ADDIS ABABA UNIVERSITY

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SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

Urbanization and Options for Sustainable Drainage Management

(A Case Study of Yeka Sub- City)

A thesis submitted and presented to the School of Graduate Studies of Addis Ababa University in partial fulfillment of the Degree of Masters of Science in Civil and Environmental Engineering (Major Hydraulic Engineering).

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APPROVAL

The undersigned have examined the thesis entitled ‘Urbanization and Options for Sustainable Drainage Management’ presented by **Ataklti Hagos**, a candidate for the degree of **Master of Science in Hydraulics Engineering** and hereby certify that it is worthy of acceptance.

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I certify that research work titled "Urbanization and Options for Sustainable Drainage Management" is my own work. The work has not been presented elsewhere for assessment. Where material has been used from other sources it has been properly acknowledged / referred.

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ABSTRACT

Flooding of urban areas is a worldwide problem as cities grow rapidly. Due to urbanization increases catchment changes strongly to impervious coverage and by decreasing pervious areas. The main objective of the study was focused on the (1) Identifying which critical parts of the drainage area are prone to frequent flooding, (2) Examine the effectiveness of sustainable urban drainage system (SUDS) on the current storm water drainage system management, (3) Quantifying the surface runoff volume and peak flow reduction after implementing Low impact development Best management practice (LID) (rain barrel and bio retention cell) to handle the excess storm water runoff in Yeka.

Different types of software and modelling tools such as DEM, ARC-GIS, and stormwater management model (SWMM5) were used to estimate the stormwater and Low Impact development (LID) infrastructures were also introduced to observe their effect on the quantity of storm water. The simulated results were calibrated and validated through field data and utilization of different methods of estimation such as manning formula. An IDF curve was developed utilizing collected rainfall data. Simulated SWMM5 parameters were calibrated and validated using the observed rainfall runoff data and the total surface runoff volume 37.51x$10^3$$m^3$ and peak runoff 21.84 m^3/sec model simulated results respectively without low impact development (LID). The Calibration result of 10 day (July rainfall) and the Validation results on the 10 day (August rainfall) NSE were 0.53 and 0.78 respectively. After implementing LID (Bio Retention and Rain Barrel) scenarios into SWMM5 of the surface runoff reduction and peak outfall reduction was 33.43x$10^3$$m^3$ (10.87%) and18.69m^3/sec (14.42%) respectively.

Generally the low impact development technology where shown the positive effect on the reduction of runoff volume in the urban infrastructure impervious areas.

Key Words; Sustainable Urban Drainage System, Storm Water Management Model (SWMM5), Low Impact Development (LID).

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ABBREVIATION

AAIT	Addis Ababa Institute Technology
AACRA	Addis Ababa City Road Authority
A.M.S.L	Above Mean Sea Level
Arc-GIS	Geographic Information System
BMP	Best Management Practice
BRC	Bio Retention Cell
CMS	Cubic Meter per Second
DEM	Digital Elevation Model
ECDF	Empirical Cumulative Distribution Function
EPA	Environmental Protection Agency
ERA	Ethiopia Road Authority
GOF	Goodness of Fit
GPS	Global Position System
Ha	Hectare
IDF	Intensity Duration Frequency
LID	Low Impact Development
L-THIA-LID	Long-Term Hydrologic Impact Assessment Low Impact Development
MM	Millimeter
MS	Meteorological Station
NMA	National Metrological Agency
NSE	Nash Sutcliffe Model Efficiency
RB	Rain Barrel
SCS	Soil Conservation Service – Curve Number
SUDS	Sustainable Urban Drainage System
SUSTAI	System for Urban Storm Water Treatment and Analysis Integration
SWMM	Storm Water Management Model
TDEC	Tennessee Department of Conservation Resources
TS25	Time Series – 25 Year Rainfall
USEPA	United States Environmental Protection Agency
WSUD	Water Sensitive Urban Design

MoWIE	Ministry of Water Irrigation and Energy

CHAPTER ONE

1. INTROUDUCTION

1.1 Background

Flooding of urban areas is worldwide problem as cities grow and the amount of impermeable surfaces increases generating more surface runoff. Existing storm water drains are typically not capable of handling such increases in runoff. Therefore, greater volumes of water are left on the surface. The reason for flooding in most areas can be attributed to the increased runoff resulting from land-use change, particularly the increase in impermeable surfaces. Also, lack of any upgrade or maintenance of the drainage system implies that the system can no longer sustain the runoff volumes.

Flood hazard is generally assessed through the evaluation of its impact parameters, such as water depth, velocity and its associated probability of occurrence. Floods interfere with efficient drainage and economic use of lands for agricultural or industrial purposes. Floods also damage drainage channel, bridge, sewer outfalls and other structures. Human influence is an important factor that many artificial changes in the river system may induce morphological changes and subsequent rising of the water or bed level.

Typically the goal of a drainage system is to convey the excess surface water through underground pipe systems or through open channel system away as quickly as possible. Effective control of rainfall runoff at-source minimizes the necessity of large flow structures. In most developed cities, approximately 40-50% of the impervious urban surface areas are roofs and paved roads (Seema Bardhipur, 2017a).

According to “best management practice technique that reduces the rate and volume of impervious surface runoff has the potential to contribute and to improved storm water management systems”. It reduces runoff and peak flow being at-source detention and retention technology. Increasing vegetated areas in cities has many other benefits regarding to water quantity and quality managements. Urbanization is known to cause increased in volume of storm water runoff and peak flow rates, which leads to changes in Natural flow regime and increases the chance of flooding.

Conventional stormwater management practices mainly focus on reducing peak flow rates and surface runoff volume reduction has traditionally been ignored. Conversely, Low Impact Development (LID) practices seek to increase infiltration to reduce runoff volume and peak flows close to the source as possible and are generally considered being more sustainable solution for urban storm water management. In this study, the effectiveness of two LID practices, rain barrel and bio retention, in mitigating urban flooding was tested within the study area.

Drainage problems in urban areas include flooding, deterioration of roads, land degradation, sedimentation, blockage of drainage facilities and water logging. Urbanization increases impermeability with the increase impervious surfaces (residential, commercial buildings, paved roads, parking lots, etc.).Drainage pattern changes, overland flow gets faster, flooding and environmental problems such as land degradation increases. It is a crucial problem facing the existing and future road infrastructure.

1.2 Statement of the Problem

Storm drainage system of city is ideally aimed to handle peak flow from rainfall return period equal to or greater than their design year. Because of such causes, there is a problem of drainage such as overtopping and flooding of the area during intensive rainfall. Those problems are due to an incremental of rainfall, increasing of pavements in the overall catchment or urbanization, improper drainage design, weakly structural and non-structural storm water management practices.

In Addis Ababa, with increasing densification and urban infrastructure development of various types, such as road, building construction, parking lots and industrial parking are leading to increased run-off, greater susceptibility to flooding hazards and pollution of rivers. Some areas in Yeka Sub-City where flood have been observed are listed as below.

- Gurdshola to Merri Railways.
- Merry Roundabout (Tsehayreal estate).
- MegenagnatoWossenRoad.
- Lambert Bus station.

1.3 Objectives of the Study

1.3.1 General Objectives

The general objective of this study was to simulate the peak flow rate and total runoff volume in Yeka sub-city and identify Options for Sustainable Drainage Management**.**

1.3.2 Specific Objectives

- Identify areas most prone to flooding problems and its causes.
- Quantifying the peak flow rate and total volume of storm water drainage system of the study areas.
- Examine the effectiveness of sustainable urban drainage technology on runoff reduction in study areas.

1.4 Significance of the Study

Studying of the urbanization and options for sustainable drainage management helps to give effective and urgent action plan that requires estimation of inundation levels. Thus, the results of the study are important for the following reasons:

- To inform the flood prone area coverage in the downstream urban area.
- To notify the effect of urban flooding on the human life and constructed infrastructure area and other developed economic activity.
- To develop LID action plan for the Yeka sub city on the flood inundation map developed and the potential risk that can exist.
- Will support other researchers to do analysis of similar SUDS technology.

1.5 Scope of the Study

The scope of the study is limited to 450ha of modeled area which is focused in flood inundations. According this, the study analysis was suggested on the development of LID (i.e. Rain Barrel and Bio Retention Cell Structure) due to cost efficiency and easy to sustain.

CHAPTER TWO

2. LITERATURE REVIEW

2.1 Urbanization

Due to urbanization, the study catchment area is rapidly changing from pervious areas to impervious areas. For example, forests are replaced with buildings and roads. Due to this, increasing the storm water runoff, peak flow and time to peak increased (Tikkanen,2013). Urbanization also causes increased pollutant and sediment delivery that contaminate lakes and streams due to the unfiltered and rapid transport of chemical, sand and nutrients (Zakia Raihan Alam, 2014). Global warming and high precipitation events have worsen the impact of hydrologic cycle on the urbanization (Moheseen, 2015).

In several situations, forest and open space are replaced by houses, roadways, commercial and industrial areas. This transition in land has environmental impacts. For instance watershed storage (interception, infiltration and depression storage) is greatly modified peak flow and runoff increase and water quality decreases. Figure 2-1 Shows how runoff, Evapo - transpiration, deep infiltration and shallow infiltration percentages are affected while the percentage of impervious surfaces increases.

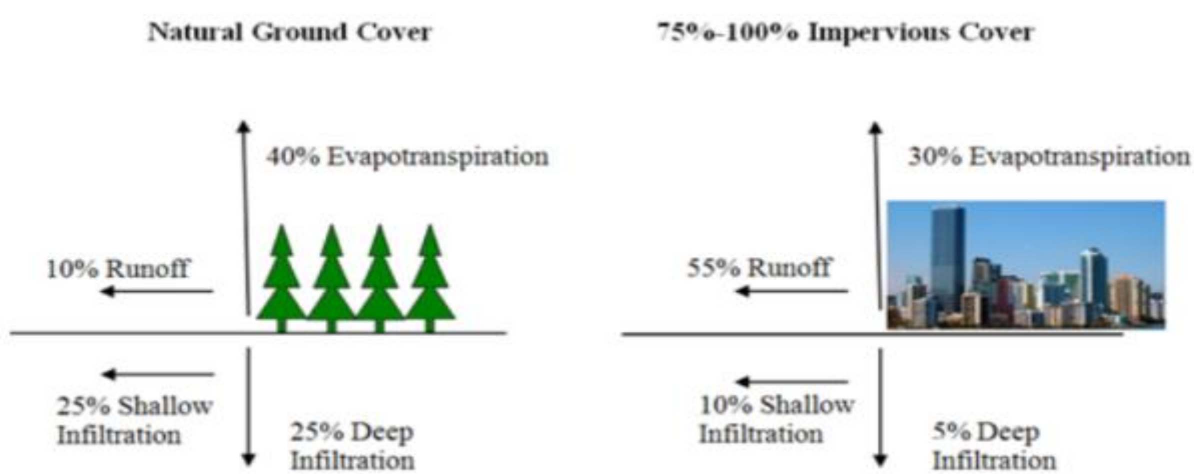

Figure 2-1. Influence of urbanization in runoff generation (Source: EPA 2003).

2.2 Rainfall Intensity-Duration-Frequency Relationship

The intensity-duration-frequency (IDF) relationship is one of the pre requisite statistics in water recourses engineering to planning, development, management and to assess the vulnerability of hydraulic structures. IDF Curves describe the amount of rainfall in a watershed area for a given period of time. IDF relationships have been established since 1932 in developed countries (see Chow (1988), Dupont and Allen (2006)).The IDF relationship is a mathematical relationship between the rainfall intensity, the duration, and the return period using extreme rainfall data. This tool used by engineers to design safe and cost effective structures for certain return periods.

The magnitude of an extreme rainfall event has an inverse relation to its occurrence frequency. Therefore, the severe rainfall events have less frequency compared to moderate rainfall events. The frequency analysis of rainfall data is to relate the magnitude of extreme events to their frequency of occurrence using the probability distribution. One input of this study was to develop IDF using measured daily rainfall depth data from Ethiopian National Metrological Station. In addition, an empirical formula derived for the rainfall intensity analysis with different returning period and rainfall duration compared to IDF-Curves.

2.2.1 Fill Missing Data and Check Consistency

Rainfall is an integral component in the hydrologic cycle. Engineers must be able to quantify rainfall in order to design structures impacted by or dealing with the collection, conveyance and storage of excess rainfall. Rainfall missing data were checked using normal ratio method were used Because of the normal annual precipitation of the index stations lies exceeds ±10% of normal annual precipitation of interpolation station. In such cases, the normal ratio method is recommended to fill missing precipitation data. For example rainfall data at day 1 is missed from station X having the mean annual rainfall of Nx and there are three surrounding stations with mean annual rainfall of N1, N2, and N3 then the missing data Px can be estimated (Yilmaz, 2011).

$$\mathrm{PX} = \frac{1}{3} * \left(\mathrm{P1} * \frac{\mathrm{NX}}{\mathrm{N1}} + \mathrm{P2} * \frac{\mathrm{NX}}{\mathrm{N2}} + \mathrm{P3} * \frac{\mathrm{NX}}{\mathrm{N3}}\right)$$

Where:

P_X: missing rainfall data (daily, monthly or yearly)

P1, P2 and P3 rainfall data at nearest different station (daily, monthly or yearly)

N_X: mean annual rainfall at missed station. N1, N2, and N3- mean annual rainfall at different nearest station.

2.2.2 Test for Outliers

An outlier is an observation that deviates significantly from the bulk of the data, which may be due to errors in data collection or recording or due to natural causes. The presence of outliers in the data causes difficulties when fitting a distribution to the data. Low and high outliers are both possible and have different effects on the analysis. To estimate in making projections of the magnitude and frequency of rainfall, hydrological data is essential (Maidment and Mays, 2015). Extreme hydrological event and long term data series is required for reliable estimations.

The Grubbs and Beck test (G-B) may be used to detect outliers. In this test the quantities x_H and x_L are calculated by using the equations mentioned here below.

$X_H = \exp\left(\bar{X} + KN.S\right)$ $X_L = \exp(\bar{X} - KN.S)$

Where X and S are the mean and standard deviation of the natural logarithm of the sample respectively and KN is the G-B statistic by Grubbs and Becks (1972). At the 10% significance level, the following approximately proposed by (Pilon et al.1985) is used.

Where N is the sample size. $K_N = -3.62201 + 6.28446N^{\frac{1}{4}} - 2.49835N^{\frac{1}{2}} + 0.491436N^{\frac{3}{4}} - 0.037911N$. Sample values greater than X_H are considered to be high outliers, while those less than X_L is considered being low outliers.

2.2.3 Design Rainfall of Shorter Duration

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

$$RRt = (t(b + 24)^n)/(24(b + t)^n)$$

Where,

R_{RT} = Rainfall depth ratio R_t/R_{24}

R_t =RainfalldepthgivendurationR24 = 24hr rainfall depth coefficients b = 0.3 and n = 0.78

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

- Using the trend line equation obtained from Log Pearson type III method of frequency analysis, i.e. y = 11.414ln(x) + 38.795 where y is 24-hour rainfall depth (R24) of a return period x under consideration, R24 is calculated for 2, 5, 10, 25, 50 and 100 year return period. Rearranging the above equation gives
- $R_t = \frac{t}{24} * \frac{(b+24)^n}{(b+t)^n} * R24$ Intensity (mm/hr) I_t =Rt/ t
- Using b = 0.3 and n = 0.92 as suggested by ERA manual results are tabulated for rainfall durations 10, 20, 30 … 180 minutes. The resulting table is graphed for each return period.
- In order to incorporate into software and automate the estimation of runoff representation of intensity duration frequency relationship with an equation is required.

The polynomial equation can be used to represent the IDF curve of 25 year return period to obtain rainfall intensity for a given duration, provided that the duration is below 3 hours. These Equations obtained and representing IDF curves for different return period are presented in the results section.

2.2.4 Extreme Maximum Probability Distributions and Formulas

Frequency analyses of hydrologic data using probability distributions to relate the magnitude of extreme events to their frequency of occurrence. It is generally assumed that a hydrological variable has a certain distribution type. Some of the most common and important probability distributions used in hydrology are the Normal, Log-Normal, Exponential, Gamma, Pearson Type I,II and III, Log Pearson, General Extreme Value I (Gumbell), General Extreme Value II and General Extreme Value III (Weibull).

The normal distribution generally fits to the annual flows of rivers. The log-normal distribution is also used for the same purpose. In hydrology, the gamma distribution has the advantage of having only positive values, since hydrological variables such as rainfall and runoff are always the Gumbal distribution is used in the frequency analysis of floods and the Weibull distribution in the analysis of low flow values observed in rivers. Positive (greater than zero) or equal to zero the assumed distribution is fitted to the sample. Different techniques are used in estimating the parameters. These are listed here in ascending order of efficiency, from the least efficient to the most efficient: the graphical method, the least squares method, the method of moments, and the maximum likelihood method. The recently developed method of probability-weighted moments (L-moments) can also be used. The graphical method for parameter estimation is particularly used for distributions which can be plotted as a straight line on a probability graph paper.

The normal distribution is the most common. It is not easy to use this method to estimate the parameters of distributions which cannot be plotted as a straight line on the probability graph paper, as in the case for gamma distribution. The sum of squares of difference between the coordinates of the observed values and their corresponding values on the fitted distribution should be the smallest in the least-squares method a lower limit value.

The historical rainfall data available is a 24hr duration rainfall hence an appropriate IDF reduction method needs to be used to obtain rainfall intensities of shorter duration. Any probability distribution can be used as the model and its reliability can be check by the goodness of fit tests. Gumbal and Log Pearson Type III methods are used as suggested by Ethiopian Road Authority Drainage Design Manual (ERA, 2013). And their goodness of fit is analyzed in the next section.

2.2.4.1 Normal Distribution

The normal or Gaussian distribution is one of the most popular distributions in statistics. It is also the basis for the Log-normal distribution, which is often used in hydrologic ssapplications.The distribution used in frequency analysis computations is provided as follows:

$$X_T = \overline{X} + K_T S \qquad w = \left[\ln\left(\frac{1}{p^2}\right)\right]^{1/2} \qquad (0 < p \le 0.5)$$

$$z = w - \frac{2.515517 + 0.802853w + 0.010328w^2}{1 + 1.432788w + 0.189269w^2 + 0.001308w^3}$$

2.2.4.2 Log-Normal Distribution

The annual maximum flow series is usually not well approximated by the normal distribution. It is skewed to the right, since flows are only positive in magnitude, while the normal distribution includes negative values. When a data series is left-bounded and positively skewed, a logarithmic transformation of the data may allow the use of normal distribution concepts through the use of the log-normal distribution. This transformation can correct this problem through the conversion of all flow values to logarithms. This is the method used in the log-normal distribution.

$YT = \bar{Y} + KT * SY$When,Cs is considered at 0

2.2.4.3 Gumbel Extreme Value Distribution

Peak discharges commonly have a positive skew. Because, one or more high values in the record result in the distribution are not being log-normally distributed. Hence, the Gumbel extreme value distribution was developed.

$$X_T = \overline{X} + K_T S$$

K_T values are calculated from different return periods

$$K_T = -\frac{\sqrt{6}}{\pi}[0.5772 + \ln(\ln(\frac{T}{T-1}))]$$

2.2.4.4 Log-Pearson Type III Distribution

The log-Pearson type III distribution applies to nearly all series of natural floods and is the most commonly used frequency distribution for peak flows in the United States. It is similar to the normal distribution except that the log-Pearson distribution accounts for the skew, instead of the two parameters, standard deviation and mean. When the skew is small, the log-Pearson distribution approximates a normal distribution.

The basic distribution is:

$$\boldsymbol{YT = \overline{Y} + KT * SY}$$

2.2.4.5 Pearson Type III Distribution

To compute the precipitation depth (XT) associated with each return period and skew coefficient (Cs) KT factor from Table with the vales of positive Skew coefficient (Cs)

$$X_T = \overline{X} + K_T S$$

2.2.5 Goodness of Fit Test

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

2.2.5.1 Chi-Square Test

One of the most commonly used tests for goodness of fit of empirical data to specify theoretical frequency distributions. The test makes comparison between the actual number of observations and the expected number of observations that fall in the class intervals. The expected numbers are calculated by multiplying the expected relative frequency by total number of observations values. The chi-square test statistic is given by the equation as:

$$\mathrm{X}^2 = \sum_{i}^{k} \mathrm{x} \frac{(\mathrm{Oi}-\mathrm{Ei})\hat{}2}{\mathrm{Ei}}$$

Where, Oi is the observed rainfall and Ei is the expected rainfall.

The best probability distribution function was determined by comparing Chi square values obtained from each distribution and the smallest Result is the Best Fit Distribution.

2.2.5.2 Kolmogorov-Smirnov Test

This test is used to decide if a sample comes from a hypothesized continuous distribution. It is based on the empirical cumulative distribution function (ECDF). Assume that we have a random sample.Fn (X) $= \left(\frac{1}{N}\right)$ [Number of Observations $\leq$ X].

The Kolmogorov Simonov statics (D) is based on the largest vertical difference between F(X) and Fn(X).it is defined as it is defined as

$$\mathrm{DN} = \left\langle \begin{matrix} \sup \\ \mathrm{X} \end{matrix} \right\rangle |\mathrm{Fn(X)} - \mathrm{F(X)}|$$

The main principle is comparing different distribution, and then the lower statistics value is better fit to the data using.

2.2.5.3 Anderson-Darling Test

This test gives more weight to the tails than the Kolmogorov-Smirnov and its general test compare the fit of an observed cumulative distribution function to an expected cumulative distribution function.

The Anderson-Darling statistic (A^2) formula is:-

$$A^2 = -n - (\frac{1}{n}) \sum_{i=1}^{n} (2i-1).\,[lnF(Xi) + \ln\big(1 - F(X\mathrm{n} - \mathrm{i} + 1)\big)]$$

2.2.6 Hydrological Models

Storm water Management Software is commonly and commercially available. Nevertheless, different software modelling has provided for different design facilities.

2.2.6.1 Software Name: DRAINS

Storm Water Drainage System design and analysis program (DRAINS) was developed by Water Com Pty Ltd in January1998 at New South Wales, Australia. DRAINS are use for designing and analyzing urban storm water drainage system catchments. This software capable to model drainage systems of all sizes, from small to up to 10 $km^2$by using sub-catchment with ILSAX hydrology and greater using storage routing model hydrology. DRAINS will simulate and convert the rainfall patterns to storm water runoff hydrographs by working through a number of time steps that are fall during the course of a storm event. The simulation runoff will route through channel, networks of pipes and streams.

The connections to CAD and GIS program, automatic design procedure for piped drainage systems and an in-built Help system is also include in this software. In a single package, DRAINS can model the hydrological model using ILSAX, rational method and a storage routing models together with unsteady hydraulic modelling of systems of pipes, open channels in the premium hydraulic model and surface overflow routes. The important functions that are not included in DRAINS are Continuous modelling over long periods including wet and dry conditions. Water quality modeling two-dimensional unsteady flow modeling Life cycle cost.

2.2.6.2 Software Name: MOUSE

Based in Pennsylvania, USA, Urban Drainage and Sewer Model (MOUSE) are created as a comprehensive modelling system. This software is used for analysis of urban drainage and sewer systems including links to GIS. MOUSE also used to simulate spatial variations in flows, water levels, sediment transport and pollution in pipes and open drains. Single or multiple events can be defined by the user and the time period up to several years. Deterministic mathematical modelling tool is use as mathematical formulation of MOUSE model. With user defined the time interval MOUSE produces output a large number of variables.

MOUSE has advantage in the aspect of no limits on the size of model area or included in the simulation and capable to branched and looped network. Continuous type of simulation is used in the MOUSE for nature of simulation. Besides that, this software also can predict the hydraulic deficiency, overflow sites, flood inundation areas, and effect of real-time control. For rainfall-runoff model, the contributing area and hydrological losses are part of model parameter. Topographic maps, drainage plans and aerial photos are an input requirement in MOUSE. The Input data requirement is in spread-sheet format.

2.2.6.3 Software Name: Info Work RS

Info Works River Simulation (Info Work RS) is hydrodynamic modelling software includes full solution modelling of open channels, floodplains, embankments and hydraulic structures.

Using both event based and conceptual hydrological methods, the rainfall-runoff simulation is also available using geographical plan views, sectional view, long sections, spreadsheet and time varying graphical data in full interactive views of data.

The underlying data can be accessed from any graphical and geographical view. In addition to presentation of results in geographical plan, tables, cross section views , long section and model also provide animation showing how a flood event progress. Full flood-mapping capability is provided based on a sophisticated flood-interpolation model overlaid onto an imported ground model. Info Works RS combine the advanced flow simulation engine, both hydrological and hydraulic models, GIS functionality and database storage within one single environment. The basic system architectures is an “Integrated Network Model” links data storage using GIS to hydrologic/hydraulic modeling software suite embedded in Info Works RS.

2.2.6.4 Software Name: HSPF

Hydrological Simulation Program-Fortran (HSPF) simulating the extended periods time of the hydrologic processes on pervious and impervious land surfaces and in streams and well-mixed impoundments. HSPF uses continuous rainfall and other meteorological records to compute stream flow hydrographs. This software is used to simulate one or many pervious or impervious unit areas discharging to one or many river reaches or reservoirs. Any time series for frequency-duration analysis can be done. From 1 minute to 1 day at any time that divides equallyinto1daycanbeused.HSPF also can simulate any period from a minute to hundreds of years. This software is generally used to evaluate the effects of land-use change, reservoir operations, point or non-point source treatment alternatives, or flow diversions. Programs which available separately, will support data in pre-processing and post processing for statistical and graphical analysis of data saved to the Watershed Data Management (WDM) file. The model contains hundreds of process algorithms developed from theory, laboratory experiments, and empirical relations from instrumented watersheds.

HSPF simulated sediment routing by particle size, channel routing, reservoir routing, and constituent routing. Meteorological records of precipitation and estimates of potential Evapo -Transpiration are required for water shed simulation. Physical measurements and related parameters are required to describe the land area, channels, and reservoirs.

2.2.6.5 Software Name: DR3M

Distributed Routing Rainfall-Runoff Model (DR3M) is one of the storm water modelling. This software is created to simulate the storm runoff. It can simulate the routing storm either in system or pipes or natural channel by using rainfall data as an input. This software model is the detail simulation of storm runoff according to user period time selected. DR3M is commonly used to simulating the storm runoff for small urban basins. To calculate the infiltration and pervious area rainfall excess, the Green-Ampt equation is used.

The weakness of this software is it does not simulate interflow and base flow of the basin. Daily precipitation, daily Evapo-transpiration, and short-interval precipitation are required for data requirement. To optimize and calibrate the model short interval discharge is needed.

2.2.6.6 Software Name: SWMM

Developed in 1971, The USEPA Storm Water Management Model (SWMM) has been used for over 30 years and widely used as utilized model in detailed hydrological and hydraulic modelling of storm water and watershed of the catchment. This software is capable to simulate the precipitation movement from the ground surface through channel and pipe network. Single event and a long continuous period of event can be simulating by using SWMM. For a long time, SWMM version is free and being maintained by several numbers of individual and organization. Now, SWMM engine has been rewritten and known as a versionSWMM5.It is managing by USEPA. In this hydrological model concept, every sub-catchment in a basin is treated as non-linear reservoir with a single inflow or rainfall input and will produce the outflow or discharge in term of infiltration, evaporation and surface runoff. The development of a number of SWMM Graphical User Interface (GUI) wrappers such as MIKE-SWMM, XP-SWMM has been created through year of revolution.

2.2.6.7 Software Name: XP-SWMM

XP Storm Water Management Model (XPSWMM) is an inclusive software package for planning, modelling and managing sustainable drainage systems. This software is used to simulate the storm water and sanitary sewer flows as well as treatment in typical LID (WSUD) systems. Hydraulically, flows are simulated in one - dimensional channel and pipes and coupled to two dimension surface grid for comprehensive flood modeling and mapping.

Scientists, engineers as well as resource and asset managers use this software to simulate natural rainfall-runoff processes and the performance of engineered systems that manage our water resources. XPSWMM is used to develop link-node and spatially distributed models that are used for the analysis, design and simulation of storm and wastewater systems. XPSWMM also models flow in natural systems including rivers, lakes, and floodplains with ground water interaction.

2.2.6.8 Software Name: MIKE-SWMM

MIKE-SWMM is one of the SWMM Graphical User Interface (GUI) wrappers. In this MIKE-SWMM, the function is similar to SWMM which is to analyses the urban drainage system and sanitary sewers Combination of hydrology, hydraulics and water quality.

In MIKE-SWMM provides complete graphical and user friendly interface for user. The hydrological model concept, every sub catchment in a basin is treated as non-linear reservoir with single inflow or rainfall input and will produce the outflow or discharge in term of infiltration, evaporation and surface runoff.

2.2.6.9 Software Name: QQS

QQS stand for Quality-Quantity Simulators. The advantage of this software is by using fine interval such as five minutes time interval. This is software capable to perform continuous or single event simulation. QQS is used to simulating the flows in channels or pipes using an implicit finite difference approximation of the kinematic wave equation, backwater analysis, storage routing and pipes under pressure. This software is suitable to use in urban area catchment to simulate it urban storm water modelling.

2.3 Low Impact Development (LID)

Due to climate change urban flooding frequently occurs in the worldwide and also High-speed urbanization has caused rapid changes to the underlying surfaces, resulting in fundamental changes in urban runoff. Urban water logging brings a series of socio-economic losses such as traffic paralysis, loss of property and even human losses.

The effective control of urban water logging is both crucial and difficult to manage for urban storm water runoff. The evaluations of the effectiveness of the various management measures are more important. Various runoff reduction measures have been implemented, including LID, and widely applied throughout the world(Asghar and Garg, 2018). LID is an innovative urban storm water management system that was jointly introduced by the storm water management experts from Programs and Planning Division of Prince George’s County Department of Environmental Resources during the mid-1990s.

At the end of the 1990LID was developed by the United States Environmental Protection Agency (USEPA) with encouragement. It has been generally recognized and adopted by countries all over the world. A series of related storm water management regulations have been formulated in Florida, Chicago, and other locales, and remarkable achievements have been made (Wang, 2015). The idea behind LID is to depress the negative influence of water quantity as well as the quality of the runoff process caused by urbanization to allow regional runoff processes to return to a natural undeveloped state to the largest degree possible. Its meaning has been extended in many countries or regions that have similar ideas regarding storm water management, such as Water-Sensitive Urban Design (WSUD) in Australia, Sustainable Urban Drainage Systems(SUDS) in the UK, and the natural drainage systems in Seattle (Nasrin, 2018).

2.3.1 Commonly Used LID Techniques

Effective low impact development includes the use of both non-structural and structural storm water management measures that are a subset of a larger group of practices and facilities known as Best Management Practices (BMP). As noted above, the BMPs utilized in low impact development, known as LID-BMPs, focus first on minimizing both the quantitative and qualitative changes to a site’s pre-developed hydrology through non-structural practices and then providing treatment as necessary through a network of structural facilities distributed throughout the site. In doing so, low impact development places an emphasis on non-structural stormwater management measures, seeking to maximize their use prior to utilizing structural BMPs. Non-structural BMPs used in low impact development seek to reduce storm water runoff impacts Structural BMPs used to control and treat runoff are also considered LID-BMPs if they perform these functions close to the runoff’s source.

LID techniques are not restricted to land development sites with limited impervious cover but can also be applied to virtually any development site regardless of the impervious coverage, to produce improved site designs and lesser storm water impacts. The common LID practices are bio-retention, green roofs, permeable pavements, rain gardens, vegetative swales, and rain cisterns (Rain Barrel)that are used to create a functionally equivalent hydrologic landscape (Enis Baltaci, 2016). These LID practices play an important role because of their ability to store water, allowing it to infiltrate or releasing it to receiving streams. They also have the benefit of lengthening the flow path and runoff time (Asghar and Garg, 2018).

The two popular LIDs currently used in residential areas are bio-retention cells and rain barrels. An analysis of LID for runoff reduction obtained the benefits of optimized LID implementation in reducing runoff and peak flow rates because LID reduces the need for expensive channel systems such as pipes, channels, and combined sewer systems (Seema Bardhipur, 2017).

2.3.1.1 Rain Barrels

Rainwater tanks are one of the widely-used WSUD approaches for non-potable reuses or outdoor uses (Mogen felt, 2017). These are popular on-site storm water rainwater collection method which store water during a storm event. These storage tanks are usually placed beneath roof downspouts, which capture roof runoff and thus prevent storm water inflow entering the sewer network(Nasrin, 2018).

2.3.1.2 Bio Retention Cells

Bio-retention cell is the most widely applied LID practice throughout the U.S., which restores the natural system function by using design techniques that infiltrate, filter, store, evaporate, and detain runoff close to its source (Trowsdale and Simcock, 2011). Bio-retention cell consists of a grass buffer strip, a sand bed, a pond area, an organic layer of mulch, planting soil, and plants. Runoff water passes across the length of the pond area which consists of organic mulch. Later, water infiltrates into planting soil and sand beds (USEPA, 2000). Some of the bio-retention facilities have under drains which convey the excess water to the storm drain system. Bio retention estimates can reduce the peak flow from 44% to 66% depending on site conditions such as soil and basin slope with substantially delayed time-to-peak (Trowsdale and Simcock, 2011).

2.3.1.3 Green Roofs

They are also known as vegetated roof covers. Green roofs have a surface layer of living plants that grow on the top of a roof a thin soil layer and a special drainage mat below the soil layer. They can retain significant amount of rainfall and roof runoff, then filter through soil layer and drain excess percolated water off the roof (Trowsdale and Simcock, 2011). They have multitude of benefits other than retarding storm water runoff and decreasing flows to sewer network during intense rainfall. This include reducing direct energy uses and urban heat island effects through evaporative cooling, removing sound pollution and improving air quality and biodiversity (Wise et al., 2010). They can also provide green space in dense urban zones and thus, improve community aesthetic.

2.3.1.4 Infiltration Trenches

These are narrow ditches filled with gravel to the ground level. They provide storage and capture storm water runoff from the impervious areas. The captured runoff then infiltrates into the natural soil (Rossman and Huber, 2016). They can significantly reduce runoff volumes that enter sewer system. They can also improve landscape and aesthetic by providing green space (Sample et al., 2014).

2.3.1.5 Swales

These are depressed areas which act as channels to route the surface runoff. Grass or vegetation is used to cover the sliding slopes of the depression areas (Rossman and Huber, 2016).Vegetative swales help to reduce the conveyance capacity of storm water runoff and provide sufficient time to infiltrate the storm water into the natural soil.

2.3.1.6 Detention ponds

They are used to retain storm water runoff from impervious area during storm event and then, completely release through some specific outlets within few hours. They store storm water runoff temporarily and thus, reduce runoff volume and peak flows (Pennino et al., 2016). They have varying styles in terms of manicured or natural appearing vegetation.

2.3.1.7 Permeable Pavements

Permeable pavements are excavated areas where gravel is used to fill the area. Here, porous concrete or asphalt mix is used for paving the surface. Storm water runoff can pass through the permeable surface, filter by the soil layer and then enter the gravel storage zone beneath the pavement. After that, runoff can easily infiltrate the natural soil or convey to storm drain through optional drainage system. They are effective reducing peak runoff and improving groundwater recharge (Patwardhan et al., 2005). They can improve water quality as well by reducing sediments, nutrients and metals (Sample et al., 2014).

2.4 Implementations of LID to SWMM5

There are many types of methods for evaluating the effects of LID measures; the one way method is using models. The relevant models include the Long-Term Hydrologic Impact Assessment Low Impact Development (L-THIA-LID) model (Ma, 2004), the improved SCS-CN model (Parketal.2014b); the System for Urban Storm-water Treatment and Analysis Integration (SUSTAIN)model (Shafique, Kim and Kyung-ho, 2018),And the SWMM model. Widely used in the evaluation and forecasting of the surface runoff process of storm floods by relevant scholars SWMM have a better simulation effect, Compared with other models.

Within LID simulation, SWMM is one of the earliest hydrological models to be supplemented with LID module. The LID module of SWMM provides five single measures for storm runoff control. The hydrological process and its key factors, such as regional surface runoff and peak discharge, can be simulated by applying the LID module combined with the hydraulic module. Hence, the runoff reduction effect of LID can be evaluated through these key factors. The SWMM for different LID measures has also been extensively used in different regions and countries, especially in Germany. Because the SWMM was well applied in both storm runoff simulations and LID evaluations and was appropriate for different types of areas, it was selected as the simulation model.

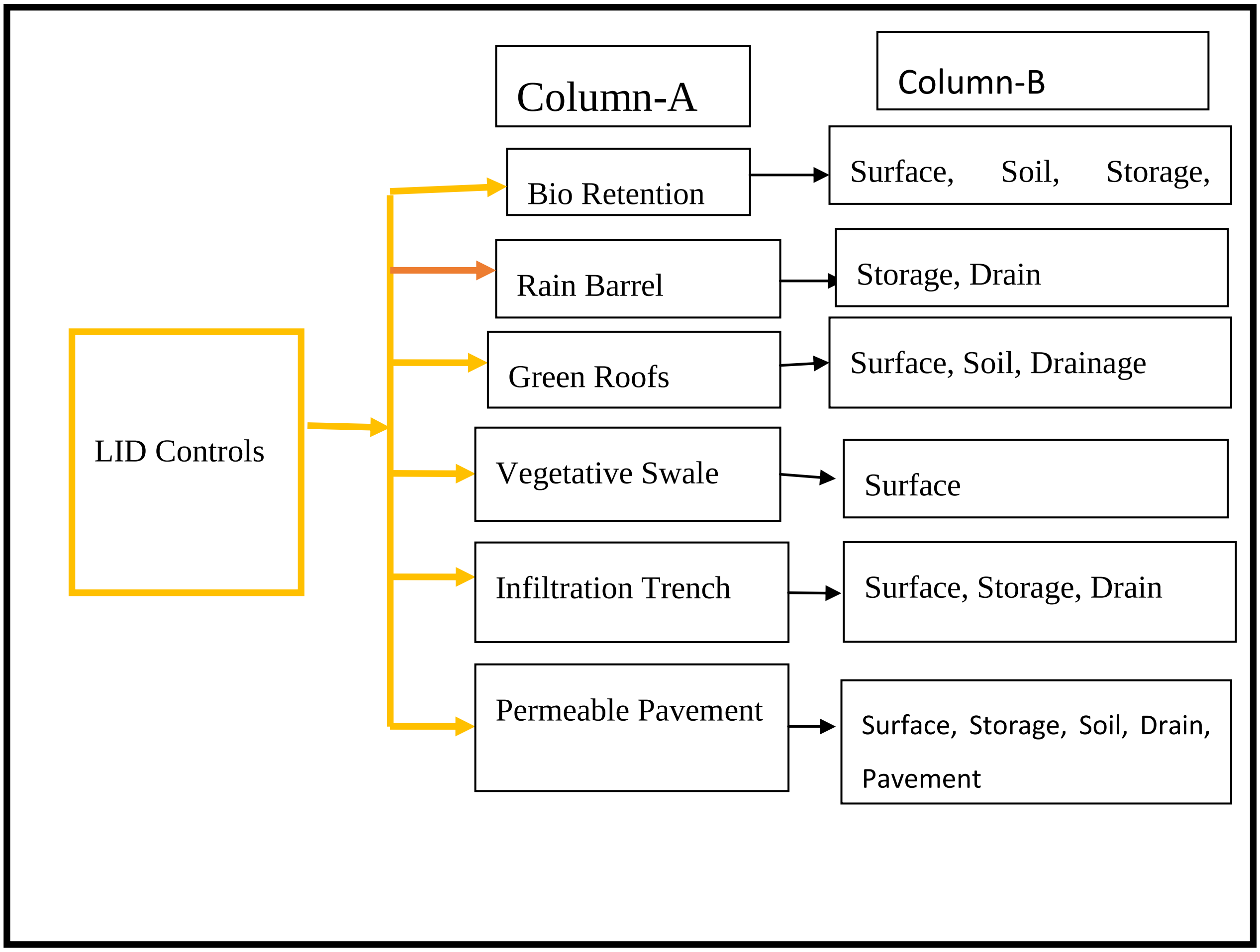

Figure 2-2. Types of different LID Controls in SWMM5 and its Layers(Source: Cleveland state university, 2017).

In the figure 2-2 types of different LID controls in SWMM5 in column -A and column- B which shows types of LID and its input property used to pactice respectively.

CHAPTER THREE

3. MATERIAL AND METHODOLOGY

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

Addis Ababa occupies the central part of Ethiopia and has been serving as the socio economic and political capital of the country since its establishment in 1879 E.C.

It is located between $8^0 45' 00$ to $9^0 05'00''$ N and $38^0 35'00'' to\ 38^0 55' 00'' E$. Its altitude ranges from 2100meter at Akaki (south) to 3139 meter at Entoto (north) .The average altitude of the city is 2380 meter (a.m.s.l) (Admasu, 2017). The city receives its maximum rainfall in the month of June, July, August and September, which is the main rain season in most parts of the country (figure 3.2)(NMA). Maximum monthly temperature is recorded in March, April and May. The main annual temperature ranges from 16^oc to18^oc. Addis Ababa receives an average rainfall of 1255mm per year (Admasu, 2017).

The population of Addis Ababa has grown from 1.4 million in 1984 to 2.1 million in 1994 and then to 2.7 million in 2007 (AACRA, 2009). This growth is mainly caused by rural to urban migration. The rate of urbanization in Addis Ababa is one of the fastest in sub-Saharan African with an annual growth rate of 3.8% Compared to other urban centers in Ethiopia.

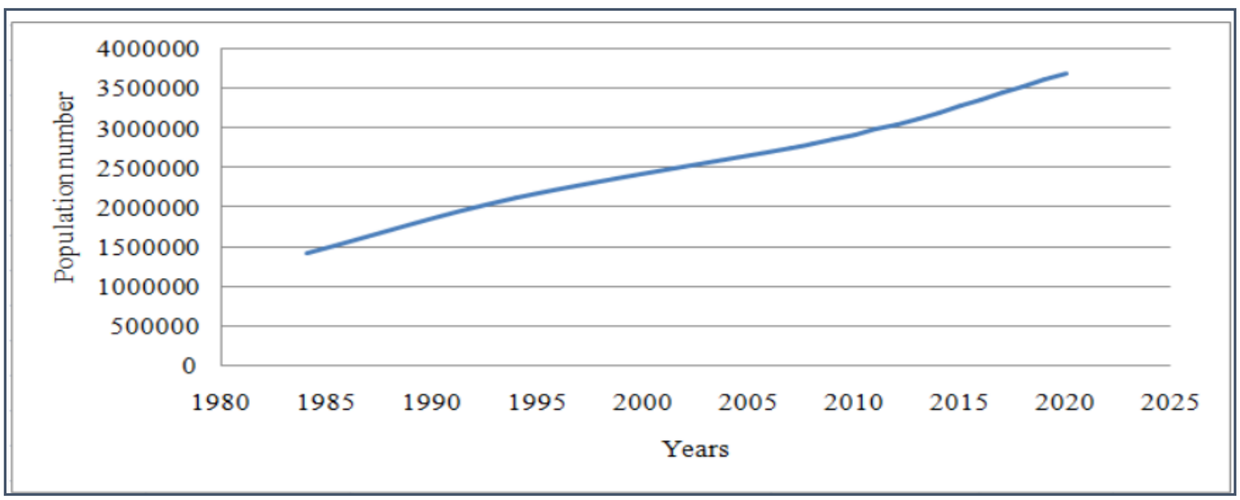

Figure 3-1. Trends of population growth in Addis Ababa (1984-2020). (Source: Central Statistical Agency, 2013).

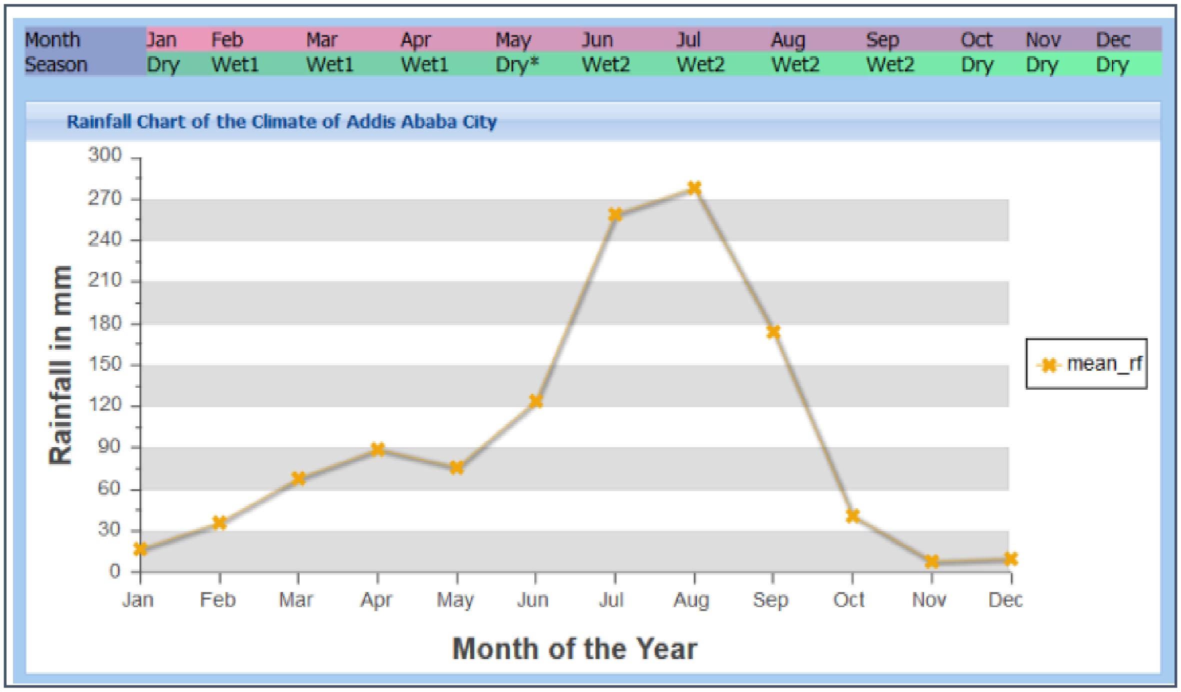

Figure 3-2. Monthly average rainfall in Addis Ababa (Source: NMA).

3.1.1 Location (Yeka Sub-City)

The study area Yeka sub-city is one of the ten sub-cities in Addis Ababa. It is found in north-eastern part of Addis Ababa is located at geographical coordinate of $9^0 01'30.73''$ N and $38^0 46' 27.55'' E$ in DMS (Degree, minutes and Second) with a total area of 86 km^2 (8600ha). It was composed of 14 Woredas and its population is 368,418 people. There is mountainous, highly stream and rural areas in the North West parts of the sub-city.

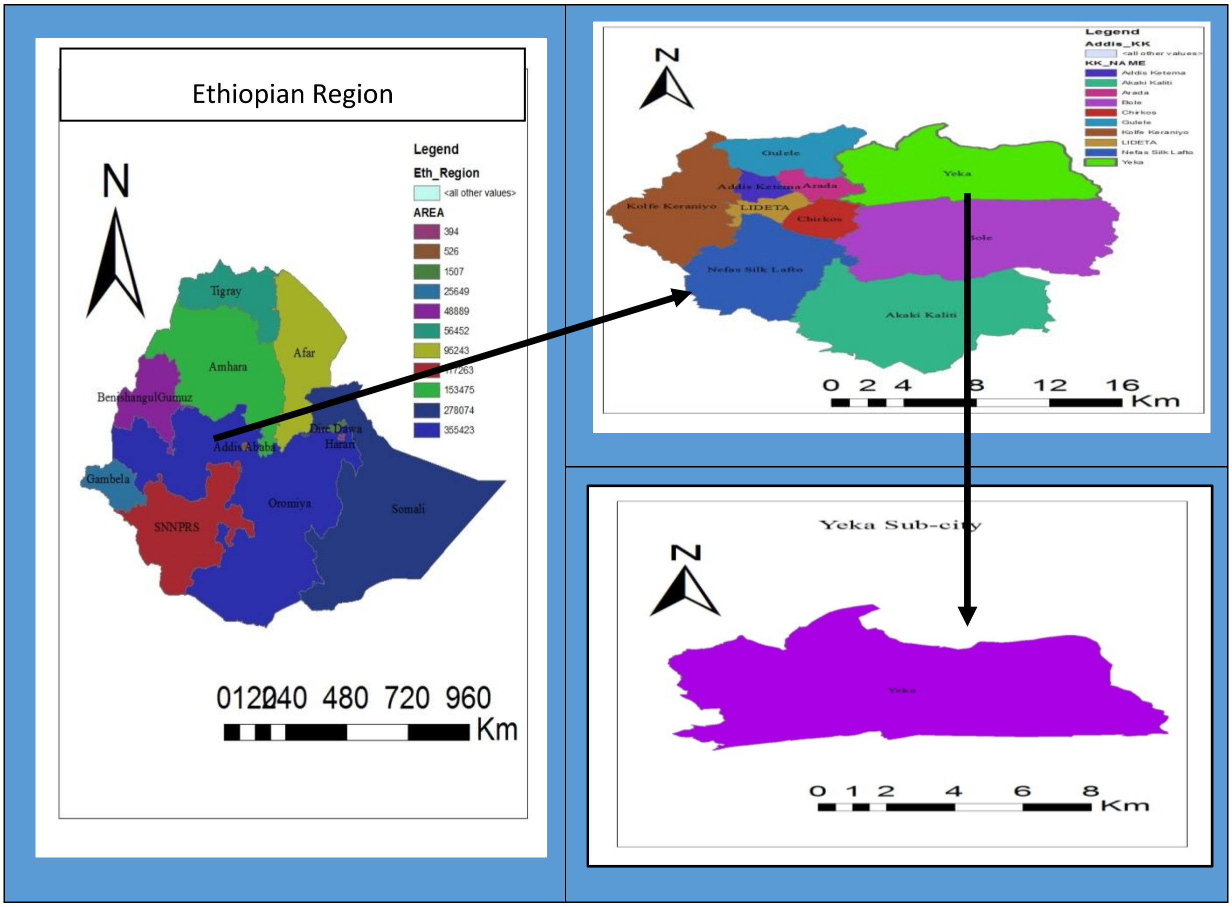

Figure 3-3. Location of the Study Area (Arc-GIS 2019).

From Arc-GIS, The highest elevation in Yeka sub city is 3139 meters, while the lowest point in the study area is 2302 meters (DEM). The steeper areas are the south edge and the western part, which consist mainly on small valleys bordering the urbanized area.

3.1.2 Land Use Land Cover Map of Yeka Sub-City

The land use and land cover of the Yeka sub-city is one of the mountainous sub-cities in the north-west of the sub city which is covered by different trees. On the top of the mountainous area there are rural areas where agricultural activities are performing. The topography of the land is hilly with steep slope drained by streams of both small and medium size.

In rainy season, the streams are highly flooded and surface runoff affects human activities and create obstacle to daily traffic in the downstream of the sub-city road. The direction of flow is into the downstream neighboring sub-city Bole in the eastern part of the sub city is perform highly urbanization activities construct high condominium houses, Parks, asphalts, business and commercial actives. Center of the sub-city is highly urbanized and also informal settlement house near the river and small streams.

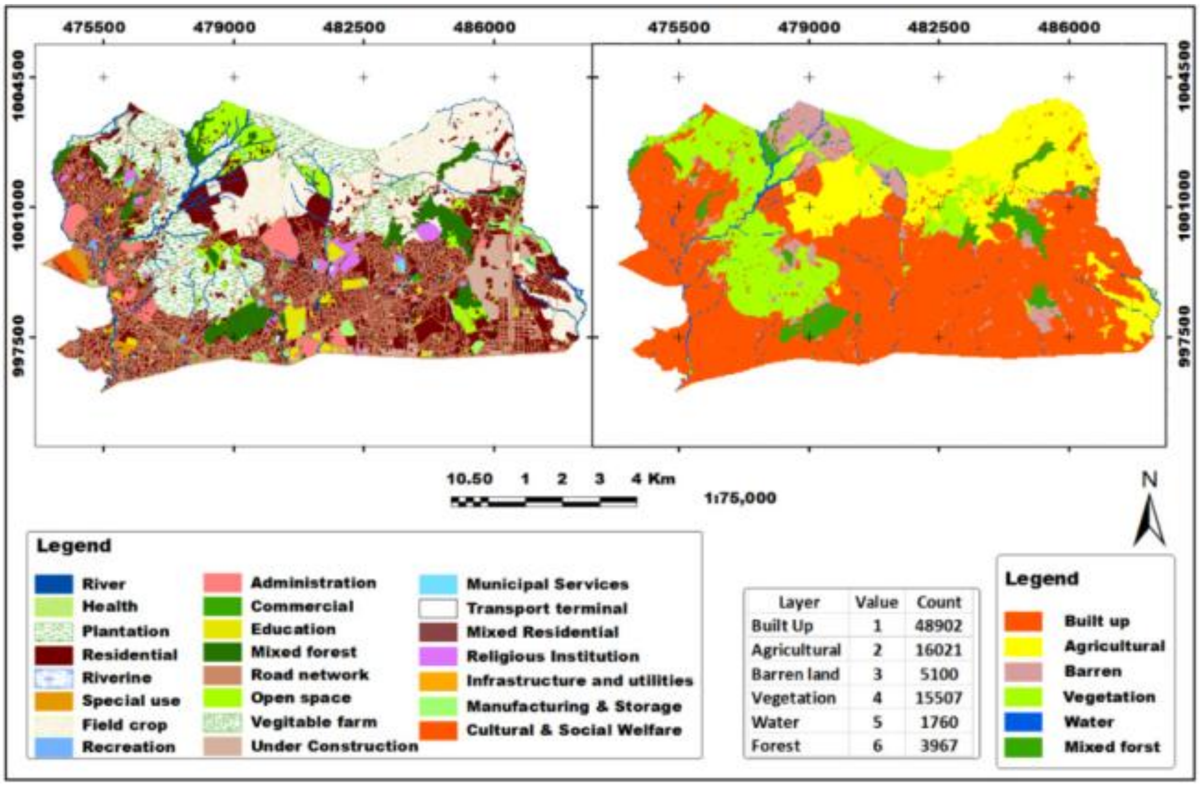

Figure3-4 Land Use Land Cover Map of Yeka Sub-City (Source: Geographic Information System, 2019).

Now a day the issues' of the health and quality and quantity of river in Addis Ababa city is under alarming. People are damping the solid waste into the streams, drainage system structures and into rivers. The local and governmental urban drainage management system is weak. The reason to the selection of the study area was based on the above personal observation. With intention of providing and recommendation to improving the storm water management system by applying the best management practice in suitable area through modeling are indicates how to reduce the runoff over the traffic roads and to introduce sustainable drainage system (SUDS) techniques for further study.

3.1.3 Flooding Area locations

The study area is located in the Yeka Sub-city and is part of the ring road which is shown in the Figure 3.5 using Google Earth Map and photo serving shows the location of flooded areas. The modeled area contains all types of urban area means, there are commercial, administration and business area and also these modeled areas contain not developed areas.

Figure 3-5. Modeled study area top view and present flooding condition. (Source; photo serving from the field study).

3.2 Data Collection

Data have been collected from both primary and secondary sources.

3.2.1 Primary Data Collection

The primary data gathering of more accurate and tangible that can be used to calibrate and validate. The simulated storm water management model is difficult to measure directly considering the whole of study area. However attempts were made to capture the situation by pictures and measure the section properties of the storm water drainage infrastructure. Data obtained from this administration is one of the plan views the land map of the whole sub -city in AUTOCAD form used for location, slope, and specific boundary of the study area. Field measurement of the geometric properties of hydraulic structure; size, shape, width and the depth of existing drainage system and identification of the type of channel material in the problem areas are carried out.

3.2.2 Secondary Data Collection

Secondary data was widely used in the research. Data were collected from.

- Sub-city and city administration
- Addis Ababa City Road Authority (AACRA),
- Ministry of Water Irrigation and Energy (MoWIE),
- The Geological Survey Institute,
- National Meteorological Agency; and
- Various reports and manuals.

3.2.3 **Materials**

Materials used in this research were:-

- ARC-GIS to obtain the hydrological and the physical parameters spatial information of the catchment of the study area.
- DEM data is used as an input data for ARC-GIS software for catchment delineation and develop of catchment elevation, flow direction and flow accumulation. This data obtain from MoWIE is used for catchment delineation with 30m by 30m.

- Google Earth software: - from internet to help the dividing the catchment to sub-catchment.
- Field Survey Materials:- digital camera for photo serving observing flood on the study area.
- GPS device, measuring Tap meter and others.

3.3 Methods of Data Analysis

The thesis focuses on the identifying of the real problem in the urban drainage on storm water management in Yeka sub-city to make sustainable and to put the engineering solution and to practice the best management practice devices.

The main objectives of this study works are:-

- Develop Thiessen polygon for Selection of nearby meteorological stations.
- Estimation of missing rain fall data using normal ratio method.
- Check the consistency of rainfall data from four nearby stations using double mass curve techniques.
- Design rainfall analysis (frequency analysis by using different statically probability distribution method formulas and select best fit using L-moment) probability distribution.
- Develop hourly and daily IDF curve for 2, 5,10,25,50 and 100 year return period for 25 year rainfall events using normal probability distribution methods.
- Setup, calibrate the parameters and validate with the observation and simulated value of the model SWMM5 using manual calibration method.
- Put best management practice (BMP) and run the model for quantifying the reduction of the peak flow rate and runoff volume.

3.3.1 Rainfall Data Analysis

There are different rainfall gauging stations in the city such as Addis Ababa Observatory station, Bole International Airport Station, Ayer Tena, Kolfe-Keraniyo School, and Yekatit 23 School, Abyssinia School, Medhaniyalem School, Akaki Kality, Asko, Kotebe TTC and Entoto. Some of these stations are inactive, not have more than ten years data and the other stations are actually old with a lot of missing data. Therefore the data are not complete enough to use the nearest station as Thiessen Polygon shows for developing frequency analysis. Based the Thiessen polygon developing, the study rainfall data analysis were to perform constancy checking from four stations(i.e. Bole, Ayer Tena, Addis Ababa observatory, Entoto And Bole international Airport Station using for model simulation.

A 25year (1994–2018GC) rainfall data were collected from Bole station, Addis Ababa Observatory, Ayertena and Entoto stations. The daily and hourly data gathering from these gauging stations have been used to develop IDF Curves.

3.4 Rainfall Data Analysis and IDF Curve Developing for Yeka Sub –City

3.4.1 Consistency Checking

The procedure is develop using the surrounding stations and Thiessen polygon was created using ARC- GIS from that for selection of the nearest metrological stations, Bole Airport Station is the nearest one to the study area. Independent rainfall data obtained from other 3 nearby stations (Addis Ababa observatory stations, Ayertena stations and Entoto station were carefully checked for accuracy and consistency. a Double Mass Curves were used to check the consistency of rain gauge record rainfall data and the correction factor were 1.023.as shown in table 3.1.

Table 3-1 AnnualAccumulated Rainfall for Bole station and Average Accumulation of three stations.

Year	Annual maximum RF (mm) of Bole station	cumulative annual precipitation at Bole station(mm)	Addis Ababa observatory station(mm)	Entoto station	Ayertena station	Average of 3 stations	Average Accumulative RF 3 station(mm)
2018	59	59	68.7	65	64.5	66.1	66.1
2017	49.1	108.1	50.8	28.6	41.6	40.3	106.4
2016	29	137.1	54.9	46	43.5	48.1	154.5
2015	60.3	197.4	47.8	38.7	51.2	45.9	200.4
2014	27.2	224.6	65.4	35.5	39.6	46.8	247.3
2013	42.6	267.2	47.2	35.4	42.6	41.7	289.0
2012	64.7	331.9	36.4	31	43.4	36.9	325.9
2011	36.9	368.8	55.8	72.2	38.1	55.4	381.3

2010	54.4	423.2	44.8	98.3	59.4	67.5	448.8
2009	51.2	474.4	54.7	69.2	38.2	54.0	502.8
2008	37.2	511.6	53.3	58.3	34.7	48.8	551.6
2007	71.2	582.8	64	42.8	32.4	46.4	598.0
2006	61.7	644.5	70.9	55.6	60.1	62.2	660.2
2005	44.5	689	58.6	40.4	37.5	45.5	705.7
2004	29.6	718.6	44.2	40	17.4	33.9	739.6
2003	34.6	753.2	54.9	61.4	43.5	53.3	792.8
2002	28.6	781.8	29.5	43.6	29.1	34.1	826.9
2001	32.4	814.2	96.3	38.5	30.2	55.0	881.9
2000	47	861.2	37.1	38.2	50.2	41.8	923.7
1999	37.8	899	37.4	56.3	30.4	41.4	965.1
1998	60.1	959.1	78.3	56.4	97	77.2	1042.3
1997	37.3	996.4	46.3	90.3	50.9	62.5	1104.8
1996	52	1048.4	67	45.7	50.2	54.3	1159.1
1995	64.7	1113.1	85.3	43.3	49.9	59.5	1218.6
1994	38.2	1151.3	57	49.7	55.4	54.0	1272.7
	Total	604.636					646.6

Corrected value = Original value x (Mc/Ma) = PX*(MC/Ma).

Where, Px the rainfall data at bole station (Mc/ma), the slope ratio of the value at change in regime indicated (511.6 -781.8).

3.4.2 Probability Distribution of Rainfall Data of Yeka Sub City

Probability distribution function is used for prediction of maximum daily rainfall for higher return periods which fit the observed rainfall data approximately. Probability analysis of one day annual maximum rainfall series was carried out by employing six probability distributions namely Normal, Log Normal, and Pearson type - III, Log – Pearson type - III, Weibull and Gumbel distributions.

Table 3-2 Different probability distribution model For Calculation of extreme value XT (mm) from different Return Period (T).

Return period(T)	Exceedance probability(p)	parameter values of the probability distribution					
		Weibulls formula(Observation)	Normal	Lognormal	Pearson type - III	Log-Pearson	Gumbel (EVI)
2	0.5	47.6	49.28	47.32	48.69	47.33	46.95
5	0.2	64.52	61.19	47.42	60.99	47.42	59.47
10	0.1	69.52	67.43	47.48	67.76	47.46	67.75
25	0.04	76.18	74.12	47.54	75.24	47.49	78.22
50	0.02	76.18	78.37	47.58	80.22	47.51	85.99
100	0.01	76.18	82.23	47.61	84.79	47.53	93.70

3.4.3 Testing the Goodness of Best Fit Probability Distribution

The best fit distributions decided by Chi-Square test for goodness of fit to observed values. The Chi-Square test statistic is given by the equation as:

$$X^2 = \sum_i^k x \frac{(Oi - Ei)^2}{Ei}$$

Where, Oi is the observed rainfall and Ei is the expected rainfall.

The best probability distribution function was determined by comparing Chi square values obtained from each distribution which have been select the function that gives the smallest value. the smallest value was the Best Fit Distribution.

3.4.4 Intensity – Duration – Frequency Curves Relationships

Intensity-Duration-Frequency (IDF) curves describe the relationship between rainfall intensity, rainfall duration, and return period (its inverse, probability of exceedence).

IDF curves are commonly used in the design of hydrologic, hydraulic, and water resource systems. The intensity-duration-frequency (IDF) relationship is one of the prerequisite statistics in water recourses engineering planning, development, and management to assess the vulnerability of hydraulic structures.IDF relationships have been established since 1932 in developed countries. but, these relationships do not exist accurately for many developing countries up to now.

The accessibility of accurate and long-term rainfall data in the former places and lack of that information in the latter countries is the major factor for the IDF curves construction (Hamaamin, 2018). IDF curves describe the amount of rainfall in a watershed area for a given period of time. The magnitude of an extreme rainfall event has an inverse relation to its occurrence frequency. Therefore, the severe rainfall events have less frequency compared to moderate rainfall events. The frequency analysis of rainfall data is to relate the magnitude of extreme events to their frequency of occurrence using the probability distribution. The IDF relationship is a mathematical relationship between the rainfall intensity, duration, and the return period using extreme rainfall data (Melesse, 2016).

IDF curves are obtained through frequency analysis of rainfall observations. The IDF curve is developed from 24-hour rainfall data of 25 years i.e. 1994 to 2018, obtained from Ethiopian Meteorological Agency rainfall gauge located around Bole, Addis Ababa. Data From rainfall measurements, for every year of record, determine the annual maximum rainfall intensity for specific durations (or the annual maximum rainfall depth over the specific durations). Common durations for design applications are: 5-min, 10-min, 15-min, 30-min, 1-hr, 2-hr, 6-hr, 12-hr, and 24-hr of shorter duration that used to SWMM5 simulation.

3.5 Storm Water Management Model (SWMM5) Setup

The SWMM software was developed in1969-1971 and is widely applied to analyze the water quantity and quality in storm water runoff, combined sewers, sanitary sewers, and other drainage systems in urban areas as well as in non-urban areas (Rossman, 2010). SWMM5 is a dynamic rainfall-runoff simulation model used for single-event or long-term continuous simulation of runoff quantity and quality from primarily urban areas.

SWMM5 is the current version used in this study to analyze the water quantity. The SWMM5 runoff component operates on a collection of sub - catchment areas that receive precipitation and generate runoff and pollutant loads. The principal input parameters for sub catchment are infiltration method, assigned rain gage, outlet node, assigned land uses, surface area, imperviousness, slope, characteristic width of overland flow path, Manning's n for overland flow path on pervious and impervious areas, depression storage in both pervious and impervious areas, and the percent of impervious areas with no depression storage (Ross man, 2010).

Table 3-3. The Sub - catchment Input parameters into SWMM5.

Characteristics	Description
Rain Gages	Refers to the rain gage where the rain intensity is defined over a time interval
Outlet	Defines which node or sub catchment is receiving the flow
Area	Area of the sub catchment including any LID controls
Width	Characteristic width of the overland flow path for sheet flow runoff from non-LID area only.
% Slope	Average percent slope of the sub catchment
% Impervious	Percent of the land area which is impervious
N - Impervious	Manning's n for overland flow over the impervious portion of
	the sub catchment
N - Pervious	Manning's n for overland flow over the previous portion of the sub catchment
D-store impervious	Depth of the depression storage on the impervious portion of the sub catchment
D store - pervious	Depth of the depression storage on the pervious portion of the sub catchment
% zero - impervious	Percent of the impervious area with no depression storage
Sub area routed	Choice of internal routing of flow between pervious and impervious sub-areas (allows directing the flow between the pervious and impervious areas within a sub catchment)
% routed	Percent of the diverted flow toward a sub-area within sub catchment.

LID Controls	This is used to edit the use of Low Impact Development controls in the sub catchment.

These sub catchment objects represent a land area that receive precipitation and produce runoff to an outlet node. SWMM5 uses three methods these are Curve Number method, Horton's method and the Green-Ampt method for infiltration computation (Agency, 2010). In this study Green-Ampt method assumes that a sharp wetting in the soil column, separating soil with some initial moisture content below from saturated soil above. Initial moisture deficit, soil's hydraulic conductivity, and suction head are the input parameters for Green-Ampt method.

This nonlinear reservoir method solves a continuity equation coupled with Manning's equation based on rainfall excess (Rossman and Huber, 2016). Using a nonlinear reservoir concept SWMM5 generate the overland flow. Surface runoff conceptual view as shown in Figure 3.11runoff flow rate, q per surface area of the sub catchment is given as

$$q = (1.49Ws^{\frac{1}{2}}(d - ds)^{\frac{5}{3}})/An$$

Assuming that flow across the sub catchment's surface behaves as if it were uniform flow within a rectangular channel of width W (m), height d–ds, and slope S, the Manning equation can beused to express the runoff's volumetric flow rate Q (CMs) as:

$$Q = \frac{1}{n}S^{1/2}Rx^{2/3}Ax$$

Where:-

n -is a surface roughness coefficient,

As the apparent or average slope of the sub catchment (m/m),

Ax- the area across the sub catchment's width through which the runoff flows (m^2) and

Rx is the hydraulic radius as associated with this area (m). W is Axis of rectangular area with width W and height d-ds. Because W will always be much larger than d it follows that.

$$\mathrm{Ax} = (\mathrm{d} - \mathrm{ds})$$

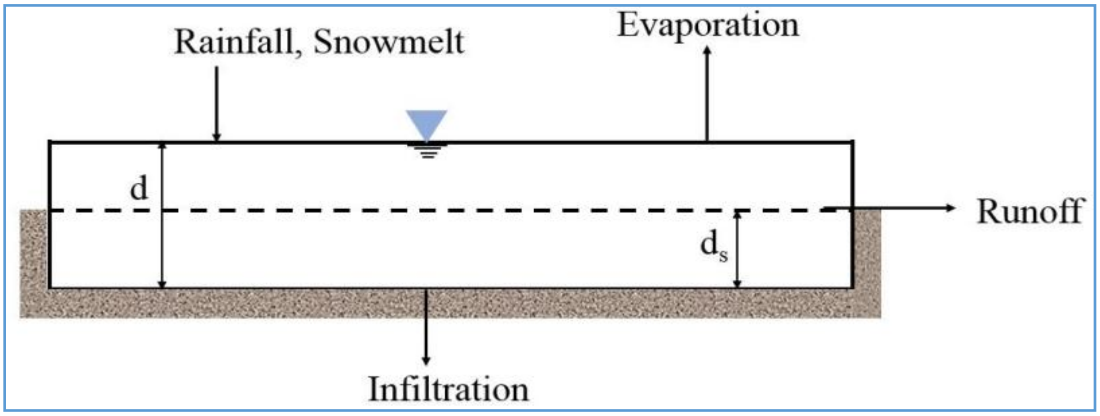

Figure 3-6. Nonlinear Reservoir Model of a Sub catchment (Source: United States Environmental Protection agency, Storm Water Management Model Reference Manual, 2016).

The three options for routing component in SWMM5aredynamic wave routing, kinematic wave routing and Steady-flow routing. The inflow to the conduit is translated to the downstream end by one of the options. Steady-flow routing assumes that within each computational time step flow is uniform and steady. Kinematic wave routing allows flow and area to vary spatially and temporally within a conduit. The outflow hydrograph is delayed as inflow through the conduit varies. Dynamic wave routing produces accurate results as it uses complete one-dimensional Saint Venant flow equations. Therefore, this study used the dynamic wave routing method to consider all possible hydraulic conditions e.g., backwater and Pressurized surcharge in conduits and junctions (manholes) accurately.

3.5.1 Depression Storage

Depression (retention) storage depth d_S are a volume that must be filled prior to the occurrence of runoff on both pervious and impervious areas. It represents a loss or “initial abstraction” caused by such phenomena as surface ponding, surface wetting, interception and evaporation. However, the depression storage depends on the permeability of the surfaces. includes different values for depression storage that are considered on SWMM given in mm (U.S, Agency, 2010).

Table 3-4. Depression storage values (Ross man, 2010).

Depressions Storage	Capacity (mm)
Impervious surface	1.2 – 2.5
lawns	0.10 -0.20
pasture	5.08
Forest litter	7.62

3.5.2 Imperviousness

The percent imperviousness of a sub catchment is another parameter that can in principle, be measured accurately from aerial photos or land use maps. In practice, unless impervious layers are included in a GIS representation of the basin, such work tends to be tedious and it is common to make careful measurements for only a few representative areas and extrapolate to the rest. Runoff volume and flow rates are strongly sensitive to estimates of imperviousness. Hence, care should be taken in imperviousness estimates.

3.6 Model Calibration and Validation Procedure

Model calibration and validation have been conducted using two intense rainfall events during the wet months of July and August 2019. The goal should be to provide good values for a set of measures, even if they are lower than single best realizations in order to include the whole dynamics of the model results(Beck et al., 2017).Conclude that a complete assessment of model performance should include one “goodness-of-fit” measure (e.g. NSE, d) and at least one absolute error measure (e.g. RMSE, MAE).The sewer model has been calibrated by using the measured sewer flow at the outlet1 flow meter location for the July rainfall event for 10 days measurement. Then, the calibrated model has been validated for another 10 day intense rainfall event that occurred in August. The time series plots of the calibration and validation rainfall events are shown in Figure below. In this section, the efficiency criteria used in this study are presented and evaluated.

3.6.1 Nash-Sutcliffe Model Efficiency (NSE)

In hydrological modeling, the coefficient of determination is commonly known as the Nash-Sutcliffe efficiency coefficient (NSE or R^2) (Nash and Sutcliffe, 1970). The NSE is a normalized statistic that determines the relative magnitude of the residual variance compared to the measured data variance (Nash and Sutcliffe, 1970). It indicates how well the plot of observed versus simulated data fits the 1:1 line and is defined as:

NSE = 1- $\left[\sum_{i=1}^{n}(Oi-Pi)^2 / \sum_{i=1}^{n}(Oi-\overline{O)2}\right]$

Where O are the observed and P the predicted values, and $\bar{O}$ is the mean of the observed values. The NSE ranges from minus infinity to 1.0, with higher values indicating better agreement and a value of 1.0 being the optimal value. Values higher than 0.0 are generally viewed as acceptable levels of performance, whereas values below 0.0 indicate that the mean observed value is a better predictor than the simulated value (i.e. the model performance is unacceptable).

3.6.2 Parameter Properties Estimation of the Study Area

Extracted from the GIS raster layers, many parameters related to the properties of the sub catchments. The following values of the parameters were extracted in this study SWMM implies sub-dividing the catchment into 31 sub catchments where each sub catchment was considered as an independent hydrologic unit and own features. Still, these features were defined according to the respective land variations, topography and the drains location. Further, table 3.8includes the different zones in the catchment. Where the catchment was divided into three different zones, where each zone included a number of sub-catchments that connected to an outlet.

Table 3-5. The water shed classification of into three zones within the sub catchment area.

Sub catchment Zone	Outlet Number	Elevation(m)	Sub catchments	Sub catchment Area (ha)
CMC	1	2400	From sub1- 10	158
Kotebe Metropolitan University	2	2440	From sub1 – 9	128
02-Gebeya to Gurd-shola	3	2420	From sub1 - 11	160

Runoff Flow Direction (S) illustrates the runoff flow different direction was used as a starting point to estimate the ideal locations for the outlet. The current location for each of the outlets was based on the elevation profile for the catchment was obtained from Addis

Design Storm: the design storm compromised a rainfall event that lasted for 180 minutes associated with 25-years return period and applied evenly to the catchment. Each sub catchment is linked to the design storm, where the storm is represented by a time series object (Rain Gage). in the model, that is named as time series 25-years rainfall (TS25) with a 15minutes time-interval and another one named 25-years Rainfall +LID /BMP plotted in The runoff rate and volume depends on design storm magnitude and distribution over the catchment. Thereafter the runoff will be routed through drainage systems.

Table 3-6. Different input parameters were in SWMM5 model.

Parameter	Value
Catchment Area (ha)	442
Number of sub-catchments	31
Manning's roughness Coefficient. For Impervious Area (N- impervious)	0.011 – 0.015
Manning's roughness Coefficient. For pervious Area (N- previous)	0.05 – 0.8
Depth of depression storage on Impervious Area (D- Impervious) (mm)	1.2 – 2.5
Depth of depression storage on pervious Area (D-Pervious)	2.5 - 8
% Imperviousness	
Residential	50
Roads	100
Green areas - Park &forest area	2
Infiltration	Green Amps Methods

3.7 Best Management Practices Low Impact Development (LID) Control Scenarios in SWMM5

The LID control in SWMM5 are different practices from these bio-retention, permeable pavement, rain garden, rain barrel, infiltration trench, rooftop disconnection, vegetative swale, and green roofs.

Each LID in SWMM5 has a variety of process layers such as: surface, soil, storage, and drain. Each sub catchment can have multiple LID controls (Seema Bardhipur, 2017b). In Bio retention cell in residential communities is used to treat runoff from roads whereas rain barrel is used to treat rooftop runoff. This study explains the two modeling techniques of Bio-Retention cell and Rain Barrel can be practice and reduce surface runoff.

3.7.1 Bio Retention Cell

It restores the natural system function by using design techniques that infiltrate, filter, store, evaporate and detain runoff close to its source (Trowsdale and Simcock, 2011).Bio-retention cell is the most widely applied LID practice throughout the U.S.

Bio-retention cell consists of a grass buffer strip, a sand bed, a pond area, an organic layer of mulch, planting soil and plants. Runoff water passes across the length of the pond area which consists of organic mulch. Later, water infiltrates into planting soil and sand beds (USEPA, 2000). Some of the bio-retention facilities have under drains which convey the excess water to the storm drain system.

Internal Water Storage (IWS) layer is also a design type of bio-retention cell included in the subsurface portion of the media which provides water storage volume in the bio retention. IWS also accounts for pollutant reduction. This study didn’t design for IWS as (Jarden et al., 2015) experimented with traditional bio-retention cell in the study site.

But usually, bio-retention cell with IWS layer showed greater reduction in volume than other type of bio-retention cells (Winston, 2016).The general components of bio-retention cell are surface layer, soil layer, storage layer, under drain, and overflow structure. A bio-retention cell can be designed with and without under drains.

3.7.1.1 Characteristics of Bio- Retention Cell

Bio retention cells have become an accepted technology for storm water management due to their potential benefits in runoff volume reduction and water quality improvement. There are four types of components in SWMM for bio-retention cell or rain garden modeling: surface layer, engineered soil layer (filter media), gravel storage layer and an optional under drain system. Figure 3.16 shows a typical bio-retention cell (rain garden) layout in SWMM.A simple, yet effective method to control storm water is through the use of rain gardens, also known as bio retention cells. Rain gardens are small vegetated depressions that collect, store, and in some cases, infiltrate storm water runoff. They contain a special soil mix, or media typically consisting of 50 percent sand, 30 percent organic material, and 20 percent topsoil (by volume)TDEC (Tennessee Department of, Conservation and Resources, 2014). It uses very little of the onsite soils and tends to be sandy.

The size and depth of a rain garden, or bio retention cell, varies depending on the drainage area and location of the storm sewer Drainage Area contributing. Urban bio retention is limited to 250m^2 and the total areas applied limited to less than 4046m^2 to 8092m^2 preferable less than 4046m^2 of drainage area. However, this is considered a general rule; larger drainage areas may be allowed with sufficient flow controls and other mechanisms to ensure proper function, safety, and community acceptance.

The drainage areas in these urban settings are typically considered to be 100% impervious. While multiple planters or swales can be installed to maximize the treatment area in ultra-urban watersheds, urban bio retention is not intended to be used as treatment for large impervious areas such as parking lots.

Drainage System Adequacy Description of Bio Retention**:** Urban bio retention practice elevations must allow the untreated storm water runoff to be discharged at the surface of the filter bed and ultimately connect to the local storm drain system.

Available Hydraulic Head of Bio Retention: In general, above 900mm of elevation difference is needed between the downstream storm drain invert and the inflow point of the urban bio retention practice. This is generally not a constraint, due to the standard depth of most storm drains systems.

Setbacks from Building and Roads: If an impermeable liner and an under drain are used, no setback is needed from the building. Otherwise, the urban bio retention practice should be 4m down gradient from the building.

Minimizing External Impacts of Bio Retention Structure (Safety issue):

Because urban bio retention practices are installed in highly urban settings, individual units may be subject to higher public visibility such as; greater trash loads, pedestrian use traffic, vandalism and even vehicular loads. These practices should be designed in ways that prevent, or at least minimize, such impacts. In addition, designers should clearly recognize the need to perform frequent landscaping maintenance to remove trash, check for clogging, and maintain vigorous vegetation. The urban landscape context may feature naturalized landscaping or a more formal design. When urban bio retention is used inside walk areas of high foot traffic, designers should not impede pedestrian movement or create a safety hazard and maintain the American with Disabilities Act (ADA) required path of travel. Designers may also install low fences such as allow garden fence, grates or other measures to prevent damage from pedestrian short-cutting across the practices.

3.7.1.2 Proposed Bio- Retention System in Study Area

The system of forty three cells was proposed in the study area. All 43 bio retention cells were assumed to be of equal size which is approximately 25 m long by 10.5m wide. Therefore, the total area for all cells is approximately 11287m^2.The media begins approximately 86.4 cm below the existing grade with the top of the media at approximately 55.9cm below the lip of the bio retention cell.

Total Design Volume Consideration: under design consideration the combination of surface ponding volume of soil media and gravel storage volume is adequate to manage the Design Volume. The storage volume of the soil media and gravel is within the void spaces, referred to as porosity (η). The accepted porosity values for the storage components are illustrated in Figure BR 3-15 and listed below:

Table 3-7. Accepted porosity (n) values for the storage components are illustrated.

Surface ponding	1
Soil media	0.25
Under daring gravel	0.4

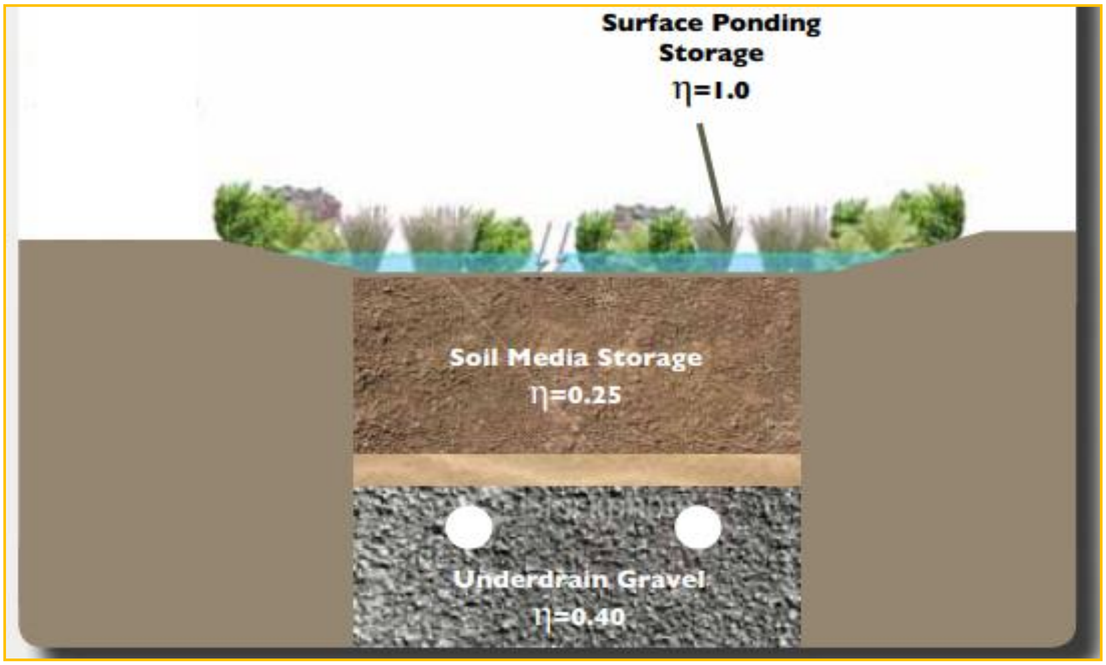

Figure 3-7.Typical Bio retention Section with Porosity (n) Values for Volume Computations (Source: West Virginia storm Water Management and design guidance Manual).

Surface Water Storage Depth Requirements:

- 6 inches maximum in high-use areas (along streets, at schools, in public landscapes, etc12 inches in less used areas (away from frequent public access).
- Bio retention Soil Depth: Between 12 and 36 inches
- Gravel storage Depth: Between 6 and 8 inches

Void ratios (Vr) are generally:

- 0.20for bio retention soils
- 0.40 for clean –washed aggregates has AASHTONo.3
- 0.85 to 0.95 for manufactured storage units depending on manufacturer

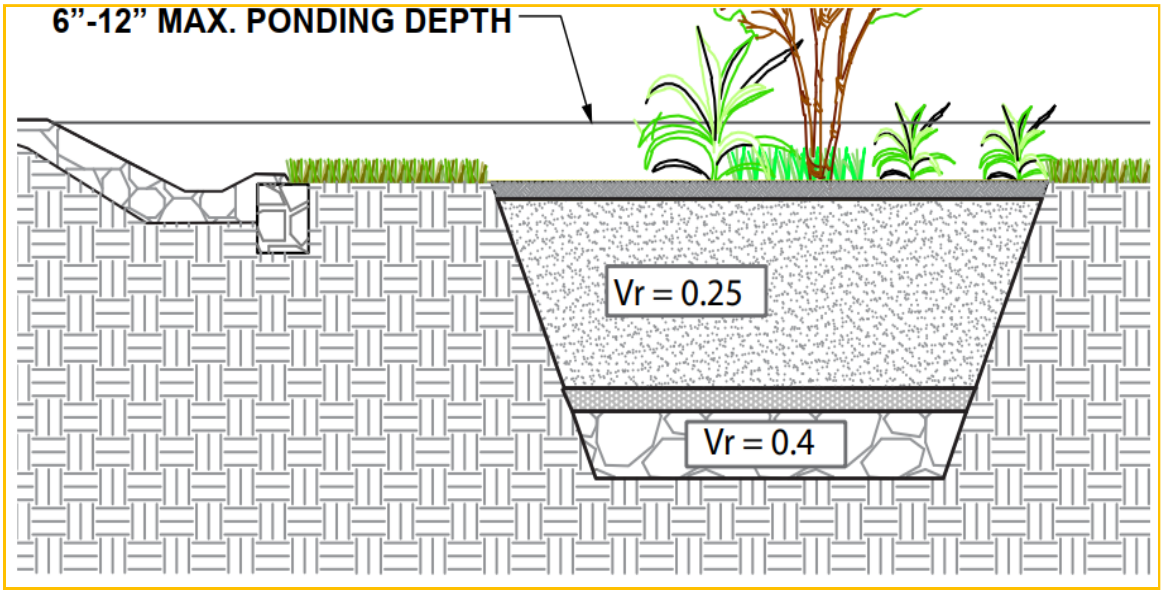

Figure 3-8. Typical bio retention section with void ratios for volume computations.

Bio-Retention in SWMM5

Shown in Figure 3-9A bio-retention cell is represented in SWMM5 by four vertical layers and each layer is parameterized using a LID control editor

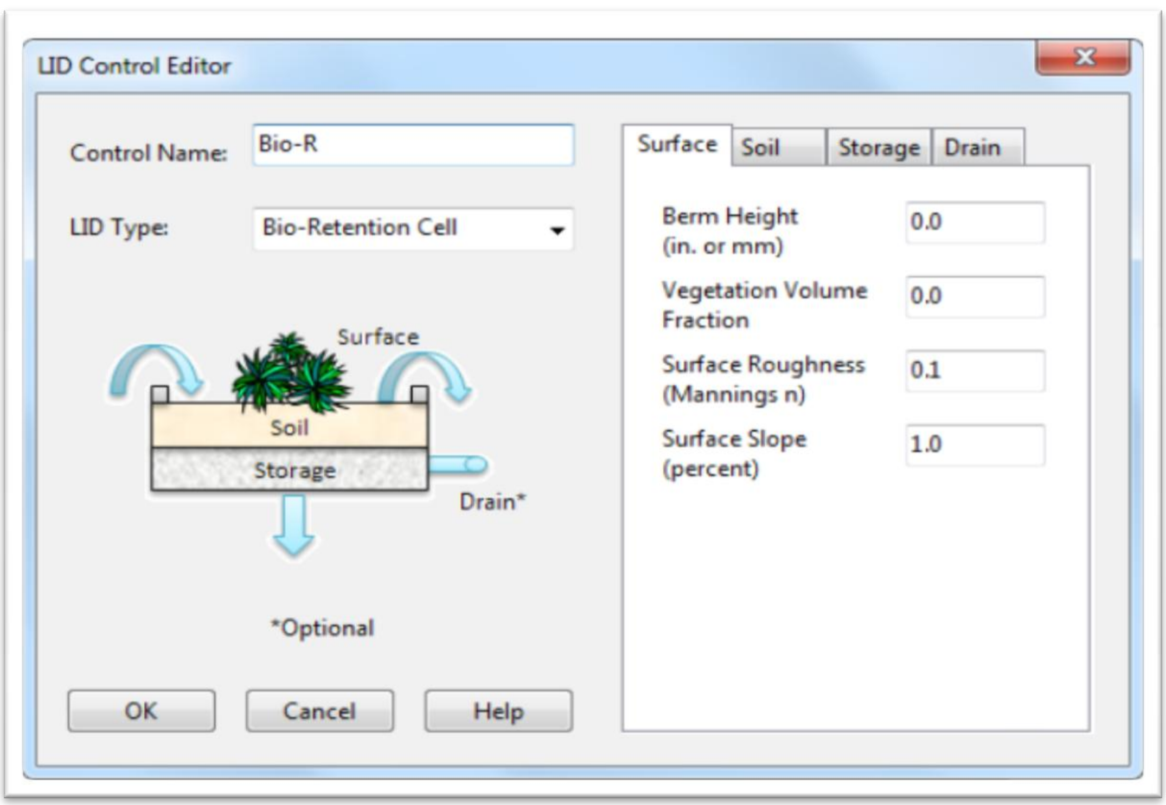

Figure 3-9. LID Control Editor in SWMM5 (Example of Bio-retention Cell).

The surface layer represents the top vegetative growth in bio-retention cell and it receives rainfall and runoff from surrounding soil. The water from the surface layer is infiltrated into the soil layer or is lost through evaporation. The soil layer contains a soil mix to support the top vegetative growth. This layer receives water through infiltration from surface layer and loses water through percolation to the storage layer below it. The storage layer consists of a stone aggregate. This layer receives water from the soil above it, and loses water through infiltration to natural soil or by an under drain pipe (Rossman and Huber, 2016). The following hydrologic assumptions were made for bio-retention cell simulation in SWMM5.

The most common design of Bio-retention cell was compared with its conceptualization in SWMM5.Itis noted that the over flow pipe connected to the under drain cannot be simulated using the SWMM5 LID controls because it was not considered in SWMM5 LIDs. However, if bio-retention cell is modeled hydraulically, overflow pipe can be modeled using other SWMM5 modules like weir or orifice. Table 3-10 explains the parameters represented in SWMM5, compared with the on-site bio-retention cell as shown in Figure 3-10.

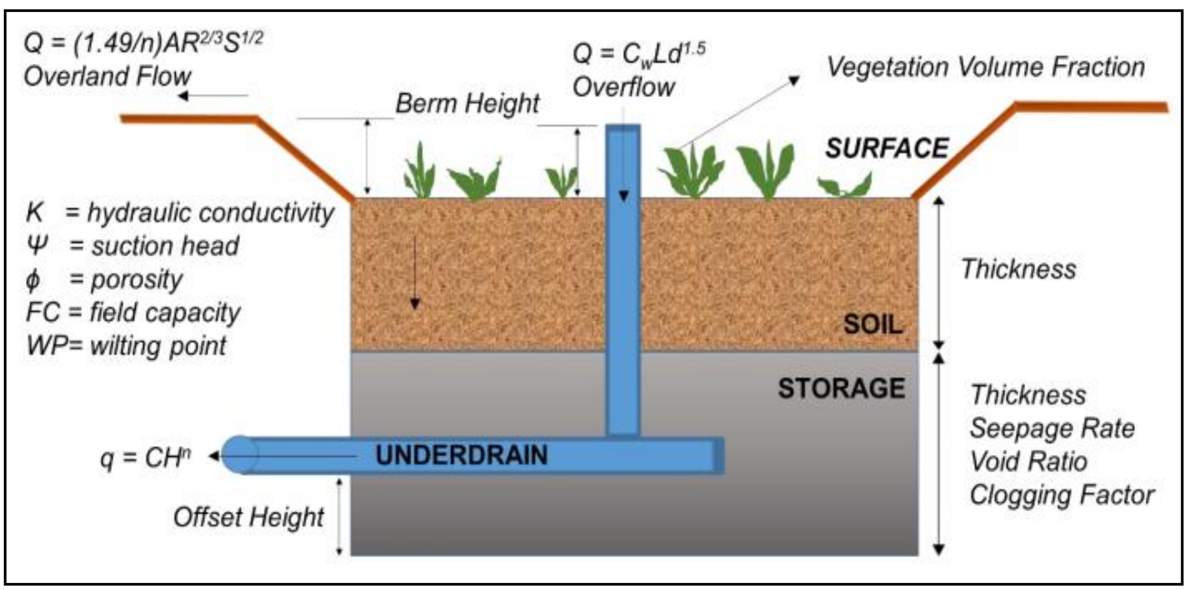

Figure 3-10. Parameterization of Bio-retention Cell for SWMM5.

Table 3-8 . Properties of Bio-retention Cell Parameter were used in SWMM5.

Layers		Value	
Surface	Berm height(mm)	200	Typical value 100 to 300mm (Rossman,2010)
	Vegetative volume friction(-)	0.05	
	Surface roughness(-)	0.23	
	Surface slope (%)	5	Typical value from 2 to 20%
Soil layer	Thickness (mm)	600	Typical value range 450 to 900mm
	Porosity(n)	0.25	Typical value for a sandy Soil(Ross man, 2010)
	Field capacity	0.062	
	Wilting point (volume fraction)	0.024	
	Conductivity (mm/h)	130	
	Conductivity slope	5	
	Suction head (mm)	1.93	
Storage layer	Height(mm)	300	Typical gravel layer 150 to 450 mm(Ross man, 2010)
	Void ratio	0.7	Typical value 0.5 to 0.75 for gravel bed (Ross man, 2010)
	Conductivity(mm/hr)	3.5	Saturated hydraulic conductivity of the sub catchment used green-Amp infiltration method
	Clogging factor	0	
Under drain	Drain coefficient	0	
	Drain exponent	0	Under drain is not considering
	Drain offset height(mm)	0	
Impervious area treated (%)		51	

3.7.2 Rain Barrel Description

These are low-cost water conservation devices used to divert runoff from storm sewer systems to back yards it collects and stores runoff from rooftops. Two types of rain barrels are commonly used in residential areas (The City of Calgary Water Resources, 2011).

The first type is the overflowing rain barrel in which inflow flows through the downspout connected to rooftop, while outflow flows as the overflow. The overflow is then routed to pervious area such as lawn yards. The second type is the continuously draining rain barrel. The inflow is from the downspout connected to rooftop where as the outflow is from the outlet drain pipe only as shown on Figure 3-11.The water from outlet pipe is routed to a pervious area.

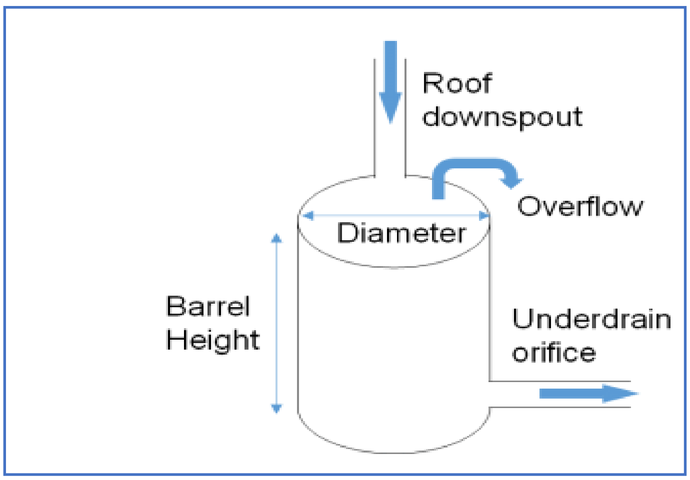

Figure 3-11. Parameterization of Rain Barrel for SWMM5.

3.7.2.1 Characteristics of Rain Barrels

Table 3-14 presents the Rainwater Tank parameters used in SWMM for the SUDs modeling. The various Rainwater Tank parameters that were analyzed included tank volume (in liters), drain time (T in hours) and the drain delay (in hours). Four different tank sizes (500, 1000, 1200 and 1500 L), four drain times (12, 24 and 36 and 48 hours) and four drain delay times (0, 12, 24 and 36 hours) were analyzed.

In SWMM, flow through the under drain from a rainwater tank is governed by the submerged orifice equation as shown in(Nasrin,2018). C represents the drain coefficient, D is the height of stored water, Hd is the drain offset and N is the drain exponent.

$$q = C(\ D - Hd)^n$$

The drain coefficient (C) can be estimated by integrating as can be seen, C is a function of two variables, namely the drain time (T) and thedepth (D) of the stored water. Drain time (T) is the time required to drain out a depth D of stored water in the rainwater tank.

$$\mathrm{C} = 2(\mathrm{D}^{0.5})/\mathrm{T}$$

With the values of D and T in mm and hours respectively the standard range of drain time for storage-based SUD strategies is 24 to 48 hours and 900 to 1200mm (Nasrin, 2018). The drain exponent has been taken as 0.5, assuming the under drain acts like an orifice (Walsh et al., 2014;(U.S Agency, 2010). Drain offset has been taken as zero, assuming that the orifice is at the bottom of the rainwater tank.

Rain Barrel Setbacks from Buildings: -Cistern overflow devices should be designed to avoid causing ponding or soil saturation within 3 meter of building foundations. Storage tanks should be designed to be watertight to prevent water damage when placed near building foundations. Tank Size: -In general, the larger the tank you select, the greater your water savings you'll be able to store more rainwater as a backup supply for dry periods when there is no rainfall. Larger tanks are typically more expensive, so it is important to select an ideal tank size one that is sized to maximize water savings while minimizing the cost of the system. Determining the ideal tank size depends upon wide variety of factors including local rainfall patterns, roof collection area, and future daily rainwater demands within the home. For most cases, the size of tank can be selected based upon what the rainwater will be used for as well as the number of bedrooms within the home, assuming a typical roof area and typical water demands. Given these assumptions, the homeowner can determine the ideal size of tank by selecting one of the following: a Small tank (tanks less than 2,500 L of storage capacity), Medium-sized tank (between 2,500 L and 5,000 L) and a large tank (tanks with more than 5,000L storage capacity).Multiple tanks or barrels can be placed together and connected with pipes to balance water levels and increase overall storage, as needed.

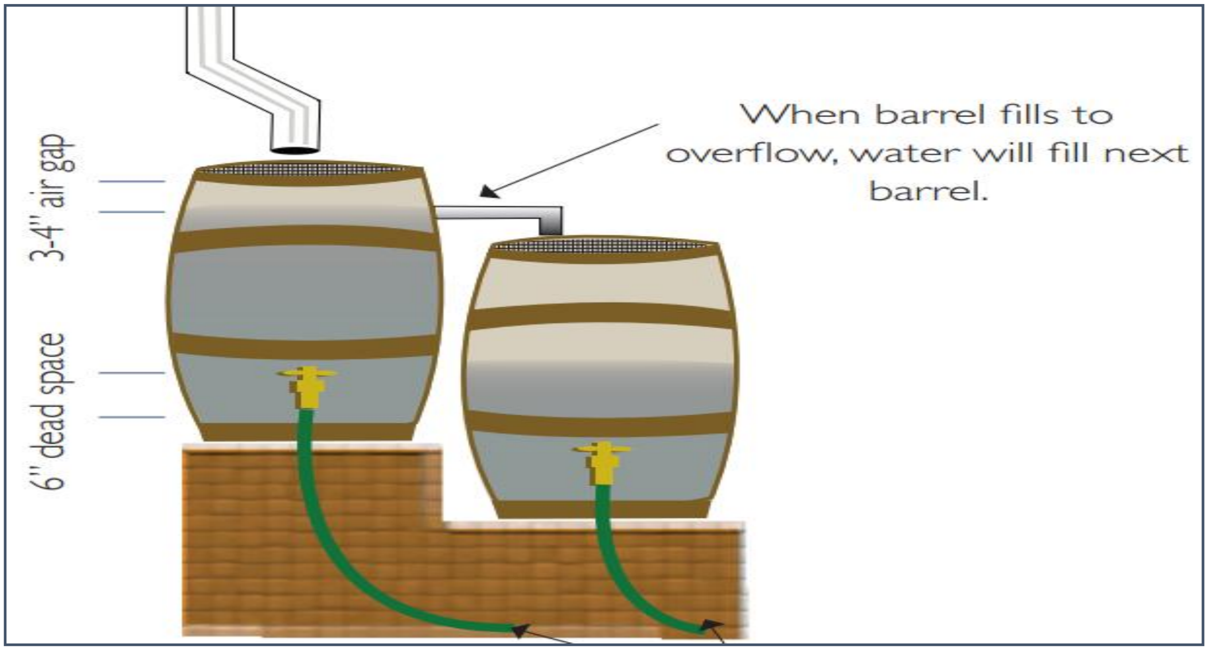

Figure 3-12. View of Alternatives for two barrel configuration rainwater harvesting systems. (Source: Tennessee permanent storm water management and design guidance manual, 2014).

This allows more storage while still using small vessels. If more than two barrels are required, a larger tank or cistern is recommended. For water quality improvement, a 25mm rainfall event is a typical recommended amount to capture in a Rainwater Harvesting tank this is the standard for most of Maryland, including Annapolis and the Eastern Shore Use the following formula to determine how many gallons a roof area will produce in a 25mm rain event.

Rain Barrel in SWMM5

A rain barrel has the drain valve placed above an impermeable bottom. It represented in SWMM5 with barrel height as shown on Figure 3-13. it modeled just as a storage layer with all void space.

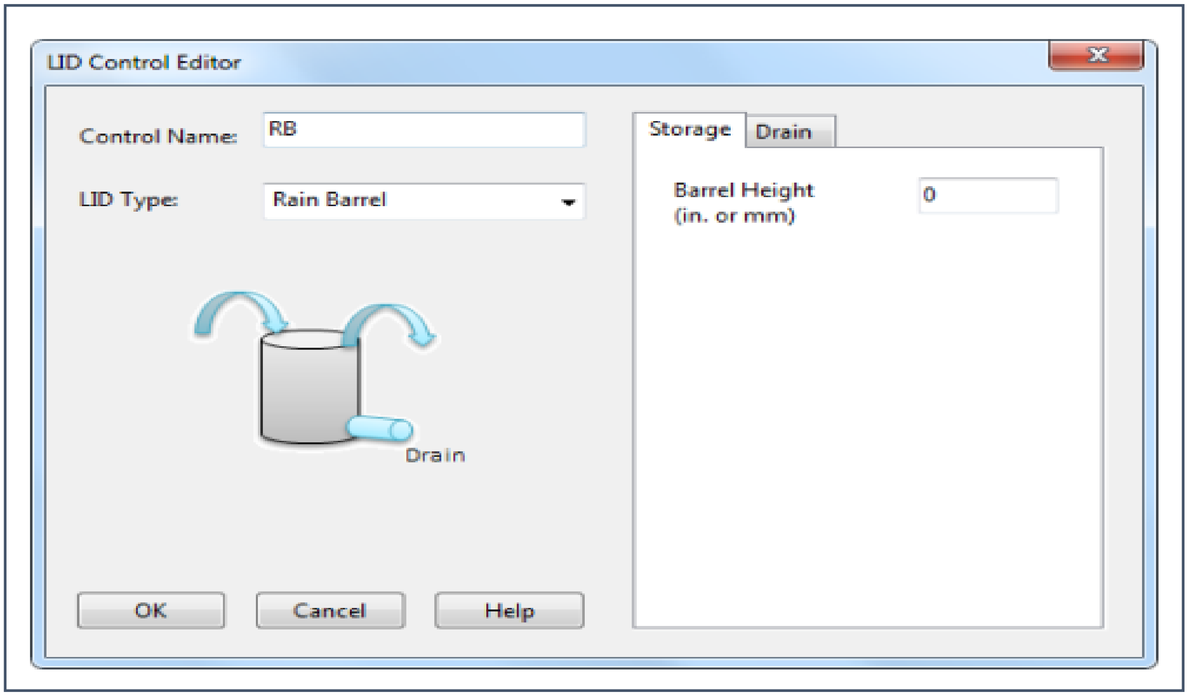

Figure 3-13. Rain Barrel Control Editor in SWMM5.

Table 3-9. Rain Barrel (Rainwater tank) parameters were used in SWMM5.

Volume(L)	1000L
Height (D)(mm)	1200
Drain coefficient(C) (mm/hr)	14.54
Drain exponent(n)	0.5
Drain offset Height(Hd)	0
Drain Delay (hour)	24
Flow coefficient	100
Average Roof area(m2) for 2-3 bed room	150m^2

50 houses were identified in sub- catchment 2 occupying 30% of the total sub - catchment area (25ha), 20 buildings in sub – catchment 3 occupying 20% of the total sub - catchment area (15ha) and 30 buildings in sub – catchment 5, occupying 28%, 25 house in sub – catchment 9 occupying 10%, 20 house in sub – catchment10 occupying 15 % its area using

site visit and earth Google helping. Once the housewere identified, 50,20,30,30,20,25 and 30 houses were picked within sub – catchment 2,3,5,6,7,9,10 respectively and the roof area of each building was 150m2 for 1 -2 bed room for each houses. 1000 litter rain barrels were simulated foreach house. Note that 1000litter rain barrel does not enough mean single rain barrel; it could be 2 size rain barrels was analyzed show in table below.

Table 3-10. The Number of Sample houses and different LID Scenarios input into SWMM5.

Sub catchment (zone1)	Number of Sample houses	Number of Sample Rain Barrel(RB)	Number of Sample Bio Retention(BR)
2	50	18	7
3	20	13	5
5	30	14	6
6	30	15	8
7	20	9	6
9	25	13	7
10	30	14	8
Total	205	96	47

CHAPTER FOUR

4. RESULTS AND DISCUSSIONS

4.1 Rainfall Data Analysis and IDF Curve Developing for Yeka Sub –City

4.1.1 Consistency Checking

The procedure is develop using the surrounding stations and thiessen polygon was created using ARC- GIS from that for selection of the nearest metrological stations, bole airport station is the nearest one to the study area. Independent rainfall data obtained from other 3 nearby stations (Addis Ababa observatory stations, Ayertena stations and Entoto station) were carefully checked for accuracy and consistency. a double mass curves were used to check the consistency of rain gauge record rainfall data and the correction factor were 1.023.

Table 4-1. Annual Accumulated Rainfall for Bole station and Average Accumulation of three stations.

Year	Annual maximum RF (mm) of Bole station	cumulative annual precipitation at Bole station(mm)	Addis Ababa observatory station(mm)	Entoto station	Ayertena station	Average of 3 stations	Average Accumulative RF 3 station(mm)
2018	59	59	68.7	65	64.5	66.1	66.1
2017	49.1	108.1	50.8	28.6	41.6	40.3	106.4
2016	29	137.1	54.9	46	43.5	48.1	154.5
2015	60.3	197.4	47.8	38.7	51.2	45.9	200.4
2014	27.2	224.6	65.4	35.5	39.6	46.8	247.3
2013	42.6	267.2	47.2	35.4	42.6	41.7	289.0
2012	64.7	331.9	36.4	31	43.4	36.9	325.9
2011	36.9	368.8	55.8	72.2	38.1	55.4	381.3
2010	54.4	423.2	44.8	98.3	59.4	67.5	448.8
2009	51.2	474.4	54.7	69.2	38.2	54.0	502.8
2008	37.2	511.6	53.3	58.3	34.7	48.8	551.6
2007	71.2	582.8	64	42.8	32.4	46.4	598.0
2006	61.7	644.5	70.9	55.6	60.1	62.2	660.2
2005	44.5	689	58.6	40.4	37.5	45.5	705.7

2004	29.6	718.6	44.2	40	17.4	33.9	739.6
2003	34.6	753.2	54.9	61.4	43.5	53.3	792.8
2002	28.6	781.8	29.5	43.6	29.1	34.1	826.9
2001	32.4	814.2	96.3	38.5	30.2	55.0	881.9
2000	47	861.2	37.1	38.2	50.2	41.8	923.7
1999	37.8	899	37.4	56.3	30.4	41.4	965.1
1998	60.1	959.1	78.3	56.4	97	77.2	1042.3
1997	37.3	996.4	46.3	90.3	50.9	62.5	1104.8
1996	52	1048.4	67	45.7	50.2	54.3	1159.1
1995	64.7	1113.1	85.3	43.3	49.9	59.5	1218.6
1994	38.2	1151.3	57	49.7	55.4	54.0	1272.7
	Total	604.636					646.6

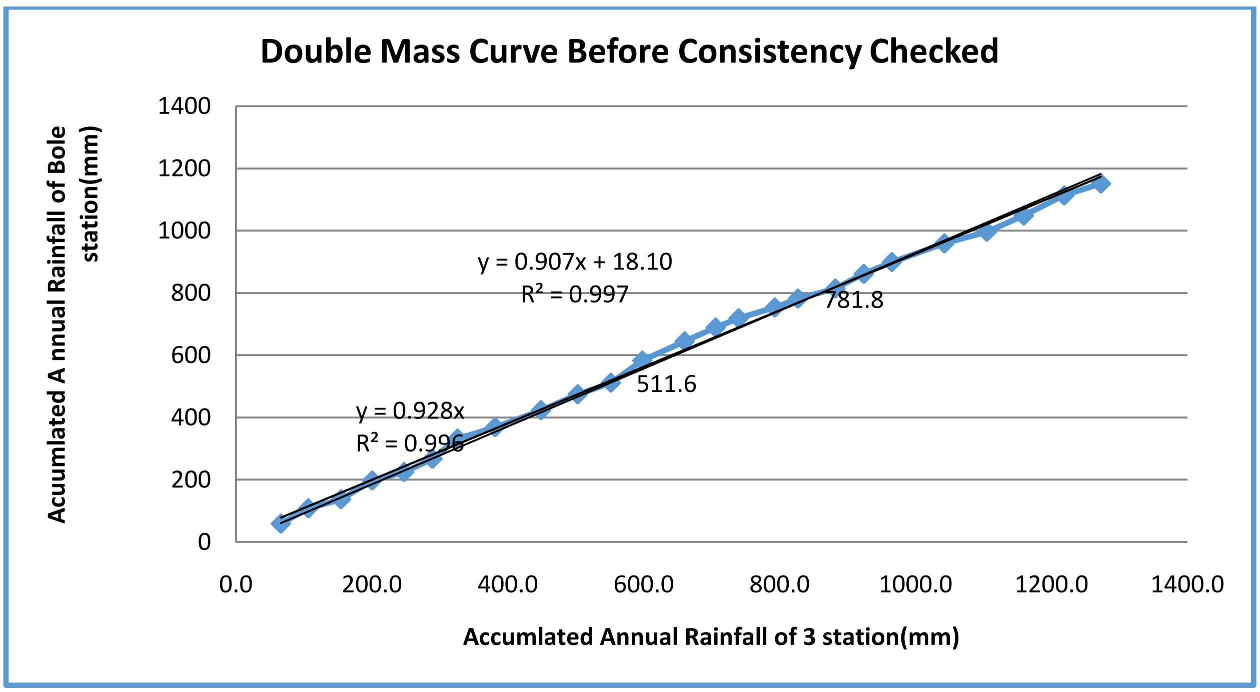

Figure 4-1 Double mass curve before consistency of rainfall data.

Where, Corrected value = Original value x (Mc/Ma) = PX*(MC/Ma).

Where, Px the rainfall data at bole station (Mc/ma), the slope ratio of the value at change in regime indicated (511.6 -781.8).

Table 4-2 The Corrected value of 25 Year Rainfall data performs at Bole station (1994 – 2018).

Year	Original Value of at Bole station (mm)	Corrected Value (mm)	Cum. Annual RF of Bole station(mm)
2018	59	63.13	63.13
2017	49.1	52.54	115.67
2016	29	31.03	146.70
2015	60.3	64.52	211.22
2014	27.2	29.10	240.32
2013	42.6	45.58	285.90
2012	64.7	69.23	355.13
2011	36.9	39.48	394.62
2010	54.4	58.21	452.82
2009	51.2	54.78	507.61
2008	37.2	39.80	547.41
2007	71.2	76.18	623.60
2006	61.7	66.02	689.62
2005	44.5	47.62	737.23
2004	29.6	31.67	768.90
2003	34.6	37.02	805.92
2002	28.6	30.60	836.53
2001	32.4	34.67	871.19
2000	47	50.29	921.48
1999	37.8	40.45	961.93
1998	60.1	64.31	1026.24
1997	37.3	39.91	1066.15
1996	52	55.64	1121.79
1995	64.7	69.23	1191.02
1994	38.2	40.87	1231.89

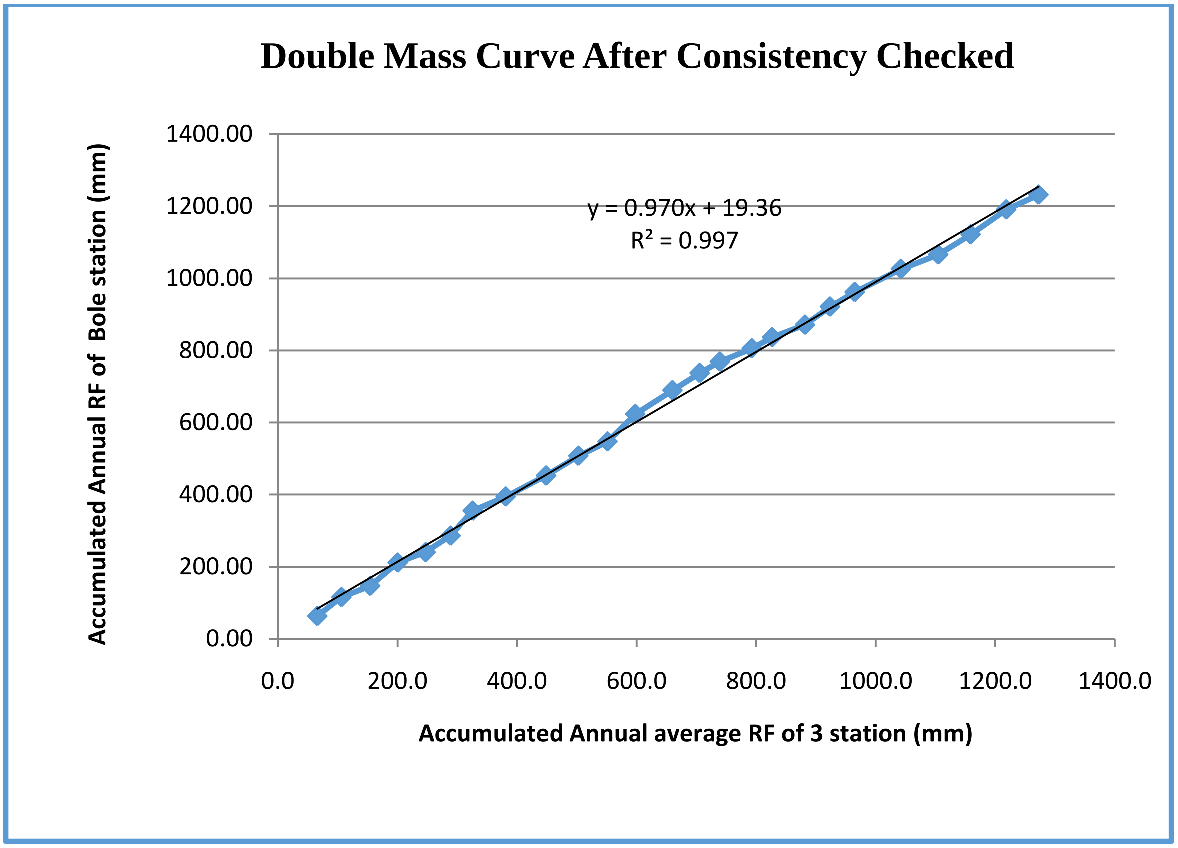

Figure 4-2. Double Mass curve After Consistency of Rainfall Checked.

4.1.2 Intensity – Duration – Frequency Curves Developing

IDF curves are obtained through frequency analysis of rainfall observations. The IDF curve is developed from 24-hour rainfall data of 25 years (i.e. 1994 to 2018), obtained from Ethiopian Meteorological Agency rainfall gauge located around Bole, Addis Ababa. Data From rainfall measurements, for every year of record, determine the annual maximum rainfall intensity for specific durations (or the annual maximum rainfall depth over the specific durations). Common durations for design applications are: 5-min, 10-min, 15-min, 30-min, 1-hr,2-hr, 6-hr, 12-hr, and 24-hr of shorter duration that SWMM5 simulation used.

The IDF curve developed by ERA under rainfall region A2 and the new developed by different distribution method checked by the goodness of fit tests and EV1 (Gumbel) were fit and as shown on figure 4.3 EV1 (Gumbel) was more maximum and used for IDF curve developed.

Table 4-3. Probability distribution of extreme value XT (mm) for Yeka Sub –City Were Compared with ERA Regional Rainfall depth (mm) -A2.

	Return period (Year) VS Rainfall Depth(mm)					
Probability Distribution	2 year	5 year	10 year	25 year	50 year	100 year
Normal	49.28	61.19	67.43	74.12	78.37	82.23
Log-normal	47.32	47.42	47.48	47.54	47.58	47.61
Log-Pearson	47.33	47.42	47.46	47.49	47.51	47.53
Gumbel (EVI)	46.95	59.47	67.75	78.22	85.99	93.70
Pearson Type -III	48.69	60.99	67.76	75.24	80.22	84.79
ERA (XT)	**51.92**	**65.52**	**74.45**	**85.70**	**94.07**	**102.45**

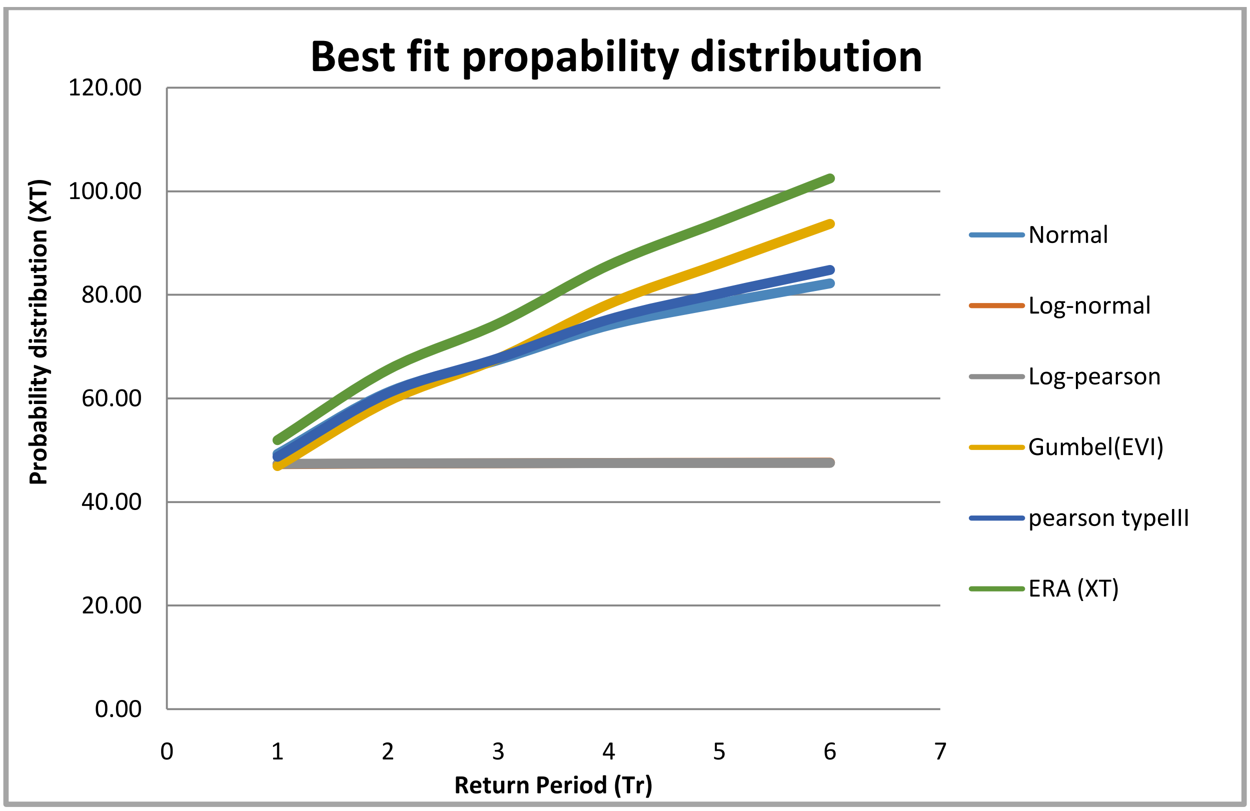

Figure 4-3. Graph best fit frequency distributions function Compare with ERA distribution.

Table 4-4. Geographic regions having similar flood frequency relationships (source: Ethiopia Road Authority Drainage Design Manual, 2013).

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

Note: RR- Rainfall Region

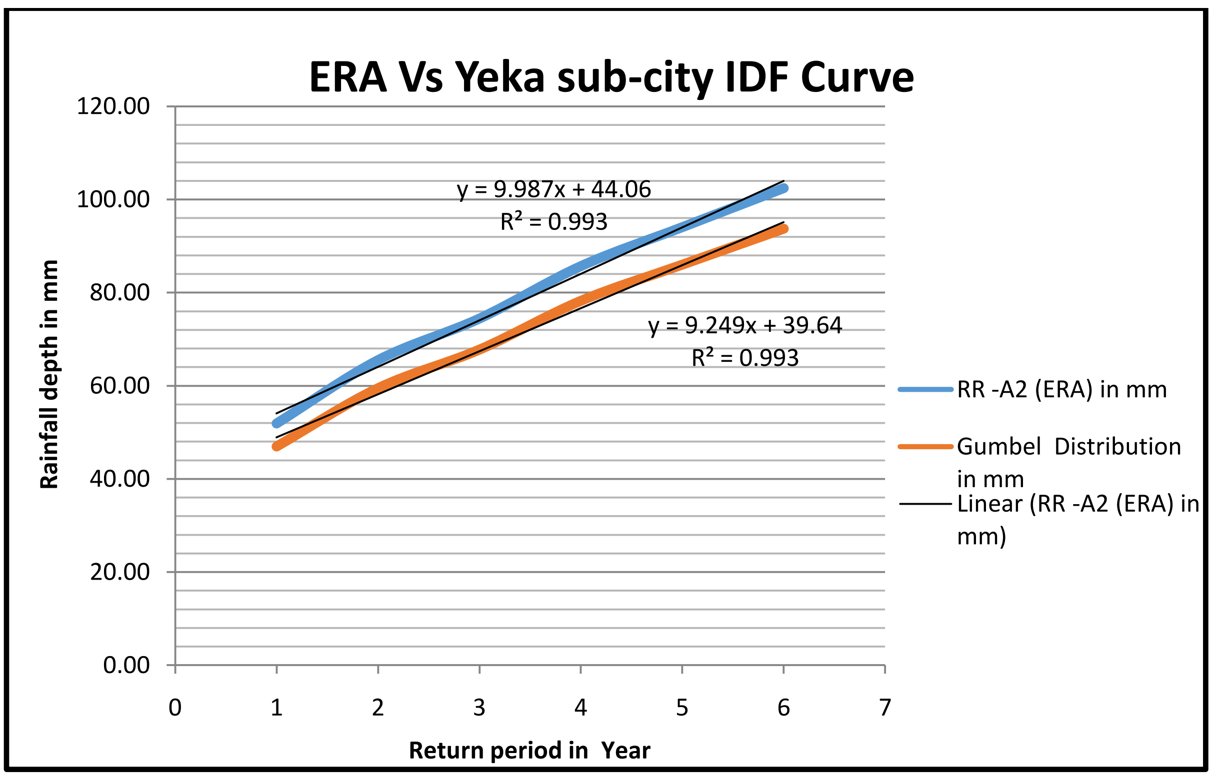

Figure 4-4. Compared of Yeka Sub-city IDF Curve with ERA

From figure 4.4 The IDF curve developed by ERA under rainfall region- A2, R^2 = 0.993 and the new IDF Curve developed for Yeka sub-city by different distribution method checked by the goodness of fit tests that R^2= 0.993 more suitable than ERA IDF curve. Therefore from result tested used the new IDF curve developed by Gumbel probability distribution where that best fit for IDF Curve of Yeka sub-city drainage systems sustainable management of study area.

Table 4-5. Intensity frequency curve (mm/hr) developing in different duration for given Return periods by Gumbel Probability Distribution.

Intensity(mm/hr) in different duration for given Return period							
Duration (mint)	Duration (hr)	2 year	5 year	10 year	25 year	50 year	1oo year
5	0.08	82.54	119.08	119.08	137.47	151.11	164.65
10	0.17	68.15	98.39	98.39	113.61	124.90	136.11
15	0.25	59.19	85.39	85.39	98.58	108.37	118.08
30	0.5	42.23	60.94	60.94	70.36	77.35	84.28
60	1	27.29	39.38	39.38	45.46	49.97	54.45
120	2	16.33	23.56	23.56	27.20	29.90	32.58
180	3	11.80	17.03	17.03	19.66	21.61	23.55
360	6	6.59	9.51	9.51	10.98	12.07	13.16
720	12	3.61	5.21	5.21	6.02	6.61	7.21
1440	24	1.96	2.82	2.82	3.26	3.58	3.90

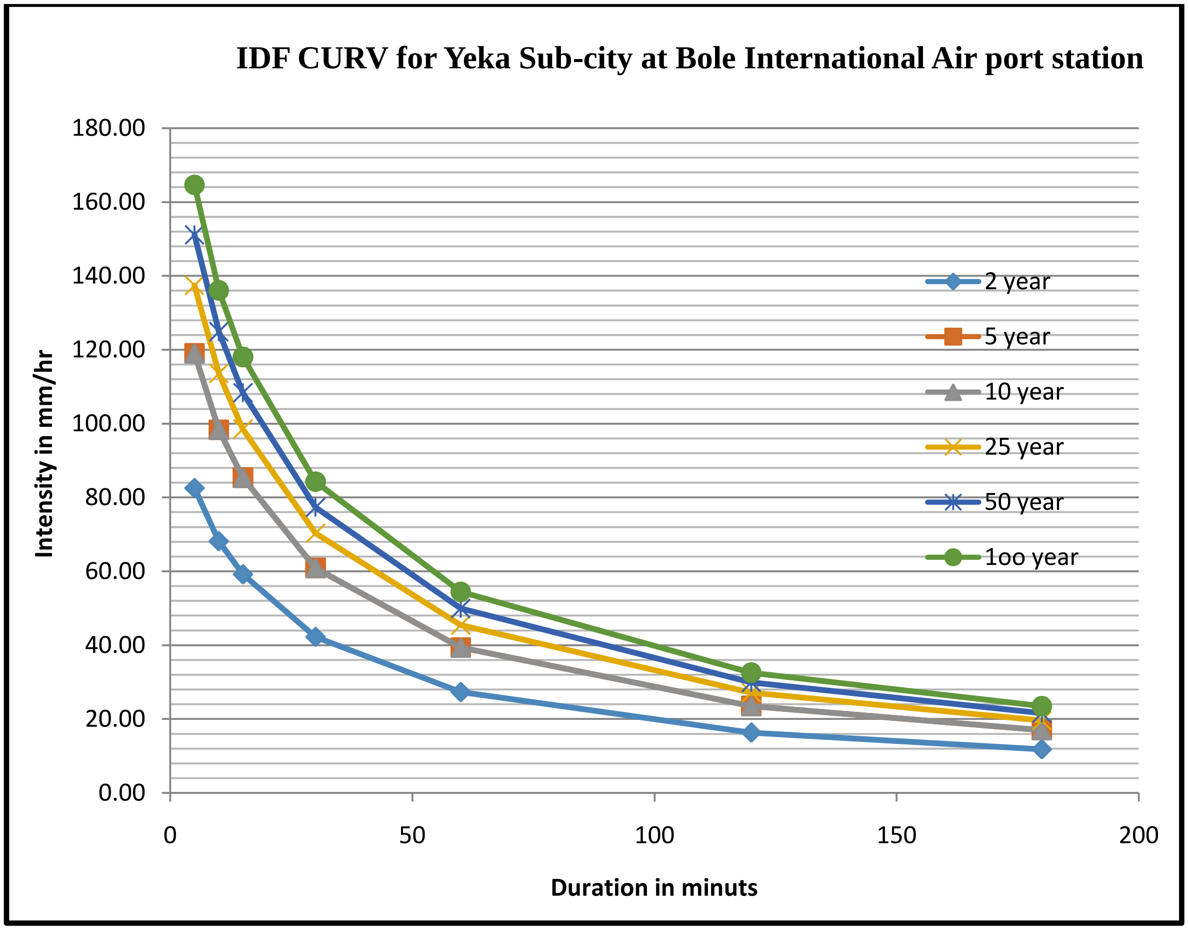

Figure 4-5. IDF Curve for Yeka sub-city from Bole Station Data.

4.2 Model Simulation and Calibration Results

The pipe flow alternative system is: where the runoff was collected and then routed into the pipes under certain pressure to fill the whole cross-section area. It is an underground system; SWMM was used to transform the rainfall into runoff and for further runoff simulation.

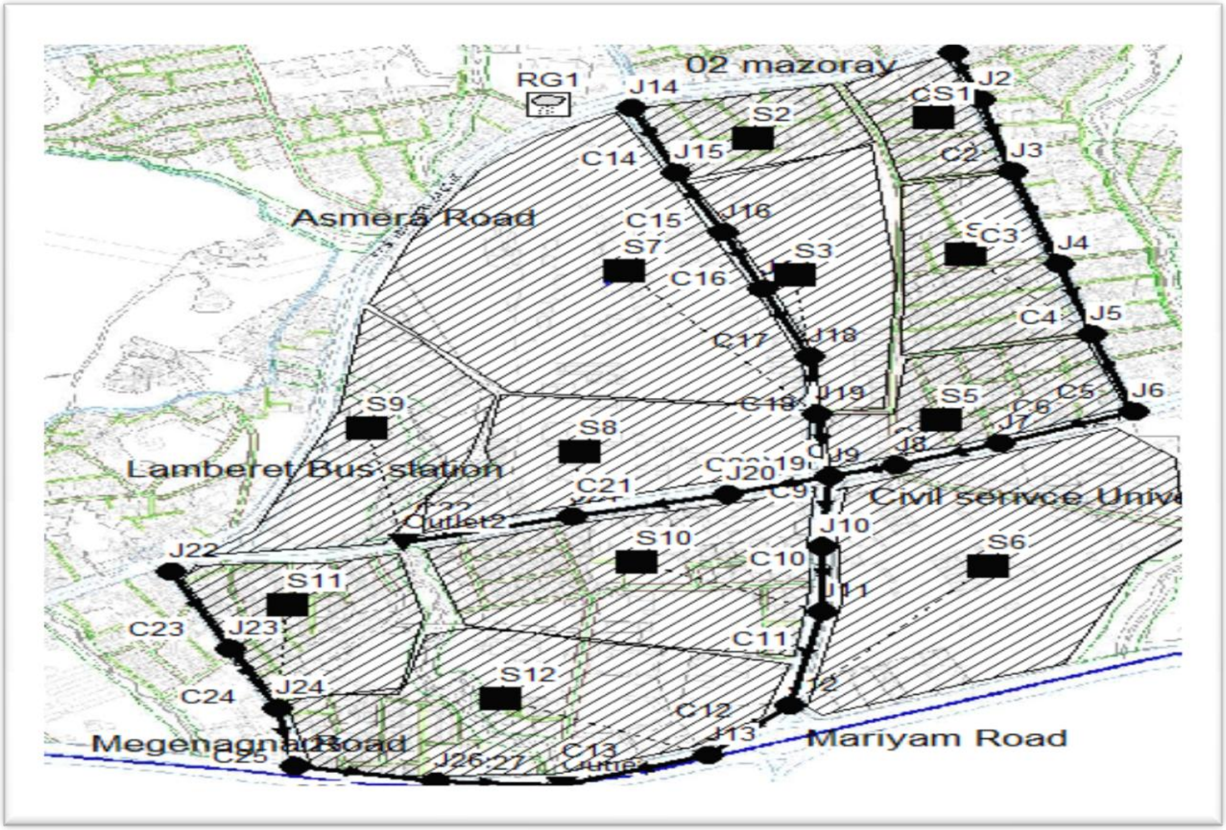

Figure 4-6.Zone 3 Schematic diagram showing conduits, junctions and sub-catchments pipe flow-based drainage system layout.

Table 4-5. Shows sub catchment groups and numbers of outlet, nodes and conduits classification.

Zone	Outlet	Sub catchment numbers	Area of Sub catchment(ha)	No of Node	Number of Conduit
CMC Roundabout	1	10	158	25	26
KotebeMetropolitan University	2	9	124	18	17
02toGurdshola	3	11	160	28	27
Total		30	442	71	70

The proposed development for the catchment not including significant changes in the topography, this implies that roads and flow pathways will follow the natural slope. The catchment will drain through a drainage system that compromised a network of 70 conduits/pipes that are connected with 71 nodes/manholes to collect the runoff from the sub catchments, and then to convey the runoff to the downstream outlet.

A total of three simulations were run with the SWMM model, for each of the two design storms (25 year return period), a simulation was run using Green - Ampt infiltration methods.

4.2.1 SWMM5 Parameter Calibration and Validation Results

As mentioned earlier, the model was calibrated for the July rainfall event and validated for the Augustrainfall event by comparing the simulated and measured flows at theoutlet1. The fitness evaluation of the calibration and validation hydrographs has been undertaken based on the Nash-Sutcliffe coefficient of efficiency (ENS), which is acommonly used goodness-of-fit measure in hydrological models. The ENS is suitable forreflecting the trends and overall fit of a flow hydrograph (Coutu et al., 2012). The ENS is represented by Equation below (taken from Nash and Sutcliffe, 1970). Where Qobs and Qsimu refer to the measured sewer flows and model simulated flows, respectively and N defines the number of observations. NSE = 1- $\left[\sum_{i=1}^{n}(Oi-Pi)^2 / \sum_{i=1}^{n}(Oi-\overline{O)2}\right]$.

If the value of the ENS is close to 1, it denotes that the prediction of the model simulated flow is as accurate as the measured flow. A comparison of the measured and simulated hydrograph for the calibration period is presented. whereas the validation hydrographs are presented in below with Manning's n using = 0.023 for open channel.

Table4-6 Calibration Event (July Rainfall)

Date	measured flow depth(m)	Bed width(m)	Perimeter (P)	Area (m2)	Hydraulic radius ®	Manning's formula V(m/sec)	measured flow (m3/sec)
1/7/2019	0.7	2.5	3.49	2.24	0.64	1.05	2.34
4/7/2019	0.75	2.5	3.56	2.44	0.68	1.09	2.66
8/7/2019	0.65	2.5	3.42	2.05	0.60	1.00	2.04
11/7/2019	0.7	2.5	3.49	2.24	0.64	1.05	2.34
14/7/2019	0.65	2.5	3.42	2.05	0.60	1.00	2.04
16/7/2019	0.7	2.5	3.49	2.24	0.64	1.05	2.34
20/7/2019	0.6	2.5	3.35	1.86	0.56	0.95	1.77
24/7/2019	0.7	2.5	3.49	2.24	0.64	1.05	2.34

25/7/2019	0.55	2.5	3.28	1.68	0.51	0.90	1.51
28//7/2019	0.75	2.5	3.56	2.44	0.68	1.09	2.66

Table 4-7. Calibration Results (July Rainfall)

Date	measured flow (m3/sec)	Simulated flow (m3/s)
1/7/2019	2.34	2.28
4/7/2019	2.66	2.77
8/7/2019	2.04	2.04
11/7/2019	2.34	1.97
14/7/2019	2.04	2.1
16/7/2019	2.34	2.04
20/7/2019	1.77	1.77
24/7/2019	2.34	2.09
25/7/2019	1.51	2.04
28//7/2019	2.66	2.7

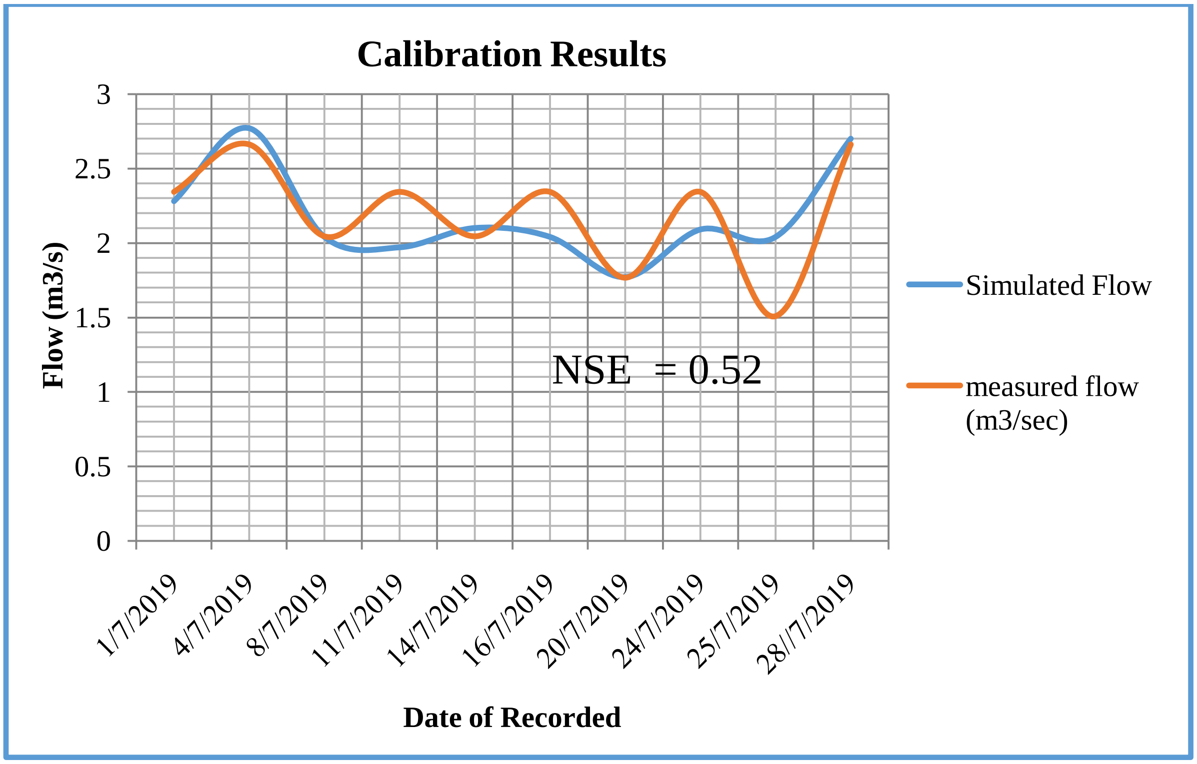

Figure 4-7. Hydrographs for the Calibration period (July rainfall event) at outlet1

Table 4-8. Validation Results (August Rainfall)

Date	measured flow (m3/sec)	Simulated flow (m3/s)
4/8/2019	2.04	2.43
5/8/2019	3.36	3.01
11/8/2019	2.34	2.18
14/8/2019	2.04	2.19
15/8/2019	1.93	2.23
17/8/2019	2.34	2.18
19/8/2019	1.76	1.89
22/8/2019	2.04	2.27
27/8/2019	2.34	2.18
29/8/2019	3.00	2.91

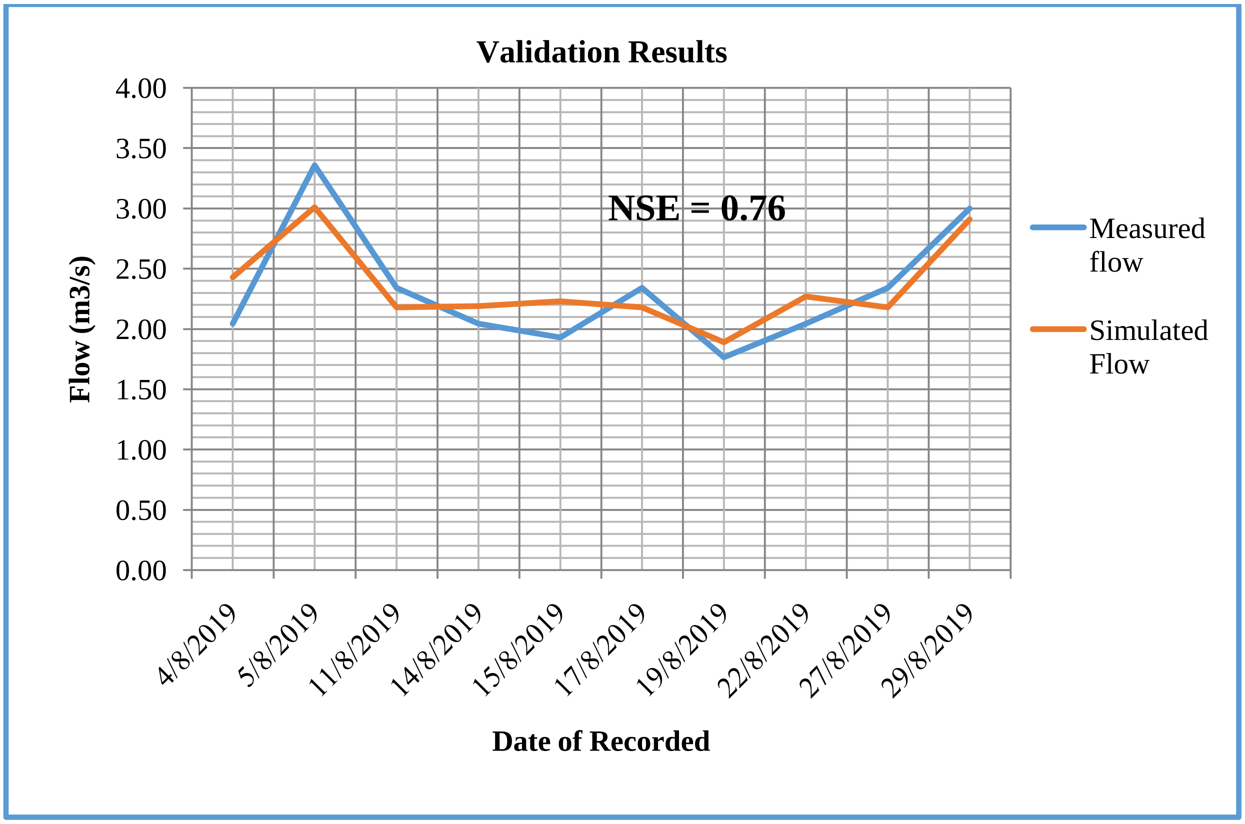

Figure 4-8. Hydrographs for the Validation period (August rainfall event) at outlet1

Performance is also achieved for the validation results, as can be seen in Figure above. The validation results again indicate good match between the measured and simulate hydrographs and a much better simulation of the peak flows (when compared to that in the calibration).

It was difficult to calibrate the model as no flow measurements have been done or any other data recorded for the study area. The rainfall intensity also was not measured except the daily average precipitation values (from National Meteorological Agency) which could not be used directly into the model. The only form of calibration that was done on the model wasmeasured using rainfall season in the outlets since the area under study was floods at least once every rain season. The rainfall intensity was extracted from IDF curves generated from short term rainfall period of 1994 to 2018 and the hourly data used on two month measurement July and August in 2019 rainfall data for calibration and validations respectively.

4.2.2 Model Simulation Results

A simulation result can be displayed and analyzed in tables, on graphs, in map views and within status reports. Table results provide tabulated values for infiltration, runoff, outflow and storage at each 15-minute interval on 25 year design storm. Graphs can be created and viewed in SWMM as a profile, time series or scatter plot. For each graph, an object (i.e. sub catchments, node, system, etc.) and a variable (i.e. precipitation, runoff, infiltration, etc.) can be selected and viewed in a graph form. The sub catchments, nodes and links of the map will be colored according to their respective Map Legends and change at each time interval for the storm duration.

Table 4-9. The Runoff simulation Results from 25 Year Rainfall Data on the Three Zone of Sub-Catchments for 25 return period Storm Events.

Sub–catchment(zone1)	Total precipitation (mm)	Infiltration (mm)	Surface Runoff (10^6 ltr)	Peak runoff (m3/s)	Runoff coefficient
S1	32.50	6.35	3.70	2.28	0.669
S2	32.50	6.45	5.13	2.77	0.632
S3	32.50	6.34	3.28	2.04	0.672
S4	32.50	6.53	4.42	1.97	0.591
S5	32.50	6.36	3.46	2.10	0.665
S6	32.50	6.34	3.28	2.04	0.672
S7	32.50	6.34	2.84	1.77	0.673
S8	32.50	6.42	3.75	2.09	0.640
S9	32.50	6.34	3.28	2.04	0.673
S10	32.50	6.34	4.37	2.74	0.673
Zone 2					
S1	55.92	12.01	3.01	1.05	0.668

S2	55.92	12.26	5.04	1.69	0.628
S3	55.92	12.25	3.10	1.04	0.628
S4	55.92	12.25	4.91	1.65	0.628
S5	55.92	12.25	7.18	2.41	0.628
S6	55.92	12.25	4.46	1.50	0.628
S7	55.92	12.25	10.94	3.68	0.628
S8	55.92	12.25	7.34	2.47	0.628
S9	55.92	12.25	3.81	1.32	0.628
Zone3					
S1	55.92	9.83	4.04	1.46	0.722
S2	55.92	9.83	3.31	1.20	0.722
S3	55.92	9.83	7.67	2.78	0.722
S4	55.92	9.83	6.06	2.19	0.722
S5	55.92	9.83	5.25	1.90	0.722
S6	55.92	9.83	8.07	2.92	0.722
S7	55.92	9.83	8.48	3.07	0.722
S8	55.92	9.83	4.04	1.46	0.722
S9	55.92	9.83	3.23	1.17	0.722
S10	55.92	9.83	5.65	2.04	0.722
S11	55.92	9.84	7.85	2.82	0.722

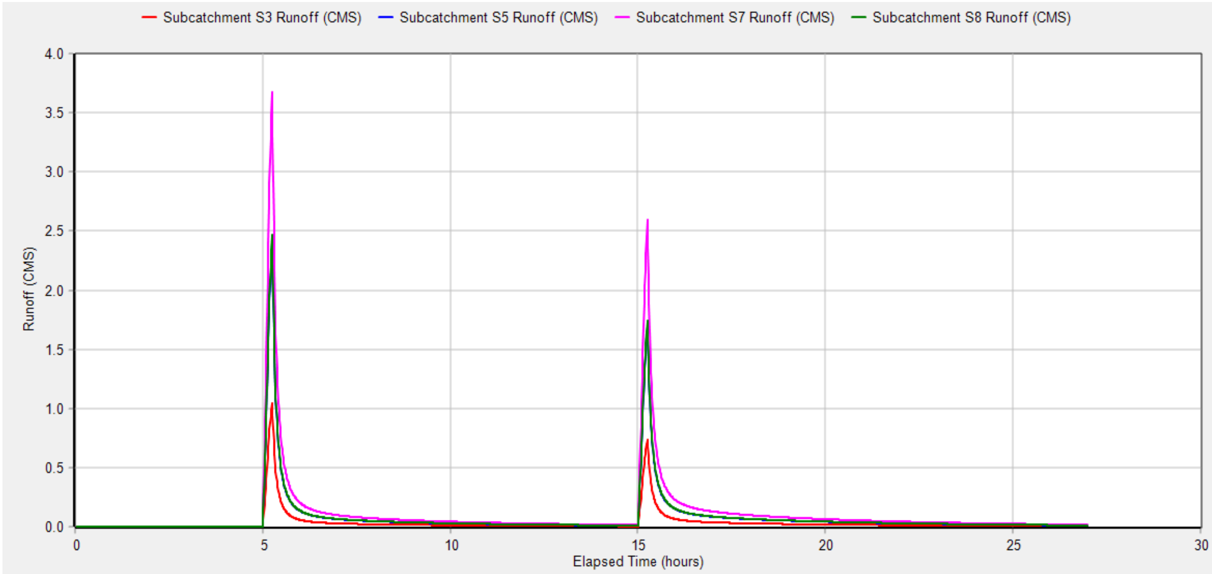

Figure 4-9. The surface runoff Hydrograph simulation on SWMM5.

4.2.3 Network Simulation

The drainage systems modeled to cope with a 25- years return period rainfall in terms of water level below the surface water elevation in drainage systems. This implies that the flooding risk must be verified in the nodes (manholes) in the systems, whereas water level at each lateral connection must be checked independently with a longitudinal profile in parts of the system which are connected The general network performance is determined by infiltration rates and the average water flow production. The water elevation profile in each node was over flooded as shown in figure below.

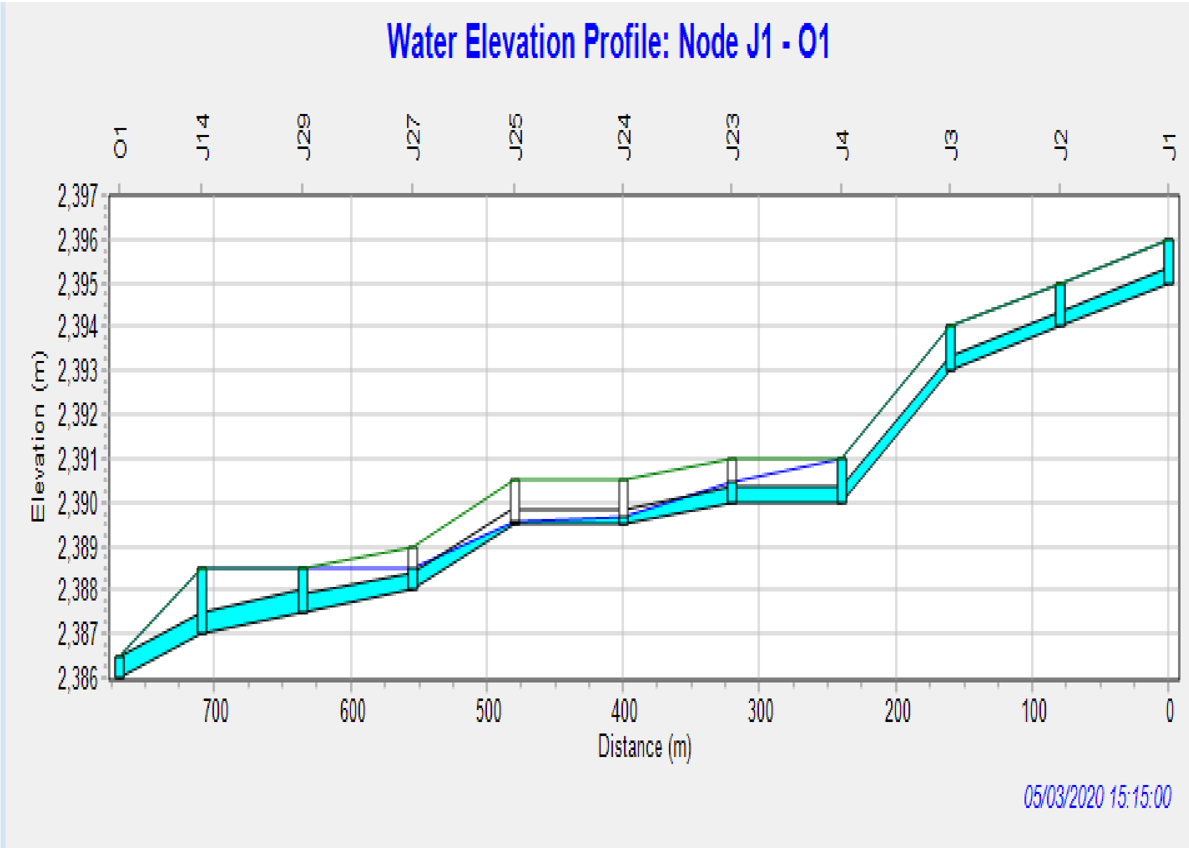

Figure 4-10. Water elevation profile of flooded junction to outlet 1 for zone1sub-catchment.

4.2.4 Flow Routing

Dynamic wave routing was used for flow calculation within the hydraulic system. Table 20 summarizes inflow, external outflow, internal outflow and percent flooded for each sub - catchments. Inflow is the surface water runoff from the previous overland flow routing methods. Thus, inflow for each storm event in Table 4.8 is equal to the surface runoff in Table 4.7 External outflow represents the water which was routed through the drainage system and discharged at the outlet location. Internal outflow occurs when there is surcharge that creates flooding within the system. Surcharge happens when all pipes entering a node are full or when the water surface at the node is between the crown of the highest entering pipe and the ground surface (James et al, 2010).

Table 4-10. Summary for Dynamic Wave Flow Routing Results for 25 return period storm event.

Sub catchment	Surface Runoff(10^6) ltr	External out flow (10^6) ltr	Internal outflow(10^6)ltr	%Continuity error	Percent flooded (%)
Zone 1	37.486	6.922	30.328	-0.004	47.70
Zone 2	49.628	29.112	20.516	-0.002	41.34
Zone 3	63.631	40.533	23.098	-0.003	36.29

Results from all simulations showed that there was internal outflow, or flooding, within the drainage network during flow routing. The percentage flooded was calculated as internal outflow divided by inflow, which exceeded 36%. Furthermore, while the simulated flooding may have a small impact on the timing of the water arrival at the study area outfall, it does not have a significant impact on the total flow rates simulated by SWMM.

4.2.5 Continuity Error

Continuity errors are quality assurance calculations performed by SWMM. There are two continuity errors calculated; one for runoff modeling and one for flow routing. These errors recalculated for the system by summing final storage and total outflow, then subtracting it from the initial storage plus total inflow (Aerts et al., 2015).The continuity errors should not exceed 10percent, otherwise the validity of the system and model should be questioned and re-examined. The value considered to be a good continuity error is often a broad range and is subjective. some researchers state that less than 1 percent as excellent and less than 5 percent as acceptable (Aertset al., 2015).Thus, as can be seen in the status reports in the continuity errors are -0.747 and -0.004 % for runoff routing continuity error and for the flow routing portion of the model respectively as shown in Table 4.8 below. Based on the criteria that less than 1%continuity is excellent, these errors are well within that range.

Table 4-11. Status Report Results for Sub Catchment Groups Zone1.

Runoff Routing				Flow routing		
Runoff quantity continuity	Volume Hectare- m	Depth (mm)	Volume (10^6 ltr)		Volume (hec –m)	Volume (10^6ltr)
Total precipitation	8.854	55.920	88.54	Wet weather inflow	6.208	62.079

Infiltration loss	1.552	9.805	15.37	External outflow	3.247	32.407
Surface runoff	6.209	39.213	62.079	Internal outflow	2.961	29.672
Final surface storage	1.159	7.320	1.56	Final storage volume	0.008	0.076
% Runoff	70.12			Percentage flooded (%)	47.7	
Continuity Error (%)	-0.746			Continuity Error (%)	-0..004	

4.3 Simulation Results after Low Impact Development Scenario

Two scenarios were considered for this combination of modeling. Firstly the number of households was taken as 100% (all the households) that had installed 1000L Rain Barrels and 262m^2 (10.5x25m) each Bio Retention. The result indicated a reduction in runoff volume by16.28%, when compared to the base case overflow volume of 62090m^3 in zone 1 sub catchment. The hydraulic performance of the case study was assessed in terms of reduction in runoff volume. It is worth mentioning that the results in terms of percentage reductions in runoff volumes are specific to this catchment and would vary from catchment to catchment. Moreover, as indicated earlier, the presented results are for the 25 year return period and would vary for a different time period (when parameters like size of the storms and recurrence interval would be different).

Table 4-12. Total Runoff Volume reduction after implementing different SUDS Scenarios was developing with 25 year return periods.

Base case (without LID)	S2	S3	S5	S6	S7	S9	S10
Areaofsubcatchment(ha)	25	15	16	15	13	15	20
Subcatchment Runoff volume (1x10^3) m3	5.13	3.28	3.46	3.28	2.84	3.28	4.37

peak flow (m3/s)	2.77	2.04	2.10	2.04	1.77	2.04	2.72
With LID (Rain Barrel)							
No of barrel	18	13	14	8	9	13	14
Area of Roofs(m^2)	2700	1950	2100	1200	1350	1950	2100
Runoff After Reduction (1x10^3)m^3	4.89	3.16	3.33	3.23	2.78	3.17	4.24
Peak flow After Reduction (m3/s)	2.28	1.82	1.87	1.96	1.67	1.83	2.48
Bio Retention (BR)							
No of Bio Retention	7	5	6	8	6	7	8
Area of BR(m^2)	1838	1313	1575	2100	1575	1838	2100
Runoff After Reduction (1x10^3)m3	4.90	3.16	1.59	2.86	2.66	3.04	4.06
Peak flow After reduction (m3/s)	2.70	1.99	1.16	1.90	1.69	1.94	1.59
Rain Barrel + Bio Retention (RB +BR) Model Runoff After Reduction Results							
Runoff Reduction(1x10^3)m3	4.66	3.04	1.48	2.91	2.61	2.93	3.92
Peak flow Reduction (m3/s)	2.41	1.76	0.71	1.82	1.59	1.70	2.32
R(%) of Runoff After reduction	8.77	7.31	57.22	11.28	8.45	10.67	10.29

R(%) of Peak flow After Reduction	12.99	13.72	66.19	10.78	10.16	16.67	14.70

Quantifying the effects of each LID installation in the different sub catchments using the following equation; this equation has been used in many of studies related to the storm-water reduction effect. $R(\%) = (\ Q_O - Q_{LID})/Q_O) \times 100$

Where R is the reduction rate, Q_0 (CMs) is the simulated runoff without LID installations and Q_{LID} (CMs) is the simulated runoff with LID installations. From the above table simulation result the value of runoff reduction and peak flow reduction are 16.28% and 20.74% respectively.

5. CONCLUSION AND RECOMMENDATION

5.1 CONCLUSIONS

This study focus on the issues of urbanization and the options for sustainable urban drainage management us an important frame work for the evaluation of the storm water runoff in the Yeka sub city. Urban catchment and also explores the impact of Low Impact Development techniques/Sustainable Urban drainage structures in an urban Environment. Drainage systems are becoming more vulnerable to failure mainly due to rapid urbanization (resulting in more impervious areas).Among these urban problems, the negative Impact of runoff on the road which affect the traffic flow, day to day activates and environmental issues is among the most serious ones.

The objectives of this research were to mitigate the negative impacts of the intense rainfall events on the performance of the drainage system. Since storm water management using sustainable urban drainage system (SUDs) and explored the impact of the commonly used LID approaches as mitigation techniques for reducing the rainfall induced drainage system over flow. LID is a land planning and storm water management technique that minimizes Storm water runoff through natural resource based site design and seeks to control runoff as near to the source of storm water as possible. The representative design storm of 25-years rainfall event that was lasted for 180minutes was applied to the contributing area then was converted into runoff. The runoff quantities were estimated using the manning's formula under current rainfall conditions.

For such urbanized study area with highly impervious surfaces, EPA's Storm Water Management Model (SWMM) was preferred and determined to be a reasonable choice for hydrological modeling. Section of the sub- city of Yeka was selected as the study area; with an outlet location was delineated based primarily on the drainage network with adjustments around the perimeter based on topography. Once the study site was defined, the area was further divided into three catchments zones and 31 sub catchments. Within these sub catchments the networks pipes were aggregate to be represented by71 junctions and 70 conduit connections. These sub-catchments were drawn into the SWMM model and the sub catchment, junction and conduit parameter were estimated and entered. These parameters were required to tailor the runoff processes and flow routing equations for the study area. The infiltration methods for the runoff processes were Green-Ampt method using. These methods required input parameters to the physical characteristics of surface impervious values. Most of this data was obtained from the MoWIE in the form of GIS data files, which were analyzed and manipulated with Arc Map software. The data required were impervious surfaces, soil type, land use type and raster Digital Elevation Model (DEM) map. The flow Routing Equation used for this model was dynamic wave routing. The hydraulic system of Junctions and Conduits required input values obtained from the Addis Ababa City Road Authority (AACRA) Engineering Department, including invert elevations and slope, shape, and diameter of the pipes.

Among various LID options, rain barrels and bio retention cell were identified as the best choice for this area because of their effectiveness in reducing peak flow and runoff as well as their low costs. Further, they occupy very small areas and require very little maintenance. Rain barrels and bio retention cell were simulated as LID practices in 7 sub catchments. It was estimated that suggested LID practices, under rainfalls corresponding to return period 25-year storm events can decrease peak flow by 14.42% and Runoff Volume by 10.87% compared to the existing conditions without LID. These results show that LID is valuable consideration for urban flood control as a storm water management planning tool.

5.2 RECOMMENDATION

- The expected change in the future rainfall patterns and the need for more urban areas are challenging the Yeka metropolitan sub-city. Provision of proper connections or integrations between the road network and drainage network systems such as the problem of integration between railway and the drainage system in phase Megenegna to CMC is required with regular maintenance and redesign.
- Once the proper adjustments are made and the flow routing process shows minimal internal flooding during a storm event the model should be calibrated. As mentioned, due to lack of material, economy, time and knowledge train events during the spring term (measuring flow depth and flow velocity using Andersen) the only precipitation data was available from national metrological agency, field data should be collected to

calibrate the model. Calibration is essential to validate the model as an accurate representation of the drainage area.

- Arc-GIS-SWMM hybrid modeling could be used as an effective tool for carrying out small and large-scale urban catchment modeling and simulations. One of the limitations of SWMM is the calibration process (manual) required is too time consuming and inefficient. A superior model like PCSWMM with PEST calibration feature or calibration using R programming is recommended for future storm water modeling to save time and increase productivity.
- Although this study properly considered the effect of LID locations and sizes in the modeling processes by introducing relative performance of each LID. The analysis of cost-to-benefit representing percent runoff volume reduction per unit cost could not be conducted due to lack of published cost information. To determine a more cost-effective LID controls a preliminary cost-to-benefit analysis is suggested to be performed in addition to site characterization.
- This study helps the communities in planning valuable step for new sustainable urban development improved esthetics which results in increased property values. Also, it reduces storm water bills of households. Furthermore, it reduces energy consumption needs, costs to build storm drain infrastructure, water treatment costs and reduces property damage from flooding which makes space healthier.
- These findings demonstrate that proper LID sizing and design are essential to maximize their benefits under given storm water management policy. Performance of rain barrels can be improved by increasing the Roof Treatment area and the capacity of a rain-barrel. Performances of bio-retention can be improved by increasing the surface area being treated by bio-retention or by changing the bio-retention design like soil composition, storage height, conductivity and berm height.
- This Study was beneficial to water resources engineer who want to evaluate the effect of different LID at particular site implementation of the Low Impact Development techniques. As evident from this Study do considerably reduce the flooding and simultaneously having the positive impact for ecology and water cycle.

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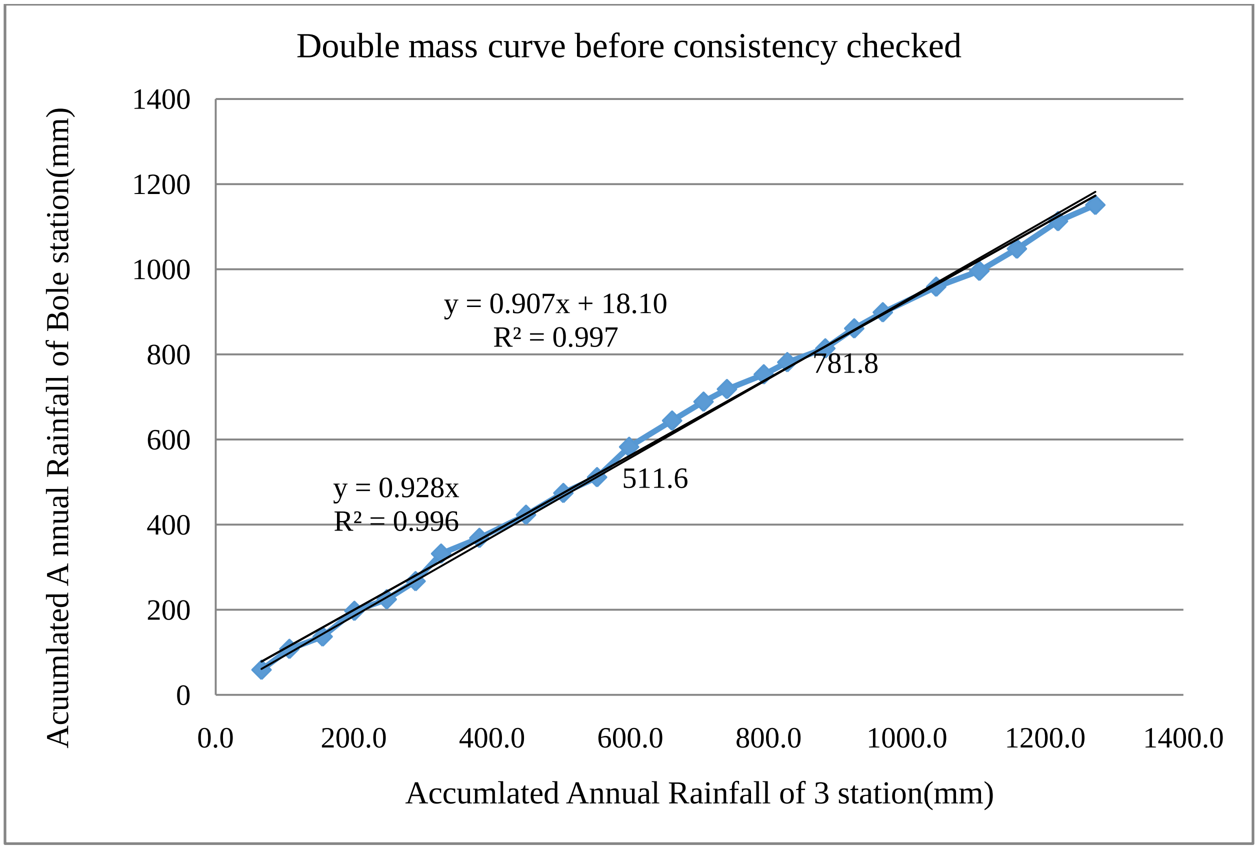
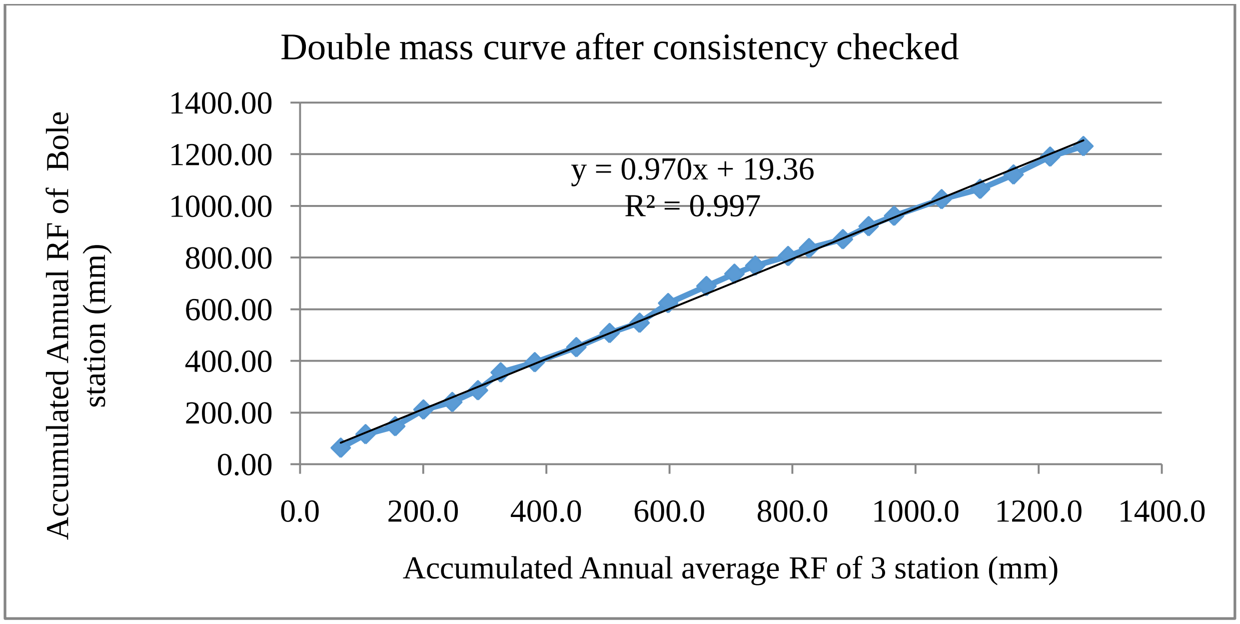
APPENDIX

Appendix A: Yearly one day maximum precipitation of the four meteorological stations.

Year	Annual maximum RF (mm) of Bole station	Accumulative annual precipitation at Bole station(mm)	Addis Ababa observatory station(mm)	Akaki station	Ayertena station	Average of 3 stations	Avrg Accumulative RF 3 station(mm)
2018	59	59	68.7	65	64.5	66.1	66.1
2017	49.1	108.1	50.8	28.6	41.6	40.3	106.4
2016	29	137.1	54.9	46	43.5	48.1	154.5
2015	60.3	197.4	47.8	38.7	51.2	45.9	200.4
2014	27.2	224.6	65.4	35.5	39.6	46.8	247.3
2013	42.6	267.2	47.2	35.4	42.6	41.7	289.0
2012	64.7	331.9	36.4	31	43.4	36.9	325.9
2011	36.9	368.8	55.8	72.2	38.1	55.4	381.3
2010	54.4	423.2	44.8	98.3	59.4	67.5	448.8
2009	51.2	474.4	54.7	69.2	38.2	54.0	502.8
2008	37.2	511.6	53.3	58.3	34.7	48.8	551.6
2007	71.2	582.8	64	42.8	32.4	46.4	598.0
2006	61.7	644.5	70.9	55.6	60.1	62.2	660.2
2005	44.5	689	58.6	40.4	37.5	45.5	705.7
2004	29.6	718.6	44.2	40	17.4	33.9	739.6
2003	34.6	753.2	54.9	61.4	43.5	53.3	792.8
2002	28.6	781.8	29.5	43.6	29.1	34.1	826.9
2001	32.4	814.2	96.3	38.5	30.2	55.0	881.9
2000	47	861.2	37.1	38.2	50.2	41.8	923.7
1999	37.8	899	37.4	56.3	30.4	41.4	965.1
1998	60.1	959.1	78.3	56.4	97	77.2	1042.3
1997	37.3	996.4	46.3	90.3	50.9	62.5	1104.8
1996	52	1048.4	67	45.7	50.2	54.3	1159.1
1995	64.7	1113.1	85.3	43.3	49.9	59.5	1218.6
1994	38.2	1151.3	57	49.7	55.4	54.0	1272.7
	Total	604.636					646.6


Double mass curve before consistency checked
Acuumlated A nnual Rainfall of Bole station(mm)
Accumlated Annual Rainfall of 3 station(mm)
y = 0.907x + 18.10
R² = 0.997
781.8
y = 0.928x
R² = 0.996
511.6
1400
1200
1000
800
600
400
200
0
0.0
200.0
400.0
600.0
800.0
1000.0
1200.0
1400.0


Double mass curve after consistency checked
Accumulated Annual RF of Bole station (mm)
Accumulated Annual average RF of 3 station (mm)
y = 0.970x + 19.36
R² = 0.997
1400.00
1200.00
1000.00
800.00
600.00
400.00
200.00
0.00
0.0
200.0
400.0
600.0
800.0
1000.0
1200.0
1400.0

Appendix B: The Corrected value of 25 Year Rainfall data performs at Bole station (1994 – 2018).

Year	Original Value of at Bole station (mm)	Corrected Value (mm)	Cum. Annual RF of Bole station(mm)
2018	59	63.13	63.13
2017	49.1	52.54	115.67
2016	29	31.03	146.70
2015	60.3	64.52	211.22
2014	27.2	29.10	240.32
2013	42.6	45.58	285.90
2012	64.7	69.23	355.13
2011	36.9	39.48	394.62
2010	54.4	58.21	452.82
2009	51.2	54.78	507.61
2008	37.2	39.80	547.41
2007	71.2	76.18	623.60
2006	61.7	66.02	689.62
2005	44.5	47.62	737.23
2004	29.6	31.67	768.90
2003	34.6	37.02	805.92
2002	28.6	30.60	836.53
2001	32.4	34.67	871.19
2000	47	50.29	921.48
1999	37.8	40.45	961.93
1998	60.1	64.31	1026.24
1997	37.3	39.91	1066.15
1996	52	55.64	1121.79
1995	64.7	69.23	1191.02
1994	38.2	40.87	1231.89

Appendix C: sub catchment input parameters into SWMM5

Characteristics	Description
Rain Gages	Refers to the rain gage where the rain intensity is defined over a time interval
Outlet	Defines which node or sub catchment is receiving the flow
Area	Area of the sub catchment including any LID controls
Width	Characteristic width of the overland flow path for sheet flow runoff from non-LID area only.
% Slope	Average percent slope of the sub catchment
% Impervious	Percent of the land area which is impervious
N - Impervious	Manning's n for overland flow over the impervious portion of
	the sub catchment
N - Pervious	Manning's n for overland flow over the previous portion of the sub catchment
D store - impervious	Depth of the depression storage on the impervious portion of the sub catchment
D store - pervious	Depth of the depression storage on the pervious portion of the sub catchment
% zero - impervious	Percent of the impervious area with no depression storage
Sub area routed	Choice of internal routing of flow between pervious and impervious sub-areas (allows directing the flow between the pervious and impervious areas within a sub catchment)
% routed	Percent of the diverted flow toward a sub-area within sub catchment.
LID Controls	This is used to edit the use of Low Impact Development controls in the sub catchment.

Appendix D: The Sub catchment Manning's Runoff Estimation

Calibration Event (July Rainfall)								
Date	measured flow depth(m)	Bed width(m)	Perimeter (P)	Values of manning's coefficient n	Area (m2)	Hydraulic radius ®	Manning'sformula V= 1/n. R^(2/3). S^(1/2)	measured flow (m3/sec)
1/7/2019	0.7	2.5	3.49	0.02	2.24	0.64	1.05	2.34
4/7/2019	0.75	2.5	3.56	0.02	2.44	0.68	1.09	2.66
8/7/2019	0.65	2.5	3.42	0.02	2.05	0.60	1.00	2.04
11/7/2019	0.7	2.5	3.49	0.02	2.24	0.64	1.05	2.34
14/7/2019	0.65	2.5	3.42	0.02	2.05	0.60	1.00	2.04
16/7/2019	0.7	2.5	3.49	0.02	2.24	0.64	1.05	2.34
20/7/2019	0.6	2.5	3.35	0.02	1.86	0.56	0.95	1.77
24/7/2019	0.7	2.5	3.49	0.02	2.24	0.64	1.05	2.34
25/7/2019	0.55	2.5	3.28	0.02	1.68	0.51	0.90	1.51
28//7/2019	0.75	2.5	3.56	0.02	2.44	0.68	1.09	2.66

Appendix E: SWMM5 sub catchment runoff simulation results.

Sub – catchment (zone1)	Total precipitation (mm)	Infiltration (mm)	Surface Runoff (10^6 ltr)	Peak runoff (m3/s)	Runoff coefficient
S1	55.92	9.77	6.47	2.18	0.708
S2	55.92	9.77	9.90	3.34	0.708
S3	55.92	9.78	5.93	2.00	0.708
S4	55.92	9.77	3.96	1.34	0.708
S5	55.92	9.77	6.33	2.14	0.708
S6	55.92	10.21	5.94	2.00	0.708
S7	55.92	9.78	4.57	1.02	0.628

S8	55.92	9.77	5.14	1.74	0.708
S9	55.92	9.77	5.94	2.01	0.708
S10	55.92	9.77	7.92	2.67	0.708
Zone 2					
S1	55.92	12.01	3.01	1.05	0.668
S2	55.92	12.26	5.04	1.69	0.628
S3	55.92	12.25	3.10	1.04	0.628
S4	55.92	12.25	4.91	1.65	0.628
S5	55.92	12.25	7.18	2.41	0.628
S6	55.92	12.25	4.46	1.50	0.628
S7	55.92	12.25	10.94	3.68	0.628
S8	55.92	12.25	7.34	2.47	0.628
S9	55.92	12.25	3.81	1.32	0.628
Zone3					
S1	55.92	9.83	4.04	1.46	0.722
S2	55.92	9.83	3.31	1.20	0.722
S3	55.92	9.83	7.67	2.78	0.722
S4	55.92	9.83	6.06	2.19	0.722
S5	55.92	9.83	5.25	1.90	0.722
S6	55.92	9.83	8.07	2.92	0.722
S7	55.92	9.83	8.48	3.07	0.722
S8	55.92	9.83	4.04	1.46	0.722
S9	55.92	9.83	3.23	1.17	0.722
S10	55.92	9.83	5.65	2.04	0.722
S11	55.92	9.84	7.85	2.82	0.722

Appendix F: sub catchment water surface elevation of the three zones and three outlets.

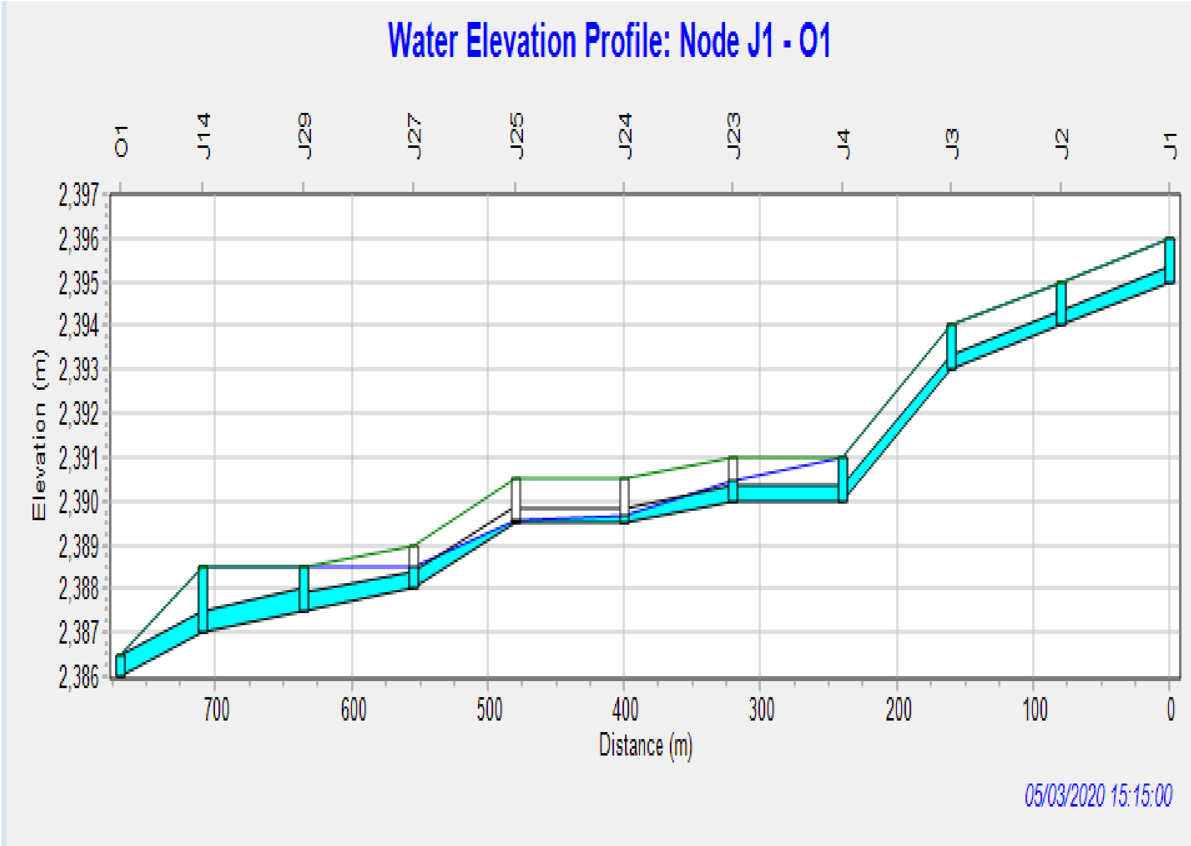


Figure 1: water elevation profile of flooded junction to outlet 1 for zone 1sub-catchment.

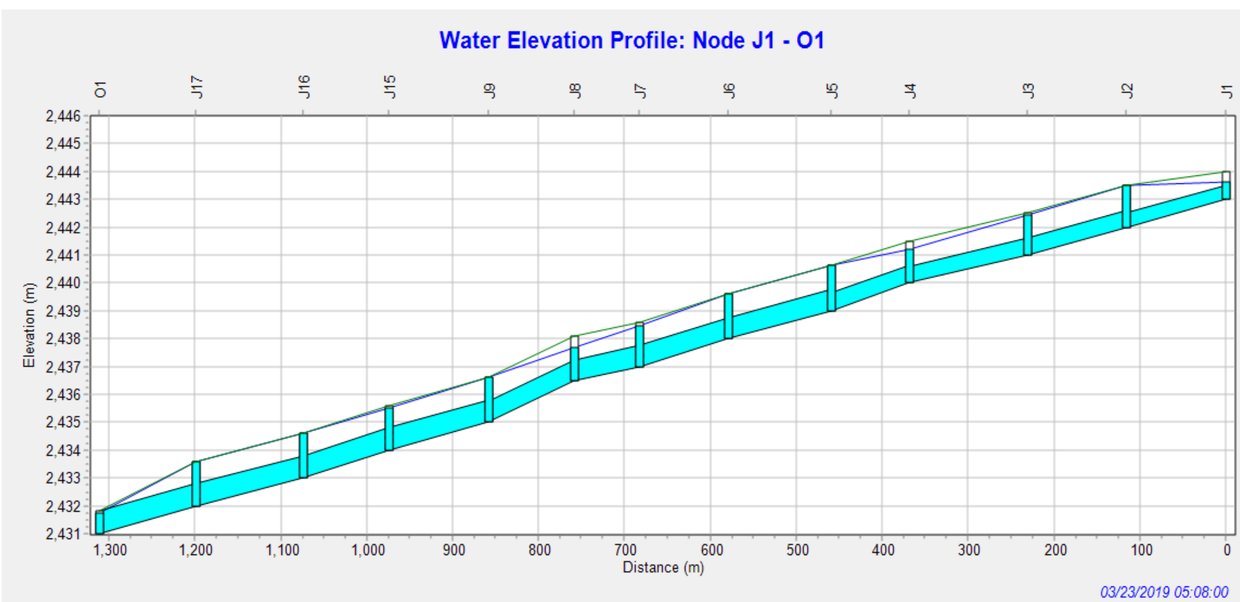


Figure 2: water elevation profile of flooded junction to outlet 2 for zone 2 sub-catchment.

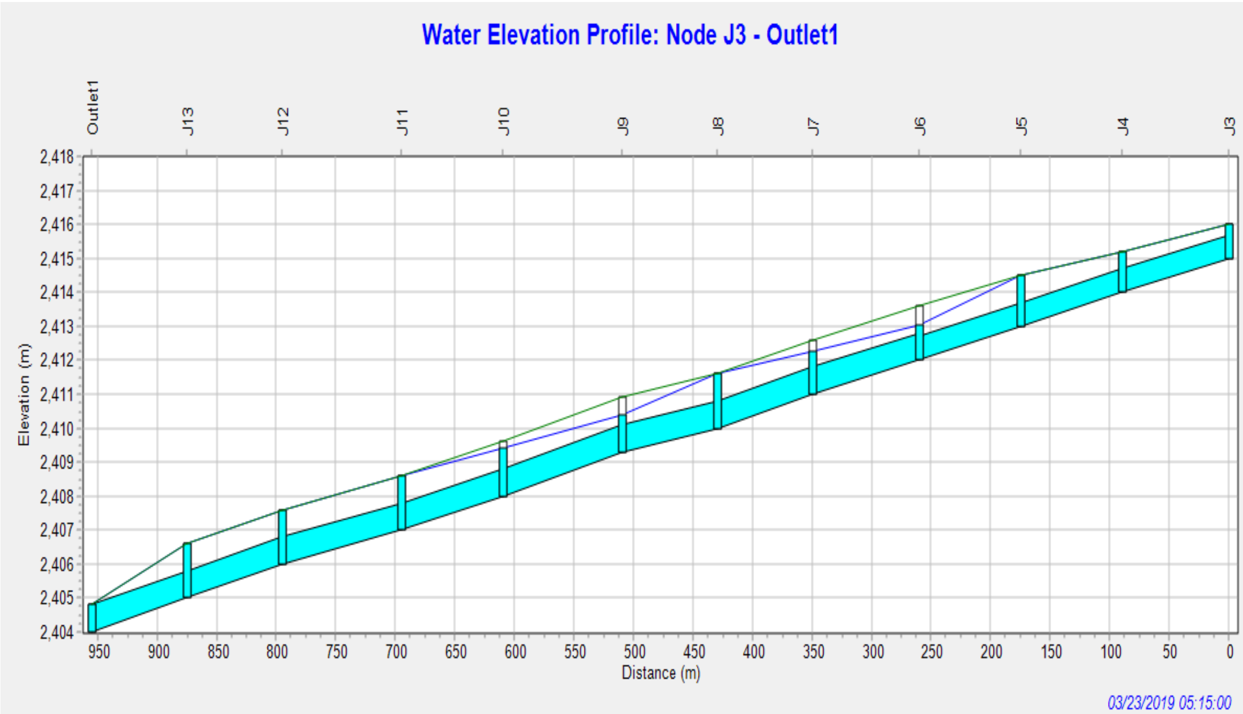


Figure 3: Water surface elevation profile of the flood junction to outlet 3 from zone 3 sub-catchment.