JIMMA UNIVERSITY

SCHOOL OF POST GRADUATE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

FACULITY OF CIVIL AND ENVIRONMENTAL ENGINEERING

ENVIRONMENTAL ENGINEERING MSc. PROGRAM

ASSESSMENT OF URBAN STORM WATER DRAINAGE SYSTEM USING STORM WATER MANAGEMENT MODEL: THE CASE OF BONGA TOWN, ETHIOPIA

BY: ANDINET G/SILASIE

A THESIS IS SUBMITTED TO SCHOOL OF POST GRADUATE STUDIES OF JIMMA UNIVERSITY PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTER IN ENVIRONMENTAL ENGINEERING

OCTOBER, 2021

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By: Andinet G/silasie Atachew

Approved by

Main advisor

Dr.-Ing. Fekadu Fufa (Ass.Professor)

Co-advisor

Mr. Desalegn Geleta (MSc.)

OCTOBER, 2021

JIMMA, ETHIOPIA

DECLARATION

I, Andinet G/silasie declare that the title “assessment of urban storm water drainage system using storm water management modeling 5.1” is the original research work that prepared with my own effort to fulfill the partial requirement to degree of master.

Andinet G/silasie Atachew signature………………..……Date…………………………

The thesis that prepared by Andinet G/silasie has been submitted for examination with my approval as a university supervisor.

Main advisor

Dr.-Ing. Fekadu Fufa (Ass.Professor) signature ………..…… Date………………..………

Co-advisor

Mr. Desalegn Geleta (MSc.) signature……………….…Date….…………...……….

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ABSTRACT

Storm water drainage is the systematic way of transporting surface runoff from urban environment to protect the risk of flooding and damage. Bonga town has a problem of storm drainage system due to lack of suitable topography and properly design of drainage system. Objective of the study is to assess *storm drainage system of Bonga town of selected area in affected sites through storm water management model version 5.1. The outputs with this study is simulation of the drainage system from initial node to outlet point without flooding problem at flow routing of drainage network and with the permissible percent of error to surface runoff. Both primary and secondary data's were collected from the field observation, municipality office and national metrology agency of Ethiopia respectively. Estimation of the storm water runoff peak discharge in the drainage network was carried out with available rainfall data and Log Pearson Type III probability distribution method was selected for frequency analysis in the study according to the value of coefficient determination. The affected part of the town around study site by existing poor drainage system was identified on roadway, ditches and residences. For comparison and evaluation of performance of model as well as its fitting, rational method was used to calculate peak discharges in all sub catchments. The study catchment was divided based on existing road access and elevation difference into twenty seven individual sub catchments and on those 121 conduits/channels, 116 node/junctions and 2 outlets were set on the model for simulation. Validation of the storm water management model 5.1 in the study was carried out through the continuity percent error of surface runoff and flow routing. The model results were 1.6 for surface runoff and zero for flow routing respectively. The performance of model was evaluated with the value that determined through statistical equations such as coefficient determination, Nash Sutcliffe and relative error and its values were 0.895, 20.5, and 0.86 respectively. The total amount of determined peak runoff discharge from study area in storm water management modeling 5.1 was 12.32 cms and whereas 9.31cms in rational method respectively. Therefore, redesign has to be required with stake holder experts to manage the storm water drainage system of study area in terms of peak runoff concentration that generated from sub catchments.*

Key words: *Drainage System, Storm water, Storm water management Modeling, Sub catchments, Simulation*

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ACRONMYS AND ABBREVIATIONS

AMS	Annual Maximum series
CMS	Cubic Meter per Second
CBE	Commercial Bank of Ethiopia
DEM	Digital Elevation Model
EPA	Environmental Protection Authority
GIS	Geographical Information System
HGL	Hydraulic Grade Line
IDF	Intensity Duration Frequency
LID	Low Impact Development
LULC	Land Use Land Cover
NMA	National Metrology Agency
NRCS	Natural resources conservation service
SCS	Soil Conservation Service
SNNPR	South Nation Nationality and People Republic
SWMM	Storm Water Management Modeling
TC	Time of Concentration

CHAPTER ONE
INTRODUCTION

1.1 Background

Urban storm water drainage facilities are part of the urban infrastructure elements and design of these facilities require due attention (Ejigu and zewudie, 2020). Urban environments were disturbed with increasing of storm water runoff quantity that occurs due to natural phenomena in water cycle and anthropogenic activities. Sewunet (2020) observe that storm water is the basic natural component challenging urban areas with urbanization effects. Storm water drainage system critically aims to keep the environmental health from surface runoff. Sanitation is a fundamental to healthy and productive urban life, and the provisions of sanitation services for fast-growing urban populations is one of the world's most urgent challenges (Kim, *et al.*, 2015).

Hydrologic cycles play a major role in surface runoff production from rainfall. Precipitation is any type of condensation of atmospheric water vapor that falls under gravity and includes rain, snow, sleet, hail, fog and generate surface runoff after all losses (Rachel, 2017). The amount of coming rainfall is affected by several factors like infiltration, interception, depression, evaporation, before reaching the earth's surface. The amount of runoff generation determined based on the land use land cover of the catchments.

The cause of densely settlement of population results deforestation and ineffective use of resources. In urban areas, due to the intense alteration of natural environmental processes by human activity, the watershed response to precipitation are also significantly altered such as reduction of infiltration, decreased travel time, higher runoff, urban flooding (Chithra, 2015). As natural landscapes are being converted to urban ones, the amount of impervious surfaces increase and storm water runoff becomes more significant as it is not able to infiltrate into soils and natural surfaces (Rachel, 2017). Many factors due to human activity are considered as possible responsible of the observed change (Mauro, 2020). An increase in impervious land cover leads to more surface runoff, faster runoff concentration and higher peak flow rate (Mipale, *et al.*, 2017). Impervious surfaces are useful indicators of the urbanization impacts on water resources (Ebrahimian, *et al.*, 2016).

The occurrence event of sudden overland flow is peak runoff due to the lack of proper surface runoff drainage management at certain urban from point of rainfall to receiving water body or downstream. However, they can happen very quickly when lots of heavy rain falls over a short period of time (Asfaw, 2016).

Environmental and natural values may be affected by runoff, despite the fact that floods are to some extent a natural phenomenon (Zelenakova, *et al*., 2016). Surface runoff problems are prominent during the wet season; other effects of poor drainages are perennial and intrinsically linked to deterioration in sanitation and environmental health conditions (Jonathan, 2016). Urban flooding is a major catastrophic trait of many cities around the world, uncertain factors such as, hydrological factors, urbanization, climate change and infrastructure inadequacy and failures that result in property damage, critical infrastructure distraction and loss of lives (Morita, 2014; Mugume and Butler, 2017; Liu, *et al*., 2016).

In most Ethiopian’s’ towns storm water management was very low relative to the available rainfall amount at a particular location. Due to inadequate installation of drainage infrastructure, poor maintenance of existing drains and the problem is more critical in cities of highland regions like Addis Ababa, Adgrat and others (Zena, 2015). Major cities of developing countries mainly suffer from the localized problem of flooding and water pollution due to presence of inefficient, unscientific and weakly maintained drainage system (Abraha, 2018).

The open storm water systems have become an increasingly acceptable solution to handle the storm water in urban areas, since these solutions are more sustainable help to reduce and retard the flow (shukri, 2010). Urban drainage system is the systematic way of transporting surface runoff from urban environment to conserve the environmental and public health. In designing of drainage, the primary objective is to properly accommodate water flow along and across the road and conveniently transport and deposit the water to the downstream without any obstruction in flow (Tiza, 2016). When road construction project is proposed at certain location, drainage design consideration is the critical issues to protect the constructing road structure up to design period operations. Drainage is one of the most important factors to be considered in the road design, construction, and maintenance projects (Getachew, 2015).

The problem of urban drainage can cause pollution of the urban environment as well as threat to community population. Poor drainage poses serious challenges in urban and sub urban areas worldwide and the drainage problems in roads can cause early distresses and lead to structural or functional failures of pavement, if counter measures are not undertaken (Magdi,2016). A drainage problem in urban areas introduces flooding, deterioration of roads, land degradation, sedimentation, water logging (Tamiru, *et al*., 2020). In poorly drained areas, urban runoff overflowing in latrines and sewers, causing pollution and a wide range of problems associated with the increased risk of water borne diseases (Jonathan, 2016).

Urban storm water drainage system is the components of highway structure that used to transport surface runoff coming from rainfall. Storm drain is that portion of the roadway drainage system that receives runoff from inlets and conveys the runoff to some point where it can be discharged into a ditch, channel, stream, pond, lake, or pipe (Asfaw, 2016). In urban areas storm water is generated by rain runoff from roofs, roads, driveways, footpaths and other impervious or hard surfaces (Birhanu, 2018). In almost all developing countries there is less comprehension to manage the urban storm water drainage system relative to available rainfall amount. However, even if with low intensity for short duration storm water quantity and its effect is more than control due to drainage system problems.

Bonga town has a problem of storm water drainage system due to lack of proper design, monitoring system during construction as well as operation process and evaluating the function of the drainage system according to desired uses. The aim of this study is assessment of existing storm water drainages, estimation of storm water quantity to model suitable drainage with storm water management model 5.1.

1.2 Statement of the problems

Urban storm water drainage system is the basis for understanding of environmental, infrastructure and public health concern. Over the world, improper design and low management of storm water drainage system causes to the losses a lot of lives and property as well as unexpected migration from local environment. In developing country, a lot of problems are faced due to storm water runoff that leading to global climatic change. Ethiopia is one of the developing countries in Africa and has a high probability to expose for storm water runoff due to natural geographical phenomena or condition of drainage system relative to the available rainfall.

Especially, south west parts of Ethiopia get extreme rainfall amount in summer season in every year and Bonga town is found in this part but has not well designed storm drainage system. From the consequences of insufficient drainage system design and management, the existing drainage system had been blocked during the summer season and runoff overtop toward middle street road access around the central square of the town. This study aims to identify the problem that causes to damage the part of the Bonga town with poor drainage between developed urban areas with rainfall dependent runoff by simulating the SWMM 5.1 model using all necessary data.

1.3 Objectives

1.3.1 General Objective

The main objective of the study is to assess storm water drainage system of Bonga town on affected sites using storm water management model version 5.1.

1.3.2 Specific Objectives

- to assess the conditions of existing drainage system with standards;
- to identify the effects on study area due to poor drainage system; and
- to determine the peak runoff by simulating SWMM 5.1 and verify the fitting of model.

1.4 Research questions

1. What were the problems of existing drainage conditions relative to hydraulic cross section standards of drainage?
2. What effects was identified due to poor existing drainage system? and
3. How to determine peak runoff discharge in the drainage system that simulated through using SWMM 5.1 and validate the fitting of model?

1.5 Significance of the study

The assessed existing drainage system problem is use as evidence to model the storm water drainage system with simultaneous scenarios for efficiently conveys the runoff that generate from rainfall. Properly designing of drainage system means the systematic way of saving the environment health from damage that may occurred by poorly design. Affected infrastructures during summer season were caused due to improper drainage system. Balancing the available rainfall data and the sizes of constructing drainage structures is the necessary consideration at a particular area. The SWMM 5.1 software was selected to model the drainage system relative to the capacity of existing drainage system and the available extreme rainfall amount. The expected output with this study is simulation of the storm water drainage system from initial node to outlet or disposal point without flooding problem at any node and the percent of error that may occur at flow routing and surface runoff below permissible level.

1.6 Scope of the study

The existing drainage system and effects that caused by improper drainage system were assessed for identification of the problem caused by poor design management system. Needs of geographical landscape identification was to determine the load of runoff during the extreme rainfall events on downstream part of the study area. The integration of hydrological data with SWMM 5.1 model was used to determine the amount of runoff at desired outlets for required number of years. The performance capacity of SWMM 5.1 was verified to know the fitting of model relative to rational method with various scenarios.

1.7 Limitation of the study

Previously, no study was done in a town with related title that used to support the current investigation in accordance to that evidence. The new plan of Bonga town includes undeveloped area but the study area covers only developed part in the town around Central Square that affected with low design and management of storm water drainage system.

CHAPTER TWO
LITERATURE REVIEW

2.1 General

The impact of environmental changes, mainly urbanization and climatic change, leads to increased runoff and peak flows which the drainage system must be able to cope with to reduce potential damage and inconvenience (Maharjan, 2009). Urbanization leads to the replacement of natural areas by impervious surfaces and affects the catchment hydrological cycle with adverse environmental impacts (Gerald, *et al*., 2016). The increase of paved surfaces, thus reducing infiltration, while causing surface runoff to exhibit higher peak flows, larger volumes and shorter times to peak and accelerated transport of pollutants and sediment from urban areas (Deresu, 2019).

Adane (2019) conclude that as the process of urbanization accelerates, drains became increasing with high load of runoff that is more than control when heavy rain fall down. The urban sanitation problem can lead to environmental pollution as well as loss of public health and it may be the consequence of improper urban drainage system. Spreading pathogens around communities and increasing risks to health from various waterborne diseases (Jonathan, 2016). The surface runoff is increase with increasing the number of population size and demands to lead their life. At an impervious surface the only rainfall loss is the initial loss, and at a pervious surface, rainfall exceeding the final infiltration capacity contributes to direct runoff (Tatsuya, *et al.*, 2016). Development of an urban area involves covering the ground with artificial surface, and it significantly increases the amount of surface runoff in relation to infiltration and evapo-transpiration (Zinabie, 2018).

The drainage problems in urban areas introduce the flooding, deterioration of roads, land degradation and sedimentation (Deresu, 2019). Urban drainage system is technological based environmental and public health protection methods of runoff conveyance from point of generation to point disposal point or outlet. Properly designed and managed drainage system is basic system of conveyance for fluid discharge from urban area. Bonga town existing drainage system was very poor and only open channel or masonary ditches are used as storm water conveyance. As the result, around the developed or center part of town’s road access damage, ditches blockage, and the storage of runoff on the street had been shown.

2.2 Necessity of urban drainage system

The importance of urban drainage system is to solve socioeconomic and environmental problems that caused by surface runoff around study area. Conveying the surface runoff in proper manner from upstream sub catchments on which point of rainfall to outlet or downstream. The main function of storm drainage system is to protect public health and safety, environment and sustainable development (Asfaw, 2016). Because of low management of drainage system in Bonga town, especially during summer season surface runoff discharge create threat to community public health. Most of the time in urban drainage design has high quantity of runoff and that may cause negative effects on the town economically and socially. Storm water drainage is an important component in the design of roadways, because it affects serviceability and usable life of the roadway, including structural strength of the pavement (Zeleke, 2018).

2.3 Condition of the existing urban drainage system

In Bonga town, urban drainage systems contain mostly rectangular masonary channels to conduct rainfall based runoff and retaining wall structures that used to support soil either from two or one side. However, the coverage of this masonary drainage was very small relative to the size of the town and low quality materials usage is carried out during construction period. In this study area there is no effective management of urban drainage system to keep the environmental quality of town and public health. According to Bonga town asset data the town has coverage of 161.798 km road access namely Federal asphalt, city asphalt, coble stone, gravel and compacted earth. Among this only certain km is covered with asphalt by both Federal and city budget. Drainage line usually following the road street according to master plan of the town that was prepared within municipality office. Rectangular masonary drainages were available in study area to transform storm water runoff through the main ditches following asphalt line that cover with slab the sub main without slab cover. There was no well-designed separate storm water drainage system in Bonga town that used to convey storm water surface runoff from the developed urban area. The figure 3.1 explains the road access and its coverage by percentage in Bonga town.

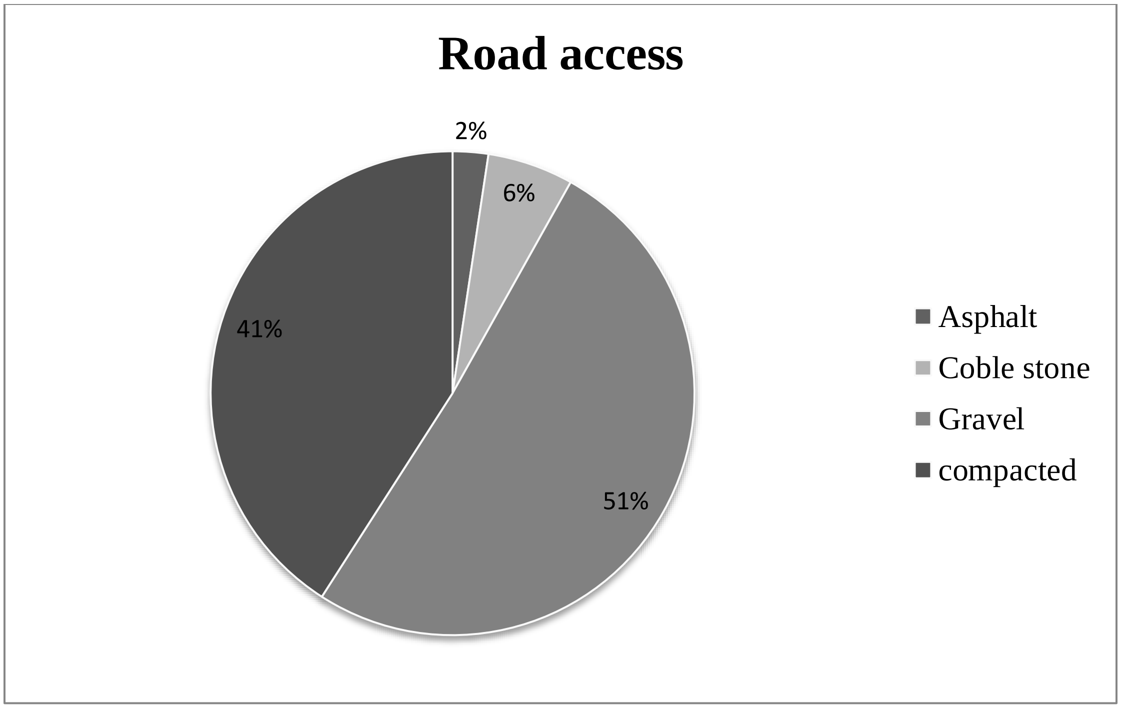

Figure 2.1: Existing different level road access in Bonga town

2.4 Storm water runoff

During the hydrologic cycle, the precipitation has the probability to reach either on pervious or impervious surface. When rain falls on a natural landscape, it soaks into the ground (infiltration), evaporates, is taken up by plants (evapo-transpiration) and some of it eventually finds its way into streams and rivers (Woods, 2015 and Abraha, 2018). Storm water is the water draining off a site from the rain that falls on the roof and land, and everything it carries with it (Asfaw, 2016). Storm water in built-up areas and other areas with closed surfaces can hardly find a natural path to reach the natural water cycle. This may result in gradual, long-lasting changes in soil structures and water régimes, entailing a reduction in the natural local ground water replenishment and impacts upon the chemical and biological conditions over and under the ground (Petr, *et al.*, 2015).

2.5 Causes of storm water surface runoff

The initial consequence of high storm water discharge is urbanization and increases the impacts on the urban environment. The development of urban areas has had a significant impact on urban storm water runoff and generation due to the replacement of natural green infiltration surfaces that mean natural soil cover with impervious surfaces such as concrete roads, rooftops and buildings (Asfaw, 2016). When there is high rainfall rate for a short period duration around certain area, there is also excessive surface runoff concentration in overland surface. A flood is an excess of water (or mud) on land that's normally dry and is a situation where in the inundation is caused by high flow, or overflow of water in an established watercourse, such as a river, stream, or drainage ditch; or ponding of water at or near the point where the rain falls.

2.6 Effects of poor drainage system

The effects of storm water runoff on urban area is very risky and lead to flooding that can damage the ground surface and its cover including human's built. The causes of poor drainage results pavement distress leading to driving problems and structural failures of road access (Tiza, 2016). Flood the consequences of poor drainage system and that alter the natural states. Floodwater can seriously disrupt public and personal transport by cutting off roads and railway lines, as well as communication links when telephone lines are damaged (Asfaw, 2016).

2.7 Storm water management

The main objective of storm water management is to protect the environment from excessive surface runoff. Storm water management is a widely used tool for urban drainage design and planning (Meharn, *et al.*, 2017). Storm water management is the effort to reduce runoff of rainwater or melted snow into streets, lawns and other sites and the improvements of water quality. Storm water runoff poses many challenges to cities, including flooded streets, strain on sewage conveyance systems and waste water treatment plants, and groundwater pollution of nearby water bodies (Nadia, 2016).

2.8 Challenges on urban drainage management system

The challenges in drainage management system is low interest of the stack holders institutions and lack of comprehension for design professions about the hydrological risk that caused by surface runoff. When society has not been impacted directly by pluvial floods, it tends to approach storm water management with a low level of interest (Andrea, 2020). Natural factors that determine urban drainage management systems in Bonga town are rainfall dependent erosion and landslides especially in summer season. Erosion of pavements resulting in reduction of service life of road infrastructure and impact of road flooding on nearby community are consequences of poor drainage system in the area (Alemu, 2017). Poor drainage quality on roads leads to a large amount of costly repairs or replacements before reaching their design life (Tiza, 2016). Factors that determines drainage management system was lack of consideration during budget classification for maintenance of drainage rather than road construction. As the result, most Ethiopian towns have problems of storm water drainage system and expose seasonal runoff.

Environmental problems relative to surface runoff and its causes are described as the destruction of natural state existences such as: soil resource, water resource (surface and ground), and climatic change or over all natural ecosystem disturbances are the result of improper management of drainage system. When the duration is increasing it may causes the ground water level became rise, the quantity of surface runoff concentration became increase. Due to this summation of water quantity on the urban surface, the structure could be useless, i.e. if there is no properly designed drainage system.

2.9 Hydrologic cycle

Every living thing requires water that occurs by hydrological cycles. Alemu (2017) conclude that the continuous movement of water between earth's surface and the atmosphere is known as hydrologic cycle. The water cycle of the Earth system and its variability at global, regional and local scales are influenced by a range of processes and mutual interactions, feedback mechanisms and as well as affected by anthropogenic processes (Michael, *et al.*, 2014).

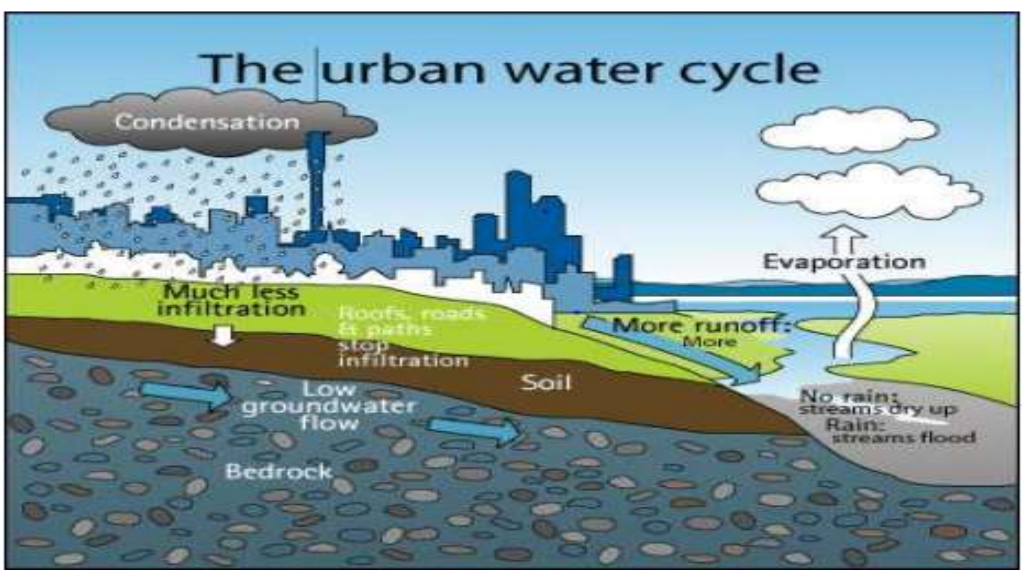

Figure 2.2: urban hydrologic cycle (source: Belachew, 2019)

2.10 Rainfall

Rainfall is liquid precipitation that can occur by natural hydrologic cycle and appear on the earth's surface as a source of water (Belachew, 2019). Saleem (2017) suggest that design of urban drainage infrastructure depends on the rainfall pattern and runoff volumes. The total rainfall received in a given period at a specific location is highly variable from one year to another. The variation depends on the type of seasons, vegetation cover, temperature, elevation and others. Estimates of rainfall or intensities that can be expected for a specific probability during a specific reference period are required for the management and design of drainage projects (Raes, *et al.*, 2013). The intensity of rainfall has important impacts on the hydrology of a system, and these impacts can be very different at small and large spatial scales (Daniel, *et al.*, 2015).

2.11 Estimation of peak runoff from rainfall

The ultimate goal of the hydrologic modeling is to obtain the design peak discharge and that of hydraulics modeling is to provide drainage structures or components for efficient and safe (Belayneh, 2016). To estimate the peak flow of storm water runoff there are a number of methods used depending up on the area of width in the catchment. The importance of peak runoff estimation is to analysis of drainage sizes for conveyance of storm water from study urban environment.

2.11.1 Rational method

Hassen (2016) determines and concludes that rational method is the best method to determining the surface runoff quantity for drainage area those less than 50 hectares. This method is used to calculate the peak runoff as the size of the study area is within the permissible limit (Alemu, 2017). In the design of storm water drainage system, the main purpose of hydrologic analysis is to determine the maximum amount of run-off (peak discharge) that can be accumulated at certain storm drainage outlet (usually a ditch) along a highway/access road alignment section (Asfaw, 2016). However, it has a limitation when the areas become complex and where sub catchments come together, the rational method tend to over or under estimate the actual flow, which results in problem of sizing of drainage facilities.

I. Runoff coefficient

The runoff coefficient estimation is used to consider pervious and impervious surface of specific study area relative to infiltration context. Runoff coefficient is expressed as a dimensionless decimal that represents the ratio of runoff to rainfall (Belachew, 2019). The study area contains sub catchments with different permeable and impermeable coverage. The estimation of runoff coefficient in all sub catchments is applicable in both rational method and SWMM 5.1 by identifying the land use land cover. The weighted runoff coefficient can be determined as the multiple each land use land cover area within one sub catchment by surface runoff coefficient and divided to its total area of sub catchment. The Proportion of the total rainfall that will reach the storm drains depends on the percent imperviousness, slope, and ponding character of the surface (Asfaw, 2016).

$$\mathrm{cw} = \frac{C1A1 + C2A2 + \cdots CnAn}{A\,total} \qquad 2.1$$

2.11.2 Storm water management modeling 5.1

The Storm Water Management Model (SWMM) is a dynamic rainfall-runoff simulation model used for single event or long-term (continuous) simulation of runoff quantity from primarily urban areas (Jorge, *et al.*, 2009). Like most hydrologic models, SWMM5.1 sub divides the overall catchment into sub catchments, predicting runoff from the sub catchments on the basis of their individual properties, and combining their outflows using a flow routing scheme (Birhanu, 2018). The runoff component of SWMM 5.1 operates on a collection of sub catchment areas that receive precipitation and generate runoff. The routing portion of it transports this runoff through a system of pipes or channels and tracks the quantity of runoff generated within each sub catchment, through channels during a simulation period comprised of multiple time steps. Storm water management model accounts for various hydrologic processes that produce runoff from urban areas. Spatial variability in all of these processes is achieved by dividing a study area into a collection of smaller, homogeneous sub catchment areas, each containing its own fraction of pervious and impervious sub-areas. Overland flow can be routed between sub-areas, between sub catchments, or between entry points of a drainage system (Lewis, *et al*., 2015). The modeling system integrates area, land use, soil type, elevation, precipitation amount, and temporal distribution to calculate runoff volume and runoff rate over time for individual storm events (Anne, *et al*., 2016).

2.11.3 Physical elements in SWMM 5.1

1. Sub catchments

Sub catchments are the small portion of study area that is divided from the proposed certain catchment to identify the geographical location on which surface runoff flows toward the common points (Lewis, *et al.*, 2015). Those runoff flows generate from precipitations are challenged by different factors such as infiltration, interception, storage depression, and others. In sub catchment themes area, width, slope, precipitation, imperviousness, infiltration, runoff and optionally evaporation were displayed while necessary all data became as required.

a. Perviousness-imperviousness

Permeability and imperviousness of surface is the major factors that determine the rainfall-runoff volume estimation. The impervious surfaces are defined as the surfaces that prohibit the infiltration of water from the land surface into the underlying soil (Chithra, 2015). This is the percentage of the sub catchment area that is covered by impervious surfaces, such as roofs and roadways, through which rainfall cannot infiltrate. Imperviousness tends to be the most sensitive parameter in the hydrologic characterization of a catchment and can range anywhere from 5% for undeveloped areas up to 95% for high-density commercial areas (Jorge, *et al.*, 2009). In the study area the percent of impermeability was analyzed according to the geographical land use land cover the study site. The Perviousness-imperviousness of the sub catchment determines directly the surface runoff coefficients of the each. The amount and characteristics runoff not only depends on the rainfall pattern, but also on the catchment properties (Deresu, 2019).

b. Sub catchment width

An initial estimate of the characteristic width is given by the sub catchment area divided by the average maximum overland flow length (Lewis, *et al.*, 2015). The maximum overland flow length is the length of the flow path from the outlet to the furthest drainage point of the sub catchment. The flow width is one the least tangible swmm5.1 parameters and the characteristics width of overflow path for street flow runoff (Deresu, 2019). Mathematically formula of

$$\text{Width} = \frac{Aea\ of\ sub\ catchments}{longest\ path} \qquad 2.2$$

c. Infiltration

Infiltration is expressed as the down ward movement of water from rainfall based on land cover types. Soil type is the most factors that determines infiltration rate. The infiltration rate of any given soil also varies over a rainfall event, usually decreasing significantly as the soil becomes saturated (Zachary, 2015). Infiltration is the process of rainfall penetrating the ground surface into the unsaturated soil zone of pervious sub catchments areas. SWMM5.1 offers four choices for modeling infiltration: among them Green-Ampt method was selected to run simulation. This method is used for modeling infiltration assumes that a sharp wetting front exists in the soil column, separating soil with some initial moisture content below from saturated soil above. The input parameters required are the initial moisture deficit of the soil, the soil's hydraulic conductivity, and the suction head at the wetting front. The recovery rate of moisture deficit during dry periods is empirically related to the hydraulic conductivity.

d. Slopes

The slope parameter tells the amount of inclination and the SWMM5.1 sub catchments are conceptually represented as rectangular planes and its all flow is directed perpendicularly toward one of the edge of the rectangle (Deresu, 2019). Average percent slope of the sub catchments are the key factor that determine elevation difference of each node flow. Slope analysis is the basic point to transport runoff effectively in the network link and node by identifying the downstream elevation from the highest one in order to characterize the flow direction and accumulation. Most of the time downstream or outlet environment has the probability to expose for extreme surface runoff. The analysis of slope around study area can be determined by differencing the higher altitude from lower altitude over the distance of each altitude.

2. Nodes

Junctions are drainage system nodes where links join together. Physically they can represent the confluence of natural surface channels, manholes in a sewer system, or pipe connection fittings. External inflows can enter the system at junctions. Excess water at a junction can become partially pressurized while connecting conduits are surcharged and can either be lost from the system or be allowed to pond atop the junction and subsequently drain back into the junction (Lewis, *et al.*, 2015). The elements determined at junctions are invert elevation depth lateral inflow total inflow flooding. Outfalls are terminal nodes of the drainage system used to define final downstream boundaries under kinematic Wave flow routing. Only a single link can be connected to an outfall node. Type of outfall boundary condition is free in outfall stage determined by minimum of critical flow depth and normal flow depth in the connecting conduit.

3. Links

Storm water runoff is transported through conduit, channels or pipes from first station point of rainfall to or other node in drainage system. The shapes of the conduit were adjusted with SWMM 5.1 software and it could be rectangular, trapezoidal, circle, and others. When filling data on the elements of drainage structures in SWMM 5.1 software the length of conduit, shape of conduit, inlet and outlet of the node, maximum depth and others were set by clicking on it and fill the well fit dimensions value. The flow pass in the drainage system had been determined by using manning formula. While using this formula to determine the discharge within the conduit or channel of the structures; the cross sectional area of conduit or channels, hydraulic radius, wetted perimeters, slope of drainage structure installation could be considered within the models. The discharge through the channels could be determined below the following formula.

$$\mathrm{Q} = \frac{A * \mathrm{R}^{2/3}\mathrm{S}^{1/2}}{n} \qquad 2.3$$

2.12 Applicable software's in the study

There were different computer applications inclusion during the study such as; Google earth to classify the study area into a number of sub catchments based on physical properties and land use land cover, arch map 10.3.1 to extract the land use land cover of the town and soil type map, storm water management modeling version 5.1 software to simulate the input parameters for checking the data consistency relative to generated result. Microsoft excels was also used to analysis of statistical metrology data, to draw intensity-duration-frequency curve of various historical records of precipitation, to draw the graph of any required data in the study progress. However, storm water management model could be used for simulation because; it is developed with environmental protection agency and it provide fast response to the researcher where mistake has been done with reasonable evidences and information's are included in it to model drainage system. In addition, suitable to rainfall-runoff analysis with precision as well as accuracy because of the study area is highly exposed to rainfall dependent effects.

2.13 Calibration and validation

The parameters that calibrated within separate storm water drainage systems are to check the consistency of the software based on the displayed result. Storm water management modeling parameters should typically be calibrated and validated against measurements to reach reliable results (Belachew, 2019). The parameters that calibrated in the storm water drainage system were sub catchments runoff, node depth, link flow rate, node flooding, outfall loading and peak runoff discharge. The model would simulate for continuous time series of precipitation within drainage system network by feeding the rainfall or precipitation data with its required various physical elements property and dimensions in SWMM5.1. After all this process and entering of the required data, calibration will carry out for peak discharge of runoff around the point of outlet to identify the depth of runoff in the separate storm water drainage system. The storm water management model 5.1 will validate through the surface runoff and flow routing from all sub catchments to an outlet of study area by checking the successfulness of model for urban drainage system on selected study site. The peak runoff determination on study site will validate by SWMM5.1 models by comparing rational methods. In this study parameters that will validate are the methods of peak runoff discharge analysis, percent of continuity errors that displayed on the surface runoff, flow routing of installed drainage network.

CHAPTER THREE

METHODS AND MATERIALS

3.1 Description of Study area

Bonga is a town of Kaffa zone and found in south nation nationality and people regions (SNNPR). The town has three kebeles and far 454 and 784km from capital city of Addis Ababa and regional city of Hawassa respectively. It is located between $07^0 11^'$ 03''- 07^0 22' 05'' North and between 36^0 11' 44''-36 $^0 15^'$ 57 East of the zone. The annual rainfall with mean monthly values found between 125-250mm whereas, the maximum and minimum temperature exists between 32^0C and 5^0C respectively. The population of town was 20, 855 in 2007 E.C according to central statically agency (CSA) data. The following Figure 3.1 indicates that the locations of study area in accordance to country, regional, and Zonal level from where it was extracted.

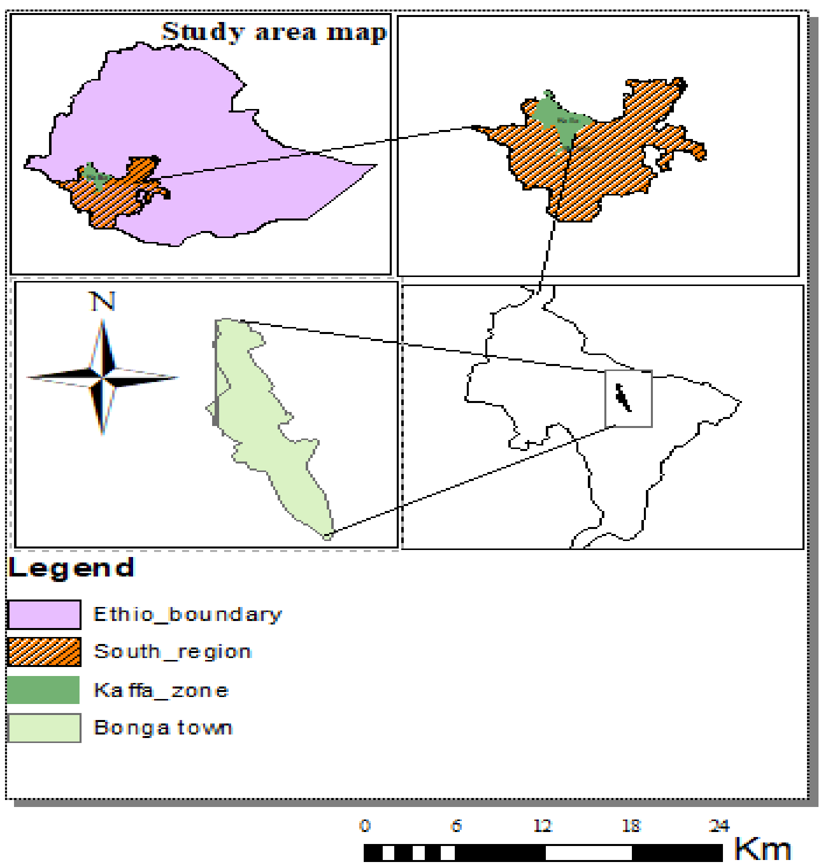

Figure 3.1: Bonga town administration boundary map

3.2 Design

In Bonga town of study site storm water drainage system assessment, the topography condition, elevation difference and flow accumulation was conduct on Google earth and arch map 10.3.1 respectively. In selected Catchment of the study area the analysis was done based on spatial or geographical landscapes, developed status of each sub catchments and existing plan of road access. The study design from data collection to conclusion and recommendation was shown in the flow diagram of figure 3.2.

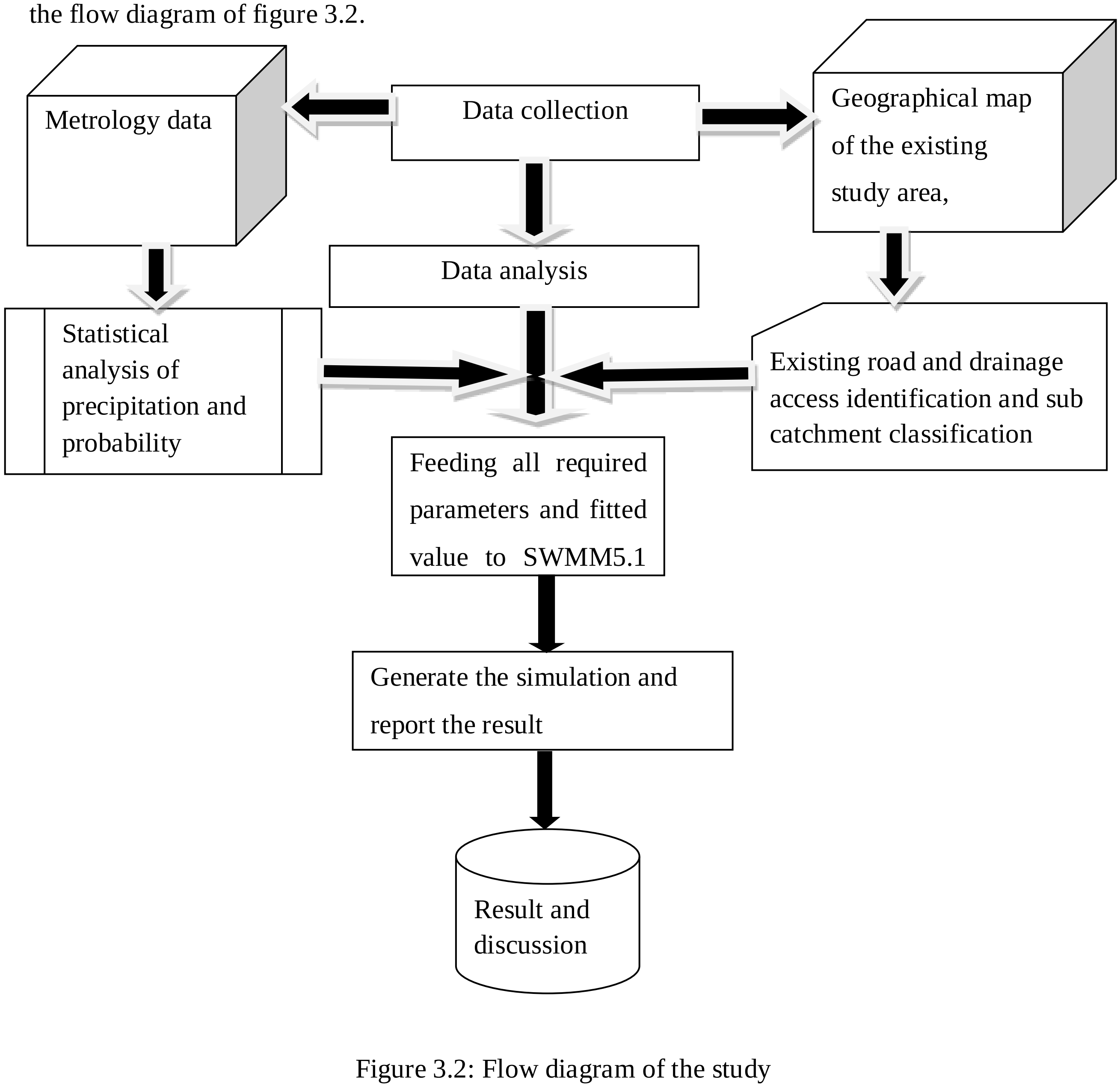

Figure 3.2: Flow diagram of the study

3.3 Road access and drainage coverage in the selected site

The town road access coverage's were asphalt, coble stone, gravel and earthen from central main to internal sub main parts of selected site of the town as identified by site observation from the current existing access. In the existing drainage system of the town only the main road (asphalt) side ditches were constructed with rectangular that cover with slab and while, the sub main or internal ditches were constructed also by rectangular shape but not covered at the top. The red color line indicated in figure 3.3 represents that the outline of existing masonary drainages relative from yellow colored coverage of road access network in a selected site of the town.

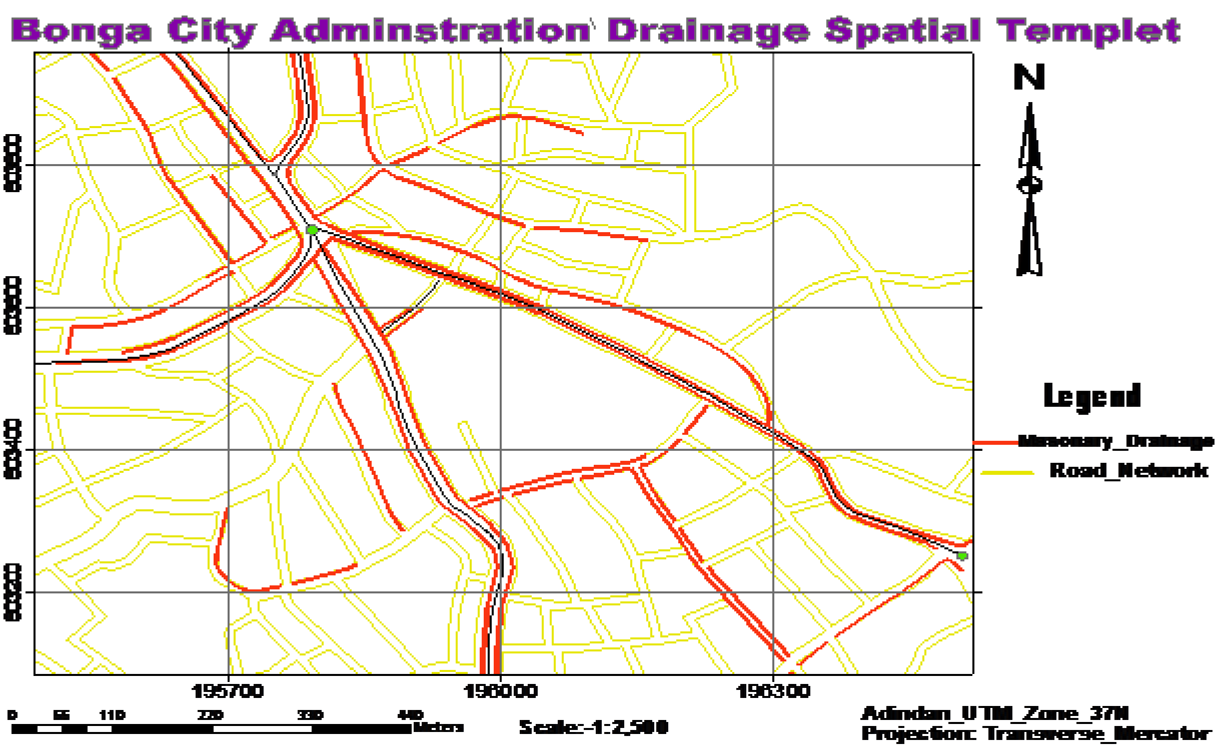

Figure 3.3: Existing road access and drainage coverage map (source: city municipal office, 2012)

3.4 Vegetation cover

A few years later, kaffa zone around Bonga town was covered by a high density of forests and vegetation's. However, now a day land use in Bonga town is mostly cover by residential houses, public institutions and sparsely grown vegetation's. At the peripheral parts of the town the vegetation like grasslands and artificial vegetation's are slightly good relative to the central part of town as shown on the land use land cover of Bonga town.

3.5 Land use land cover

Land use land cover identification in a certain area is required to determination of rainfall-runoff relationships relative to the amount of loses through infiltration, interception, evaporation and others from precipitation after reach the earth's surface. Understanding LULC change is one of fundamental importance for environmental monitoring, urban planning, and governmental decision making around the world (Nega, 2016). Sub catchments of land use land cover in selected site were the factor that determines the runoff coefficient. The value of runoff coefficient in a sub catchment also determines peak discharge at designed outlets. This means that area of sub catchments with impervious cover such as paved street, buildings, parking lots has the high probability to produce the surface runoff while, area of sub catchments with pervious cover such as vegetation cover, undeveloped part has the probability to produce low amount of surface runoff.

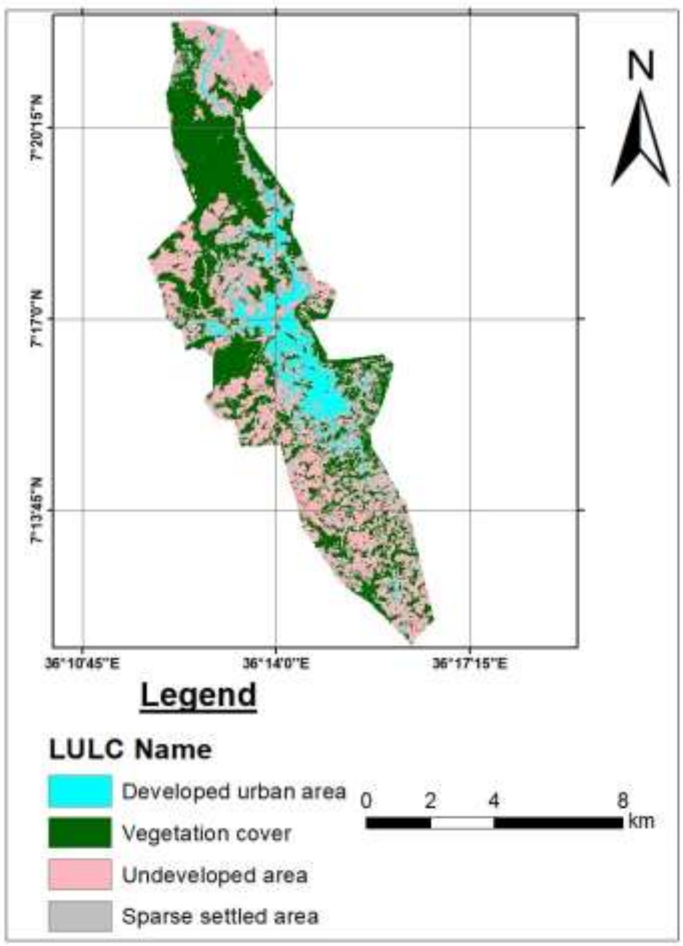

Figure 3.4: Land use land cover of Bonga town (source: arc map 10.3.1)

3.6 Soil type

The type of soil determines runoff coefficients based on infiltration rate capacity of the given soil group in a particular location. According to soil conservation service curve number method, there are four classifications of soil groups (A-D). The group of soils has their own runoff production potential from the range of low to high and those values has its contribution on saturated conductivity mm/hr as shown in the table 3.5. In this study area group-B soil is available and it helps to know the value of hydraulic conductivity for setting its value the SWMM 5.1 software.

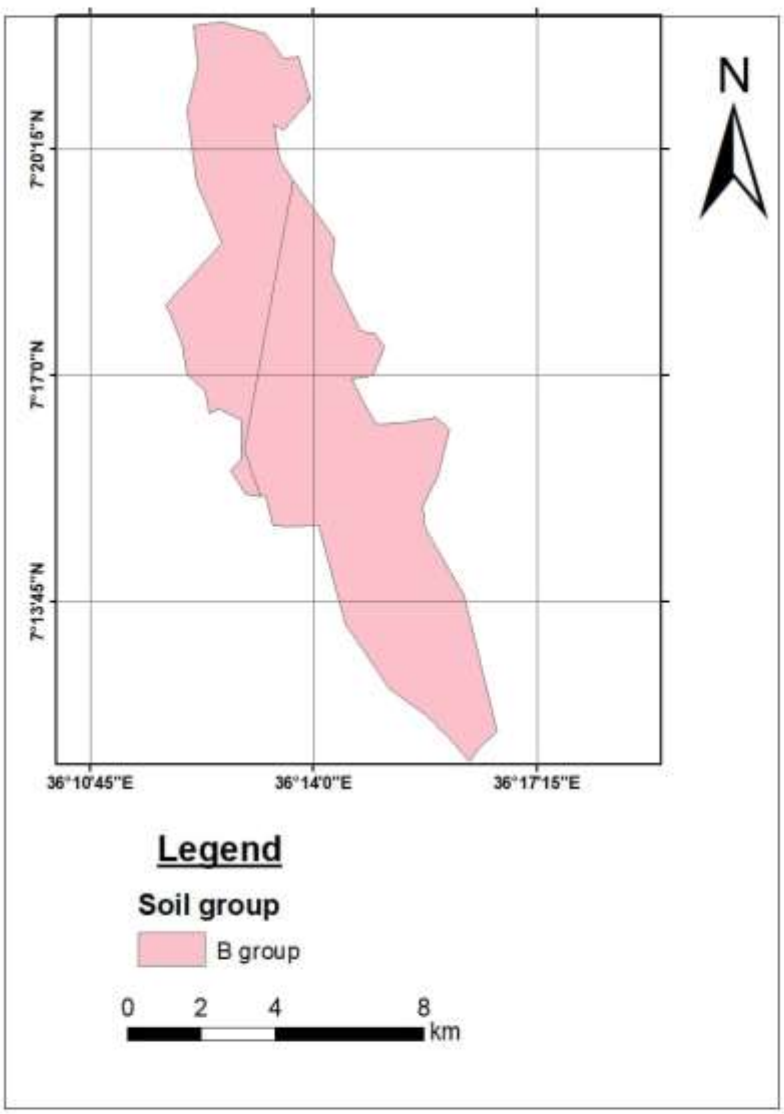

Figure 3.5: Soil group of Bonga town (source: arc map 10.3.1)

Table 3.1: The group of soil and is saturated conductivity

Group	Meaning	saturated Conductivity in/hr.
A	Low runoff potential. Water is transmitted freely through the soil. Group A soils typically have less than 10 percent clay and more than 90 percent sand or gravel and have gravel or sand textures.	> 1.42
B	Moderately low runoff potential. Water transmission through the soil is unimpeded. Group B soils typically have between 10 percent and 20 percent clay and 50 percent to 90 percent sand and have loamy sand or sandy loam textures.	0.57 - 1.42
C	Moderately high runoff potential. Water transmission through the soil is somewhat restricted. Group C soils typically have between 20 percent and 40 percent clay and less than 50 percent sand and have loam, silt loam, sandy clay loam, clay loam, and silty clay loam textures.	0.06 - 0.57
D	High runoff potential. Water movement through the soil is restricted or very restricted. Group D soils typically have greater than 40 percent clay, less than 50 percent sand, and have clayey textures.	< 0.06

(Source: NRCS, U.S. Department of Agriculture, January 2009).

3.6.1 Slope

Slope is expressed as the difference between vertical altitudes to horizontal distances and it uses to identify the condition of geographical landscapes. According to drainage system, the higher elevated part of the town has the probability to less infiltration rate due to sufficient slope and produce high runoff relatively lower elevated one. The slope of whole Bonga town is exist within the range of zero to more than 20%. However; the study site was exist in the range between 1 to 7% that colored in blue to green selected. Generally, it means that as elevation increases infiltration rate became decreases and vice versa for lower elevation. The figure 3.6 represents that the slope of new developed map of the Bonga town.

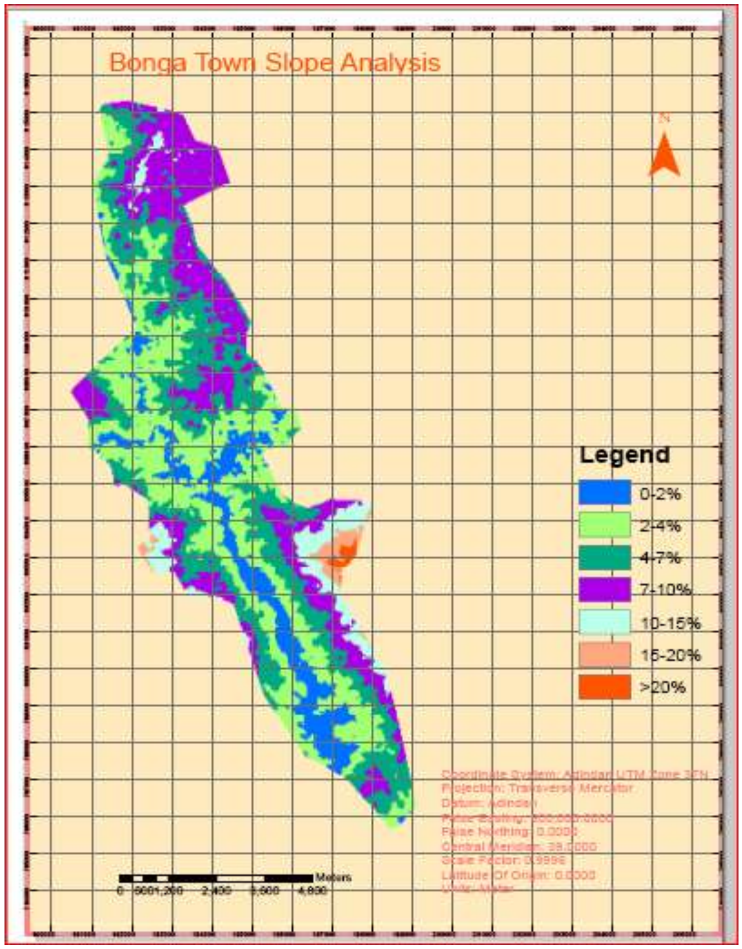

Figure 3.6: slope analysis of the study area (Source: Bonga town municipal office)

3.7 Topography and climate condition

The topography of the Bonga town is surrounded by mountain with including the sparsely settled forest at the periphery of the newly planned town. The topography of landscapes determined by its slope difference and the condition of topography affect the infiltration of surface runoff when flowing on overland surface. The weather condition of Bonga town is mostly temperate zone during summer season. The need of showing the elevation profile on selected study area was to fix the outlets point accordingly. The topography of the study site elevation ranges between 1664 and 1778m as shown in the figure 3.7.

Figure 3.7: Elevation profile of the study area

3.8 Data collection

3.8.1 Primary data collection

The images taken on field represented in the figures 4.2, 4.3 and in an appendix part A1-A3 were to identify the real status of existing storm water drainage systems, functioning, stressed with runoff and sediment at various parts of the study site. Additionally, primary data's were collected on field by measuring the dimensions of existing ditches on various main and sub main drainage points within selected study area.

3.8.2 Secondary data

The secondary data's such as precipitations, minimum and maximum temperatures and elevations were collected from national metrological agencies of Ethiopia data user office. The current urban plan and existing urban road access as well as storm water drainage system of the town were taken from Bonga City mayor and municipality office. Different previous research papers, articles and journals were used as reference to develop this thesis work.

3.9 Variables

3.9.1 Dependent variable

Dependent variable in this study is assessment of the storm water drainage system in Bonga town within affected site due to poor drainage system by simulating on SWMM 5.1 software.

3.9.2 Independent variables

Independent variables are the variables which could affect the dependent variable either negatively or positively to precede the output. Under the study the factors that affect the assessments were rainfall amount, topography, slope of the town, land use land cover of each sub catchment's, shape of the drainage system and roughness of the channels.

3.10 Hydrological data analysis

3.10.1 Rainfall analysis

In order to estimation of storm water runoff peak discharge in the drainage network, available historical rainfall data has to be required. The rainfall intensity-duration-frequency relationship is the basic point to predict the storm water runoff for different return periods and duration of the rain falls within certain study area (Hassen, 2016). Therefore, a rainfall record was obtained from the national meteorological agency of Ethiopia for Bonga town station for consecutive forty seven years. By this available historical rainfall data for the station of Bonga could collect and analyzed in order to prepare the necessary depth or intensity of extreme rainfall input data for peak discharges computation. The analysis and processing was aimed for determination of appropriate intensity-duration relationship to differentiate the worst/severe scenario events of precipitation.

3.10.2 Missing value estimation

The point observation from precipitation gauge may have a short break in the record and missed due to several factors such as: failure of the instrument, absence of observer. Rainfall data having significant portion of the historic record missing, needs to be estimated (Belachew, 2019). The historical daily rainfall data of Bonga town was obtained from national meteorological agency according to the request letter of environmental engineering chair. However, from forty seven consecutive years (1970-2016) precipitation data, there was a certain missed daily precipitation data at various duration. To fill those missed data there are different missing data determination methods such as simple arithmetic method, Normal ratio method, Modified normal ratio method, Inverse distance method Linear, and programming method. But in this study simple arithmetic method was selected due to the simplicity and accuracy of the results.

3.10.3 Simple arithmetic method

Simple arithmetic average is one of missing data determination method in which the unknown value of precipitation. In some extents the precipitation data became missed in Bonga town due to the recording failed emergency. However, around various neighbors' shebe, Cida and Wushwush stations there were recorded data at the same time. In the determination of hydrological data less than 5% precipitations' were missed with various problems identifying while analyzing the available data to find daily extreme precipitation that received from NMA of Ethiopia. The allowable maximum missed data should be 25 to 30% to preceed the further study analysis (Shuvayan, 2015). Therefore, it could be get by using simple arithmetic method to find the maximum precipitation that may occur within missed range of data.

$$\mathrm{Px} = \frac{1}{3}(\mathrm{Pa} + Pb + Pc) \qquad 3.1$$

Where, P_x- unknown precipitations P_a, P_b and P_c are known precipitations.

3.10.4 Checking the consistency of rainfall data

The consistency of rainfall data at recorded station of study area corrected by double mass curve. Double mass curve was developed from the cumulative rainfall data series of study area and the neighbor stations to verify the accuracy of recorded rainfall data. It is evaluated with the coefficient determination or R-square value. Therefore, as shown in the figure 3.8 double mass curve was corrected through statistical coefficient determination value (R^2= 0.9977). Already it closes to 1 or linear line and this indicates that it is best consistency between the study area and neighbor's station recorded precipitation values.

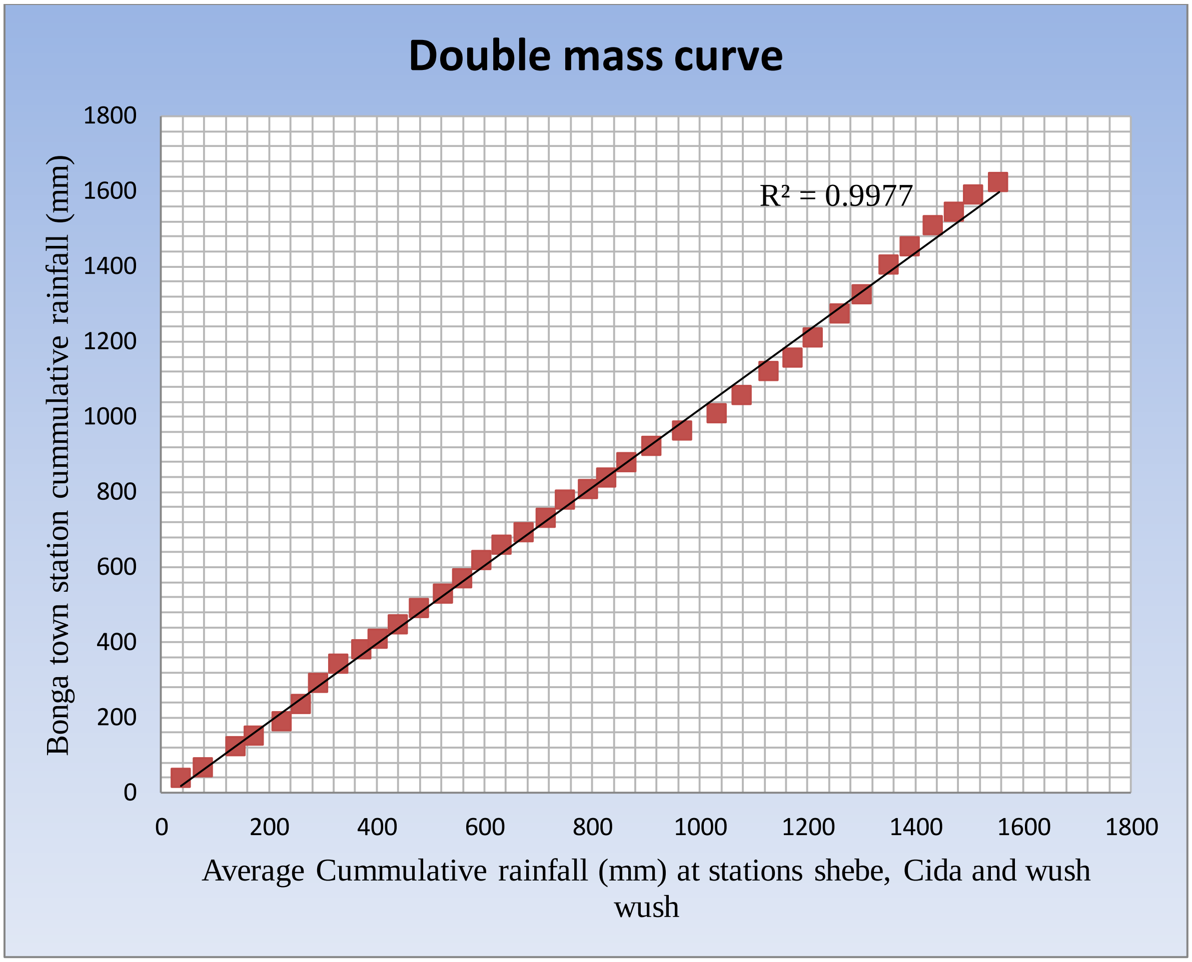

Figure 3.8: Double mass curve to check consistency

3.10.5 Extreme rainfall data analysis

Rainfall data was taken from national metrology agency of Ethiopia as daily with international standard units of mm. However, within 28, 30, 31 days of the different months in a year due to various problem missing of data was occurred. To determine those missed data for this thesis the neighbor (Cida, shebe and wush wush) town's station data was used. By using simple arithmetic method the missed data could be determined but no precipitation value was taken in the list of extreme value. Therefore, after filling all missed data within Bonga town station for consecutive forty seven years, the extreme or maximum daily precipitation was selected for the assessment of the existing and simulating of drainage system structures size by using SWMM 5.1 software. The table 3.2 shows that daily maximum precipitation data 1970-2016 Bonga town.

Table 3.2: Extreme rainfall analysis of Bonga town

Years and dates of	Max. daily rainfall	Max and min temp		Date of records	Max. daily rainfall	Max and min temp	
11/4/1970	58.4	24	13	25/7/1994	50	25.7	14
9/8/1971	33.2	21	12	23/4/1995	39.2	26	9.5
17/7/1972	46.4	23	8.5	6/8/1996	35.2	24	10
2/7/1973	47.4	25	6.5	3/11/1997	38.4	28	11
20/5/1974	46.8	22.5	5.5	13/8/ 1998	48.6	24.5	11.5
23/5/1975	48.7	23	7	19/7/1999	28	24.8	11.4
10/5/1976	40.5	25	10.5	10/8/2000	29.6	24	14.6
1/7/1977	47.5	21.5	8	1/8/2001	40.5	23.5	13.5
6/5/1978	75.4	24	10	6/3/2002	42.8	25.7	11.5
1/5/1979	64.5	26.5	12	11/4/2003	41.4	29	13
21/8/1980	37.8	24.5	7	12/5/2004	46.1	20.5	13.4
2/2/1981	28	32	15	3/5/2005	49.2	25	13
21/7/1982	54.5	25.5	13.5	3/8/2006	63.3	27	12.5
16/7/1983	30	26	12	29/5/2007	36.7	27	13.5
29/2/1984	38.3	24	5	28/10/2008	52.1	27	12
15/5/1985	44.2	17.3	13.8	30/11/2009	65.5	27.5	8.5
18/7/1986	56.7	26.5	11.5	5/9/2010	49.7	26	12.5
4/12/1987	50.2	27.8	12.7	5/4/2011	80.1	29	5
6/6/1988	39.9	25.7	12.7	16/6/2012	48.8	26	11.5
3/6/1989	27.8	30	10	14/6/2013	55.7	25.6	12.5
17/2/1990	37.2	29	14	6/5/2014	36.3	29	13
7/5/1991	44.5	26.7	14.5	27/4/2015	45.6	29	13
30/8/1992	37.6	26	9.9	3/5/2016	32.9	26	9
13/5/1993	40.4	24	9.2				

3.11 Probability distribution

Frequency analysis of hydrological data requires probability distributions related to the magnitude of extreme events to their frequency of occurrence. The general idea for this assumption is the probability of maximum rainfall re occurrence events once within simultaneous years (2, 5, 10, 25, 50 and 100). However, generally various applicable probability distributions are used to determine maximum precipitations within particular study area to know the worst scenarios of rainfall event that may lead to risk of flooding.

- Gumbel extreme Value type I distribution
- Log Pearson Type III distribution
- Normal distribution
- Log-normal distribution (two parameter)

However, for this study the extreme value type I distribution, also known as the Gumbel and log Pearson type three distributions were selected to compare the most fit method for rainfall distributions and for various reoccurrence periods to the available rainfall data. Estimates of maximum rainfall depths for different return periods (T) were obtained by statistical technique of frequency analysis on excel work sheet. The extreme value type I distribution and Log Pearson Type III distribution selection was carried out in terms of determination of coefficient value.

i. Gumbel Extreme Value distribution

The type of distribution method in which the average value of the historical precipitation data, the multiple of frequency factor and standard deviation determine the extreme rainfall depth. The mathematical expression of this method is indicated as the following equation.

$$\mathrm{XT} = Xavg + \mathrm{KT} * \mathrm{Sy} \qquad 3.2$$

Where: X_T- Frequency of Rainfall depth (mm) at return period T (years), X_{avg}-Mean value of logarithmic daily rainfall data (mm), Sy - Standard deviation (mm), K_T - frequency factor and express mathematically in the equation 3.3.

$$\mathrm{KT} = \frac{-\sqrt{6}}{\pi}[0.5772 + \ln(\ln(\left(\frac{T}{T-1}\right))] \qquad 3.3$$

This method determines the extreme rainfall at various scenarios of return periods by using the parameters (average value of rainfall, standard deviation and frequency factor) by using equation

3.2. However, the probability distribution method was selected for further analysis based on the R^2 value. The Gumbel method of probability distribution was calculated in the table 3.3.

Table 3.3: Gumbel probability distribution method

Gumbel method, XT=Xm+Sy*KT		
T	KT	XT
2	-0.164	42.8
5	0.7198	52.3
10	1.305	58.6
25	2.0445	66.5
50	2.592	72.3
100	2.912	75.8

ii. Log Pearson Type III distribution

The type of frequency probability distribution expressed in terms of logarithm annual maximum precipitation and its average value, standard deviation and skewness coefficient of the data. Log Pearson type III distribution is a three-parameter gamma distribution with a logarithmic transform of the variable (Zewdu, 2015). Mathematically expressed as the equation 3.4.

$$\mathrm{xT} = 10^{YT} \qquad 3.4$$

Where; YT-logarithm value of maximum rainfall.

$$\mathrm{YT} = Yavg + \mathrm{Sy} * \mathrm{KT} \qquad 3.5$$

Where, K_T -logarithm frequency factors, Sy-standard deviation, Y_{avg}- mean value of logarithm function of precipitation.

$$\mathrm{KT}=z+ (z^2-1)\, k + \frac{1}{3}(z^3-6z)\, K^2 - (z^2-1)\, k3 + zk^4 + \frac{1}{3}k^5 \qquad 3.6$$

Where, k is coefficient and can be calculated as skewness divided by 6, coefficient z can determine from one over return period and enter into normsinv from statistical formula at excel work sheet. The analysis of this method has many steps and more related calculations on the excel worksheet to get results that presented in the table 3.4 by using the equations 3.4-3.6.

Table 3.4: Log Pearson type three distribution system

Log Pearson type three distribution system			$\text{YT} = \text{Log mean} + \text{Sy} * \text{KT}$ and $\text{XT} = 10^{YT}$			
T	1/T	Z (Normsinv (1/T))	k	KT	YT	XT
2	0.5	-1.39E-16	-0.0410675	0.040998189	1.638586571	43.5
5	0.2	-0.841621234	-0.0410675	-0.827161847	1.713712589	51.7
10	0.1	-1.281551566	-0.0410675	-1.304751959	1.759351233	57.5
25	0.04	-1.750686071	-0.0410675	-1.832459678	1.80977913	64.5
50	0.02	-2.053748911	-0.0410675	-2.183624771	1.843336564	69.7
100	0.01	-2.326347874	-0.0410675	-2.506465246	1.874187289	74.8

i. probability distribution comparisons

Graphical representation is a method of reporting the analyzed result and the consistency of data. For selecting the best fit method of probability distribution, R-squared value has its own governing interval (0-1). Therefore, in this study a higher R-square value has scored on log Pearson type three probability distribution relative to Gumbel or type one extreme value method. Therefore, log Pearson type three distributions was selected to determine the further analysis. The trend line drawn on vertically coordinated precipitation data and horizontally coordinated reoccurrence period as indicated in the figure 3.9.

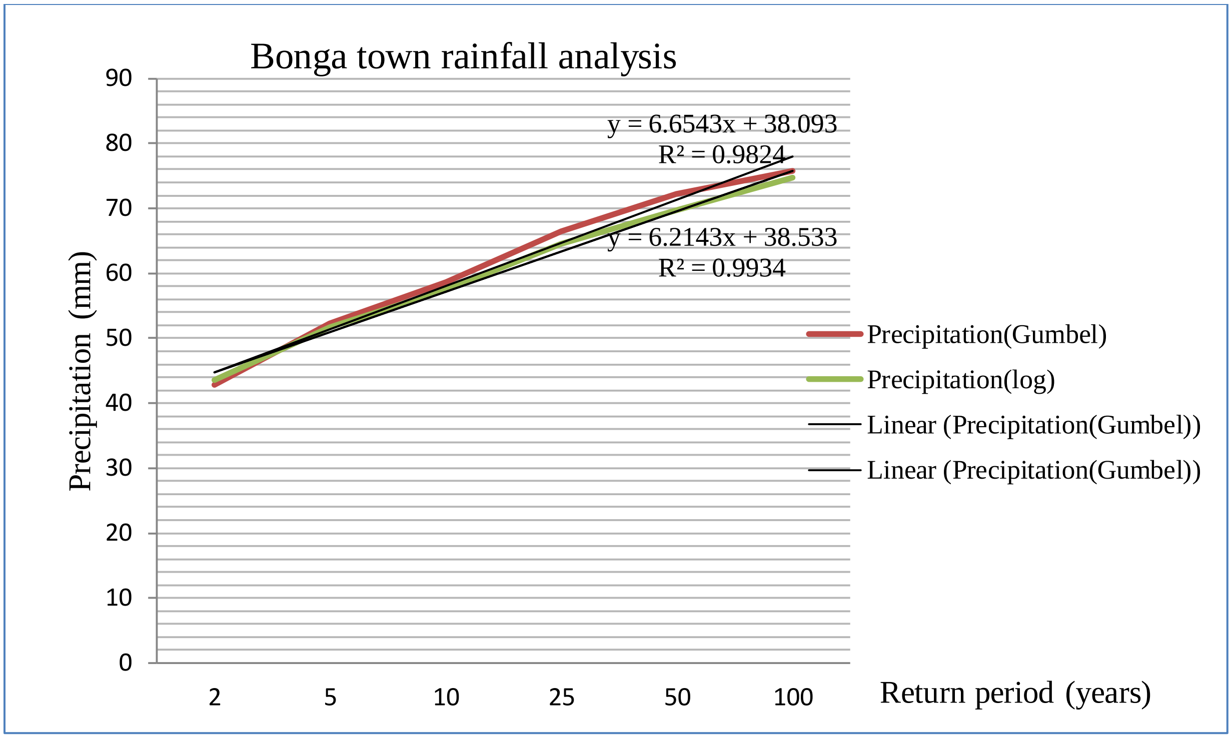

Figure 3.9: Probability distribution method of Bonga town rainfall

3.11.1 Design rainfall of shorter duration

In small watershed hydrology, the peak discharge is related to the time of concentration and, through the applicable intensity-duration-frequency curve, to the rainfall intensity. Small drainage areas would have a short time of concentration and this could produce a high intensity. However, since the area is small, the peak discharge will be correspondingly small. The maximum rainfall depths obtained from gauging station were 24hrs duration. Design and analysis of drainage structures require rainfall intensity, duration and its reoccurrence intervals relationship of shorter duration. Because rainfall data of shorter duration is unavailable, it is necessary to find the plot of appropriate IDF curve to identify the worst scenarios of rainfall falling rate to various return periods and this has low accuracy. By using Ethiopian road authority it is possible to find the short duration rainfall. Maximum rainfall amount was taken from NMA as daily and this was reduced to as required convenient durations both hourly and in minutes to get more accuracy of the intensity values. The available rainfall amount for this thesis was reduced to 12, 6, 2, 0.5 and 0.25 hours of short durations. For conversions of the maximum daily rainfall to the above short duration, Ethiopian road authority formula was used and shown in the table 3.5.

$$Rt = \frac{t}{24}\left(\frac{b+24}{n+t}\right)^{n} * R24 \qquad 3.7$$

Where: R_t-Rainfall depth in a given duration t; R_{24} -24hr rainfall depth; Coefficients (b -0.3 and n=0.92) according to the manual of (ERA, 2003)

Table 3.5: Short duration and intensity analysis

Duration(min)	Return Period (years)					
	2	5	10	25	50	100
	max (R24) for various return periods					
	43.5	51.7	57.5	64.5	69.7	74.8
	$I = \frac{R24}{24}\left(\frac{b+24}{n+t}\right)^{n}$					
15	59	70	78	87.7	94.8	101.7
30	41.9	49.8	55.4	62	67	72
60	26.8	31.85	35.43	39.7	42.95	46
120	15.86	18.85	21	23.5	25	27.3
360	6.3	7.46	8.3	9.3	10	10.8
720	3.4	4	4.5	5	5.4	5.8

3.11.2 IDF curve developing

An IDF is a three parameter curve, in which intensity of a certain return period is related to required duration of rainfall event. IDF curve enables the researcher to develop hydrologic systems by identifying the worst case scenarios of rainfall intensity and duration during a given interval of time. If local rainfall data is available, IDF curves can be developed using frequency analysis and minimum of 20 years data is desirable to develop. IDF curve was constructed by using the intensity that determined through short durations and basically aimed to identify the highest intensity to enter it as input on SWMM5.1 model instead of rain fall to generate runoff. After analysis of short durations, 15 minutes duration were scored the maximum falling rate of rain or intensity (101.7mm/hr) in Bonga town at 100 years of return period as calculated on the table 3.5. The frequency analysis of precipitation data was done for 2, 5, 10, 25, 50 and 100 years and shown in the figure 3.10.

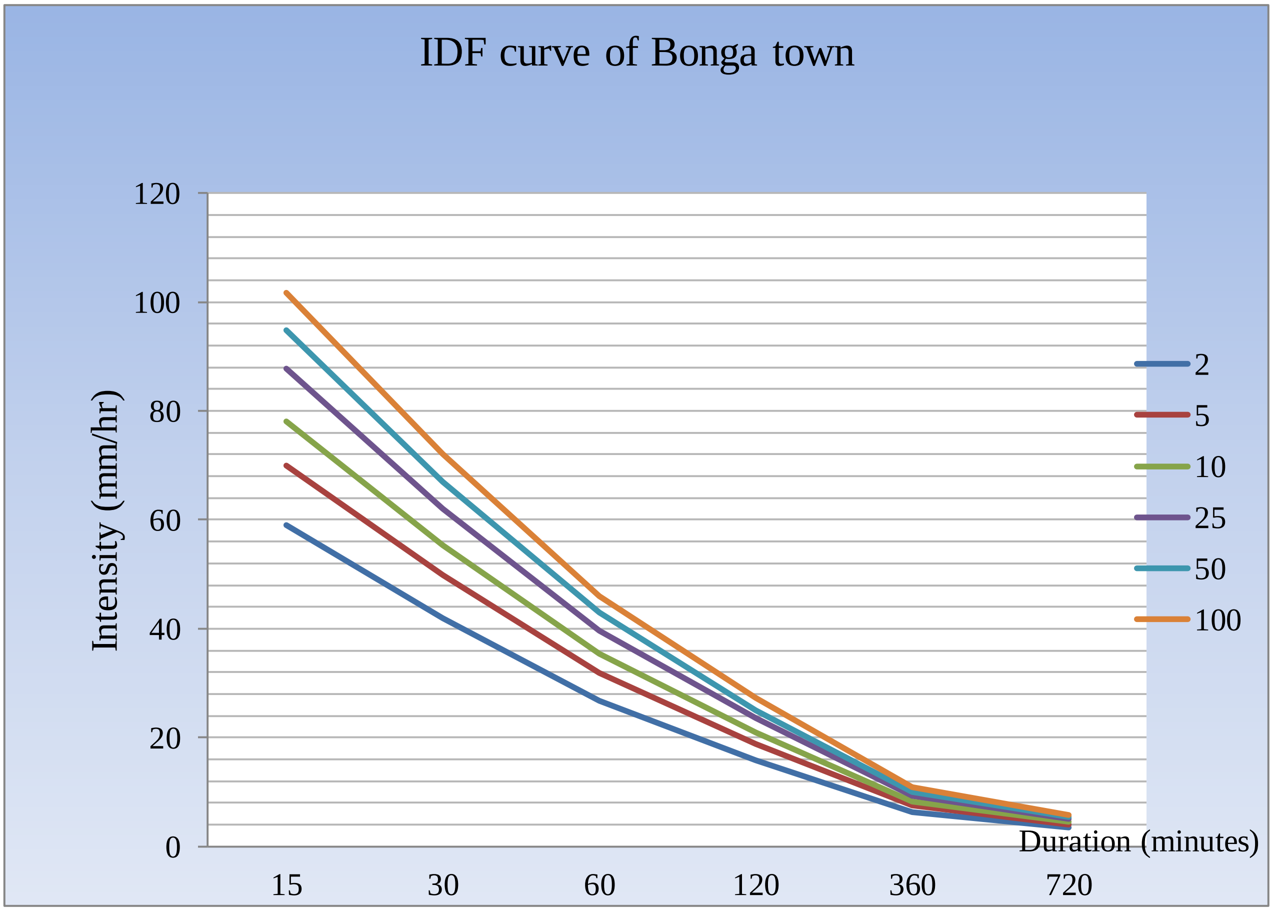

Figure 3.10: Intensity, duration and frequency graph

3.11.3 Land use land cover of selected study area

In Bonga town the selected Study area was more developed in comparison to others parts of the town and the problem of storm water runoff causes to damaging of roadway and disrupt the urban environment's sanitation. However, the topographical conditions were not suitable at all round of developed and settled study area of the town to convey the surface runoff simply. As the result, draining of the storm water surface runoff from the urban part was not simply manageable and makes the existing drainage capacity stressed during the heavy rainfall event. The land use land cover with commercial or developed part that indicated as purple color in the figure 3.12 of LULC has the probability to produce high surface runoff and contribute peak runoff to the undeveloped and residential area of downstream parts by increasing outfall loading. The aim of classifying study area with its land use was to identify the percent of impervious and pervious area that determine the runoff coefficient during setting the SWMM5.1 model.

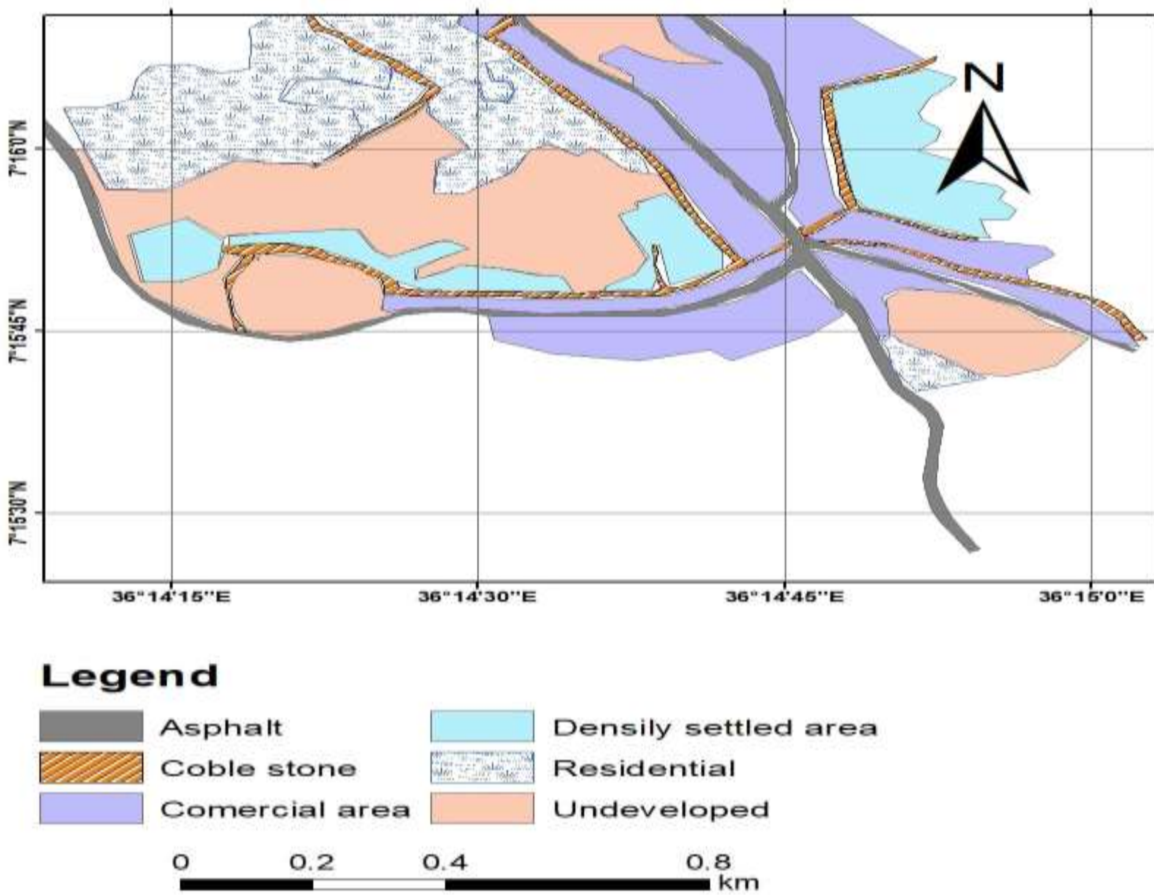

Figure 3.11: Land use land cover of selected study site (source: arc map 10.3.1)

3.12 Determination of peak runoff

3.12.1 Rational method

In the determination of storm water peak runoff discharge rational method was conduct for the sub catchments its area less than 50 hectares. In the design of storm water drainage system, the main purpose of hydrologic analysis is to determine the maximum amount of run-off or peak discharge that can be accumulated at certain storm drainage outlet usually a ditch along an access road alignment section (Belachew, 2019). The various parameters taken as the factor that determines the rate of runoff discharge in each sub catchments were land use with high percent of impervious, time duration, area and precipitation amount. Area of sub catchments could be known by classifying the selected study catchment into a numbers based on the existing plan of the town. The mathematical formula of rational method is as follows:

$$Q = 0.00278CIA \qquad 3.9$$

Where; C-coefficient of runoff, I-intensity and A-area of sub catchments

However, runoff coefficient determination was done by identifying the impervious and pervious land use from each of the sub catchment and calculating of the weighted C by using the equation 2.1. The table 3.6 indicates that the various runoff coefficient values for various land use land cover that taken from the manual of (ERA, 2013).

Table 3.6: Runoff Coefficient value for various drainage areas (source: ERA, 2013)

Type of Drainage Area	Runoff Coefficient C
Business: Downtown areas	0.7-0.95
Neighborhood areas	0.5-0.7
Residential: Single-family	0.3-0.5
Multi units, detached	0.4-0.6
Multi units, attached	0.6-0.75
Suburban	0.25-0.4
Residential (0.5 hectares lots or more)	0.3-0.45
Apartment dwelling areas	0.5-0.7
Industrial: Light areas	0.5-0.8
Heavy areas	0.6-0.9
Parks, cemeteries	0.1-0.25
Playgrounds	0.2-0.4
Railroad yard areas	0.2-0.4
Unimproved areas	0.1-0.3

i. Time of concentration

The time taken by storm water from longest distance of sub catchments to outlet point on which peak discharge to be determined. In Rational Method time of concentration was used to determine the intensity of rainfall per desired duration which would result in maximum runoff (Asfaw, 2016). Flow path from the upstream sub catchment to the outlet was well defined because of the volume of the water; hence the equation developed by United States Soil Conservation Service its formula is shown in the equation 3.11

$$Tc = (\frac{0.87L^2}{1000Sav})^{0.385} \quad 3.11$$

Where; Tc- is time of concentration, L-length of the flow on sub catchments from longest path, Sav-average slope

3.13 Storm water management modeling 5.1

The model of SWMM 5.1 was developed in United States (US) of environmental protection authority (EPA). This model assumes that any types of drainage line that connects the consecutive junction nodes as conduit. Rain gage was used as a rainfall source that determined from daily maximum NMA precipitation data as intensity using short duration intervals. Because, the time series setting for intensity and rain gage time interval should be equal; unless it never display the results. However, hydraulic cross section of the existing ditches or channels was rectangular and adjusted with optimum dimensions to assess the drainage network in accordance to the available amount of rainfall in the town.

The runoff component of SWMM 5.1 operates on a collection of sub catchment areas that receive precipitation and generate runoff. The routing portion transports this runoff through a system of channels. It tracks the quantity of runoff generated within each sub catchment, and the flow rate as well as flow depth in each channels during a simulation period comprised of multiple time steps. Precipitation data and time series were the key components in the software that determine rainfall-runoff relation. The modeled data results could report with three basic themes (sub catchments, nodes and links) of legends by running the simulation for achievement of successful analysis on the window as represented at the appendix-C.

The peak discharge determination for all sub catchments in this model was done by running the simulation after entering of all necessary data. The aim of simulations is to evaluate the surface runoff flow depth through the drainage network in the study area. It is possible to minimize the effects of runoff that occur through poor drainage system by setting suitable dimensions to depth of nodes, links and outlets on SWMM 5.1 model by following their elevation differences. The runoff amount that passes through drainage system determined based on the peak runoff generated from the sub catchments. The maximum capacity of drainage sizes set on the model was $1m^3$ as existing drainage system capacity with rectangular shape and while running the simulation the storm drainage system became stressed and creates flooding in some nodes. The drainage system was modeled to convey storm water runoff safely with separate drainage system relative to the amount of runoff received from sub catchments to drainage network. The advantages of using storm water model rather than sewer cad and others software applications is integrating precipitation data with physical property of landscapes of study area and sent quick responses where the error had been occurred while running the simulation after entering all necessary data.

Rain gage was taken as a function of rainfall source and sub catchments were receive this rainfall and generate runoff toward the connected nodes according to their land use land cover. The amounts of available precipitation data in the town was enter as intensity form to 15 minutes interval of time series within two hours of simulations. Within classified sub catchments; 121 conduits, 116 junctions and 2 outlet nodes were installed by following the road access in order to characterize the available rainfall amount with geographical landscapes of the study area.

3.13.1 SWMM 5.1 parameters

1) Sub catchments

Storm water management modeling is the software in which the coming precipitation is taken as input of rainfall source for rain gage to generate surface runoff within installed network of storm water drainage system until outfall with physical elements (nodes, links and outfalls). The storm water management modeling software display or generate the peak discharge surface runoff from input data of precipitation by considering losses that may be caused due to infiltration, interception, evaporation, storage of depression. Sub catchments were the classified part of the study area catchments and based on the direction of runoff discharge toward the common outlet points.

2) Infiltrations model

In general conditions there are four classifications (Horton, modified Horton, Green-Ampt, and modified green-Ampt) of infiltration models on which surface runoff move down with saturated soil layer. The required input parameters were the initial moisture deficit of the soil, the soil's hydraulic conductivity, and the suction head at the wetting front (Lewis, *et al.*, 2015). The Green-Ampt infiltration method was used for this study and it is targeted for determination of the quantity of surface runoff infiltrate into the ground according to soil type as well as its property (suction head, conductivity and initial deficit). The recovery rate of moisture deficit during dry periods is empirically related to the hydraulic conductivity. The table 3.7 indicates soil texture class that can determine the infiltration rate on various sub catchments.

Table 3.7: Soil texture class (Source: Rawls, *et al*., (1983)

K = hydraulic conductivity, in/hr Ψ = suction head, in. ϕ= porosity, fraction FC = field capacity, fraction WP= wilting point, fraction					
Soil Texture Class	K	Ψ	Φ	FC	WP
Sand	4.74	1.93	0.437	0.062	0.024
Loamy Sand	1.18	2.40	0.437	0.105	0.047
Sandy Loam	0.43	4.33	0.453	0.19	0.085
Loam	0.13	3.5	0.463	0.232	0.116
Silt Loam	0.26	6.69	0.501	0.284	0.135
Sandy Clay Loam	0.06	8.66	0.398	0.244	0.136
Clay Loam	0.04	8.27	0.464	0.31	0.187
Silty Clay Loam	0.04	10.63	0.471	0.342	0.21
Sandy Clay	0.02	9.45	0.43	0.321	0.221
Silty Clay	0.02	11.42	0.479	0.371	0.251
Clay	0.01	12.6	0.475	0.378	0.265

3) Routing model

Lewis, *et al*., (2015) conclude that flow routing in within a channels link in SWMM 5.1 is governed by the conservation of mass and momentum equations for gradually varied, unsteady flow (the Saint Venant flow equations). Kinematic wave model was the choice for this study to conduct the simulation analysis and it solves the problem of continuity equation along the simplified form while flow passing through conduits. The rout model is interconnected from rain gauge to outlets through junctions and channels link to convey the storm water that generate from rainfall. The accuracy of flow rout determined through the precipitation amount, sizes of drainage and condition landscapes.

i. Junction

Junctions are drainage system nodes where links join together and on which external flow entering to it. The importance of junctions in the drainage networks are during direction change of links, conduit size increasing or decreasing, and at the location of slope change. Physical elements of junction may be represented as manhole. According to Lewis, *et al*., (2015), the points that must be considered in junction relative to input parameters were elevation differences and maximum depth of the junction nodes that connect conduits/ channels. The needs of junction points in this model were done during direction change, joining of sub main line with mains and at outlets points.

ii. Conduit/channels

The SWMM 5.1 software understands any drainage line as conduit and the shape as well as its dimensions has been adjusted on it with try and error in terms of the available rainfall amount. Rectangular channels were selected in the model to assess the flow pattern through storm water drainage system of study site. According to the Lewis, *et al*., (2015), the storm water management modeling uses the manning formula to express the relation between flow rate, cross sectional area, hydraulic radius, wetted perimeter and slope in all conduits or channels. The length of conduit or channels can be measured on Google earth and also making the auto length on the map of SWMM 5.1 that found at the bottom of window. The length of conduit/channels shortness or too long between junctions can determine the percent of flow routing during simulation. In the manual of SWMM 5.1 the surface roughness or smoothness could determines the value of error percent as shown in the table 3.8 for various channel surface type.

Table 3.8: Channels types and its manning coefficient

Conduit Material	Manning n
Asbestos-cement pipe	0.011 - 0.015
Brick	0.013 - 0.017
Cast iron pipe	
- Cement-lined & seal coated	0.011 - 0.015
Concrete (monolithic)	
- Smooth forms	0.012 - 0.014
- Rough forms	0.015 - 0.017
Concrete pipe	0.011 - 0.015
Corrugated-metal pipe	
(1/2-in. x 2-2/3-in. corrugations)	

- Plain	0.022 - 0.026
- Paved invert	0.018 - 0.022
- Spun asphalt lined	0.011 - 0.015
Plastic pipe (smooth)	0.011 - 0.015
Vitrified clay	
- Pipes	0.011 - 0.015
- Liner plates	0.013 - 0.017

Source: EPA, 2015

3.13.2 Performance evaluation criteria of the model and acceptable range

The need of evaluation for model performance is to governing the error from exceeding the permissible range. The performance of model used in this study was evaluated by using statistical data analysis parameters (coefficient determination, relative error, and Nash-Sutcliffe coefficient) in excel worksheet between calculated and simulated peak discharge of runoff in all sub catchments as expressed in the equations 3.12-3.14.

I. Coefficient determination (R^2)

$$R^2 = \frac{(\sum_{i=1}^{n} (qi^{calc} - q^{calc\ avg}) \sum_{i=1}^{n} (qi^{simul} - q^{simul\ avg}))\text{^}2}{\sum_{i=1}^{n} (qi^{calc} - q^{calc\ avg})^2 \sum_{i=1}^{n} (qi^{simul} - q^{simul\ avg})\text{^}2} \qquad 3.12$$

Exist between 0 and 1 and higher square-R value tends to more fit

II. Relative error (RE)

$$RE = \frac{\sum_{i=1}^{n} | qi^{calc} - qi^{simul} |}{\sum_{i=1}^{n} qi^{calc}} \qquad 3.13$$

Its values less than 30 %

III. Nash-Sutcliffe model efficiency coefficient (RNS)

$$RNS = 1 - \frac{\sum_{i=1}^{n} (qi^{calc} - simul)\text{^}2}{\sum_{i=1}^{n} (qi^{calc} - q^{calc\ avg})^2} \qquad 3.14$$

If RNS=1, the model fit perfect; RNS >0.75, the model fit very well; 0.74< RNS<0.64, the model fit good and RNS<0, the calculated value predict more than the simulated one.

CHAPTER FOUR

RESULTS AND DISCUSSIONS

4.1 Existing drainage and road network

Drainage system in an urban area plays the critical role to the safety of environment and the health of community population. Increase in the built up areas would alter the hydrological makeup by blocking natural streams and reducing the water absorption capacity of the surface. Drainage system could construct from masonary concrete, free fabricated steel, PVC plastic conduit and others. In the study area of the town no well-designed drainage network had been constructed to convey storm water runoff that sourced from available precipitation amount of urban environment to disposal point. The existing plan of the study area was obtained from Bonga town municipal and mayor office and it clearly indicates that only small portions of the town has masonry rectangular ditches or drainage coverage. However, the size of drainage system did not consider extreme events of rainfall in a town and these leads the existing drainage became blocked with sediment and storm water surface runoff flows over the drainage system toward road access. The figure 4.1 indicates that the current image of town taken from Google earth and existing masonry drainage, road network access in Bonga town that was prepared by Bonga municipal office.

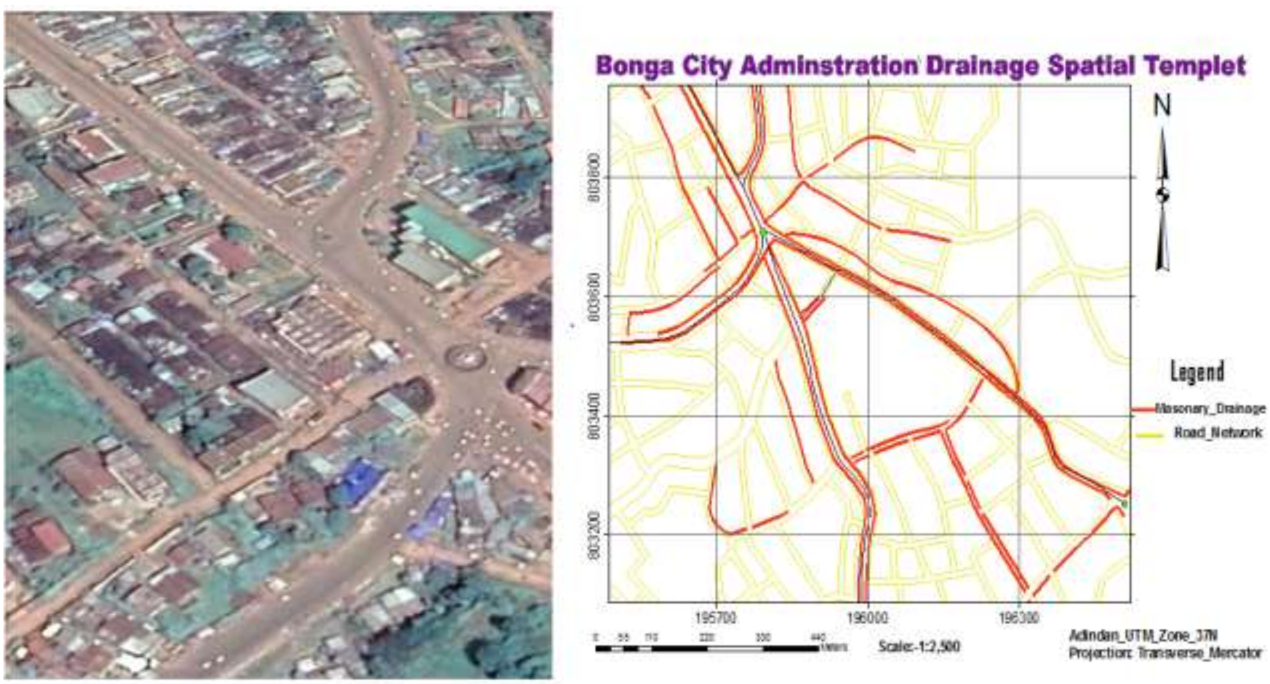

Figure 4.1: Captured image from Google earth and the existing drainage of study site

4.1.1 Classification of existing drainage in the study site

I. Rectangular ditches with slab cover

The drainage system that constructed with rectangular channels is useful simply to build, operate and maintains while closed with slab cover. The portions of drainage system that constructed with geometrical shape of rectangular by following the main asphalt road access was 3001.59m far that financed with Bonga town mayor, municipality office and federal road authority as an inventory document of Bonga town. At this drainage system, peak runoff flows through inside of the ditches during the low rainfall events and whereas at high rainfall events, the flow overtops from ditches to Road Street and allow the accumulation of mud on it. Around line between J13 and J56, at the back of line between J40 and J90 and also back of line between J18 to J20 were exemplary affected sites during heavy rain events. The figure in the Appendix-A1 indicates that the existing main rectangular ditches taken on field observation from study site.

II. Rectangular ditches without slab cover

The open rectangular ditches are not advisable due to the various risks for community population and environmental sanitation. The disadvantages of ditches without slab cover at a particular location is greater than its advantages because lose of property, health problem, pollution of environment may occurs suddenly. The existing open drainages were constructed by following the internal coble stone and gravel road access and contribute the runoff toward the main drainage as the elevation difference conditions. In the study site the 1685.69m coble stone and gravel road sides internal drainages were blocked with solid wastes and the surface runoff that flow from undeveloped part of sub catchments with high silt concentration. The ditches were open rectangular masonary concrete and exposed to entering of construction demolition wastes, domestic wastes and silted runoff that originates from the periphery of the town as shown in the appendix-A2.

4.1.2 Road and drainage access in selected site of study area

The life span of road access in a certain area depends upon the availability of effective and efficient drainage system that transport storm water surface runoff which generated from rainfall. The capacity of drainage system and the volume of discharge are the parameter that has to be considered from point of generation to disposing point. Around the site of study area the road and drainage access was assessed with site observation on fields and using Google earth.

Different types of road levels were exist on which the drainage system constructed by following its sides with masonary rectangular channels or ditches. In the study sites developed part of the town were covered with road access such as asphalt, coble stone, concrete with ditches, concrete without ditches and earthen around the undeveloped area. The maximum depth of the ditches from Central Square of the town to the downstream site of study area with following the main and sub main road access were range between 0.8-1m for asphalt, coble stone and concretes respectively while measuring their dimensions during field observation. Therefore, during the extreme rainfall events, the surface runoff flows overtop from the drainage system and disrupt the road access service by permitting the flow of excess runoff on the streets. The maximum sizes of assessed existing drainage at a particular location were represented for sample in the table 4.1.

Table 4.1: The existing ditches dimensions taken during site observations

<table>
<tr><td>Location</td><td>Asphalt length (m)</td><td colspan="2">Rectangular Ditches along two side (m)</td></tr>
<tr><td>J71 to J87</td><td>796.28</td><td rowspan="2">Depth and width</td><td rowspan="2">Max=1, Max=0.8
Min=0.6, Min=0.5</td></tr>
<tr><td>J80 to J76</td><td>449</td></tr>
<tr><td>J20 to J57</td><td>411</td><td rowspan="3">Depth and width</td><td>Max=1, Max=1</td></tr>
<tr><td>J20 to J34</td><td>1243</td><td rowspan="2">Min=0.8, Min=0.6</td></tr>
<tr><td>J13 to J15</td><td>102.31</td></tr>
<tr><td>Total</td><td colspan="3">3001.59</td></tr>
<tr><td>Location</td><td>Cobblestone length(m)</td><td colspan="2">Rectangular Ditches along two side</td></tr>
<tr><td>J10 to J 96</td><td>258.6</td><td rowspan="2">Depth and width</td><td>Max=1, Max=0.8</td></tr>
<tr><td>J103 to J41</td><td>371.14</td><td>Min=0.7, Min=0.5</td></tr>
<tr><td>J3 to J82</td><td>482.95</td><td>Depth and width</td><td>Max=1, Max=0.8</td></tr>
</table>

J58 to S13	43		Min=0.8, Min=0.5
J111 to J16	150		
Total	1685.69		

4.2 Affected part of the town by poor drainage system

4.2.1 Existing drainage effects on roadway of study site

The appropriate performance of urban drainage systems plays a key role in preventing urban flooding. The primary aim of an urban storm water management system is to ensure storm water generated from developed catchments causes minimal nuisance, danger and damage to people, property and the environment. The storm water drainage system in the urban is aimed to transport surface runoff from the town to downstream point or disposal outlet site without causing a problem on health of community and urban environment. Improper design of drainage system cause to lose of various property and damage of infrastructures without their design life spans time. In study area, road access was damaged due to the overtopped surface runoff while heavy rain falling on upstream part of the town. Destruction of infrastructures with non-managed storm water surface runoff increases stress on vehicles service and challenges the daily activities of population. During field assessment, due to design problems of storm water drainage system several sites were detected and among this some of them were represented in the figure 4.2.

Figure 4.2: Effects of existing drainage system at in front of Bonga town municipality office (photos taken on field, June 29/2021 at 5:30)

4.2.2 Poor monitoring and poor drainage

The reduction of functionality and capacity for transferring the runoff flow, and their level of service decreases due to degradation in time. Improper maintenance, inappropriate design, aging, sedimentation and siltation, increase in materials' roughness, and structural deterioration are also the factors that determines drainage systems. Constructing one infrastructure around the drainage system disturbs its service or completely destructs from its desired uses with simultaneous factors. Building construction, water supply pipe network installations, electric line instillation, road construction and maintenance are the expected factors that may overlap on common areas. In Bonga study area drainage system of channels or ditches was damaged due to the construction material quality and maintenance inadequacy for existing storm water drainage systems. As the result, while heavy vehicles pass through road access between two side ditches, it became instable or destruct completely from service. Rectangular ditches should be cover with slab to protect the entering of solid wastes and the safety of the community population. However, the existing study area main roadside ditches were cover with simply broken and less strengthens slabs. The photo taken on the site shown in the figure 4.3 represents that the current conditions of damaged drainages.

Figure 4.3: Damaged and sedimented ditches with poor existing drainage system at in front of bust station (photos on field, June 30/2021 at 11:00)

4.2.3 Factors that affect existing urban drainage management system

i. Monitoring system

The existing main storm water drainage system in Bonga town study site was constructed in 2005 E.C together with asphalt road at both sides according to information of inventory asset of the town. The progress of any construction should be leads with stake holder professions in various institutions to check the quality of construction from earth excavation up to finishing work as per drawing and contract agreement. The geometric elements of best hydraulic cross sections for drainages have its own standard guidelines as country and international levels, whereas on the table 4.1 the existing available maximum depth and width of ditches didn't follow the standards. Ineffective monitoring system of storm water drainage structures during design and construction results undersized and low quality relative to desired use and design periods in the study area of Bonga town. The best hydraulic cross sections of drainage system should be designed based on standard for various shapes and among them some of them were shown in the table 4.2.

Table 4.2: The geometric elements of best hydraulic cross sections for drainage (Pierre, 2020)

cross section	Area	wetted perimeter	Hydraulic radios	Top width	hydraulic depth
Rectangle	b*y	b+2*y	b*y/(b+2y)	b	y
Trapezoid	(b+ t*y)*y	b+2y*$\sqrt{(1+t^2)}$	A/P	b+2ty	A/B

Note: A-cross sectional area, B-top width, b=bottom width, P-wetted perimeter, y- depth of flow in the channels

ii. The awareness of community

The solid wastes that disposed into the line of drainage or ditches increases the load on the effectiveness of drainage service and back flow in the drainage network and results overtopping of runoff from its system. Jonathan (2003) suggests that problems related to poor drainage are exacerbated by poor solid waste management, as uncollected solid waste often enters surface drains and causing blockages and reduced flow capacity. The problem that detected on site due to improper disposal of solid waste was clogging of internal drainage channels/ditches and alters its desired uses. The grasses that grow on the accumulated sediment and domestic wastes in the channels were identified at various part of the study site during field work. Lack of comprehensions for the community about the advantages and disadvantages of storm water drainage system result the detected problems in study site shown in the Appendix-B.

iii. The available rainfall amount

The amount of rainfall in a certain area is key parameters to design storm water drainage system to transport surface runoff toward desired outlet points. In rainfall region classification map of Ethiopia, Bonga town was found in the western part as represented with figure in the Appendix-D3. Extreme rainfall events were identified for all available years in Bonga town station from the data that taken from national metrology agency of Ethiopia. During available hydrological data analysis the falling rate of rain in the study area, 101.7mm/hr was recorded in 100 years of return period. However, the available rainfall amount and the observed existing drainage system capacity were not matched to convey the surface runoff that generated from study area sub catchments to disposing outlet points.

4.2.4 The existing and simulated drainage system

The finding is emphasis on assessing of existing storm water drainage system and simulating the model with suitable dimensions of main as well as sub main drainage system for the conveyance of runoff discharge through rectangular channels. The determination of flow through drainage channels is to identify the problem of existing drainage and propose the suitable dimensions on the model up to outlets with try and errors. The existing and simulating drainages were compared and validate based on the various scenarios and to select the fit method through evaluation criteria. The selected 10 channels/conduits around outlet-1 and outlet-2 were determined for sample calculations by using both rational and SWMM 5.1. The discharge through existing drainage that calculated by using the manning equation and channels through simulation of the model were shown in the table 4.3 and the remaining table of outlet-2 was presented in the appendix-B.

Table 4.3: The existing and simulated flows

Outlet1	Ditches	D (m)	W (m)	A (m2)	P (m)	R (m)	S	n value	Calculated Q (cms)	simulated
existing	C42	0.8	0.8	0.64	2.4	0.267	0.04	0.012	0.025	0.45
existing	C43	0.85	0.8	0.68	2.5	0.272	0.04	0.016	0.021	0.23
existing	C44	0.85	0.6	0.51	2.3	0.222	0.05	0.012	0.017	0.42
existing	C50	0.85	0.5	0.425	2.2	0.193	0.05	0.012	0.011	0.06
existing	C60	0.85	1	0.85	2.7	0.315	0.04	0.012	0.047	0.08
existing	C61	0.9	0.9	0.81	2.7	0.3	0.04	0.012	0.041	0.13
existing	C64	0.95	0.8	0.76	2.7	0.281	0.04	0.012	0.033	0.51
existing	C65	0.95	1	0.95	2.9	0.328	0.06	0.012	0.085	0.19
existing	C70	1	1	1	3	0.333	0.06	0.012	0.093	0.35
existing	C71	1	0.9	0.9	2.9	0.31	0.06	0.012	0.072	0.24

4.2.5 Sub catchments classification

The study site catchment was classified to twenty seven small individual sub catchments. The division was based on the existing plan of the main and sub main road access, geographical landscapes, elevation difference between upstream sub catchment and downstream common outlets. Those downstream and upstream values were corrected on Google earth to the required point for each proposed sub catchments to laying out the junction nodes, conduits and outlets. The amount of peak runoff production rate depends up on the imperviousness and width of land use land cover. Sub catchments in study site were grouped into some undeveloped and developed with commercial, densely settled area, various road access respectively as identified in the LULC figure 3.12. The aim of catchment classification is transporting surface runoff from upstream urban part of the first junction node to outlets point with effective way, efficient ability and permissible error in the flow routing system. The figure 4.4 shows that the classified sub catchments in the SWMM 5.1 window.

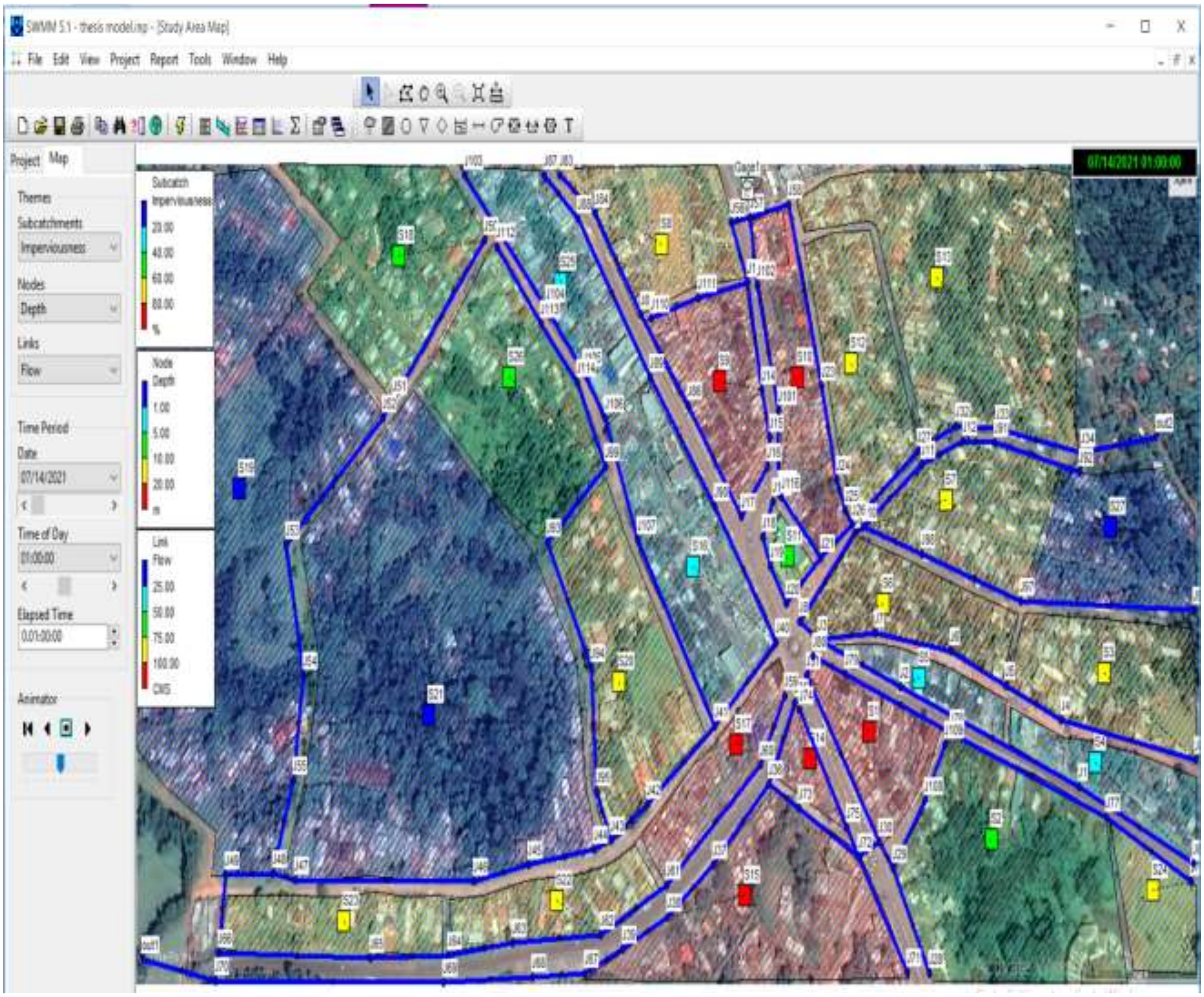

Figure 4.4: Image of study site classified sub catchments

The area of sub catchments play the key role for determining surface runoff amount relative to their land use land cover during for simulation of model. It was measured on the Google earth and set in the model with the SI unit of hectare as an input. The value of width for each sub catchments determines the error of surface runoff while running simulation for validation of the drainage system adequacy. An initial estimate of the characteristic width is given by the sub catchment area divided by the maximum overland flow length. The maximum overland flow length is the length of the flow path from the outlet to the furthest drainage point of the sub catchments as indicated in the table 4.4.

Table 4.4: Calculated Width of each sub catchments

Sub catch	Area (ha)	Area (m2)	path length	Width
S1	0.7809	7809	137	57
S2	2.786	27860	201.53	134.2
S3	1.332	13320	206.33	64.6
S4	0.7295	7295	213.55	34.2
S5	0.4081	4081	168.36	24.2
S6	0.9819	9819	203	44.2
S7	5.61	56100	189.3	296.4
S8	1.157	11570	152.77	55.4
S9	1.027	10270	110.19	58.3
S10	1.074	10740	136	34.5
S11	0.7258	7258	74.41	97.5
S12	1.239	12390	152.35	49.1
S13	3.415	34150	230.5	107.1
S14	0.225	2250	80.58	27.9
S15	2.13	21300	139.38	89
S16	1.637	16370	111.45	69.8
S17	0.6984	6984	166.92	41.8
S18	1.796	17960	261.21	66.2
S19	7.47	74700	207.48	197.9
S20	1.732	17320	164.16	77.3
S21	9.021	90210	206.55	260.3
S22	0.7934	7934	179.59	37.9
S23	0.847	8470	232	36.5
S24	0.5683	5683	103.38	37.1
S25	1.16	11600	161.79	50
S26	2.14	21400	189.71	82.4
S27	5.61	56100	189	280.5

4.3 Determination of peak discharge of runoff

4.3.1 Time of concentration

Time of concentration is defined as the time taken by storm water runoff from longest path of the sub catchments to the point on which peak runoff to be determined. Various possible options are available for determinations of intensity. If the determined value of TC is less than 15 minutes, it is taken as minimum value of intensity to determine the peak runoff whereas, if it is greater than 15 minutes, the intensity value should be taken from developed IDF curve to those durations.

Fortunately, for this thesis its value for all sub catchments less than 15 minutes. TC value could be determined by using equation 3.11 and represented in the table 4.5.

Table 4.5: Time of concentration in all sub catchments

Sub catch	S_{av}	L	TC
S1	0.03	137	10.9464
S2	0.03	201.53	14.7346
S3	0.03	206.33	15.0041
S4	0.03	203.55	14.8482
S5	0.03	168.36	12.8293
S6	0.03	203	14.8173
S7	0.03	189.3	14.0412
S8	0.02	152.77	13.9156
S9	0.01	110.19	14.1299
S10	0.02	136	12.7238
S11	0.01	74.41	10.4435
S12	0.02	152.35	13.8861
S13	0.04	230.5	14.6269
S14	0.02	80.58	8.50331
S15	0.02	139.38	12.9666
S16	0.01	111.45	14.2541
S17	0.03	166.92	12.7447
S18	0.05	261.21	14.7798
S19	0.05	207.48	12.3782
S20	0.02	164.16	14.7078
S21	0.04	206.55	13.4421
S22	0.03	179.59	13.4833
S23	0.04	232	14.7002
S24	0.01	103.38	13.4526
S25	0.02	161.79	14.5441
S26	0.04	189.71	12.59
S27	0.03	189	14.0241

4.3.2 Rational method

Rational method is one of peak discharge runoff determination method for the sub catchments whose width is less than 50 hectares. In this thesis the peak runoff discharge determination by rational method was carried out to compare runoff amount that simulated with SWMM 5.1. During calculation of the peak discharge of runoff by using rational method; intensity, area of the sub catchments and runoff coefficients should be required. The study site catchment was classified to 27 sub catchments and each of them also contains their own various land use land cover that was identified on Google earth. By using the equation 3.9 peak runoff discharge was calculated in the table 4.6.

Table 4.6: The peak runoff calculated by rational method

Sub	L	A (ha)	cw	TC	I (mm/hr)	Factor	Q (cms)
S1	310.73	0.7809	0.95	10.9464	101.7	0.00278	0.21
S2	302.73	2.786	0.495	14.7346	101.7	0.00278	0.39
S3	213.68	1.332	0.56	15.0041	101.7	0.00278	0.21
S4	238.4	0.7295	0.5275	15.4068	101.7	0.00278	0.11
S5	197.83	0.4081	0.9225	12.8293	101.7	0.00278	0.11
S6	316.83	0.9819	0.8575	14.8173	101.7	0.00278	0.24
S7	429.08	5.61	0.76	14.0412	101.7	0.00278	1.21
S8	130.69	1.157	0.615	13.9156	101.7	0.00278	0.2
S9	344.17	1.027	0.9225	14.1299	101.7	0.00278	0.27
S10	446.34	1.074	0.6875	12.7238	101.7	0.00278	0.21
S11	128	0.7258	0.8525	10.4435	101.7	0.00278	0.17
S12	86.35	1.239	0.7925	13.8861	101.7	0.00278	0.28
S13	183.88	3.415	0.5325	14.6269	101.7	0.00278	0.51
S14	117.08	0.225	0.9225	8.50331	101.7	0.00278	0.06
S15	304.98	2.13	0.69	12.9666	101.7	0.00278	0.42
S16	311.71	1.637	0.8675	14.2541	101.7	0.00278	0.4
S17	183.91	0.6984	0.876	12.7447	101.7	0.00278	0.17
S18	88.3	1.796	0.515	14.7798	101.7	0.00278	0.26
S19	363.38	7.47	0.4675	12.3782	101.7	0.00278	0.99
S20	216.93	1.732	0.755	14.7078	101.7	0.00278	0.37
S21	328.73	9.021	0.3325	13.4421	101.7	0.00278	0.85
S22	209.68	0.7934	0.89	13.4833	101.7	0.00278	0.2
S23	250.8	0.847	0.89	14.7002	101.7	0.00278	0.21
S24	102.76	0.5683	0.6225	13.4526	101.7	0.00278	0.1

S25	179.61	1.16	0.9175	14.5441	101.7	0.00278	0.3
S26	271.97	2.14	0.405	12.59	101.7	0.00278	0.25
S27	354.42	5.61	0.383	14.0241	101.7	0.00278	0.61

4.3.3 Processing of model

i. Rainfall-runoff

Intensity from highest to lowest one were an input for the model to its short durations because the highest rain falling rate was recorded at lowest time as described on IDF curve. As durations of time increases the falling rate of rain became decreases and the production rate of surface runoff at the particular time was zero. Because of rain has changed to runoff after reaching to the surface of the earth and remains from loses. The table 4.7 showed that the selected sub catchment 7 and 21 respectively for sample calculations due to having a high peak runoff from other sub catchments the connected to both outlets of drainage systems. They produces 0.68, 1.2 cms runoff at 30 minutes of rainfall and 1.23, 1.45 cms at 45 minutes then decreases continuously up to last of two hours simulation as shown on precipitation (mm/hr) vs runoff (cms) for both S7 and S21. Generally, at heavy rain the runoff amount exist at initial point and whereas as rain decreasing continuously for a particular time the runoff amount reach at critical point and also reduces gradually. The rainfall-runoff relation could affected by land use land cover, width of sub catchments and time series. The storm water drainage has to be carry the maximum load of runoff produced at that particular time. The maximum capacity of existing drainage system could carry $1m^3$ but the amounts of peak runoff generated were 1.23 and 1.45cms. This indicates that there were excess runoff discharges on receiving J10 and J45 nodes at 45 minutes of duration from S7, S21 respectively. Therefore, the maximum nodes and channels depth in the main road access should greater than 1m at a various particular location to convey runoff effectively. As the result, the required size of drainage at all main asphalt road sides of storm water drainage in study area, the routing from center square to outlet 1 and from kalima building to outlet 2 has to be more than $1.5m^3$ to convey the storm water runoff without stressing and overflow.

Table 4.7: Rainfall-runoff at sub catchment 7

Sub catchment 7 and sub catchment 21			
Hours	Precipitation	Runoff (cms) on S7	Runoff (cms) on S21
0:15:00	101.7	0	0
0:30:00	72	0.68	1.2
0:45:00	48.9	1.23	1.45
1:00:00	46	0.77	1.23
1:15:00	39.2	0.7	1.13
1:30:00	34.2	0.61	1
1:45:00	30.3	0.53	0.88
2:00:00	27.3	0.47	0.78

The rainfall- runoff relation on sub catchment shown in the figure 4.5 states that the amount of rainfall in the form of intensity was 101.7mm/hr and at particular time the runoff concentration was zero cubic meter per second. Because, the accumulation of runoff occur and start to flow over land surface after some duration of rainfall.

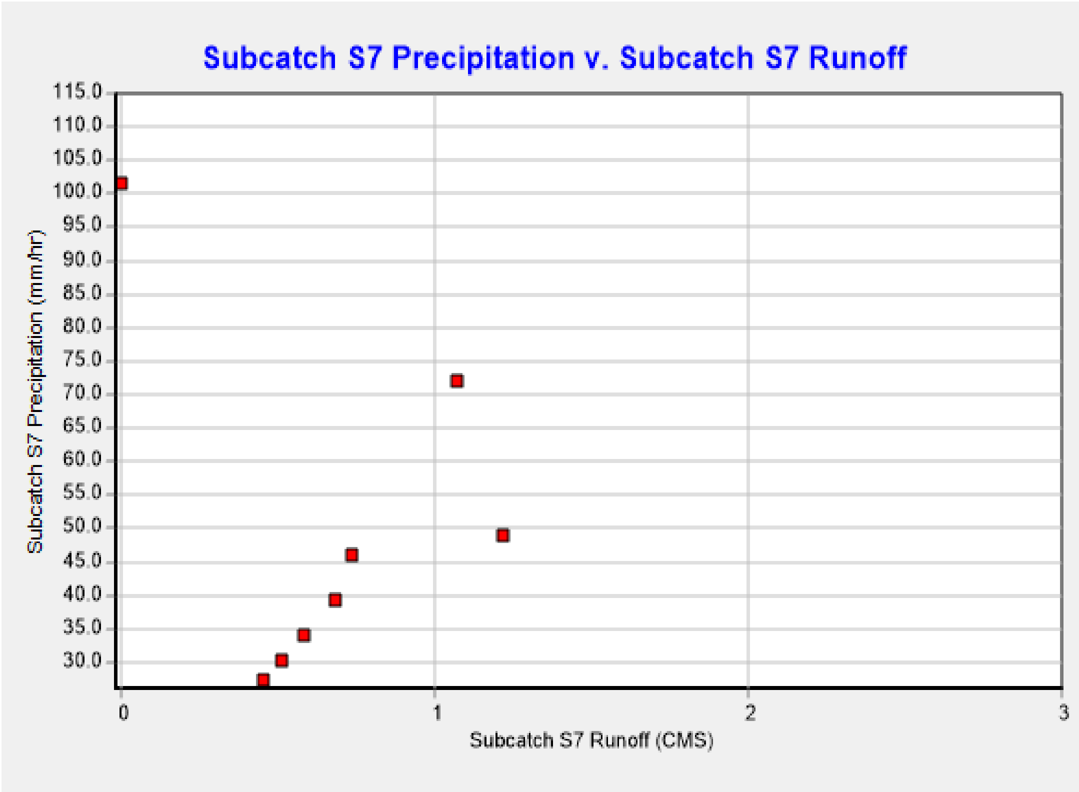

Figure 4.5: Rainfall-runoff relation for sample sub catchment from outlet one

In the same way, sub catchment that taken as a sample from flow routing of outlet-2 was generate the runoff discharge after some durations of rainfall. At the maximum intensity, no runoff generation has occurred in the drainage system that was reported on scatter plot selection by adjusting x and y variables in SWMM 5.1 as shown in the figure 4.6.

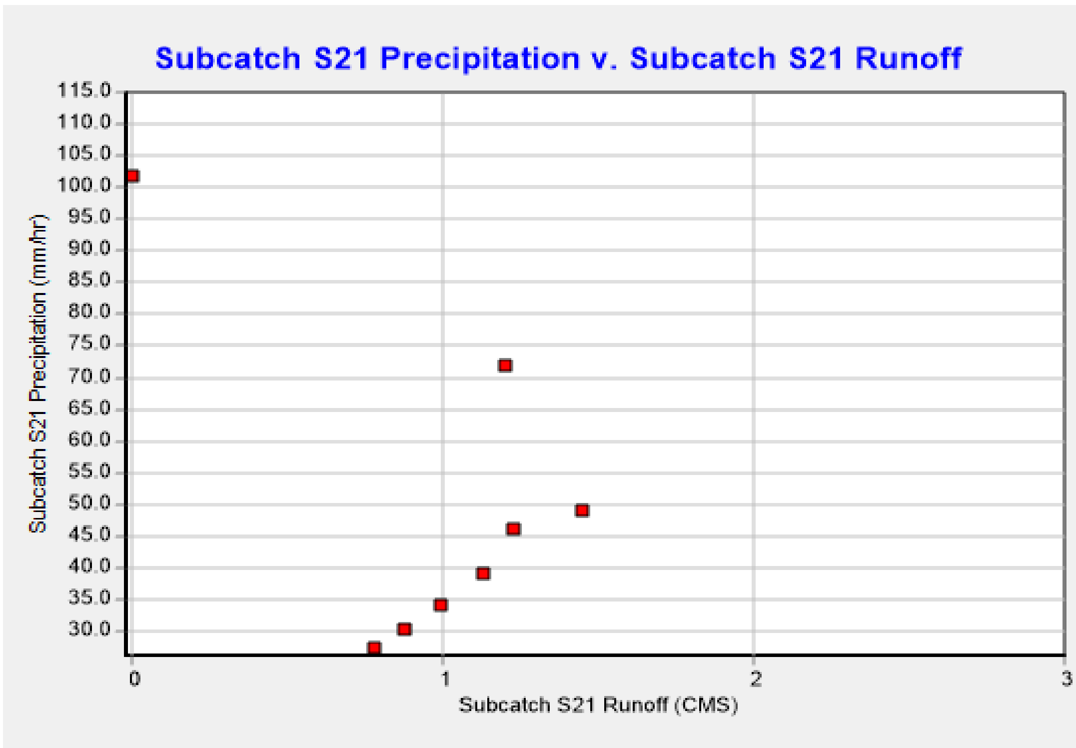

Figure 4.6: Rainfall-runoff relation for sample sub catchment from outlet two

ii. Flow routing

The network of storm water drainage system a structure (nodes, conduit/channels and outlets) were interconnected and gives the successful simulation while the necessary data entered well. However, from those parameters if there is none connected node or elevation difference problems from first node up to the end of outlet point the model never display the result or warning messages about errors. The flow routing in the link governed by saint venant flow equations that means unsteady flow or gradually varied flow conditions (Lewis, *et al.*, 2015). The selected flow routing for Bonga town of study site was kinematic wave flow conditions because, this method allows the flow variation in storm water drainage system spatially and temporally during runoff flow on the channels. The flow routing was connected with both outlet 1 and outlet 2 but they receive precipitation from single rain gage as represented in the figure

4.9. The 59 conduits/channels and 57 junctions were connected with outlet 1whereas 62 conduits and 59 junctions were connected to outlet 2. The storm water drainage system of study site flow routing was carefully follows the elevation difference of each consecutive nodes and it results no flooding drains in both modeled outlets shown in the figure 4.7.

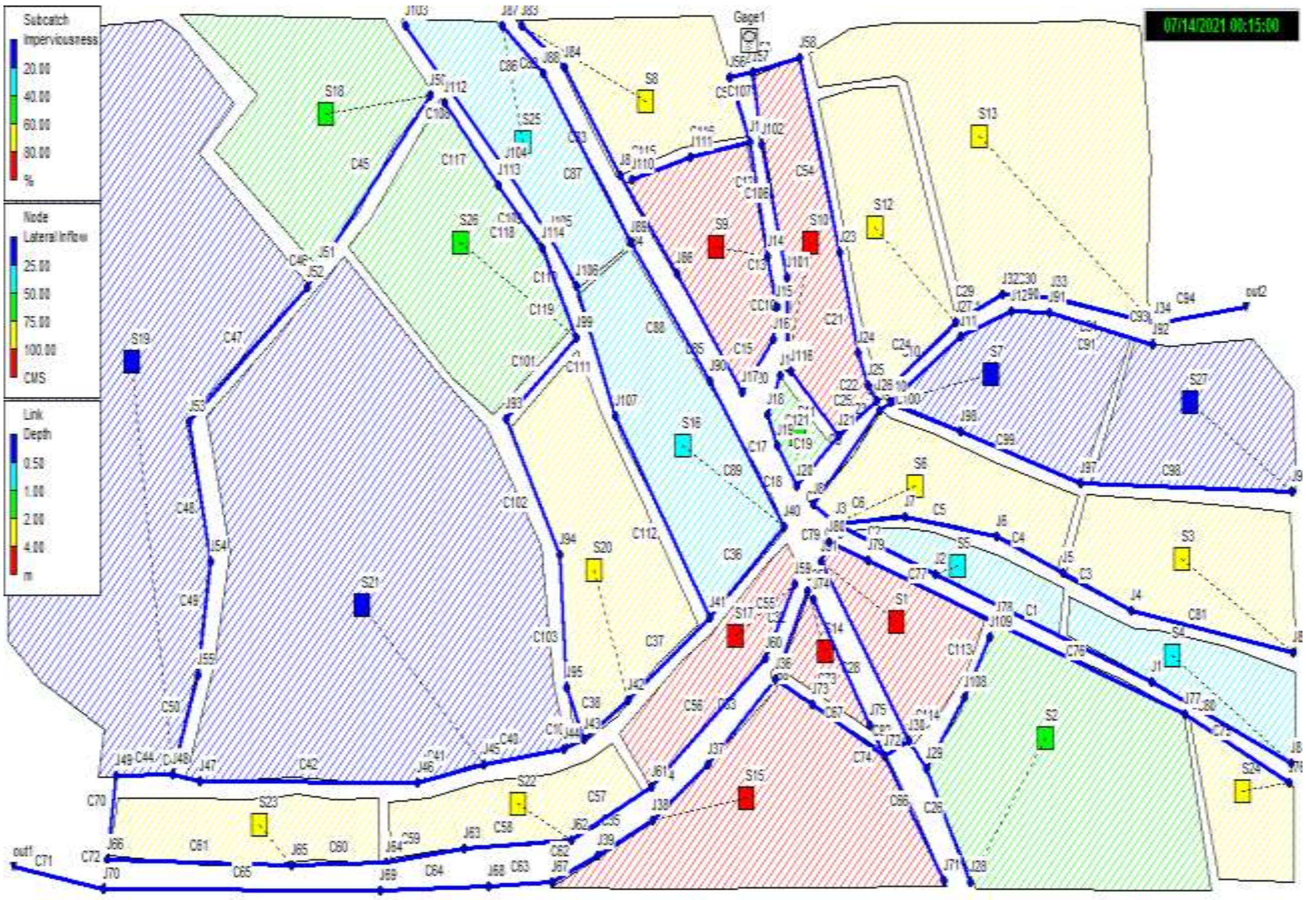

Figure 4.7: The flow routing of drainage network at selected site

4.3.4 Infiltration model

I. Green-Ampt method

Infiltration of surface runoff has its own advantage to recharge of ground water and reduce the formation of flooding. Green-Ampt method was selected to carry out the infiltration rate of study site sub catchments and necessary data (conductivity, suction head and deficit) were filled depending up on the soil textures. Group-B soils typically have between 10 percent and 20 percent clay and 50 percent to 90 percent sand and have loamy sand or sandy loam textures that indicated at table 3.1 (NRCS, 2009). The value of infiltration parameters was entered based on soil texture in the study area and the description that given in soil group that prepared by natural resource conservation service. During setting the infiltration parameters in all sub catchments, it was taken within the accepted range of soil mixture that fixed with United State natural soil conservation service and represented in form of table at Appendix-E.

4.3.5 Calibration

4.3.6 Surface runoff in sub catchments

The surface runoff in each sub catchments were generated through SWMM 5.1 remaining after loses due to infiltration based on the percentage of impervious land use land cover. The amounts of infiltration rate for undeveloped sub catchments (S16, S19, S21 and S27) were higher than developed (densely settled area, commercial area and others). When the amount of infiltration rate in particular sub catchments became high, the production rate of runoff became low and besides of this, the area of sub catchments determine surface runoff amount. The runoff that produced for certain sub catchments were high relative to others that have small area and width. The table 4.8 represents that each Sub catchments of total precipitation, total infiltration, total runoff and peak runoff.

Table 4.8: Surface runoff in all sub catchments

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

Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Total Runoff mm	Total Runoff 10^6 ltr	Peak Runoff CMS
S1	101.70	0.00	0.00	0.11	101.63	0.79	0.24
S2	101.70	0.00	0.00	2.54	95.57	2.66	0.65
S3	101.70	0.00	0.00	1.32	99.83	1.33	0.38
S4	101.70	0.00	0.00	2.94	92.71	0.68	0.14
S5	101.70	0.00	0.00	3.31	93.81	0.38	0.09
S6	101.70	0.00	0.00	1.30	100.05	0.98	0.28
S7	101.70	0.00	0.00	1.96	95.17	5.19	1.23
S8	101.70	0.00	0.00	1.22	99.99	1.16	0.33
S9	101.70	0.00	0.00	0.22	101.30	1.04	0.30
S10	101.70	0.00	0.00	1.11	99.82	1.07	0.30
S11	101.70	0.00	0.00	1.85	99.08	0.72	0.20
S12	101.70	0.00	0.00	1.95	97.41	1.21	0.32
S13	101.70	0.00	0.00	1.54	98.91	3.38	0.93
S14	101.70	0.00	0.00	0.21	101.53	0.23	0.07
S15	101.70	0.00	0.00	1.07	100.26	2.14	0.61
S16	101.70	0.00	0.00	4.37	86.09	1.41	0.26
S17	101.70	0.00	0.00	0.78	100.92	0.70	0.21
S18	101.70	0.00	0.00	3.33	94.75	1.70	0.41
S19	101.70	0.00	0.00	6.03	82.66	6.17	1.14
S20	101.70	0.00	0.00	1.36	98.12	1.70	0.44
S21	101.70	0.00	0.00	6.46	85.30	7.70	1.49
S22	101.70	0.00	0.00	1.31	100.19	0.79	0.23
S23	101.70	0.00	0.00	2.03	99.15	0.84	0.24
S24	101.70	0.00	0.00	2.24	97.57	0.55	0.15
S25	101.70	0.00	0.00	4.85	88.83	1.03	0.20
S26	101.70	0.00	0.00	3.61	94.88	2.03	0.51
S27	101.70	0.00	0.00	4.80	88.65	4.07	0.90

4.3.6.1 The sub catchments peak runoff

The study emphasis on identification of the sub catchments area that has the probability to produce peak surface runoff and the capacity of storm water drainage that receives it with node connected as an outlet. The occurrence of more peak runoff on those sub catchments was based on their infiltration capacity, width and area relatively and this allows overland flow from the longest path to the desired outlet points. The figure 4.8 sub catchments were selected to show their peak runoff vs elapsed time that generated by SWMM 5.1 model in terms to other sub catchments as identified on the surface runoff table 4.7. The reason to identifying of the sub catchments with higher peak runoff in sub catchments were to take consideration during setting the size of drainage system on SWMM 5.1 at that particular location of storm water drainage system. The maximum runoff was occurred at 15 minutes in all four sub catchments around 1.3 cms and but increases for S21 up to the some extents and curve down continuously. Therefore, the separate storm water drainage system should consider to carry the load of peak runoff generated from those sub catchments to nodes and consecutive channels or conduits.

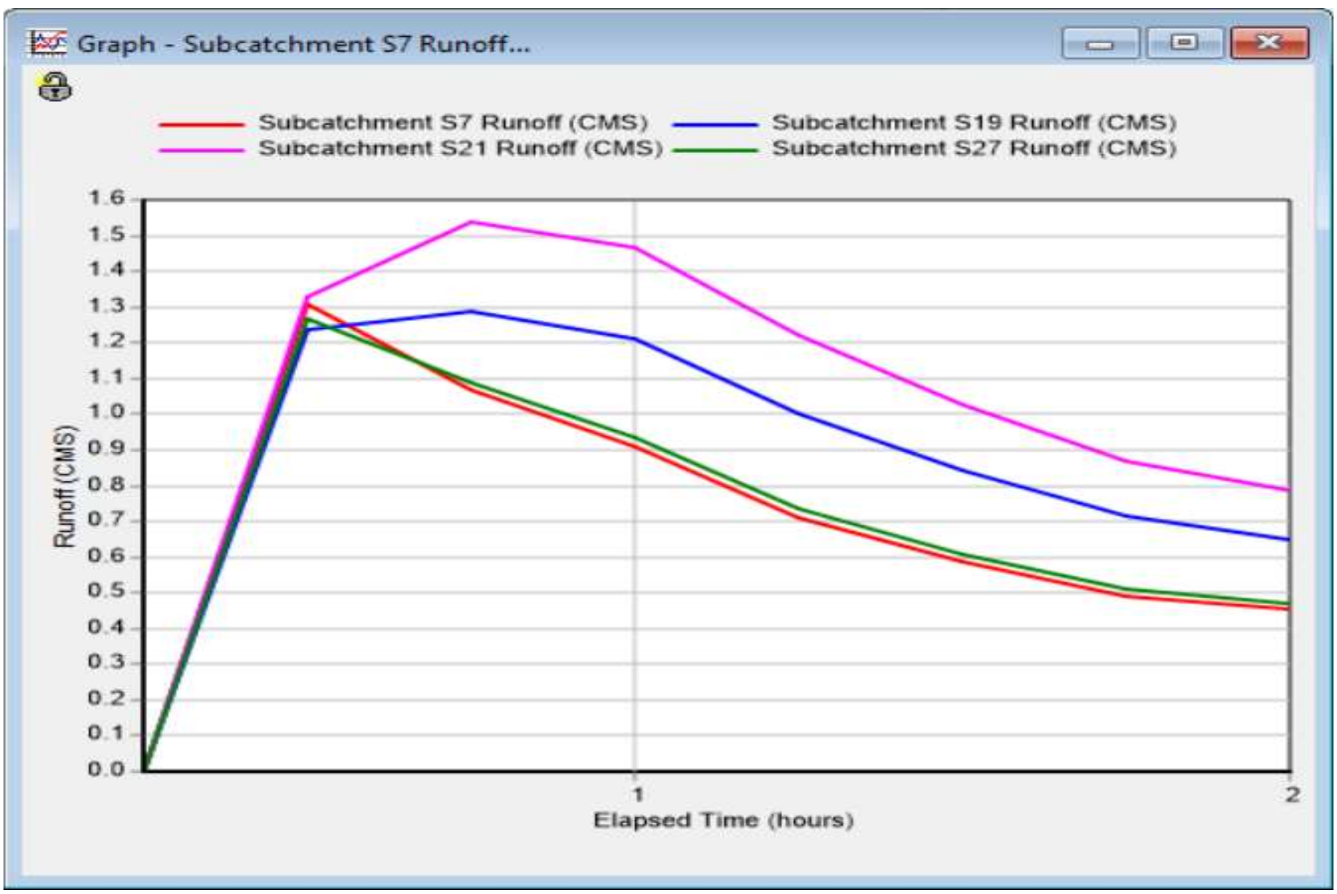

Figure 4.8: Sub catchments runoff comparison

4.3.7 Node depth

The average and maximum depth of storm water at each node were variable for simultaneous simulation time series. The node depth value increasing or decreasing rate was depends on the conditions of interconnection between sub catchments and nodes. This means that the node that connected as outlet to sub catchments has the probability to produce more depth of inflow in the system relative to other non-connected nodes. Average and maximum depth of flow in the junction, maximum hydraulic grade line and various duration on which maximum depth of runoff flow to be occurred were the parameters carefully identified while running simulation. As shown in the table 4.9, 12 junctions were selected as a sample representation and among of them no one depth of flow in storm water drainage system was exceeded the maximum depth of node set in the model.

Table 4.9: Depth of node

Summary Results

Node Depth | Click a column header to sort the column.

Node	Type	Average Depth Meters	Maximum Depth Meters	Maximum HGL Meters	Day of Maximum Depth	Hour of Maximum Depth
J1	JUNCTION	0.04	0.06	1766.06	0	00:33
J2	JUNCTION	0.05	0.07	1739.07	0	00:33
J3	JUNCTION	0.08	0.14	1731.14	0	00:33
J4	JUNCTION	0.08	0.14	1752.14	0	00:33
J5	JUNCTION	0.08	0.14	1750.14	0	00:33
J6	JUNCTION	0.06	0.11	1739.11	0	00:33
J7	JUNCTION	0.08	0.14	1733.14	0	00:33
J8	JUNCTION	0.13	0.22	1729.22	0	00:33
J9	JUNCTION	0.13	0.22	1725.22	0	00:33
J10	JUNCTION	0.29	0.45	1720.45	0	00:45
J11	JUNCTION	0.30	0.47	1715.47	0	00:45
J12	JUNCTION	0.31	0.48	1712.48	0	00:45

However, the depth of nodes through storm water drainage system transports surface runoff in safe condition within adjusted value of maximum depth on SWMM 5.1. The colored bottom part of the conduit or channels indicates that the flow pattern of storm water variation in drainage system within two hours simulation. The simulation hours were divided into equal interval of 15 minutes to characterize with rain gage time interval as well as for checking flow variation from initial depth until maximum points. The flow depth calibrated in the figure 4.9 was conduct on certain part of downstream study site between outlets and neighboring nodes to check the capacity of modeled storm water drainages. During the simulations of outlet-1 and outlet-2 with neighbor junctions were selected to identify the flow of runoff load pattern around the outfall relative to maximum depth of the nodes. Maximum runoff flow in the ditches was shown between the junctions J69 to outlet-1 in the flow routing of outlet-1 whereas between J91 and outlet-2 in the routing of outlet-2 respectively. However, the flow in the simulated two hours duration in the drainage system indicated in the figures 4.9-4.10 of water elevation profile of outlet-1 and outlet-2 with their neighbors channels were below the adjusted maximum depth and width with 1.5m. Therefore, the available rainfall data and the modeled drainage network fit well to transport storm water runoff from study area upstream sub catchments to both outlets.

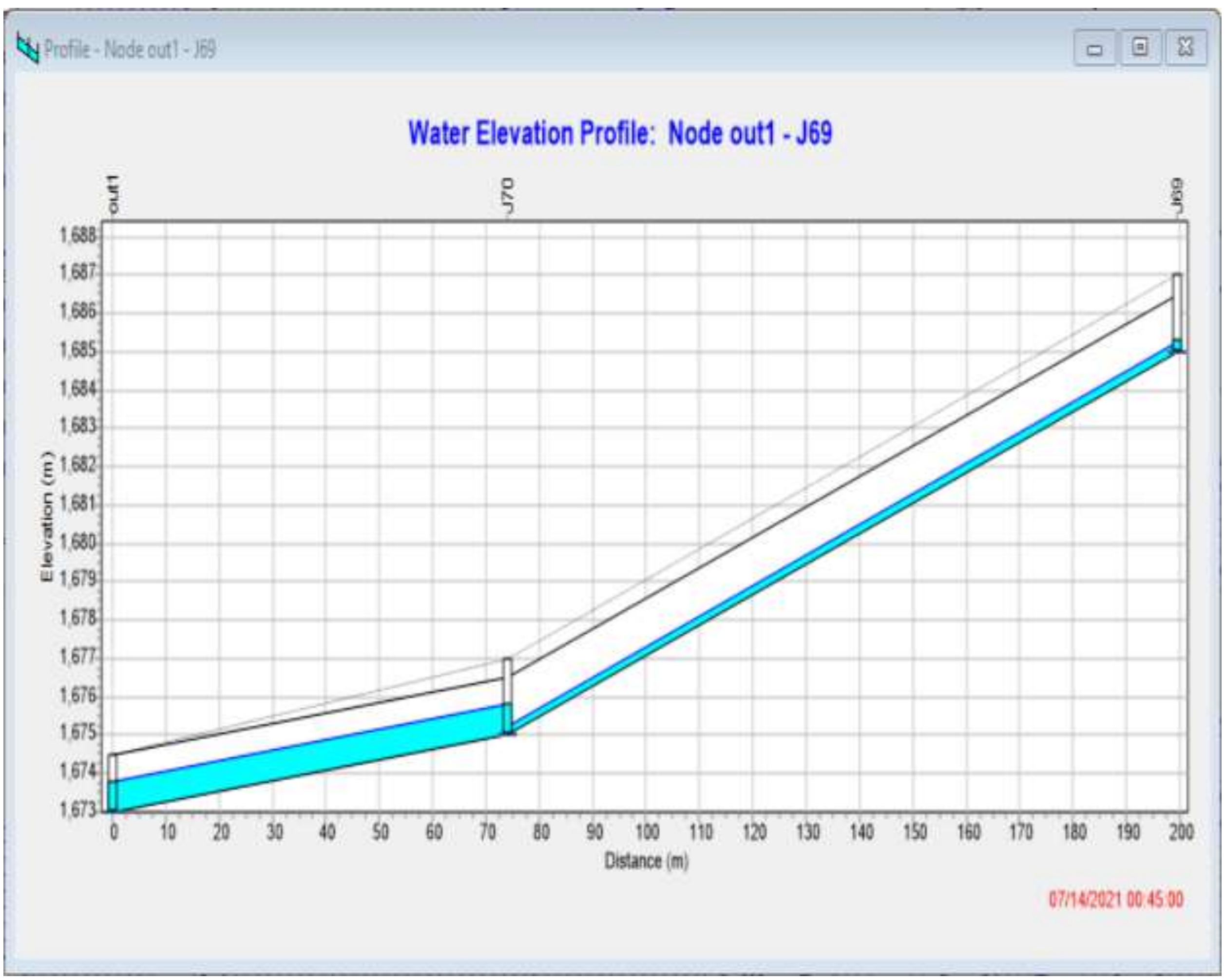

Figure 4.9: Water elevation profile around outlet-1

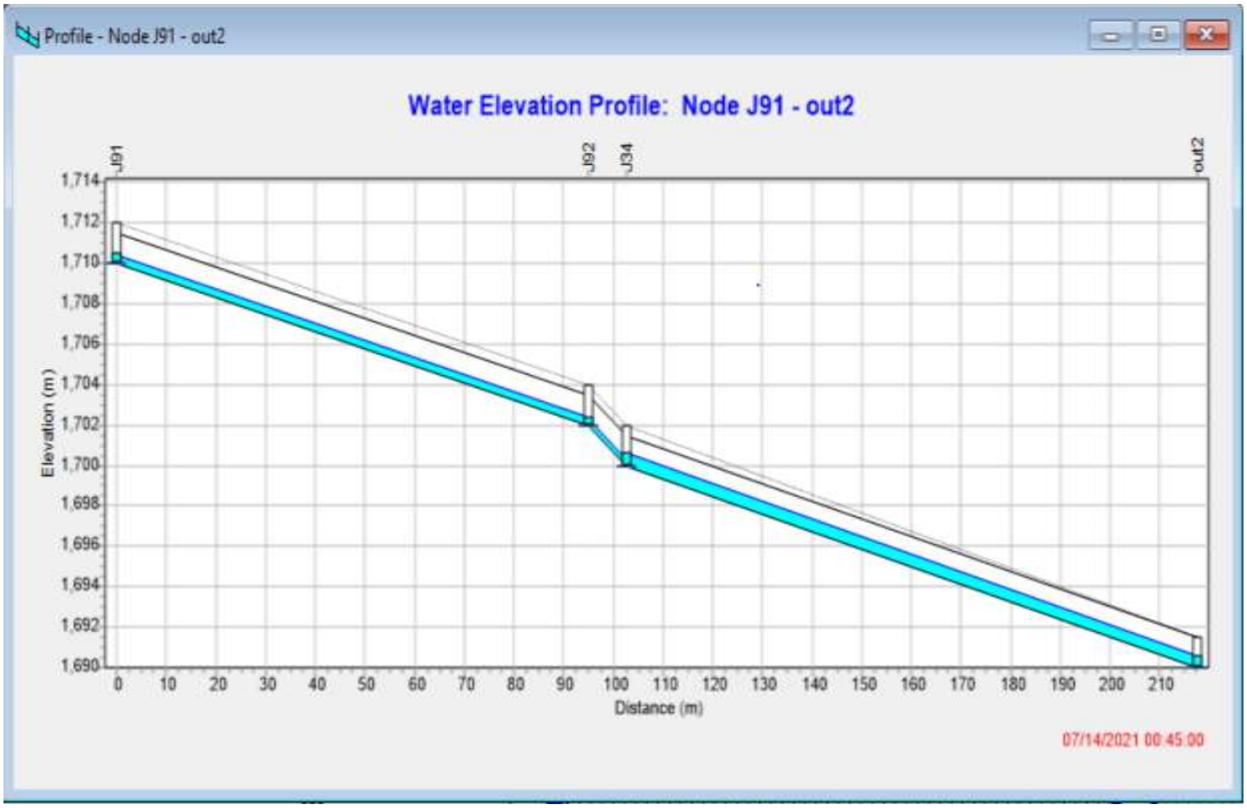

Figure 4.10: Water elevation profile around outlet-2

4.3.8 Link flow

Flow routing includes link on which storm water runoff flow on the channels or ditches after adjusting its depth and length. Flow in link is characterized by several factors such as geometrical shape, roughness of surface, inlet nodes, outlet nodes and elevation differences. The maximum flow, hours of maximum flow occurs, maximum velocity, maximum full depth and flow were the parameters that has variable scenarios when generating two hours simulation on the software. When flow move through conduit or channels it vary from one conduit to another as simulation time series variation. From the installed 121 conduits on SWMM 5.1, 12 of them were selected for sample representation as indicated in the table 4.10 and all the remaining displayed results were shown in the appendix-G.

Table 4.10: Link flow

Summary Results

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

Link	Type	Maximum |Flow| CMS	Day of Maximum Flow	Hour of Maximum Flow	Maximum |Velocity| m/sec	Max / Full Flow	Max / Full Depth
C1	CONDUIT	0.133	0	00:33	2.81	0.01	0.05
C2	CONDUIT	0.214	0	00:33	2.91	0.02	0.05
C3	CONDUIT	0.346	0	00:33	2.49	0.07	0.14
C4	CONDUIT	0.347	0	00:33	4.38	0.03	0.08
C5	CONDUIT	0.349	0	00:33	3.31	0.04	0.11
C6	CONDUIT	0.351	0	00:33	2.54	0.07	0.14
C7	CONDUIT	0.815	0	00:33	6.37	0.06	0.13
C8	CONDUIT	0.818	0	00:33	3.70	0.06	0.11
C9	CONDUIT	0.818	0	00:33	7.16	0.02	0.06
C10	CONDUIT	2.703	0	00:45	5.98	0.22	0.30
C11	CONDUIT	2.701	0	00:45	5.72	0.24	0.32
C12	CONDUIT	0.586	0	00:33	2.36	0.16	0.25

4.3.9 Node flooding

Node flooding is directly related to flow routing within the network of drainage system from upstream nodes point up to downstream outlets point. Existence of flooded node within the drainage network of study area can confirm inadequacy of existing channel (Abew, 2016). Ketema (2018) reports that if the drainage systems have nodes flooding it could overflow there by resulting damages to road surface material. The factors that determine the node flooding in the drainage systems were invert elevation, maximum depth of node, shape of the channels and its depth. During setting of those parameters, the maximum depth of nodes and channels or ditches was taken as existing 1m for checking, but this creates flooding at particular nodes around downstream on which it connected with sub catchments having peak runoff. Therefore, maximum depth of nodes were adjusted again as 1.5m by following the main road and results no flooding in all selected site storm water drainage successfully to zero at both outlet-1 and outlet-2 for two hours simulation periods as represented in the figure 4.11. When the result of nodes flooding became low or neglected, the flow routing system in the storm water drainage adjusted in the model was successful by considering all necessary data relative to conditions of study area.

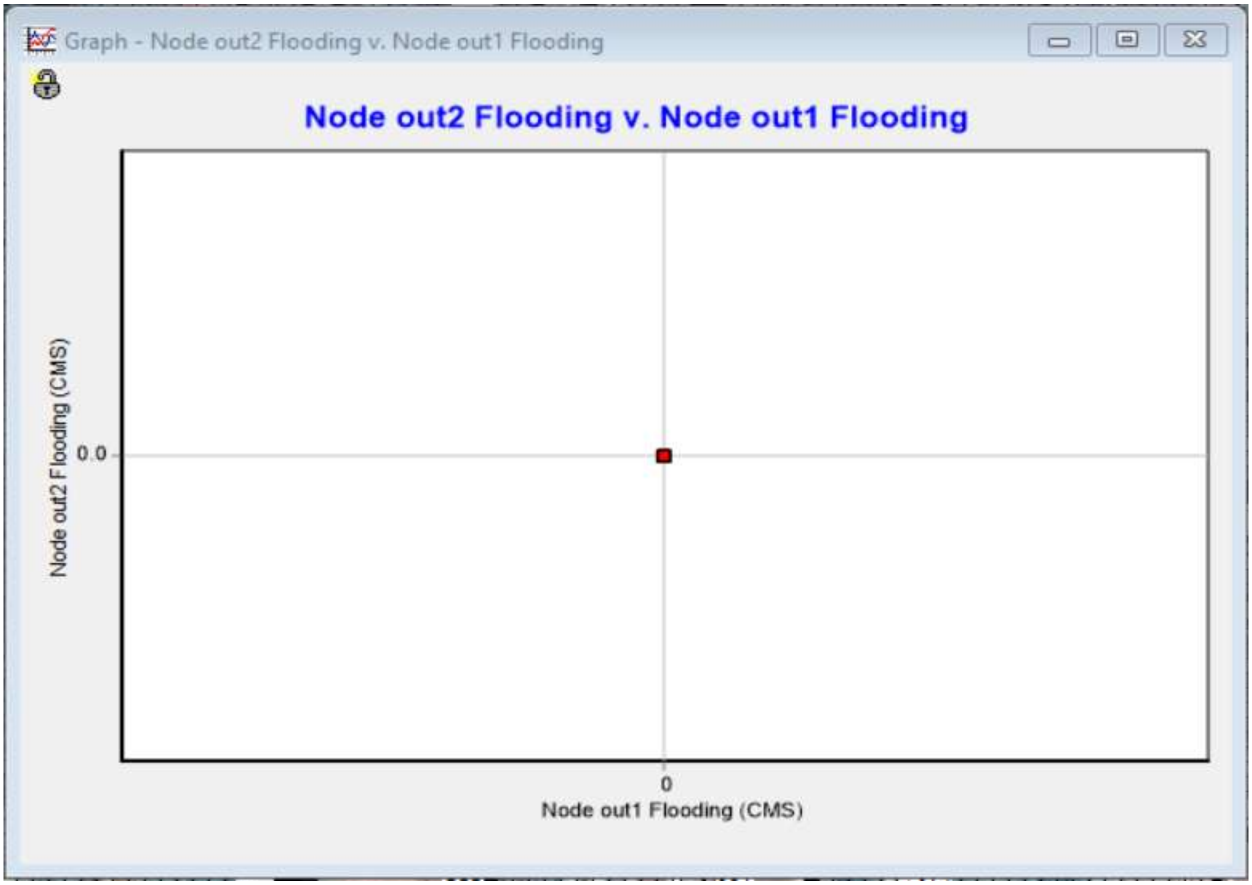

Figure 4.11: Node flooding

4.3.10 Outfall loading

All interconnected parameters through SWMM 5.1 in study area were aims to dispose runoff at outfall nodes generated from precipitation amount received by sub catchments from rain gages. Out fall loading is finally disposed surface runoff through storm water drainage system in the two outlets. The percent of frequent flow discharges equal to 87.8 and their average as well maximum flow were varied because of the number of sub catchments connected for two outlets not equal. In outlet-1 flow routing, surface runoff amount from 14 sub catchments were determined through the model; whereas the remaining 13 sub catchment's surface runoff flow were throughout the routing of outlet-2. Therefore, the total volumes of runoff in sub catchments discharge at both outlets were 26.179 and 18.937 million liter respectively as shown in the table 4.11.

Table 4.11: Outfall loading at downstream point

Summary Results

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

Outfall Node	Flow Freq. Pcnt.	Avg. Flow CMS	Max. Flow CMS	Total Volume 10^6 ltr
out1	87.80	4.080	5.951	26.179
out2	87.80	2.950	4.664	18.937

4.4 Validation

The modeled storm water drainage system was validated with the peak runoff discharges that generated from the sub catchments relative to the dimensions set in SWMM 5.1 with try and errors to protect the stress and surcharge of runoff flow in the routing. Peak discharge determination shown in the table 4.12 was carried out through rational method to make comparisons and using SWMM 5.1 to assess the existing storm water drainage system by various depth scenarios and elevation differences from upstream sub catchments to downstream outlet points.

Table 4.12: Peak runoff discharge determination

sub	A	$A-A_{avg}$	$(A-A_{avg})^2$	B	$B-B_{avg}$	$(B-B_{avg})^2$	(B-A)
S1	0.22	0.056	0.003136	0.21	-0.135	0.018225	-0.01
S2	0.63	0.03	0.0009	0.39	0.045	0.002025	-0.24
S3	0.35	0.011	0.000121	0.21	-0.135	0.018225	-0.14
S4	0.13	0.106	0.011236	0.11	-0.235	0.055225	-0.02
S5	0.08	0.141	0.019881	0.11	-0.235	0.055225	0.03
S6	0.26	0.038	0.001444	0.24	-0.105	0.011025	-0.02
S7	1.31	0.729	0.531441	1.21	0.865	0.748225	-0.1
S8	0.31	0.021	0.000441	0.2	-0.145	0.021025	-0.11
S9	0.3	0.024	0.000576	0.27	-0.075	0.005625	-0.03
S10	0.28	0.031	0.000961	0.21	-0.135	0.018225	-0.07

S11	0.2	0.066	0.004356	0.17	-0.175	0.030625	-0.03
S12	0.29	0.028	0.000784	0.28	-0.065	0.004225	-0.01
S13	0.73	0.075	0.005625	0.51	0.165	0.027225	-0.22
S14	0.06	0.157	0.024649	0.06	-0.285	0.081225	0
S15	0.57	0.013	0.000169	0.42	0.075	0.005625	-0.15
S16	0.26	0.038	0.001444	0.4	0.055	0.003025	0.14
S17	0.19	0.071	0.005041	0.17	-0.175	0.030625	-0.02
S18	0.38	0.006	0.000036	0.26	-0.085	0.007225	-0.12
S19	1.29	0.696	0.484416	0.99	0.645	0.416025	-0.3
S20	0.41	0.002	0.000004	0.37	0.025	0.000625	-0.04
S21	1.54	1.175	1.380625	0.85	0.505	0.255025	-0.69
S22	0.22	0.056	0.003136	0.2	-0.145	0.021025	-0.02
S23	0.23	0.051	0.002601	0.21	-0.135	0.018225	-0.02
S24	0.14	0.1	0.01	0.1	-0.245	0.060025	-0.04
S25	0.2	0.066	0.004356	0.3	-0.045	0.002025	0.1
S26	0.47	0.125	0.015625	0.25	-0.095	0.009025	-0.22
S27	1.27	0.663	0.439569	0.61	0.265	0.070225	0.4356
Average	0.456			0.345			
Sum		4.575	2.952573		-0.005	1.995075	-1.9144

Note: A-SWMM 5.1, B-rational method, A_{avg}-simulated average, B_{avg}-average rational method

The statistical parameters that used for evaluation of model performance were: relative error, coefficient of determination, Nash-Sutcliffe coefficient respectively as explained on the above equation 3.12-14. After calculating the peak runoff with rational method and simulating with SWMM 5.1, the performance of model were determined and results R^2=0.895, RE=20.5 and RNS=0.86). Therefore, these models fit well with all statistical parameters and exist in the acceptable range. The previous study with the title "sustainable storm water management by implementing low impact development" was done in Jemo (Ketema, 2018) to determines the statistical parameters and had got the value of RNS=0.8954, R^2=0.99. Another study was done in Debrebirhan town with the title of "performance assessment of storm water drainage systems"

(Birhanu, 2018) determine the performance evaluation of model and had got R^2=0.852, RNS=0.932 and RE=9.7% respectively.

4.4.1 Peak runoff

Validation is the process of correcting the better fit method that used in the study progress of storm water drainage system assessment. The drainage network consists of sub catchments, links, nodes and outlets. In selected site of study area from coming precipitation some amount is lose with various factors and the remaining flow overland surface as runoff . The peak discharge produced at designed outlet of sub catchments could be determined by rational and storm water management modeling 5.1. The SWMM5.1 simulation was selected for the study site of Bonga town drainage networks and rational method was conduct to comparisons of the analysis. Generally, during the determination of peak runoff the in selected site of study area in both cases, total amount of runoff could be recorded as 12.32cms for SWMM5.1, and 9.31cms for rational method respectively. The same study was done on shire indaslasse city with title of "improving urban drainage system" (Belachew, 2015) and had got peak runoff that calculated by using SWMM5.1 9.28cms, whereas by using rational method 7.459 cms respectively. Another study was carried out on Addis Ababa Bole city with title "Modeling and analysis of urban flooding" (Hassen, 2016) and had got the value of peak runoff discharge by using SWMM 5.1 3.47cms, whereas by using rational method 3.14cms respectively. The figure 4.12 indicates that peak discharge determined by SWMM5.1 and rational methods for all sub catchments as shown in the legends.

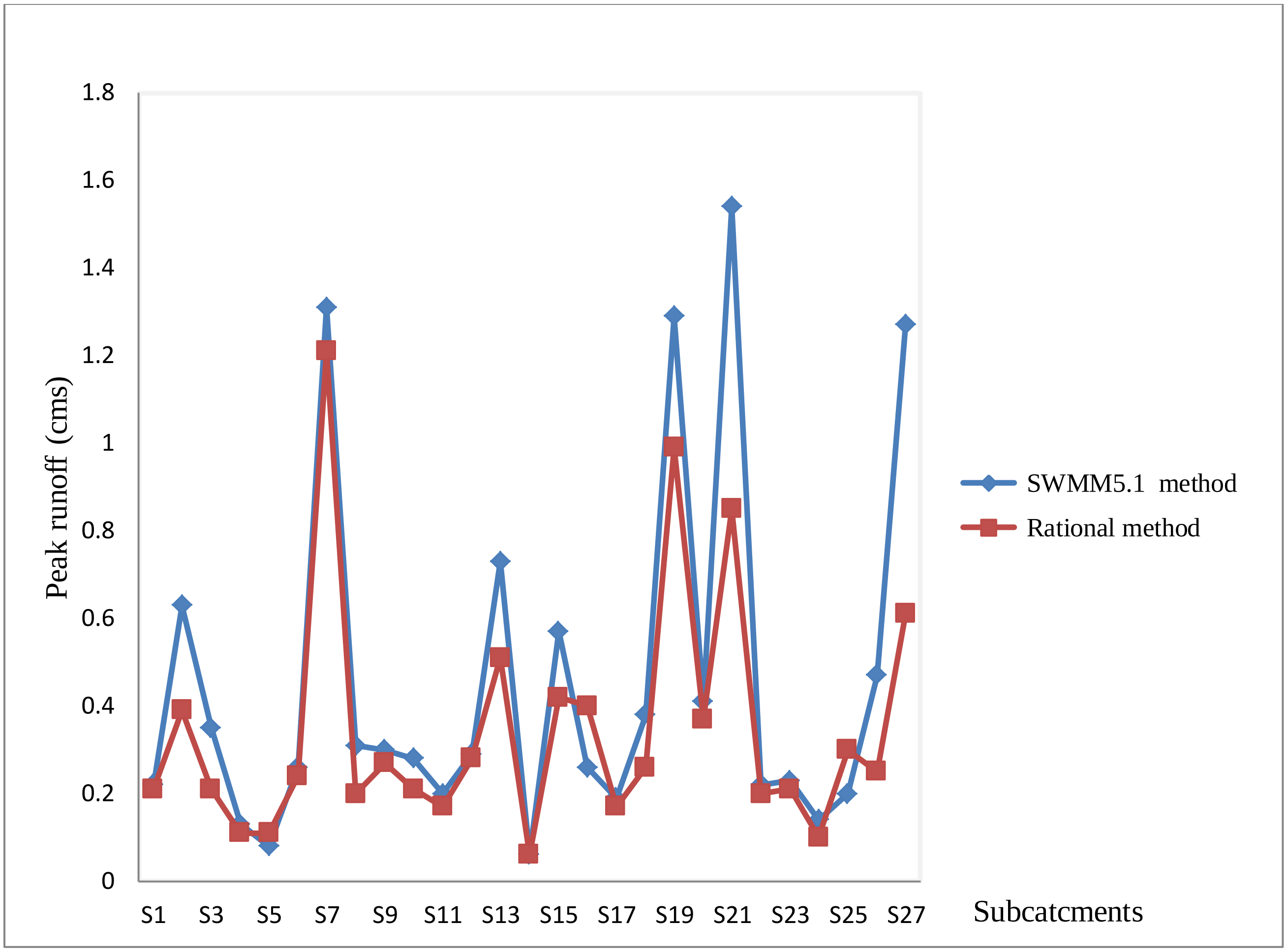

Figure 4.12: Peak runoff graph in all sub catchments

4.4.2 Percent error of flow routing and surface runoff

Model performance for the validation period is usually quantified using the same measures of goodness of fit that was used for calibration process. For calibration, several parameters were examined through flow routing and surface runoff on selected sub catchments. In SWMM 5.1 for validation of flow routing in the drainage system included parameters were nodes (depth, lateral inflow, total inflow and flooding), links or channels (maximum depth, roughness, flow, slope, velocity) and whereas for surface runoff validation area of sub catchments, slope, imperviousness, width, precipitation data were the required and filled input parameters. Validation of the study was carrying out through the percent continuity error of surface runoff and flow routing. The continuity error displayed in the run status window while generate the simulation represent that the percent difference between initial storage plus total inflow and final storage plus total out flow for the entire drainage system (Lewis, *et al.*, 2015). The percent of error in this study area of selected site was recorded zero for flow routing whereas 1.6% for surface runoff. Therefore, it seems good results relative to error standards for both flow routing and surface runoff. Lewis (2015) concludes that flow routing and surface runoff exceed some reasonable level of 10 percent, and then the validity of the analysis results must be questioned. Figure 4.13 defines the status of simulated output for validation of storm water drainage network on selected site.

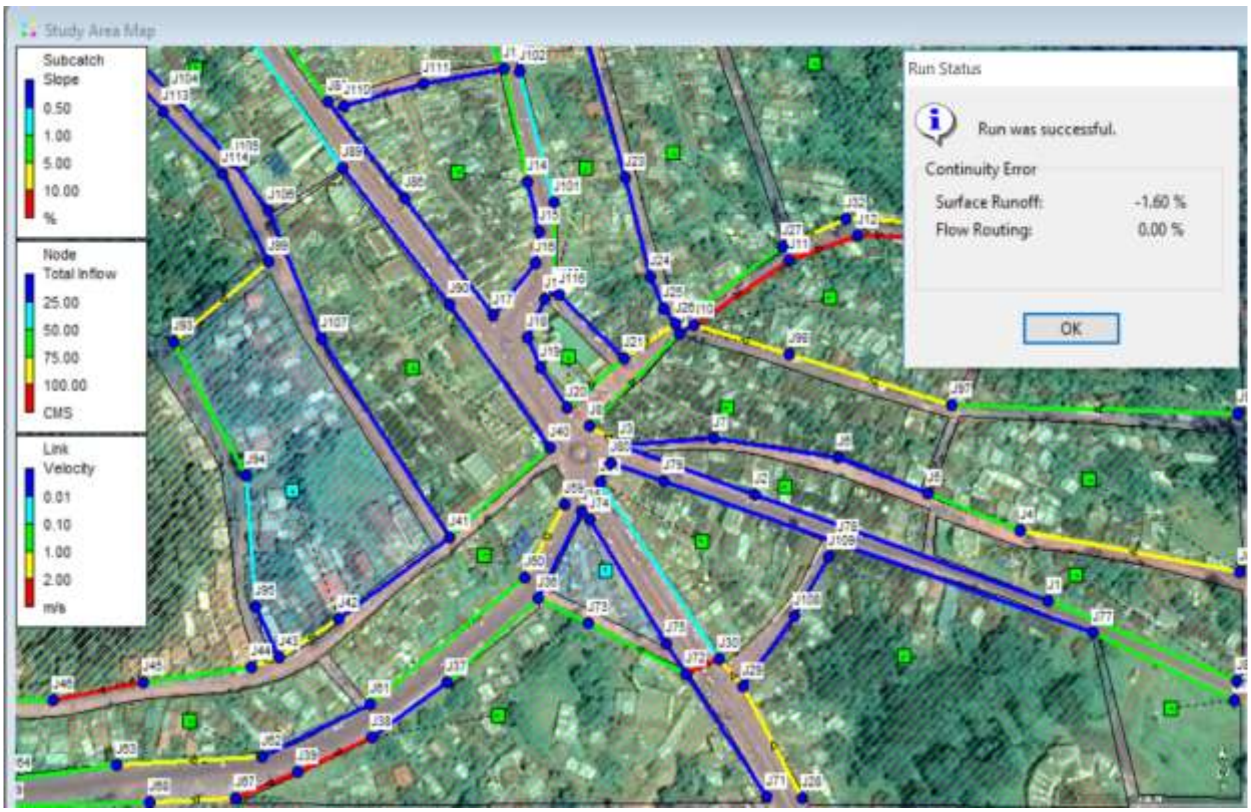

Figure 4.13: Validation of the simulated drainage system

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The existing drainage systems in study site were covered with rectangular channel or ditches which is constructed by masonary and aligned at the sides of roadway. However, the maximum volume of the drainage system capacity was 1m^3 and it couldn’t carry the load of runoff that generated from storm water. As the result, during heavy rain events excess runoff those more than drainage capacity overtops from drainage system to road access. Therefore, it is conclude that the exiting drainage system was not well planned and designed in according to the available rainfall amount in the town.

The efficiency of drainage network was evaluated based on the flow pattern variation in the channels from the upstream to outlets through maximum and minimum rainfall event. The cause of poor existing drainage system in selected study site was result in damaging of the roadways especially asphalt, stressing of the drainage line with runoff and accumulated sediment.

The assessment of storm water drainage system in study site was done by using SWMM 5.1 software. The available rainfall data, time series, rain gage time interval, shape, maximum depth of conduit, the depth of nodes and outlets were used as input in the model of drainage networks to calibrate the flow depth pattern in drainage system. The flow routing and surface runoff were the parameters that used to verify the fitting of model. But, the percent of errors displayed when running the simulation were 1.6% to surface runoff and 0% to flow routing respectively. So, SWMM 5.1 is applicable software for this study site and fit well.

The peak discharge determination around two outlets was carried out through SWMM5.1 and rational method. Validation was carried out with statistical parameters (coefficient determination, Nash Sutcliffe and relative error) that govern the performance capacity of the model. But according to the determined values it exists within the acceptable level and its values were 0.895, 20.5 and 0.86 respectively. Therefore, it is concluded that the SWMM 5.1 model fit well to storm water drainage design assessment of this study area relative to the benefits of community populations and safety of the urban.

5.2 Recommendations

- The existing poor drainage system causes to damaging of road access, stressing of the channels or ditches with accumulated sediment and overtopping of runoff from drainage network on the street that exceed more than its capacity and disrupt the environment. Therefore, the stake holder experts should identify the available extreme rainfall in the town for various reoccurrence periods before starting the storm water drainage design process.
- The most problem of drainage system service has been blocked by community who dispose the solid and domestic wastes into the storm water drainage line. So, town municipal office has to be increase their integration with community population to solid waste collection and hauling activities from the town. The disposed wastes with runoff may pollute the receiving water body and so more study has to be required to quality assessments of effluent.
- SWMM 5.1 and rational method were used to determine the peak surface runoff that generated from sub catchments of study area. Based on various performance evaluations SWMM 5.1 was selected as best fit model relatively. So, the researcher should use this model to conduct the study rather than rational method to determine the peak runoff discharge for any particular location.
- The existing storm water drainage system was constructed with masonary rectangular ditches and the monitoring system that carried out from construction to operation periods was very week. Therefore, effective replanning, redesign and operation with good management should require to solve the problem that occurs between the available rainfall amount of the town and the capacity of existing drainage system.

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APPENDIXES

Appendix-A1: Existing main rectangular ditches with slab cover

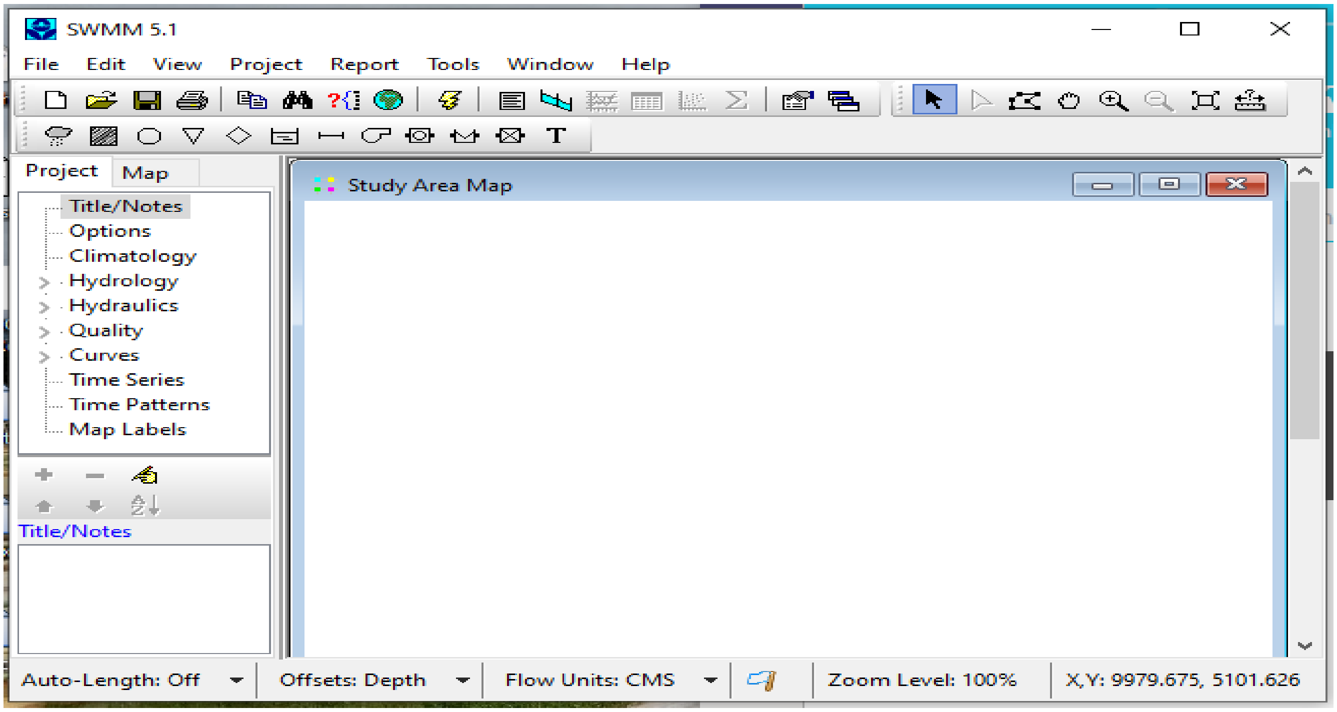
Appendix-A2: The internal ditches following the road access

Appendix-A3: Solid and domestic wastes disposal and existing drainage

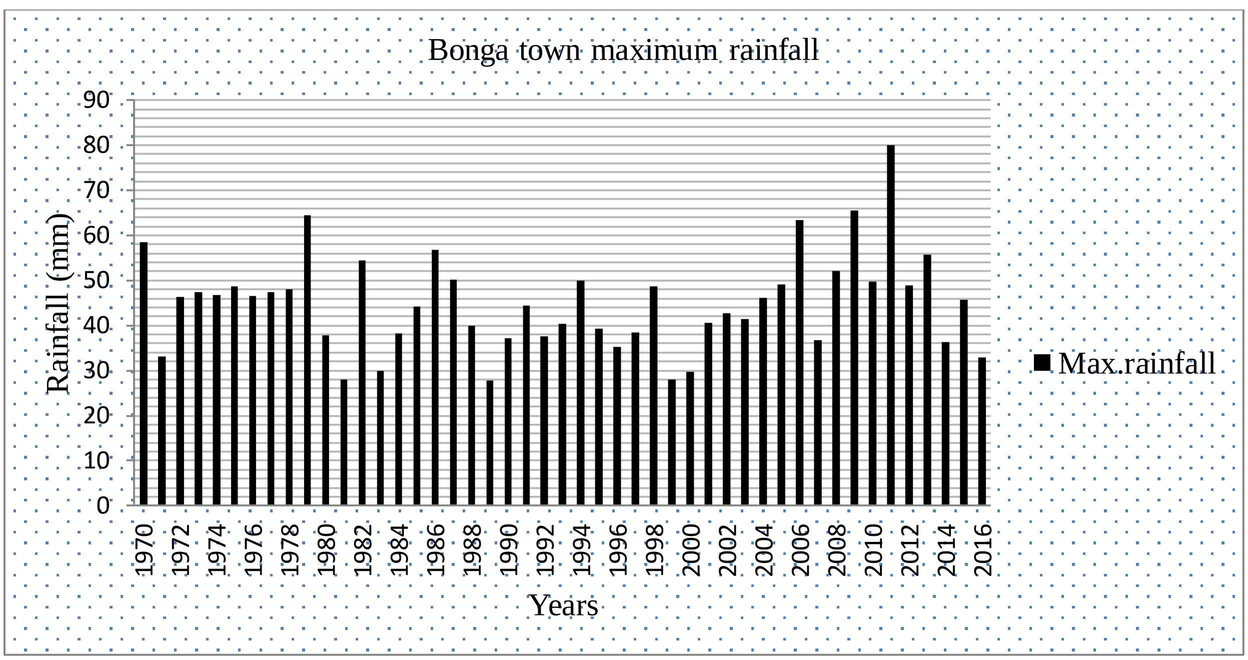
Appendix-B: existing and simulated flows in ditches around outlet-2

Outlet- 2	Ditches	D (m)	W (m)	A (m2)	P (m)	R (m)	slope	n value	Calc (cms)	simul Q (cms)
existing	C10	0.8	0.6	0.48	2.2	0.218	0.03	0.012	0.01	0.3
existing	C11	0.85	0.7	0.595	2.4	0.248	0.03	0.016	0.011	0.31
existing	C90	0.85	0.75	0.6375	2.45	0.26	0.03	0.012	0.018	0.32
existing	C91	0.85	0.75	0.6375	2.45	0.26	0.03	0.012	0.018	0.28
existing	C93	0.85	0.8	0.68	2.5	0.272	0.05	0.012	0.035	0.18
existing	C94	0.9	0.8	0.72	2.6	0.277	0.05	0.012	0.038	0.41
existing	C24	0.9	0.85	0.765	2.65	0.289	0.03	0.012	0.027	0.29
existing	C29	0.9	0.85	0.765	2.65	0.289	0.03	0.012	0.027	0.18
existing	C30	1	0.95	0.95	2.95	0.322	0.03	0.012	0.041	0.18
existing	C31	1	1	1	3	0.333	0.03	0.012	0.046	0.18

Appendix-C: The window of SWMM5.1 used for this thesis



Appendix-D1: Graphical rainfall data in Bonga town



Appendix-D2: Cumulative maximum rainfall of Bonga town and neighbor stations

Max. daily rainfall (Bonga)	Max. daily rainfall (Cida)	Max. daily RF (shebe)	Max. daily rainfall (wush wush)	cumulative of Bonga town	cumulative of neighbor stations			mean of neighbor cumulative
37.8	39.6	32.0	40.4	37.8	39.6	32.0	40.4	37.33
28	41.6	38.7	44.7	65.8	81.2	70.7	85.1	79.00
54.5	86	45.8	46.1	120.3	167.2	116.5	131.2	138.30
30	35.7	30.2	39.8	150.3	202.9	146.7	171	173.53
38.3	30.5	64.5	56.5	188.6	233.4	211.2	227.5	224.03
R44.2	48.2	29.8	31.7	232.8	281.6	241.0	259.2	260.60
56.7	31.3	31.5	30.8	289.5	312.9	272.5	290	291.80
50.2	27.9	33.2	54.3	339.7	340.8	305.7	344.3	330.27
39.9	41	34.2	49.6	379.6	381.8	339.9	393.9	371.87
27.8	21.3	43.7	31.1	407.4	403.1	383.6	425	403.90
37.2	30.8	41.7	34.9	444.6	433.9	425.3	459.9	439.70
44.5	43.5	24.2	51.2	489.1	477.4	449.5	511.1	479.33
37.6	26.6	62.5	46.9	526.7	504	512.0	558	524.67
40.4	28.2	33.0	46.4	567.1	532.2	545.0	604.4	560.53
50	21.4	36.0	49.2	617.1	553.6	581.0	653.6	596.07
39.2	26.5	31.0	53.7	656.3	580.1	612.0	707.3	633.13
35.2	31.5	42.2	46.6	691.5	611.6	654.2	753.9	673.23
38.4	37.3	43.5	44.2	729.9	648.9	697.7	798.1	714.90
48.6	34	38.1	35.6	778.5	682.9	735.8	833.7	750.80
28	38	42.0	49	806.5	720.9	777.8	882.7	793.80
29.6	32	35.3	35.9	836.1	752.9	813.1	918.6	828.20
40.5	28.2	42.2	38.3	876.6	781.1	855.3	956.9	864.43
42.8	32.2	56.6	54.1	919.4	813.3	911.9	1011	912.07
41.4	24.1	90.3	54.9	960.8	837.4	1002.2	1065.9	968.50
46.1	38	108.0	47.4	1006.9	875.4	1110.2	1113.3	1032.97
49.2	35.6	45.2	54.5	1056.1	911	1155.4	1167.8	1078.07

63.3	26.5	55.0	72.7	1119.4	937.5	1210.4	1240.5	1129.47
36.7	34	44.0	52.5	1156.1	971.5	1254.4	1293	1172.97
52.1	24	30.9	55.9	1208.2	995.5	1285.3	1348.9	1209.90
65.5	60.4	43.0	48.5	1273.7	1055.9	1328.3	1397.4	1260.53
49.7	52	28.8	40.6	1323.4	1107.9	1357.1	1438	1301.00
80.1	60	34.6	59.8	1403.5	1167.9	1391.7	1497.8	1352.47
48.8	30	37.9	48.9	1452.3	1197.9	1429.6	1546.7	1391.40
55.7	59.9	32.2	34.8	1508	1257.8	1461.8	1581.5	1433.70
36.3	32.2	43.6	43.3	1544.3	1290	1505.4	1624.8	1473.40
45.6	37.5	32.0	38.6	1589.9	1327.5	1537.4	1663.4	1509.43
32.9	50.4	48.1	38.4	1622.8	1377.9	1585.5	1701.8	1555.07

Appendix-D3: Rainfall region classifications of Ethiopia and Bonga town was found in a region of B1

Appendix-E: Infiltration parameters

Sub catch	Suction head	Conductivity	Initial deficit
S1	2.5	0.5	0.22
S2	2.15	0.8	0.2

S3	2.25	0.95	0.221
S4	3.5	0.67	0.18
S5	2.1	0.85	0.201
S6	2	1.3	0.209
S7	2.25	0.8	0.19
S8	1.8	1.25	0.175
S9	1.85	1.4	0.18
S10	2.15	1.3	0.22
S11	1.75	0.95	0.195
S12	1.7	1	0.206
S13	2.43	1.1	0.211
S14	2.35	1.3	0.18
S15	2.2	1.35	0.202
S16	2.1	1.05	0.203
S17	2.05	1.45	0.21
S18	2.08	1.22	0.2
S19	1.65	1.32	0.209
S20	2.18	0.97	0.269
S21	1.9	1.3	0.29
S22	2.11	1.27	0.243
S23	1.68	1.4	0.4
S24	1.88	1.15	0.25
S25	1.65	1.42	0.19
S26	2.32	1.34	0.27
S27	2.31	1.25	0.15

Appendix-F: Node depth

Node	Type	average depth(m)	maximum depth (m)	maximum HGL (m)	Hour of Depth

J1	Junction	0.04	0.06	1766.06	0:33
J10	Junction	0.29	0.45	1720.45	0:45
J100	Junction	0.08	0.14	1729.14	0:30
J101	Junction	0.09	0.16	1728.16	0:30
J102	Junction	0.09	0.16	1727.16	0:33
J103	Junction	0	0	1731	0:00
J104	Junction	0	0	1729	0:00
J105	Junction	0	0	1726	0:00
J106	Junction	0	0	1725	0:00
J107	Junction	0	0	1724	0:00
J108	Junction	0	0	1734	0:00
J109	Junction	0	0	1736	0:00
J11	Junction	0.3	0.47	1715.47	0:45
J110	Junction	0	0	1735	0:00
J111	Junction	0	0	1732	0:00
J112	Junction	0	0	1727	0:00
J113	Junction	0	0	1726	0:00
J114	Junction	0	0	1725	0:00
J115	Junction	0	0	1731	0:00
J116	Junction	0	0	1728	0:00
J12	Junction	0.31	0.48	1712.48	0:45
J13	Junction	0.14	0.25	1728.25	0:33
J14	Junction	0.14	0.25	1729.25	0:33
J15	Junction	0.07	0.12	1730.12	0:33
J16	Junction	0.08	0.13	1731.13	0:33
J17	Junction	0.1	0.17	1732.17	0:33
J18	Junction	0	0	1730	0:00
J19	Junction	0	0	1729	0:00
J2	Junction	0.05	0.07	1739.07	0:33
J20	Junction	0.06	0.12	1728.12	0:30

J21	Junction	0.06	0.12	1727.12	0:30
J23	Junction	0.21	0.4	1719.4	0:33
J24	Junction	0.17	0.31	1718.31	0:33
J25	Junction	0.11	0.19	1715.19	0:33
J26	Junction	0.24	0.44	1714.44	0:33
J27	Junction	0.23	0.43	1713.43	0:33
J28	Junction	0.1	0.16	1735.16	0:30
J29	Junction	0.1	0.16	1729.16	0:33
J3	Junction	0.08	0.14	1731.14	0:33
J30	Junction	0.15	0.22	1727.22	0:33
J31	Junction	0.13	0.22	1728.22	0:33
J32	Junction	0.16	0.28	1710.28	0:33
J33	Junction	0.15	0.27	1707.27	0:33
J34	Junction	0.39	0.62	1700.62	0:36
J35	Junction	0.02	0.03	1727.03	0:30
J36	Junction	0.12	0.21	1721.21	0:33
J37	Junction	0.11	0.19	1712.19	0:33
J38	Junction	0.14	0.25	1706.25	0:33
J39	Junction	0.14	0.25	1700.25	0:33
J4	Junction	0.08	0.14	1752.14	0:33
J40	Junction	0.1	0.14	1727.14	0:45
J41	Junction	0.1	0.14	1723.14	0:45
J42	Junction	0.15	0.22	1705.22	0:33
J43	Junction	0.15	0.22	1703.22	0:33
J44	Junction	0.23	0.35	1701.35	0:33
J45	Junction	0.24	0.35	1699.35	0:45
J46	Junction	0.45	0.68	1692.68	0:45
J47	Junction	0.45	0.68	1688.68	0:45
J48	Junction	0.43	0.63	1685.63	0:45
J49	Junction	0.43	0.63	1682.63	0:45

J5	Junction	0.08	0.14	1750.14	0:33
J50	Junction	0.05	0.08	1727.08	0:30
J51	Junction	0.05	0.08	1719.08	0:33
J52	Junction	0.05	0.08	1710.08	0:33
J53	Junction	0.05	0.08	1699.08	0:33
J54	Junction	0.06	0.09	1692.09	0:33
J55	Junction	0.06	0.1	1688.1	0:33
J56	Junction	0.1	0.17	1726.17	0:33
J57	Junction	0.12	0.21	1722.21	0:33
J58	Junction	0.22	0.4	1720.4	0:33
J59	Junction	0.04	0.07	1726.07	0:30
J6	Junction	0.06	0.11	1739.11	0:33
J60	Junction	0.04	0.07	1722.07	0:30
J61	Junction	0.04	0.07	1709.07	0:33
J62	Junction	0.05	0.09	1703.09	0:30
J63	Junction	0.07	0.12	1693.12	0:33
J64	Junction	0.07	0.12	1688.12	0:33
J65	Junction	0.11	0.19	1683.19	0:33
J66	Junction	1.16	1.43	1679	0:33
J67	Junction	0.16	0.27	1693.27	0:33
J68	Junction	0.2	0.35	1688.35	0:33
J69	Junction	0.2	0.35	1685.35	0:33
J7	Junction	0.08	0.14	1733.14	0:33
J70	Junction	1.04	1.25	1676.5	0:33
J71	Junction	0	0	1731	0:00
J72	Junction	0.15	0.25	1725.25	0:33
J73	Junction	0.15	0.25	1722.25	0:33
J74	Junction	0	0	1727	0:00
J75	Junction	0	0	1726	0:00
J76	Junction	0.03	0.06	1769.06	0:30

J77	Junction	0.03	0.06	1761.06	0:33
J78	Junction	0.04	0.07	1738.07	0:33
J79	Junction	0.04	0.07	1732.07	0:33
J8	Junction	0.13	0.22	1729.22	0:33
J80	Junction	0.03	0.06	1729.06	0:33
J81	Junction	0.04	0.06	1773.06	0:45
J82	Junction	0.05	0.09	1769.09	0:30
J83	Junction	0.06	0.1	1741.1	0:30
J84	Junction	0.08	0.13	1738.13	0:30
J85	Junction	0.08	0.13	1736.13	0:33
J86	Junction	0.1	0.17	1733.17	0:33
J87	Junction	0.06	0.08	1742.08	0:45
J88	Junction	0.06	0.09	1740.09	0:45
J89	Junction	0.06	0.09	1735.09	0:45
J9	Junction	0.13	0.22	1725.22	0:33
J90	Junction	0.06	0.08	1730.08	0:45
J91	Junction	0.31	0.48	1710.48	0:45
J92	Junction	0.27	0.42	1702.42	0:45
J93	Junction	0.07	0.1	1710.1	0:30
J94	Junction	0.1	0.17	1705.17	0:33
J95	Junction	0.1	0.17	1704.17	0:33
J96	Junction	0.23	0.34	1753.34	0:45
J97	Junction	0.23	0.34	1750.34	0:45
J98	Junction	0.12	0.17	1738.17	0:45
J99	Junction	0.05	0.07	1722.07	0:30
out1	Outfall	0.51	0.76	1673.76	0:33
out2	Outfall	0.39	0.62	1690.62	0:36

Appendix-G: Link inflow

Link	Type	maximum flow CMS	Flow	hour of maximum Flow	maximum velocity m/sec	maximum full Flow	maximum full Depth

C1	Conduit	0.133	0	0:33	2.81	0.01	0.05
C10	Conduit	2.669	0	0:45	5.96	0.22	0.3
C100	Conduit	0.961	0	0:45	9.03	0.04	0.11
C101	Conduit	0.463	0	0:30	6.23	0.02	0.07
C102	Conduit	0.464	0	0:33	4.45	0.04	0.1
C103	Conduit	0.467	0	0:33	2.76	0.09	0.17
C104	Conduit	0.468	0	0:33	3.64	0.06	0.13
C105	Conduit	0.271	0	0:30	2.01	0.06	0.14
C106	Conduit	0.274	0	0:33	1.75	0.08	0.16
C107	Conduit	0.274	0	0:33	3.6	0.03	0.08
C108	Conduit	0	0	0:00	0	0	0
C109	Conduit	0	0	0:00	0	0	0
C11	Conduit	2.666	0	0:45	5.7	0.23	0.31
C110	Conduit	0	0	0:00	0	0	0
C111	Conduit	0	0	0:00	0	0	0
C112	Conduit	0	0	0:00	0	0	0
C113	Conduit	0	0	0:00	0	0	0
C114	Conduit	0	0	0:00	0	0	0
C115	Conduit	0	0	0:00	0	0	0
C116	Conduit	0	0	0:00	0	0	0
C117	Conduit	0	0	0:00	0	0	0
C118	Conduit	0	0	0:00	0	0	0
C119	Conduit	0	0	0:00	0	0	0
C12	Conduit	0.586	0	0:33	2.36	0.16	0.25
C120	Conduit	0	0	0:00	0	0	0
C121	Conduit	0	0	0:00	0	0	0
C13	Conduit	0.304	0	0:33	2.49	0.05	0.12
C14	Conduit	0.305	0	0:33	2.9	0.04	0.11
C15	Conduit	0.305	0	0:33	2.26	0.06	0.13
C17	Conduit	0	0	0:00	0	0	0

C18	Conduit	0	0	0:00	0	0	0
C19	Conduit	0.2	0	0:30	1.74	0.05	0.12
C2	Conduit	0.214	0	0:33	2.91	0.02	0.05
C21	Conduit	0.849	0	0:33	2.73	0.13	0.21
C22	Conduit	0.848	0	0:33	5.79	0.04	0.1
C23	Conduit	0.847	0	0:33	4.45	0.07	0.13
C24	Conduit	1.033	0	0:33	2.4	0.21	0.29
C25	Conduit	0.199	0	0:30	4.2	0.01	0.05
C26	Conduit	0.594	0	0:33	3.73	0.08	0.16
C27	Conduit	0.594	0	0:33	4.2	0.07	0.14
C28	Conduit	0.346	0	0:33	1.55	0.13	0.22
C29	Conduit	1.313	0	0:33	4.77	0.11	0.18
C3	Conduit	0.346	0	0:33	2.49	0.07	0.14
C30	Conduit	1.31	0	0:33	4.83	0.11	0.18
C31	Conduit	1.31	0	0:36	4.98	0.1	0.18
C32	Conduit	0.061	0	0:30	1.9	0.01	0.03
C33	Conduit	1.004	0	0:33	5.48	0.06	0.12
C34	Conduit	1.004	0	0:33	5.25	0.07	0.13
C35	Conduit	1.549	0	0:33	6.3	0.09	0.16
C36	Conduit	0.45	0	0:45	3.12	0.07	0.14
C37	Conduit	0.45	0	0:45	5.05	0.02	0.06
C38	Conduit	0.816	0	0:33	3.71	0.08	0.15
C39	Conduit	1.284	0	0:33	5.78	0.08	0.15
C4	Conduit	0.347	0	0:33	4.38	0.03	0.08
C40	Conduit	1.286	0	0:33	3.64	0.16	0.24
C41	Conduit	2.651	0	0:45	7.47	0.16	0.24
C42	Conduit	2.643	0	0:45	3.9	0.38	0.45
C43	Conduit	2.642	0	0:45	7.58	0.16	0.23
C44	Conduit	4.085	0	0:45	6.48	0.34	0.42
C45	Conduit	0.378	0	0:33	4.97	0.02	0.05

C46	Conduit	0.378	0	0:33	9.23	0.01	0.03
C47	Conduit	0.381	0	0:33	4.8	0.02	0.05
C48	Conduit	0.383	0	0:33	4.74	0.02	0.05
C49	Conduit	0.384	0	0:33	4.25	0.02	0.06
C5	Conduit	0.349	0	0:33	3.31	0.04	0.11
C50	Conduit	0.384	0	0:33	3.99	0.02	0.06
C51	Conduit	0.585	0	0:33	3.53	0.09	0.17
C52	Conduit	0.581	0	0:33	6.19	0.02	0.1
C53	Conduit	0.856	0	0:33	4.04	0.08	0.14
C54	Conduit	0.854	0	0:33	2.16	0.18	0.26
C55	Conduit	0.19	0	0:30	2.67	0.02	0.07
C56	Conduit	0.189	0	0:33	2.95	0.01	0.04
C57	Conduit	0.19	0	0:33	2.85	0.01	0.04
C58	Conduit	0.398	0	0:33	4.3	0.02	0.06
C59	Conduit	0.401	0	0:33	3.46	0.03	0.08
C6	Conduit	0.351	0	0:33	2.54	0.07	0.14
C60	Conduit	0.403	0	0:33	3.26	0.03	0.08
C61	Conduit	0.623	0	0:33	3.22	0.07	0.13
C62	Conduit	1.551	0	0:33	7.11	0.08	0.15
C63	Conduit	1.554	0	0:33	5.65	0.11	0.18
C64	Conduit	1.556	0	0:33	4.41	0.16	0.24
C65	Conduit	1.554	0	0:33	5.6	0.11	0.19
C66	Conduit	0	0	0:00	0	0	0
C67	Conduit	0.942	0	0:33	3.83	0.15	0.25
C68	Conduit	0.942	0	0:33	4.57	0.07	0.14
C7	Conduit	0.815	0	0:33	6.37	0.06	0.13
C70	Conduit	4.083	0	0:45	7.75	0.27	0.35
C71	Conduit	5.643	0	0:33	7.4	0.44	0.51
C72	Conduit	4.106	0	0:33	2.76	1	1
C73	Conduit	0	0	0:00	0	0	0

C74	Conduit	0	0	0:00	0	0	0
C75	Conduit	0.135	0	0:33	2.41	0.02	0.06
C76	Conduit	0.136	0	0:33	2.82	0.01	0.05
C77	Conduit	0.136	0	0:33	2.08	0.02	0.07
C78	Conduit	0.136	0	0:33	2.43	0.02	0.06
C79	Conduit	0.136	0	0:33	2.31	0.02	0.06
C8	Conduit	0.818	0	0:33	3.7	0.06	0.11
C80	Conduit	0.134	0	0:33	2.11	0.02	0.06
C81	Conduit	0.343	0	0:33	3.85	0.03	0.09
C82	Conduit	0.303	0	0:30	3.14	0.04	0.1
C83	Conduit	0.3	0	0:33	2.29	0.06	0.13
C84	Conduit	0.302	0	0:33	2.66	0.05	0.11
C85	Conduit	0.305	0	0:33	1.75	0.06	0.12
C86	Conduit	0.197	0	0:45	2.36	0.03	0.08
C87	Conduit	0.197	0	0:45	2.27	0.03	0.09
C88	Conduit	0.196	0	0:45	2.4	0.03	0.08
C89	Conduit	0.196	0	0:45	2.63	0.03	0.07
C9	Conduit	0.818	0	0:33	7.16	0.02	0.06
C90	Conduit	2.665	0	0:45	5.57	0.24	0.32
C91	Conduit	2.661	0	0:45	6.37	0.2	0.28
C92	Conduit	0.941	0	0:33	4.89	0.03	0.11
C93	Conduit	2.661	0	0:45	9.74	0.11	0.18
C94	Conduit	4.576	0	0:36	7.38	0.34	0.41
C98	Conduit	0.965	0	0:45	2.83	0.24	0.34
C99	Conduit	0.962	0	0:45	5.69	0.09	0.17

Appendix-G: Node inflow

Node	Type	maximum lateral inflow CMS	maximum inflow CMS	hour of maximum Inflow	lateral inflow volume 10^6 ltr	total inflow volume 10^6	flow balance error %

						ltr	
J1	Junction	0	0.134	0:33	0	0.589	0
J10	Junction	1.029	2.671	0:45	4.21	11.4	0
J100	Junction	0.277	0.277	0:30	0.962	0.954	0
J101	Junction	0	0.271	0:30	0	0.95	0
J102	Junction	0	0.274	0:33	0	0.943	0
J103	Junction	0	0	0:00	0	0	0
J104	Junction	0	0	0:00	0	0	0
J105	Junction	0	0	0:00	0	0	0
J106	Junction	0	0	0:00	0	0	0
J107	Junction	0	0	0:00	0	0	0
J108	Junction	0	0	0:00	0	0	0
J109	Junction	0	0	0:00	0	0	0
J11	Junction	0	2.669	0:45	0	11.4	0
J110	Junction	0	0	0:00	0	0	0
J111	Junction	0	0	0:00	0	0	0
J112	Junction	0	0	0:00	0	0	0
J113	Junction	0	0	0:00	0	0	0
J114	Junction	0	0	0:00	0	0	0
J115	Junction	0	0	0:00	0	0	0
J116	Junction	0	0	0:00	0	0	0
J12	Junction	0	2.666	0:45	0	11.4	0
J13	Junction	0	0.586	0:33	0	1.94	0
J14	Junction	0.296	0.582	0:33	0.951	1.95	0
J15	Junction	0	0.305	0:33	0	1.01	0
J16	Junction	0	0.305	0:33	0	1.01	0
J17	Junction	0	0.305	0:33	0	1.01	0
J18	Junction	0	0	0:00	0	0	0
J19	Junction	0	0	0:00	0	0	0
J2	Junction	0.08	0.213	0:33	0.341	0.92	0

J20	Junction	0.204	0.204	0:30	0.663	0.658	0
J21	Junction	0	0.2	0:30	0	0.655	0
J23	Junction	0	0.854	0:33	0	2.85	0
J24	Junction	0	0.849	0:33	0	2.84	0
J25	Junction	0	0.848	0:33	0	2.84	0
J26	Junction	0	1.045	0:33	0	3.49	0
J27	Junction	0.292	1.314	0:33	1.08	4.54	0
J28	Junction	0.598	0.598	0:30	2.37	2.35	0
J29	Junction	0	0.594	0:33	0	2.34	0
J3	Junction	0.264	0.814	0:33	0.883	2.95	0
J30	Junction	0	0.94	0:33	0	3.52	0
J31	Junction	0.22	0.343	0:33	0.714	1.19	0
J32	Junction	0	1.313	0:33	0	4.54	0
J33	Junction	0	1.31	0:33	0	4.53	0
J34	Junction	0.858	4.575	0:36	3.03	18.9	0
J35	Junction	0.063	0.063	0:30	0.205	0.204	0
J36	Junction	0	1.003	0:33	0	3.7	0
J37	Junction	0	1.004	0:33	0	3.69	0
J38	Junction	0.568	1.543	0:33	1.92	5.59	0
J39	Junction	0	1.549	0:33	0	5.58	0
J4	Junction	0	0.343	0:33	0	1.18	0
J40	Junction	0.255	0.451	0:45	1.24	2.1	0
J41	Junction	0	0.45	0:45	0	2.09	0
J42	Junction	0.405	0.814	0:33	1.52	3.59	0
J43	Junction	0	1.284	0:33	0	5.37	0
J44	Junction	0	1.284	0:33	0	5.36	0
J45	Junction	1.451	2.651	0:45	6.76	12	0
J46	Junction	0	2.651	0:45	0	12	0
J47	Junction	0	2.643	0:45	0	11.9	0
J48	Junction	1.102	4.087	0:45	5.38	18.7	0

J49	Junction	0	4.085	0:45	0	18.7	0
J5	Junction	0	0.346	0:33	0	1.17	0
J50	Junction	0.383	0.383	0:30	1.52	1.5	0
J51	Junction	0	0.378	0:33	0	1.5	0
J52	Junction	0	0.378	0:33	0	1.5	0
J53	Junction	0	0.381	0:33	0	1.49	0
J54	Junction	0	0.383	0:33	0	1.49	0
J55	Junction	0	0.384	0:33	0	1.49	0
J56	Junction	0	0.585	0:33	0	1.94	0
J57	Junction	0	0.855	0:33	0	2.87	0
J58	Junction	0	0.856	0:33	0	2.87	0
J59	Junction	0.194	0.194	0:30	0.634	0.629	0
J6	Junction	0	0.347	0:33	0	1.17	0
J60	Junction	0	0.19	0:30	0	0.627	0
J61	Junction	0	0.189	0:33	0	0.623	0
J62	Junction	0.216	0.398	0:30	0.715	1.33	0
J63	Junction	0	0.398	0:33	0	1.33	0
J64	Junction	0	0.401	0:33	0	1.32	0
J65	Junction	0.224	0.615	0:33	0.754	2.07	0
J66	Junction	0	4.562	0:45	0	20.8	0
J67	Junction	0	1.551	0:33	0	5.58	0
J68	Junction	0	1.554	0:33	0	5.57	0
J69	Junction	0	1.556	0:33	0	5.56	0
J7	Junction	0	0.349	0:33	0	1.17	0
J70	Junction	0	5.66	0:33	0	25.6	0
J71	Junction	0	0	0:00	0	0	0
J72	Junction	0	0.941	0:33	0	3.51	0
J73	Junction	0	0.942	0:33	0	3.5	0
J74	Junction	0	0	0:00	0	0	0
J75	Junction	0	0	0:00	0	0	0

J76	Junction	0.137	0.137	0:30	0.496	0.492	0
J77	Junction	0	0.135	0:33	0	0.489	0
J78	Junction	0	0.136	0:33	0	0.485	0
J79	Junction	0	0.136	0:33	0	0.481	0
J8	Junction	0	0.815	0:33	0	2.95	0
J80	Junction	0	0.136	0:33	0	0.48	0
J81	Junction	0.134	0.134	0:45	0.6	0.594	0
J82	Junction	0.353	0.353	0:30	1.19	1.18	0
J83	Junction	0.307	0.307	0:30	1.04	1.03	0
J84	Junction	0	0.303	0:30	0	1.03	0
J85	Junction	0	0.3	0:33	0	1.02	0
J86	Junction	0	0.302	0:33	0	1.02	0
J87	Junction	0.197	0.197	0:45	0.91	0.901	0
J88	Junction	0	0.197	0:45	0	0.899	0
J89	Junction	0	0.197	0:45	0	0.891	0
J9	Junction	0	0.818	0:33	0	2.94	0
J90	Junction	0	0.196	0:45	0	0.885	0
J91	Junction	0	2.665	0:45	0	11.4	0
J92	Junction	0	2.661	0:45	0	11.3	0
J93	Junction	0	0.463	0:30	0	1.79	0
J94	Junction	0	0.464	0:33	0	1.79	0
J95	Junction	0	0.467	0:33	0	1.78	0
J96	Junction	0.971	0.971	0:45	4.39	4.34	0
J97	Junction	0	0.965	0:45	0	4.31	0
J98	Junction	0	0.962	0:45	0	4.3	0
J99	Junction	0.468	0.468	0:30	1.81	1.8	0
out1	Outfall	0	5.643	0:33	0	25.6	0
out2	Outfall	0	4.576	0:36	0	18.8	0