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Assessment and Modeling of Urban Drainage System Performance (A Case Study of Mojo Town)

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

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ABSTRACT

The drainage system is one of the most important factors to be considered in the urban drainage infrastructure plan. Now inadequate urban stormwater drainage problems represent one of the most common sources of compliant numerous towns of Ethiopia. Drainage problem in mojo town the worst issue of over flooding on the surface runoff at high rainfall season. This difficulty occurred due to inadequate drainage network infrastructure and the increase in the urban pavement on most urban sub-catchments. The objective of this study to identify the critical condition and related problem with a drainage system, assess the hydraulic performance of stormwater drainage infrastructures, and to evaluate alternatives for drainage problem mitigation measures are achieving this specific objective SWMM5.1 model and LID control was used in the study. The calibration and validation of the SWMM5.1 model well-done and its performance was tested by the goodness of fit using the coefficient of determination (R^2) =0.97, the Nash –Sutcliffe coefficient (NSE) =0.96, and Relative error (RE) =11.16%. The simulated area of the study subdivide to four outlets of the total area of sub-catchment is 745.2ha joint to drainage system infrastructure of 93 nodes, 101 channels, and one divider are simulated by SWMM 5.1 model. From model result greater than 60% of drainage infrastructure is flooded, at the outfall total sub-catchment runoff is 6.88m^3/s average flow, 22.44m^3/s maximum flow and 583.07*10^3 m^3 total volume of all outfall. The LID control used in this study for reduced the peak runoff overflow and select the best alternative on the outlet one of the project in the same sub-catchment S1, S2, S3, S5, S6, S9, S10, and S12 were the reduced the total outlet one volume by 28.12% using bio-retention Cell scenario, 25.9% infiltration trench scenario, 26.71% using permeable pavement scenario and also on sub-catchment reduced total volume outfall one reduced by 3.02 % using vegetation swale scenario. All scenarios used two of the highest value of runoff decreased by Bio-retention Cell scenario and permeable pavement for total study applied. by using two selected Lid control total volume of all outfall from 583.07*10^3 m^3 is reduced to 425.89*10^3 m^3 by 36.92% from the total study. Generally, the mojo town urban drainage system performance infrastructures overflow assessed and controlled for the problem occurred using the best alternative mitigation measures by improved LID control for all study areas.

Keywords:-Drainage systems, SWMM5.1, performance, LID and alternative mitigation

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF TABLES IN APPENDIX

LIST OF ABBREVIATION

AAIT---------------------Addis Ababa Institute of Technology

AASHTO------------------American Association of State Highway and Transportation Official

ASTM--------------------American Society for Testing and Material

BMP-------------------------Best Management Practice

DEM------------------------Digital Elevation Model

EPA-------------------------Environmental Protection Agency

ERA-------------------------Ethiopian Road Authority

GIS--------------------------Geographical Information System

IDF-------------------------- Intensity duration frequency

LID------------------------Low Impact Development

OUPI-----------------------Oromia urban planning institute

SWMM 5.1------------------Storm water management model version 5.1

SUDS-----------------------Sustainable Urban Drainage System

CHAPTER 1 INTRODUCTION

1.1 General Background

Urban Drainage system is one of the most significant factors to be considered in the urban road design and other infrastructure. Drainage overflow is affected due to the expansion of urban infrastructure, without concern of drainage characteristics and an increase in paved surfaces. Such happenings effect can be minimized through modeling of flood events and preparing action plans.(Rangari V., 2018)

Drainage difficulties in urban areas introduce flooding, decline of roads, land degradation, sedimentation, waterlogging and etc. With growth an urban increase in impervious surfaces, the drainage pattern changes, the overland flow gets faster, flooding and it becomes a critical problem facing the existing and future road infrastructure.(Manual P.D., 2002)

Insufficient urban stormwater drainage complications represent one of the greatest common sources of compliant from the citizens in many towns of Ethiopia and this problem is getting worse and worse with the current high rate of urbanization in different parts of the country.

Stormwater discharges are produced when the capacity of the land to retain precipitation is exceeded and run-off occurs. A run-off will be influenced by rainfall and intensity (millimeter of rainfall per hour)) and duration, antecedent storms and a number of watersheds, and land use characteristics such as slope, soil type, and impervious surfaces.(Biniyam A., 2016)

Drainage design for highway facilities must strive to maintain compatibility and minimize interference with existing drainage patterns, and control flooding of the roadway surface for design flood events. Due to inadequate integration between the road and urban stormwater drainage infrastructure facilities, many areas are exposed to flooding problems and road damages in urban roads. (Belete., 2011).

1.1.1 Background of Drainage in Mojo City

A river and its tributaries drain a drainage base Mojo town. This drainage boundary is marked by a ridge of highland beyond which any precipitation will drain into adjacent basins. The town is located within the Awash River drainage basin. In the town, the runoff from the upper part of Kolba and nearby plateau toward the town is draining through natural and human-made drainage canals.

1.2 Problem Statements

When observed study place of drainage systems problem such as overflowing on the road surface, eroding the highway at the overflowing area, and taking water in open ditches or on the surface at rainfall season. This problem may due to not adequate drainage ditches, the network of the canal's infrastructure is not well connected, an incremental of rainfall, the increase of pavement in overall the catchments, or an increase in urbanization and in most flat areas the water stagnation is the other critical problem during the rainy season.

The following figure 1.1, 1.2 1.3 and 1.4 are taken from August 15/2019 to September 8/2019 shows that the current problems of Town.

Figure 1-1 The drainage system problems on infrastructures

The figure 1.1 express that Left side the drainage structure for not constructed the abutment and asphalt road scoring. On the right side the network of drainage is not a function and water-logging happens due to this nearby site community compline on the problems.

Figure 1-2 The drainage system problems on infrastructures

At the figure 1.2 shows flooding over asphalt road that overcrowding the transport service, overflowing runoff on the sidewalk and marketing area, silt deposited at service road and destructive the marketplace. Due to this, the consumers of the market and shopping on this road side do not work service at every rain season time.

Figure 1-3 Silt loading and road failures by flood

The figure 1.3 shows that flooding on roads area that not traveling spatially for the small vehicle to surface ground and the road destroyed and the right side image the road is filled by sediment that decreasing the transport service. This both side road not only the problem of town it also problem of drainage on the road the crossing country highway of Mojo to Addis Ababa and Hawasa.

Figure 1-4 Water logging under bridge and flooding on the highway

The left side figure 1.4 problem is the waterlogging under the railway bridge and on the highway and also on the Side walkway that may due to insufficient drainage canal or surface slope flooded. The right side also shows that overflowing the runoff on the highway and scoring material and losing the strength of the road. On this road the contribution a large number of vehicle movements enter into and exist through the town is high. The absence of adequate roadside channels in Mojo town the roads that from Addis Ababa, Adama, Ejere, and Hawasa highway with four direction part of the country passes through the town is overflowing, storming water, and flooding are surveyed during this study.

Therefore Due to a lack of management in urban stormwater there is a problem of drainage flooding of the Study area during intensive rainfall. To overcome these problems the management has to reduce the amount of runoff entering into the drainage system. Recently, Mojo town road is facing broad water logging during the rainy season as result of this serious problem of drainage systems need to solve through answering the following objectives.

1.3 Objectives of the study

1.3.1 General Objective

The overall objective of the study is to assess the performance of drainage system of Mojo town with possible mitigation measure.

1.3.2 Specific Objectives

- To identify the critical condition and related problem with drainage system
- To assess the hydraulic performance of storm water drainage infrastructures
- To evaluate alternatives for drainage problem mitigation measures

1.4 Research Questions

This research was conducted in line with the following question to be answered:

- What is the critical conditions and related problem of urban drainage system?
- What is the hydraulic performance of storm water drainage infrastructures?
- What are the alternatives for drainage problem and mitigation measures?

1.5 Importance of the Study

The importance this study also may help in filling the gaps by identifying problems to Sustainability, taking proper modeling of the Stormwater drainage system, and proper functioning of Drainage schemes in the town. The Concerned body and organizations working in the area can use it as a reference for proper design, implementation and maintenance of urban drainage system. Generally mojo town is the part of which drainage problem is overflow, for this result further analysis where involve the hydraulic performance of storm water drainage problem and sustainable drainage system evaluations for future use in the town.

1.6 Scope and limitation of the Study

The scope of study is really on assessing and modeling of drainage system performance using SWMM tool to compare the results with actual measurement for calibrating the two results and also for sustainability in Drainage schemes that contribute for better services of mojo town drainage system performance. The runoff from each sub catchments were modeled and flood routing was done through drainage canals and nodes. Also the drainage networks were simulated by considering each sub catchments.

The study is limited to the critical and related problem of drainage system in the mojo town which not includes the agricultural area of the town. This study also does not include detail structural design of drainage infrastructures except modeling the type and size of essential drainage structures.

1.7 Outline of the Research Study

The study structure is dependent into five chapters. First chapter introduction or background with related study, statement of the problem, objectives of study, questioners of study, significance of study, scope and limitation of study. Second chapter review the literatures that related to study with storm water and drainage system performance. Third chapter material and methodology used that start from the description of the study area, material used for study, methodology used to address the objectives. Forth chapter contents are result and discussion parts. Fifth chapter conclusion and recommendation with the possible mitigation measures and the last reference and appendix are included in this study.

CHAPTER 2 LITERATURE REVIEW

2.1 Theory of urban drainage system development

Urban drainage was firmly established as a vital public works system in the early parts of the twentieth century. Engineers continued to improve design concepts and methods. During the second half of the twentieth century regulatory elements were spread in the United States, Europe, and other locations addressing urban drainage issues. Computer modeling tools advanced the methods used to design and analyze urban drainage systems. (Zhou Q., 2014).

Communities worldwide are yet searching for innovative techniques to capture, detain, and use rainwater within the watershed instead of constructing massive drainage structures. Urban drainage has indeed expanded significantly during the past few decades beyond a technical challenge to drain the urban area rapidly to include the consideration of social, economic, political, environmental, and regulatory factors. (Roesner, L.A., 2010)

2.2 Stormwater and urban drainage system

2.2.1 Stormwater

Stormwater is the water draining off a site from the rain that falls on the land and everything it carries with it. in an urban area, stormwater is generated by rain runoff from the roof, roads, driveways, footpath and other impervious or hard surfaces (Ericson T., 2010)

2.2.2 Stormwater management

Strom water management is important in consideration of modeling the urban drainage system. Stormwater management practices, when properly selected, modeling, and applied, can be utilized to mitigate the adverse hydrologic and hydraulic impacts caused by drainage facilities, thus protecting downstream areas from flooding and erosion. Present downstream conveyance constraints, particularly in cases where the roadway drainage system connects to existing drainage systems, may limit the peak discharge to the capacity of the downstream system.

Stormwater management, whether structural or non-structural, on or off-site, must fit into the natural environment, be functional, safe, and aesthetically acceptable. Revegetation with native, non-invasive grasses, shrubs, and possibly trees may be required to achieve compatibility with the surrounding Urban Storm Water. (Manual, P.D., 2002)

The stormwater results from all kind of precipitation (snowmelt, rainfall, etc...) and comprises the water flowing in the surface Therefore, the characteristics of both the rainfall and the catchment area represent important factors in the stormwater properties. Indeed, part of the water of the rainfall goes to initial losses as interception, depression storage, infiltration, and evapotranspiration. The remaining water is the runoff. (Davis A,P., 2009)

2.2.3 Storm water drainage system

Stormwater drainage is the process of draining excess water from streets, sidewalks, roofs, buildings, and other areas. The systems used to drain stormwater are often referred to as storm drains.

Many cities and towns have carefully planned stormwater drainage systems that consist of inlets, outlets, and pipes. The inlets of storm drains are often covered by protective grates that help to ensure that large items don't fall in while water can enter freely. Since it's important for large amounts of water to flow into these drains, the bars of the grates must be spaced some distance from each other. This concession means that some smaller objects do fall into the drain.

2.2.4 Functions of storm water drainage system

The drainage system's functions are to collect surface water and/or groundwater and direct it away, thereby keeping the strength of the bed drained. The drainage system must also protect the substructure from erosion, Public health and safety, Environmental protection, and Sustainable development. Additionally, the impact of drain and sewer systems on the receiving waters shall meet the requirements of any national or local regulations or the relevant authority. (Manual, P.D., 2002).

2.3 Urban drainage system

City drainage structures have been considered a dynamic natural resource, leftover transport medium, and a flooding difficulty. climate, topography, geology, clinical knowledge, engineering and construction talents, not unusual values, and other elements have subjective the local evaluation of urban drainage for constructing cities those elements have guided and restrained the improvement of urban drainage solutions. Historic accounts offer attractions of many thrilling and particular urban drainage techniques. (Biniyam A., 2016).

2.3.1 Sustainable urban drainage system

The indication with Sustainable Urban Drainage Systems (SUDS) is that in the best potential way redevelop the natural system of stormwater handling, in order to reduce peak flows and deliver treatment for the stormwater on its way to the recipients. The consequences of high peak flows, which arrive quickly after the storm starts. Since the traditional pipe-systems in the cities normally aren´t designed to handle these infrequent peak flows, flooding is often the result. With the introduction of SUDS, the water will be late on its" way downstream, in similarity to nature's way of handling stormwater runoff (ERA Storm et al., 2013).

2.3.2 Component of the drainage system

A whole storm drainage gadget layout consists of the attention of both the most important and minor drainage structures. The minor device consists of curbs, gutters, ditches, inlets, get right of entry to holes, pipes, and different conduits, open channels, pumps, detention basins, water quality manipulate facilities, and so forth. The minor device is generally designed to carry runoff from 10-yr frequency storm activities.

The fundamental machine gives an overland remedy for stormwater flows exceeding the ability of the minor gadget. This generally takes place at some stage in extra infrequent storm activities, including the 25-, 50-, and one hundred-year storm. The essential gadget consists of pathways that can be supplied – knowingly or unknowingly -for the runoff to drift to herbal or artificial receiving channels which include streams, creeks, or rivers.

Usually, storm drainage design efforts have focused on components of the minor system with little attention being paid to the major system. Although the more significant design effort is still focused on the minor system, lack of attention to the supplementary functioning of the major storm drainage system is no longer acceptable. (Roesner V.A., 2010)

2.4 Flooding

Flooding, by its very nature, is often a result of each meteorological and hydrologic strategies; the character of a flood is determined each with the aid of the exact conduct of the precipitation and with the aid of the nature of the situation in which the occasion is probably to arise (soil situations, quantity of antecedent rainfall, and so on). It isn't probable that exactly unique forecasts of flooding events will ever be possible, even though it is honestly well inside our capability to assume the opportunity of most flood occasion.

Greater rainfall, or effective rainfall, is that rainfall is neither retained at the land surface nor inflated into the soil. After flowing throughout the watershed floor, excess rainfall will become direct runoff on the watershed outlet below the assumption of Hononian overland flow. (Ven Te Chow., 1988)

2.4.1 Factors Affecting Flood Runoff

For all hydrologic analyses, the following factors will be evaluated and included if they have an enormous impact on the final outcomes:-

- Drainage basin characteristics including length, shape, slope, land use, geology, soil kind, surface infiltration, and storage;
- movement channel characteristics including geometry and configuration, natural and artificial controls, channel modification, aggradation/degradation, and particles;
- Flood plain characteristics; and
- Meteorological characteristics along with precipitation amounts and sort (rain, hail, or combinations thereof), hurricane mobile size and distribution traits, storm route, and time charge of precipitation.(Authority ERA., 2012)

2.5 Hydraulic Modeling and Design

Urban flooding is two vital hydraulic ideas that arise: surcharge and backflow. A drainage gadget is surcharged while its potential is passed, i.e. it gets extra extent of water than the gadget can carry. As an end result, the water stage rises upstream due to the community overloading (Ericson, 2010) if the power line reaches higher points downstream than upstream, water may alternate the normal waft direction leading to backflow.

Many method-based hydrological models, inclusive of SWMM, have the functionality to generate rainfall and, thereafter, floor runoff, on each day, month-to-month, and annual bases. An ok method to calibrate and validate the SWMM modeling is an essential problem. A version ought to be tested appropriately earlier than using it to broaden effective watershed management plans, especially if generated rainfall is the fundamental input. This examines turned into undertaken to estimate runoff of a watershed and evaluation and modeling on city drainage machine performance development in mojo town the use of the SWMM5.1 model.

The weather variable needed for a maximum of the hydrological models consists of precipitation, maximum and minimum temperatures, solar radiation, and different. (Hassen N., 2016)

2.5.1 Flood frequency Analysis

The hydrologic simulation models the rainfall-runoff relationship within the basin, it could also be used to test the validity of the probabilistic distribution selected for gauged unregulated watersheds with long stream go with the flow information. Meteorological data inside the have a look at watersheds increase from15 to 30years. stream go with the flow statistics, if to be had in any respect, are frequently a whole lot shorter and most of the watersheds are ungauged non-stop hydrologic simulations can use the located metrological statistics to be had to extend the prevailing stream waft document form some years to 20 or 25 years.25 years.

The calculated runoff intensity above positive return intervals greatly exceeds the precipitation for the identical return duration. for the reason that continuous hydraulic simulation keeps a continuous accounting of soil moisture, it gives a completely unique tool to analyze the complicated dating between frequencies of precipitation, soil moisture, and runoff. The fundamental requirement for any watershed hydrology version is its capability to estimate floor runoff thoroughly because it influences the improvement of hydraulic structures. Correct simulation of yield from a watershed is pretty vital, as it is often required to determine the magnitude of flood from the watershed. (ERA et al., 2012)

2.5.2 Hydraulic capacity

The hydraulic capacity of a storm drain is managed by its length, form, slope, and friction resistance. Numerous glide friction formulas have been superior which outlines the connection among flow capability and these parameters. The maximum extensively used components for gravity and stress flow in storm drains is Manning's Equation. (Guo and Mackenzie., 2005)

2.6 Model Justification

SWMM (U.S. Environmental Protection Agency., 1992) is a comprehensive computer model for simulation of urban runoff quantity and quality in storm and combined sewer systems. SWMM 5.1 stands for Storm Water Management Model version 5.1. All aspects of the city hydrologic and such as floor runoff, shipping via the drainage community, and storage. Like most hydrologic fashions, SWMM5 subdivides the general catchment into sub-catchments, predicting runoff from Sub catchments on the basis of their individual properties, and combining their outflows using a float routing scheme. SWMM5 can also simulate backwater outcomes. In SWMM5, sub-catchments are represented mathematically as spatially lumped, nonlinear reservoirs and their outflows are routed via the channel/pipe. Sub catchments are subdivided into 3 subareas, impervious area with and without despair storage, and pervious areas with depression garage. Go with the flow from one subarea isn't always routed over another subarea. Overland waft is generated from each of the three subareas by way of approximating them as nonlinear reservoirs. This nonlinear reservoir is hooked up by means of combining the continuity equation with Manning's equation. Infiltration from pervious regions can be computed through both Horton and inexperienced-Ampt equation. Float routing in channel/pipes is also carried out via a nonlinear reservoir by combining the continuity equation with Manning's equation. (EPA States., 2015).

2.6.1 Selection criteria of SWMM model

There are various number of techniques for evaluation of storm water runoff on bases of water balance equation, empirical equation and viable models like STORM model, CIVIL CAD, RAFTS, MOUSE and SWAT by such strategies calculations for penetration, overflow of surface, routing of flow, and slacking of surface overflow have been re arranged to permit simulation of flow with a hydraulic and Hydrologic Simulation Program For each one of those number of techniques they have some limitations fixing urban drainage systems networks. (Sidek L, Desa M, Basri N& Tenaga U., 2011)

2.6.1.1 STORM Model

The U.S. Army Corps of Engineers (1977) developed Storm water Runoff Model (STORM) to analyze quantity and quality of runoff from urban and nonurban catchments. STORM was primarily developed to evaluate the storm water storage and treatment capacity required to reduce untreated overflows below specified values. Computations of treatment, storage and overflow proceed in an hourly basis by simple runoff volume and pollutant mass balance for the entire catchment. Since this model runs on hourly time step, this model is not suitable for small catchments where time of concentration is less than one hour. STORM is a continuous simulation model. This model is basically a planning model and therefore, not suitable for detailed quantity or quality modeling.

2.6.1.2 MOUSE

MOUSE (Danish Hydraulic Institute, 1988) stands for Modeling of Urban Sewers and is a hydrologic-hydraulic model applicable only for modeling of urban catchments. This model is used extensively for sewerage design in Australia compared to the design of stormwater drainage networks (Lindberg and Car, 1992). The hydrologic part of the model deals with simulation of runoff using two methods: a simple method based on time-area diagram and a complex method based on kinematic wave theory and continuity equation.

2.6.1.3 CIVILCAD

CIVILCAD (Surveying and Engineering Software, 1997) is a multipurpose design computer package. It was mainly a design tool for road design, although it provides facilities for drainage design. However, this package is rarely used only for drainage design by city/shire councils in Victoria (personal communication with R. Silva, Buloke Shire Council, Victoria, 1999).

2.6.1.4 RAFTS

RAFTS can be used in event or continuous mode, with appropriate rainfall inputs. Like most rainfall-runoff models, RAFTS requires the catchment to be sub-divided into several sub catchments. Each sub catchment is then divided into 10 subareas within RAFTS based on lines of equal travel time or isochrones. Runoff from each subarea is routed using the Laurenson's (1964) runoff routing procedure to obtain the outflow hydrograph of a Sub catchment. RAFTS can model pervious and impervious areas separately. However, it does not consider directly connected impervious area and supplementary area separately as in ILSAX and SWMM. RAFTS use initial loss-continuing loss model or Philip's infiltration equation to simulate the excess runoff. Pipe flow is determined using Manning's equation. Overflow is computed as the portion of the total sub catchment inflow, which cannot flow through the pipe because of inadequate capacity. Pit inlet capacity restriction is not considered in this model. For flood routing through pipes and trunk drainage system, the Muskingum procedure is used. As an alternative to channel routing where physical data is lacking.

2.6.1.5 WBNM

The WBNM (Boyd et al., 2000) model is an event based nonlinear runoff routing model, capable of modeling runoff from small and large catchments. In WBNM, a catchment is divided into a number of sub catchments and is represented by a separate storage element. Each urbanized sub catchment is divided into pervious and impervious subareas, with separate rainfall losses to compute the rainfall excess. Five alternative loss models (i.e. initial loss-constant loss rate, initial loss-loss rates varying in steps, initial loss–runoff proportion, Horton continually varying loss rate and Green-Ampt varying loss) are available in WBNM to model rainfall losses. Overland flow in each sub catchment is modeled by a nonlinear reservoir with time-lag.

EP SWMM5 is utilized in urban areas flood modeling and analyses to fix the drainage size by way of considering the pervious and impervious areas. So that modeling in urban drained the **SWMM** software program is more **comfortable** for **Mojo** drainage systems Hydraulic and Hydrologic Simulation program for this study objectives, the SWMM5 has been favored for city runoff estimation.(Kong F., 2017)

2.6.2 Model calibration and validation

An adequate procedure to calibrate and validate the SWMM modeling is an important issue. A model should be tested adequately before using it to develop effective watershed management modeled.(Weaver H and Nachabe M., 2018)

SWMM changed into used, adapted, and calibrated for the Bellona Creek Watershed, a catchment in Southern California. A geographic statistics system (GIS) turned into used to system the enter facts and generate the spatial distribution of precipitation. Catchment became delineated with the aid of 1579 catchments, 2648 channels, and over 263 km long pipes

Calibration and validation of urban drainage infrastructure consideration and it becomes demonstrated in our country in Jimma and Debere Berhan town. (Warati G, Demissie T., 2015 and Eyosias B.,2018)

2.7 Low Impact Development (LID) controls

Basically, LID is a land re-improvement technique to manage stormwater. the principle intention of LID is to lessen the bad effects of precipitation flooding waters through maintaining the predevelopment hydrology of a site through decentralizing micro-scale controls. .LID practices successfully reduce water-related problems via infiltration and evaporation of the typhoon water resulting in environmental, social, and economic blessings. The common LID practices are bio-retention, inexperienced roofs, permeable pavements, rain gardens, vegetative swales, and rain containers that are used to create a functionally equal hydrologic landscape. (Kong F., 2017)

2.7.1 Selection criteria of LID techniques

A range of things to be considered while choosing Suds such that the maximum suitable method is followed and SUDS are successfully applied. Such elements encompass website online suitability, available land area, value, maintenance troubles, and network popularity. (Square P and Place E., 2010)

Overland flow from LID controls can be modeled in three ways. (Eyosias B., 2018)
The first approach is to route impervious sub catchment to pervious sub catchment to receiving node as pervious area properties are to be matched to LID control design. The pervious area of the sub catchment acts as LID control. This approach is not realistic and does not give accurate Results.
The second approach is to create LID sub catchment as a separate sub catchment and to route the original sub catchment to the LID sub catchment to receiving node. The LID design is to be matched to sub catchment properties. LID area is to be extracted from Original pervious or impervious area.
The last approach is to create LID as part of original sub catchment and to route runoff through LID prior to receiving node from LID area is to be added to original pervious or impervious area. (USEPA., 2000)
If multiple LID units are placed in a sub catchment, then the LID units take the impervious area runoff of a sub catchment. Different capture ratios can be given to different LID units. The options for routing the surface flow and underdrain flow of the LID units are as follows: a) both surface overflow and underdrain flow is routed to the sub catchment's outlet; and b) Underdrain flow can be routed to a separate outlet other than its sub catchment pervious/impervious area.(USEPA., 2000)
Sub catchment of Mojo town are contain both pervious and impervious and no area used separately for LID be for receiving node to control the stormwater. because of this the area were LID control applied are considered as part of original subcatchment.so the second alternative is selected to route runoff through LID prior to receiving node .

2.7.1.1 Permeable Pavements

Permeable pavement is an opportunity to standard paving in which water filters thru the paved structure in place of going for walks off it.each floor and the sub-grade need to be designed with this feature in thoughts. Water may be allowed to infiltrate directly into the subsoil where conditions are suitable. As a substitute, it is able to be held in a reservoir shape beneath the paving to be used once more, infiltration, or late discharge. The permeable paving may be crafted from materials inclusive of gravel, grass Crete, concrete blocks designed for the cause, or porous asphalt.(Ericson T., 2010)

Figure 2-1: Permeable pavements method of structure view

2.7.1.2 Infiltration Trench

An infiltration trench is a shallow, excavated trench that has been connected with a geotextile and backfilled with stone to create an underground reservoir. Stormwater runoff flowing into the ditch regularly infiltrates into the subsoil. An overflow can be required for extreme rainfalls that exceed the potential of the reservoir. The overall performance of the trench relies largely on the permeability of the soil and the depth to the water table. Infiltration trenches generally serve small catchment regions up to 2-3 hectares in commonplace with other source control strategies. The closer they may be to the source of the runoff the extra powerful they will be. The operational existence of the trench may be enhanced by means of presenting pre-remedy for the inflow, which includes a filter strip, gully, or sump pit, to dispose of immoderate solids. Everyday maintenance may be required for maximum pretreatment designs.

Figure 2-2: Infiltration trench structural view on side road

2.7.1.3 Bio retention

Bio retention areas are answers which have few limits, and are well proper for retrofitting purposes in extremely-urban areas(Sidek L.M., 2011) According to study of (Laddimath R.S., 2016) the most ponding intensity for water is 0.2 meters above the extent of the soil. a further freeboard of at least 5 centimeters over the maximum intensity of the water needs to be added to the construction, consistent with the identical supply. On account that space is very limited inside the imperative areas of Xiamen, vertical aspects are supposed to preserve the ponding water within the bio-retention region. Brown, Stein, if local soil-situations are restricting, underdrains may be used to lead the water away from the bio-retention area.

Figure 2-3: The construction practice of bio retention method

Mojo town are contain both pervious and impervious and no area used separately for LID be for receiving node to control the storm water. Because of this the area were LID control applied are considered as part of original subcatchment.so applied the best alternative mitigation measure LID control for the town of sub catchment by increasing the infiltration runoff to soil and decrease the runoff from surface using the above alternative methods of LID technique.

Bio-retention cells, infiltration trenches, and permeable pavement systems can contain optional drain systems in their gravel storage beds to convey excess captured runoff off of the site and prevent the unit from flooding. They can also have an impermeable floor or liner that prevents any infiltration into the native soil from occurring. Infiltration trenches and permeable pavement systems can also be subjected to a decrease in hydraulic conductivity over time. Although some LID practices can also provide significant pollutant reduction benefits, at this time SWMM only models the reduction in runoff mass load resulting from the reduction in runoff flow volume.(EPA SWMM., 2015)

CHAPTER 3 MATERIALS AND METHODOLOGY

3.1 General Description of the Study Area

Mojo town is one of the pre-Italian towns of the country that owned its emergence to the Djibouti to Addis Ababa railway line. Which began as a settlement around the Mojo River, is believed to have been emerged around 1916/17 near mojo river from which the rail required water. The foundation and development of the town is closely related on Addis Ababa to Djibouti railway line. The town recognized as town since 1941 and it gained a municipal status in 1944. (Mojo master plan Study reports., 2014)

3.1.1 Geographical setting

3.1.1.1 Location and Area

Mojo Town is located in East Shewa zone which is one of the zones in the National Regional State of Oromia, in Lume district. It is located south east of Addis Ababa along the highway to Adama and Djibouti at 57 km. It is 28 km from zonal capital i.e. Adama town.

The coordination of town extends from 8° 32' 44" N to 8° 37' 58" N latitudes and 39° 05' 5" E to 39° 10' 38" E longitudes. In local coordinates the town is located 509,333 to 519,501meters Easting (X) Longitude and 944,621 to 954, 257 meters Northing (Y) Latitude.

The study area existing built up area of the town that is developed for different urban land use was about **5,108.1** hectare.

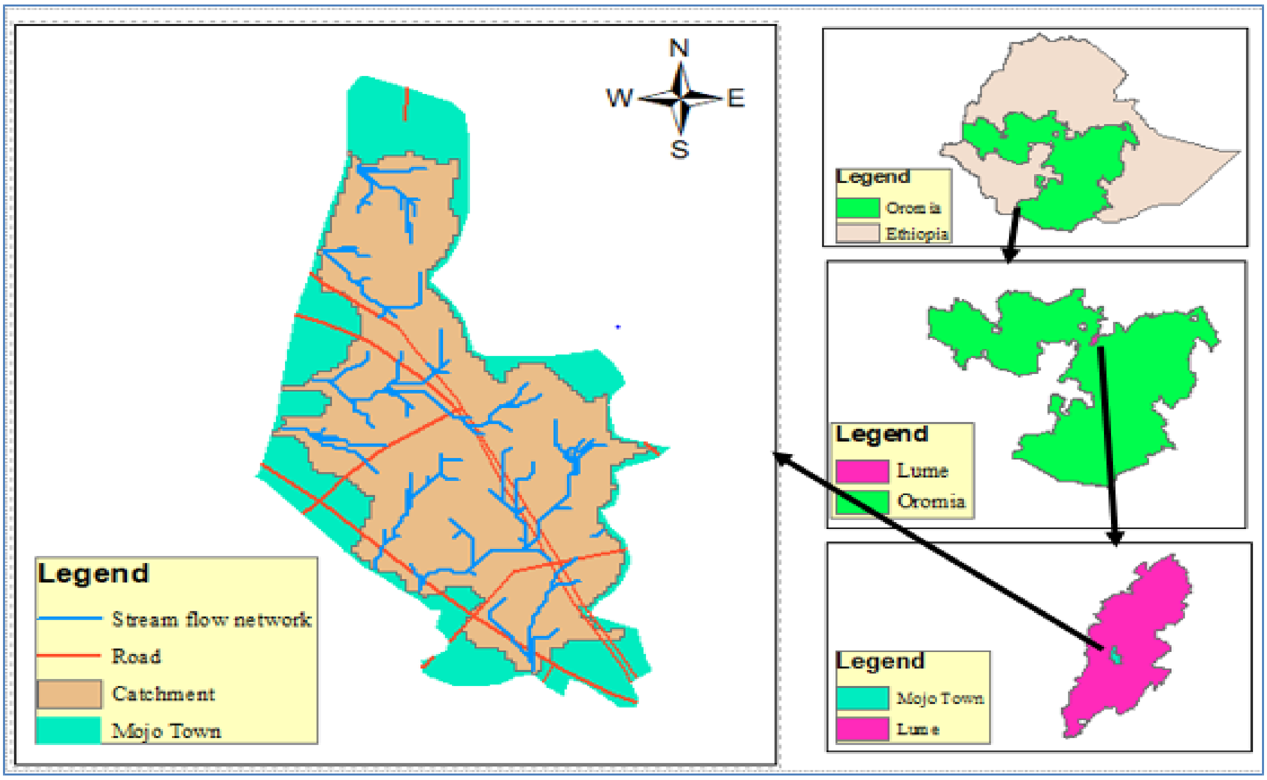


Figure 3-1: Location maps study area of Mojo town
(source by Arc GIS)

3.1.1.2 Topography

Different relief features we observe today in Ethiopia are the result of the geological events happened in different times. However, these relief features have been subjected to forces of erosion and weathering hence subjected to change through time. Mojo town is a part and parcel of the Great East African Rift Valley. The floor of the Rift Valley is an area of low elevation marked mainly by undulating and rolling surfaces.

A. Altitude/elevation

Mojo town lies on plateau and plain landscape within these landscapes there are also gullies and river course. Altitude analysis in the town indicates that elevation generally increases from west to east and from south to north and vice versa. The elevation ranges from **1,738.06 to 1,801** meters above sea level (m.a.s.l).

B. slope analysis

Slope analysis contributes for the selection of preferable landscape for urban expansion and determining different urban uses in the urban centers.

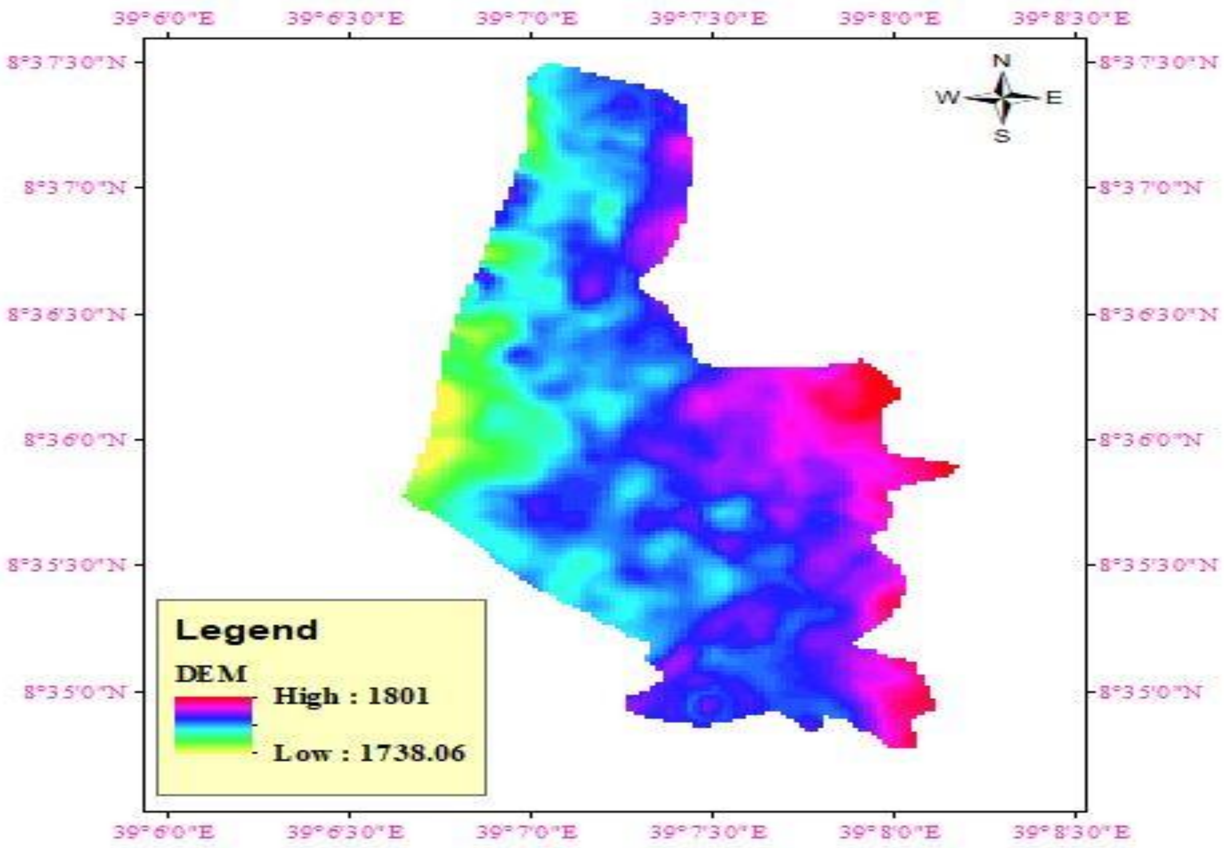


Figure 3-2: Digital elevation model (DEM) of mojo town
(Source: ARC GIS)

The slope of the town is analyzed using xyz coordinates through GIS tools. In the town excessively plain slope less than 1% occupies 12% of the total area. These are floodable areas so they are exclusive for different constructions and also they are exposed for storing stagnate water that may negatively affect the health and safe movement of the town dwellers. There is no excessive steep slope that hinders the development of urban infrastructures like road network in the town.

Table 3-1 Slope Classification and Area Covered

Slope classification (%)	m^2	Hectare	Percentage (%)
0-1	5,944,979.7	594.5	11.6
1-2	13,907,792.7	1,390.8	27.2
2-5	22,142,074.3	2,214.2	43.3
5-10	7,109,608.4	711.0	13.9
10-15	1,342,173.2	134.2	2.6
15-20	545,092.6	54.5	1.1
>20	88,993.8	8.9	0.2
Total	**51,080,714.7**	**5,108.1**	**100**

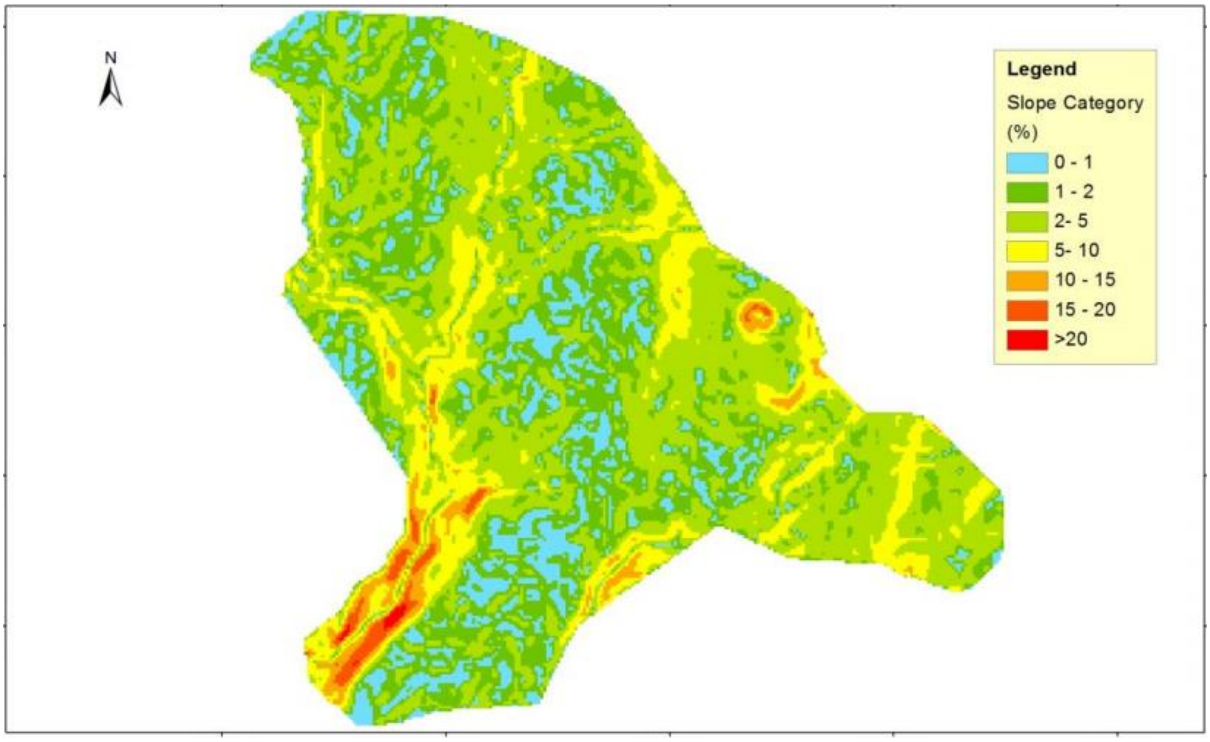


Figure 3-3 Slope distribution of Mojo town
(Source: Mojo master plan, 2014)

3.1.1.3 Climate

Climate is one of the physical factors that influence human activity especially in agricultural society through its impacts on soils and vegetation cover of an area. Climate consists of several elements. The major ones are temperature, rainfall, wind and pressure. Mojo town experiences sub-tropical climate. The general agro climatic condition of the town is categorized under **upper kola** climate.

A. Temperature

Ethiopia temperature distribution is affected to greater extent by variation in altitude than latitude. Accordingly, the mean annual temperature is about **26.5ºc** conventionally this is characteristic of tropical climate.

B. Precipitation

The mean annual rainfall is about **74.33mm** shown on table 3.2 below. The meteorological region A2 gauged station of precipitation data of the mojo town.

Table 3-2: Mean monthly rainfall of Mojo town

monthly	Jan	Feb	March	April	may	Jun	July	August	Sept	Oct	Nov	Dec
Rainfall mm	11.71	21.37	51.11	59.97	62.05	99.72	272.39	251.50	117.80	33.08	8.04	2.59

The distribution highest rainfall occurs during summer particularly from June and September. The smallest rain fall season starts from October to February.

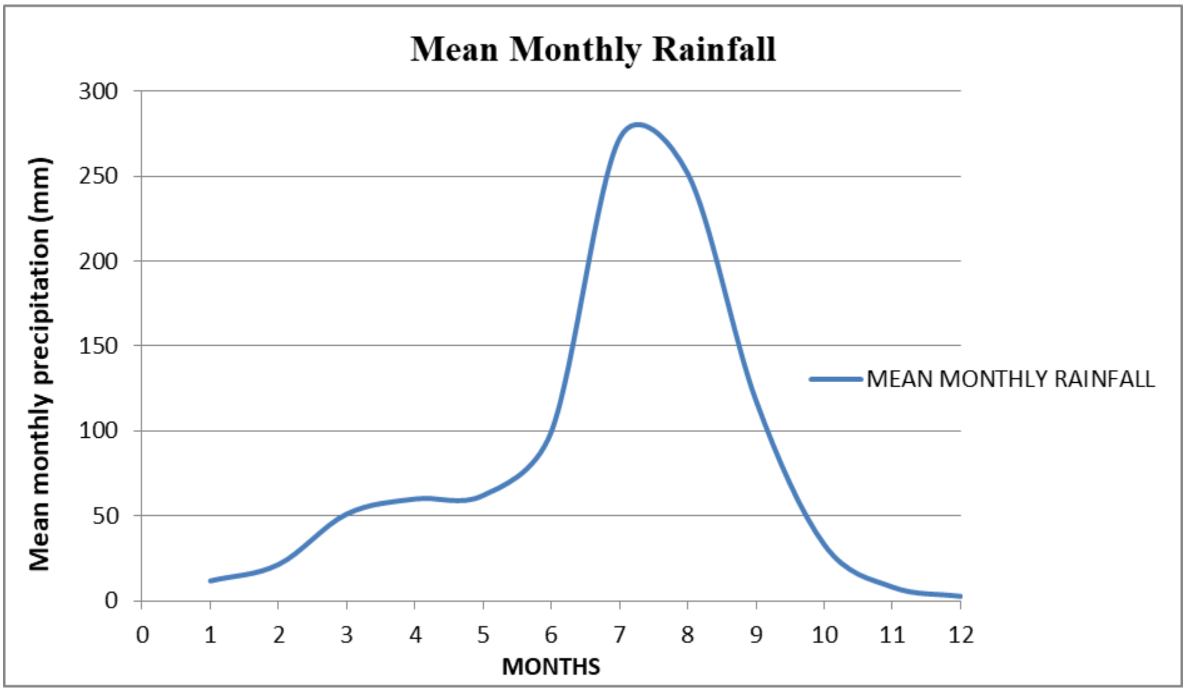


Figure 3-4 Average monthly rainfalls of Mojo town

C. Population

According to the recent (2011) Central Statistical Agency (CSA) projection, town population projection indicates **34,968** of which male and female accounts 16,989 and 17979 respectively

3.1.2 Land Use, Land Cover and soil type of the area

3.1.2.1 Land use land cover

The land use land cover data will be an essential input for the calculation of run off coefficient in the determination runoff by SWMM5.1 model.

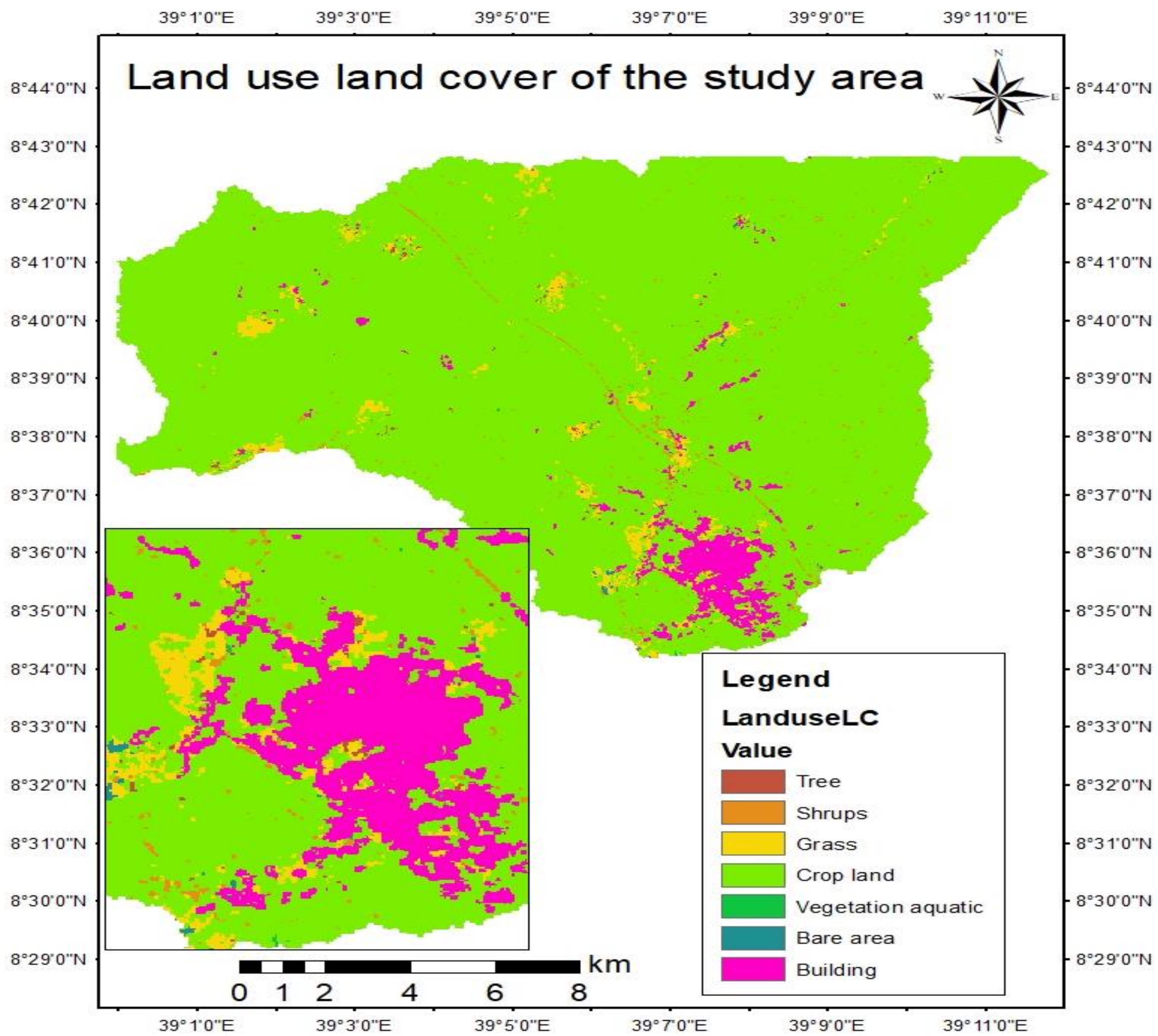


Figure 3-5: Land use land cover area of Mojo town neighboring

(Source, Arc GIS for 2019**)**

The natural vegetation surrounding the town is depleted and causes land degradation. The natural vegetation region is categorized as the Coniferous Forest Region where the climax vegetation are Junipers woodland and Pod carpus forest but these forests are found as manmade forest in some compounds of individuals, religious and government institutions. Planting these natural vegetation's are very important for rehabilitation of degraded land within and surrounding the town and this contributes for modifying micro climate of the area.

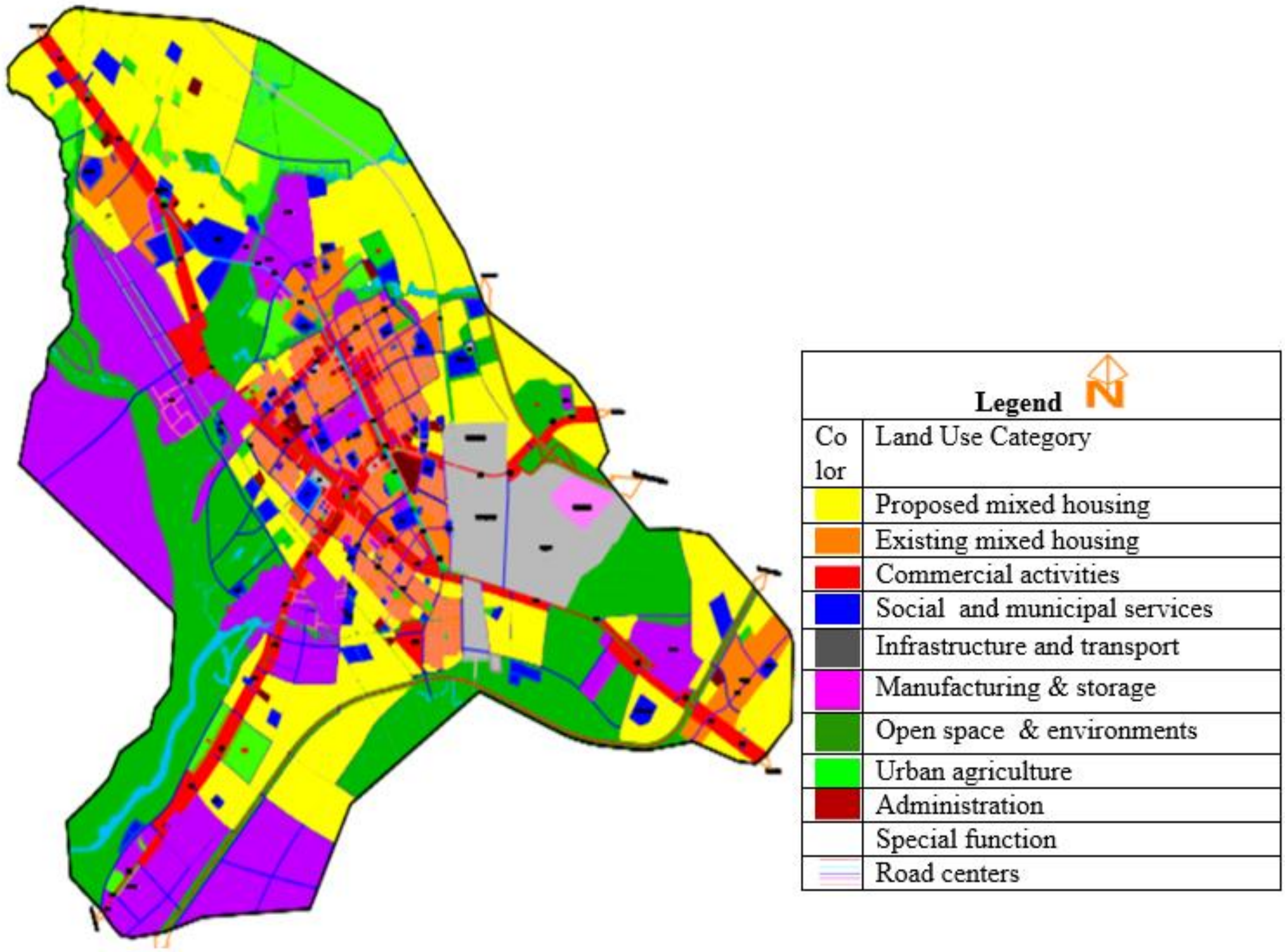


Figure 3-6: The current Mojo town structural Land use land cover
(Source: Oromia urban planning management, 2014)

3.1.2.2 Soil type

The development of soils depends primarily on geologic and climatic conditions. The FAO Soil Map of Ethiopia classifies 19 soil units.

According to FAO‟s soil classification (Region Study of Mojo), about 9 major types of soils are found in East Shewa Zone from that 3 type's soil classification hydrologic soil group D in mojo town as followed figure 3.8.

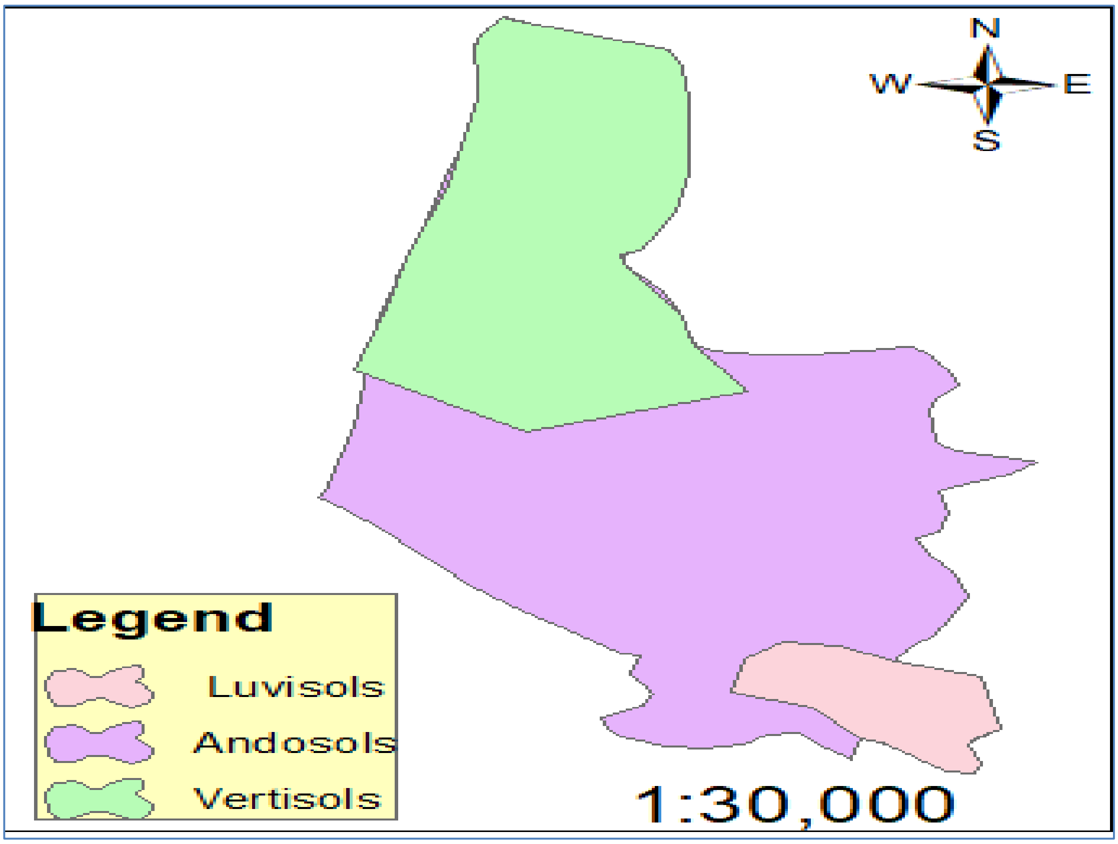


Figure 3-7 Types of soil classification in map
(Source: Arc GIS)

A. Andosols

From figure 3.8 Andosols are very fragile soil and thus get easily detached by rain droplets as well as Transportable by wind. On flat surfaces like the floor of the rift valley, they have limited agricultural value. Which accounting for about **57.46%** of the land area of the study.

B. Vertisols

Vertisols are mostly dark colored clay soils containing more than 30% clay content. They are heavenly cracking soils found in lume wareda mojo town with long dry season of gently sloping and large flat areas. Such soils are limited on agricultural potential, Land preparation is difficult because during the dry season they become hard, while during wet season. Vertisols cover the northern Lume and it makes up **35.84%**of study land surface.

C. Luvisols

Luvisols soils are formed in zone of relatively pronounced wet and dry season.Luvisols are mainly in the Lume districts of the Mojo town are caver **6.7%.** With high base saturation and high content of weather able minerals they have high agricultural potentiality.

3.1.3 Existing drainage networks of town

Mojo town and its vicinity is part of the areas which are found in awash valley. This means Mojo town and its surrounding are areas which are drained by Awash River and its tributaries. When we consider specifically the drainage pattern of Mojo town and its neighborhood type of drainage pattern is observed. Naturally this drainage pattern is developed more or less by chance on the present land surface and is not controlled by topographic features of the area. (Source mojo study report) The town and its neighborhood are found in Ethiopian Rift Valley zone.

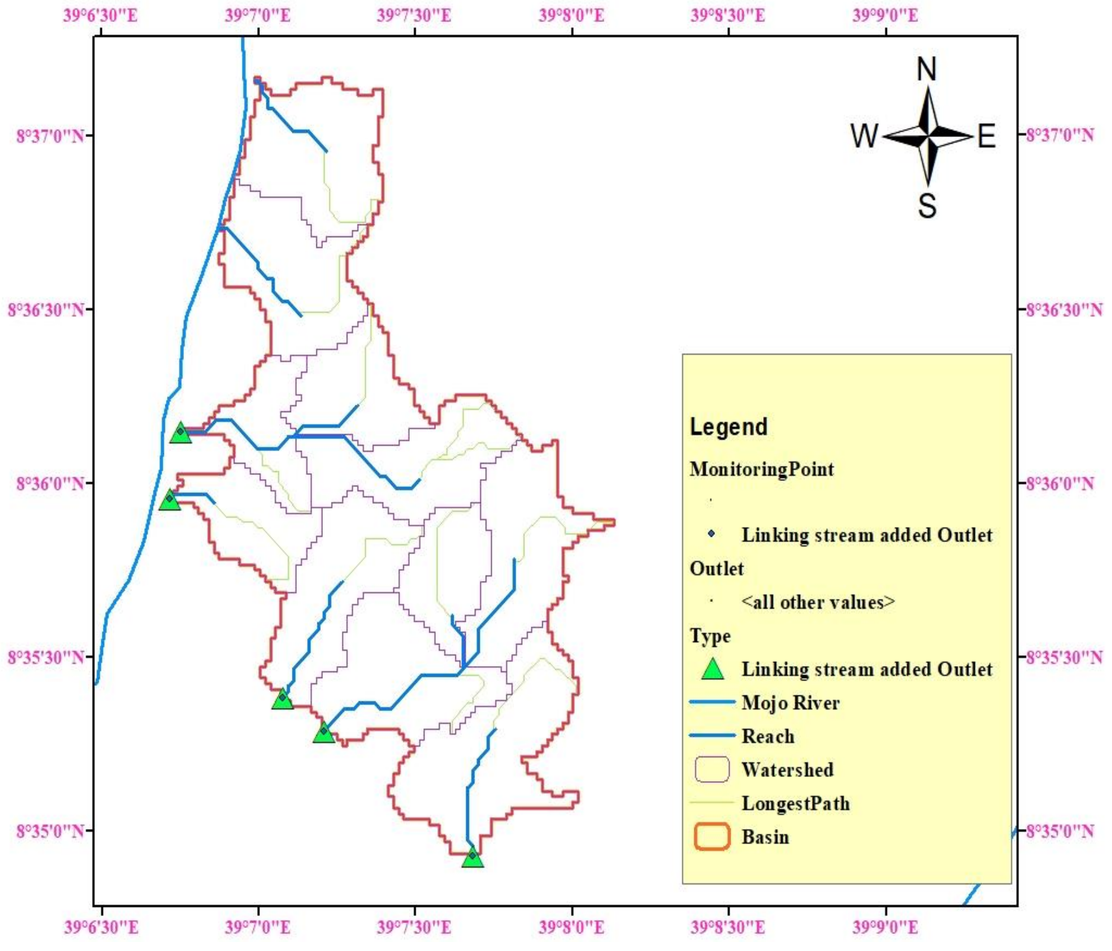


Figure 3-8: The existing natural drainage networks of Mojo town

(Source: Arc GIS)

3.2 Data collection and Material used

3.2.1 Primary Data collection

- Site Observation the overall of drainage system condition and booked camera.
- Measured the drainage dimensions using GPS and TAPE material for the input of swmm model simulation.(detail data appendix table 9 to 11)
- Measured flow depth in drainage system that used for calibration and validation of model.

3.2.2 Secondary Data collection

3.2.2.1 Hydrological data collection

A. Meteorological Data

- From the National Meteorological Services Agency collected weathered data of rainfall in daily from 1986 to 2017(32year) different stations of Mojo, Debrzite, Koka and Adama Observatory gauged station.

3.2.2.2 Catchment Data Sources

Topography and shape file

- From Ministry of water, Irrigation and energy use shape file of Awash River basin.
- from Oromia urban planning management office use shape file of mojo town and structural master plan

Soil maps

- From ministry of agriculture soil classification of Ethiopia
- From mojo master plan Scio economic report

Land use and land caver

- From Oromia urban planning management office use the land cover with delineated by Arc GIS and the Google earth maps views used for 2019.

3.2.3 Material used

All rainfall data recorded daily intervals were, collected from National meteorological Agency. The data recorded period was for 32 year (1986 – 2017 G.C). Generally the data collection and material used for each results of study used as follows:

- Shape file data is used as an input for ARC-GIS software for catchment delineation and estimation of catchment characteristic.

- ARC-GIS to obtain hydrological and physical parameters and spatial information of the catchments of the study area.
- Google Earth 2019 Software to verify water shed and divides of catchments of the study area.
- Hydrological and hydraulic data where used as input for SWMM tool
- GPS and TAPE meter to measure elevation of nodes and drainage demission that input for SWMM tool.
- Storm water management model to determine the peak runoff
- All the primary and secondary data organized used the study
- Referring different journals/thesis, books, design documents and manuals used as guideline

3.3 Method of the study

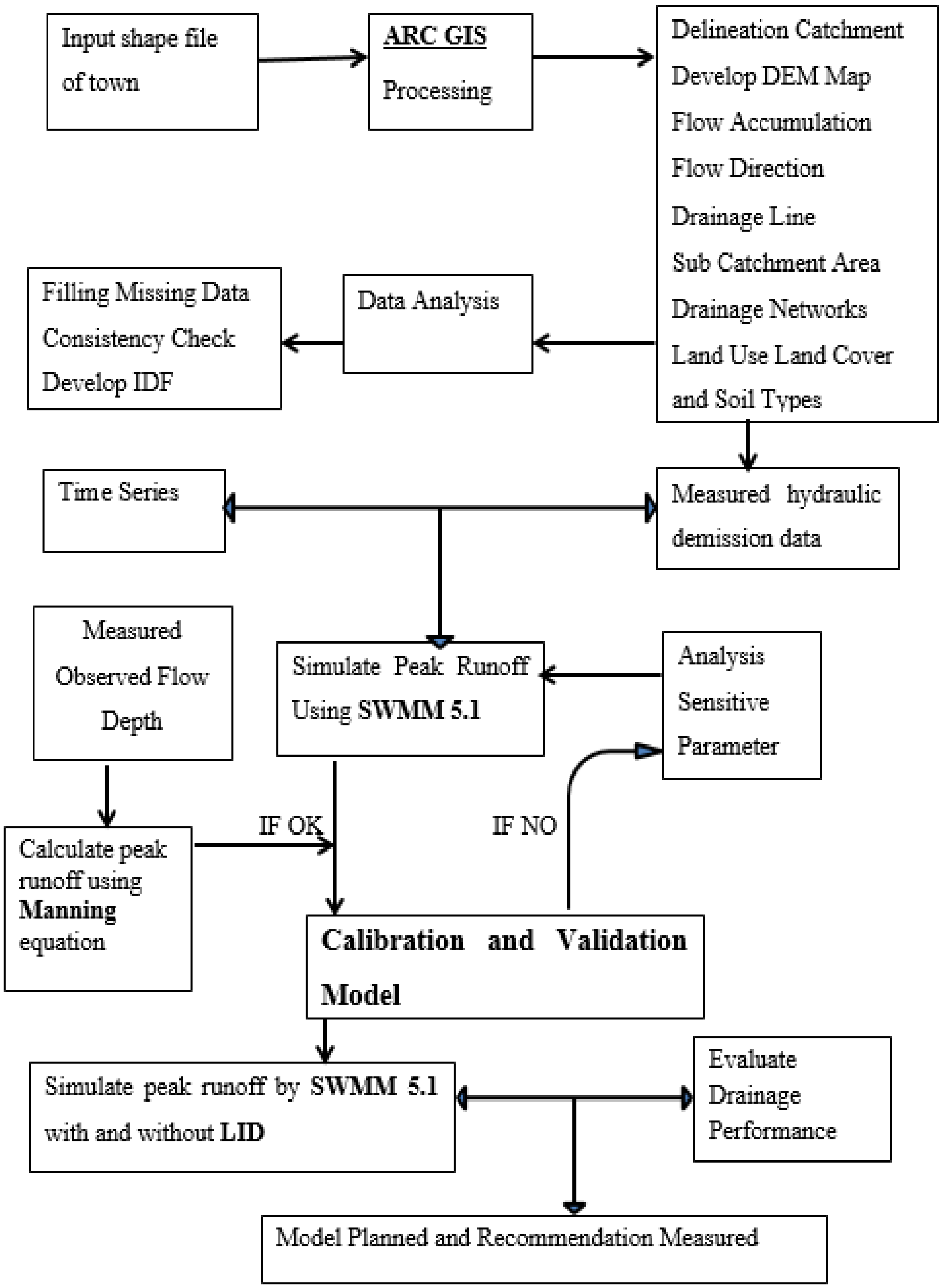


Figure 3-9: Overall framework methodology

3.4 Rainfall Analysis

3.4.1 Filling of Missing Data

Rainfall surrounds missing values which attributes to different reasons such as weathered, instrumental failures, or human error data entry .estimation of filling missing rainfall value becomes the first priority in the data preparation process. For this study four (4) stations are considered one target station and three (3) neighboring stations in the radius of 10km to 30km.the data consists daily rainfall from 1986 to 2017 G.C(32 Years') in order to have better estimation results(Kang and Yusof, 2012).

Give the yearly precipitation values, p1, p2, …, pm at neighboring m stations 1,2,3,..,m respectively , it is required to find the missing annual rainfall px at a station x not included in the above m stations. Further, the normal annual precipitations N1, N2, Ni … at each of the above (m+1) stations, including station, are known. If the normal annual precipitation at various stations are within 10% of the normal annual precipitation at station x then a simple arithmetic average procedure is followed to estimate PX by equation 3.1.

$$px = \frac{1}{M}(P1 + P2 + \cdots + Pm) \quad \text{3.1}$$

If the normal annual precipitation more than 10% **PX** estimated by normal ratio methods equation 3.2 gives as follow

$$px = \frac{NX}{M}\left(\frac{P1}{N1} + \frac{P2}{N2} + \cdots + \frac{PM}{NM}\right) \quad \text{3.2}$$

Whereas: - N1, N2 and NM are annual rainfall of known stations

NX annual rainfall (mm) of unknown station

3.4.2 Checking the accuracy of the data

3.4.2.1 Test for consistency of record

Inconsistency would arise in the rainfall data of that station. Some of the common causes for inconsistency of record are shifting of a rain gauge station to a new location, the neighborhood of the station undergoing a marked change, change in ecosystem due to calamities such forest fires, landslides and occurrence of observational error from a certain date. Testing for inconsistency of record is done by the double mass curve technique. The procedure is that accumulated rainfall at the gauge station whose record is in doubt is plotted as ordinate versus the average concurrent accumulated average rainfall of nearby station of rainfall data are available. Subramanian book)

To resolve break in the slope of the resulting plot indicates a change in precipitation regime of station the Precipitation values at station x beyond the period of change is corrected by using the following equation 3.3 relation.

$pcx = px(\frac{mc}{ma})$…………………………………………………………...3.3

Where as

Pcx=corrected precipitation at any time period t1 at station x

PX=original recorded precipitation at time period t1 at station x

Mc=corrected slope of double mass curve

Ma=original slope of the mass curve

3.4.2.2 Testing the quality of data by relative standard error

Before proceeding to the other analysis the adequacy of rainfall data series should be checked and it should be realized. The data series should be considered and adequate if relative standard error ≤10%, where is the calculated as follows

$Average\ daily\ maximum\ annualy\ rainfall\ of\ logx(ym) = \sum X/N$……….3.4

$Aerage\ daily\ maximum\ annualy\ rainfall(xm) = \sum X/N$………………..…3.5

$Standard\ deviation(xi), \delta n - 1x = \frac{\sqrt{\sum(x-xm)2}}{N-1}$…………………………….....3.6

$Standard\ deviation\ (yi), \delta n - 1y = \frac{\sqrt{\sum(y-ym)2}}{N-1}$……………………………..3.7

$Skweness\ coefficient(Cs)\ = \frac{N\sum(y-ym)3}{(N-1)(N-2)Sy3}$…………………………………..3.8

$relative\ standard\ error\ (\delta e)\% = \frac{se}{xm} * 100$………………………………….3.9

$se = \frac{\delta n-1x}{\sqrt{N}}$……………………………………………………………………..3.10

Whereas N= Number of year recorded rainfall data

If the Relative standard error $(\delta e) \leq 10\%$ the recorded data is reliable and adequate (appendix table 2)

3.4.2.3 Testing outlier

Outliers are data points that proceed from the trend the remaining data. As shown from the above calculations the station skew is less than -0.4, so based on the following principle the skewedness (Cs) value falls in the first case. Therefore, it needs checking for lower outlier.

Case1; if Skweness coefficient (Cs) <-0.4 check for lower outlier

Case 2; if Skweness coefficient (Cs >+0.4 check for higher outlier

Case3; if Skweness coefficient (Cs –o.4< Cs <+0.4 check for both outlier.

Since it is stated that the Skweness coefficient (Cs is between -0.4 and +0.4 our recorded with respect to both outlier is within reasonable range. Thus the data is checked for both lower and higher outlier.

Since it is stated that the skewedness coefficient is between -0.4 and +0.4 our recorded with respect to both outlier is within reasonable range. Thus the data is checked for both lower and higher outlier.

A. Lower outlier determination

To detect the lower outlier the following equation are applied.

Lower outlier $yi = ym - kn * sy$…………………………………….…..3.11

B. Higher outlier determination

To detect the higher outlier the following equation are applied

Higher outlier $yh = ym + kn * sy$………………………………………..3.12

Where

Yi and yh=antilog of value calculated

Ym=mean of data in log unit

KN=read from table for sample size N (32 year) find KN from (Ven Te cow)

Sy=for Skweness coefficient (Cs) value calculated by above equation 3.8 find **sy** from log transformation series garg book

3.5 Development of IDF curve for Mojo town

Intensity duration frequency (IDF) curve describe as the relationship between rainfall intensity, rainfall duration and return period. This curve commonly used in design of hydrologic, hydraulic and water resource system. IDF curve are obtained through frequency analysis of rainfall observation.

The development of IDF curve requires the frequency analysis is be performed of each set of annual maximum rainfall. The frequency analysis to determine the exceedance probability distribution of the rain intensity for each duration. There are two options for this frequency analysis, the first one Use an empirical plotting position approach to estimate the exceedance probabilities based on the observation and fit a theoretical extreme value distribution(Gumbel EVT-1 , Person type III,Nnormal, Log Normal and Log Person type III distribution) to estimate the rainfall associated with given exceedance probabilities. Because of absence extreme rainfall values for periods less than 24hr (12.6.3…) in mojo town station IDF curve develop to obtain the depth and intensity of 15 minute interval that used input for swmm 5.1 model. The following equation adopted for IDF development of rainfall in a given durations (M.L Waikar* and Undegaonkar Namita U, 2015)

$$\frac{R_t}{R_{24}} = t/24\left(\frac{(b+24)^n}{(b+t)^n}\right) \qquad 3.13$$

Where: - Rt/R24=rainfall ratio

Rt= rainfall in given duration (hr.)

R24= rainfall in 24hr

n (constant) =0.3

b (constant) =0.9 and

t=time (hr.) based on studies of large number of rainfall gauged in east Africa with constant value of n and b.

The relation adopted for IDF development at a given station, all probability distribution function were compared by chis-Square test of goodness fit and selecting the best probability function can be used for this is research is normal distribution method(VenTe Chow)

$$X_T = \bar{x} + K_T S \qquad 3.14$$

Where XT= the magnitude computed runoff (mm)

$\bar{x}$=mean value of annual daily rainfall data (mm))

S=standard deviation value of annual daily rainfall data (mm)

KT=the Frequency Factor

$$K_T = Z = W - \frac{2.515517+0.802885W+0.00328W^2}{1+1.432788W+0.189269W^2+0.001308W^3} \qquad 3.15$$

$$W = \left(\ln\left(\frac{1}{P^2}\right)\right)^{1/2} \quad (0<P\leq 0.5) \qquad 3.16$$

$P = \frac{1}{T}$……………………………………………………………..3.17

Where Z=standard normal variation

W= an intermediate variable

P=probability function

T= return period (years)

When p>0.5, 1-p is substituted for p in equation W and Z computed is given a negative sign. The error in this formula less than 0.00045 in Z (Ven Te Chow)

3.6 Modeling hydrological and hydraulic Using EP SWMM5

3.6.1 General explanation EP SWMM

SWMM is a dynamic rainfall-runoff simulation model used for single event or long-term (continuous) simulation of runoff quantity and quality from primarily urban areas. The runoff component of SWMM operates on a collection of sub catchment areas that receive precipitation and generate runoff and pollutant loads. The routing portion of SWMM transports this runoff through a system of pipes, channels, storage/treatment devices, pumps, and regulators. The quantity of runoff generated within each sub catchment, and the flow rate, flow depth, and channel during a simulation period included of multiple time steps. Even for small catchments, runoff and consequent model predictions. (EPA, 1992)

3.6.2 Model capability

SWMM accounts for various hydrologic process that produce runoff from urban area spatial variability in all of these process is achieved by dividing a study area into a collection of smaller sub catchments areas; each contains its own fraction of pervious and impervious sub-areas. The model contains flexible set of hydraulic modeling capability used to route runoff and external inflows through drainage system network of channels, pipes and storage unit’s structures. These include the ability to

- Handle networks of unlimited size
- Use wide variety of standard closed and open conduit shapes as well as natural channels
- Utilize full dynamic wave flow routing methods
- Model various flow regimes such backwater, surcharging, reverse flow and surface ponding.

For modeling accuracy to more specifically and successful calibration of SWMM essential rain gages be located within and adjacent to the catchment. (EPA SWMM 5.1 Manual)

3.6.3 Model set up procedure

- Set the coordinates of area map/image
- Draw network representative and describe sub catchments
- Edit the properties of the object that make up the system
- Describe how the system is operated
- Select a set of investigation options
- Run Simulation for Rainfall/Runoff and Flow routing

3.6.4 Model preparation area

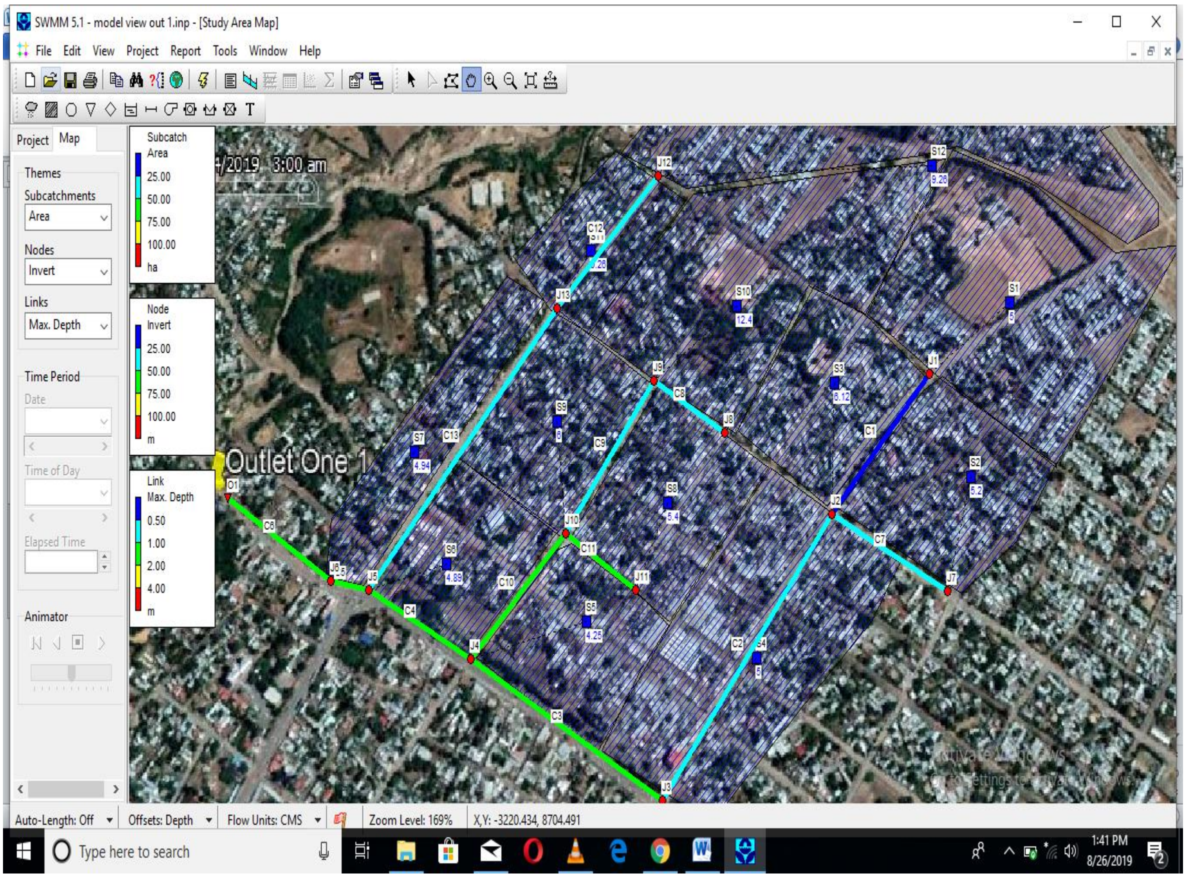


Figure 3-10: The outlet one map area prepared

The above figure 3.8 shows modeled area of outlet one starts from gorge's church to bowa gorge near to mojo oil factory. That simulated model of each 12 sub catchments connected into 13 nodes and 12 channels and flow routing outfalls into bowa gorge that drainage line contribution to Mojo River.

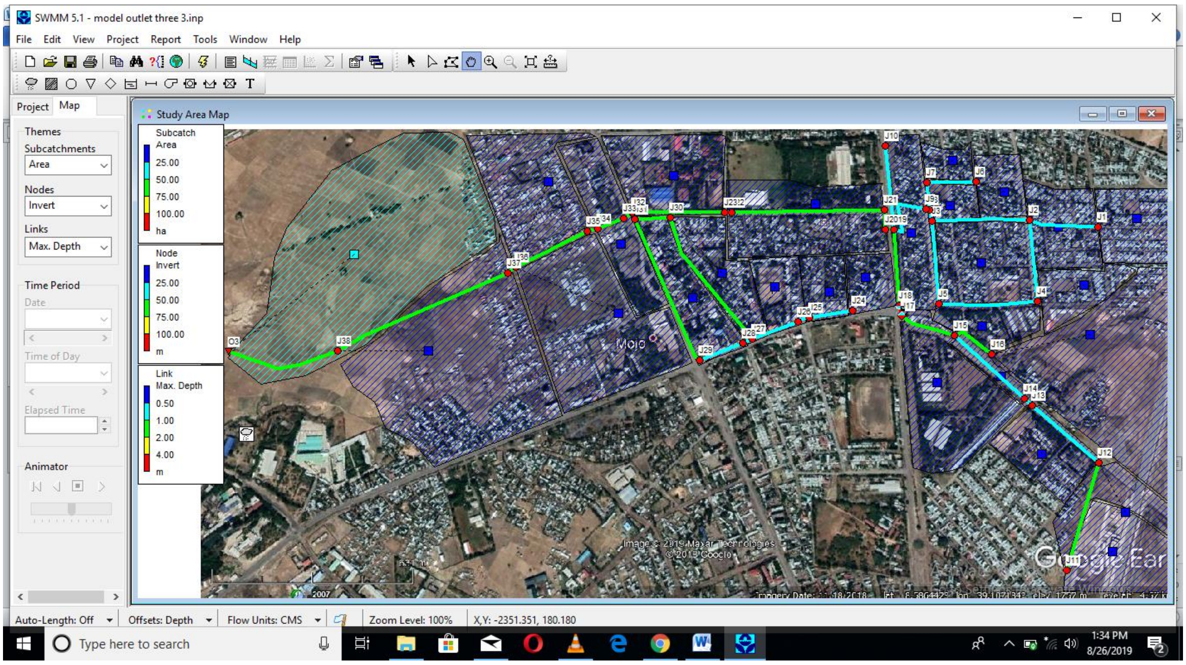


Figure 3-11: The outlet three map area prepared

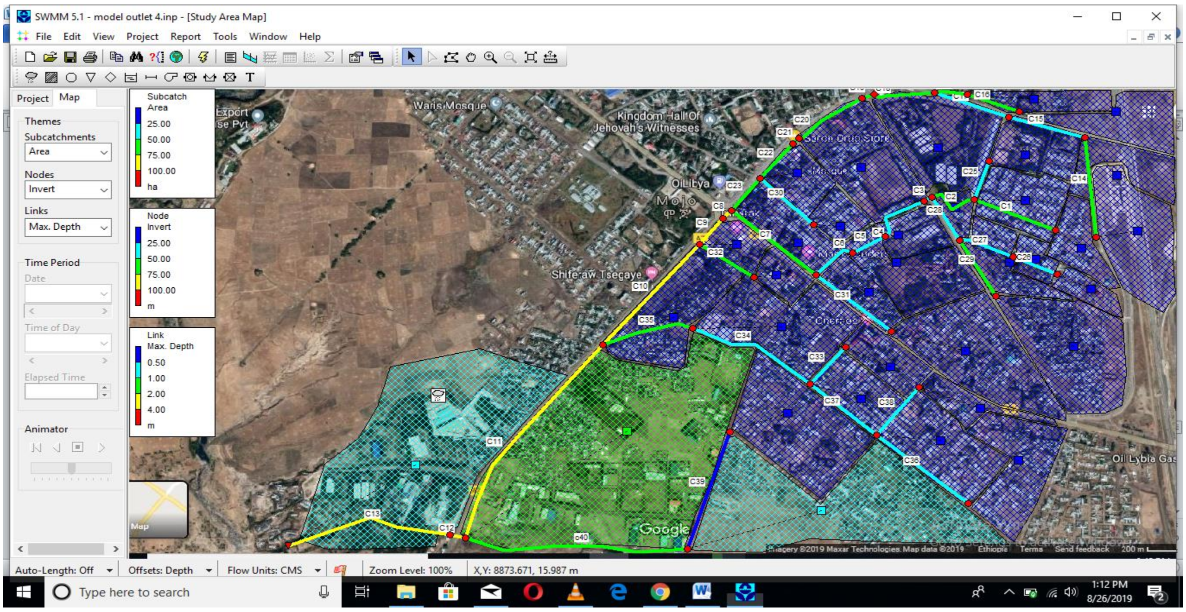


Figure 3-12: The outlet four map area prepared

The total model area prepared of mojo town drainage system networks consists of 745.2ha area from the whole 78 sub catchments. 93 junctions and 101 channels and 1 divider in all 4four outfalls which showed on the above figure 3.8, 3.9 and 3.10.

The manholes/Junctions were modeled as rectangular channel with different depth. It had assumed that there are no energy losses in the manholes. The precipitations were introduced into the model by associating each sub-catchment to the rainfall time-series.

3.7 Governing Equations

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

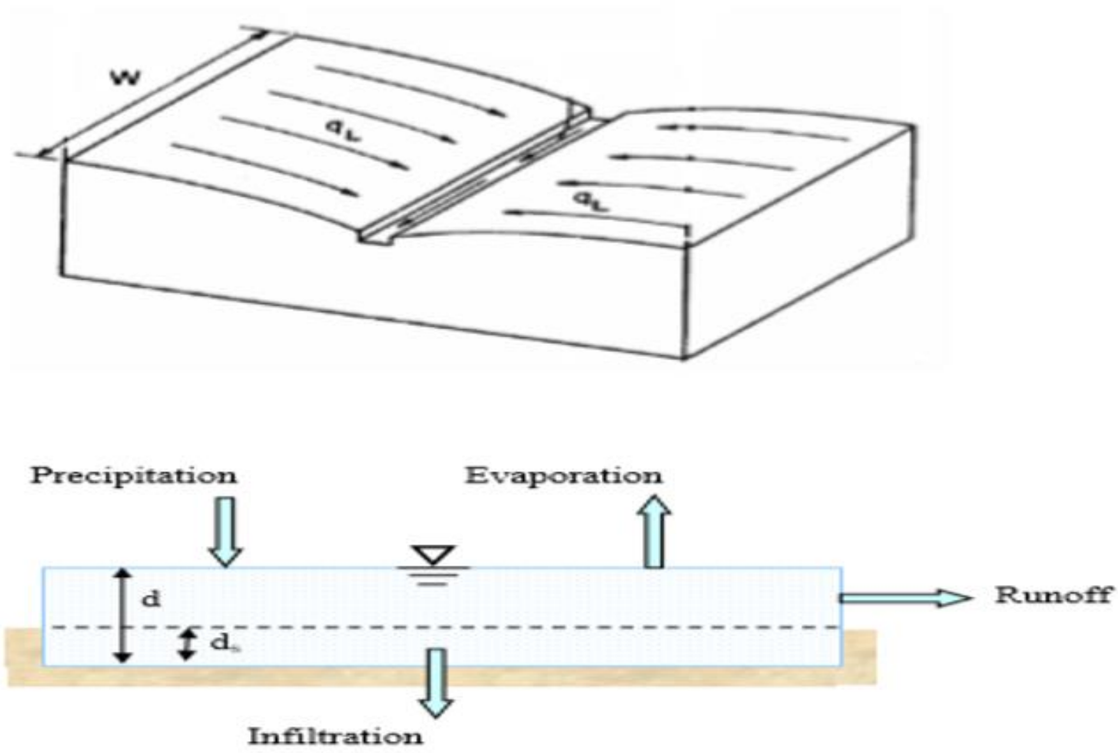


Figure 3-13: Idealized and Nonlinear reservoir model of a subcatchment
(Source swmm model manual)

In the above figure 3.13 representation, the sub catchment experiences inflow from precipitation (rainfall and snowmelt) and losses from evaporation and infiltration. The net excess ponds atop the sub catchment surface to a depth d. Ponded water above the depression storage depth ds can become runoff outflow q. Depression storage accounts for initial rainfall abstractions such as surface ponding, interception by flat roofs and vegetation, and surface wetting. From conservation of mass, the net change in depth d per unit of time t is simply the difference between inflow and outflow rates over the sub catchment:

$\frac{ad}{at} = i - e - f - q$……………………….3.18

Where: i = rate of rainfall + snowmelt (m/s) e = surface evaporation rate (m/s)

f = infiltration rate (m/s) and

q = runoff rate (m/s).

Note that the fluxes i, e, f, and q are expressed as flow rates per unit area (cms/m^2 = m/s). The required rain input data in urban drainage applications depends on the nature of the engineering task. In this study both point Rainfall and block rains will be considered. The rain data is commonly measured as intensity (mm/h) or depth (mm) and it is related to a statistical concept: frequency. (Durrans & Haestad Methods, 2003).

Assuming the 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 be used to express the runoff's volumetric flow rate Q(cms) as:

$$Q = \frac{1.49}{n}\left(A * R^{\frac{2}{3}} * S^{\frac{1}{2}}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.19$$

Here n is the surface roughness coefficient dimensionless, A the area across the sub catchment's width through which the runoff flow (m^2), R the hydraulic radius associated with this area (m) and S the apparent or average slope of the sub catchment (m/m).

A is a rectangular area with width W and height d-ds. Because W will always be much larger than d it follows that

$$A = W * (d - ds) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.20$$

$$R = d - ds \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.21$$

Substituting these expression into equation above

$$Q = \frac{1.49}{n} * W * (d - ds)^2 * S^{1/2} \ldots\ldots\ldots\ldots 3.22$$

To obtain a runoff flow rate per unit of surface area, q, equation 3.22 is divided by the surface area of the sub catchment

$$q = \frac{1.49 * W * S^{\frac{1}{2}}}{An} * (d - ds)^{5/3} \ldots\ldots\ldots\ldots 3.23$$

3.8 Calibration and validation of model

The calibration model was able to predict the observed output with the reasonable accuracy; the sensitivity analysis was performed by changing each parameter while keeping all others constants and observing the changing in model output using the recent swmm5.1 obtained from the hydraulic model calibration and validation process were done for mojo town. The most parameters used for sensitivity analysis and allowable range of change proposed by (Weaver and Nachabe, 2018)

The Observed data were used in the model calibration, a process made by a manual trial and error method and the simulated and the observed runoff in the outlets were compared.

Table 3-3 some keys parameters used for sensitivity analysis

Parameter	Description	Allowable range of change
N-Imperv	Manning’s roughness coefficient for impervious areas	0.005-0.05
N-perv	Manning’s roughness coefficient for pervious areas	0.05-0.5
Dstore –Imperv	Depth of surface storage in impervious areas(mm)	1.3-2.5
Dstore-Perv	Depth of surface storage in pervious areas(mm)	2.5-7.6
Zero-Imperv	Impervious areas without surface storage (%)	50-80

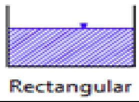
3.8.1 Calibration and validation data

SWMM model calibration and validation checked rainfall through basin and recorded stream flow at drainage system was needed. Because of no flow gauges installed in mojo town drainage system and unavailability of recorded data is difficult to obtained data.

Figure 3-14: The recorded flow depth at site and drainage network

In the figure 3.14 In order to model accuracy recorded flow depth data in open rectangular trapezoidal channel from Mojo rail station, trough ethio japan factory ends to Mojo mizzen was selected around outlet three (3).this flow depth was recorded at different interval for 12 days from June to quarter of September to calibration of sensitivity parameter and validate swmm5.1 model for the area.

Table 3-4: Recorded flow depth for calibration and validation model

Gauged area	Type channel	Data of measured date	Recorded flow depth(m)	Method calculation flow discharge $Q=\frac{1}{n}*A*R^{\frac{2}{3}}*S^{\frac{1}{2}}$
at outlet 3 for **calibration** used		June 16 2019	0.55	Each Values are filling in result and discussion part
		July 8 2019	0.62	
		July 15 2019	0.8	
		August 5 /2019	0.78	
		August 22 /2019	0.84	
		Sept. 3 /2019	0.76	
at outlet 3 for **validation** used		June 2 2019	0.42	
		July 22 2019	0.6	
		June 28 2019	0.85	
		July 30/2019	0.79	
		August 12/2019	0.98	
		Sept 8/2019	0.7	

From the above table 3.4 six days rainfall data observer used for calibration the model with flow calculated using maximum flow depth (m) recorded and the other left six days used for validation of the model.

The method of calculation calibration and validation of observed data is using the manning equation.

$Q=\frac{1}{n}*A*R^{\frac{2}{3}}*S^{\frac{1}{2}}$ ……………………………………3.24

Whereas Q=flow discharge in m^3/s

n=manning roughness coefficient

A=cross-sectional area in m^2 that derived from recorded flow depth (m) with given drainage dimensionless

R (A/P) =hydraulic radius in m

P=wetted parameter in m

S=channel slope in m/m fraction

3.8.2 Criteria of evaluation Model performance

The consistency of the flow checking data is evaluated to calibration and validation process. The model simulation errors were quantified by measuring the difference between observed and simulated hydrographs. Analyzed the results as following equations.

The Nash-Sutcliffe efficiency (NSE)

$$\text{The Nash Sutcliffe efficiency (NSE)} = 1 - \frac{\sum_{t=1}^{n}(q_t^{obs} - q_t^{sim})^2}{\sqrt{\sum_{t=1}^{n} q_t^{obs} - q_t^{avrg.obs})^2}} \ldots\ldots\ldots\ldots..3.25$$

Coefficient of determination (R^2)

$$R^2 = \left[\frac{\sum_{t=1}^{n}(q_t^{obs} - q_t^{avrg.obs})(q_t^{sim} - q_t^{avr.sim})}{\sqrt{\sum_{t=1}^{n}(q_t^{obs} - q_t^{avrg.obs})^2}\sqrt{\sum_{t=1}^{n}(q_t^{sim} - q_t^{avr.sim})^2}}\right]^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.26$$

Relative error (RE)

$$RE = \frac{\sum_{t=1}^{n}|q_t^{obs} - q_t^{sim}|}{\sqrt{\sum_{t=1}^{n} q_t^{obs}}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots...3.27$$

Whereas q_t^{obs} and $q_t^{avrg.obs}$ = are the calculated and average flow respectively

$q_t^{sim}$and $q_t^{avrg.sim}$=area the simulated and average flow respectively

t=at time t and

n= is the total number of time steps.

3.8.3 Acceptable level of calibration

- NSE value Between 0 and 1 shows acceptable model

  IF < 0 indicates poor models,

  = 1 perfect models,

  = 0 Model is no better than using as an Estimator
- R2- Approach to one and indicates adequate models.
- RE < 30%

These three attributes are important in design and analysis of urban drainage systems. Peak discharge is required in urban drainage design for sizing pipes, culvert and bridges. Runoff volume is required for design and operation of flood control structures such as retarding basins. Time to peak discharge is required for flood forecasting and operation of control structures during storm events. Flow data was calibrated for daily flows(“Eyosais b, 2018)

3.9 Evaluate alternatives mitigation actions for drainage overflow

The excess storm water runoff increase in urban drainage areas overflow risks and leads to significant economic losses, traffic disturbances and even occur deaths on community. So that is need to decrease this flood risk from urban area are by improving existing drainage system capacity, evolution Sustainable Urban Drainage Systems (SUDS), adding best management practices (BPMS) and low impact development(LID) techniques in the drainage system.(Zhou Q., 2014)

3.9.1 LID Improvement Systems

The low impact development practices designed to capture surface runoff and provide some combination of detention, infiltration, and evapotranspiration to it. They are considered as properties of a given sub catchment..(States U., 2015)

When a user adds a specific type of LID control object to a SWMM project the LID Control Editor is used the design properties of each relevant layer (such as thickness, void volume, hydraulic conductivity, drain characteristics, etc.). These LID objects can then be placed within selected sub catchments at any desired sizing (or areal coverage) by editing the sub catchment’s **LID Controls** property.(Eyosias B., 2018)

LIDs are modeled as a number of interconnected, fully mixed layers representing the surface, pavement, soil, storage, and underdrain portions of a LID unit. Infiltration, drainage, and overflow control the storage in each of the layers dynamically. SWMM can explicitly model bio retention cells, infiltration trenches, porous pavement, rain barrels, vegetated swales and green roofs.(Niazi M., 2017)

3.9.1.1 Infiltration trenches

Infiltration trenches are engineered structures that provide storage and facilitate infiltration of runoff into the subsurface. Infiltration trenches are typically long and narrow and filled with aggregate. Runoff from the study area was routed through an infiltration trench in the LID area. They can be simulated as fully pervious sub-catchment whose depression storage depth equals the equivalent depth of the pore space available within the trench.

Infiltration Trenches are narrow ditches filled with gravel that intercept runoff from upslope impervious areas. They provide storage volume and additional time for captured runoff to infiltrate the native soil below(Acharya A., 2018)

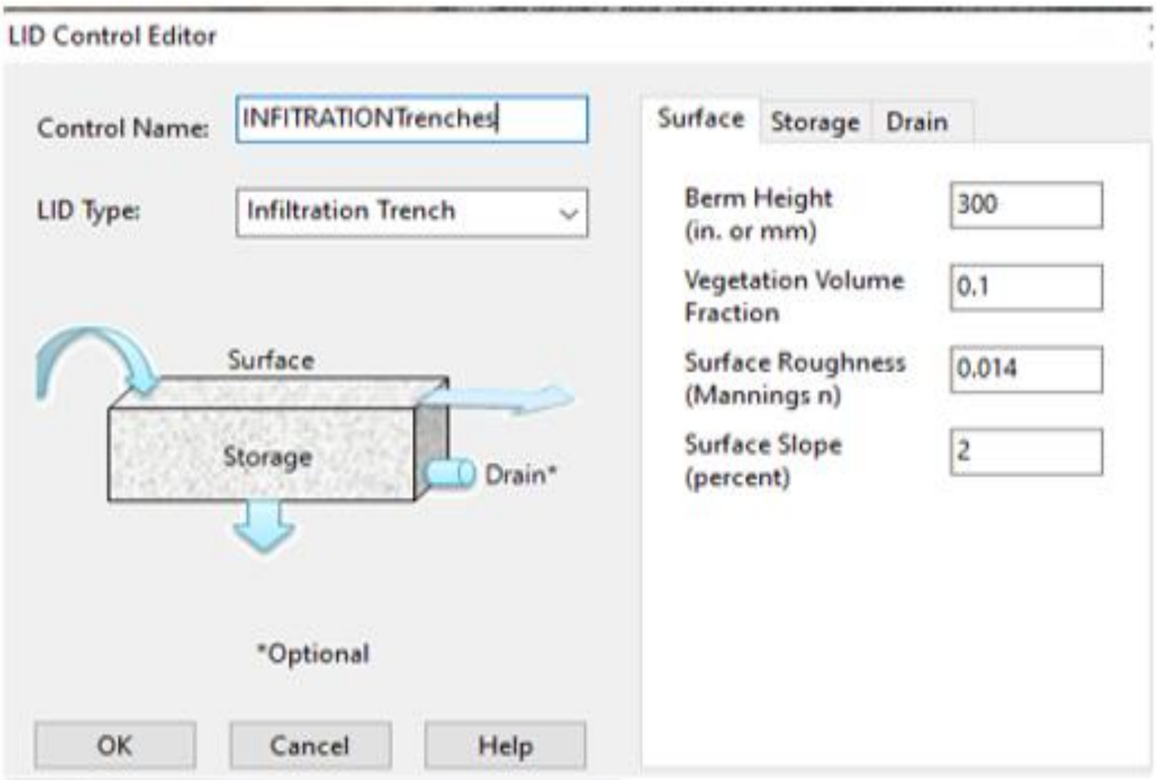


Figure 3-15: LID editor techniques of Infiltration trenches within swmm5.1

3.9.1.2 Bio-retention Cells

Bio retention cells are depressions that contain vegetation grown in an engineered soil mixture placed above a gravel drainage bed. They provide storage, infiltration and evaporation of both direct rainfall and runoff captured from surrounding areas.(States U., 2015)

Bio-retention cells remained set in dual characteristic housing communities located in china urban sub-catchment. The depth of the surface storage was 150 mm, with 90% vegetation coverage; the soil thickness was 500 mm, with 0.5 porosity. The field capacity was 0.35 and the wilting point was 0.187. The conductivity gradient of the bio-retention cell was 10, with 36 mm/h hydraulic conductivity and the height of the gravel layer was 400 mm with 210 mm suction of the soil moisture.(Luan Q., 2010)

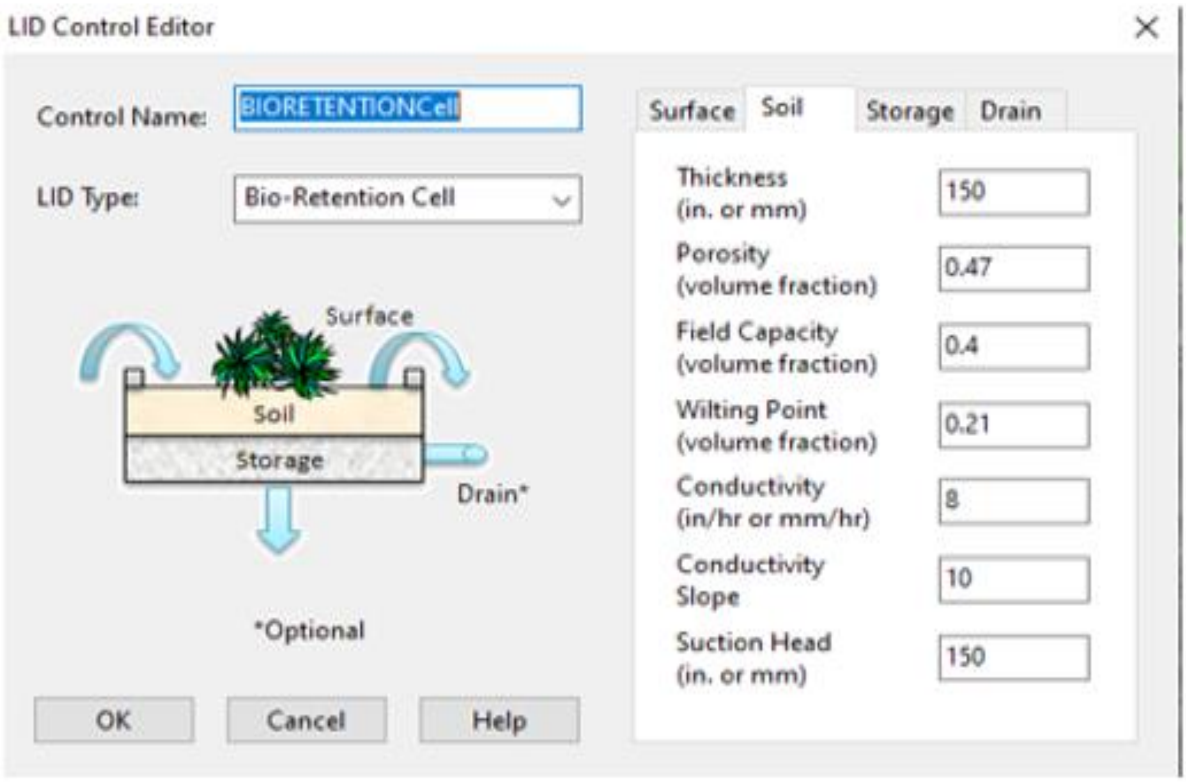


Figure 3-16: LID editor of bio retention within swmm5.1

This study used LID control editor in Bio-retention soil parameter value shows in figure 3.14 above are used 150mm soil thickness, 0.47 porosity in volume fraction .0.4 field capacity in volume fraction, 0.21 wilting point in volume fraction, 8mm/hr. hydraulic conductivity, 10 conductivity slope and 150mm suction head of the soil moisture all value depends on soil texture class of town.

3.9.1.3 Permeable Pavement

Permeable Pavement systems are excavated areas filled with gravel and paved over with a porous concrete or asphalt mix. Block Paver systems consist of impervious paver blocks placed on a sand or pea gravel bed with a gravel storage layer in the figure below

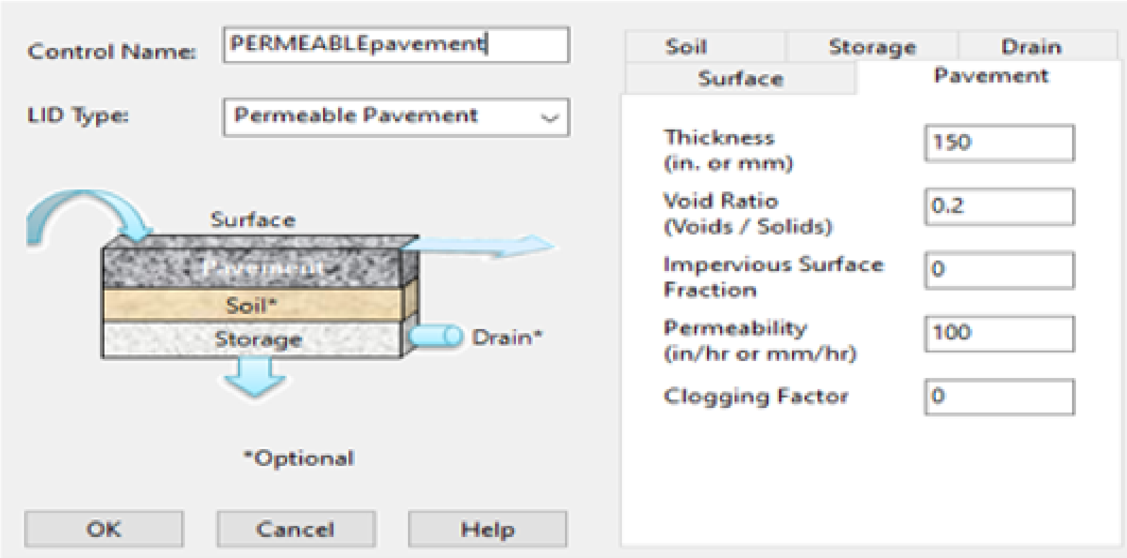


Figure 3-17: LID editor of Permeable Pavement within swmm5.1

3.9.1.4 Vegetative Swales

Vegetative Swales are channels or depressed areas with sloping sides covered with grass and other vegetation. They slow down the conveyance of collected runoff and allow it more time to infiltrate the native soil.

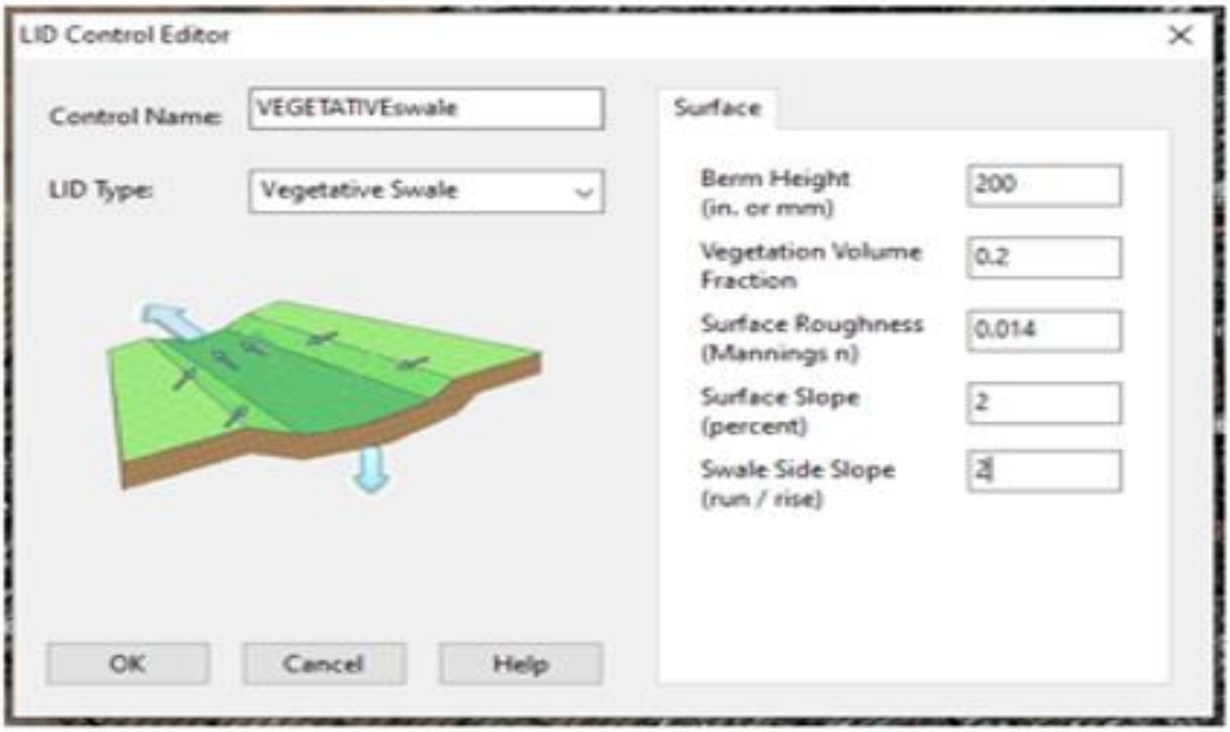


Figure 3-18 LID editor of Vegetative Swales within swmm5.1

The above figure 3.13, 3.14, 3.15 and 3.16 are used LID controls editors within SWMM tool for modeling the best alternative mitigation activities to decreasing the drainage overflow of urban drain systems in their gravel storage beds to convey excess captured runoff off of the site and prevent the unit from flooding.

Table 3-5 Designing and modeling parameters of LID types

Designing and modeling LID	layer	Properties	LID types developed			
			Infiltration trenches	Bio retention	Pavement permeable	Vegetation swales
Designing parameters of LID used	Surface	Berm height(mm)	300	300	300	200
		Vegetative volume fraction	0.1	0.1	0.1	0.2
		Surface roughness	0.014	0.024	0.024	0.014
		Surface slope (%)	2	1.5	1.5	2
		Swale side slope(run/rise)				2
	storage	Thickness(mm)	400	350	300	
		Void ratio(voids/solid)	0.75	0.75	0.7	
		Seepage rate (mm/hr.)	0.5	5	0.5	
		Clogging factor	0.1	0.1	0	
	Soil	Thickness(mm)		150	750	
		Porosity(volume fraction)		0.47	0.47	
		Field capacity(volume fraction)		0.34	0.34	
		Wilting points(volume fraction)		0.21	0.21	
		Conductivity (mm/hr.)		0.04	0.04	
		Conductivity slope		40	20	
		Suction head(mm)		10.63	10.63	
	Pavements	Thickness(mm)			150	
		Void ratio(voids/solids)			0.2	
		Impervious surface fraction			0	
		Permeability (mm/hr.)			100	
		Clogging factor			0	
	Drain	Flow coefficient	0	0	0	
		Flow exponent	0	0	0	
		Offset height(mm)	5	0	5	
Modeling LID parameters	Areas of each units from total (%)		25	25	25	25
	Number of units		2	2	2	2
	Surface width per(m) unit		3	3	2	3.5
	Initially saturated (%)		2	2	2	2
	Impervious area treated (%)		80	95	85	85

In The Table 3.5 the designing and modeling of Lid control editor’s layer of surface, storage, soil, and pavements parameters for each types of LID developed with the recommended value from EPA SWMM5.1 tool manual are used in the drainage systems.

CHAPTER 4 RESULTS AND DISCUSSIONS

4.1 Data Availability

4.1.1 Double Mass Curve

Double mas curve is used to check the consistency of hydrological data of MOJO station by comparing with other station (NEZERET station. KOKA station and DEBRAZIET station) in the surrounding and that can used to adjust inconsistency precipitation data. The graph 4.1shows cumulative annual rainfall of mojo station versus the average accumulative annual rainfall of neighboring stations ration between variables is fixed by correction factor 1.052. (Appendix table 1)

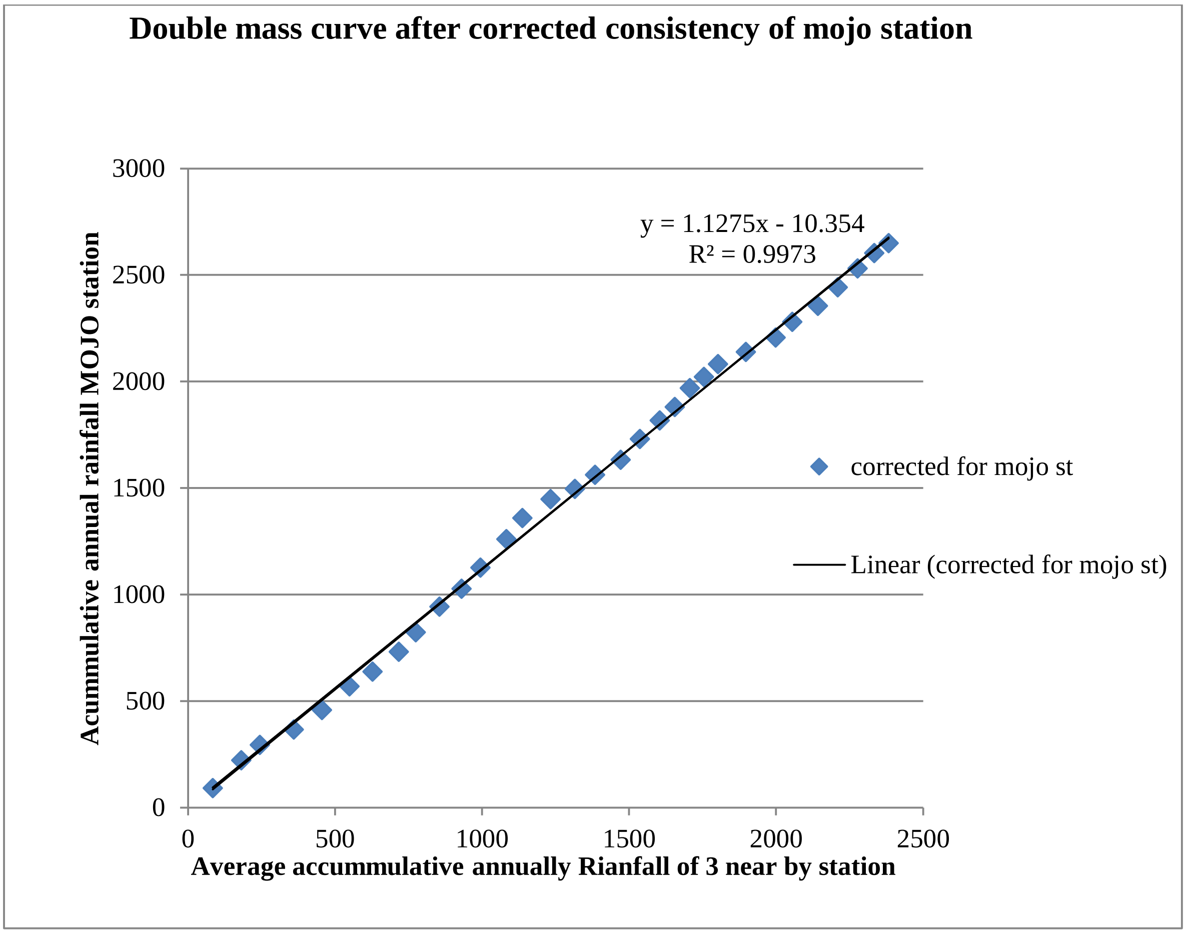


Figure 4-1: Double mass curve after corrected inconsistency of mojo rainfall station

4.1.2 Testing Relative standard error

Before proceeding to the other analysis the adequacy of rainfall data series should be checked and it should be realized. The data series should be considered and adequate, if relative standard error ≤10%.

$$se = \frac{21.67}{(32)^{0.5}} = 3.83$$

$$\delta e = \frac{3.83}{82.8} * 100 = 4.63\%$$

4.63%≤10%......ok therefore the data is adequate and reliable. (Appendix 2)

4.1.3 Testing outlier

Outliers area the data points that depart from the trend of remaining data as shown at (appendix table 2) calculated the Skweness coefficient is between -0.4 and +0.4 our data recorded with respect to both outlier is reasonable range. This data is checked for both lower and higher outlier.

4.1.3.1 Lower outlier test

YL=the lower outlier Boundary in log unit

From table sample size for N (32year) rainfall data read Kn=2.591(Ven Te cow table 12.5.3 outlier test kn values)

Ym=mean of data in log unit (1.9036) and

For Ym=1.9036, read sy=0.1144356(from log-transformation series garg text)

YL=1.9036-2.591*0.1144356=1.607

AntilogYL=**40.46mm**

The lowest recorded daily maximum rainfall is **47.03mm** in the 2002(Appendix table 2) which higher than boundary of lower outliers. Hence the daily maximum rainfall data recorded with respect to lower outlier is within reasonable range.so there are no lower outliers in this sample data.

4.1.3.2 Higher outlier test

$$YH = 1.9036 + 2.592 * 0.1144357 = 2.001$$

AntilogYH=**158.527mm**

The higher recorded daily maximum rainfall is **132.45mm** in the 2005 which lower than boundary value of higher outliers. Hence there are no higher outlier's data recorded.

4.2 IDF curve development

4.2.1 IDF curve development for Mojo town

The intensity duration frequency curve developed from daily rainfall data of 32 years from 1986 to 2017 obtained in Ethiopia meteorological agency rainfall gauge station located in Mojo city. For this study the rainfall intensity obtains from IDF curve is used to SWMM model as input parameter.

Table 4-1: Mojo IDF curve development

Duration (minute)	Duration (hour)	2 year Intensity (mm/hr)	5year Intensity (mm/hr)	10year Intensity (mm/hr)	25year Intensity (mm/hr)	50year Intensity (mm/hr)	100year Intensity (mm/hr)
5	0.08	151.01	184.32	201.76	220.46	232.34	243.14
10	0.17	119.07	145.33	159.08	173.83	183.19	191.71
15	0.25	104.76	127.86	139.96	152.93	161.17	168.66
30	0.5	74.76	91.26	99.89	109.16	115.03	120.38
60	1	48.30	58.96	64.53	70.52	74.31	77.77
120	2	28.90	35.28	38.62	42.20	44.47	46.54
180	3	20.89	25.49	27.90	30.49	32.13	33.63
360	6	11.67	14.25	15.59	17.04	17.96	18.79
720	12	6.39	7.80	8.54	9.33	9.83	10.29
1440	24	3.46	4.23	4.63	5.06	5.33	5.58

All the probability distribution functions were compared by chi-square test of goodness of fit and the selecting the function that gave the smallest chi-square value determined the best probability function. From the final result table 4.1 the best fit probability is **normal distribution** and that used to developed IDF curve of Mojo town as shown figure below 4.3 and detail analysis in appendix Table 3 to 9 is specified.

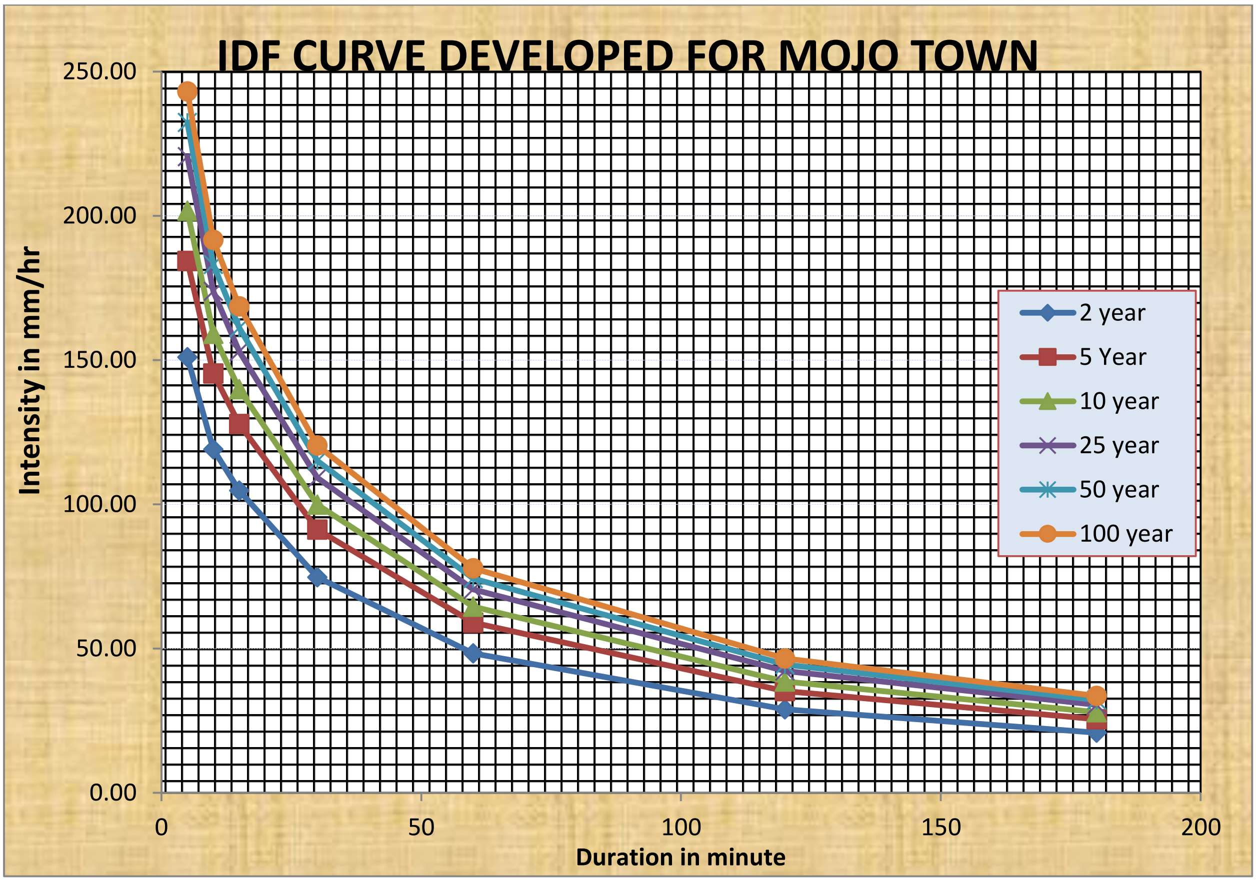


Figure 4-2 Developed IDF curve for Mojo town

4.2.2 Relationship of Mojo town IDF curve with ERA

The relationship adopted for IDF development at a given station, any probability distribution can be used but the reliability of the distribution were assessed whether a given distribution are suited to a data set with ERA developed.

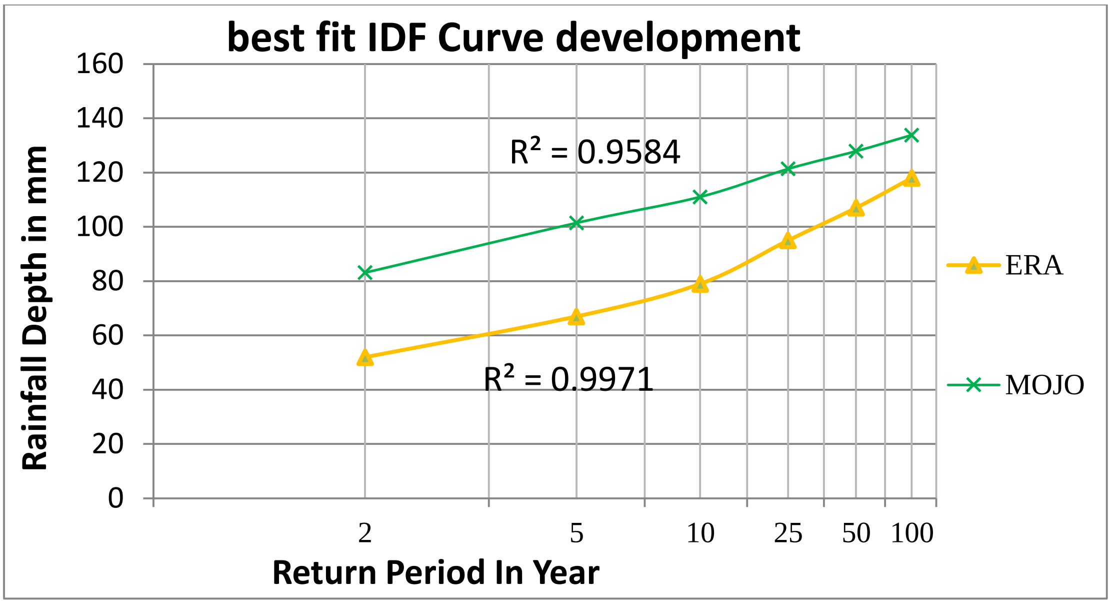


Figure 4-3 Compared of Mojo IDF Curve with ERA

From figure 4.3 The IDF curve developed by ERA under rainfall region A2 R^2 =0.8971 and the new IDF Curve developed for Mojo town by different distribution method checked by the goodness of fit tests that R^2=0.9584 more suitable than ERA IDF curve. Therefore from result tested used the new IDF curve development of Mojo town for assessment and modeling drainage systems performance of study area.

4.3 Calibration and validation of the model

The calibration and validation of SWMM model in this study depends on the sensitive parameter value adjusting until to fit the observation is comparable to that achieved in the calibrate by using goodness of fit measured with performance simulation model.

4.3.1 Model calibration

The calibration proceed has been confirmed by varying sensitive parameters to different model parameter input and check the output model result until it match closely within proper range compare with the observed results For the modeled accuracy and successful calibration of SWMM.

Table 4-2 Sensitivity parameter used for the calibration of model

Parameters	Manning Description	Recommended Value Range	Initial Used Proceed Sensitive Parameters	Final used Sensitive Parameters

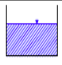
N-Impervious	Manning Roughness For Impervious Area	0.011-0.015	0.013	0.015
N- Pervious	Manning Roughness For Pervious Area	0.05-0.8	0.3	0.5
Dstore-Impervious	Depth Of Depression Storage On Impervious Area	0-3	2	2.5
Dstore-Pervious	Depth Of Depression Storage For Pervious Area	3-10	4	7
Smooth concrete roughness	Manning Roughness Coefficient For Open Smooth Concrete	0.012	0.012	0.012
Infiltration Method	Suction	3.5	3.5	3.5
	Conductivity	0.5	0.5	0.5
	Initial Deficit	0.25-0.26	0.26	0.26

The input parameter's used in the above table 4.2 for the calibration of model the final output Calibrated results of peak discharge flow rate simulated by swmm5.1 model is 1.403m^3/s and the observed flow depth at site is calculated peak discharge flow rate by manning equations is 1.478m^3/s.

4.3.2 Model validation

The validation of this model fit to the observed at the outlet three root is verified the velocity and the discharge flow rate in the drainage systems is listed in the table 4.3 below.

Table 4-3: Validation flow depth observed result

Channel type	Record depth (m)	Manning roughness used(n)	channel slope(S)	Side Slope H/V (m)	Width (m)	Area (m^2)	Parameter (m)	Hydra radius (m)	Velocity (m/s)	Discharge Q(m^3/s)= $ARS^{1/2}/n$
 Rectangular	0.60	0.013	0.0072		0.8	0.48	2.0	0.24	2.485	1.210
	0.42		0.0214		0.8	0.34	1.64	0.21	3.913	1.315
 Trapezoidal	0.85	0.024	0.0094	0.2358	0.8	0.85	2.55	0.312	1.944	1.653
	0.79		0.0076	0.219	0.8	0.77	2.42	0.317	1.692	1.301
	0.98		0.0014	0.196	1.2	1.36	3.2	0.425	0.88	1.205
	0.8		0.0154	0.2222	0.8	0.78	2.44	0.319	2.42	1.895

From result table 4.3 the channel velocity 2.42m/s, 2.485m/s and 3.913m/s are high value. The rectangular open channel type material is constructed from high smooth quality concrete reinforced and the trapezoidal open channel one is constructed from stone masonry with floor stone pavement jointed by concrete material. Hence the high value of velocity above table 4.3 there is no any effect on channel. due to high strength of channel material constructed.

Table 4-4: Summary of the model result and observed values

Date of observed data	Recorded average flow depth(m)	Determine flow rate using manning equation Q(m^3/s)=$AR^{\frac{2}{3}}S^{1/2}/n$	Simulated flow rate using SWMM5.1 (m^3/s)
June 2/2019	0.42	1.315	1.214
June 28/2019	0.85	1.654	1.613
July 22/2019	0.6	1.193	1.188
July 30/2019	0.79	1.301	1.327
August 12/2019	0.98	1.205	1.239
September 8/2019	0.8	1.895	1.792

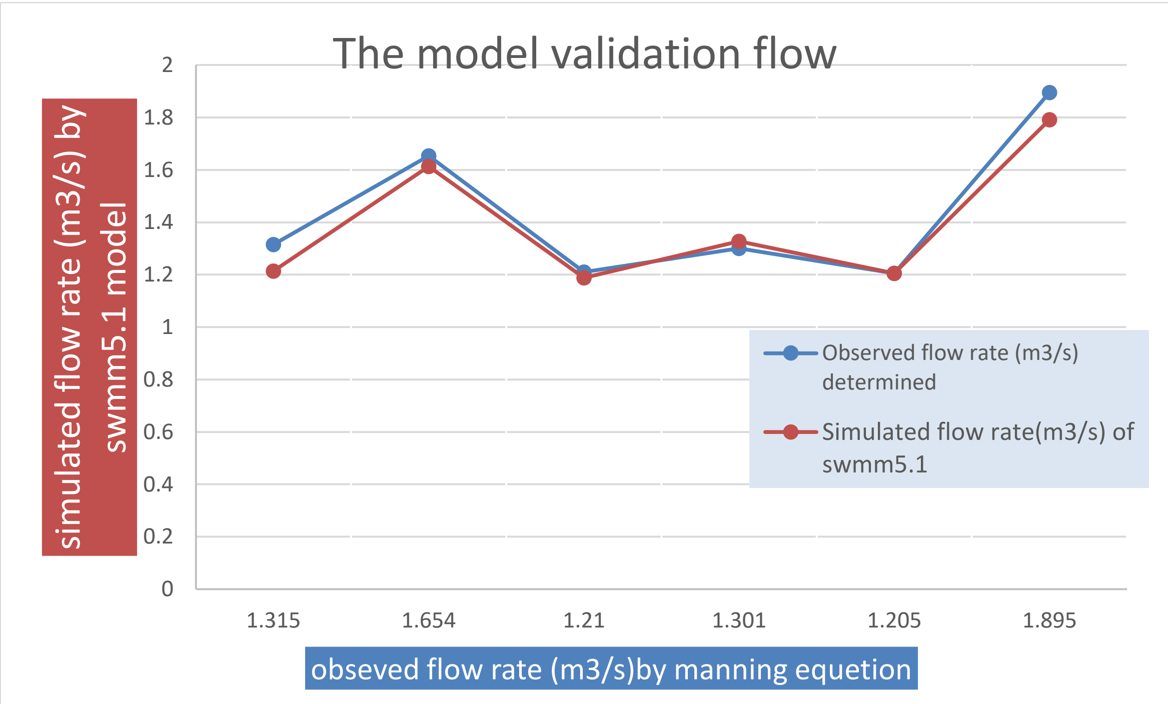


Figure 4-4 The Model Validation Flow chart

The validation of the model with the observed flow rate calculated by manning equation and simulated by SWMM 5.1 used at the same site network of channel is validated showed the above figure 4.4 chart.

4.3.3 Model performance evaluation

The Model performance evaluation were quantified by the difference between observed and simulated using method of Coefficient of Determination (R^2), Nash-Sutcliffe Efficiency (NSE) and Relative Error (RE).in the detail Evaluation in below table 4.6.

Table 4-5: Correctness function of the model performance

Recorded flow depth(m)	Simulated flow rate (using SWMM5) CMS	Calculated (observed) flow rate (using manning's) CMS	$q_t^{ob}- q_t^{avrob}$	$q_t^{sim}-q_t^{avrsim}$	$(q_t^{ob}-q_t^{avrob})*(q_t^{sim}-q_t^{avrsim})$	$(q_t^{ob}-q_t^{avrob})^2$	$(q_t^{sim}-q_t^{avrsim})^2$	$(q_t^{ob}- q_t^{smi})^2$	$(q_t^{ob}-q_t^{avrob})^2$	$\|q_t^{ob}- q_t^{smi}\|$
0.42	1.214	1.315	-0.115	-0.181	0.0209	0.0132	0.0329	0.01020	0.0196	0.101
0.65	1.613	1.654	0.224	0.2175	0.0485	0.0724	0.0473	0.0017	0.0724	0.041
0.6	1.188	1.049	-0.22	-0.208	0.0457	0.1128	0.0431	0.019	0.1129	0.022
0.79	1.327	1.322	-0.129	-0.069	0.0045	0.0088	0.0047	2.5E-05	0.0039	0.026
0.98	1.239	1.205	-0.225	-0.157	0.0282	0.0352	0.0245	0.0012	0.0324	0.034
0.7	1.792	1.835	0.465	0.3965	0.1844	0.2025	0.1572	0.0019	0.2025	0.103
Total	**8.373**	**8.58**			0.3437	0.6287	0.55651	0.02491	0.6287	0.327
Average	**1.3955**	**1.43**								

Coefficient of determination (R^2)= $\left(\frac{0.344}{0.349}\right)^2$

$$R^2=0.97$$

Nash-Sutcliffe Efficiency (NSE)$= 1 - (0.0249/0.6287)$

NSE =0.96

Relative Error (RE) $= \frac{0.327}{(8.58)^{0.5}}$

RE=0.1116=11.16% ….which is acceptable RE<30%

Overall, the calibration resulted in a good fit with observed daily flows for the simulation period, with $\mathbf{R^2 = 0.97}$, **NSE = 0.96** and relative error **(RE) =11.16%.**This calibration and validation result indicated that the model structure and parameters matched the runoff-producing pattern and the calibrated model was suitable for simulating runoff in the study area.

4.4 Model result of from swmm5.1 tool

This study result of the model fully understanding of the town drainage system performance under multiple working condition one to identify the critical condition and related problem with drainage system, secondly to assess the hydraulic performance of storm water drainage infrastructures and finally to Evaluate alternatives for drainage problem mitigation measures by using hydrological model.

The total sub catchment of the study area are generating runoff from daily precipitation taken as extreme events from each year from 1986-2017 rainfall data. The project of sub catchment used as input for drainage systems connect through each nodes and simulate runoff in the network system of swmm5.1 model in all four outfall study.

4.4.1 Result of Outlet one study

4.4.1.1 Sub catchments Runoff

Topic: Subcatchment Runoff   Click a column header to sort the column.

Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Total Runoff mm	Total Runoff 10^6 ltr	Peak Runoff CMS	Runoff Coeff
S1	120.92	0.00	0.00	16.48	104.62	5.23	0.60	0.865
S2	120.92	0.00	0.00	14.11	107.08	5.57	0.80	0.886
S3	120.92	0.00	0.00	12.40	108.54	6.64	0.71	0.898
S4	120.92	0.00	0.00	13.99	107.26	5.36	0.78	0.887
S5	120.92	0.00	0.00	17.40	103.78	4.41	0.48	0.858
S6	120.92	0.00	0.00	20.04	101.00	4.94	0.51	0.835
S7	120.92	0.00	0.00	13.82	107.42	5.31	0.79	0.888
S8	120.92	0.00	0.00	16.02	105.08	5.67	0.67	0.869
S9	120.92	0.00	0.00	12.89	108.41	6.50	0.76	0.897
S10	120.92	0.00	0.00	23.57	97.05	12.03	1.17	0.803
S11	120.92	0.00	0.00	21.48	99.24	9.19	0.93	0.821
S12	120.92	0.00	0.00	32.56	88.17	8.16	0.71	0.729

Figure 4-5 Outlet one sub catchment runoff result

From figure 4.5 the simulation results the total area of the whole 12 sub catchments is 89.9ha and produced the runoff from the area of all sub catchment is 9.84m^3/sec and the peak runoff generated at sub catchment (S11) is 0.98m^3/sec.

4.4.1.2 Junctions and channels flooded

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

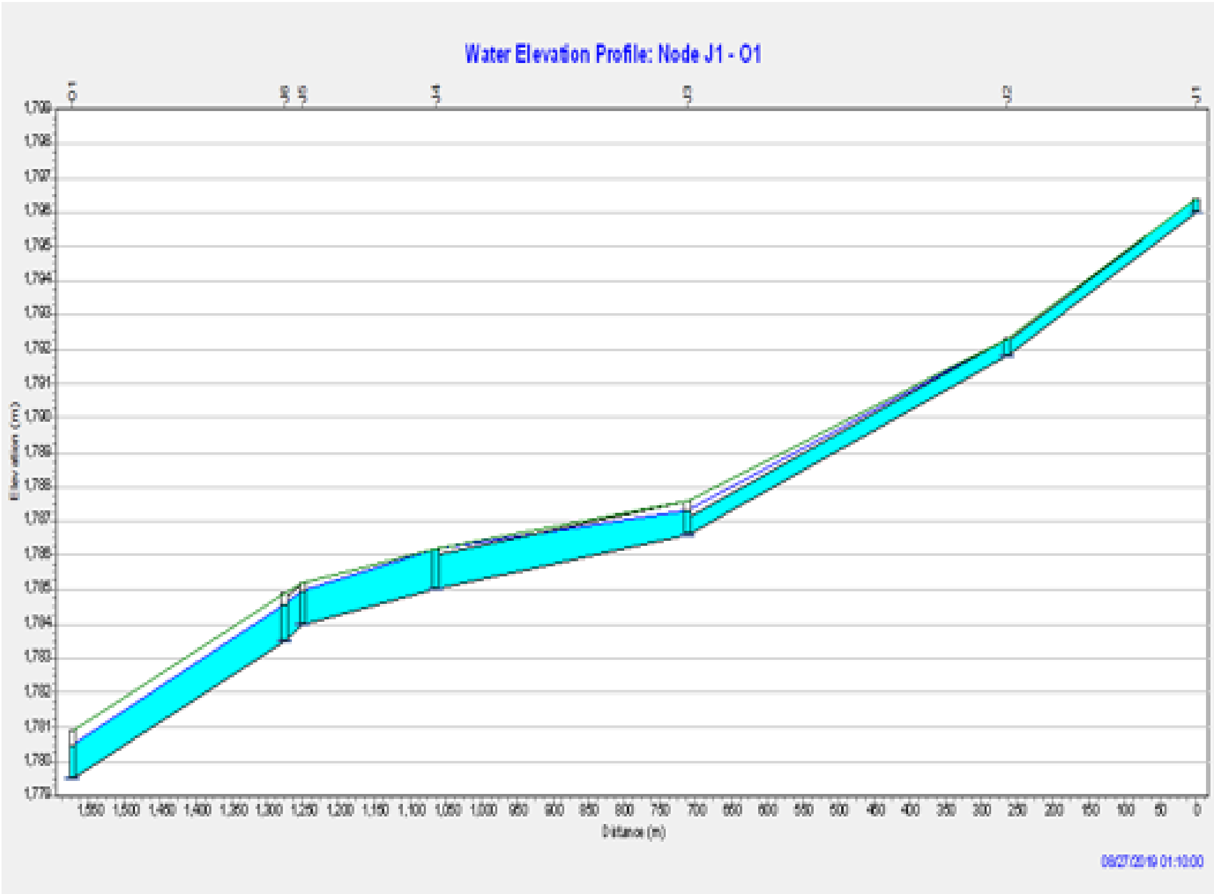
Node	Hours Flooded	Maximum Rate CMS	Day of Maximum Flooding	Hour of Maximum Flooding	Total Flood Volume 10^6 ltr	Maximum Ponded Depth Meters
J1	1.58	0.222	0	00:30	0.702	0.000
J2	3.33	0.811	0	00:12	5.344	0.000
J4	1.29	0.809	0	00:36	2.009	0.000
J7	3.28	0.674	0	00:36	3.608	0.000
J9	1.81	0.603	0	00:18	1.949	0.000
J10	0.16	0.081	0	00:30	0.036	0.000
J12	2.61	0.438	0	00:30	1.363	0.000
J13	1.46	0.389	0	00:18	1.050	0.000

Conduit	Hours Both Ends Full	Hours Upstream Full	Hours Dnstream Full	Hours Above Normal Flow	Hours Capacity Limited
C1	0.01	0.01	4.00	0.01	0.01
C2	0.01	0.01	3.40	0.01	0.01
C3	0.01	0.01	2.44	0.01	0.01
C4	0.01	1.29	0.01	0.01	0.01
C5	0.01	0.01	0.27	1.27	0.01
C7	0.01	0.01	0.01	3.25	0.01
C9	0.01	0.01	3.19	0.01	0.01
C10	0.01	0.01	2.44	0.01	0.01
C13	0.01	0.01	3.93	0.01	0.01

Figure 4-6 The outfall one flooded junctions and channels

From the above table 4.6 result, it was studied that the drainage system channels and junctions overflow in the network simulation status report shows that the sections between junctions J1, J2, J4, J7, J9, J10, J12, and J13 are nodes flooded and links overflows are.C1 to C5, C7, and C9. C10 and C13 which are limited in capacity and that not to convey flow inside the channels at different high rain season time.

4.4.1.3 Water profile and outfall charged



Topic: Outfall Loading   Click a column header to sort the column.

Outfall Node	Flow Freq. Pcnt.	Avg. Flow CMS	Max. Flow CMS	Total Volume 10^6 ltr
O1	68.15	1.608	4.884	63.556

Figure 4-7 Outlet one water profile and discharge

The figure 4.7 view of the each sub catchment used simulated with water profile plot is obtained. Form this network profile of J1 to outlet one (O1) the water elevation is overflowed at J1 (junction one), J2 and J4.

An adequate junction or not flooded junction three, five and six are showed in above figure 4.7 water elevation profile is sufficient hydraulic performance.

From the total simulated results of outfall node the average flow rate is 1.608m^3/s, maximum flow rate at ends to outlet one is 4.884m^3/s and total volume of runoff is $63.556*10^3 m^3$.

4.4.2 Result from Outlet two study

4.4.2.1 Sub catchment runoff

Topic: Subcatchment Runoff   Click a column header to sort the column.

Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Total Runoff mm	Total Runoff 10^6 ltr	Peak Runoff CMS	Runoff Coeff
S1	120.92	0.00	0.00	6.64	113.00	0.89	0.21	0.935
S2	120.92	0.00	0.00	6.72	112.85	0.59	0.13	0.933
S3	120.92	0.00	0.00	10.33	104.78	5.08	0.59	0.866
S4	120.92	0.00	0.00	10.53	104.19	4.01	0.45	0.862
S5	120.92	0.00	0.00	10.60	103.92	4.26	0.47	0.859
S6	120.92	0.00	0.00	12.53	96.71	7.24	0.63	0.800
S7	120.92	0.00	0.00	7.66	110.24	7.42	1.25	0.912
S8	120.92	0.00	0.00	7.39	109.69	5.71	0.98	0.907
S9	120.92	0.00	0.00	10.46	104.46	4.54	0.51	0.864
S10	120.92	0.00	0.00	35.53	81.87	6.25	0.94	0.677
S11	120.92	0.00	0.00	11.34	100.53	9.62	1.01	0.831
S12	120.92	0.00	0.00	11.60	99.08	9.34	0.97	0.819
S13	120.92	0.00	0.00	10.03	105.92	2.61	0.33	0.876
S14	120.92	0.00	0.00	9.36	105.85	7.20	0.99	0.875
S15	120.92	0.00	0.00	10.33	103.13	4.96	0.60	0.853
S16	120.92	0.00	0.00	9.82	102.80	13.57	1.78	0.850

Figure 4-8 Sub catchment Runoff at outlet two

From the figure 4.8 simulation results 16 sub catchments of total area **88.63ha** at outlet two study. The runoff from whole area of sub catchment by SWMM5.1 model is **11.84m3/sec and** the peak runoff generated at sub catchment (S16) is **1.78m3/sec.**

4.4.2.2 Junctions and channels flooded

From the table 4.8 result, it was observed that the drainage system channels and nodes overflow in the network simulation status report shows that sections between junctions J4, J5, J6, J8, J10, J11, J14, J15 and J18 are nodes flooded.

Topic: Node Flooding   Click a column header to sort the column.

Node	Hours Flooded	Maximum Rate CMS	Day of Maximum Flooding	Hour of Maximum Flooding	Total Flood Volume 10^6 ltr	Maximum Ponded Depth Meters
J4	2.08	1.354	0	00:30	5.550	0.000
J5	3.48	0.871	0	00:30	7.931	0.000
J6	4.90	1.303	0	00:14	14.291	0.000
J8	0.54	0.688	0	00:30	0.872	0.000
J10	1.37	0.654	0	00:18	1.430	0.000
J11	0.95	0.121	0	00:30	0.274	0.000
J14	0.32	0.367	0	00:30	0.364	0.000
J15	3.51	1.069	0	00:18	7.170	0.000
J18	0.24	0.241	0	00:36	0.113	0.000

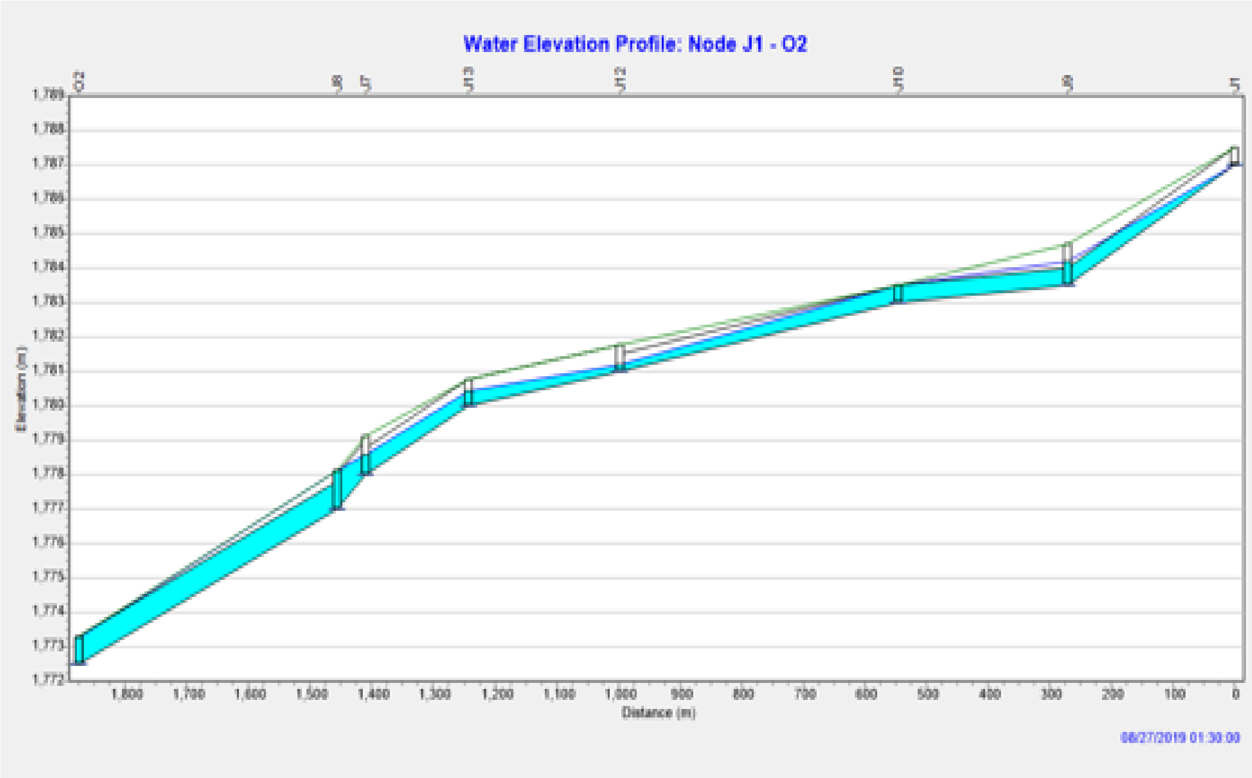
Topic: Conduit Surcharge   Click a column header to sort the column.

Conduit	Hours Both Ends Full	Hours Upstream Full	Hours Dnstream Full	Hours Above Normal Flow	Hours Capacity Limited
C3	0.01	0.01	3.45	0.01	0.01
C4	2.71	2.71	3.70	2.53	2.53
C5	0.01	0.01	0.01	0.02	0.01
C7	0.01	0.01	0.75	0.01	0.01
C8	0.01	1.09	0.01	0.90	0.01
C10	0.01	1.43	0.01	1.37	0.01
C14	0.01	0.01	7.43	0.01	0.01
C16	0.01	0.01	5.09	0.01	0.01
C17	0.01	0.01	0.32	0.01	0.01
C18	0.32	0.32	2.08	0.01	0.01
C19	0.01	0.01	3.50	0.01	0.01
C21	0.01	0.01	2.30	0.01	0.01

Figure 4-9 Junction and Channels flooded at outfall two

The channels overflows occurred are C1, C4, C7, C8, C10, C14, C16, C17, C18, C19 and C21 which are limited their capacity to convey flow in the channels infrastructure constructed.

4.4.2.3 Water profile and outfall charged



Topic: Outfall Loading | Click a column header to sort the column.

Outfall Node	Flow Freq. Pcnt.	Avg. Flow CMS	Max. Flow CMS	Total Volume 10^6 ltr
O2	99.70	1.885	4.731	55.383

Figure 4-10 Outlet two water elevation profile and outfall discharge

From the figure 4.10 shows the water profile plot is obtained from J1 to outlet two (O2) the maximum flow rate at ends of this outlet is 4.731m3/s and total volume 55.383*$10^3$$m^3$.

4.4.3 Result From Outlet Three and Four Study

4.4.3.1 Sub catchment runoff

The figure 4.11 showed that on the outlet three and four of total 50 sub catchments simulated area is 566.67ha and total runoff from the whole sub catchments is 80.51m3/sec and the maximum runoff simulated at sub catchment (S48) is 5.28m3/s .the detail result showed in (Appendix table 13)

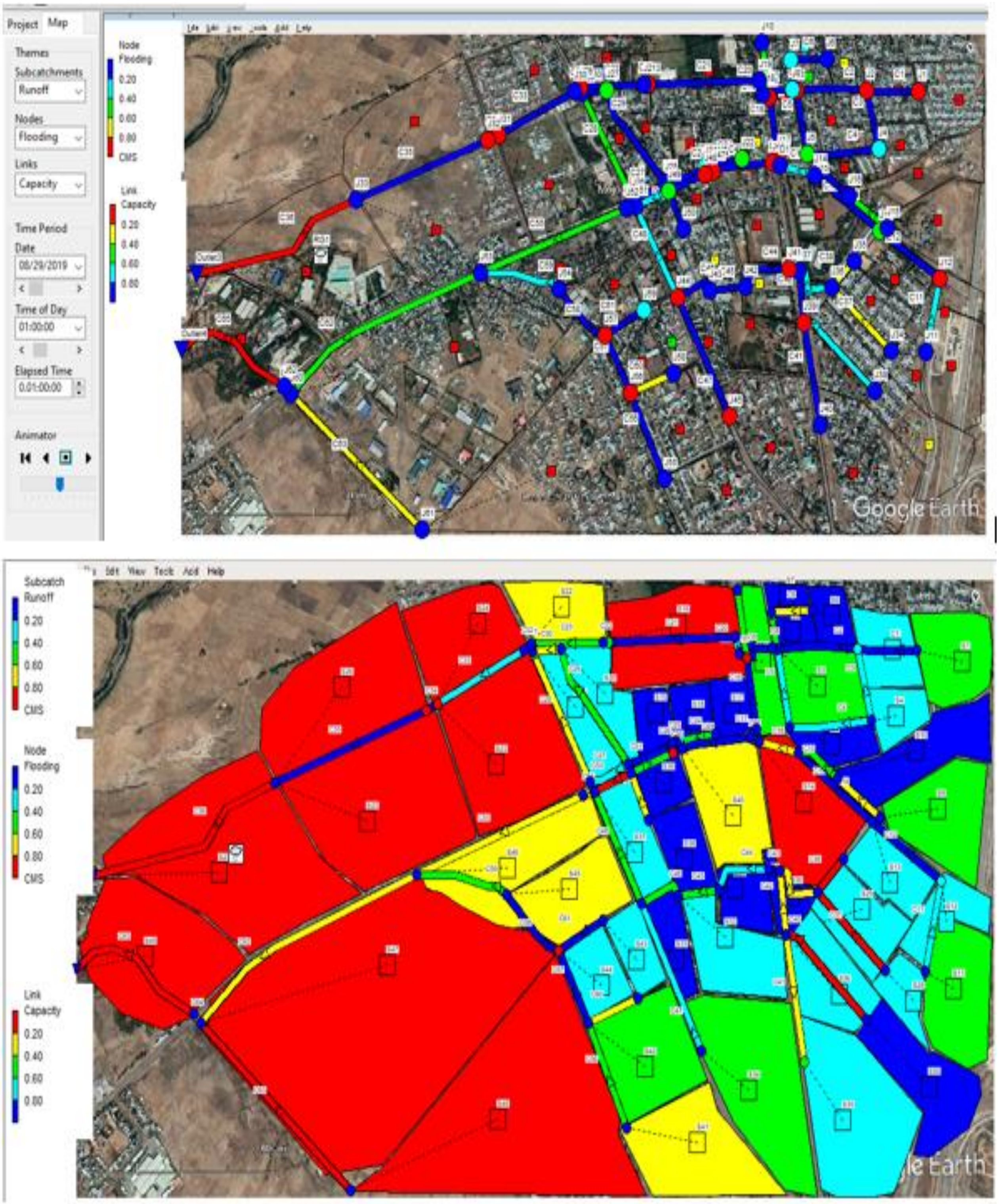


Figure 4-11 Outfall three and four runoff simulated maps

4.4.3.2 Junction and conduit flooded

From total 62 junctions and 1 divider(node) The simulated SWMM5.1 results status report showed at (Appendix 14) that sections between junctions J1 -J5, J7-J9, J12-J13, J17, J22-J24,J28 -J32, J37, J40-J41, J44 -J48, J50, J56-J57 and J59 are nodes flooded from this outlet three and four study project identified the critical problems drainage systems. From this two outfalls greater 65% junctions and conduits are flooded in the detail the result showed in (appendix table 14)

From total 66 channels simulated in this study outfalls project The flooded channels from the simulated result of C2, C4-C9, C11-C13, C17,C19-C23, C26, C27, C29-C34, C39, C41-C49, C56-C58, C61 and C66 are Overflowed. In this drainage infrastructure the capacity of channels where limited to carry discharged in the network that runoff from each sub catchments. The overflow difference on those channels are determined in different time period on. (Appendix table 15).

4.4.3.3 Water profile and outfall discharged

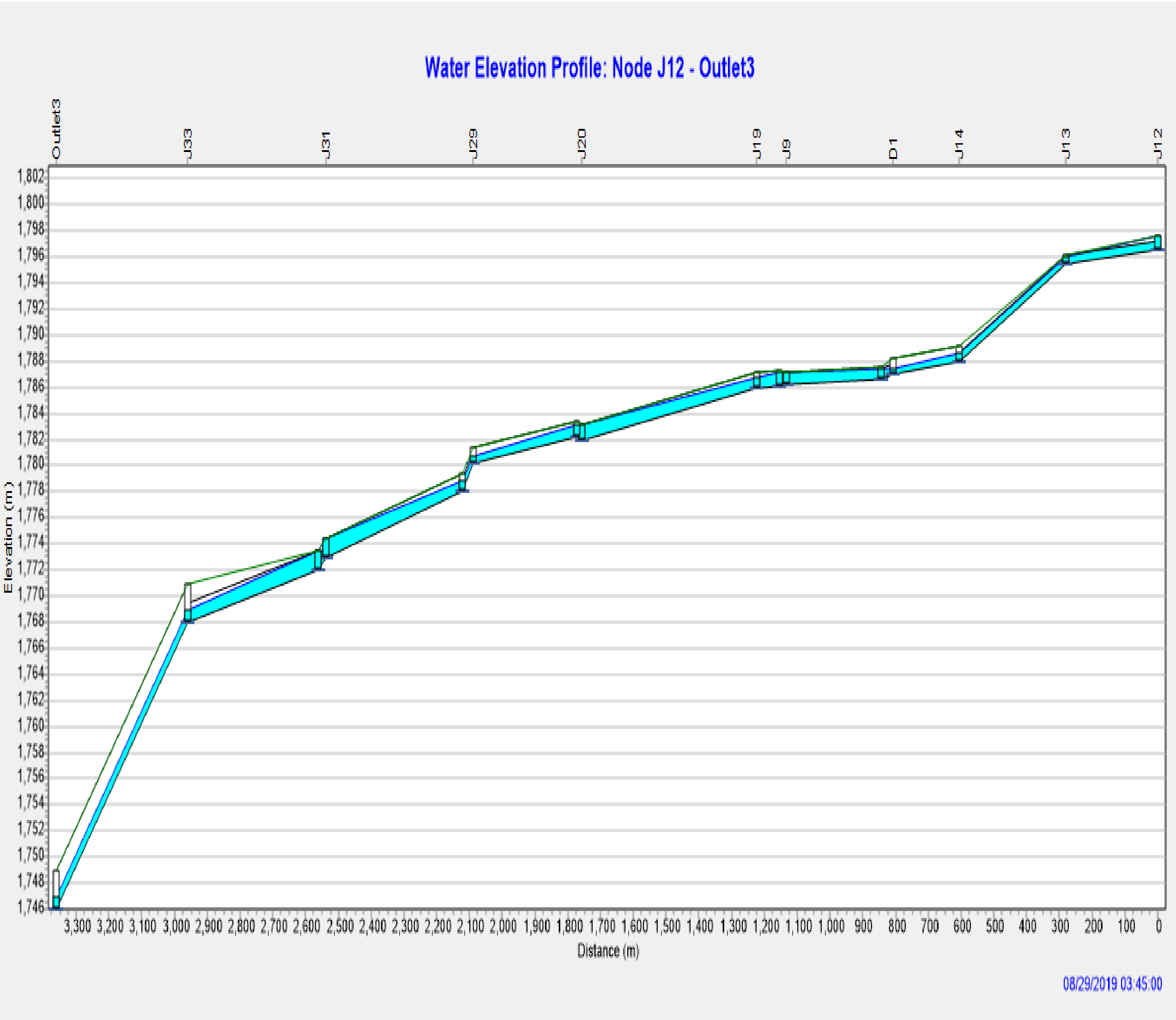


Figure 4-12 Water elevation profile from junction J12 to outfall 3

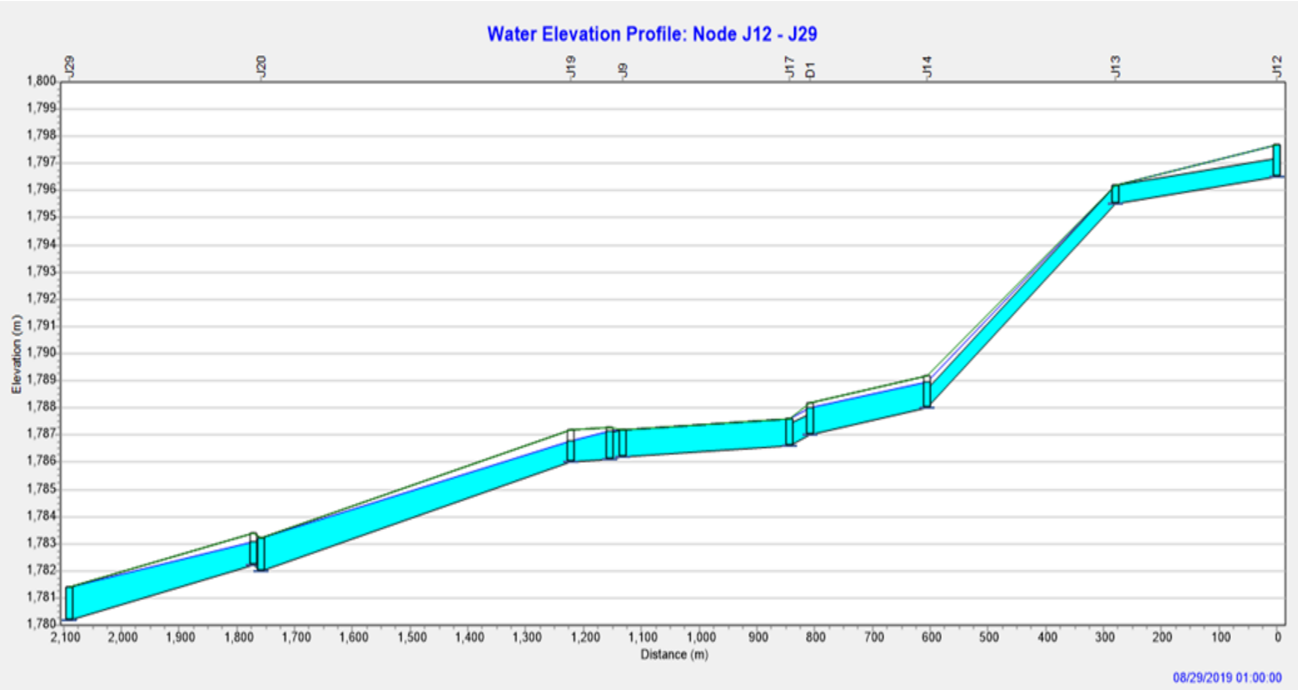


Figure 4-13 Water profile from J12 to J29 at the outlet three and four

Summary Results

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

Outfall Node	Flow Freq. Pcnt.	Avg. Flow CMS	Max. Flow CMS	Total Volume 10^6 ltr
Outlet3	99.69	9.133	13.095	181.262
Outlet4	99.69	14.405	22.244	282.889

Figure 4-14 Summary result of outfall of outlet three and four runoff

From the above figure 4.12 and 4.13 the water elevation profile shows the drainage systems networks performance and figure 4.14 summarized at ends of two outfalls is the maximum flow rate of 13.095m^3/sec and 22.244m^3/sec, total volume runoff 181.262*103m^3 and 282.889*103m^3 for outlet three and Four respectively.

Form the entirely drainage system of hydroulic perfomance capacity is checked by swmm5.1 model. The systems simulation the runoff generated from each sub catchment and flow routed through network of the channels and junction infrastructure constructed under multiple working conditions the outcomes of flooded drainage system is well evaluated.

Table 4-6: Outflow discharge and volume of runoff or each outfalls

Outfall nodes Names	Average flow rate (m^3/sec)	Maximum flow rate in m^3/sec	Total volume to outlet $*10^3m^3$
Outlet one	2.09	4.884	63.556
Outlet two	1.885	4.731	55.383
Outlet three	9.133	13.095	181.24
Outlet four	14.405	22.44	282.889
Overall outfalls results	6.88	22.44	583.07

The generally study simulated results in table 4.6 all four outlets the total simulation area of sub catchment is 745.2ha (7.452km^2), the combined drainage system are 93 nodes and 101 channels with total length canal flow routed through 26.94km and one divider to distributing the runoff flow through the networks channel constructed for outlet three and four.

Overall outfalls results in table 4.6 showed that 6.88m^3/sec average flow rate, 22.4488m^3/sec maximum flow rate and 583.07$*10^3m^3$ total volume runoff collective drainage systems involved each regular or irregular shape simulation applied

The same study using swmm5.1 model located in Athens, Greece, and covers an area of 0.89 km^2 (89 ha). Most of the drainage area is closely developed. The portion of the combined drainage network corresponding to the catchment consisted of 79 combined pipes and 112 junctions with a total length of 5.34 km. The combined drainage system comprised either regular shape or irregular simulation performed.(Eyosias B., 2018)

4.5 The alternative measurement of drainage system management

The best practice Management of reduce peak flows the attentiveness with sustainable urban drainage system(SUDS) is applied for this study to redevelop the natural system of storm water management which is Low impact development (LID) technique used to reduce the negative influence of water quantity LID measures mostly reduce the flood peak and volume by increasing the infiltration. Only the reduction of surface runoff quantity was involved in this study, and the relevant practice measure includes infiltration trench, bio-retention, vegetative swales and permeable pavement is applied.

In this study the targets of reducing peak runoff and runoff volume quantify using different techniques of the LID practices and compared with the existing condition using in EPA SWMM5.1 results.

In the urban planning technique with the key objective of urban flood management through source control of storm water runoff. Using LID in urban areas is becoming one of the most popular methods for sustainable storm water management and flood mitigation. LID is a smarter and more sustainable urban development and flood management technique. LID reduces the risk of flood through several non-structural and structural measures. Some of these measures include Green Infrastructure like as Bio retention cells, green roofs. Infiltration trench, vegetable swales and permeable pavements.

4.5.1 LID Development with SWMM 5.1 result

The LID technique results for infiltration trenches, bio retention Permeable pavement, and vegetative swales are compared the value with and without lid technique in SWMM5.1 shows the table below to modeling drainage system network.

Table 4-7: SWMM5.1 results without and with LID

Item	swmm5.1 results without LID (Current condition)	swmm5.1 results with low impact development							
		Infiltration trench		Bio-retention Cell		Permeable pavement		vegetable swales	
		value	%	Value	%	Value	%	Value	%
Peak runoff (m^3/s)	4.89	4.65	5.16	4.51	8.43	4.55	7.45	4.84	1.04
Total volume runoff at outlet one (10^3m^3)	63.56	50.46	25.96	61	**28.12**	50.16	**26.71**	61.70	3.02

The above table 4.7 For selecting the best alternative to evaluate the runoff decrease in this four out fall projects is compared in the outlet one of the project on the same sub catchment S1,S2,S3,S5,S6,S9,S10 and S12 were the reduced the total outlet one volume by **28.12%** using

Bio-retention Cell**,** 25.96 infiltration trench, **26.71%** using Permeable pavement and also on the same sub catchment reduced total volume outfall one by 3.02 % using vegetable swales.

The same study using the LID technique (Kamal Ahmed, 2017) was used to compare the simulation results before and after LID structure installation. The study area infiltration trench was indicates that total volume are reduced by 20.95% for two sub-catchments S1 of highest by area and S6 of lowest by area used.

Therefore for this study select the best alternative of LID control to Evaluate the mojo town drainage system problem mitigation measures is solved using the acceptable LID control types **Bio-retention Cell and Permeable pavement** are the most recommended from result observed in table 4.7.

Table 4-8: Summary results of SWMM5.1 With and without LID

Outfall name	Before LID use total volume of runoff	After LID control used Bio-retention Cell and Permeable pavement	
		Value	total volume of runoff Reduced by %
Outlet one total volume $(10^3)m^3$	63.56	49.82	27.58
Outlet two total volume $(10^3)m^3$	55.38	41.17	34.515
Outlet three total volume $(10^3)m^3$	181.26	137.79	31.548
Outlet four total volume $(10^3)m^3$	282.89	197.11	43.518
Sum of all outfall volume of (m3)m3	583.10	425.89	36.92

At ends of results in table 4.8 the total volume of runoff in each outlet is reduced by different percentage volume and lastly the sum of all outfall before LID control develop in the drainage system total volume of runoff $583.1*10^3$ m^3 and After LID control developed in the drainage systems used Bio-retention Cell and Permeable pavement in the sub catchments the first total volume results reduced to $425.89*10^3$ m^3 by 36.92%.

4.5.2 Modeled area of LID control

The LID control planned area is applied to sub catchment using bio retention and Permeable pavement method to the whole four outfall networks. The selected sub catchments depends the results of runoff flow from each sub catchments in the systems.

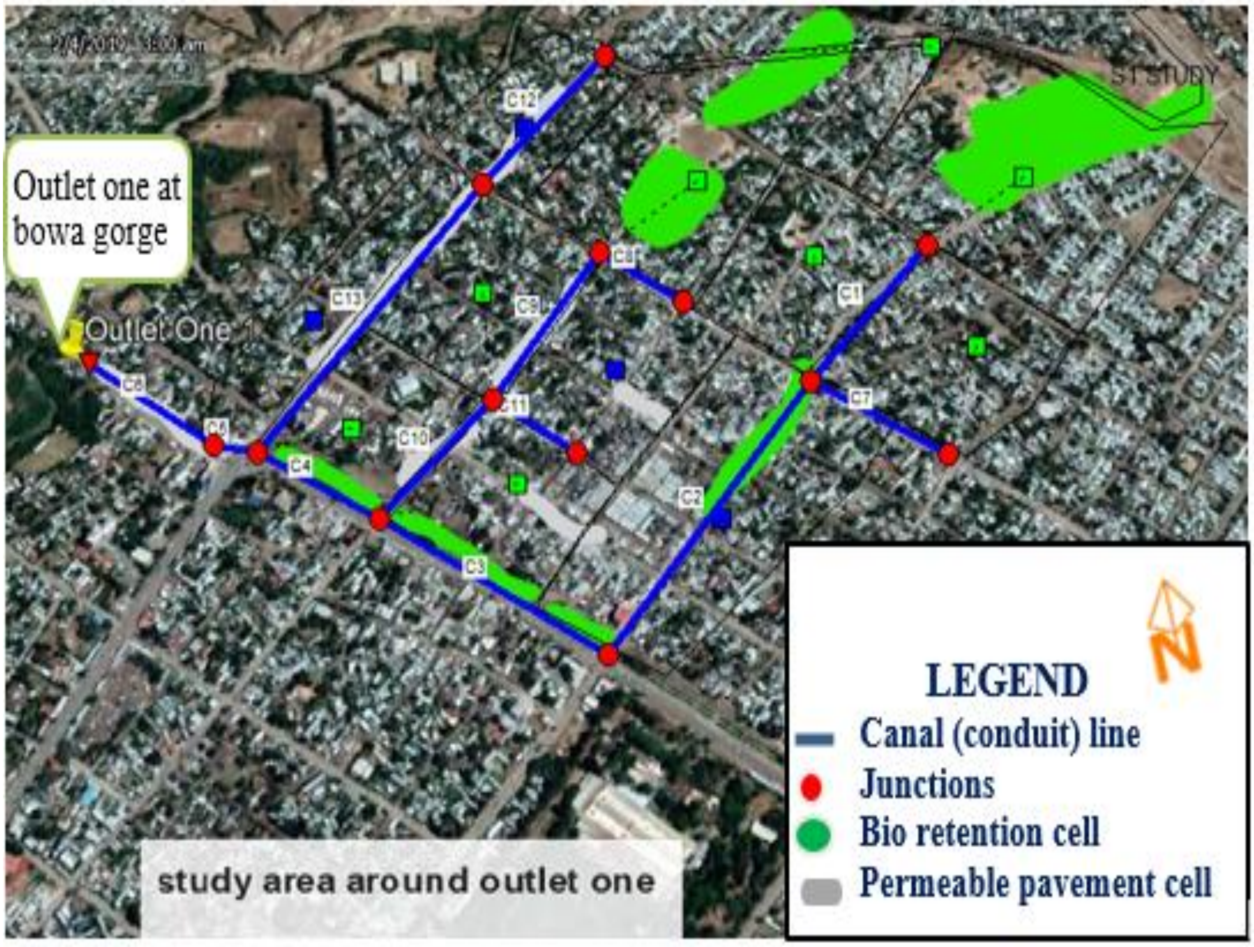


Figure 4-15 Outfall One study of LID control area modeled

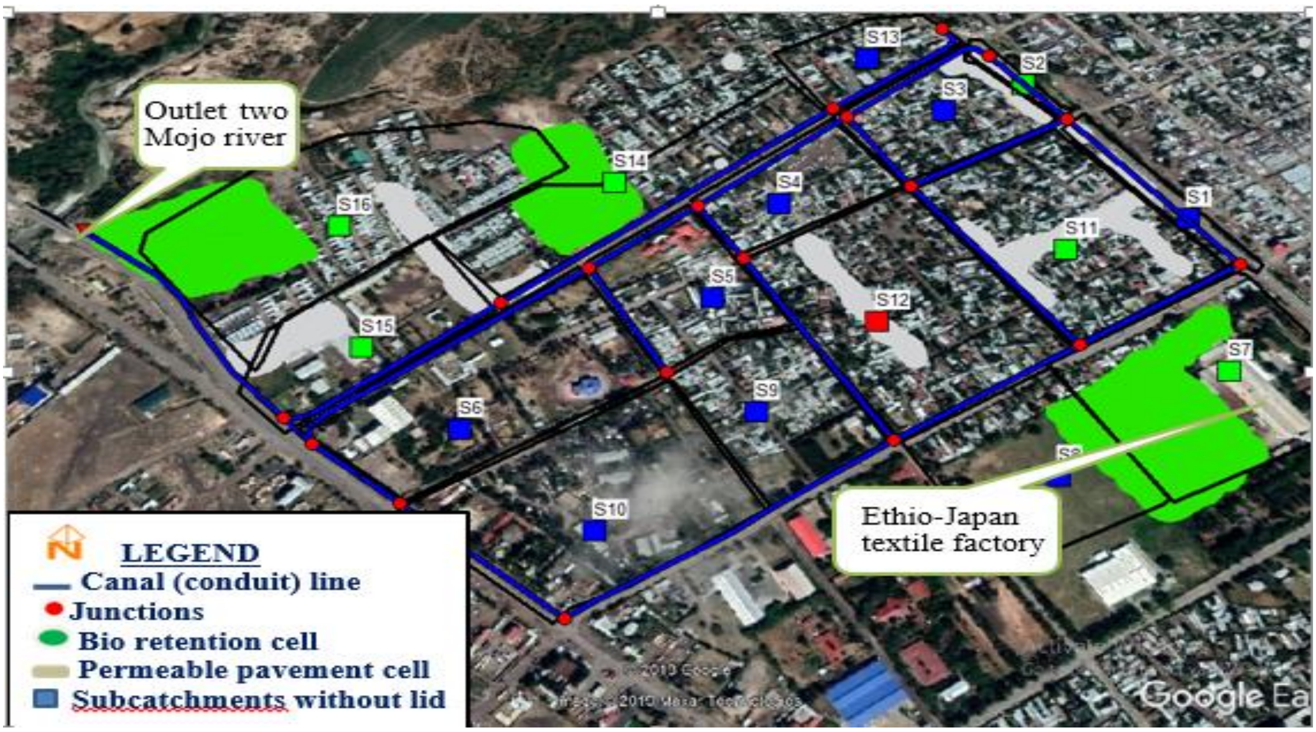


Figure 4-16 Outfall two of LID control area modeled

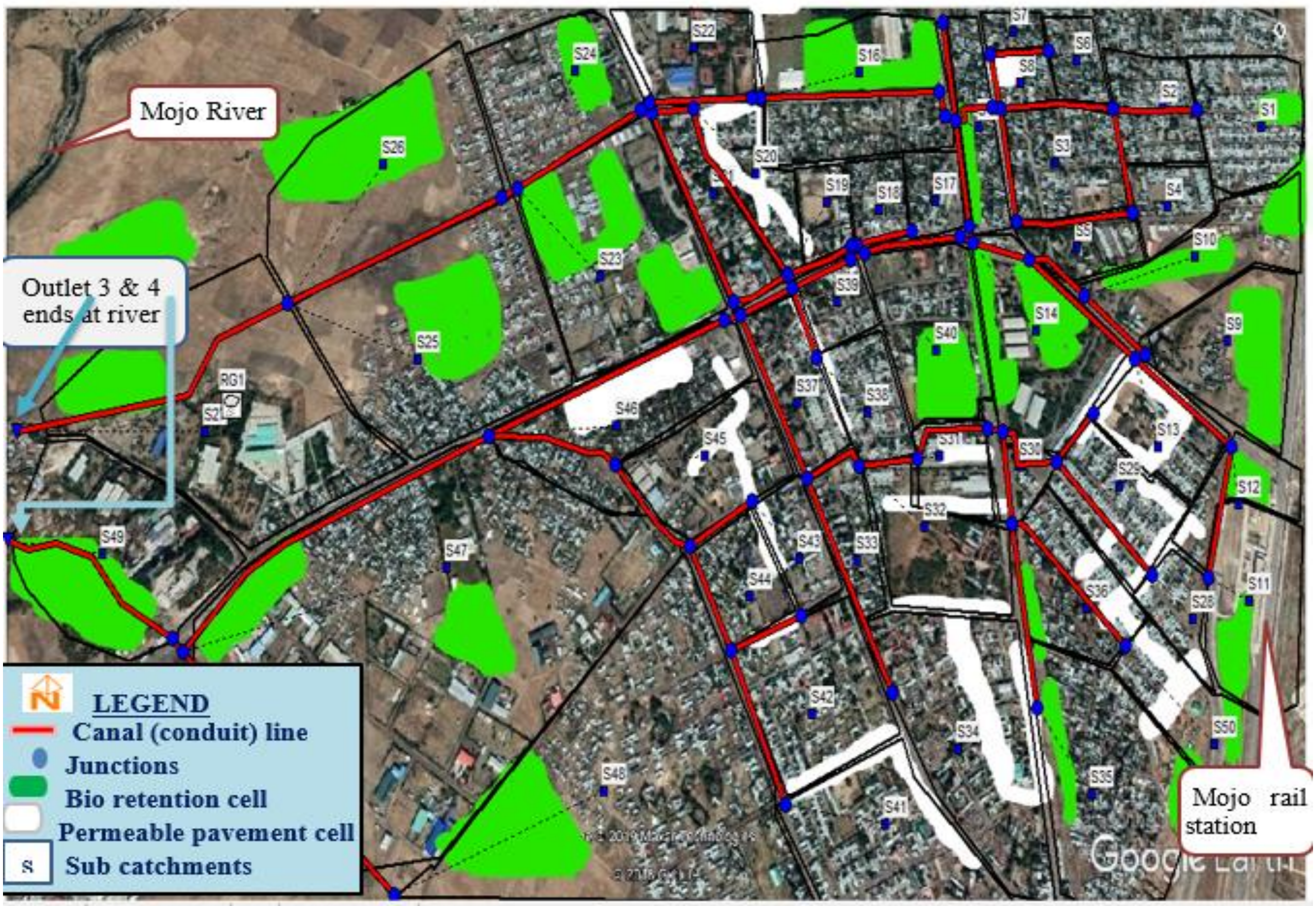


Figure 4-17 Outfall three and four study area of LID control modeled

CHAPTER 5 CONCLUSION AND RECOMMENDATION

5.1 Conclusion

The study assessed and modeled urban drainage system performance on mojo town by swmm5.1 model has totally identified the critical problem condition of drainage systems with related subjects, assessed the hydraulic performance of stormwater drainage infrastructures, and also evaluated the alternative for drainage problem mitigation measured.

The hydraulic and hydrology performance of drainage was simulated by EPA SWMM5.1 with and without LID .the calibration and validation were used for 12 days of observed flow depth compared with the model result. For calibration used 6days flow depth data recorded is calculated by the manning equation is 1.473 m^3/s and compared by adjusted sensitive parameters of model results 1.403m3/s on the same site are almost equivalent. The other 6 days flow depth data used for validation without changed the sensitive parameters of model results parallel with observed are verified.

The swmm5.1 model performance for the certified area was tested by the goodness of fit using the coefficient of determination (R^2) =0.97, the Nash –Sutcliffe coefficient (NSE) =0.96, and Relative error (RE) =11.16% which are simulated values indicated in an acceptable range.so that the SWMM5.1 model is a powerful application for assessed and modeled urban drainage system performance and also controls the overflow drainage infrastructure for this study.

The simulated area in this study subdivide to four outlets of the total area of sub catchment is 745.2ha , the combined drainage system are 93 nodes and 101 channels with total length canal flow routed through 26.94km and one divider to distribute the runoff flow for outlet three and four. According to simulated result greater than 65% of drainage infrastructure are over flooded .From the result of SWMM5.1 simulated all total sub catchment average flow rate 6.88m^3/sec, maximum flow rate 22.44m^3/sec and total volume of all outfall 583.1*10^3 m^3

The LID control used in this study for reduced the peak runoff overflow and select the best alternative on the outlet one of the project in the same sub-catchments S1, S2, S3, S5, S6, S9, S10, and S12 were the reduced the total runoff at outlet one by 28.12% using bio-retention Cell, 25.9% infiltration trench, 26.71% permeable pavement and 3.02 % vegetable swales. From all select highest value of runoff decreased by Bio-retention Cell and permeable pavement for total study applied and after selected two LID control types to the total volume of all outfall is reduced from $583.11*10^3$ m^3 to $425.89*10^3$ m^3 by 36.92% from the total study.

Commonly, the mojo city urban drainage systems performance infrastructure's overflow assessed and controlled for the problem occurred the usage of the excellent alternative mitigation measures by improved LID control. For all have a look at regions, the LID technique of Bio-retention cellular and permeable pavement are the maximum acceptable of sustainable drainage systems management.

5.2 Recommendation

Upgrading the drainage system layout as per to the master plan of mojo town and applying the updated IDF curve for town only and urban stormwater drainage design manual of the country to assessing accurate measurement and improve storm drainage systems of the area

This study properly considered only the effect of LID locations and sizes in the modeling processes by introducing the relative performance of each LID. Therefore when planning and designing LID systems considering the first count the cost and decide if it is difficult to use another alternative or if it is improvement connecting the LID techniques in the all drainage network to reduce the overflow runoff.

The current drainage network line totally overflows so we recommend that removing the silt from the channel, reconstructed the failed infrastructures and the existing drainage system must be properly controlled management system using LID technique awareness is providing for any concerned body on this area.

Applying this research study results for future plans of the urban drainage systems in mojo town in order to standardize all sub-catchment of drainage networks in a Sustainable way.

The upcoming study should focus on concerning model parameters of LID practices and the predictable measures further **with cost-benefit** studies must be included in the design process, in order to determine the feasibility of predictable and LID solutions regarding the success of sustainability goals.

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APPENDIX

Appendix Table 1: Double mass curve after consistency checked

Year	Annually RF (mm) of MOJO station	Accumulative annual rainfall MOJO station (mm)	DEBRZEIT station (mm)	NEZERET station (mm)	KOKA station (mm)	Average RF of the 3 Neighboring stations	Average Accumulative Rainfall of nearby station(mm)
2017	91.281	91.281	86.127	83.107	82.641	83.958	83.958
2016	130.400	221.681	133.117	81.522	76.280	96.973	180.931
2015	71.617	293.298	82.125	59.813	48.860	63.599	244.530
2014	72.925	366.223	196.400	70.650	78.029	115.026	359.557
2013	91.367	457.589	148.304	75.350	63.725	95.793	455.350
2012	110.650	568.239	93.958	101.142	84.967	93.356	548.705
2011	68.158	636.398	94.101	62.058	79.250	78.470	627.175
2010	92.533	728.931	90.874	86.292	93.342	90.169	717.344
2009	92.358	821.289	64.280	49.058	57.458	56.932	774.276
2008	120.117	941.406	80.497	92.267	69.408	80.724	855.000
2007	83.467	1024.873	79.894	76.017	69.417	75.109	930.110
2006	99.925	1124.798	68.904	59.967	64.700	64.524	994.633
2005	131.768	1256.566	86.125	93.892	83.958	87.992	1082.625
2004	99.723	1356.289	60.172	51.325	51.300	54.266	1136.891
2003	88.342	1444.631	81.736	70.800	135.475	96.004	1232.894
2002	47.025	1491.656	63.717	88.558	97.058	83.111	1316.005
2001	65.675	1557.331	67.863	70.775	67.883	68.840	1384.846
2000	71.008	1628.339	81.175	80.133	99.475	86.928	1471.773
1999	98.420	1726.759	48.825	73.050	73.250	65.042	1536.815
1998	86.158	1812.918	59.958	84.267	58.475	67.567	1604.382
1997	63.358	1876.276	34.617	70.858	47.775	51.083	1655.465
1996	87.742	1964.018	34.617	70.858	47.775	51.083	1706.548
1995	52.839	2016.857	40.433	68.633	34.833	47.967	1754.515
1994	59.469	2076.325	40.433	68.633	34.833	47.967	1802.482
1993	57.367	2133.692	68.475	69.917	145.250	94.547	1897.029
1992	66.500	2200.192	46.067	73.033	184.550	101.217	1998.246
1991	73.217	2273.409	48.516	76.761	44.683	56.653	2054.899
1990	75.792	2349.201	72.171	89.567	100.700	87.479	2142.378
1989	86.133	2435.335	68.500	62.908	69.417	66.942	2209.320
1988	88.458	2523.793	71.567	64.942	68.242	68.250	2277.570
1987	72.083	2595.876	69.192	72.325	26.067	55.861	2333.431
1986	47.650	2643.526	71.233	61.342	14.108	48.894	2382.325

From the above figure 4.1 in the result and discussion part the consistency check is more related to the original data but there is some correction of rainfall data for mojo station is need at year

2008-2012, 2005-2003, 1998-1994 and 1991-1990 by correction value of=1.1244/1.1186 in the appendix table 1

=1.005185 is the correction value of the slope 1990 to 2012 year rainfall data

Appendix Table 2: Data quality check by relative standard error

year	24hr max annual rainfall (mm) MOJO station (X)	y=LogX	Y-Ym	$(Y-Ym)^2$	$(y-ym)^3$	$(x-xm)^2$
1986	47.65	1.678	-0.226	0.051	-0.011	1235.523
1987	72.08	1.858	-0.046	0.002	0.000	114.847
1988	88.46	1.947	0.043	0.002	0.000	32.017
1989	86.13	1.935	0.032	0.001	0.000	11.111
1990	76.19	1.882	-0.022	0.000	0.000	43.737
1991	73.60	1.867	-0.037	0.001	0.000	84.688
1992	66.50	1.823	-0.081	0.007	-0.001	265.690
1993	57.37	1.759	-0.145	0.021	-0.003	646.854
1994	59.78	1.777	-0.127	0.016	-0.002	530.005
1995	53.11	1.725	-0.178	0.032	-0.006	881.278
1996	88.20	1.945	0.042	0.002	0.000	29.138
1997	63.69	1.804	-0.100	0.010	-0.001	365.276
1998	86.61	1.938	0.034	0.001	0.000	14.488
1999	98.42	1.993	0.089	0.008	0.001	243.987
2000	71.01	1.851	-0.052	0.003	0.000	139.043
2001	65.68	1.817	-0.086	0.007	-0.001	293.266
2002	47.03	1.672	-0.231	0.053	-0.012	1279.851
2003	88.80	1.948	0.045	0.002	0.000	36.013
2004	100.24	2.001	0.097	0.009	0.001	304.204
2005	132.45	2.122	0.218	0.048	0.010	2465.489
2006	99.93	2.000	0.096	0.009	0.001	293.266
2007	83.47	1.922	0.018	0.000	0.000	0.444
2008	120.12	2.080	0.176	0.031	0.005	1392.534
2009	92.84	1.968	0.064	0.004	0.000	100.773
2010	93.01	1.969	0.065	0.004	0.000	104.336
2011	68.51	1.836	-0.068	0.005	0.000	204.125
2012	111.23	2.046	0.143	0.020	0.003	808.002
2013	91.37	1.961	0.057	0.003	0.000	73.388
2014	72.93	1.863	-0.041	0.002	0.000	97.516
2015	71.62	1.855	-0.049	0.002	0.000	125.067
2016	130.40	2.115	0.212	0.045	0.009	2265.760
2017	91.28	1.960	0.057	0.003	0.000	71.930
sum	2649.67	60.916		0.406	-0.006	14553.647
average(xm)	82.80	1.9036				
	xm=82.8					
	ym=1.90362					
st,dev xi	21.66731					
st,dev yi	0.11444					
Skweness coff(cs)	-0.12785					

Before proceeding to the ather other analysis the aduquacy of rainfall data series be checked

And it should be realized. The data series should be adequate if relative standard error ≤10% whereas relative standard error =se/xm se=standard deviation/ (number of year) ½= 3.83019

Relative standard error=3.83019/xm*100

=4.625832947%≤10% ok therefore the data is reliable and adequate

Appendix Table 3: Frequency Analysis and IDF Development

	24Hr Max. Annual RF (mm)	5min =0.08hr	10 min =0.17hr	15 min = (0.25 hr.)	30min =0.50 hr.	1hr	2hr	3hr	6hr	12hr	24hr
Year											
1986	47.650	6.93	11.60	15.01	21.43	27.69	33.14	35.92	40.15	43.97	47.65
1987	72.083	10.48	17.56	22.71	32.42	41.89	50.14	54.34	60.73	66.52	72.08
1988	88.458	12.86	21.54	27.87	39.79	51.41	61.52	66.69	74.53	81.63	88.46
1989	96.133	13.97	23.41	30.29	43.24	55.87	66.86	72.47	80.99	88.71	96.13
1990	76.160	11.07	18.55	24.00	34.26	44.26	52.97	57.41	64.17	70.28	76.16
1991	73.579	10.70	17.92	23.18	33.10	42.76	51.18	55.47	61.99	67.90	73.58
1992	66.500	9.67	16.20	20.95	29.91	38.65	46.25	50.13	56.03	61.37	66.50
1993	57.367	8.34	13.97	18.08	25.80	33.34	39.90	43.25	48.33	52.94	57.37
1994	59.78	8.69	14.56	18.84	26.89	34.74	41.58	45.07	50.37	55.16	59.78
1995	53.11	7.72	12.94	16.74	23.89	30.87	36.94	40.04	44.75	49.01	53.11
1996	88.20	12.82	21.48	27.79	39.67	51.26	61.34	66.49	74.31	81.39	88.20
1997	63.69	9.26	15.51	20.07	28.65	37.01	44.30	48.01	53.66	58.77	63.69
1998	86.61	12.59	21.09	27.29	38.96	50.33	60.24	65.29	72.97	79.92	86.61
1999	98.42	14.31	23.97	31.01	44.27	57.20	68.45	74.20	82.92	90.82	98.42
2000	71.01	10.32	17.29	22.37	31.94	41.27	49.39	53.53	59.83	65.53	71.01
2001	65.68	9.55	15.99	20.69	29.54	38.17	45.68	49.51	55.33	60.60	65.68
2002	47.03	6.84	11.45	14.82	21.15	27.33	32.71	35.45	39.62	43.39	47.03
2003	88.80	12.91	21.63	27.98	39.94	51.61	61.76	66.94	74.82	81.94	88.80
2004	100.24	14.57	24.41	31.59	45.09	58.25	69.72	75.57	84.45	92.50	100.24
2005	132.45	19.25	32.26	41.74	59.58	76.98	92.12	99.85	111.59	122.23	132.45
2006	99.93	14.52	24.34	31.49	44.95	58.07	69.50	75.33	84.19	92.21	99.93
2007	83.47	12.13	20.33	26.30	37.54	48.51	58.05	62.92	70.32	77.02	83.47
2008	120.12	17.46	29.25	37.85	54.03	69.81	83.54	90.55	101.20	110.84	120.12
2009	92.84	13.49	22.61	29.25	41.76	53.95	64.57	69.99	78.22	85.67	92.84
2010	93.02	13.52	22.65	29.31	41.84	54.06	64.69	70.12	78.37	85.83	93.02
2011	68.51	9.96	16.69	21.59	30.82	39.82	47.65	51.65	57.72	63.22	68.51
2012	111.23	16.17	27.09	35.05	50.03	64.64	77.36	83.85	93.71	102.64	111.23

2013	91.37	13.28	22.25	28.79	41.10	53.10	63.55	68.88	76.98	84.31	91.37
2014	72.93	10.60	17.76	22.98	32.80	42.38	50.72	54.98	61.44	67.29	72.93
2015	71.62	10.41	17.44	22.57	32.21	41.62	49.81	53.99	60.34	66.09	71.62
2016	130.40	18.95	31.76	41.09	58.65	75.78	90.70	98.30	109.86	120.33	130.40
2017	91.28	13.27	22.23	28.76	41.06	53.05	63.49	68.81	76.91	84.23	91.28
	Average	**12.08**	**20.24**	**26.19**	**37.38**	**48.30**	**57.81**	**62.66**	**70.02**	**76.70**	**83.11**
	St,dev (Sy)	3.17	5.31	6.87	9.80	12.66	15.15	16.43	18.36	20.11	21.79
	Skew Coeff (Cs)	0.47	0.47	0.47	0.47	0.47	0.47	0.47	0.47	0.47	0.47

From the Appendix table 3 shows Mean annual rainfall (mm) l volume for the short durations

The Reducing the daily precipitation data to smaller durations use appendix table 4

Appendix Table 4: Reduction daily precipitation for each time t

		$R_t = \frac{t}{24}\frac{(b+24)^n}{(b+t)^n} * R_{24}$	
	given t	Rt	t in hr.
	5min	0.145356	0.0833
	10min	0.243544	0.1667
	15min	0.3151	0.25
	30min	0.449804	0.5
	1hr	0.581146	1
	2hr	0.69552	2
	3hr	0.753864	3
	6hr	0.842518	6
	12hr	0.922786	12
	24hr	1	24

Whereas $R_t = \frac{t}{24}\frac{(b+24)^n}{(b+t)^n} * R_{24}$ = Rain fall Redaction formula

n = 0.9 and b = 0.3 is content

Probability distribution and development of IDF curves

There are two options for this frequency analysis these are:

1) Fit a theoretical Extreme Value (EV) distribution (e.g., Gumbel Type I) to the observations and then use the theoretical distribution to estimate the rainfall events Associated with given exceedance probabilities.

2) Use an empirical plotting position approach to estimate the exceedance probabilities

Based on the observations.

Appendix Table 5: Probability distribution of extreme value XT in (mm)

PROBABILITY DISTRIBUTION AND GOODENSE FIT DISTRIBUTION						
Distribution	2 year	5 year	10 year	25 year	50 year	100 year
Normal	83.11	101.45	111.04	121.34	127.87	133.82
Lognormal	80.37	100.46	112.91	127.98	138.57	148.97
Log-Pearson	80.93	100.46	112.36	126.01	138.57	148.97
Gumbel(EVI)	79.53	98.79	111.54	127.65	139.60	151.46
Pearson typeIII	81.40	100.77	111.91	124.55	135.23	141.22
Weibull's Formula:	86.61	99.93	120.12	130.40	132.45	132.45

Testing The Goodness of Fit of Probability Distribution the best fit distributions decided

By chi-square test for statistic is given by the equation

$$x^2 = \sum_{i=1}^{k} \frac{(O_i - E_i)^2}{E_i}$$

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

Appendix Table 6: Tested Goodness of Fit by Probability Distribution

return period(T)	Expected rainfall(mm)=Ei					observed rainfall(mm)=Oi
	Normal	Lognormal	Log-Pearson	Gumbel(EVI)	Pearson typeIII	Weibull's Formula:
2 year	83.113	80.372	80.928	79.534	81.405	86.606
5 year	101.447	100.464	100.464	98.790	100.766	99.925
10 year	111.042	112.908	112.356	111.539	111.905	120.117
25 year	121.338	127.981	126.011	127.647	124.554	130.400
50 year	127.871	138.573	138.573	139.597	135.225	132.454
100 year	133.816	148.970	148.970	151.459	141.222	132.454

Appendix Table 7: Chi - Square Test Result in dimensionless of X^2 values

Chi - Square Test Result in different return period of probability distributions					
Return period(T)	Normal v	Lognormal	Log-Pearson	Gumbel(EVI)	Pearson typeIII
2 year	0.15	0.48	0.40	0.63	0.33
5 year	0.02	0.00	0.00	0.01	0.01
10 year	0.74	0.46	0.54	0.66	0.60
25 year	0.68	0.05	0.15	0.06	0.27
50 year	0.16	0.27	0.27	0.37	0.06
100 year	0.01	1.83	1.83	2.38	0.54
Total CHI-SQUARE VALUE=X^2	1.77	3.09	3.19	4.11	1.82

All probability distribution functions were compared by chi square test of goodness of fit

And then selecting the function that gave the smallest chi-square value determined the best probability function.

Therefore for this study the best fit probability distribution is **normal distribution**

Appendix Table 8: Rainfall depth (mm) by normal distribution at time t (yrs.)

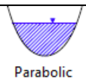
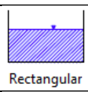
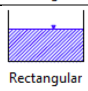
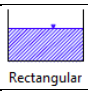
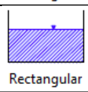
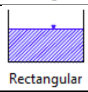
Duration(min t)	Duration(hr.)	2 year	5 year	10 year	25 year	50 year	1oo year
5	0.08	12.08	14.75	16.14	17.64	18.59	19.45
10	0.17	20.24	24.71	27.04	29.55	31.14	32.59
15	0.25	26.19	31.97	34.99	38.23	40.29	42.17
30	0.5	37.38	45.63	49.95	54.58	57.52	60.19
60	1	48.30	58.96	64.53	70.52	74.31	77.77
120	2	57.81	70.56	77.23	84.39	88.94	93.07
180	3	62.66	76.48	83.71	91.47	96.40	100.88
360	6	70.02	85.47	93.55	102.23	107.73	112.74
720	12	76.70	93.61	102.47	111.97	118.00	123.48
1440	24	83.11	101.45	111.04	121.34	127.87	133.82
	MAX	83.11	101.45	111.04	121.34	127.87	133.82

Appendix Table 9: Mojo Idf Curve (mm/hr.) developed

Duration(mint)	Duration(hr.)	2 year	5 year	10 year	25 year	50 year	1oo year	x axis
5	0.08	151.01	184.32	201.76	220.46	232.34	243.14	
10	0.17	119.07	145.33	159.08	173.83	183.19	191.71	
15	0.25	104.76	127.86	139.96	152.93	161.17	168.66	
30	0.5	74.76	91.26	99.89	109.16	115.03	120.38	
60	1	48.30	58.96	64.53	70.52	74.31	77.77	
120	2	28.90	35.28	38.62	42.20	44.47	46.54	
180	3	20.89	25.49	27.90	30.49	32.13	33.63	
360	6	11.67	14.25	15.59	17.04	17.96	18.79	
720	12	6.39	7.80	8.54	9.33	9.83	10.29	
1440	24	3.46	4.23	4.63	5.06	5.33	5.58	
					y axis			

The Idf developed in appendix 9 for given return period is used in swmm5.1 as input data

Appendix Table 10: Measured input parameters of model in outfall 1

Name of junction code	invert elevation of nodes	Name of canal code	Inlet node of canal	Outlet nodes of canal	Elven b/n nodes	channel demission parameters						channel slope= eleven/length	Roughness depend on material (N)
						Type of channel	Bottom width (m)	TOP width (m)	Max depth (m)	Side Slope H/V	Length conduit(m)		
J1	1787	C1	J1	J2	1			1.5	0.5		341	0.0121	0.03
J2	1786	c2	J2	J3	2		0.5	0.5	0.6		188	0.0116	0.024
J3	1784	c3	J3	J4	2		0.6	0.6	0.6	0.33	268	0.0045	0.024
J4	1782	c4	J4	J5	1		0.7	0.7	1	0.25	273	0.0054	0.024
J5	1780	c5	J5	J6	0.2		>>	>>	>>		180	0.008	0.024
J6	1779	c6	J6	J7	0.5		0.7	0.7	1.1	0.167	416	0.0128	0.024

J7	1778.5	c7	J7	J8	0.5	Circular			D= 1m		36.5	0.0009	0.012
J8	1778	c8	J8	O2	5.5	Trapezoidal					270	0.0025	0.024
J9	1783.5	c9	J1	J9	3.5		1.3	1.8	0.8		267	0.0173	0.024
J10	1783	c10	J9	J10	0.5	Parabolic		1.5	0.5		276	0.0099	0.03
J11	1781.5	c11	J10	J12	0.5	Parabolic		1.5	0.5		452	0.0039	0.03
J12	1781	c12	J11	J12	1.7	Trapezoidal	0.85	1.6	0.75		246	0.0061	0.024
J13	1780	c13	J13	J7	1.5	Trapezoidal	0.85	1.6	0.75		157	0.0146	0.024
J14	1782.5	C14	J11	J6	2	Parabolic		1	0.5		209	0.0095	0.03
J15	1781.5	C15	J10	J15	1.5	Parabolic		>>	>>		344	0.0044	0.03
J16	1783.5	C16	J15	J5	1.5	Parabolic		>>	>>		110	0.014	0.03
J17	1782	C17	J9	J14	2	Rectangular	1	1	1.2		337	0.006	0.024
J18	1778.8	C18	J14	J4	0.6	Rectangular	1	1	1.2		152	0.004	0.024
O2	1772.5	C19	J2	J14	3.5	Parabolic		1.5	0.5		271	0.013	0.03
		C20	J16	J17	1.5	Rectangular	0.5	0.5	0.5		225	0.0067	0.024
		C21	J17	J18	4.2	Rectangular	0.5	0.5	0.6		586	0.007	0.024
		C22	J18	J8	0.8	Rectangular	0.7	0.7	1		305	0.003	0.024

Appendix Table 11: Measured input parameters of model in outfall 2

Name of junction code	invert elevation of nodes	Name of canal code	Inlet node of canal	Outlet nodes of canal	Elven b/n nodes	channel demission parameters						channel slope= elev/length	Roughness depend on material (N)
						Type of channel	Bottom width (m)	TOP width (m)	Max depth (m)	Side Slope H/V	Length conduit (m)		
J1	1795	C1	J1	J2	3.2	Parabolic		1	0.4		265	0.0121	0.03
J2	1791.5	c2	J2	J3	5.2	Parabolic		1	0.5		447	0.0116	0.03

J3	1786.6	c3	J3	J4	1.6	Trapezoidal	0.8	1.4	1	0.33	352	0.0045	0.024
J4	1785	c4	J4	J5	1	Trapezoidal	1	1.6	1.2	0.25	187	0.0054	0.024
J5	1784	c5	J5	J6	0.2	Closed Rectangular	1	1.2	1.2		25	0.008	0.02
J6	1783.5	c6	J6	O1	3.8	Trapezoidal	1.2	1.6	1.4	0.16 7	297	0.0128	0.024
J7	1792	c7	J7	J2	0.2	Parabolic		1	0.4		219	0.0009	0.03
J8	1792.3	c8	J8	J9	0.9	Parabolic		1	0.5		321	0.0025	0.03
J9	1792	c9	J9	J10	4.5	Parabolic		1	0.5		260	0.0173	0.03
J10	1787	c10	J10	J4	2	Rectangular	1	1	1		202	0.0099	0.024
J11	1787.5	c11	J11	J10	0.5	Parabolic		1	0.5		130	0.0039	0.03
J12	1792	c12	J12	J13	1.7	Parabolic		1	0.5		277	0.0061	0.03
J13	1790.8	c13	J13	J5	6.8	Parabolic		1.5	0.5		465	0.0146	0.03
O1	1779												

Appendix Table 12: Measured input parameters of model in outfall 3 and 4

Input data used in swmm5.1 at Outlet 3 and 4 canals and junctions parameter													
Name of junction code	invert elevation of nodes	Name of canal code	Inlet node of canal	Outlet nodes of canal	Elven b/n nodes	channel demission parameters						channel slope= elev/length	Roughness depend on material (N)
						Type of channel	Bottom width (m)	TOP width (m)	Max depth (m)	Side Slope H/V(m)	Length of conduit (m)		
J1	1796	C1	J1	J2	4	Parabolic		1	0.5		228	0.0175	0.03
J2	1792	c2	J2	J3	4	Trapezoidal	0.8	1.4	1.2	0.2 5	341	0.0117	0.022
J3	1788	c3	J3	J4	5	Parabolic		1.5	0.5		326	0.015	0.03
J4	1793	c4	J4	J5	4	>>		>>	>>		273	0.0146	0.03
J5	1789	c5	J5	J3	1	Trapezoidal	0.7	0.8	1.4	0.4 4	310	0.0032	0.022

J6	1789	c6	J6	J7	0.5	>>	0.8	1.4	1.2	0.25	171	0.0029	0.022
J7	1788.5	c7	J7	J8	1	>>	0.8	1.4	1.2	0.25	150	0.0067	0.022
J8	1787.5	c8	J3	J8	0.5	Closed Rectangular	1.2	1.2	0.8		15	0.03	0.018
J9	1786.2	c9	J8	J9	3.5	Trapezoidal	0.9	1	1	0.27	372	0.0094	0.022
J10	1788	c10	J9	J10	1.8	Rectangular	0.5	0.5		0.8	384	0.00469	0.02
J11	1805	c11	J11	J12	4	Circular			D= 1		372	0.0108	0.011
J12	1801	c12	J12	J13	6	Rectangular	0.6	0.6	0.75		279	0.0215	0.02
J13	1795	c13	J13	J7	6.5	Closed Rectangular	0.6	0.6	0.7		327	0.019	0.015
J14	1788	C14	J15	J16	7	Rectangular	1	1	1.5		167	0.04	0.025
J15	1795	C15	J16	J14	2	Circular			D= 1.2		173	0.0116	0.011
J16	1790	C16	J14	JD1	1	Irregular		2.4	1.2		202	0.00495	0.45
J17	1786.6	C17	D1	J17	0.4	Closed Rectangular	1	1	0.8		37	0.024	0.015
J18	1786.1	C18	J17	J9	0.4	Trapezoidal	1.2	1	1.6	0.2	290	0.0014	0.025
J19	1786	C19	J9	J18	0.4	Closed Rectangular	1	1	1.2		21	0.019	0.02
J20	1781.8	C20	J18	J19	0.5	>>	>>	>>	>>		69	0.0075	0.02
J21	1782	C21	J19	J20	4.2	Trapezoidal	0.8	1.4	1.2	0.25	535	0.00785	0.025
J22	1784	C22	J20	J21	0.2	>>	>>	>>	>>	0.25	13	0.0154	0.025
J23	1783	C23	J21	J29	3	>>	>>	>>	>>	0.25	319	0.0094	0.025
J24	1782.5	C24	J22	J23	0.5	Rectangular	0.5	0.5	0.6		149	0.0034	0.02
J25	1782	C25	J23	J24	0.5	>>	>>	>>	>>		10	0.05	0.022
J26	1781	C26	J24	J25	0.5	>>	>>	>>	>>		195	0.0025	0.022
J27	1780	C27	J25	J26	1	>>	>>	>>	>>		179	0.0056	0.022
J28	1779.2	C28	J26	J27	1	Trapezoidal	0.7	1.2	1.4	0.29	543	0.0018	0.025
J29	1779	C29	J25	J27	3	>>	>>	>>	>>		518	0.0058	0.025
J30	1778	C30	J27	J28	0.8	>>	>>	>>	>>		120	0.00667	0.025

J31	1773	C31	J28	J29	0.8	Closed Rectangular	1	1	0.8		18	0.044	0.015
J32	1772	C32	J29	J30	1	>>	1.2	1.2	1		32	0.0313	0.015
J33	1768	C33	J30	J31	5	Trapezoidal	1.2	1.8	1.4	0.22	416	0.012	0.022
J34	1804	C34	J31	J32	1	Closed Rectangular	1	1	1.2		23	0.043	0.02
J35	1794	C35	J32	J33	4	Trapezoidal	1.2	1.8	1.4	0.22	515	0.0078	0.025
J36	1793	**C36**	J33	03	14	Irregular		3.8	3		450	0.0311	0.5
J37	1790	C37	J34	J36	11	Trapezoidal	1	1.6	1.4	0.214	309	0.036	0.025
J38	1801	C38	J35	J36	7	Parabolic		1.2	0.4		160	0.044	0.03
J39	1794	C39	J36	J37	11	Trapezoidal	1	1.6	1.4	0.21	227	0.049	0.025
J40	1790.4	C40	J38	J39	7	Parabolic		1.2	0.5		355	0.0197	0.03
J41	1789.8	C41	J40	J39	3.5	Irregular		2.4	1.3		221	0.015	0.45
J42	1789	C42	J39	J37	4	>>		2	1		212	0.0188	0.45
J43	1786	C43	J37	J41	0.2	Closed Rectangular	1	1	0.8		16.5	0.012	0.013
J44	1784	C44	J41	J42	0.8	Trapezoidal	1	1.2	0.8	0.125	243	0.0033	0.025
J45	1787	C45	J42	J43	3	>>	>>	>>	>>	>>	157	0.019	0.025
J46	1786.4	C46	J43	J44	2	>>	>>	1.4	1.2	0.167	161	0.012	0.025
J47	1782.4	C47	J45	J44	0.6	>>	1	1.6	0.8	0.375	550	0.001	0.025
J48	1782	C48	J44	J51	4	>>	1.2	1.8	1	0.3	409	0.009	0.025
J49	1784	C49	J46	J47	4	Rectangular	0.8	1	1		298	0.013	0.02
J50	1785	C50	J46	J48	4.4	>>	>>	>>	>>		16.5	0.27	0.02
J51	1780	C51	J48	J49	2	>>	>>	>>	>>		191	0.0105	0.02
J52	1779	C52	J50	J49	1	Parabolic		1	0.5		266	0.0038	0.03
J53	1778	C53	J49	J51	4	Closed Rectangular	0.9	0.9	1.1		162	0.025	0.02
J54	1783	C54	J51	J52	1	>>	2.4	2.4	2		58.5	0.017	0.013

J55	1790	C55	J52	J53	1	Rectangular	3	3	2.2		669	0.0015	0.013
J56	1786	C56	J54	J56	3	Trapezoidal	1	1.8	0.8	0.3 75	411	0.0073	0.025
J57	1784	C57	J56	J57	2	>>	>>	>>	>>	>>	302	0.0066	0.025
J58	1787	C58	J57	J54	1	>>	>>	>>	>>	>>	503	0.0019	0.025
J59	1785	C59	J54	J53	5	Trapezoidal	1	1.6	1	0.3	308	0.013	0.025
J60	1765	C60	J58	J56	1	Parabolic		1.2	0.4		230	0.0044	0.03
J61	1775	C61	J59	J57	1	>>		>>	>>		232		0.03
J62	1764	C62	J53	J60	13	Rectangular	3	3	2.2		835	0.0155	0.015
D1	1787	C63	J61	J60	10	Irregular			2.5		785	0.013	0.75
O3	1754	C64	J60	J62	1	Closed Rectangular	2.5	2	2		43.8	0.023	0.015
04	1748	C65	J62	**O4**	16	Irregular		5.2	4		720	0.022	0.75
		C66	D1	J46	0.6	Closed Rectangular	1	1.2	1.2		17	0.035	0.013

Appendix table 13: Input data in swmm5.1 at outlet 3 and 4 sub catchments parameters

Name of sub catchment(S)	Out Let of S	Area of S (ha)	width	% of slope	% of Imperv	N-imp	N-perv	Destore-imprev	Destore-perv	% zero-imprev	Land use land cover observation from total sub catchment area in percentage (%)
S1	J1	10.2	460	1.8	25	0.015	0.5	2.5	7	75	5%free area and 95%builling
S2	J2	7.25	228	1.8	30	0.015	0.5	2.5	7	70	All covered by house build
S3	J3	9.85	341	1.5	25	0.015	0.5	2.5	7	75	All covered by house build
S4	J4	6.35	230	1.8	20	0.015	0.5	2.5	7	80	4% free area and 96%bld
S5	J5	4.35	341	1.5	25	0.015	0.5	2.5	7	75	20 %(Muslim area),30%free and 50%bld
S6	J6	4.7	384	1.2	25	0.015	0.5	2.5	7	75	All bldg. housing
S7	J7	2.67	200	0.3	25	0.015	0.5	2.5	7		All housing cover
S8	J8	2.71	250	2.15	25	0.015	0.5	2.5	7	75	15%(cattle and shape market).85% bldg. house
S9	J9	9.5	350	2.15	20	0.015	0.5	2.5	7	80	75% agricultural and 25%household area
S10	J10	5.18	450	2.15	20	0.015	0.5	2.5	7	80	95% free agricultural and 5% household

S11	J11	13	420	1.1	35	0.015	0.5	2.5	7	65	95%(rail way station and road) and 5%free area cover
S12	J12	6.37	200	1.1	30	0.015	0.5	2.5	7	70	60%water supply office and 40%house bldg.
S13	J14	6.19	280	2.15	20	0.015	0.5	2.5	7	80	90%fana school 10%house
S14	D1	14.8	350	1.9	20	0.015	0.5	2.5	7	80	Union Area And Agriculture Cover(40%free Area,20%tree,40%bld)
S15	J17	8.45	300	0.5	25	0.015	0.5	2.5	7	75	60%bld.40%tree And Grass
S16	J20	14.8	530	0.8	20	0.015	0.5	2.5	7	80	30%ethiojapan, 80 %(10%health. 5%library.55% house bldg.)
S17	J22	3.22	250	5	25	0.015	0.5	2.5	7	75	All bldg.
S18	J23	3.46	250	5		0.015	0.5	2.5	7		All bldg.
S19	J25	3.99	250	5	30	0.015	0.5	2.5	7	70	20%bld House And 80% Daima School(25%bld ,55%green)
S20	J27	7.28	520	0.6	25	0.015	0.5	2.5	7	75	90% bldg. house and 10% youth club area
S21	J28	6.64	543	0.7	30	0.015	0.5	2.5	7	70	50%bld House and 50%(Hotel,Shoping Area)
S22	J29	12.4	320	0.95	30	0.015	0.5	2.5	7	70	80% Mechine, Church, School And Clinic) 20%hotel And Shopping Area
S23	J31	24.6	420	1.2	30	0.015	0.5	2.5	7	70	50%hospital,20%beqele Mola Oil And Restaurant, 20%Micro Entries Office And 10%car Weight
S24	J31	21.3	420	1.2	25	0.015	0.5	2.5	7	75	20%hotel and 80 house life bldg.
S25	J33	23.7	515	0.8	20	0.015	0.5	2.5	7	80	70%agricultural And 30% House Bldg.
S26	J33	22.2	515	0.8	20	0.015	0.5	2.5	7	80	90%agricultural And 10%house Bldg.
S27	03	33.1	600	3.1	25	0.015	0.5	2.5	7	75	40% agricultural ,50% bldg. of factors and 10%house
S28	J34	5	200	1.8	30	0.015	0.5	2.5	7	70	All bldg. house life
S29	J39	7.3	310	3.6	30	0.015	0.5	2.5	7	70	All bldg. house life
S30	J41	1.4	220	4.9	25	0.015	0.5	2.5	7	75	30%free ,60%bld and 10%raod
S31	J42	3.7	240	2.5	25	0.015	0.5	2.5	7	75	100% light factory
S32	J43	7.66	400	2.5	30	0.015	0.5	2.5	7	70	All high school
S33	J44	4.27	550	2.5	30	0.015	0.5	2.5	7	70	All house hold
S34	J45	14.3	650	2.5	25	0.015	0.5	2.5	7	75	80%household and 20%hotels and shops
S35	J40	13.6	600	45	25	0.015	0.5	2.5	7	75	30%free area and 70% house
S36	J39	8.4	355	3	30	0.015	0.5	2.5	7	70	90%house build and 10 free area

S37	J51	6.64	410	2.5	30	0.015	0.5	2.5	7	70	100 %house hold
S38	J50	2.81	320	2.5	30	0.015	0.5	2.5	7	70	100% house hold
S39	J49	4.42	300	1	30	0.015	0.5	2.5	7	70	100% house hold
S40	47	12.2	320	2	25	0.015	0.5	2.5	7	75	40%stadium,30% electric and red cross office,20 shops and10%muslikus
S41	J55	13.6	410	0.4	30	0.015	0.5	2.5	7	70	All bldg.
S42	J56	9.65	480	0.4	30	0.015	0.5	2.5	7	70	95% household and 5%hotels
S43	J59	4.17	420	0.1	25	0.015	0.5	2.5	7	75	50%household and 50% hotels
S44	J57	5.89	302	0.7	25	0.015	0.5	2.5	7	75	20%church and 80%household bldg.
S45	J54	13.6	460	0.15	25	0.015	0.5	2.5	7	75	15free area and 85% bldg.
S46	J53	14.75	650	0.15	30	0.015	0.5	2.5	7	70	70% road district office, 20%hotels and10%house bold
S47	J60	58.7	830	1.55	20	0.015	0.5	2.5	7	80	40%house,20%factories and40%free agricultural area
S48	J61	45.3	850	1.55	20	0.015	0.5	2.5	7	80	60%house, and 40%free agricultural area
S49	04	17.9	720	2.2	20	0.015	0.5	2.5	7	80	20%trees and 80%factory of goje
S50	J38	3.15	450	1.8	25	0.015	0.5	2.5	7	75	25%church 40%house build and 35%free area
Total											

Appendix Table 14: Sub catchment runoff at outlet three and four

Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Total Runoff mm
S1	193.62	0.00	0.00	7.96	172.17
S2	193.62	0.00	0.00	50.37	133.60
S3	193.62	0.00	0.00	8.49	166.98
S4	193.62	0.00	0.00	8.94	166.41
S5	193.62	0.00	0.00	7.37	176.55
S6	193.62	0.00	0.00	51.43	137.75
S7	193.62	0.00	0.00	8.54	166.43
S8	193.62	0.00	0.00	6.96	178.78
S9	193.62	0.00	0.00	8.13	170.61
S10	193.62	0.00	0.00	7.58	176.94
S11	193.62	0.00	0.00	7.46	169.26
S12	193.62	0.00	0.00	8.18	165.65
S13	193.62	0.00	0.00	8.47	170.91
S14	193.62	0.00	0.00	9.59	158.45
S15	193.62	0.00	0.00	9.28	156.20
S16	193.62	0.00	0.00	10.81	136.06
S17	193.62	0.00	0.00	6.69	180.00
S18	193.62	0.00	0.00	6.77	179.66
S19	193.62	0.00	0.00	6.39	180.34
S20	193.62	0.00	0.00	8.10	170.93
S21	193.62	0.00	0.00	7.20	175.50
S22	193.62	0.00	0.00	8.58	159.89
S23	193.62	0.00	0.00	9.00	152.68
S24	193.62	0.00	0.00	9.51	152.31

Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Total Runoff mm
S25	193.62	0.00	0.00	9.91	153.57
S26	193.62	0.00	0.00	9.80	155.31
S27	193.62	0.00	0.00	8.92	161.67
S28	193.62	0.00	0.00	7.51	172.92
S29	193.62	0.00	0.00	7.01	176.83
S30	193.62	0.00	0.00	6.08	182.15
S31	193.62	0.00	0.00	7.28	177.04
S32	193.62	0.00	0.00	6.97	177.12
S33	193.62	0.00	0.00	6.05	181.77
S34	193.62	0.00	0.00	7.74	173.97
S35	193.62	0.00	0.00	6.23	181.70
S36	193.62	0.00	0.00	7.12	176.12
S37	193.62	0.00	0.00	6.78	178.33
S38	193.62	0.00	0.00	6.15	181.35
S39	193.62	0.00	0.00	7.21	175.42
S40	193.62	0.00	0.00	8.69	164.63
S41	193.62	0.00	0.00	8.95	153.61
S42	193.62	0.00	0.00	8.24	164.81
S43	193.62	0.00	0.00	8.92	161.62
S44	193.62	0.00	0.00	8.08	171.12
S45	193.62	0.00	0.00	10.24	136.99
S46	193.62	0.00	0.00	7.50	172.99
S47	193.62	0.00	0.00	9.20	163.53
S48	193.62	0.00	0.00	10.12	149.98
S49	193.62	0.00	0.00	8.62	169.52
S50	193.62	0.00	0.00	6.60	180.38

Appendix Table 15: Junction flooded

Node	Hours Flooded	Maximum Rate CMS	Day of Maximum Flooding	Hour of Maximum Flooding	Flood Volume 10^6 ltr
J1	3.25	1.153	0	01:00	9.097
J2	4.13	1.331	0	00:30	13.305
J3	3.07	1.934	0	01:00	16.464
J4	2.50	0.278	0	01:45	1.550
J5	2.78	0.452	0	01:00	2.409
J7	1.91	0.255	0	01:00	1.022
J8	3.27	0.635	0	00:24	3.664
J9	4.73	2.925	0	01:45	37.473
J12	3.71	2.303	0	00:30	19.992
J13	2.79	0.512	0	01:15	3.332
J17	3.01	0.905	0	01:16	7.262
J20	5.00	0.990	0	02:00	12.047
J22	2.95	0.475	0	00:45	2.524
J23	3.00	0.588	0	01:00	3.681
J24	3.38	0.327	0	01:01	3.304
J27	1.58	0.431	0	01:00	1.422
J29	1.90	0.866	0	01:00	3.854
J31	3.97	6.988	0	00:42	71.081
J32	5.29	1.392	0	04:21	21.720
J37	0.99	0.054	0	01:02	0.081
J39	2.88	4.577	0	00:45	24.096
J41	3.71	2.887	0	00:45	31.345
J44	2.84	2.240	0	01:00	13.848
J45	2.43	0.979	0	01:00	4.825

Appendix Table 16: Channels overflowed in different time t

Conduit	Hours Both Ends Full	Hours Upstream Full	Hours Dnstream Full	Hours Above Normal Flow	Hours Capacity Limited
C11	0.01	0.01	3.71	0.01	0.01
C12	2.79	4.33	2.79	4.07	2.79
C13	2.79	2.79	3.25	0.14	0.01
C17	2.87	2.87	3.90	0.86	0.86
C19	0.01	0.01	5.05	0.01	0.01
C2	0.01	0.01	3.56	0.01	0.01
C20	0.01	0.01	0.01	5.63	0.01
C21	0.01	0.01	5.00	0.01	0.01
C22	0.01	0.01	5.00	0.01	0.01
C23	0.01	0.01	1.90	0.01	0.01
C26	0.01	0.01	3.16	0.01	0.01
C27	0.01	3.16	0.01	3.13	0.01
C29	0.01	0.01	1.58	0.01	0.01
C30	0.01	1.58	0.01	1.41	0.01
C31	2.05	2.05	2.32	0.01	0.01
C32	2.32	2.32	3.06	0.23	0.01
C33	0.01	0.01	3.97	0.01	0.01
C34	4.91	4.91	5.65	0.01	0.01
C39	0.01	0.01	1.46	0.01	0.01
C4	0.01	0.01	3.40	0.01	0.01
C41	0.01	0.01	5.69	0.01	0.01
C42	0.01	0.01	2.62	0.01	0.01
C43	2.99	2.99	3.27	3.03	2.99
C44	0.01	3.27	0.01	0.01	0.01
C45	0.01	0.01	2.52	0.01	0.01
C46	0.01	0.01	2.86	0.01	0.01
C47	2.43	2.43	3.23	0.01	0.01
C48	0.01	3.02	0.01	0.01	0.01
C49	0.01	3.29	0.01	3.29	0.01
C5	2.78	2.78	3.07	0.01	0.01
C56	0.01	0.01	2.92	0.01	0.01
C57	2.92	2.92	4.38	0.01	0.01
C58	0.01	4.38	0.01	0.01	0.01
C6	0.01	0.01	1.91	0.01	0.01
C61	0.01	0.01	5.62	0.01	0.01
C66	0.01	0.01	3.16	0.01	0.01
C7	1.91	1.91	3.07	0.01	0.01
C8	3.07	3.07	3.76	0.01	0.01
C9	0.01	0.01	0.01	0.03	0.01