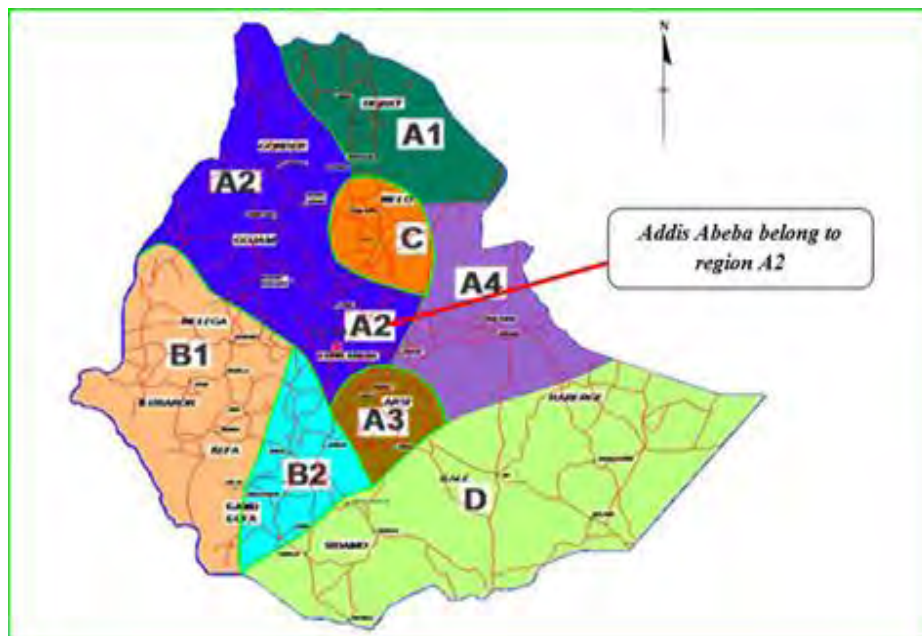


Figure 17: Rainfall regions of Ethiopia where Addis Ababa is found in region A2  
(Source: Ethiopia Road Authority drainage design manual, 2013)

### Rainfall Intensity

Rainfall intensity, duration, and frequency curves are necessary to use the rational method. Regional IDF curves available in ERA drainage modeling manual in relation to AACRA used to determine the rainfall intensity for duration equal to the time of concentration ( $T_c$ ) of desired frequency.

### Frequency Factors

ERA drainage modeling manual 2013 recommends the use of frequency factor  $C_f$  Table below to account for different return periods while computing the peak discharge for smaller catchments using rational formula. For minor drainage structures of return periods other than 5 & 10 frequency factors corresponding to required frequency is considered in peak discharge estimation.

Table 8: Frequency factor  $C_f$  for the rational formula

Recurrence Interval (yrs)	$C_f$
5	1.00
10	1.00
25	1.10
50	1.20
100	1.25

(Source: Ethiopia Road Authority drainage modeling manual 2013)

The runoff is simulated by means of a hydrological SWMM which is a numerical model (computer program) that includes a large set of methods to simulate watershed, channel and water control structure behavior thus predicting flow, stage and timing. On one hand, the surface runoff model considers just the surface runoff generated during a rainfall event, resulting in discontinuous runoff hydrographs. On the other hand, the continuous hydrological model computes the precipitation volume balance including both overland and subsurface runoff. The most suitable model for an urbanized area affected by heavy and intense rainfall events (this case study) is a surface runoff model, since most of the runoff is generated on impervious surfaces during rains and ceases when the event finishes (DHIa, 2010).

To compute the runoff the total catchment area is divided in a certain number of smaller cells depending on the simulation time step, see Equation below,

$$n = \frac{tc}{\Delta t}$$

Where  $t_c$  = concentration time and  $\Delta t$  = simulation time step

The area of each cell is calculated according to the defined time-area curve and the total area of all cells is the specified impervious area. The time-area curve accounts for the catchment reaction speed and shape, relating the flow time and the corresponding catchment sub-area (DHIa, 2010).

The surface runoff starts when the rain depth reaches the value of the initial loss, which determines the water held in the land surface either in depressions and irregularities or by surface tension. In the same way, the runoff generation ceases once the rain depth is below the initial loss parameter (DHIa, 2011). The volume of runoff in each cell is calculated as a balance of inflow from the upstream cell, the outflow to the downstream cell and the rainfall in the cell. The outflow in the most downstream cell is the surface runoff hydrograph. However, the runoff is then reduced in the impervious surfaces to account for the specified hydrological reduction evapotranspiration, imperfect imperviousness, etc. (DHIa, 2010) the computed runoff will be the load of the collection network.

Once the intensity for duration equal to the time of concentration at the desired frequency determined from IDF curve, the momentary peak discharge from the pavement surface and from the adjacent watershed area contributing to the pavement surface determined independently through rational method and the cumulative flow effect is taken into consideration for the modeling of pavement and storm drain drainage system facilities.

In the following subchapters the modeling of intercepting catchment and all drainage utilities are modeled and tabulated as below. Runoff and network system performance is also checked in next subchapters using SWMM5.

### **3.2.3 Storm drainage hydraulic elements modelling and analyses**

#### **Pavement drainage**

The objective of highway storm drainage modeling is to provide safe passage of vehicles during the designing storm event. Effective drainage of highway pavements is essential to the maintenance of highway service level and to traffic safety.

Pavement drainage requires consideration of surface drainage, gutter flow, and inlet capacity. The modeling of a drainage system for a curbed highway pavement section is to collect runoff in the gutter (which carries runoff generated from pavement surface and from the adjacent areas contributing to the pavement at a permissible level) and convey it to pavement inlets in a manner that provides reasonable safety for traffic and pedestrians at a reasonable cost.

The modeling of these elements is dependent on storm frequency and the allowable spread of storm water on the pavement surface.

### **Curb and gutter modeling**

Curbs are normally used at the outside edge of pavements for low-speed, highway facilities, and in some instances adjacent to shoulders on moderate to high-speed facilities. They serve the following purposes:

- Contain the surface runoff within the roadway and away from adjacent properties
- Prevent erosion on fill slopes
- Provide pavement delineation
- Enable the orderly development of property adjacent to the roadway

Gutter sections can be categorized as conventional or shallow swale type, Conventional curb and gutter sections usually have a triangular shape with the curb forming the near-vertical leg of the triangle as shown in Fig below. The distance of the spread,  $T$ , is measured perpendicular to the curb face to the extent of the water on the roadway. Conventional curb and gutter with uniform triangular section and constant cross slope ( $S_x$  equal to the slope of travel lane adjacent to the gutter)

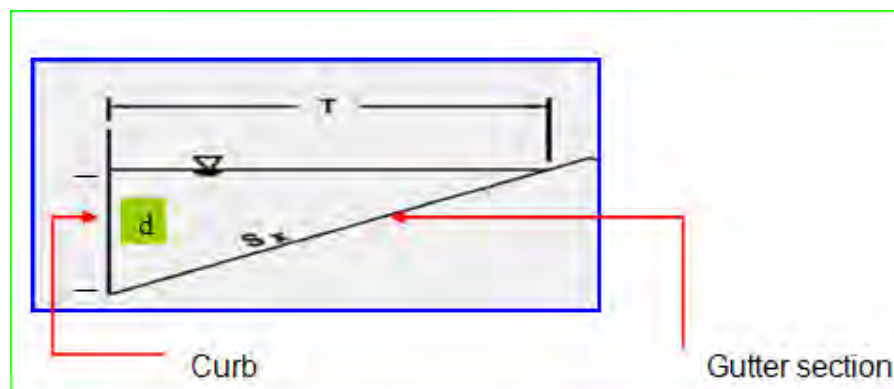


Figure 18: Curb-gutter section showing spread distance and pavement cross slope  $S_x$

### Capacity relationship

Gutter flow calculations are necessary to establish the spread of water on the shoulder, parking lane, or pavement section. A modification of the Manning's equation is used for computing flow in triangular channels.

To compute gutter flow, the Manning's equation is integrated for an increment of width across the section. The resulting equation is:

$$Q = \frac{K_u}{n} S_x^{1.67} S_L^{0.5} T^{2.67}$$

Where:

$$K_u = 0.376$$

$n$  = Manning's coefficient:  $Q$  = flow rate, m<sup>3</sup>/s

$T$  = width of flow (spread), m:  $S_x$  = cross slope, m/m

$S_L$  = longitudinal slope, m/m

The depth of water at the vertical edge of the gutter section, computed as follow:

$$d = T S_x$$

### Drainage inlet modeling

The receptors for surface water collected in ditches and gutters, which serve as the mechanism whereby surface water enters storm drains; refers to drainage inlets structures. Primarily Inlet structure is used to allow surface water to enter the storm drainage system but it also serve as an access point for cleaning and inspection.

The hydraulic capacity of a storm drain inlet depends upon its geometry as well as the characteristics of the gutter flow. Inlet capacity governs both the rate of water removal from the gutter and the amount of water that can enter the storm drainage system. Inadequate inlet capacity or poor inlet location may cause flooding on the roadway resulting in a hazard to the traveling public.

Inlets are typically located in gutter sections, paved medians, and roadside and median ditches.

Inlets used for the drainage of highway surfaces can be divided into the following four classes:

- Grate inlets; Curb-opening inlets; Slotted inlets and

➤ Combination inlets

The most commonly used Curb –opening drop inlet structure shown in Figure below with sump portion to settle and to collect sediment is used in modeled area approach storm water drainage system which serves both as water intercepting structure and access structure for underground pipe drain system.

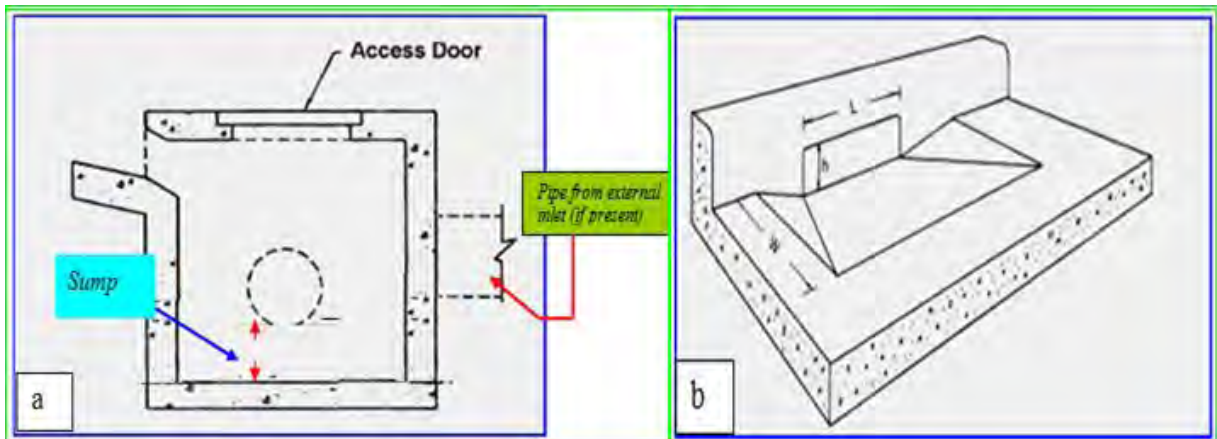


Figure 19: (a) Curb-opening drop inlet with access door, (b) Curb-opening inlet over view

### Inlet interception capacity and efficiency

Inlet interception capacity,  $Q_i$ , is the flow intercepted by an inlet under a given set of conditions. The efficiency of an inlet,  $E$ , is the percent of total flow that the inlet will intercept for those conditions. The efficiency of an inlet changes with changes in cross slope, longitudinal slope, total gutter flow, and, to a lesser extent, pavement roughness. In mathematical form, efficiency,  $E$ , is defined by the following equation:

$$E = \frac{Q_i}{Q}$$

Where:  $E$  = inlet efficiency;  $Q$  = total gutter flow,  $m^3/s$  ( $ft^3/s$ );  $Q_i$  = intercepted flow,  $m^3/s$  ( $ft^3/s$ ) Flow that is not intercepted by an inlet is termed carryover or bypass and is defined as follows:

$$Q_b = Q - Q_i$$

Where;  $Q_b$ - bypasses flow  $m^3/s$

The interception capacity of all inlet configurations increases with increasing flow rates, and inlet efficiency generally decreases with increasing flow rates. Factors affecting gutter flow also affect inlet interception capacity.

Interception capacity of a curb-opening inlet is largely dependent on flow depth at the curb and curbs opening length. Flow depth at the curb and consequently, curb-opening inlet interception capacity and efficiency, is increased by the use of a local gutter depression at the curb-opening or a continuously depressed gutter to increase the proportion of the total flow adjacent to the curb.

### **Curb-opening inlets**

Curb-opening inlets are effective in the drainage of highway pavements where flow depth at the curb is sufficient for the inlet to perform efficiently. Curb openings are less susceptible to clogging and offer little interference to traffic operation. Curb opening heights vary in dimension and the length of the curb-opening inlet required for total interception of gutter flow on a pavement section with a uniform cross slope is expressed by equation below:

$$L_T = K_u Q^{0.42} S_L^{0.3} \left( \frac{1}{n S_x} \right)^{0.6}$$

Where:

$K_u = 0.817$ ;  $L_T$  = curb opening length required to intercept 100 percent of the gutter flow, m

$S_L$  = longitudinal slope;  $Q$  = gutter flow, m<sup>3</sup>/s

The efficiency of curb-opening inlets shorter than the length required for total interception is expressed by:

$$E = 1 - \left( 1 - \frac{L}{L_T} \right)^{1.8}$$

Where:  $L$  = curb-opening length, m

Flow interception capacity of depressed curb-opening inlets is much higher as compared to un-depressed once. With this ground to increase the flow interception capacity of the inlet structure and to restrict the spread and depth of water on the curbed section depressed curb-opening inlet is used in modeled area approach roadway storm water drainage system.

The length of inlet required for total interception by depressed curb-opening inlets or curb openings in depressed gutter sections can be found by the use of an equivalent cross slope,  $S_e$ , in equation below in place of  $S_x$ .  $S_e$  can be computed using equation below:

$$S_e = S_x + S'_w E_o$$

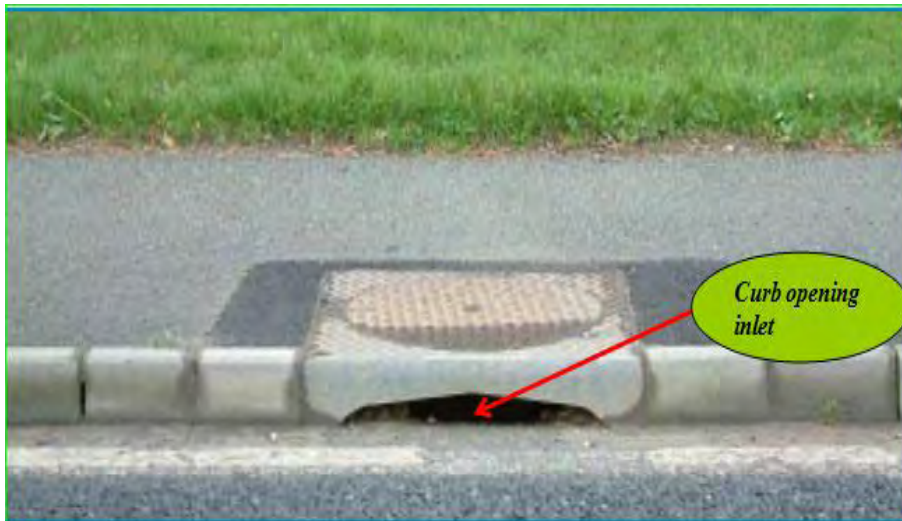


Figure 20: Curb opening inlet over view picture

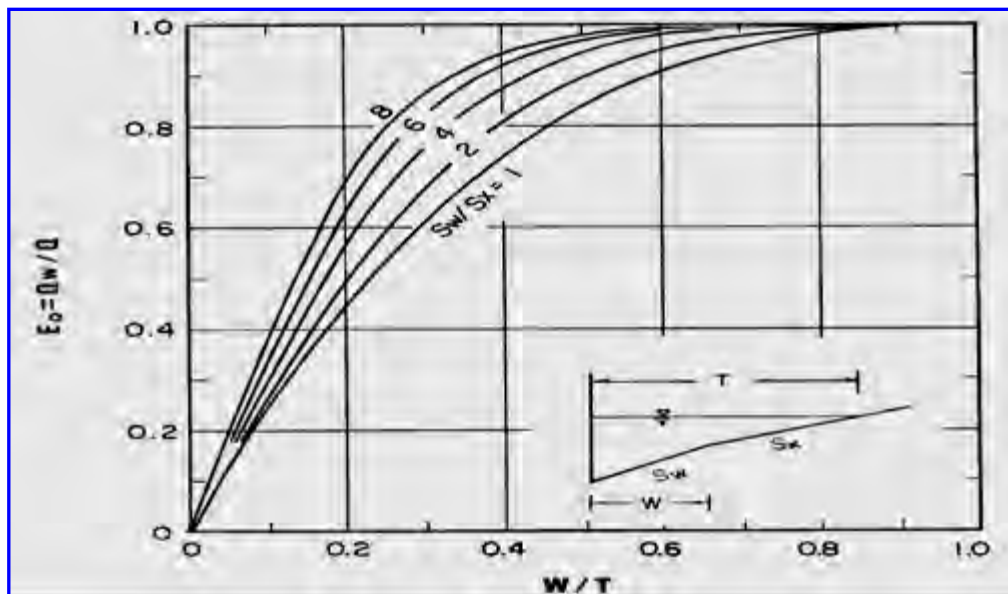


Figure 21: Chart showing the ratio of frontal flow to the total gutter flow  
(Source: Ethiopian Roads Authority drainage design manual, 2013)

Where:

$S_w$  = cross slope of the gutter measured from the cross slope of the pavement,

$S_x$ , m/m

$S'w = a / [1000 W] W$ , for  $W$  in m; or  $= S_w - S_x$

$a$  = gutter depression, mm (in)

$E_o$  = ratio of flow in the depressed section to total gutter flow determined by the Gutter configuration upstream of the inlet

### Catchment flow interception inlets

As stated before runoff from cut slopes, adjacent catchments and pavement surfaces of the feeder roads draining towards the approach road way should be intercepted before it reaches the highway. This limits the probability of sediment and debris deposition on the road way and as well reduce the amount of water carried in the gutter section there by restricting the spread and depth of water on the pavement surface of the road.

Taking into account these conditions, two external inlet structures on the right side Hospital to bridge approach and two on the right side Federal water works to bridge approach are modeled once the probable outlets identified based on physical observation and contoured map indication. Exact position and any adjustment of these structures should go along with the construction process to meet the requirement on a better manner.

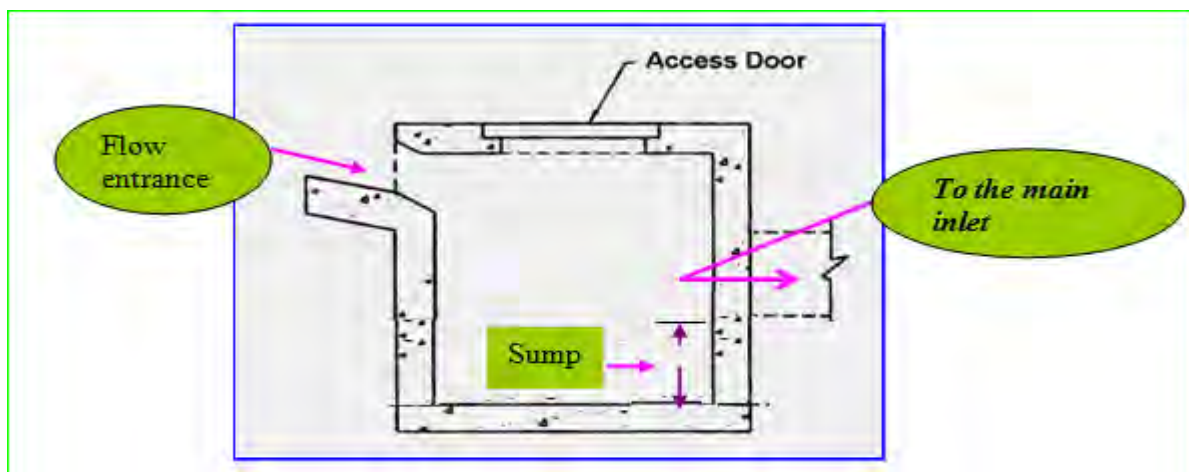


Figure 22: External inlet detail for catchment flow interception

### Pipes (Concrete)

Pipes are considered in SWMM5 as one-dimensional water conduits that convey water between manholes (nodes). In this case, the model just includes closed conduits links (pipes), although there is the possibility to model open channel links. Links are defined by three main properties: cross-section geometry, bottom slope and friction properties (DHIa, 2011).

The cross-section geometry of the pipes can be defined by either adopting a standard section (circular, rectangular, O-shaped, Egg-shaped or quadratic) or using CRS to define alternative shapes. The slope is determined by the upstream and downstream levels and the length of the pipes.

The link material is characterized by a hydraulic friction loss coefficient, which can be the Manning coefficient, the Colebrook White coefficient or the Hazen-Williams coefficient. The choice of one hydraulic friction loss formulation is of importance to determine the flow pattern at pipes (DHIa, 2011).

The flow description can be simulated with three different approximations. The dynamic wave approach uses the mass conservation equation and includes acceleration forces to fully simulate transients and backflow profiles. The diffusive wave approach just models the friction bed, gravity force and the hydrostatic gradient in the momentum equation, so that it is suitable for backflow analysis when the link bed and wall resistance forces dominate. The kinematic wave approach calculates the flow as a balance between friction and gravity forces, so that backflow effects are not considered (DHIa, 2010).

The hydraulic capacity of a storm drain is controlled by its size, shape, slope, and friction resistance. Several flow friction formulas have been advanced which define the relationship between flow capacity and these parameters. Manning's Equation, the most widely used formula for gravity flow is used in storm drain modeling of Modeled area approach road.

For circular storm drains flowing full, Manning's Equation becomes:

$$Q = \frac{K_Q}{n} D^{2.67} S_o^{0.4}$$

Where:  $V$  = mean velocity, m/s;  $Q$  = rate of flow, m<sup>3</sup>/s;  $K_Q = 0.312$ ;

$n$  = Manning's coefficient (0.016 for pipe concrete pipe surface);  $D$  = storm drain diameter, m  
 $S_o$  = slope of the hydraulic grade line (assumed to be equal with bed slope), m/m.

As described in AACRA'S drainage modeling manual, in modeling of underground pipe drain is based on the following considerations:

- Maximum velocity for pipe flowing full shouldn't be greater than 6m/s
- Minimum velocity for pipe flowing full shouldn't be less than 0.9m/s
- Manning's roughness coefficient of 0.013 used for modeling

With these modeling considerations, the storm drains in modeling area approach road are modeled accordingly as shown in tables below. However, low computed discharge values of small magnitude for certain storm drain segments results in a very small pipe diameter size which may not be applicable to apply it during construction phases. Provision of the smallest applicable diameter pipes from availability and clearance point of view under this conditions results in smaller flow velocity values. Hence, to avoid siltation problem in pipe drains it is important to flush and clear the underground pipes regularly and at times whenever it is required.

### **3.3 Modeling Rainfall Using EP SWMM5**

#### **3.3.1 EP SWMM 5**

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

Even for small catchments, runoff and consequent model predictions (and prototype measurements) may be very sensitive to spatial variations of the rainfall. For instance, thunderstorms (convective rainfall) may be highly localized, and nearby gages may have very dissimilar readings.

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

→ **Model set up procedure**

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

Run Simulation for Rainfall/Runoff and Flow routing

### 3.3.2 Governing equations

SWMM conceptualizes a sub catchment as a rectangular surface that has a uniform slope  $S$  and a width  $W$  that drains to a single outlet channel as shown in Figure below. Overland flow is generated by modeling the sub catchment as a nonlinear reservoir, as sketched in Figure 26.

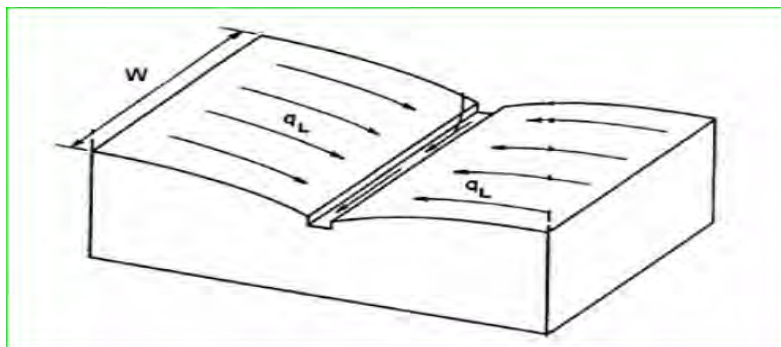


Figure 23: Idealized representation of a sub catchment

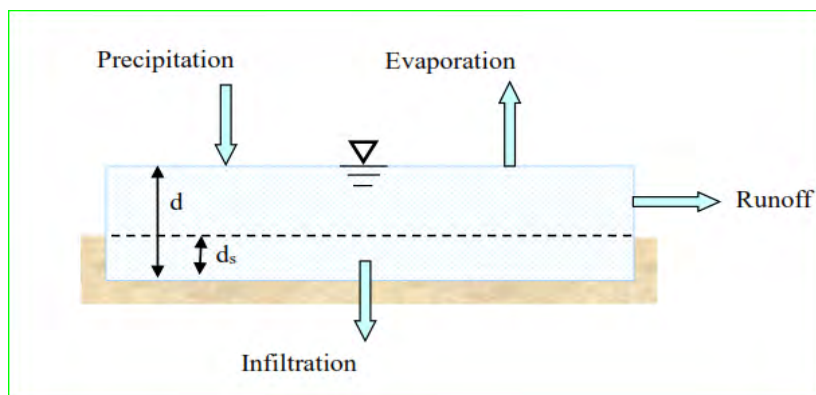


Figure 24: Nonlinear reservoir model of a sub catchment

In this representation, the sub catchment experiences inflow from precipitation (rainfall and snowmelt) and losses from evaporation and infiltration. The net excess ponds atop the sub catchment surface to a depth  $d$ . Pondered water above the depression storage depth  $d_s$  can become runoff outflow  $q$ . Depression storage accounts for initial rainfall abstractions such as surface ponding, interception by flat roofs and vegetation, and surface wetting.

From conservation of mass, the net change in depth  $d$  per unit of time  $t$  is simply the difference between inflow and outflow rates over the sub catchment:

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

Where:

$i$  = rate of rainfall + snowmelt (ft/s);     $e$  = surface evaporation rate (ft/s)

$f$  = infiltration rate (ft/s);                       $q$  = runoff rate (ft/s).

Note that the fluxes  $i$ ,  $e$ ,  $f$ , and  $q$  are expressed as flow rates per unit area ( $\text{cfs/ft}^2 = \text{ft/s}$ ).

The required rain input data in urban drainage applications depends on the nature of the engineering task. In this study both point Rainfall and block rains will be considered.

The rain data is commonly measured as intensity (mm/h) or depth (mm) and it is related to a statistical concept: frequency. The frequency is normally represented as the return period, which is the probability that a rainfall with certain intensity will be exceed or equaled in any year (Durrans & Haestad Methods, 2003).

### 3.3.3 Runoff evaluation

To simulate the generation of runoff, some parameters must be determined in the program to define the sub-catchments properties. The catchment was divided in 14 smaller sub-catchments connected to four nodes. This involves that all 14 nodes are linked to sub-catchments and collect the runoff generated within the area. The required input data to define the properties of the sub-catchments is set by the choice of one hydrological model. Green Ampt method is selected for infiltration model and Dynamic Wave is selected as a routing model.

According to Durrans & Haestad Methods (2003), the runoff is the amount of water in a rainfall that is not lost to interception, evapotranspiration or infiltration, so that it ends on water bodies or storm water collection structures after running through the surface.

Hence, the amount and characteristics of runoff not only depends on the rainfall pattern, but also on the catchment properties.

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

The imperviousness was calculated for each sub-catchment according to the percentage of different surfaces, see Equation below.

$$\phi = \frac{(A1 * \phi1 + A2 * \phi2 + \dots + An * \phi n)}{(A1 + A2 + \dots + An)}$$

Where  $\phi$  =imperviousness of the whole sub-catchment,  $\phi_i$ =imperviousness of each type of surface,  $A_i$  =area of each surface

Table 9: Impervious coefficient for different types of surface

Type of surface	Imperviousness coefficient
Roofs	90%
Concrete and asphalt surfaces	80%
Paved surfaces with gravel joints	70%
Gravel road, sharply slope mountainous park area without significant vegetation	40%
Outcrop with not significant slope	30%
Gravel paths with undeveloped parts of soil	20%
Parks with lush vegetation and rugged mountainous woodland	10%
Cultivated land , lawn or grassland	0-10%
Flat woodland	0-10%

(Source: Svenskt Vatten, 2004)

### 3.3.4 Model calibration and validation

- ✓ Model calibration: Manually
  - Using rational formula result
  - Trial and error method and
  - Statistical method

**Secondary data**

Table 10: Daily heaviest rainfall data for Bole sub city (mm)

<b>Year</b>	<b>24hr Max. Rainfall to Bole Sub City</b>
<b>2001</b>	32.4
<b>2002</b>	28.6
<b>2003</b>	34.6
<b>2004</b>	29
<b>2005</b>	44.5
<b>2006</b>	61.7
<b>2007</b>	71.2
<b>2008</b>	37.2
<b>2009</b>	51.2
<b>2010</b>	54.4
<b>2011</b>	36.9
<b>2012</b>	64.7
<b>2013</b>	42.6
<b>2014</b>	27.2
<b>2015</b>	60.5

Table 11: Nodes and outlets /outfalls invert elevation

Nodes	Invert elevation	Nodes	Invert elevation	Nodes	Invert elevation	Nodes	Invert elevation	Outlet	invert elevation
J1	2371.0	J21	2332	J41	2337	J61	2329	Outlet 1	2329
J2	2369.5	J22	2330	J42	2338	J62	2328.6	Outlet 2	2328.85
J3	2368.5	J24	2329.1	J43	2339	J63	2333.5	Outlet 3	2328.85
J4	2366.5	J25	2329.17	J44	2340	J64	2336	Outlet 4	2329
J5	2364.0	J26	2329.3	J45	2338	J65	2338.5		
J6	2362.0	J27	2329.5	J46	2337	J66	2341		
J7	2360.0	J28	2330	J47	2336	J67	2343.5		
J8	2358.5	J29	2330.5	J48	2335	J68	2346		
J9	2356.5	J30	2331	J49	2334.5	J69	2348.5		
J10	2354.0	J31	2331.5	J50	2334.3	J70	2351		
J11	2353.0	J32	2332	J51	2334	J71	2353.5		
J12	2351.0	J33	2332.5	J52	2333.5	J72	2356		
J13	2349.0	J34	2333	J53	2332.7	J73	2358.5		
J14	2347.0	J35	2333.5	J54	2332	J74	2361		
J15	2345.5	J36	2334	J55	2331.8	J75	2363.5		
J16	2343.5	J37	2334.5	J56	2331.5	J76	2366		
J17	2341.5	J38	2335	J57	2331	J77	2368.5		
J18	2340.0	J39	2335.5	J58	2330.5	J78	2371		
J19	2337.0	J40	2336	J59	2330	J79	2331		
J20	2335.0			J60	2329.5				

(Source: Addis Ababa City Road Authority)

## CHAPTER FOUR

### 4 RESULTS AND DISCUSSIONS

#### 4.1 Results Using Rational Formula

Using the daily maximum rainfall from metrological agency, 24 hour design rainfall was calculated using Log Pearson type III distribution methods. The values are compared with Ethiopian Roads Authority recommended values and the maximum was taken, as it is recommended by ERA. The rainfall of ERA is attached on the appendix part.

Table 12: Log Pearson type III daily heaviest rainfall analyses for Bole sub city

Applying Log Pearson Type III Distribution, 24 hours rainfall design $Y_T = Y_{avg} + K_{(T,n)} * \sigma_Y$							
Return period( $T_r$ )	Exceedance Probability	Skew Coefficient.	K (T,n)	$S_y$	$Y_T$	$X_T$	ERA, $X_T$
2	0.50	0.1	-0.17	0.14	-0.193	40.68	51.92
5	0.20		0.836	0.14	1.281	56.29	65.52
10	0.10		1.292	0.14	2.236	65.21	74.45
25	0.04		1.785	0.14	3.401	76.46	85.7
50	0.02		2.107	0.14	4.237	84.83	94.07
100	0.01		2.4	0.14	5.046	93.24	102.45

$X_T$  and ERA,  $X_T$  are calculated and Ethiopian roads authority recommended 24 hours design rainfall for a given return period T, respectively.

The intensity of the rainfall for design purposes is obtained from the intensity-duration-frequency (IDF) relationship for the sub catchment. Duration of the design rainfall is taken as the time of concentration ( $t_c$ ) which is obtained by most commonly used Kirpich formula as mentioned in chapter three. Use a minimum  $t_c$  value of 7 minutes for asphaltic and developed urban areas and a minimum  $t_c$  value of 15 minutes for areas that are not developed and intercepting catchments. (ERA, 2013)

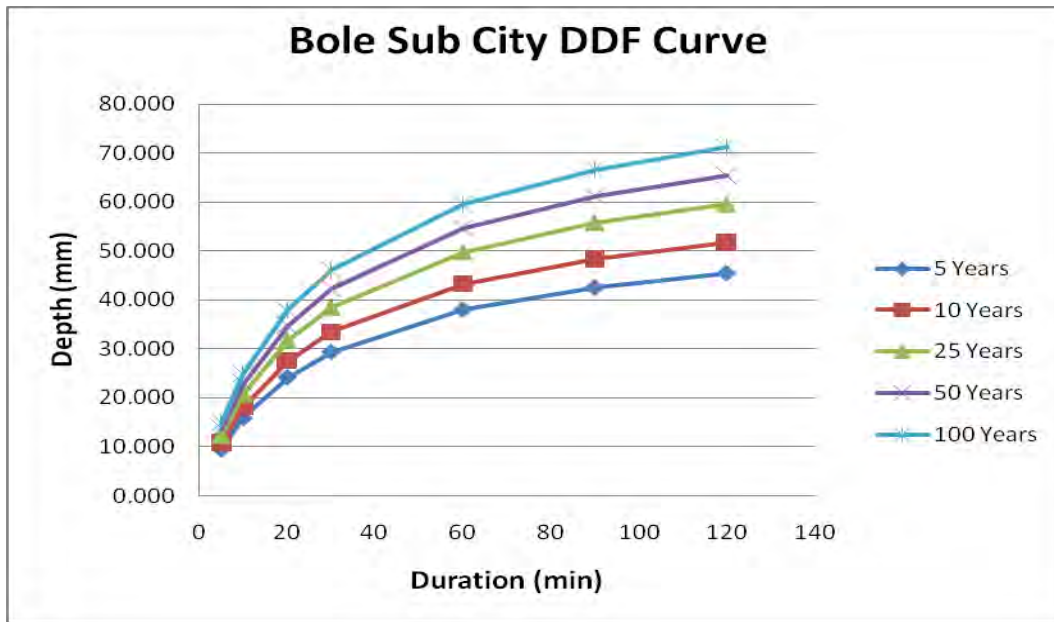


Figure 25: Bole sub city station DDF curve

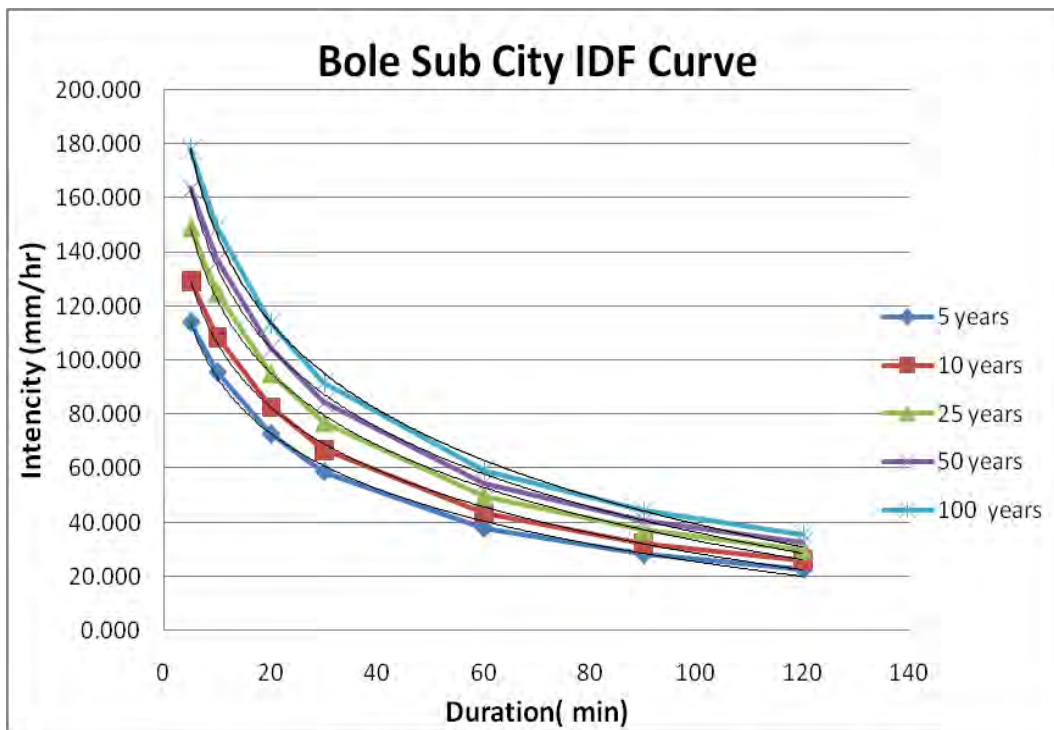


Figure 26: Bole sub city station IDF curve

The table below shows the trend equations for Bole sub-city with different return periods.

Table 13: IDF frequency curve trend equations and  $R^2$  values for Bole sub-city station

Return period (yr)	DDF Equation	DDF $R^2$ - values	IDF Equation	IDF $R^2$ - values
5	$y = 11.65\ln(x) - 10.10$	0.998	$y = -29.5\ln(x) + 161.6$	0.996
10	$y = 13.23\ln(x) - 11.48$	0.998	$y = -33.6\ln(x) + 183.6$	0.996
25	$y = 15.24\ln(x) - 13.22$	0.998	$y = -38.7\ln(x) + 211.4$	0.996
50	$y = 16.72\ln(x) - 14.51$	0.998	$y = -42.4\ln(x) + 232.0$	0.996
100	$y = 18.21\ln(x) - 15.80$	0.998	$y = -46.2\ln(x) + 252.7$	0.996

Table 14: Runoff coefficient values for different types of drainage areas

S. No.	Catchment	Surface Condition		Runoff Coeff., C	C*A	C weighted
		Description	Area Proportion (%)			
1	From Federal water works to Bridge/outlet	Unimproved areas	10	0.2	0.02	0.664
		Asphalt streets	40	0.83	0.332	
		Asphalt streets	30	0.78	0.234	
		single family area	20	0.4	0.08	
2	From Emperial Hotel to Bridge/outlet	Multi unit detached Resident	30	0.5	0.15	0.598
		Multi unit attached Resident	60	0.68	0.408	
		single family area	10	0.4	0.04	
3	From Bole airport to Bridge/outlet	Unimproved areas	20	0.2	0.04	0.643
		Asphalt streets	30	0.83	0.249	
		Asphalt streets	30	0.78	0.234	
		single family area	30	0.4	0.12	
4	From Hayat Hospital to Bridge/outlet	Multi unit detached Resident	20	0.5	0.10	0.602
		Multi unit attached Resident	65	0.68	0.442	
		single family area	15	0.4	0.06	

Weighted runoff coefficient was calculated using area proportion of each land use for all sub-catchments as shown on the above table. Runoffs from each sub-catchment were calculated using rational formula is tabulated as follows.

Table 15: Federal water work to the outlet interception catchment runoff

Sub catchment label	Area (ha)	Total Tc (min)	Min. Tc (min)/ ERA	IDF(for 10 years frequency)	Weighted Runoff coeff., C	Discharge (m <sup>3</sup> /s)
C1	1.151	2.936	15	95	0.664	0.20
C2	1.102	1.852	15	95	0.664	0.19
C3	1.22	1.726	15	95	0.664	0.21
C4	0.9	4.084	15	95	0.664	0.16

Table 16: Hayat Hospital to the outlet intercepts catchment runoff modeling

Sub catchment label	Area (ha)	Total Tc (min)	Minimum tc(min)/ERA	IDF(for 10 years frequency)	Weighted Runoff coeff., C	Discharge (m <sup>3</sup> /s)
C5	1.8	8.68	15	95	0.602	0.29
C6	1.78	7.25	15	95	0.602	0.27
C7	1.7	9.116	15	95	0.602	0.27

Table 17: Bole Airport to the outlet intercepts catchment runoff modeling

Sub catchment label	Area (ha)	Total Tc (min)	Minimum tc (min)/ERA	IDF(for 10 years frequency)	Weighted Runoff coeff., C	Discharge (m <sup>3</sup> /s)
C8	1.2	8.68	15	95	0.602	0.19
C9	1.2	7.25	15	95	0.602	0.19
C10	1.5	9.116	15	95	0.602	0.24

Table 18: Emperial Hotel to the outlet interception catchment runoff modeling

Sub catchment label	Area (ha)	Total Tc (min)	Min. Tc (min)/ ERA	IDF(for 10 years frequency)	Weighted Runoff coeff., C	Discharge (m <sup>3</sup> /s)
C11	1	2.936	15	95	0.68	0.18
C12	1.4	1.852	15	95	0.68	0.25
C13	1.3	1.726	15	95	0.68	0.23
C14	1.5	4.084	15	95	0.68	0.27

**Flow considered in the gutter**

Gutters formed in combination with curbs have varying widths. Gutter cross slopes may be the same as that of the pavement or may be modeled with a steeper cross slope. The cross slope should be adequate to provide proper drainage) usually 80 mm per meter (1 inch per foot) steeper than the shoulder or parking lane. AASHTO geometric guidelines state that an 8% slope is a common maximum cross slope. New Stadium bridge approach road pavement surface has a cross slope of 2.5%, hence the gutter section is modeled with a cross slope equal to the pavement cross slope. The minimum longitudinal slope gradient is more important for curbed pavement as it is more susceptible to the spread of storm water against the curb.

Pavement longitudinal slope of the approach road ranges from 0 to 5.5 %. The immediate approach road portion of the bridge has a longitudinal slope of 0%, but to avoid spread of water at these segments a minimum gutter grade of 0.35 percent is desirable (Recommended by FHWA, Urban drainage modeling manual).

A curb and gutter combination forms a triangular channel that can convey runoff equal to or less than the modeling flow without interruption of the traffic. When a modeling flow occurs, there is a spread or widening of the conveyed water surface. The water spreads to include not only the gutter width, but also parking lanes or shoulders, and portions of the traveled surface.

Gutter sections can be categorized as conventional or shallow swale type, conventional curb and gutter sections usually have a triangular shape with the curb forming the near-vertical leg of the triangle as shown in Fig below. The distance of the spread,  $T$ , is measured perpendicular to the curb face to the extent of the water on the roadway. Conventional curb and gutter with uniform triangular section and constant cross slope. For each sub catchments the runoff from nearby pavement and walkway (side walk) were calculated and inflow to the nearest and possible nodes and join the network.

The table below shows the runoff calculated for each gutter segment length on the street from former Emperial Hotel to the new stadium bridge/ outlet on this street there is four intercepting sub catchments which produce the runoff to the drainage network and also there is runoff from pavement and sidewalk this are the most runoff producer and making the drainage network to flood. The runoffs produced at each gutter length are calculated and join the drainage network by means of near permissible nodes. The total runoff from pavement and sidewalk for these street is  $0.0952\text{m}^3/\text{sec}$  this amount of flow is joining the drainage network.

Table 19: Emperial Hotel to outlet pavement and side walk discharge modeling

Gutter segment	Gutter segment length (m)	Pavement width(m)	Side walk width (m)	Runoff coeff., C		IDF (mm/hr)	Q <sub>pav.</sub> (m <sup>3</sup> /s)	Q <sub>sidew.</sub> (m <sup>3</sup> /s)	Q <sub>total</sub> (m <sup>3</sup> /s)
				pave ment	side walk				
EB1-EB2	28.04	10	5	0.83	0.78	108	0.0086	0.0040	0.0126
EB2-EB3	28.79	10	5	0.83	0.78	108	0.0088	0.0041	0.0129
EB3-EB4	28.71	10	5	0.83	0.78	108	0.0088	0.0041	0.0129
EB4-EB5	28.60	10	5	0.83	0.78	108	0.0031	0.0014	0.0045
EB5-EB6	28.00	10	5	0.83	0.78	108	0.0031	0.0014	0.0045
EB6-EB7	28.80	10	5	0.83	0.78	108	0.0019	0.0009	0.0028
EB7-EB8	28.80	10	5	0.83	0.78	108	0.0029	0.0013	0.0042
EB8-EB9	28.50	10	5	0.83	0.78	108	0.0029	0.0013	0.0042
EB9-EB10	28.80	10	5	0.83	0.78	108	0.0022	0.0010	0.0032
EB10-EB11	28.80	10	5	0.83	0.78	108	0.0028	0.0013	0.0041
EB11-EB12	28.00	10	5	0.83	0.78	108	0.0022	0.0010	0.0032
EB12-EB13	28.00	10	5	0.83	0.78	108	0.0031	0.0014	0.0045
EB13-EB14	28.79	10	5	0.83	0.78	108	0.0088	0.0041	0.0129
EB14-EB15	28.71	10	5	0.83	0.78	108	0.0088	0.0041	0.0129

Table 20: Federal water work to outlet pavement and side walk discharge modeling

Gutter segment	Gutter segment	Pavement width(m)	Side walk width(m)	Runoff coeff., C		IDF (mm/hr)	Q <sub>pav.</sub> (m <sup>3</sup> /s)	Q <sub>sidew.</sub> (m <sup>3</sup> /s)	Q <sub>total</sub> (m <sup>3</sup> /s)
				For pavement	For side walk				
FB1-FB2	32.80	10	5	0.83	0.78	108	0.0056	0.0027	0.0083
FB2-FB3	32.60	10	5	0.83	0.78	108	0.0027	0.0013	0.004
FB3-FB4	32.00	10	5	0.83	0.78	108	0.0055	0.0031	0.0086
FB4-FB5	30.60	10	5	0.83	0.78	108	0.0058	0.0041	0.0099
FB5-FB6	28.90	10	5	0.83	0.78	108	0.0053	0.0025	0.0078
FB6-FB7	28.69	10	5	0.83	0.78	108	0.0058	0.0011	0.0069
FB7-FB8	28.62	10	5	0.83	0.78	108	0.0043	0.0023	0.0066
FB8-FB9	28.62	10	5	0.83	0.78	108	0.0064	0.0034	0.0098
FB9-FB10	28.62	10	5	0.83	0.78	108	0.0078	0.0027	0.0105
FB10-FB11	28.62	10	5	0.83	0.78	108	0.0086	0.0033	0.0119
FB11-FB12	28.62	10	5	0.83	0.78	108	0.0081	0.0021	0.0102
FB12-FB13	28.62	10	5	0.83	0.78	108	0.0060	0.0029	0.0089
FB13-FB14	28.77	10	5	0.83	0.78	108	0.0087	0.0038	0.0125
FB14-FB15	28.80	10	5	0.83	0.78	108	0.0082	0.0047	0.0129
FB15-FB16	28.80	10	5	0.83	0.78	108	0.0076	0.0049	0.0125

During intensive rainfall the discharges from pavement and side walk is highly loaded the installed pipe, its runoff were calculated for the streets starting from federal water work to the outlet. On this street there are four sub catchments which all are contributing the runoff to the drainage networks. The sidewalk and pavement generate a large amount of runoff because there is low infiltration rate due to high runoff coefficient. The run off coefficient used to calculate flow in the sidewalk and pavement is 0.78 and 0.83 respectively from the table in chapter three.

Table 21: Bole airport to outlet approach pavement and side walk discharge modeling

Gutter segment	segment length (m)	Pavement width(m)	Side walk width (m)	Runoff coeff.,			Q <sub>pav</sub> (m <sup>3</sup> /se).	Q <sub>sidew</sub> (m <sup>3</sup> /se)	Q <sub>total</sub> (m <sup>3</sup> /se)
				pavement surface	For side walk	IDF (mm/hr)			
BB1-BB2	29.6	10	5	0.83	0.78	108	0.003	0.001	0.004
BB2-BB3	29.6	10	5	0.83	0.78	108	0.003	0.002	0.005
BB3-BB4	29.8	10	5	0.83	0.78	108	0.003	0.001	0.004
BB4-BB5	29.4	10	5	0.83	0.78	108	0.004	0.002	0.006
BB5-BB6	29.8	10	5	0.83	0.78	108	0.003	0.001	0.004
BB6-BB7	29.22	10	5	0.83	0.78	108	0.005	0.003	0.008
BB7-BB8	29.7	10	5	0.83	0.78	108	0.009	0.004	0.013
BB8-BB9	27.9	10	5	0.83	0.78	108	0.004	0.004	0.008
BB9-BB10	27.7	10	5	0.83	0.78	108	0.003	0.004	0.007
BB10-BB11	27.5	10	5	0.83	0.78	108	0.007	0.004	0.011
BB11-BB12	27.48	10	5	0.83	0.78	108	0.008	0.004	0.012
BB12-BB13	28.64	10	5	0.83	0.78	108	0.009	0.004	0.013
BB13-BB14	28.64	10	5	0.83	0.78	108	0.002	0.004	0.006
BB14-BB15	28.64	10	5	0.83	0.78	108	0.005	0.001	0.006
BB14-BB14'	28.64	10	5	0.83	0.78	108	0.002	0.002	0.004
BB15-BB14	28.64	10	5	0.83	0.78	108	0.006	0.004	0.01
BB15-BB15'	28.65	10	5	0.83	0.78	108	0.007	0.004	0.011
BB16-BB15	28.73	10	5	0.83	0.78	108	0.008	0.004	0.012
BB17-BB16	29.6	10	5	0.83	0.78	108	0.009	0.004	0.013

The above table shows the total runoff calculated for the street from Bole Airport overpass to the new stadium bridge/ outlet on this street there is three sub catchments which produce the runoff to the drainage network and also there is pavement and sidewalk this are the most runoff producer and making the drainage network to flood. The total runoff from pavement and sidewalk for these street is 0.15m<sup>3</sup>/sec this amount of flow is joining the drainage network.

Totally in the study area the drainage network is classified into four streets with four outfall or outlets for all the streets the flow in all drainage utilities are calculated and added in the network simulation for analyzing and modeling purpose. Therefore on this road side from Hayat Hospital to the bridge/outlet the total runoff from sidewalk and pavement were greater than the above mentioned two streets, the reason were in this street the slope is larger than both streets.

Table 22: Hayat Hospital to outlet pavement and side walk discharge modelling

Gutter segment	Gutter segment length (m)	Pavement width(m)	Side walk width (m)	Runoff coeff., C		IDF (mm/hr)	Q <sub>sidew.</sub> (m <sup>3</sup> /s)	Q <sub>total</sub> (m <sup>3</sup> /s)	Q <sub>pav</sub> (m <sup>3</sup> /s)
				For pavement surface	For side walk surface				
HB1-HB2	28.93	10	5	0.83	0.78	108	0.0089	0.0042	0.0131
HB2-HB3	28.79	10	5	0.83	0.78	108	0.0088	0.0041	0.0129
HB3-HB4	28.90	10	5	0.83	0.78	108	0.0077	0.0036	0.0113
HB4-HB5	28.60	10	5	0.83	0.78	108	0.0023	0.0011	0.0034
HB5-HB6	29.80	10	5	0.83	0.78	108	0.009	0.0045	0.0135
HB6-HB7	28.77	10	5	0.83	0.78	108	0.0088	0.0041	0.0129
HB7-HB8	28.77	10	5	0.83	0.78	108	0.0088	0.0041	0.0129
HB8-HB9	27.90	10	5	0.83	0.78	108	0.0046	0.0022	0.0068
HB9-HB10	29.60	10	5	0.83	0.78	108	0.0028	0.0013	0.0041
HB10-HB11	26.87	10	5	0.83	0.78	108	0.0028	0.0013	0.0041
HB11-HB12	29.76	10	5	0.83	0.78	108	0.0025	0.0008	0.0033
HB12-HB13	28.84	10	5	0.83	0.78	108	0.0089	0.0041	0.0130
HB13-HB14	28.82	10	5	0.83	0.78	108	0.0088	0.0036	0.0124
HB14-HB15	28.80	10	5	0.83	0.78	108	0.0077	0.0011	0.0088
HB15-HB16	28.78	10	5	0.83	0.78	108	0.0023	0.0046	0.0069
HB16-HB17	28.76	10	5	0.83	0.78	108	0.009	0.0045	0.0135
HB17-HB18	28.74	10	5	0.83	0.78	108	0.0088	0.0041	0.0129
HB18-HB19	26.87	10	5	0.83	0.78	108	0.0088	0.0044	0.0132
HB19-HB20	30.76	10	5	0.83	0.78	108	0.0085	0.0041	0.0126

Gutter flow computation in New Stadium Bridge Approach Road involves momentary peak discharge determination from the pavement surface draining towards the gutter and incorporating runoff expected to enter to the gutter section from side walk portion and adjacent watersheds which don't enter directly to the inlet structure.

Where ever possible runoff from adjacent areas draining towards the roadway should be intercepted before it reaches the highway. By doing so, the deposition of sediment and other debris on the roadway as well as the amount of water carried in a gutter section will be restricted to the desired level. In case of modeling area approach roadway external inlet structures are laid at appropriate drainage out let points in the right side so as to intercept the runoff generated from the adjacent catchment areas.

Modeling area approach road curbed gutter section has a varying flow depth and water spread width against the curb. The curbed gutter section longitudinal slope is governed by the pavement edge longitudinal slope obtained from geometric modeling of the approach road. Curbed gutter section modeling primarily targeted to limit the water depth and water spread on the pavement surface of the road to a permissible range.

Water spread width of less than half of the travel lane (1.7m in case of modeled area approach roadway) and gutter flow depth at curbed edge not greater than 4.0 cm (equivalent to AACRA's & Urban drainage manuals recommendation) is used in gutter section modeling of the approach road but for some gutter segments maximum spread of 1.76m and flow depth of 4.4cm adopted to avoid frequent provision of inlet structures which is not economical in relation to the very limited spread and depth rise in the gutter section. Flow depth and water spread width for each gutter run segment is shown in four tables below.

Table 23: Hayat Hospital to outlet gutter modeling detail

Gutter segment	Gutter segment length (m)	Pavement width (m)	Side walk width (m)	Runoff coeff., C		IDF (mm/hr)	Q <sub>sidew</sub> .m <sup>3</sup> /s	Q <sub>total</sub> , m <sup>3</sup> /s	Q <sub>pav</sub> (m <sup>3</sup> /s)
				For pavement surface	For side walk surface				
		10	5	0.83	0.78	108	0.0089	0.0042	0.02
HB1-HB2	28.93	10	5	0.83	0.78	108	0.0088	0.0041	0.019
HB2-HB3	28.79	10	5	0.83	0.78	108	0.0077	0.0036	0.017
HB3-HB4	28.90	10	5	0.83	0.78	108	0.0023	0.0011	0.005
HB4-HB5	28.60	10	5	0.83	0.78	108	0.009	0.0045	0.023
HB5-HB6	29.80	10	5	0.83	0.78	108	0.0088	0.0041	0.019
HB6-HB7	28.77	10	5	0.83	0.78	108	0.0088	0.0041	0.019
HB7-HB8	28.77	10	5	0.83	0.78	108	0.0046	0.0022	0.01
HB8-HB9	27.90	10	5	0.83	0.78	108	0.0028	0.0013	0.006
HB9-HB10	29.60	10	5	0.83	0.78	108	0.0028	0.0013	0.006
HB10-HB11	26.87.	10	5	0.83	0.78	108	0.0025	0.0012	0.006
HB11-HB12	29.76	10	5	0.83	0.78	108	0.0089	0.0041	0.019
HB12-HB13	28.84	10	5	0.83	0.78	108	0.0088	0.0036	0.017
HB13-HB14	28.82	10	5	0.83	0.78	108	0.0077	0.0011	0.005
HB14-HB15	28.80	10	5	0.83	0.78	108	0.0023	0.0045	0.023
HB15-HB16	28.78	10	5	0.83	0.78	108	0.009	0.0041	0.019
HB16-HB17	28.76	10	5	0.83	0.78	108	0.0088	0.0041	0.019
HB17-HB18	28.74	10	5	0.83	0.78	108	0.0028	0.0022	0.01
HB18-HB19	26.87.	10	5	0.83	0.78	108	0.0028	0.0013	0.006
HB19-HB20	30.76	10	5	0.83	0.78	108	0.0025	0.0013	0.02

Table 24: Emperial Hotel to outlet approach gutter modeling detail

Gutter segment	Gutter segment length (m)	Slope (m/m)		Manning roughness for the pavement surface	Flow considered in gutter (m <sup>3</sup> /s)	Gutter water spread, T (m)	Gutter flow depth, d (m)
		Pavement cross slope, S <sub>x</sub>	Gutter Longitudinal slope, S <sub>o</sub>				
EB1-EB2	28.04	0.025	0.0283	0.016	0.019	1.36	0.034
EB2-EB3	28.79	0.025	0.01465	0.016	0.025	1.71	0.043
EB3-EB4	28.71	0.025	0.0129	0.016	0.026	1.77	0.044
EB4-EB5	28.6	0.025	0.001	0.016	0.007	1.73	0.043
EB5-EB6	28	0.025	0.001	0.016	0.007	1.73	0.043
EB6-EB7	28.8	0.025	0.0008	0.016	0.004	1.51	0.038
EB7-EB8	28.8	0.025	0.0008	0.016	0.006	1.76	0.044
EB8-EB9	28.5	0.025	0.0008	0.016	0.006	1.76	0.044
EB9-EB10	28.8	0.025	0.0008	0.016	0.005	1.6	0.04
EB10-EB11	28.8	0.025	0.0008	0.016	0.006	1.73	0.043
EB11-EB12	28	0.025	0.0008	0.016	0.005	1.58	0.04
EB12-EB13	28	0.025	0.0008	0.016	0.007	1.65	0.04
EB13-EB14	28.79	0.025	0.0008	0.016	0.004	1.72	0.042
EB14-EB15	28.71	0.025	0.0008	0.016	0.005	1.7	0.043

Runoff considered in the gutter length, all intercepting sub-catchments, pavements and sidewalk are used to fix or decide the drainage pipe diameter (section size). According to the simulation the existing pipe diameter is insufficient to detain the runoff from all drainage utilities.

Table 25: Federal water work construction to outlet gutter modeling detail

Gutter segment	Gutter segment length (m)	Slope (m/m)		Manning roughness for pavement	Flow considered in gutter (m <sup>3</sup> /s)	Gutter water, spread, T (m)	Gutter flow depth, d (m)
		Pavement cross slope, S <sub>x</sub>	Gutter Longitudinal slope, S <sub>o</sub>				
FB1-FB2	32.8	0.025	0.007	0.016	0.008	1.29	0.032
FB2-FB3	32.6	0.025	0.012	0.016	0.025	1.78	0.044
FB3-FB4	32	0.025	0.012	0.016	0.026	1.78	0.045
FB4-FB5	30.6	0.025	0.015	0.016	0.025	1.71	0.043
FB5-FB6	28.9	0.025	0.015	0.016	0.026	1.73	0.043
FB6-FB7	28.69	0.025	0.015	0.016	0.019	1.53	0.038
FB7-FB8	28.62	0.025	0.022	0.016	0.023	1.54	0.038
FB8-FB9	28.62	0.025	0.021	0.016	0.026	1.63	0.041
FB9-FB10	28.62	0.025	0.018	0.016	0.028	1.71	0.043
FB10-FB11	28.62	0.025	0.032	0.016	0.028	1.54	0.039
FB11-FB12	28.62	0.025	0.038	0.016	0.031	1.54	0.039
FB12-FB13	28.62	0.025	0.05	0.016	0.033	1.50	0.038
FB13-FB14	28.77	0.025	0.05	0.016	0.036	1.55	0.039
FB14-FB15	28.8	0.025	0.055	0.016	0.037	1.55	0.039
FB15-FB16	28.8	0.025	0.055	0.016	0.039	1.58	0.039
FB16-FB15	28.8	0.025	0.047	0.016	0.040	1.64	0.041
FB17-FB16	28.8	0.025	0.042	0.016	0.026	1.42	0.036
FB18-FB17	28.8	0.25	0.042	0.016	0.058	1.42	0.036
FB19-FB18	28.77	0.025	0.046	0.016	0.019	1.25	0.031
FB20-FB19	28.77	0.025	0.031	0.016	0.027	1.52	0.038

Table 26: Bole Airport to outlet gutter modeling detail

Gutter segment	length (m)	Slope (m/m)			roughness for the pavement surface	Flow considered in gutter (m <sup>3</sup> /s)	Gutter water spread, T (m)	Gutter flow depth, d (m)
		Pavement cross slope, $\alpha$	itudinal slope, $\beta$	transverse				
BB1-BB2	29.6	0.025	0.0066	0.016	0.008	1.29	0.032	
BB2-BB3	29.6	0.025	0.0066	0.016	0.012	1.53	0.038	
BB3-BB4	29.8	0.025	0.015	0.016	0.019	1.54	0.038	
BB4-BB5	29.4	0.025	0.015	0.016	0.023	1.66	0.041	
BB5-BB6	29.8	0.025	0.015	0.016	0.013	1.31	0.033	
BB6-BB7	29.22	0.025	0.005	0.016	0.013	1.61	0.040	
BB7-BB8	29.7	0.025	0.008	0.016	0.007	1.18	0.029	
BB8-BB9	27.9	0.025	0.005	0.016	0.007	1.28	0.032	
BB9-BB10	27.7	0.025	0.005	0.016	0.009	1.45	0.036	
BB10-BB11	27.5	0.025	0.005	0.016	0.007	1.28	0.032	
BB11-BB12	27.48	0.025	0.005	0.016	0.012	1.58	0.039	
BB12-BB13	28.64	0.025	0.017	0.016	0.019	1.51	0.038	
BB13-BB14	28.64	0.025	0.0183	0.016	0.024	1.61	0.040	
BB14'-BB15	28.64	0.025	0.0316	0.016	0.026	1.50	0.038	
BB14-BB14'	28.64	0.025	0.038	0.016	0.029	1.52	0.038	
BB15-BB14	28.64	0.025	0.058	0.016	0.032	1.45	0.036	
BB15-BB15'	28.65	0.025	0.065	0.016	0.036	1.47	0.037	
BB16-BB15	28.73	0.025	0.065	0.016	0.039	1.52	0.038	
BB17'-BB16	29.6	0.025	0.065	0.016	0.026	1.31	0.033	

Table 27: External drop inlet modeling description

Inlet Label	Inlet joining to	Approach side	Pipe length, m	Pipe diameter	
				Provided Pipe dia. m	Recommended pipe diam. m
HBO01	J37	Hayat Hospital	180	0.6	1.0
HBO02	J42	Hayat Hospital	220.8	0.6	1.0
EBO03	J71	Emperial Hotel	307.78	0.6	1.0
EBO04	J78	Emperial Hotel	165.3	0.6	1.0

### Storm drain outfalls

All storm drains have an outlet where flow from the storm drainage system is discharged. The discharge point can be a natural river or stream, an existing storm drainage system, or a channel which is either existing or proposed for the purpose of conveying the storm water away from the highway. The procedure for calculating the energy grade line through a storm drainage system begins at the outfall. Therefore, consideration of outfall conditions is an important part of storm drain modeling.

Discharge should always be in the direction of flow of the river or stream. Protection of the bed and edges of the watercourse at the point of entry (by means of rock amour, gabions, headwalls etc.) will help to prevent erosion by water discharging from the pipe during heavy storms.

This should be confirmed at the site level through keen observation at the outfall points during flood time and also by investigating the stability of the bank during construction phase.

### Curbed gutter section modeling

First. Preparing a Preliminary layout plan showing locations of inlets, drain pipes, direction of flow, number or label assigned to each structure and out falls. Detail schematic plan layout description is given by Geometric modeling of the road as shown in appendix

Second. Contributing drainage area determination to the pavement drainage facility  
(Cured –gutter section)

- ✓ The pavement width of the approach road contributing to the gutter from the geometric modeling 9.75 (~10m), and the width side walk from geometric 4.75 (~5m).
- ✓ The length of the pavement gutter run considered (distance between two consecutive inlets i.e. EB1 & EB2 is 28.04m

Hence, pavement surface area = width X run length

$$= 10\text{m} \times 28.04\text{m}$$

$$= 0.028\text{ha}$$

Likely, side walk (pedestrian road) area = 5m X 28.04m

$$= 0.014\text{ha}$$

- ✓ Here at this gutter run segment there is no adjacent catchment area draining towards the gutter section but flow considered from upstream pipe system is nearly 0.237m<sup>3</sup>/s.

Third. Time of concentration determination for the water to reach to the gutter section from remote point of the contributing area.

- ✓ Flow for contributing area usually less than 100m run is sheet flow where TC determined first by determining the flow velocity (based on the land use) & dividing it to the flow distance
- ✓ From Fig 23, the flow velocity for paved area and shallow gutter flow with cross slope of 2.5 % ( pavement cross slope) is nearly 1 m/s, again for
- ✓ Pedestrian road surface with slope of 3%, the flow velocity become 1.1 m/s

Then, applying equation for Time of Concentration,

$$T_c = D / (60 * V)$$

$$= 10 / (60 * 1)$$

$$= 0.17 \sim 0.2 \text{ minutes for pavement surface \& } 0.2 \text{ minute for Side walk reach.}$$

But the minimum  $T_c$  value used shouldn't be less than 7 minutes (FHWA, Urban drainage manual recommendation)

Rainfall intensity determination for duration equal to  $T_c$  of given return period (10 years in this case) from IDF curve of the region in Figure 24

For  $T_c$  equal to 7 minutes from IDF curve,  $I = 108\text{mm/hr}$

Fourth. Appropriate Runoff coefficient determination for the surface

- ✓ For asphalted surface, from table 9,  $C = 0.83$  & for brick side walk surface

$$C = 0.78$$

Fifth. Discharge estimation using rational method

$$= 0.83 \times 108 \times 0.028 \times 1/360$$

= 0.00693 m<sup>3</sup>/s (from pavement surface) & 0.004 m<sup>3</sup>/s (from pedestrian road)

Total discharge to on gutter section, (0.00693+0.004) m<sup>3</sup>/s & considering discharge flexibility factor of 1.5 to accommodate discharge which may enter to the gutter section from different direction, the total discharge become = 0.0164 m<sup>3</sup>/s.

Sixth. Gutter section determination, applying equation below, with pavement longitudinal Slope of 0.0283 and cross slope of 0.025, the water spread on pavement surface against the vertical edge (curb),

$$Q = \frac{K_u}{n} S_x^{1.67} S_L^{0.5} T^{2.67}$$

Spread, T=1.36m and depth of water at vertical section, d=0.034m

Hence, the gutter has a triangular section with curb vertical edge, cross slope 2.5% (equal to pavement cross slope), curb height 0.17~0.2m, flow depth 0.034m, width of spread, T=1.36m, and longitudinal slope for run segment considered, S<sub>o</sub>=0.0283

### **Curb opening inlet modeling**

Considering depressed curb opening inlet shown in document to increase flow interception capacity of the inlet, applying equation below, the total length of the curb opening to intercept 100% of the gutter flow become,

$$L_T = K_u Q^{0.42} S_L^{0.3} \left( \frac{1}{n S_x} \right)^{0.6}$$

$$L_T = 3.08\text{m}$$

Where S<sub>w</sub>=8%, S<sub>x</sub>=2.5%, W/T=0.4, E<sub>o</sub>=0.85, and other parameters remain the same as with gutter section.

But to meet the existing standard and to be economical the curb opening inlet is not modeled to intercept all the gutter discharge. Hence, a curb opening inlet length of 1.5m provided. With this opening length,

The flow interception efficiency of the inlet, E given by equation below, become

$$E = 1 - \left( 1 - \frac{L}{L_T} \right)^{1.8}$$

$$E = 0.70$$

Hence, the flow intercepted,  $Q_i = Q * E$

$$= 0.013 \text{ m}^3/\text{s}$$

Flow bypass =  $Q - Q_i$

$$= 0.006 \text{ m}^3/\text{s} \text{ \& transferred to the next gutter run segment}$$

### Storm drain pipe modeling

Seventh. Underground pipe modeling involves pipe diameter determination, flow velocity checking (non-silting velocity) for the intercepted flow.

Applying equation below, pipe diameter for

$$Q = Q_i + Q_{\text{carried from upstream}}$$

$$S_o = 0.02558, n = 0.013$$

$D = 1.17 \text{ m}$  (but to meet clearing and maintenance operation more easily and to use the available pipe size 1.5m dia. Pipe adopted in the system & flow velocity of 2.52m/s ( $> 0.9 \text{ m/s}$ ) obtained from hydraulic calculation.

### 4.2 EP SWMM5 Model Results

The model covers a total area of 18.75ha of sub catchment which consist of impervious surfaces as roofs or roads, resulting in an average value of 50% of non-permeable surfaces and 7.4km length of network simulation.

### 4.2.1 Description of the network model

As previously mentioned, the modelled area is divided into 14 sub-catchments and flows from asphaltic road and walkway and pavements. The network consists of 82 nodes means 78 Junctions and 4 outfalls, 79 links, see Figure below.

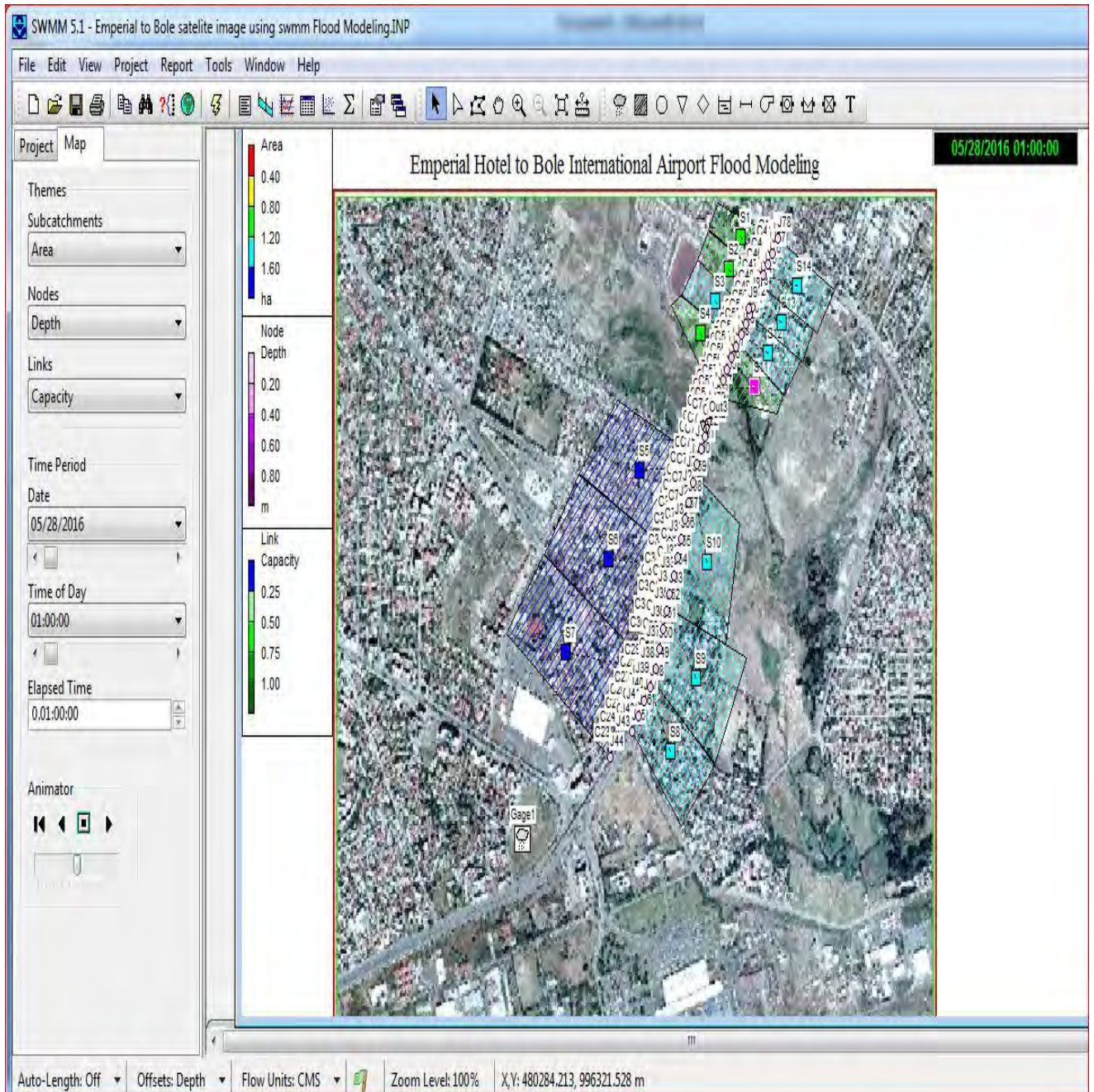


Figure 27: Sketch of the model map

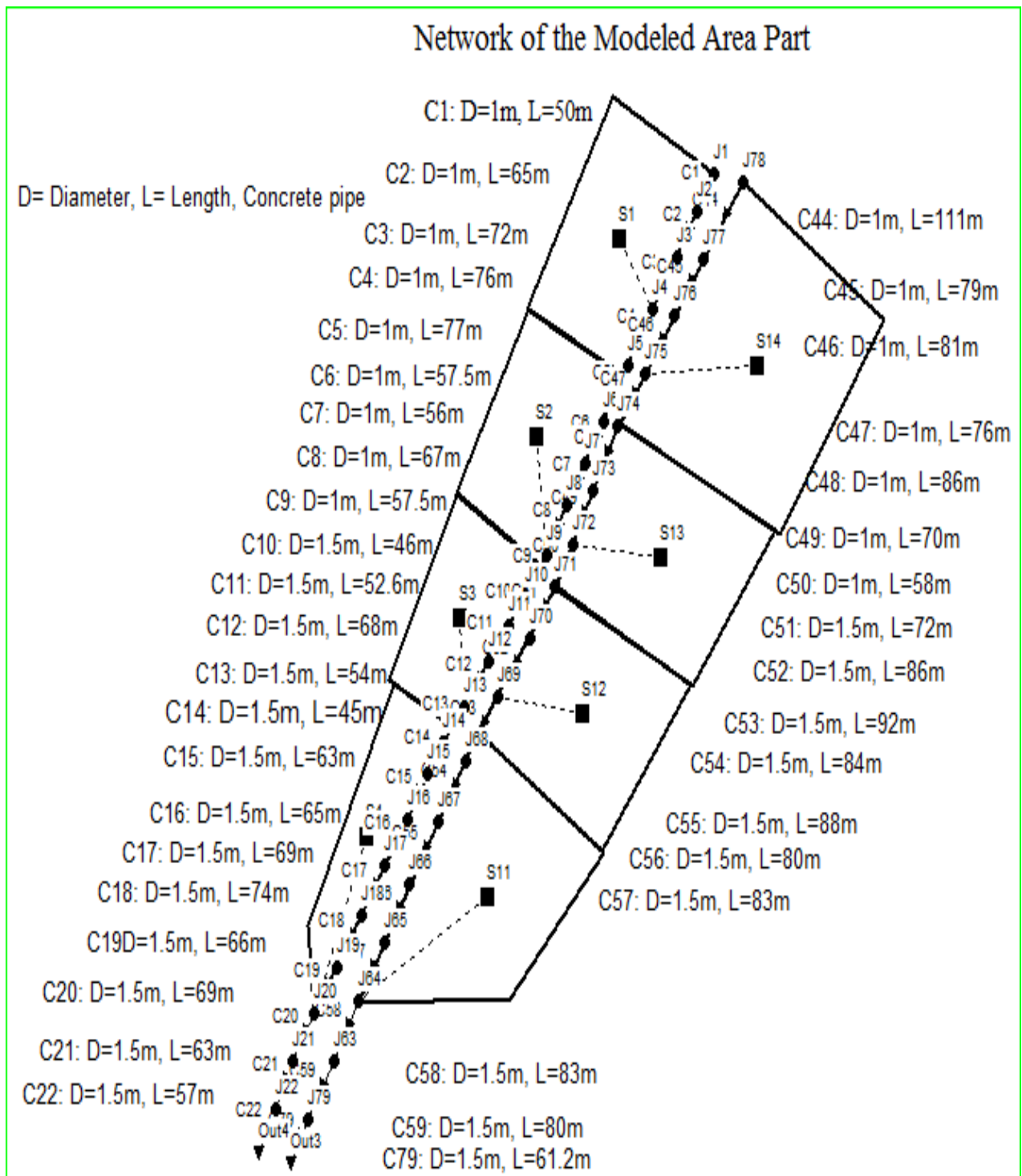


Figure 28: Network of the modeled area part one

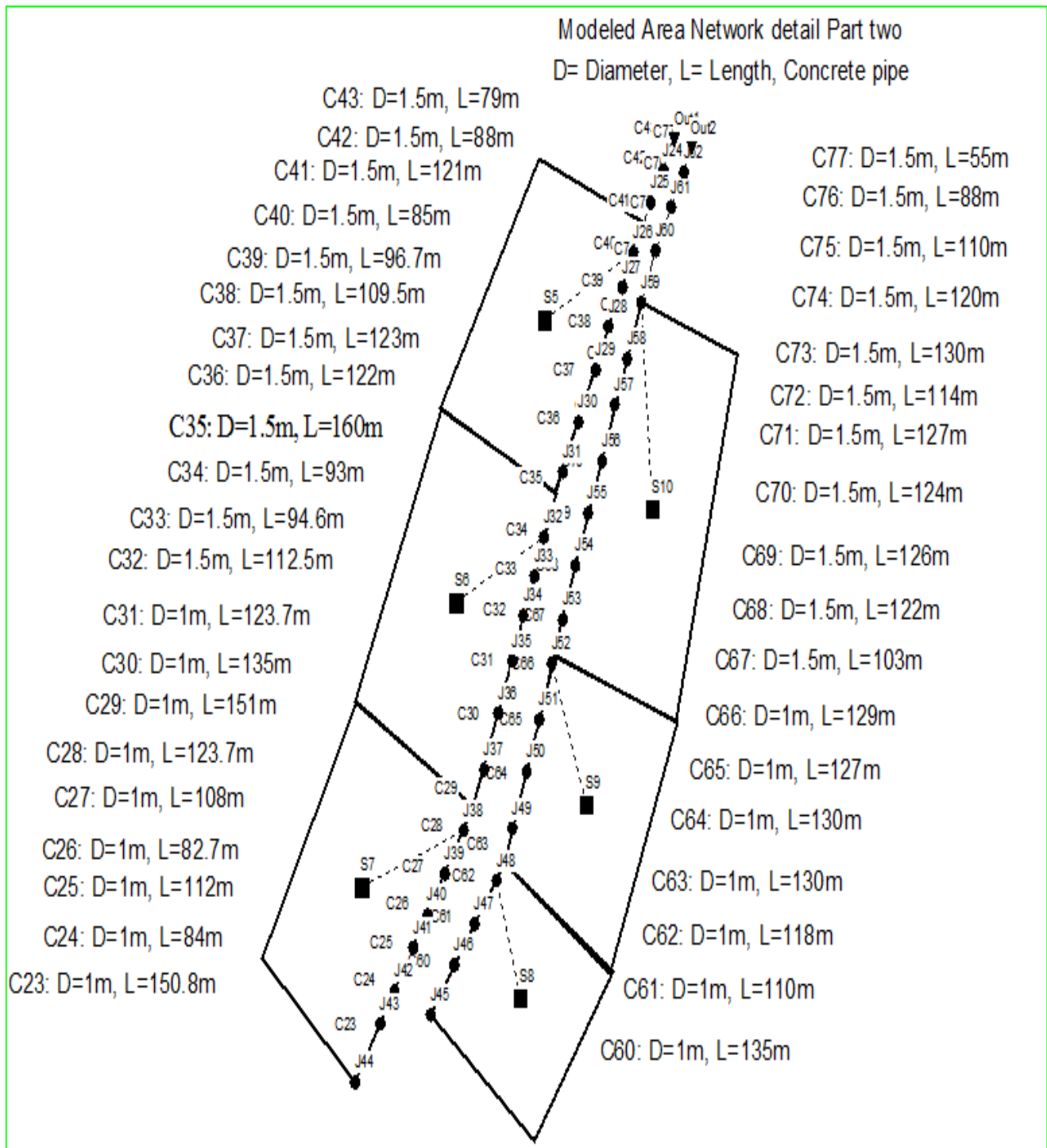


Figure 29: Network of the modeled area part two

The manholes/Junctions are all modeled as circular manholes of one meter of diameter. It has been assumed that there are no energy losses in the manholes.

Moreover, the model includes boundary conditions to represent various types of water loads, as infiltration or fixed water levels. The precipitation is introduced into the model by associating each sub-catchment to the rainfall time-series. The general network performance is determined by infiltration rates and the average water flow production.

Hence, the average water flow production in each node was multiplied by a cyclic variation that has been created to account for these variations in the water production, see Figure below.

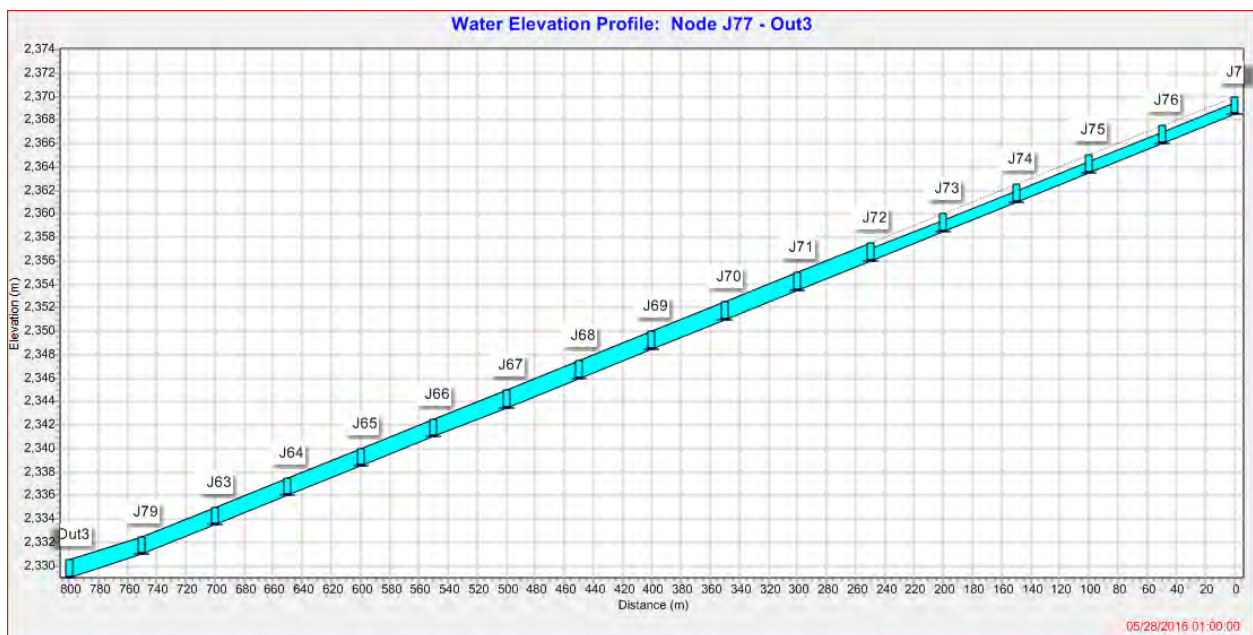


Figure 30: Water flow profile in the modeled area

The water profile plot is obtained for nodes from junction J72 to outlet3 is as shown in figure above. The simulation status report shows that sections between these junctions are surcharged (flooded). The depth of surcharging at junction J63 and at junction J79 is 0.35 m above crest level, whereas for junction J64 and junction J65 is 0.2 m.

### 4.2.2 Simulation results

Different case scenarios have been considered in this study to obtain a fully understanding of the system performance under multiple working conditions. Firstly, the model has been run with the continuous rainfall events with different return periods to analyze the current performance, as shown in Table below.

Table 28: Results of simulations with the model

Days	Hours	Precipitation (mm/hr)	Infiltration (mm/hr)	Runoff (CMS)	Total Inflow (CMS)	Outflow (CMS)
0	00:30:00	0.00	0.00	0.00	0.01	0.00
0	01:00:00	95.00	1.36	0.00	0.01	0.00
0	01:30:00	95.00	2.83	2.79	2.80	2.30
0	02:00:00	0.00	2.13	4.10	4.10	3.90
0	02:30:00	0.00	1.15	1.14	1.15	1.67
0	03:00:00	0.00	0.85	0.54	0.55	0.71
0	03:30:00	0.00	0.72	0.30	0.31	0.40
0	04:00:00	0.00	0.64	0.19	0.20	0.25
0	04:30:00	0.00	0.59	0.13	0.13	0.17
0	05:00:00	0.00	0.56	0.09	0.09	0.12
0	05:30:00	0.00	0.53	0.06	0.07	0.09
0	06:00:00	0.00	0.51	0.04	0.05	0.07
0	06:30:00	0.00	0.48	0.03	0.04	0.05
0	07:00:00	0.00	0.47	0.02	0.03	0.04
0	07:30:00	0.00	0.46	0.02	0.02	0.03
0	08:00:00	0.00	0.41	0.01	0.02	0.03
0	08:30:00	0.00	0.31	0.01	0.02	0.02
0	09:00:00	0.00	0.30	0.01	0.01	0.02
0	09:30:00	0.00	0.30	0.01	0.01	0.02
0	10:00:00	0.00	0.26	0.00	0.01	0.01
0	10:30:00	0.00	0.14	0.00	0.01	0.01
0	11:00:00	0.00	0.12	0.00	0.01	0.01
0	11:30:00	0.00	0.12	0.00	0.01	0.01
0	12:00:00	0.00	0.07	0.00	0.01	0.01

Secondly, different local improvements in the system nearby the properties which have been flooded in previous years were modelled and analyzed. Table below shows a summary of the simulations carried out, although further information is included in appendices and next sub chapters.

Table 29: Sub catchments runoff

Hours	S1 Runoff (CMS)	S2 Runoff (CMS)	S3 Runoff (CMS)	S4 Runoff (CMS)
1:00:00	0	0	0	0
2:00:00	0.24	0.23	0.24	0.21
3:00:00	0.04	0.04	0.04	0.02
4:00:00	0.01	0.01	0.01	0.01
5:00:00	0.01	0.01	0.01	0
6:00:00	0	0	0	0

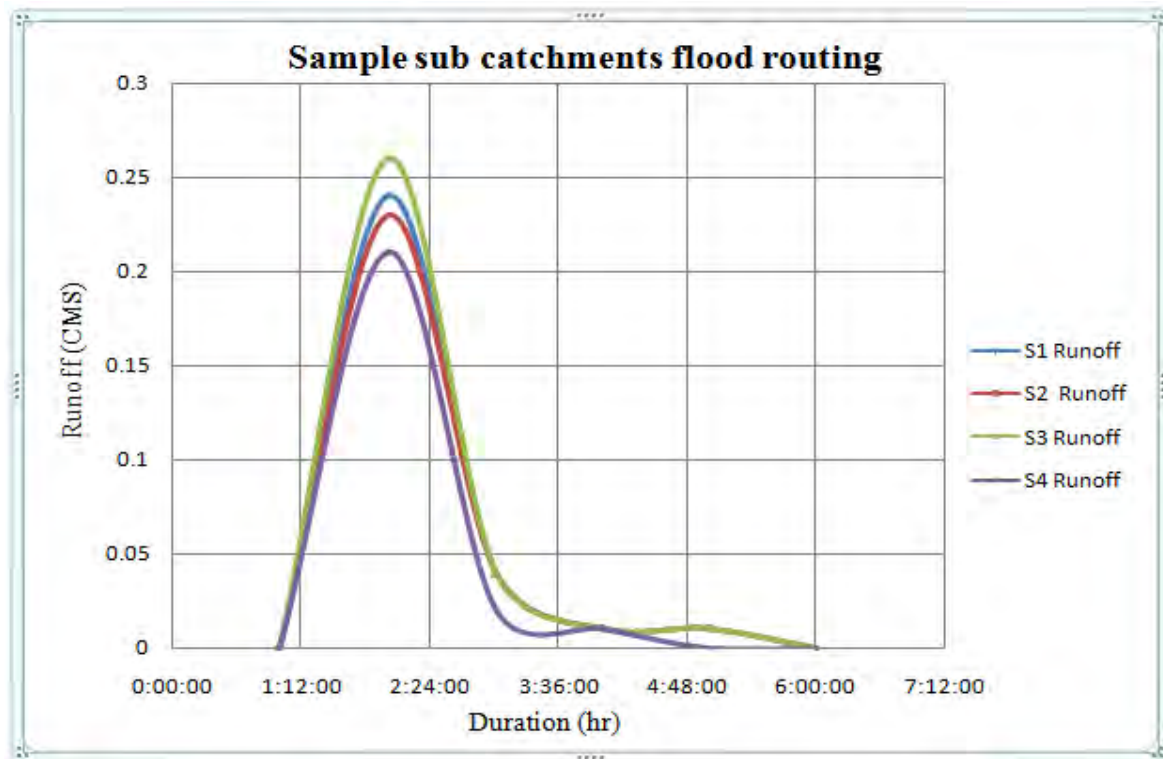


Figure 31: Runoff from some sub catchments

By running the hydrological model with the rainfall data presented in Chapter three, the runoff generation within the area was obtained. This runoff generation was then used as an input for the network simulation. The results obtained in both simulations are presented in the following subchapters. It is important to point out that a greater effort was put in the 5- and 10-year return period rainfall analysis since they are the modeling rains included in the ERA.

#### 4.2.3 Runoff Simulation

The total amount of runoff generated during the different rainfall events is included in Table 38 and Table 39. As expected, the continuous rainfalls with variable intensity produce more runoff than the block rains with constant intensity due to the duration of the rainfall data.

Table 30: Total runoff generation for continuous rainfalls with different return periods

Rain event	T5	T10	T25	T50	T100
Total runoff (mm)	65.52	74.45	85.7	94.07	102.45

Table 31: Total runoff generation for block rainfalls with different return periods

Rain event	Block T5					Block T10				
Duration (min)	7	10	15	20	30	7	10	15	20	30
Runoff (mm/hr)	95	90	80	74	60	108	102	95	82	70

Moreover, the total runoff is distributed within the area according to their parameters. That shows the accumulated discharge of each sub-catchment for a T5 and T10 rainfalls. The block rains produces a similar distribution of runoff and the difference just lies in the values. By comparing both maps it is noticeable that the total discharge is considerably greater in the T10 scenario. The sub-catchments with higher runoff values are located in the north-eastern and central part of the area, along with a group of 14 sub-catchments in the western part.

In addition, the condition illustrates the influence of both the area and the imperviousness of the sub-catchments in the hydrological model.

Thus, the majority of the sub-catchments with higher runoff discharge correspond to those with big drainage area, although there are some of them with low runoff rate due to the

imperviousness. Some values of imperviousness are included inside the sub-catchment to illustrate it.

In addition, it is easy to recognize green areas that are represented in the model with small-medium size sub-catchments since the runoff generation is very low. This has its explanation in the low imperviousness in combination with the small-medium area.

In order to evaluate the pattern of the runoff discharge, the water generated within one randomly chosen sub-catchment during a 5-year rain and a 10-year rain was plotted. Figure 31 shows the discharge of the sub-catchment number 11, which has 1.0 ha of area and 38.7 of imperviousness coefficient.

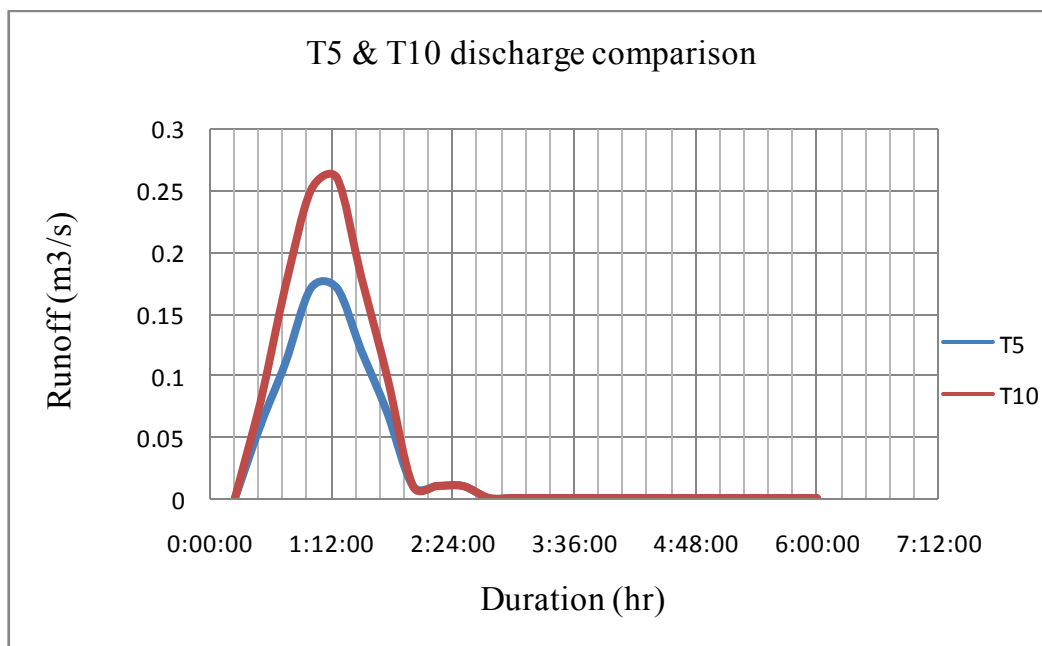


Figure 32: Discharge of the sub-catchment S11.

Both runoff discharges follow the same distribution with the singularity that the T5 discharge is slightly lower and shorter in time. Indeed, the peak discharge is 0.08 m<sup>3</sup>/s greater during the T10 rain. This means that the main difference between the two rain events is just the peak discharge value during a limited period of time (approximately 1 hour). This trend is also found with all the other continuous rainfall.

#### 4.2.4 Network Simulation

The ERA claims that pipes in a drainage system must not work under pressure for all selected return period rainfall event. Moreover, drainage systems should be modeled to cope with a 10-years return period rainfall in terms of water level below the surface in our systems.

This implies that the flooding risk must be verified in the nodes (manholes) in the systems, whereas water level at each lateral connection must be checked independently with a longitudinal profile in parts of the system which are connected. Nevertheless, in order to facilitate the analysis of the flooding risk in the whole area it was assumed that the maximum pipe flooding (water level minus ground level) is representative for all the lateral pipes connected to the link under consideration. The basement levels were assumed to be 1.5 meters below the surface, so that the pipes and the junctions are classified in three regions: water level above surface (link flooding greater than zero), risky area (link flooding varies between minus one and a half to zero meters) and safe area (link flooding smaller than minus one and a half meters).

The results obtained in the simulations are gathered in Table 40. It contains the classification of nodes and pipes regarding node flooding, pipe flooding and pipe filling.

Table 32: Classification of nodes and pipes flooding and pipe filling.

	depth interval	T5	T10	T20	T50	T100
Nodes Flooding	< -1.5m	60.0%	52.0%	34.0%	19.0%	3.0%
	-1.5-0	25.5%	30.7%	31.0%	28.0%	2.0%
	> 0m	14.5%	17.3%	35.0%	53.0%	95.0%
Pipe Flooding	< -1.5m	51.0%	36.0%	28.0%	14.0%	10.0%
	-1.5-0	28.0%	33.0%	28.0%	22.0%	18.0%
	> 0m	21.0%	31.0%	44.0%	64.0%	72.0%
Pipe Filling	< 90%	30.0%	21.0%	12.5%	4.5%	3.7%
	90% - 100%	3.0%	2.5%	2.5%	1.0%	0.6%
	> 100%	67.0%	77.5%	85.0%	94.5%	95.7%

According to the simulations, all of pipes have worked under full flow during the T10 rain, whereas others are flow two third of the pipe during the T5 rain. These numbers represent 66.7% and 62.5% of all pipes, respectively. In addition, J63 and J79 nodes are flooded in the T10 and in the T5 rain respectively, which involves 32.6% of the nodes in the case of T10 and

19.3% of the junctions in the T5 rain. Therefore, the rains with continuous hydrographs are the worst case scenario since the amount of junctions and pipes with water level below 1.5 meters, i.e. water level below basement level, is smaller than in the block rains.

Moreover, link flooding follows also this trend. Figure 32 shows the comparison of the depth of pipes with water level of T5 and T10 in the modeled area of importance points above the surface, from surface to 1.5 meters below and lower than 1.5 meters from the surface.

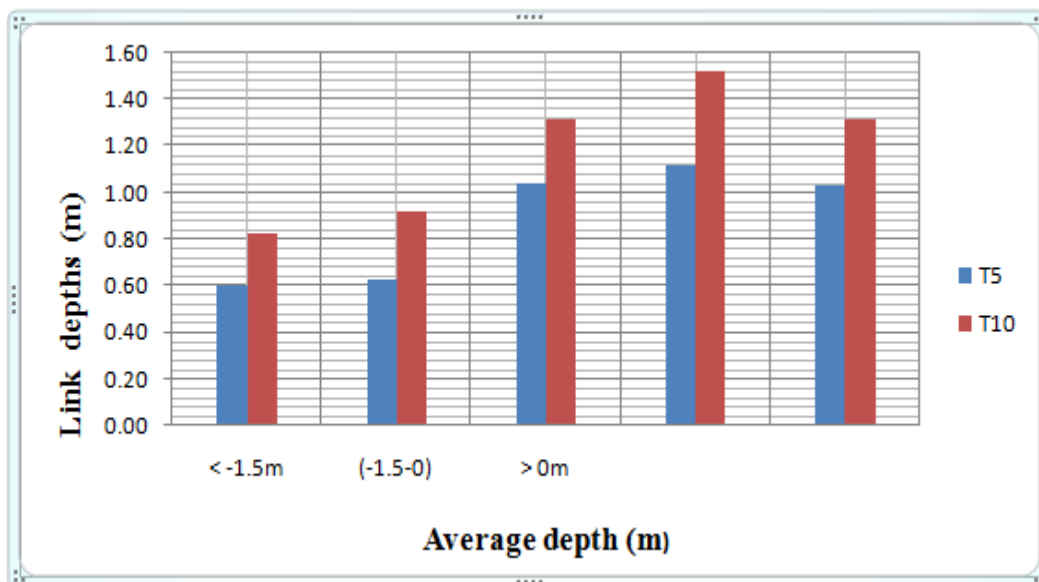


Figure 33: Depth of links with water level in five conduits/links of the modeled area

Table 33: Conduits or links flooded

Links	Link C7	Link C8	Link C9	Link C10	Link C11
T5	0.60	0.63	1.04	1.12	1.03
T10	0.83	0.92	1.32	1.52	1.32

In the case of the T5 rain, the percentage of links with water level below 1.5 meters from the surface is slightly lower than those with the water approaching the surface. However, in the case of the T10 rain the distribution is barely the same, but the percentage of pipes classified as risky or flooded is greater than in the T5 rain. Indeed, 64% of the pipes have the water level line either above the surface or 1.5 meters below it (i.e., the water level line is higher than the lateral connection to the contributing catchment). In the same way the pipes with water level higher than 1.5 meters below the surface during a T10 rain are also spread among the network.

However, in this case there is a higher concentration of flooded links within the modeled area of the network.

#### 4.2.5 Model calibration and validation

Calibration: - an iterative procedure of parameters evaluation and adjusting sensitive parameter by comparing calculated and simulated.

Calibration parameters for SWMM hydrology and hydraulic Parameters					
Name of parameter	Meaning		Value range	Initial values	Used values/ Sensitivity to peak flow
N-Imperv	Manning's roughness coefficient for impervious area		0.011–0.015	0.012	0.013
N-Perv	Manning's roughness coefficient for pervious area		0.05–0.8	0.13	0.8
Destore-Imperv	Depth of depression storage on impervious area		0–3	1	1.5
Destore-Perv	Depth of depression storage on pervious area		3–10	6	9
Conduit Roughness	Manning's roughness coefficient for conduit		0.011–0.024	0.014	0.016
Infiltration method	Green Ampt	Suction head	3.5	3.5	3.5
		Conductivity	0.5	0.5	0.5
		Initial deficit	0.25-0.26	0.25	0.26

#### 4.2.6 Model performance evaluation criteria

- Coefficient of Determination ( $R^2$ )

$$R^2 = \left( \frac{\sum_{t=1}^n (q_t^{obs} - \bar{q}_t^{obs})(q_t^{sim} - \bar{q}_t^{sim})}{\sqrt{\sum_{t=1}^n (q_t^{obs} - \bar{q}_t^{obs})^2} \sqrt{\sum_{t=1}^n (q_t^{sim} - \bar{q}_t^{sim})^2}} \right)^2$$

- The Nash-Sutcliffe coefficient (RNS)

$$R_{NS} = 1 - \frac{\sum_{t=1}^n (q_t^{obs} - q_t^{sim})^2}{\sum_{t=1}^n (q_t^{obs} - \bar{q}_t^{obs})^2}$$

- Relative Error (RE)

$$RE = \frac{\sum_{t=1}^n |q_t^{obs} - q_t^{sim}|}{\sum_{t=1}^n q_t^{obs}}$$

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

#### 4.2.7 Acceptable level of calibration

$R_{NS}$  Between 0 and 1 indicates acceptable models

< 0 indicates poor models

= 1 perfect models

= 0 Model is no better than using as an estimator

RE < 30%

$R^2$  Approach to one and its shown on the graph.

#### Flow calibration and validation

Sub catchments label	Rational method 10years runoff (CMS)	EP SWMM5 10 years runoff (CMS)	Differences
S1	0.20	0.24	-0.04
S2	0.19	0.23	-0.04
S3	0.21	0.24	-0.03
S4	0.16	0.18	-0.02
S5	0.29	0.31	-0.02
S6	0.27	0.30	-0.03
S7	0.27	0.28	-0.01
S8	0.19	0.20	-0.01
S9	0.19	0.20	-0.01
S10	0.24	0.25	-0.01
S11	0.18	0.21	-0.03
S12	0.25	0.27	-0.02
S13	0.23	0.25	-0.02
S14	0.27	0.29	-0.02
Total, Q	3.14	3.45	-0.31

The calibration and validation of the hydrological parameters gave excellent results. The model provided good simulation of both the total and peak flows.  $R_{NS}$  values of 0.622 were deemed acceptable, and the RE values of the calculated outflow were less than 30% of the simulated outflow, which shows that the simulated curves were a good fit for the simulated curves. The simulated and calculated values for runoff were correlated, and the  $R^2$  values of 0.935 were deemed acceptable. The calibration and verification results indicated that the model structure and parameters matched the runoff-producing pattern and that the calibrated model was suitable for simulating storm runoff in the study area.

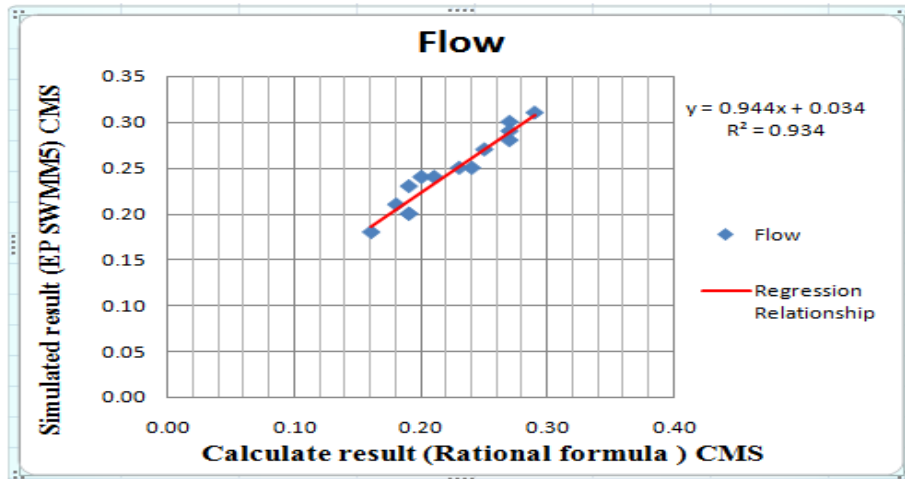


Figure 34: Correlated plot of calculated and simulated Flow

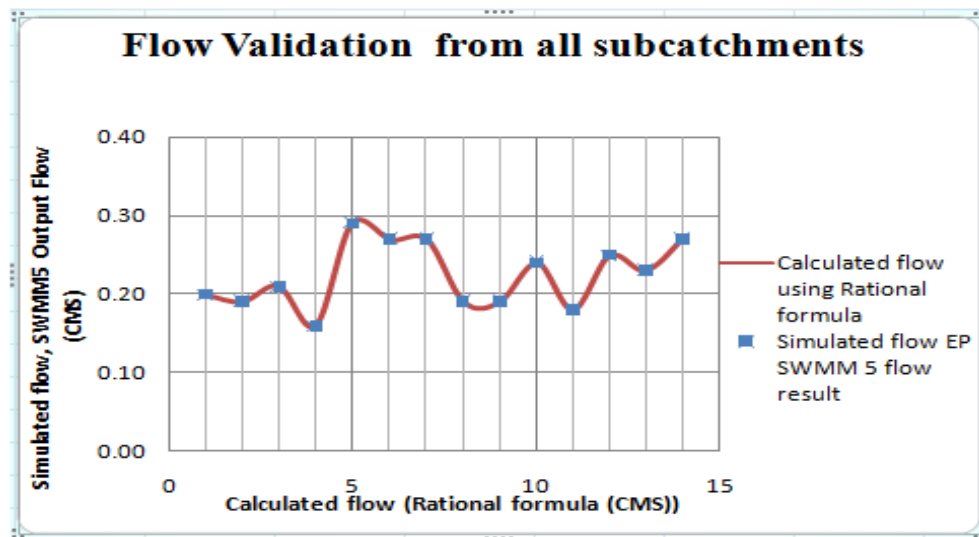


Figure 35: Flow validation from all sub-catchments

The maximum values obtained from model study and rational methods are given in Table35 below. These values are plotted in graphical format for comparison purpose and are shown in Figure27 shown below.

Table 34: Runoff values obtained using both methods

Sub catchments label	EP SWMM5 10 years runoff (CMS)	Rational method 10years runoff (CMS)	Differences
S1	0.24	0.2	-0.04
S2	0.23	0.19	-0.04
S3	0.25	0.21	-0.04
S4	0.18	0.16	-0.02
S5	0.31	0.29	-0.02
S6	0.3	0.27	-0.03
S7	0.28	0.27	-0.01
S8	0.2	0.19	-0.01
S9	0.2	0.19	-0.01
S10	0.25	0.24	-0.01
S11	0.21	0.18	-0.03
S12	0.28	0.25	-0.03
S13	0.25	0.23	-0.02
S14	0.29	0.27	-0.02
Total, Q	3.47	3.14	-0.33

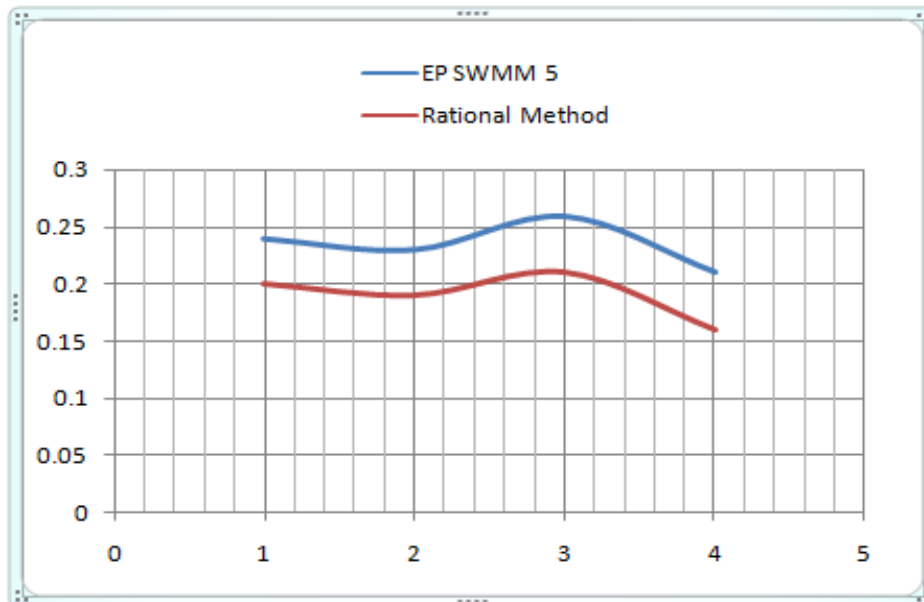


Figure 36: Graphical plot for runoff obtained using SWMM and rational method.

The US EPA SWMM 5 model and GIS were applied to an urban catchment in Bole sub city and rational method is used for comparison. Calculations are presented, the total runoff from whole sub-catchments by SWMM is 3.47m<sup>3</sup>/sec, whereas by rational method is 3.14m<sup>3</sup>/sec.

The runoff obtained in the simulation is then used as input data at each node connected to a catchment. The same study was done in India and the total runoff from seven sub catchments by SWMM is 2.177m<sup>3</sup>/sec, whereas by Rational Method is 1.109m<sup>3</sup>/sec. (M. L. Waikar\* and Undegaonkar Namita U, January, 2015) Urban Flood Modeling by using EPA SWMM 5 SRTM University's Research Journal of Science April 2015 /Spl. Vol., no 1; (ISSN : 2277 - 8594 Print)

#### **4.5 Storm Water Management Plan**

The general goal of the management plan is to reduce the amount of water entering into the system, as well as to improve the current network performance. Environmental-friendly practices are the major solutions to offset the impacts caused by urbanization by returning to nature developed spaces (Durrans & Haestad Methods, 2003). Therefore, the study was firstly focused on the implementation of sustainable drainage systems to avoid the overloading of the network. Nevertheless, other approaches based on conventional infrastructural modifications were also considered due to the critical situation of this area.

##### **4.5.1 Management plan considerations**

Several ideas should be kept in mind in order to modeling suitable solutions to avoid flooding, as well as to adjust them to the specific area. Some of these factors are presented below (Durrans & Haestad Methods, 2003):

- ✓ Ground conditions: Clay and bedrock are the main materials under the soil. The high imperviousness of these materials hinders the infiltration of water into depth ground.
- ✓ Spaces: the location of the solution should consider the topographic conditions (i.e. slopes and lands height, among others), as well as free surface space circumstances due to the high concentration of urbanized areas.
- ✓ Aesthetic: it is necessary to avoid negative aspects as eutrophication, the possibility of odours, mosquitoes and so forth since the actions will be located in an urban area.

##### **4.5.2 Description of potential solutions**

In the following sub-chapters a brief description of the considered solutions is presented.

### **4.5.3 Infrastructural actions**

Of importance in the modeling of storm water storage facilities is to establish the level that the storm water can reach. Figure below represents a basic hydrograph used to determine the required storage volume in order to modeling an optimal reservoir. The area under the hydrographs is the volume of runoff in a storm event. The required storage volume is the minimum capacity that the reservoir should have to be able to handle the rain represented in the hydrograph.

### **4.5.4 Enlargement of pipe diameters and new pipeline connections**

The enlargement of diameters is considered as a possible solution when simulations prove enough capacity at downstream pipes, whereas upstream pipes cannot handle a certain rain. In such case, the line is undersized and an enlargement of the pipes diameter is the best option. Colebrook diagram allows, by means of the flow discharge, the preliminary estimation of the modeling diameter. This first approximation of the required pipe capacity is then verified with the simulations in SWMM5.

### **4.5.5 Sustainable urban drainage systems (SUDS's)**

SUDS's are strategies used to improve sustainable urban development and performance. The aim of the Sustainable Urban Drainage Systems is the management of the storm water from an environmental-friendly point of view. The major modeling aim of these systems is to maximize runoff volume reduction and nutrient removal. Moreover, the reduction of water entering in the combined system avoids over flows and improves water quality by infiltration and sedimentation of pollutants (Pettersson, 2011).

### **4.5.6 On-site management: Green roofs and permeable artificial pavements**

Green roofs and permeable artificial pavements are on-site approaches for storm water management. Their aim is to mitigate the peak flow from each sub-catchment individually, i.e. the peak flow reduction for the whole watershed is neglected.

Green roofs capture precipitation, evapotranspire a small amount of water taken up by plants and temporally store the runoff before water flows into storm drain systems (EPA, 2003).

Vegetated roofs consist on a layered system of roofing, as can be seen in Figure below. The layered modeling is required to maintain the vigorous cover which captures and stores water, as well as to avoid the water to enter in the building. Of importance is the building structural capacity since the weight of the additional water retained in the roof has to be supported by the structure.

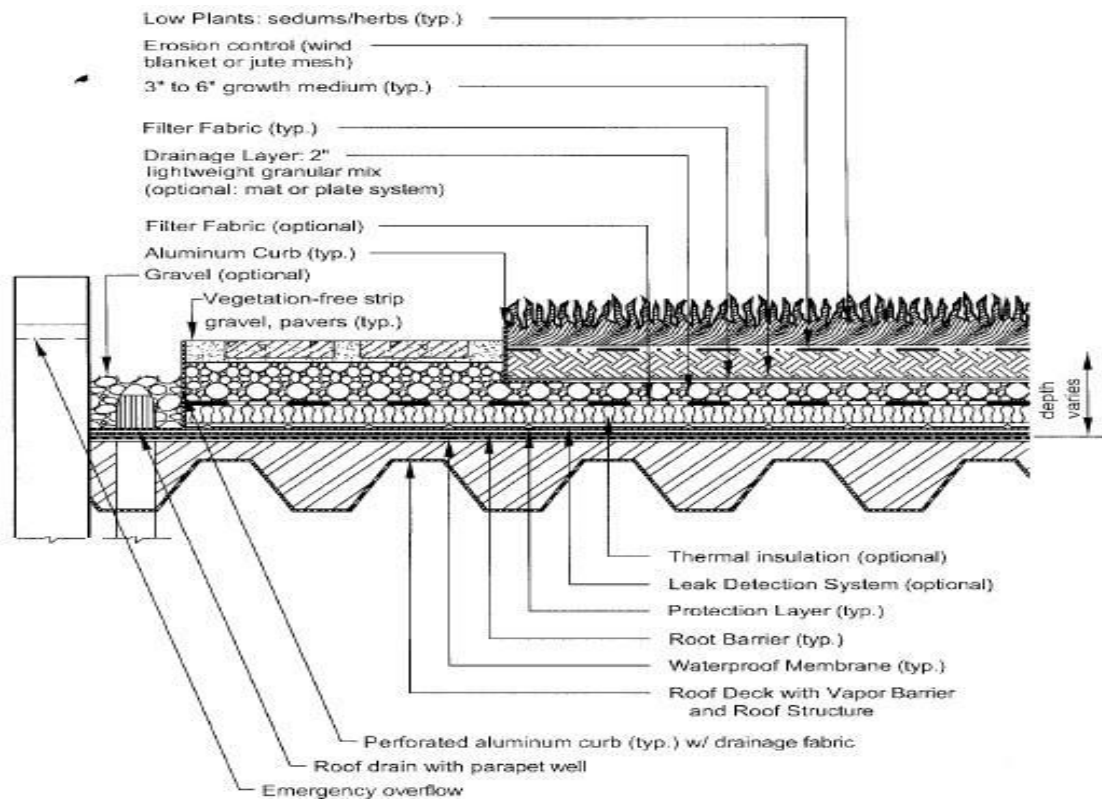


Figure 37: Cross section of green roof layers deployment  
(Source: Chesapeake Storm water Network, 2011).

Local climate and modeling objectives govern plant selection in order to match the plant species to the appropriate plant hardiness zone. In this case study, it was assumed that the plant to be used in the green roofs is sedum. According to a study performed inland (Sweden), the runoff generation can be reduced up to a 65% during rains with variable intensity if sedum green roofs are implemented (Villarreal, 2007). However, this effectiveness depends on multiple factors, such as rain characteristics. Therefore, an imperviousness value of 0.5 was assumed for the current investigation.

#### 4.5.7 Retention and infiltration management: Grass ditch

Grass ditches and filter strips are vegetated features modeling to retain the flow and remove pollutants dragged by surface runoff. The process is developed in the following steps: filtration, infiltration, absorption and biological uptake (Chesapeake Storm water Network, 2011). The precipitation is retained and conveyed by the filter strips to the grass ditch, where flow velocity is reduced, so that pollutant sedimentation is allowed, see Figure below.

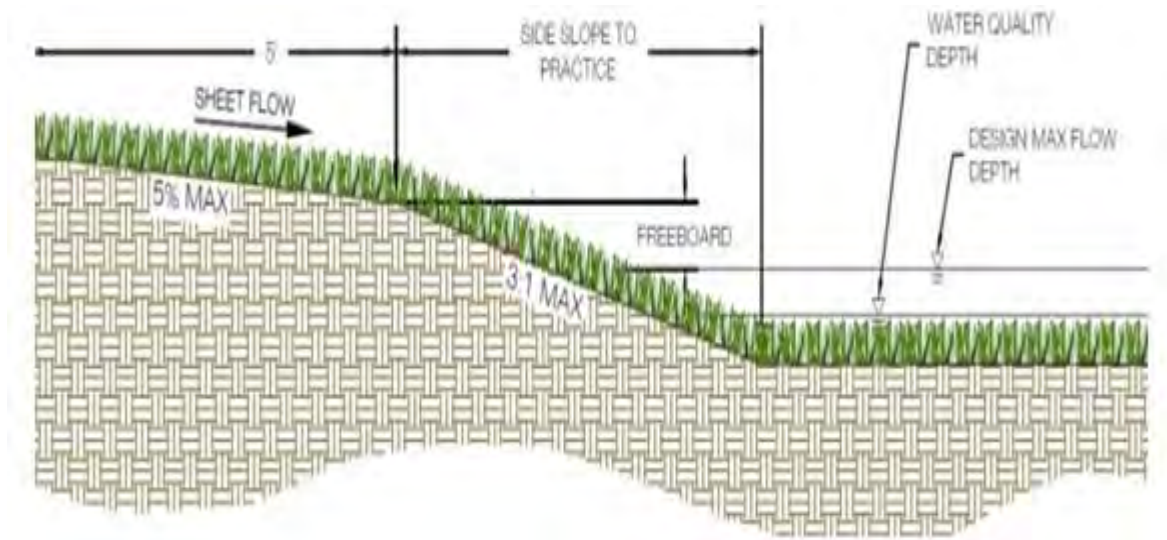


Figure 38: Grass filter for sheet flow and grass ditch (Chesapeake Storm water Network, 2011).

Grass ditches are shallow channels located close to roads, parking lots or foothills. Its performance provides filtration and attenuation of a modest amount of runoff, i.e. detaining and reducing the inflow and the pollutant concentration at the network inputs (Chesapeake Storm water Network, 2011). Filter strips are sloped surfaces leading the runoff into the ditch by a surface covered by vegetation ensuring that water spends enough time to reduce the discharge and allow the infiltration (Wilson et al. 2009). Figure above shows a grass ditch layout.

#### 4.5.8 Detention management: basin and gravel ponds

Basins and ponds are open areas with or without permanent water, where water may be stored in case of heavy rainfall in order to manage the excess of runoff volume.

The temporary storage allows pollutants and sediments to settle on the bottom of the pond providing environmental benefits (Wilson et al., 2009).

The operation is based on the collection of water from the surface runoff or from a storm water concrete pipe, which discharge runoff into the outfall. The outflow is then evacuated in the same way: by a pipe or by a controlled surface discharge. Depending on the permeability soil features some water will infiltrate into the ground decreasing the flow discharge (CIRIA, 2012).

#### **4.5.9 Evaluation of potential solutions**

Based on the results of the simulations in the base system, different modifications were included in the model to improve its capacity and efficiency. As the worst case scenario corresponds to a continuous intensive rain, the actions required in the system were modeling to cope with this rain, although the capacity was also tested for the rains. Moreover, it is important to point out that the continuous intensive rainfall used in this study corresponds to a one-day rainfall, whose peak discharge occurs.

##### **Local measures**

Local measures refer to specific system modifications whose aim is to reduce the water level in the pipes where properties with previous flooding are connected to, as well as in the surrounding ones. Thus, pipe filling values over one means that the flow is running under gravity and negative values of the difference between water level and drainage height level indicates basements under flooding risk.

## 5. CONCLUSION AND RECOMMENDATION

### 5.1 Conclusion

- ✓ Continuous rains have shown to be the worst case rainfall scenario in this drainage system. Indeed, the duration of the rainfall is not as influent as the intensity when it comes to flooding risk, i.e. heavy instantaneous rains are more dangerous than medium-intensity long rains.
- ✓ With 67% of concrete pipes running two third of pipe during 5-year return period rain and a 32.6% of nodes flooded during a 10-year return period rain it can be affirmed that the drainage system situation is critical. The total runoff from whole catchments by SWMM is 3.47m<sup>3</sup>/sec, whereas by rational method is 3.14m<sup>3</sup>/sec. The total runoff from seven sub catchments by SWMM is 2.177m<sup>3</sup>/sec, whereas by Rational Method is 1.109m<sup>3</sup>/sec. (M. L. Waikar\* and Undegaonkar Namita U, January, 2015)
- ✓ The flooding risk is very high due to the drainage system is undersized to cope with the current rainfall rates, but also is very limited to face the upcoming predicted rainfall. The most affected areas are the main line and ring road drainage system, although other singular spots are also likely to be affected by flooding.
- ✓ Sustainable drainage systems are suitable options to reduce the peak flows in the system, but they are not enough to avoid flooding in this area. Hence, it is necessary to combine environmentally friendly measures with new infrastructure to increase the system capacity.
- ✓ Mainly replacements of pipes are a suitable approach to solve local flooding problems. However, this kind of system modifications should be carefully replaced since it could move the problem to other parts of the system instead of solving the flooding risk.
- ✓ In this study, the SWMM model was successfully used to model the quantity of runoff in an urbanized area of Bole sub-city.
- ✓ Data from rainfall events were used to calibrate and validate the model. Sensitivity analysis showed that the parameters that had most influence on the hydrology and hydraulic parts were destore impervious and conduit roughness. The road and roof quality parameters were significantly more sensitive than the green field parameters.

- ✓ The road exponent was the most sensitive parameter for estimating the total and peak runoff. The calibration and verification results indicate that the model structure and parameters fitted the runoff producing pattern. The total simulation accuracy of the runoff and network systems as assessed by statistical methods, were  $R_{NS} = 0.622$ ,  $RE = 0.098$ , and  $R^2 = 0.935$ .
- ✓ Pavement drainage requires consideration of surface drainage, gutter flow, and inlet capacity. The modeling of a drainage system for a curbed highway pavement section is to collect runoff in the gutter (which carries runoff generated from pavement surface and from the adjacent areas contributing to the pavement at a permissible level) and convey it to pavement inlets in a manner that provides reasonable safety for traffic and pedestrians at a reasonable cost.
- ✓ Depth duration frequency curve and intensity duration frequency curve for the study area were constructed and their value of IDF is less than the recommended value by Ethiopian road authority drainage design manual.
- ✓ Generally it can be concluded that road surface drainage of the study area found to be inadequate due insufficient road profile, insufficient drainage structures provision, improper maintenance and lack of proper interconnection between the road and drainage infra structures thereby resulting damages to road surfacing material and flooding problems in the area.

## 5.2 Recommendation

- ✓ Pipe diameters should be completely updated according to new ERA hydrology manual. The system includes some pipes whose dimensions do not correspond to the drainage system model, so that a review of the system model is recommended to increase the model reliability.
- ✓ Since the main line has shown to be severely overloaded, it is recommended to study the feasibility of increasing its capacity either by duplicating the network or replacing the current pipes.
- ✓ Evaluation of a grass ditch network in the whole area connected to main line in order to reduce the runoff to be conveyed by the drainage system, as well as to lower the cost of pipe by means of decreasing diameter. The performance of the grass ditch network should be optimized to collect as much water as possible from more impervious areas.
- ✓ Study local system improvements for critical areas in Bole sub city previously mentioned. According to the obtained results for similar areas, the problem in these areas should be the local insufficient capacity, so that pipe enlargement stands as a feasible solution that should be investigated.
- ✓ Proper road geometry need to be maintained to provide required crown and proper side drain drainage structures need to be provided for roads without drainage structures.
- ✓ Provision of proper connections or integrations between the road network and drainage network systems is required with regular maintenance.
- ✓ Developing the skill of SWMM soft ware for planning, analysis and modeling of storm water runoff and drainage systems in urban areas and monitoring the infrastructures.
- ✓ Use of the research study results for further study of other sub catchment of the Addis Ababa city in order to have a standardized and harmonized urban drainage systems.
- ✓ Existing drainage system lay on feeder roads of the new stadium bridge approach road way not functioning properly at this time. Hence, it is advisable to undertake a comprehensive clearing and maintenance work on the existing storm drain system along with the approach road storm drain system development otherwise there may be a danger of traffic interruption due to concentrated flow condition on the roadway.

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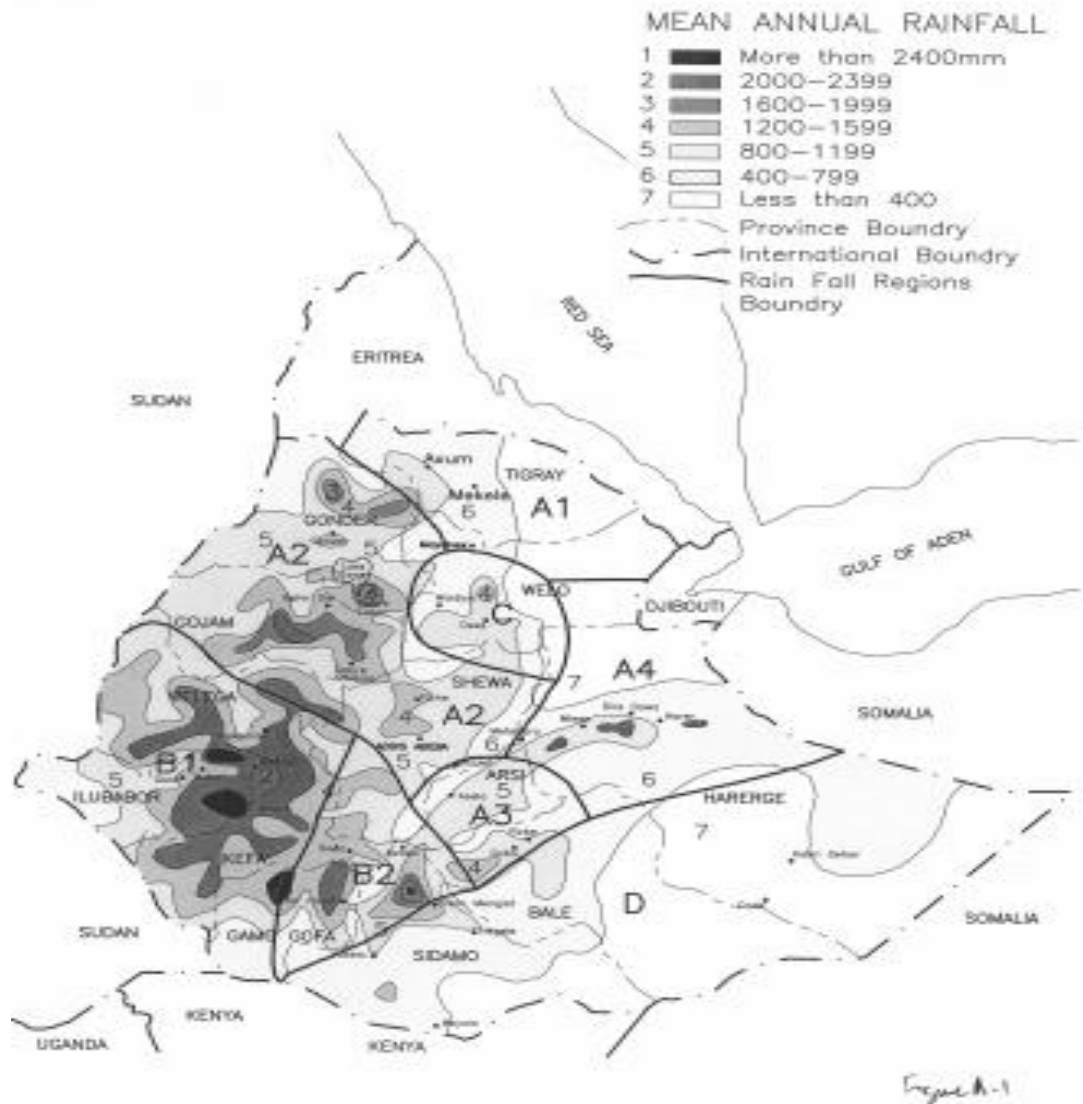
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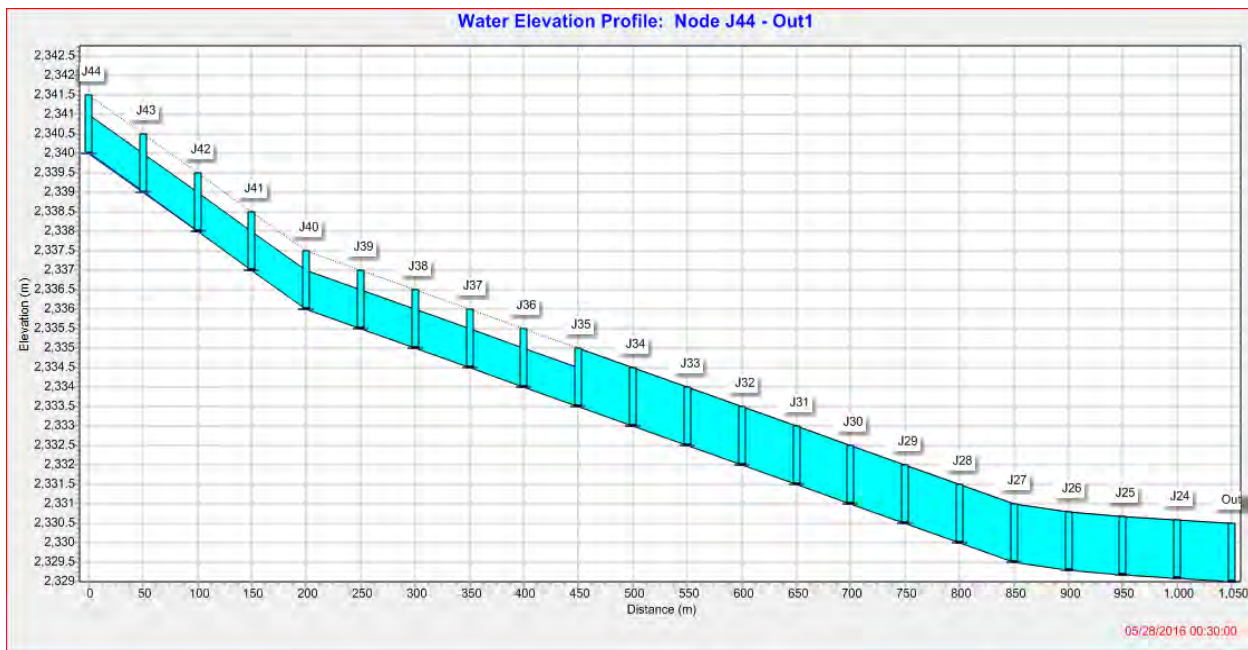
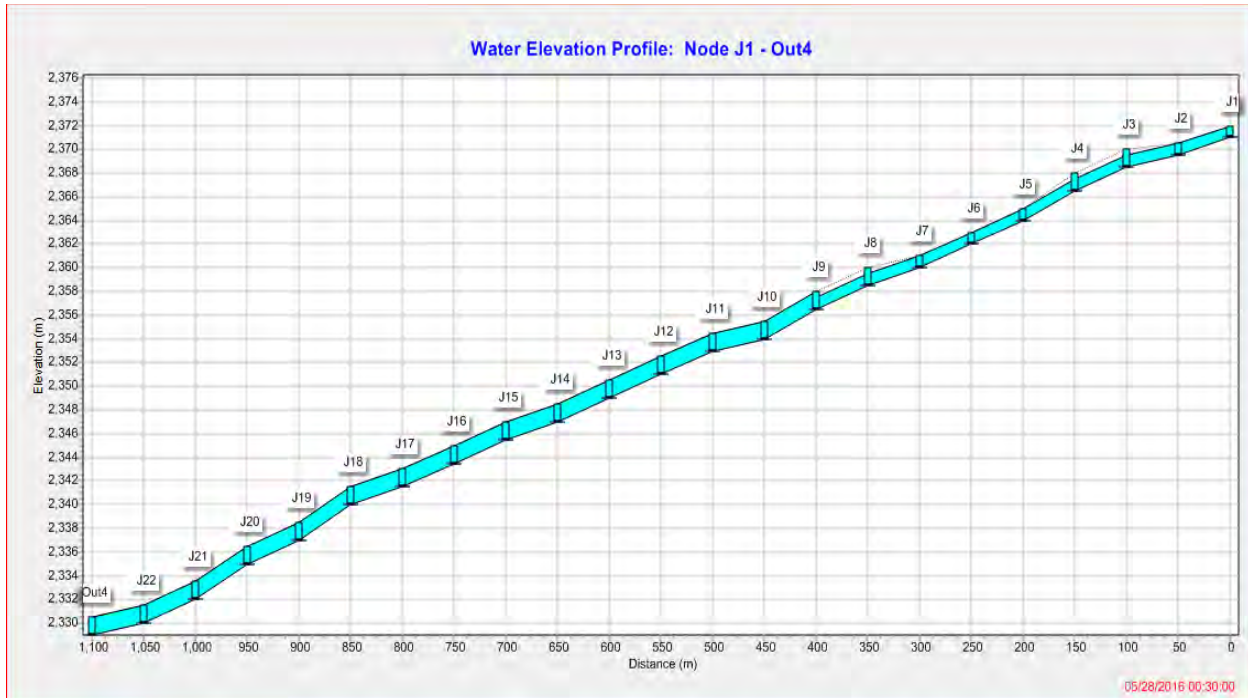
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## APPENDICES

## Appendix 1: Mean Annual Rainfall for Ethiopia (Ethiopian Roads Authority)



## Appendix 2: Water flow from starting J1-out4 and J44-out1



## Appendix 3: Intensity Duration Frequency Curve Development for Bole Sub-city

Intensity Duration Frequency Curve Development for Bole Sub-city												
Duration(mts)	Duration (hr)	Rainfall Ratio	Depth for given Return periods(mm)					Intensity for given return periods (mm/hr)				
			5 years	10 years	25 years	50 years	100 years	5 years	10 years	25 years	50 years	100 years
5	0.08	0.145	9.500	10.795	12.427	13.640	14.855	114.005	129.543	149.118	163.682	178.263
10	0.17	0.243	15.921	18.091	20.825	22.859	24.895	95.528	108.548	124.951	137.154	149.372
20	0.33	0.370	24.242	27.547	31.709	34.806	37.907	72.727	82.640	95.127	104.418	113.720
30	0.50	0.449	29.418	33.428	38.479	42.237	46.000	58.837	66.856	76.959	84.475	92.000
60	1.00	0.581	38.067	43.255	49.792	54.655	59.523	38.067	43.255	49.792	54.655	59.523
90	1.50	0.650	42.588	48.393	55.705	61.146	66.593	28.392	32.262	37.137	40.764	44.395
120	2.00	0.695	45.536	51.743	59.562	65.379	71.203	22.768	25.871	29.781	32.689	35.601

## Appendix 4: Coefficient of skewness

**$K_T$  values for Pearson Type III distribution (positive skew)**

Skew coefficient $C_s$ or $C_w$	Return period in years						
	2	5	10	25	50	100	200
	Exceedence probability						
	0.50	0.20	0.10	0.04	0.02	0.01	0.005
3.0	-0.396	0.420	1.180	2.278	3.152	4.051	4.970
2.9	-0.390	0.440	1.195	2.277	3.134	4.013	4.909
2.8	-0.384	0.460	1.210	2.275	3.114	3.973	4.847
2.7	-0.376	0.479	1.224	2.272	3.093	3.932	4.783
2.6	-0.368	0.499	1.238	2.267	3.071	3.889	4.718
2.5	-0.360	0.518	1.250	2.262	3.048	3.845	4.652
2.4	-0.351	0.537	1.262	2.256	3.023	3.800	4.584
2.3	-0.341	0.555	1.274	2.248	2.997	3.753	4.515
2.2	-0.330	0.574	1.284	2.240	2.970	3.705	4.444
2.1	-0.319	0.592	1.294	2.230	2.942	3.656	4.372
2.0	-0.307	0.609	1.302	2.219	2.912	3.605	4.298
1.9	-0.294	0.627	1.310	2.207	2.881	3.553	4.223
1.8	-0.282	0.643	1.318	2.193	2.848	3.499	4.147
1.7	-0.268	0.660	1.324	2.179	2.815	3.444	4.069
1.6	-0.254	0.675	1.329	2.163	2.780	3.388	3.990
1.5	-0.240	0.690	1.333	2.146	2.743	3.330	3.910
1.4	-0.225	0.705	1.337	2.128	2.706	3.271	3.828
1.3	-0.210	0.719	1.339	2.108	2.666	3.211	3.745
1.2	-0.195	0.732	1.340	2.087	2.626	3.149	3.661
1.1	-0.180	0.745	1.341	2.066	2.585	3.087	3.575
1.0	-0.164	0.758	1.340	2.043	2.542	3.022	3.489
0.9	-0.148	0.769	1.339	2.018	2.498	2.957	3.401
0.8	-0.132	0.780	1.336	1.993	2.453	2.891	3.312
0.7	-0.116	0.790	1.333	1.967	2.407	2.824	3.223
0.6	-0.099	0.800	1.328	1.939	2.359	2.755	3.132
0.5	-0.083	0.808	1.323	1.910	2.311	2.686	3.041
0.4	-0.066	0.816	1.317	1.880	2.261	2.615	2.949
0.3	-0.050	0.824	1.309	1.849	2.211	2.544	2.856
0.2	-0.033	0.830	1.301	1.818	2.159	2.472	2.763
0.1	-0.017	0.836	1.292	1.785	2.107	2.400	2.670
0.0	0	0.842	1.282	1.751	2.054	2.326	2.576

(Source: Vent Chow)

## Appendix 5: Model Functionality Comparison

Topic	Item	Model		
		SWMM 5.0	MOUSE / MIKE URBAN	INFOWORKS CS
Hydraulics	Flow Routing	Dynamic Wave	Dynamic Wave	Dynamic Wave
	Routing Engine	Explicit numerical engine can have stability issues if the model not constructed and reviewed carefully.	Implicit numerical engine, a stable and fast hydraulic engine. Though it is considered to be a slightly slower engine than InfoWorks.	Implicit numerical engine. Generally considered the fastest and most stable fully dynamic engine.
	Inlet Control	No	No	Yes
	Detention Storage	Yes	Yes	Yes
	RTC	Yes	Yes	Yes
	Pumps	Yes	Yes	Yes
	Irregular XS	Yes	Yes	Yes
Hydrology	Surface Runoff	Utilizes a non-linear reservoir model to simulate runoff.	Provides a number of surface runoff models, such as a time area method and a Kinematic wave model(Non Linear Reservoir Model). This model behaves exactly the same as the SWMM non-linear reservoir model	Provides a number of surface runoff models, including the SWMM non-linear reservoir model.
	Infiltration	Provides three infiltration options, Curve Number, Horton's Equation and Green Ampt.	In addition to the RDII model (see below) MOUSE utilizes the Horton's Equation or SCS Curve Number to simulate infiltration.	Fixed PR Model (simple percentage) Green Ampt Model Horton Infiltration Model New UK PR Model Wallingford Procedure Model Constant Infiltration Model US SCS Model
	RDII	Provides either unit hydrographs to simulate RDII or a groundwater infiltration module to simulate the influence of groundwater table on infiltration flow.	MOUSE employs a complex RDII model.	Provides either unit hydrographs to simulate RDII or a groundwater infiltration module to simulate the influence of groundwater table on infiltration flow. As per SWMM.
	Continuous Simulation	Yes	Yes	Yes
Water Quality	Pollutant Build Up / Washoff	Yes	Yes	Yes
	Pollutants Modeled	Yes	Yes	Yes
	Treatment	Yes	Yes	Yes
Miscellaneous	LTS - Job List	No	Yes - MOUSE provides a job list file which allows a selected number of events to be run by the HD model.	No
	Statistics	Yes	Yes	Yes
Use Ability	User Interface	Basic user interface.	Good user interface.	Sophisticated user interface.
	Data Management	None	Reasonable data management with the scenario manager.	Excellent data management.
	Result Display	Reasonable	Good	Excellent
	Support	No formal support. A SWMM Users List server, allows subscribers to ask questions and exchange information.	Comprehensive	Comprehensive
Price	Purchase Cost	Free	~\$15k to \$40k dependant on pipe limitation and modules selected.	~\$30k to \$80k dependant on node limitation selected.
	Maintenance Cost	Free	~10% of the purchase price	~15% of the purchase price

(Source: Ashley et al., 2007)

## Appendix 6: Rainfall for all classified Regions

24 hr Rainfall Depth (mm) vs Frequency (yr)								
Return Period Years	2	5	10	25	50	100	200	500
RR-A1	50.30	66.02	76.28	89.13	98.63	108.06	117.48	130.00
RR-A2	51.92	65.52	74.45	85.70	94.07	102.45	110.91	122.27
RR-A3	47.54	59.61	67.66	77.92	85.62	93.34	101.13	111.58
RR-A4	50.39	63.83	72.28	82.55	89.97	97.20	104.32	113.63
RR-B1	58.87	71.26	79.29	89.35	96.84	104.37	112.02	122.41
RR-B2	55.26	69.95	79.68	92.03	101.29	110.61	120.07	132.87
RR-C	56.52	71.04	80.54	92.52	101.48	110.50	119.66	132.06
RR-D	56.23	76.84	90.37	107.46	120.23	133.05	146.00	163.44

**Note: RR- Rainfall Region**

Source: Ethiopian Roads Authority, 2013. Drainage Design Manual

## Appendix 7: Drain pipe model values for Hayat Hospital to bridge

Gutter segment	Gutter segment length (m)	Discharge in a pipe in a system (m <sup>3</sup> /s)	Pipe longitudinal slope	Provided dia, m	Recomded. Pipe dia, m	pipe flow velocity (m/s)
BB1-BB2	29.6	0.028	0.00562	1	1.5	0.19
BB2-BB3	29.6	0.041	0.010204	1	1.5	0.28
BB3-BB4	29.8	0.056	0.01473	1	1.5	0.38
BB4-BB5	29.4	0.073	0.0108	1	1.5	0.51
BB5-BB6	29.8	0.084	1.0108	1	1.5	0.58
BB6-BB7	29.22	0.086	0.00492	1	1.5	0.59
BB7-BB8	29.7	0.093	0.00148	1	1.5	0.64
BB8-BB9	27.9	0.100	1.00148	1	1.5	0.69
BB9-BB10	27.7	0.102	0.0124	1	1.5	0.70
BB10-BB11	27.5	0.106	1.0124	1	1.5	0.73
BB11-BB12	27.48	0.114	0.01302	1	1.5	0.79
BB12-BB13	28.64	0.129	0.01504	1	1.5	0.89
BB13-BB14	28.64	0.146	0.0195	1	1.5	1.01
BB14'-BB15	28.64	0.162	0.0292	1	1.5	1.12
BB14-BB14'	28.64	0.179	0.0389	1	1.5	1.23
BB15-BB14	28.64	0.195	0.0486	1	1.5	1.34
BB15-BB15'	28.65	0.211	0.0556	1	1.5	1.45
BB16-BB15	28.73	0.228	0.0575	1	1.5	1.57
BB17'-BB16	29.6	0.242	0.0555	1	1.5	1.67

## Appendix 8: Drain pipe model values for Emperial Hotel to bridge

Gutter segment	Gutter segment length (m)	Discharge in a pipe system	Pipe longitudinal slope	Provided dia., m	Recommended diam., m	pipe flow velocity (m/s)
EB1-EB2	28.04	0.25	0.026	1	1.5	1.62
EB2-EB3	28.79	0.269	0.016	1	1.5	1.55
EB3-EB4	28.71	0.295	0.011	1	1.5	1.93
EB4-EB5	28.6	0.301	0.01	1	1.5	1.98
EB5-EB6	28	0.308	0.01	1	1.5	1.22
EB6-EB7	28.8	0.312	0.01	1	1.5	1.45
EB7-EB8	28.8	0.319	0.01	1	1.5	1.70
EB8-EB9	28.5	0.325	0.01	1	1.5	1.74
EB9-EB10	28.8	0.33	0.01	1	1.5	1.57
EB10-EB11	28.8	0.336	0.01	1	1.5	1.81
EB11-EB12	28	0.341	0.01	1	1.5	1.55
EB12-EB13	28	0.356	0.006	1	1.5	1.45
EB13-EB14	28.79	0.364	0.005	1	1.5	1.51
EB14-EB15	28.71	0.372	0.004	1	1.5	2.16

## Appendix 9: Drain pipe model values for federal water work construction to outlet

Gutter segment	Gutter segment length (m)	Discharge in a pipe system (m <sup>3</sup> /s)	Pipe longitudinal slope	Provided dia., m	Recommended dia., m	pipe flow velocity (m/s)
FB1-FB2	32.8	0.051	0.00563	1	1.5	1.72
FB2-FB3	32.6	0.096	0.00995	1	1.5	1.85
FB3-FB4	32	0.116	0.0142	1	1.5	2.03
FB4-FB5	30.6	0.134	0.01503	1	1.5	2.08
FB5-FB6	28.9	0.153	0.015	1	1.5	2.12
FB6-FB7	28.69	0.168	0.015	1	1.5	2.15
FB7-FB8	28.62	0.184	0.0165	1	1.5	2.19
FB8-FB9	28.62	0.202	0.015	1	1.5	2.24
FB9-FB10	28.62	0.221	0.0195	1	1.5	2.27
FB10-FB11	28.62	0.238	0.0302	1	1.5	2.31
FB11-FB12	28.62	0.255	0.0409	1	1.5	2.35
FB12-FB13	28.62	0.434	0.0515	1	1.5	2.45
FB13-FB14	28.77	0.452	0.0534	1	1.5	2.51
FB14-FB15	28.8	0.469	0.0548	1	1.5	2.56
FB15-FB16	28.8	0.488	0.0548	1	1.5	1.72
FB16-FB15	28.8	0.507	0.0526	1	1.5	1.85
FB17-FB16	28.8	1.146	0.0524	1	1.5	2.03
FB18-FB17	28.8	1.146	0.0017	1	1.5	2.08
FB19-FB18	28.77	1.158	0.0449	1	1.5	2.12
FB20-FB19	28.77	1.669	0.044	1	1.5	2.15

Appendix 10: Drain pipe model values for Hayat Hospital to bridge

Gutter segment	Gutter segment length (m)	Discharge in a pipe system (m <sup>3</sup> /s)	Pipe longitudinal slope	Provided dia, m	Recomded. Pipe dia, m	pipe flow velocity (m/s)
HB1-HB2	28.93	0.028	0.00562	1	1.5	1.45
HB2-HB3	28.79	0.041	0.010204	1	1.5	1.70
HB3-HB4	28.9	0.056	0.01473	1	1.5	1.74
HB4-HB5	28.6	0.073	0.0108	1	1.5	1.57
HB5-HB6	29.8	0.084	1.0108	1	1.5	1.81
HB6-HB7	28.77	0.086	0.00492	1	1.5	1.55
HB7-HB8	28.77	0.093	0.00148	1	1.5	1.45
HB8-HB9	27.9	0.100	1.00148	1	1.5	1.45
HB9-HB10	29.6	0.102	0.0124	1	1.5	1.60
HB10-HB11	26.87	0.106	1.0124	1	1.5	1.54
HB11-HB12	29.76	0.114	0.01302	1	1.5	1.37
HB12-HB13	28.84	0.129	0.01504	1	1.5	1.41
HB13-HB14	28.82	0.146	0.0195	1	1.5	1.71
HB14-HB15	28.8	0.162	0.0292	1	1.5	1.12
HB15-HB16	28.78	0.179	0.0389	1	1.5	1.23
HB16-HB17	28.76	0.195	0.0486	1	1.5	1.34
HB17-HB18	28.74	0.211	0.0556	1	1.5	1.45
HB18-HB19	26.87	0.228	0.0575	1	1.5	1.57
HB19-HB20	30.76	0.242	0.0555	1	1.5	1.67

## Appendix 11: Imperial Hotel to outlet inlet modeling detail

Gutter segment	Inlet depression cross slope, $S_w$	Flow ratio, $E_o$ for $w/T=0.4$ & $S_w/S_x=3.2$	Inlet Equivalent cross slope, $S_e$	Curb opening length, m		Flow interception efficiency of inlet	Flow intercepted, $Q_{im3/s}$	Flow by pass $(m^3/s)$
				total flow, $L_T$	provided length, $L$			
EB1-EB2	0.08	0.85	0.07175	1.08	1.5	0.7	0.013	0.69
EB2-EB3	0.08	0.85	0.07175	1.05	1.5	0.74	0.019	0.72
EB3-EB4	0.08	0.85	0.07175	1.11	1.5	1	0.026	0.97
EB4-EB5	0.08	0.85	0.07175	1.42	1.5	1	0.007	0.99
EB5-EB6	0.08	0.85	0.07175	1.31	1.5	1	0.007	0.99
EB6-EB7	0.08	0.85	0.07175	1.21	1.5	1	0.004	1.00
EB7-EB8	0.08	0.85	0.07175	1.52	1.5	1	0.006	0.99
EB8-EB9	0.08	0.85	0.07175	1.33	1.5	1	0.006	0.99
EB9-EB10	0.08	0.85	0.07175	1.09	1.5	1	0.005	1.00
EB10-EB11	0.08	0.85	0.07175	1.50	1.5	1	0.006	0.99
EB11-EB12	0.08	0.85	0.07175	1.42	1.5	1	0.005	1.00
EB12-EB13	0.08	0.85	0.07175	1.43	1.5	1	0.005	1.00
EB13-EB14	0.08	0.85	0.07175	1.22	1.5	1	0.0048	1.00
EB14-EB15	0.08	0.85	0.07175	1.07	1.5	1	0.0046	1.00

## Appendix 12: Bole Airport to outlet approach inlet modeling detail

Gutter segment	Inlet depression cross slope, Sw	Flow ratio, Eo for w/T=0.4 & Sw/Sx=3.2	Inlet Equivalent cross slope, Se	Curb Opening length, m		Flow interception efficiency of inlet, E	Flow intercepted, Q <sub>i</sub> (m <sup>3</sup> /s)	Flow by pass(m <sup>3</sup> /s)
				For total flow, L.T	Provided length,			
BB1-BB2	0.08	0.85	0.07175	1.38	1.38	1.1	0.008	1.092
BB2-BB3	0.08	0.85	0.07175	1.38	1.38	1.1	0.008	1.092
BB3-BB4	0.08	0.85	0.07175	1.67	1.5	0.98	0.012	0.968
BB4-BB5	0.08	0.85	0.07175	2.55	1.5	0.8	0.015	0.785
BB5-BB6	0.08	0.85	0.07175	2.78	1.5	0.75	0.018	0.732
BB6-BB7	0.08	0.85	0.07175	2.14	1.5	0.89	0.011	0.879
BB7-BB8	0.08	0.85	0.07175	1.54	1.5	1.0	0.013	0.987
BB8-BB9	0.08	0.85	0.07175	1.5	1.5	1.0	0.007	0.993
BB9-BB10	0.08	0.85	0.07175	1.5	1.5	1.0	0.007	0.993
BB10-BB11	0.08	0.85	0.07175	1.5	1.5	1.0	0.009	0.991
BB11-BB12	0.08	0.85	0.07175	1.5	1.5	1.0	0.007	0.993
BB12-BB13	0.08	0.85	0.07175	1.5	1.5	1.0	0.012	0.988
BB13-BB14	0.08	0.85	0.07175	2.67	1.5	0.77	0.015	0.755
BB14'-BB15	0.08	0.85	0.07175	2.97	1.5	0.72	0.017	0.703
BB14-BB14'	0.08	0.85	0.07175	3.64	1.5	0.62	0.016	0.604
BB15-BB14	0.08	0.85	0.07175	4.04	1.5	0.57	0.017	0.553
BB15-BB15'	0.08	0.85	0.07175	4.77	1.5	0.49	0.016	0.474
BB16-BB15	0.08	0.85	0.07175	5.15	1.5	0.46	0.016	0.444
BB17'-BB16	0.08	0.85	0.07175	5.33	1.5	0.45	0.017	0.433

## Appendix 13: Federal water work construction to outlet inlet modeling detail

Gutter segment	Inlet, depression, cross slope, $S_w$	Flow ratio, $E_o$ for $w/T=0.4$ & $S_w/S_x=3.2$	Inlet Equivalent cross, slope, $S_e$	Curb opening length, m		Flow, interception, efficiency of inlet	Flow intercepted $Q_i$ (m <sup>3</sup> /s)	Flow by pass(m <sup>3</sup> /s)
				For total flow, LT(m)	Provided length, L(m)			
FB1-FB2	0.08	0.85	0.072	4.07	1.5	0.56	0.021	0.54
FB2-FB3	0.08	0.85	0.072	4.56	1.5	0.51	0.045	0.47
FB3-FB4	0.08	0.85	0.072	2.71	1.5	0.77	0.020	0.75
FB4-FB5	0.08	0.85	0.072	2.88	1.5	0.73	0.019	0.71
FB5-FB6	0.08	0.85	0.072	2.92	1.5	0.73	0.019	0.71
FB6-FB7	0.08	0.85	0.072	2.54	1.5	0.80	0.015	0.79
FB7-FB8	0.08	0.85	0.072	3.11	1.5	0.69	0.016	0.67
FB8-FB9	0.08	0.85	0.072	3.24	1.5	0.67	0.018	0.65
FB9-FB10	0.08	0.85	0.072	3.19	1.5	0.68	0.019	0.66
FB10-FB11	0.08	0.85	0.072	3.78	1.5	0.60	0.017	0.58
FB11-FB12	0.08	0.85	0.072	4.12	1.5	0.56	0.017	0.54
FB12-FB13	0.08	0.85	0.072	4.61	1.5	0.51	0.017	0.49
FB13-FB14	0.08	0.85	0.072	4.76	1.5	0.49	0.018	0.47
FB14-FB15	0.08	0.85	0.072	5.00	1.5	0.47	0.018	0.45
FB15-FB16	0.08	0.85	0.072	5.10	1.5	0.47	0.018	0.45
FB16-FB15	0.08	0.85	0.072	4.93	1.5	0.48	0.019	0.46
FB17-FB16	0.08	0.85	0.072	3.97	1.5	0.57	0.015	0.56
FB18-FB17	0.08	0.85	0.068	4.93	1.5	1.00	0.018	0.98
FB19-FB18	0.08	0.85	0.072	3.60	1.5	0.62	0.012	0.61
FB20-FB19	0.08	0.85	0.072	3.66	1.5	0.61	0.016	0.59

## Appendix 14: Hayat Hospital to outlet inlet modeling detail

Gutter segment	inlet depression cross slope, $S_w$	Flow ratio, $E_o$ for $w/T=0.4$ & $S_w/S_x=3.2$	Inlet Equivalent cross slope, $S_e$	Curb opening length, m		Flow interception efficiency of inlet, $E$	Flow intercepted, $Q_i$ (m <sup>3</sup> /s)	Flow by pass (m <sup>3</sup> /s)
				For total flow, $L_T$	provided length, $L$			
HB3-HB4	0.08	0.85	0.072	2.790	1.500	0.750	0.015	0.74
HB4-HB5	0.08	0.85	0.072	3.040	1.500	0.710	0.017	0.69
HB5-HB6	0.08	0.85	0.072	1.500	1.500	1.000	0.024	0.98
HB6-HB7	0.08	0.85	0.072	1.153	1.500	1.070	0.028	1.04
HB7-HB8	0.08	0.85	0.072	2.790	1.500	1.195	0.032	1.16
HB8-HB9	0.08	0.85	0.072	3.040	1.500	0.750	0.037	0.71
HB9-HB10	0.08	0.85	0.072	1.500	1.500	0.710	0.041	0.67
HB10-HB11	0.08	0.85	0.072	1.153	1.500	1.000	0.046	0.95
HB11-HB12	0.08	0.85	0.072	2.790	1.500	1.070	0.050	1.02
HB12-HB13	0.08	0.85	0.072	3.040	1.500	1.195	0.055	1.14
HB13-HB14	0.08	0.85	0.072	1.500	1.500	0.750	0.059	0.69
HB14-HB15	0.08	0.85	0.072	1.153	1.500	0.710	0.064	0.65
HB15-HB16	0.08	0.85	0.072	2.790	1.500	1.000	0.068	0.93
HB16-HB17	0.08	0.85	0.072	3.040	1.500	1.070	0.015	1.06
HB17-HB18	0.08	0.85	0.072	1.500	1.500	1.195	0.017	1.18
HB18-HB19	0.08	0.85	0.072	1.153	1.500	0.750	0.024	0.73
HB19-HB20	0.08	0.85	0.072	2.790	1.500	0.710	0.028	0.68



Appendix 7: Rainfall Data

Bole Metrological data for Bole Sub city Study area																																						
Name	Eleva	Geogr	Geog	Elemen	Year	Mont	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2001	01	0.0	0.0	0.0	0.0	0.0	0.0	0.0	8.8	1.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2001	02	0.0	0.0	0.0	0.4	4.5	11.3	3.4	2.5	22.6	2.7	18.9	6.5	6.3	6.5	0.0	0.0	0.5	20.1	0.3	0.0	0.0	0.9	7.9	0.0	2.4	0.6	0.4	19.0	31.9	4.7	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2001	03	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.2	0.0	0.0	0.0	0.0	0.0	0.0	2.9	0.2	0.9	1.3	0.0	0.0	0.0	0.0	4.0	3.3	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2001	04	7.6	1.4	2.8	3.1	5.9	10.0	0.0	1.5	7.6	7.3	11.0	1.8	1.3	0.5	17.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.6	12.5	8.4	0.0	5.6	2.8	0.0	6.6	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2001	05	2.6	0.0	0.0	6.1	1.2	9.7	4.5	4.6	0.5	8.5	1.4	1.8	3.6	5.6	4.0	16.4	14.0	6.5	2.6	7.1	2.3	5.9	2.4	14.4	2.6	8.9	0.0	3.5	11.7	13.6		
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2001	06	5.9	2.8	32.4	18.3	3.6	5.7	0.0	11.8	4.7	23.5	0.0	6.0	14.9	16.5	3.4	8.2	9.7	12.8	22.1	13.2	1.8	2.9	5.4	4.5	2.4	9.9	9.4	0.0	7.0	2.6	28.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2001	07	17.6	8.8	6.6	0.0	2.4	0.0	0.0	5.4	17.4	5.5	1.2	3.8	11.2	2.5	4.0	0.0	6.8	19.0	8.1	16.3	0.0	11.8	7.7	0.0	18.5	0.0	0.0	19.5	6.2	0.2	6.8	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2001	08	10.7	6.2	0.0	16.0	2.4	16.8	11.8	0.0	6.1	0.0	6.0	8.6	3.5	0.5	3.5	9.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	4.2	7.4	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2001	09	8.3	0.0	0.0	0.0	1.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2001	10	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2001	11	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2001	12	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2002	01	0.0	11.7	3.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	12.4	2.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2002	02	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	9.6	15.9	0.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2002	03	0.0	21.8	1.4	28.6	0.0	0.0	0.0	0.0	0.0	0.0	3.5	0.0	0.0	0.0	0.0	13.6	0.0	0.0	0.0	9.0	0.0	0.0	0.0	0.0	0.0	0.0	1.5	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2002	04	0.0	0.0	0.0	11.2	0.0	0.0	0.0	0.0	4.6	1.6	0.0	0.0	11.4	4.6	0.0	0.0	0.0	0.3	2.0	0.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2002	05	0.0	0.0	0.0	15.0	0.3	0.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.9	0.0	13.8	3.3	0.0	0.0	0.0	13.5	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2002	06	0.8	5.4	0.0	6.5	0.0	0.0	2.2	2.0	0.7	3.3	0.0	0.0	0.0	13.8	17.0	5.8	2.0	13.1	9.5	7.3	9.4	3.4	0.0	0.0	1.0	0.0	0.0	0.3	12.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2002	07	4.6	2.7	11.0	3.3	10.0	0.7	0.0	1.8	15.8	3.6	14.0	1.0	8.4	3.7	3.1	0.0	1.1	10.3	15.4	6.3	6.2	22.1	5.0	9.3	2.5	16.7	8.0	7.4	18.4	0.0	1.5	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2002	08	17.5	17.9	1.2	0.4	17.6	6.3	23.7	0.0	3.7	6.3	2.6	0.0	0.0	15.7	0.6	6.7	2.0	15.4	0.9	9.6	0.0	15.4	11.2	16.8	2.5	0.5	10.6	1.8	0.0	3.9	22.8	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2002	09	2.4	1.7	15.3	1.4	0.0	3.8	5.1	7.1	0.0	0.0	2.0	0.0	12.5	8.8	4.6	2.5	0.0	3.5	0.5	0.0	0.0	0.0	0.0	1.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2002	10	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.5	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2002	11	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2002	12	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	25.1	0.0	0.0	4.8	2.9	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2003	01	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	4.8	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2003	03	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.5	4.7	0.0	21.7	4.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.4	16.8	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2003	02				0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	6.3	23.5	1.7	0.2	0.0					
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2003	04	1.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	17.3	1.2	4.1	5.8	1.0	4.7	4.5	0.0	8.8	0.0	0.0	22.3	6.2	34.6	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2003	05	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.4	15.4	0.0	0.0	0.0	0.0	0.0	1.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2003	06	0.0	0.0	3.1	0.0	0.0	1.5	14.2	0.0	0.0	0.0	0.4	1.0	0.0	1.8	0.0	4.6	6.5	2.0	0.0	12.6	7.3	9.3	7.3	2.2	5.8	4.6	0.0	4.6	5.0	17.2		
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2003	07	0.4	19.4	18.6	2.1	0.1	2.7	3.0	0.1	0.0	20.0	2.7	3.5	13.4	11.5	1.5	1.9	1.1	16.0	1.5	8.8	11.7	0.5	3.5	15.2	21.8	1.5	0.0	3.4	7.8	10.3	0.3	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2003	08	0.3	8.3	8.5	7.7	0.0	1.3	0.0	9.3	17.0	21.3	3.8	14.7	3.4	1.0	0.0	0.0	18.7	0.0	0.0	0.0	15.2	16.6	4.5	24.6	12.8	0.0	33.2	0.5	14.0	0.0	1.7	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2003	09	9.3	0.1	0.0	4.6	7.6	8.7	0.1	25.2	7.7	11.7	2.2	2.6	0.1	0.0	1.4	1.4	19.4	1.4	0.0	0.0	2.8	7.1	0.0	7.5	0.0	0.5	0.0	0.0	0.0	8.8		
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2003	10	0.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	4.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2003	11	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	

Rainfall data cont...

Bole Metrological data for Bole Sub city Study area																																						
Name	Eleva	Geogr	Geog	Elemen	Year	Mont	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2003	12	0.0	0.0	0.0	0.0	0.0	33.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2004	01	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	8.5	0.0	0.0	0.0	0.0	0.0	0.0	14.0	0.0	3.0	0.0	0.0	0.0	0.0	0.6	0.0	0.0		
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2004	02	0.0	11.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2004	03	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	12.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.8	4.4	2.5	0.0	12.1	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2004	05	0.5	0.0	2.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	4.2	0.0	0.1	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2004	06	0.0	0.4	0.0	0.3	0.0	4.4	3.6	0.0	3.6	2.3	4.5	3.8	3.0	0.2	0.0	0.1	0.0	1.7	7.5	18.5	0.0	1.1	0.0	1.5	15.3	14.9	6.3	2.7	14.4	4.4		
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2004	07	4.4	13.9	1.2	12.4	18.8	7.3	7.1	8.3	0.0	0.0	4.1	5.4	1.2	14.6	4.8	9.7	23.0	4.5	14.0	0.0	8.6	1.0	29.0	1.3	11.5	9.8	4.4	0.0	16.6	0.0	3.7	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2004	08	5.0	14.7	27.8	5.0	4.3	2.3	6.3	0.2	15.9	10.5	4.2	4.2	9.9	6.5	3.3	0.5	4.4	5.6	5.3	12.4	12.5	9.2	3.0	0.0	10.8	23.8	1.4	6.4	0.1	0.2	14.4	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2004	09	4.9	0.0	3.8	9.7	16.7	1.6	0.7	1.6	2.3	0.0	22.6	11.4	8.5	2.5	0.0	4.3	0.9	1.8	0.4	0.0	3.5	0.0	0.0	0.0	11.6	6.3	7.0	0.0	0.0	0.0		
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2004	10	0.0	0.0	12.0	0.0	0.0	9.6	4.0	23.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.8	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2004	11	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.6	0.0	0.0	0.0	0.0	0.0	0.0		
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2004	12	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2005	01	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	28.3	13.0	4.4	0.0	0.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	6.6	0.0	2.8	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2005	02	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.5	0.0	13.6	0.0					
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2005	03	0.0	0.0	0.0	0.0	0.0	1.6	8.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	6.7	10.0	2.1	9.6	3.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2005	04	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	5.8	0.0	5.5	4.8	17.8	0.2	1.7	10.7	4.5	0.0	8.7	0.0	5.0	10.0	23.9	0.0	0.0	17.6		
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2005	05	0.0	0.5	33.5	5.0	2.0	44.5	0.7	7.8	20.5	11.0	1.4	14.8	0.0	15.6	0.0	0.0	0.0	7.2	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2005	06	0.0	0.0	0.7	7.5	0.0	0.0	1.1	4.1	0.2	0.0	0.0	0.6	1.2	0.9	0.0	0.8	8.6	3.5	15.4	2.0	10.4	1.8	5.4	5.6	26.4	2.0	29.1	8.9	13.2	9.7		
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2005	07	0.0	0.0	3.4	3.6	20.9	0.0	2.0	2.5	5.5	6.2	10.8	14.5	0.0	3.8	4.4	3.6	21.8	8.4	0.1	0.9	16.6	10.4	0.1	9.3	0.0	15.1	0.5	0.5	6.3	2.8	0.3	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2005	08	9.2	2.0	2.6	7.2	16.5	0.0	12.9	0.0	10.8	0.0	8.5	0.0	6.6	3.0	17.6	27.9	22.0	0.4	10.3	0.0	0.0	2.1	0.0	0.0	0.0	16.2	0.8	32.4	10.2	27.9	0.9	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2005	09	0.0	1.8	0.0	3.1	10.5	0.8	6.4	4.0	2.3	0.4	8.3	1.2	0.1	12.9	2.0	6.0	0.6	6.2	0.1	0.0	0.0	0.0	0.0	2.4	0.0	0.0	8.5	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2005	10	0.0	0.0	0.0	2.4	0.0	0.3	0.0	0.4	0.0	22.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2005	11	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	3.8	3.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2005	12	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2006	01	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2006	02	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.6	9.0	5.8	1.9	0.0	2.3	0.0	16.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2006	03	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.5	0.0	3.3	0.0	0.0	0.0	2.4	0.0	13.0	0.0	0.0	44.5	0.0	1.3	0.0	0.0	0.0	2.3	4.1	36.4	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2006	04	12.0	0.0	0.0	8.8	0.0	0.0	22.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	11.2	5.2	0.0	0.0	1.0	0.0	1.8	0.0	23.4	8.0	0.0	0.0	0.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2006	05	1.5	0.0	0.5	13.1	2.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	7.5	4.6	2.0	0.6	1.5	0.4	0.0	2.1	1.8	0.0		
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2006	06	10.2	4.1	6.1	0.0	4.6	2.4	3.4	6.4	6.0	0.4	0.4	2.3	4.5	5.0	0.0	0.0	2.0	4.0	2.7	7.8	0.2	0.4	5.8	1.5	14.1	0.8	2.2	4.1	12.7	1.0		
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2006	07	0.0	2.3	5.0	2.8	6.0	6.2	0.7	4.0	17.1	19.3	11.8	11.8	0.4	7.3	2.2	1.8	24.7	38.3	2.4	1.8	6.3	26.8	33.4	2.9	1.6	4.7	19.6	16.0	1.2	30.0	4.8	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2006	08	4.7	0.2	8.8	3.5	1.8	0.8	16.8	8.3	4.4	41.6	2.4	0.6	61.7	1.2	8.6	0.4	10.4	4.7	19.3	4.3	36.0	13.2	9.0	5.8	3.0	0.5	0.1	0.2	8.8	13.5	36.5	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2006	09	36.5	3.4	11.3	0.0	1.4	0.0	13.0	1.3	0.0	5.9	3.4	13.3	2.8	0.0	3.3	7.2	2.3	13.5	0.0	0.0	0.0	1.0	10.9	0.0	2.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2006	10	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.8	0.0	0.0	0.0	0.0	0.0	0.0	1.5	0.0	31.7	0.0	0.0	0.9		
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2006	11	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	

Rainfall data cont...

Bole Metrological data for Bole Sub city Study area																																						
Name	Elevat	Geogr	Geog	Elemen	Year	Mont	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2006	12	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2007	01	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	5.3	0.0	0.0	0.0	0.0	4.5	0.1	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2007	02	3.4	8.2	9.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2007	03	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.4	1.8	23.2	24.8	0.0	0.0	0.0	0.0	0.0	0.0	9.9	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2007	04	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	5.5	12.6	30.0	3.8	0.0	13.3	1.4	3.1	2.2	0.0	8.6	0.0	0.0	0.0	0.0	0.4	5.9	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2007	05	0.0	0.0	0.0	0.0	0.0	0.0	0.0	3.5	0.0	0.0	18.8	0.0	3.7	12.6	27.8	0.8	0.0	0.0	0.0	0.0	0.0	0.0	1.2	3.0	1.4	35.7	0.0	0.4	19.4	5.7		
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2007	06	6.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	28.4	1.5	0.6	1.8	2.3	0.0	11.6	0.2	12.6	0.1	12.1	6.4	17.4	17.6	5.1	12.0	5.3	0.3	4.2	11.2		
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2007	09	15.0	1.3	3.2	10.3	9.2	0.0	0.0	0.0	21.6	0.0	0.0	0.0	7.8	4.7	8.4	3.5	3.0	0.0	0.0	10.5	3.4	0.0	11.3	1.4	0.3	0.0	0.0	8.0	1.5	6.5		
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2007	10	12.0	12.8	9.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2007	08	21.3	13.2	15.5	6.5	0.3	7.4	11.4	1.4	14.6	1.0	5.6	9.5	3.0	19.7	1.2	2.6	13.1	0.0	1.3	0.0	71.2	18.1	2.9	9.8	3.4	15.7	4.1	14.4	0.3	1.9	15.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2007	11	0.0	0.0	0.0	0.0	0.0	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2007	07	0.9	5.9	5.5	8.9	1.5	0.2	1.2	24.5	5.4	1.7	18.3	0.0	3.5	8.1	4.3	0.3	0.0	0.0	8.8	5.8	5.8	7.0	2.6	2.1	2.2	5.5	7.4	38.7	0.0	0.1	15.1	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2008	01	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2008	02	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2008	03	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2008	04	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.8	30.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.4	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2008	05	0.0	0.0	10.2	3.8	1.0	4.3	26.4	0.0	0.0	0.0	0.0	0.0	1.8	0.3	0.0	0.0	0.0	0.0	10.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.2	12.6	3.4
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2008	06	7.0	2.7	3.9	0.0	16.6	0.0	3.1	1.8	1.8	0.2	0.0	2.5	0.0	0.0	0.0	0.0	0.5	7.3	9.1	3.1	8.3	0.6	0.0	0.0	0.0	4.6	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2008	07	16.3	0.5	1.5	17.3	25.5	5.4	15.6	0.2	0.0	18.2	3.4	6.8	6.5	11.1	0.0	0.0	5.3	2.6	27.4	18.1	6.1	11.0	5.2	11.7	13.6	9.9	10.7	4.6	13.7	13.5	13.4	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2008	08	13.5	0.2	14.0	3.1	12.6	8.4	1.1	7.8	12.0	6.5	1.0	0.4	4.8	5.0	15.7	37.2	21.4	0.0	7.2	5.5	0.0	4.6	1.0	4.2	1.0	17.0	2.7	10.6	8.0	14.4	18.2	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2008	09	15.9	27.6	12.0	4.2	1.6	12.1	4.2	7.1	0.0	0.0	15.7	1.0	19.7	3.0	0.0	0.0	9.0	12.4	0.0	0.0	0.0	25.5	10.8	1.7	1.4	0.0	0.0	0.0	0.0	7.8	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2008	10	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	14.4	0.0	0.0	5.7	2.1	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2008	11	4.7	24.1	0.0	23.7	0.0	0.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2008	12	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2009	01	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.7	1.4	11.5	2.4	2.6	6.8	1.3	12.2	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2009	02	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2009	03	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	6.3	3.4	2.7
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2009	04	0.0	0.0	4.3	9.1	3.4	2.6	0.0	0.0	1.0	0.0	17.8	3.2	0.0	0.0	0.0	3.4	1.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2009	05	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.1	19.1	15.0	2.8	0.1	2.4	6.7	3.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2009	06	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.1	1.3	1.3	0.2	18.6	2.0	0.0	0.0	0.0	5.5	0.0	1.0	4.1	14.6	7.0	9.8	11.0	0.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2009	07	11.0	0.0	0.0	1.4	0.1	6.7	20.8	10.0	0.9	7.0	1.4	1.5	6.7	16.1	22.5	11.2	2.2	11.0	6.4	9.1	3.3	2.2	5.4	4.8	23.4	0.5	9.2	7.3	14.4	2.0	19.7	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2009	08	16.9	5.8	4.6	5.0	12.2	11.2	2.0	0.5	0.0	5.5	3.4	3.4	4.7	12.2	10.5	3.4	9.8	32.3	9.0	16.7	7.3	7.0	18.0	9.7	0.0	25.2	2.7	7.6	12.5	3.8	6.6	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2009	09	0.2	0.8	9.0	5.2	7.3	29.5	2.3	7.4	15.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.8	8.3	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2009	10	0.0	0.0	0.0	0.0	0.0	0.0	0.0	5.8	13.0	13.8	9.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2009	11	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.0	0.0	0.0	0.0	0.0	

Rainfall data cont...

Bole Metrological data for Bole Sub city Study area																																							
Name	Elevat	Geogr	Geog	Elemen	Year	Mont	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31		
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2009	12	0.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	3.2	51.2	13.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.3	3.8	2.0	0.0	3.4	0.0	0.0	0.0	0.0	2.0		
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2010	01	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2010	02	0.0	0.0	0.0	1.0	0.0	2.4	54.4	6.9	0.0	27.4	0.0	3.7	1.2	0.8	0.0	0.0	2.3	0.0	0.0	0.0	0.0	0.0	0.0	1.3	0.7	0.6	0.0	12.5					
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2010	03	0.0	28.4	2.7	0.0	0.0	1.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.3	0.0	21.4	0.0	0.0	0.0	0.0	0.0	0.0	6.4	0.0	4.6	7.0	2.5	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2010	04	3.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	9.1	13.3	40.0	17.8	35.8	17.8	0.2	0.0	0.0	1.7	9.7	7.7	0.0	0.8	0.0	0.0	1.3	0.9	0.0	0.0		
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2010	05	0.0	0.0	0.0	2.9	6.8	2.6	10.5	1.4	2.9	26.6	2.2	1.3	2.5	15.1	5.6	0.0	2.1	1.7	0.0	0.0	0.0	0.0	4.9	0.6	2.0	0.0	0.0	0.0	0.0	0.0	0.0	3.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2010	06	0.0	0.0	0.0	1.2	1.2	0.0	0.8	1.6	0.9	3.2	7.2	5.4	0.0	1.7	0.0	0.0	0.0	5.0	11.2	14.0	4.5	0.0	0.0	10.4	14.6	3.8	3.0	3.4	3.4	10.7			
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2010	07	1.5	1.3	0.3	15.5	11.2	6.7	2.5	9.8	15.9	0.0	33.0	0.0	25.0	20.5	0.5	4.4	25.0	0.0	0.3	20.4	12.2	48.3	6.3	13.2	5.7	12.9	7.8	0.0	2.5	4.3	13.2		
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2010	08	0.0	0.0	0.0	1.7	3.4	4.8	14.6	9.7	0.0	10.4	3.3	6.0	1.5	0.2	0.0	15.8	1.6	6.7	0.2	4.3	9.5	4.0	4.0	0.0	0.0	0.2	3.6	15.9	3.0	8.5	5.9		
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2010	09	14.8	0.0	3.3	0.0	2.1	1.7	3.7	3.0	0.0	14.2	0.3	0.0	2.1	13.3	0.2	23.1	0.0	3.2	3.3	0.6	0.0	0.0	0.0	0.0	9.7	0.0	0.0	0.0	0.0	0.0	6.4		
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2010	10	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2010	11	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	12.5	1.3	0.0	0.0	0.0	0.0	0.0	0.0		
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2010	12	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	13.6	2.1	0.0		
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2011	01	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	3.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2011	02	0.0	12.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2011	03	0.0	0.0	0.0	0.0	0.0	0.0	0.4	7.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.8	13.2	3.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2011	05	26.0	1.1	0.0	2.0	3.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.0	4.3	0.0	0.0	5.9	16.5	0.0	3.3	6.0	0.0	0.0	12.9	3.4	0.8			
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2011	06	0.0	0.2	0.0	0.0	4.7	0.4	9.8	22.5	0.0	2.9	4.7	3.2	18.2	0.0	4.0	11.0	0.0	0.0	0.3	5.6	0.0	4.7	3.5	2.5	6.3	2.4	32.2	4.4	4.5				
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2011	07	0.0	2.2	0.0	0.0	1.6	19.6	2.8	2.5	0.0	2.6	8.5	4.5	1.9	0.0	1.2	0.3	5.0	6.8	0.0	2.7	17.3	0.6	15.8	10.9	0.0	19.6	3.9	7.0	36.9	7.8	1.1		
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2011	08	33.9	11.8	10.2	5.2	0.0	8.4	14.8	5.6	4.2	7.3	4.5	0.0	23.6	2.9	17.5	16.3	7.6	31.0	0.5	2.9	17.2	8.0	15.1	7.0	19.7	0.5	6.0	11.1	0.1	2.2	1.4		
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2011	09	5.0	0.0	0.0	5.1	19.3	12.2	0.2	3.5	1.2	9.3	13.4	1.8	0.0	2.4	11.4	0.0	0.0	8.7	4.9	0.0	0.0	2.4	0.0	7.0	15.1	7.8	10.6	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2011	10	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2011	11	0.0	0.0	0.0	4.8	6.7	0.0	0.0	0.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2011	12	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2012	01	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2012	02	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2012	03	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	16.4	18.1
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2012	04	18.0	5.4	1.4	0.0	6.0	5.0	1.8	0.0	0.0	1.8	0.0	0.0	12.0	0.0	2.5	0.0	2.4	10.8	0.4	0.0	2.6	0.0	0.0	0.8	0.0	0.6	0.0	2.6	1.0	0.0			
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2012	06	0.0	0.2	2.4	0.4	0.6	0.0	0.0	0.0	0.0	0.4	12.3	0.0	0.0	0.0	0.0	0.0	0.0	3.7	7.9	0.0	15.7	3.5	0.0	0.0	0.2	9.0	1.4	0.0	11.3	3.8			
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2012	05	0.0	0.0	0.0	0.0	0.9	1.4	5.8	31.1	18.9	0.0	0.0	0.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2012	07	3.8	3.4	9.0	0.7	9.9	13.4	0.0	4.6	1.8	5.0	13.0	11.1	26.9	2.5	4.0	15.6	1.5	3.5	10.2	1.0	9.8	0.0	11.0	4.6	17.0	19.6	0.2	8.5	4.2	12.3	0.7		
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2012	08	0.5	26.2	9.3	5.9	3.8	0.0	22.8	0.0	64.7	23.9	2.3	4.7	6.5	0.4	5.0	30.0	16.6	1.8	0.0	2.8	8.6	3.1	6.0	0.0	11.5	4.4	1.5	2.0	8.3	2.1	6.9		
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2012	09	0.3	10.7	4.4	22.5	9.8	6.2	0.1	2.8	6.1	0.0	29.0	0.7	0.0	3.6	43.9	0.6	0.5	0.2	10.0	0.0	18.4	0.0	3.0	4.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2012	10	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.2	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2012	11	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Rainfall data cont...

Bole Metrological data for Bole Sub city Study area																																							
Name	Elevat	Geogr	Geog	Elemen	Year	Mont	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31		
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2013	07	20.4	12.8	1.6	1.8	16.0	7.6	3.1	10.4	0.2	1.5	0.0	0.3	0.0	7.7	0.0	2.3	0.2	10.4	0.0	12.9	1.2	13.0	1.8	5.5	0.0	15.3	4.2	0.6	3.4	2.4	1.0		
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2013	08	20.6	0.0	3.0	10.4	12.4	9.0	1.2	3.7	1.2	5.0	7.3	7.6	11.8	28.7	11.4	2.0	18.4	14.7	0.6	0.0	14.4	13.0	5.5	7.7	5.1	9.5	9.0	2.8	3.8	1.4	29.0		
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2013	02	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2013	03	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.6	0.0	3.7	4.2	9.3	9.2	13.0	19.9	0.0	3.2	0.0	0.4	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2013	04	0.0	0.0	9.2	6.7	6.0	0.7	14.8	0.0	1.0	2.5	0.0	0.0	0.0	2.3	0.0	0.0	42.6	4.0	0.0	0.0	9.5	0.0	0.0	0.0	6.5	7.0	0.0	0.0	1.6	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2013	05	0.0	0.0	17.0	11.9	7.1	1.0	14.6	2.0	0.5	0.0	0.0	0.0	1.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	3.6	0.0	1.0	11.0	5.0	2.4	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2013	06	2.0	15.0	0.2	0.0	0.0	1.7	0.0	0.0	6.8	0.0	3.0	4.9	2.6	3.9	5.8	1.0	12.8	12.3	0.0	1.1	0.7	1.7	8.8	0.7	7.1	0.0	2.4	0.0	2.5	4.4	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2013	01	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2013	09	0.6	22.0	4.7	7.9	8.2	9.9	0.0	4.3	4.2	0.2	0.0	12.6	3.7	5.4	0.3	23.1	5.4	3.4	0.0	0.0	0.0	4.0	0.6	0.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	6.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2013	10	6.0	9.2	2.0	0.0	13.3	8.1	1.6	2.8	0.0	0.6	1.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2013	11	0.0	0.0	0.0	0.0	1.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2013	12	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2014	12	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2014	01	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2014	02	0.0	0.0	0.0	0.0	0.0	0.0	1.1	12.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	27.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2014	03	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	7.0	0.0	1.6	2.6	0.7	0.0	0.0	0.0	0.0	0.2	0.0	1.2	0.7	0.8	0.0	5.1	0.0	5.6	4.2	0.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2014	04	10.2	6.6	3.4	0.0	0.0	13.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2014	05	0.0	3.1	0.0	2.0	0.0	0.0	0.0	0.0	0.0	14.2	0.0	0.0	0.6	0.0	0.0	0.0	0.0	4.1	0.0	0.0	0.0	6.4	0.7	14.2	6.4	5.7	0.0	4.7	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2014	07	0.4	5.7	14.4	0.5	8.5	16.0	7.4	0.4	7.7	3.8	4.2	18.4	7.8	0.5	12.8	4.2	10.0	0.3	15.3	0.4	0.0	0.0	1.6	0.0	10.8	4.4	0.8	15.6	1.5	0.0	0.0	6.3	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2014	08	3.0	1.6	0.0	1.8	5.4	22.0	22.5	10.5	11.2	9.0	20.0	11.5	23.5	0.0	24.2	5.2	3.0	6.5	7.0	8.9	5.0	4.5	1.5	9.0	27.0	3.3	2.7	0.0	0.0	3.0	0.8	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2014	09	5.0	1.3	0.0	17.2	0.7	0.0	1.4	5.6	0.0	0.0	0.0	2.0	11.4	3.0	5.2	0.0	0.2	0.0	0.0	13.8	1.2	0.0	12.5	0.0	1.0	11.0	2.6	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2014	10	14.5	0.0	3.9	6.4	10.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2014	11	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2014	06	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.4	1.2	8.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.0	0.0	6.8	8.5	1.0	0.0	8.6	0.0	0.0	3.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2015	01	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2015	02	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2015	03	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.4	0.0	18.2	0.0	2.0	0.0	0.0	0.0	0.7	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2015	09	2.6	0.0	0.0	5.8	7.6	11.4	5.8	16.6	12.0	3.0	0.0	0.0	22.7						0.0	0.0		5.0	0.8	0.0	0.0	0.0	0.0	0.0	0.0	1.8	0.0	0.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2015	11	0.0	0.0	0.0	0.0	0.0	1.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2015	10	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2015	05	0.0	0.0	0.0		1.5	9.0	0.0	5.6	0.0	5.0	0.0	0.0		0.0	0.0	0.0	0.0	0.0	0.0	0.5	0.0	0.0	60.5	60.5	21.5	0.0	2.3	2.7	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2015	06	31.2	9.7	0.0	0.0	0.0	0.0	4.5	5.0	1.0	0.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	6.7	9.5	19.3			4.2	0.0	0.2	19.2	12.6	11.7	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2015	12	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2015	07	8.8	3.5	0.0	0.0	0.0	1.2	1.6	0.4	7.0	2.1	7.0	1.0	4.0	5.0	21.5	1.6	7.7	9.4	5.5	12.6	3.0	4.4	7.5	5.4	0.0	11.5	2.2	1.0	14.6	4.0	9.5	0.0	0.0
Addis Ababa Bole	2354	38.75	9.03	PRECIP	2015	08	0.0	2.0	5.6	0.0	0.0	6.5	0.0	14.0	11.2	17.4	0.0	15.2	12.6	12.0	1.6	11.6	12.3	3.0	6.0	22.2	13.6	4.0	6.5	14.6	0.2	0.0	0.0	0.0	0.0	0.0	6.5	4.4	0.0