



ADDIS ABABA UNIVERSITY
ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

**Assessment of Storm Water Drainage System Performance of Mojo Town,
Central Ethiopia.**

A thesis submitted and presented to the school of graduate studies of Addis Ababa University in partial fulfillment of the degree of Masters of Science in Civil and Environmental Engineering (Major in Water Supply & Environmental Engineering).

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December, 2020
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Certificate

This is to certify that the thesis prepared by Mezmur Hawaz Debru, entitled: Assessment of Storm Water Drainage System Performance of Mojo Town, Central Ethiopia and submitted in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering (major in Water Supply and Environmental Engineering).


As a member of the board of examiners of the MSc, Thesis open defense examination, we certify that we have read, evaluated the thesis prepared by Mezmur Hawaz Debru and examined the candidate.

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Declaration

I the undersigned, declare that this thesis is my original work and has not been presented for the award of a degree in any university and all the materials used for this thesis work have duly acknowledged.

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December, 2020.

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December, 2020, Addis Ababa, Ethiopia

Abstract

The main objective of this study was to assess the existing storm water drainage performance of Mojo town and to provide the possible recommendations or engineering measures that enables to alleviate the prevailing problems. Storm Water Management Model (SWMM), which is a dynamic rainfall runoff simulation model widely applied for urban drainage, was chosen for the performance evaluation to assess the triggering factors as well. The model was developed based on a selected area of 354 ha, that has been a flood prone area mainly due to the construction of two mega projects. The model area is divided in to 45 sub catchments with a drainage network of 94 conduits, 74 junctions and 2 outfalls. Three important inputs for the model were organized: Rainfall, infiltration and physical characteristic of the model area. 1. Rainfall depths of twenty-one years (1997-2017) of Mojo metrology station were obtained from National Metrological Agency. An IDF curve for different return periods (2,5,10,25,50,100yrs) formulated by Log person type III distribution method was used as an input for the model due to its slightly better coefficient of determination than Gumbel method.2. Infiltration of the model area is represented by Green-Ampt equation.3 Physical characteristics of the sub catchments including topography of the model area was analyzed using Global mapper, ArcGIS software. Besides, three successive Google earth images were used to analyze the land use land cover change of the model area through 11 years (2009-2019) period. Based on the inputs the model is built and model parameters were derived based on characteristics of the sub catchments and drains infrastructures. According to the sensitivity analysis, catchment width and Manning roughness coefficient for pervious areas were found the most sensitive parameters. Three drains (D_48, D_89 & D_90) were selected for calibration and validation. Three rainfall event observations for calibration and one event for validation were used on the two most sensitive parameters until the simulated and observed values reached at acceptable level. The performance of SWMM is insured based on an evaluation that is made by coefficient of determination (R^2) and Nash-Sutcliffe Efficiency Coefficient (NSE). The model is simulated for one rainfall event, for different return periods and 3 hours rainfall duration. According to 10 years return period of simulation regarding storm water discharge, 25 conduits and 28 junctions were flooded which is 27.0 % and 37.8% respectively. An increment in terms of Peak runoff, total runoff & peak discharge is observed due to LULC change. The other important issue is a significant proportion of the drainage system is subjected to below 0.9m³/s flow velocity that insists deposition of solid material in drains which affects the system performance. Among the various low impact development (LID) structural measures, Bio retention and Infiltration trench are incorporated and simulated in the model. Based on 10 yrs & 25 yrs return period simulation for LID, improvements in terms of peak discharges decrease and elongated peak time discharge were observed where LID is applied. Regarding 10 yrs period ,the magnitude of peak discharge of drains is decreased with the minimum of 1.5% and maximum of 35.1. Generally, the drainage system of the town is not suffice for the runoff generated from the town because of intensive land use/land cover (LULC) change as well as lack of green areas in required proportion. The major source of overflow of drains is highly related to the construction of the two mega projects. Minimum flow velocity, which is below cleansing velocity, is the other problem that affects the drainage system performance. In terms of quality, different studies verified that storm water of Mojo town is highly polluted due to mainly industrial pollution. This study mainly recommends proper implementation of the town master plan in such a way to regulate reasonable land use land cover. Redesigning of drains and related drainage infrastructure is critically required in such a way to meet the minimum limit of flow velocity and to use as remedy for the problem of overflow of drains. Diversified application of structural and nonstructural measures of LID help to alleviate the prevailing drainage problem of the town.

Keyword: Urban drainage, SWMM, LID, calibration, validation.

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Abbreviations

AAiT: Addis Ababa Institute of Technology

AASHTO: American Association of State Highway and Transportation Officials

asl: above sea level

BC: Before Christ

BMP: Best Management Practice

BOD: Biological Oxygen Demand

CHRS: Center for hydro metrology & remote sensing

COD: Chemical Oxygen Demand

CSA: Central Statistics Agency

DEM: Digital Elevation Model

E: East

EGL: Energy Grade Line

EPA: Environmental Protection Agency

ERA: Ethiopian Road Authority

GPS: Geographic Information System

GTP: Growth and Transformation plan

HGL: Hydraulic Grade Line

IDF: Intensity Duration Curve

LID: Low Impact Development

LULC: Land Use Land Cover Change

N: North

NMA: National Metrological Agency

NSC: Nash–Sutcliffe model efficiency

R²: Coefficient of determination

SDG: Sustainable Development Goals

STORM: Storage, Treatment, Overflow, Runoff Model

SWMM: Storm Water Management Model

Sq. Km: Square Kilometer

TDS: Total dissolved solid

US EPA: United States Environmental Protection Authority

USGS: United State Geological Survey

WSUD: Water Sensitive Urban Design

Chapter One: Introduction

1.1. Background

In a country like Ethiopia, where a great effort is being employed to boost the economy, there has been massive infrastructural development like commercial and residential buildings, industries, roads, car parking in urban areas. This circumstance totally altered the natural pervious characteristics of the land surface as a result the natural hydrological cycle has been negatively affected. This fact is highly being manifested on the aspect of urban drainage poor performance. This situation is exacerbated due to improper solid waste management and massive population growth and migration from rural to urban areas.

Developing countries experience accelerated urbanization without adequate investment in infrastructure, and against a background of deficient public services for water treatment, collection and treatment of foul sewage, garbage collection, urban drainage, transport and health. Urban concentrations have environmental consequences in the form of urban flooding and pollution of water courses, soil and air. Settlements are established in inappropriate areas such as those originally set aside for environmental preservation, and on steep hillsides and areas liable to flooding. (A.Silveira, 2002). Development of an urban area, involving covering the ground with artificial surfaces, has a significant effect on these processes. The artificial surfaces increase the amount of surface runoff in relation to infiltration, and therefore increase the total volume of water reaching the river during or soon after the rain. Surface runoff travels quicker over hard surfaces and through sewers than it does over natural surfaces and along natural streams. This obviously increases the danger of sudden flooding of the river. It also has strong implications for water quality(D. Butler & J.W.Davies, 2011). With urbanization, runoff increases which yields more frequent and severe floods. This leads to the requirement of proper drainage solutions to reduce flooding and the possible occurrence of inconvenience, damage and adverse health impacts (M.Maharjan et al., 2009).

In the natural environment, ground surfaces allow water infiltration which balances groundwater supply and water quality. Surface runoff has decreased to a relatively low level because most precipitation events are not large enough to fully saturate the ground soil. However, urban impervious areas such as roads, buildings and roofs produce significant storm water runoff under the big storm events that may cause flash flooding and other water-related problems. Urban storm water drainage facilities are part of the urban infrastructure elements and design of these facilities require due attention (Ministry of Works & Urban Development, 2008). Urban drainage network

system is constructed with a purpose to collect the stormwater generated from the urban sub - catchments properly and transport safely without causing problem in the urban environment.

Due to inadequate integration between road and urban storm water drainage infrastructure provision and poor management significant proportion of the area is exposed to flooding hazards/risks. This has resulted in negative impacts on urban storm water drainage provision and management. (Dagnachew A., 2011).

In Ethiopian context, where watersheds of many urban centers receive significant amount of annual rainfall and where rainfall intensity is generally high, control of runoff at source, flood protection, and safe disposal of excess water/runoff through proper drainage facilities becomes essential (Ministry of Works & Urban Development, 2008). The pattern of urbanization and modernization in Ethiopia has meant increase densification along with urban infrastructure development. This has led to deforestation, use of corrugated roofs and paved surfaces. The combined effect of this results in higher rainfall intensity and consequently accelerated and concentrated runoff in the urban areas. Due to inadequate integration between road and urban storm water drainage infrastructure provision, many areas are exposed to flooding problems and road damages in urban roads (Dagnachew A, 2011). A number of cities and towns in Ethiopia like Addis Ababa, Dire Dawa, Adama, Sebeta, Mojo, Awasa have been facing the aforementioned problems that induced loss of infrastructures, personal properties including human life.

Metropolitan Addis Ababa, sprawling at the foothill of Entoto mountain range is traversed by several small streams originating from the mountain range. Torrential rains which are common during the rainy season in the city, cause sudden rise in flow of these streams which bring about flood damages to settlements along the bank of these streams. Such damages have often caused losses of property (WMO & GWP, 2003). According to the study (Dagnachew A. et al., 2019) conducted in Addis Ababa (Werada 03 of Nefas Silk-Lafto sub-city) the results showed that 14% of the drains in new city parts and 28% in old city parts were in conditions inadequate for removal of stormwater, resulting in flash flooding and infrastructure degradation in the associated watersheds. Further, although more than 72% of the surveyed drains were oversized, stormwater overtopping reoccur as a season-to-season problem, ascribed to illegal dumping of waste into drains, reducing their hydraulic capacity. In similar fashion, the Drainage system Evaluation study (Eskedar T., 2013) of Kebena stream catchment conducted in Addis Ababa showed that the causes for the prevailing urban drainage problems in the area are primarily flood regime change due to mainly the expansion of built environment and inadequate integration between road and urban

storm water drainage lines, lack of sustainable urban storm management, extraordinary challenge of damping solid and liquid waste water in the drainage system.

Although the magnitude is different, all the drainage problems have been exhibited in Mojo town which is an emerging industrial town in Oromia regional state where different huge industrial and infrastructural development projects have been implemented including two mega projects. Those are Addis Ababa-Adama express way and Addis Ababa –Djibouti Railway projects. This thesis report is compiled based on the detailed urban drainage study conducted in a selected model area of Mojo town using SWWM model in such a way to identify the causes and the consequences of the problems as well as the remedy.

1.2. Statement of the Problem

During rainy season, some parts of mojo town were inundated by the flood that often comes from the upland of the town. As a result, individual properties like houses, fences etc. were damaged and sometimes peoples were displaced from their home temporarily. On the other hand, there exists demolition of urban utility networks like water supply lines, electric distribution lines, telephone cables and road infrastructures. Moreover, as most of the up land area of the town is farm land, flood has caused soil erosion as a result gullies are formed in different parts.



Figure 1. Flood is exposing water supply pipes & uprooting Acacia trees

Significant proportion of the area is exposed to flooding during the rainy season. There is also a problem of overtopping, blockage of drainage facilities and water logging in some of the drainage areas and wastes that flow along the drainage system have bad smell, and unwelcoming environment (Eskedar T., 2013).Hence ,those things could affect human health because of poor environmental sanitation .



Figure 2. Overflow of drains nearby Ethio-Japan Textile Factory (8:30 AM, July 15/2020)

The malfunctioning of the urban drainage system of Mojo town is associated to the absence of urban drainage master plan that induces to improper urban drainage management. There are indications which implies that the existing urban drainage infrastructures development are not suffice and the network is not good to accommodate the overall runoff from the catchment. The problem is highly being aggravated due to the combined effect of solid and liquid waste disposal. As the town has been highly urbanized because of different types of industrial development like

Tannery, textile, abattoir, edible oil, soap & detergent, there has been a possible ways where effluent with hazardous chemicals from those industries are released to the environment. The storm water could be a means for the dispersion of those chemical to the environment which are dangerous for the health of human being and other biotic communities.

The other important problem that negatively affecting urban drainage system of the town is lack of integration among stakeholders. The two mega projects (Addis Ababa-Adama express way & Addis Ababa-Djibouti railway projects) were found a threat for the town that could be an example to cause flood problems in the town due to lack of integration with Ethiopian Road Authority (ERA) during construction period. In order to alleviate the prevailing drainage problem of Mojo town, it important to assess the root causes negatively affecting the system on the other hand that enables to improve the drainage service.

1.3. Research Questions

1. What does the existing land use look like?
2. What is the hydraulic performance of the existing urban drainage system?
3. What is the impact of land use land cover change of the town on the hydraulic performance of the drainage system?
4. Will there be a risk of flooding in the town with respect to the town drainage system capacity?
5. What are the possible recommendations to alleviate the problems taking best management systems (BMS) in to account?

1.4. Objective

1.4.1. General Objective

- ✓ To assess the existing storm water drainage performance and challenges of Mojo town and to provide the possible recommendations or engineering measures that enables to alleviate the prevailing problems.

1.4.2. Specific Objectives

- ✓ To evaluate the hydraulic capacity of the existing drainage system of the town
- ✓ To determine the land use land cover change of the town
- ✓ To determine change of runoff due to land use change through time.
- ✓ To determine the impact of best management practices in terms of runoff reduction.
- ✓ To provide recommendations that enables to alleviate the prevailing problems

1.5. Significance of the study

The attempt to assess the exiting Mojo town storm water drainage performance has a great significance to recognize the major storm water management problems of the town in terms of

storm water quantity. Moreover, it also helps to identify the possible engineering and non-engineering measures that should be implemented in the town in order to alleviate the prevailing problems. Based on the facts indicated in the research, if the Ethiopian government is going to take strategic actions in the area, it really helps to improve the situation as well as to have a clean environment for the people that has ultimately a great contribution to meet SDG goal 3 & 6.

Sustainable Development Goals were formulated by United Nations General Assembly in 2015 in such a way nations to attain development in a sustainable manner that enables to alleviate poverty and to maintain the welfare of the people. Under SDG 3 (Ensure healthy lives and promote well-being for all at all ages) ,out of nine targets one (target 3.3)is designed with an intention to combat water-borne diseases in order to secure health of human being .Moreover ,under SDG 6 (Ensure availability and sustainable management of water and sanitation for all) , out of six targets one (target 6.3) is designed with an intention to improve water quality by reducing pollution, avoid solid waste dumping and minimizing release of hazardous chemicals and etc. If improvement are going to be made on the overall planning and management of urban drainage systems, it has a great contribution to meet the aforementioned SDG goals at country level. In case of Mojo town, there were significant negative impacts being observed in the area on the dwellers of the town in terms of health, economy & environment because of lack of proper drainage master plan, drainage infrastructure network & poor integration among concerned internal stakeholders in the municipality and other external stakeholders like Ethiopian Road Authority. Hence, if proper attention is given by the municipality, this research could be a preliminary signal to take practical action that helps to reverse the aforementioned problems step by step. The significance of the study may not be limited to only these, but it can also be used as a reference for other researchers who are interested to conduct a research in the related areas.

1.6. Limitations of the study

There were some potential limitations during the study. Due the fact that a well-organized existing drainage data was not available in the municipality's office, a lot of information related to the drainage system supposed to be collected from the ground that makes the research work laborious. Localized insecurity and violence due to political instability, restrictions of movement because of Covid -19 affected primary and secondary data collection.

1.7. Organization of the Study

This thesis report is organized into five chapters. Chapter one deals with background, statement of the problem, objectives and significance of the study. Chapter two explains about review of related literature associated to the objectives of the study. Chapter three addresses Materials and Methods

employed for the research as well as description of the study area and other related issues. Chapter four is about results and discussion that deals about analysis and explanation of facts based on the output of the model .Finally, Chapter five presents conclusion and recommendations based on the result of the study.

Chapter Two: Literature review

2.1. Historical outlook

Several thousand years BC may seem a long way to go back to trace the history of urban drainage, but it is a useful starting point. In many parts of the world, we can imagine animals living wild in their natural habitat and humans living in small groups making very little impact on their environment. Natural hydrological processes would have prevailed; there might have been floods in extreme conditions, but these would not have been made worse by human alteration of the surface of the ground.(David Butler & John W.Davies, 2011). Several ancient civilizations (Mesopotamians, the Minoans and the Greeks) showed great care when constructing urban drainage systems, combining the objectives of collecting rainwater, preventing nuisance flooding, and conveying wastes. During the Roman Empire Age, significant advances were introduced in urban drainage systems (Burian & Edwards, 2002).

The beginning of modern urban drainage practices was initiated in European cities during the nineteenth century. Most sewers constructed before the nineteenth century were not planned or designed by an engineer using numerical calculations. Instead a trial-and-error process was executed, which in some cases eventually produced well-functioning systems. During the nineteenth century the perspective of urban drainage design changed to incorporate the opinions of technical experts (Burian & Edwards, 2002).

2.2. Urban Drainage

Urban Drainage includes the removal of all unwanted water from urban areas through man made and artificial structures. It includes waste water like sewage, grey water and storm water. Grey water, sometimes called sullage, is domestic wastewater predominantly from kitchens, baths, basins and washing machines. The unwanted water may or may not, be used for other purposes with or without, treatment. Indeed, it is part of the philosophy of sustainable that there is, ultimately, no waste; all 'waste' from one process should be input for another (McDonough & Braungrat, 2002). Urban drainage has two interfaces: the public and environment. The interaction among the public and the drainage system on the other hand the environment and drainage system should be healthy both in terms of water quality and quantity in order to maintain ecological balance of nature. Otherwise, if the interactions are not healthy, it induces distortion on natural system that is not good for human being & the environment.

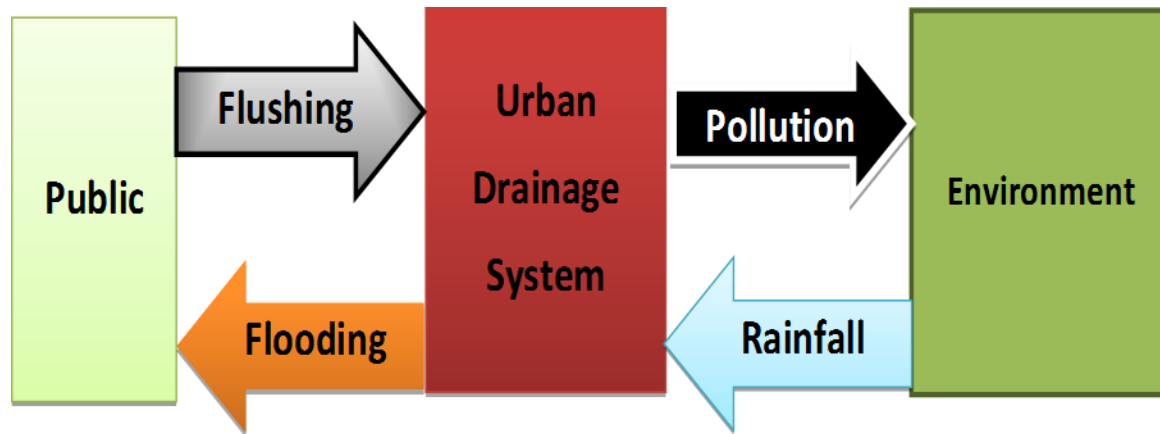


Figure 3. Interfaces with the public and the environment (Butler and Davies, 2011).

The major objectives of urban drainage remain public hygiene, flood protection and pollution control. In developed countries, the first two objectives have been accomplished and emphasis is placed mainly on pollution control. However, in developing countries, hygiene and flood protection are still major issues. (B. Chocat et al., 2001). In the large cities of developing countries, it appears that the conventional sanitary solutions to urban drainage have reached saturation point. Much money has been spent on canalizing water courses, but this has not solved the problems. The present time is a period of transition, in which the 19th-century sanitary model is being abandoned in favor of the environmental model (A.Silveira, 2002).

2.2.1. Storm Water

Stormwater is the water that is received from infrastructures like rooftops, roadways, parking lots, etc after a rainfall event; and runs over the surface of the earth, drainage lines. It ultimately joins the nearby water bodies. Its properties in terms of quantity and quality are determined by the nature and characteristics of both the rainfall and the catchment.

Urban drainage network channels try to overcome the runoff to much extent by accommodating the generated runoff within it. But the increased human activity and their produced conditions such as imperviousness and manmade water courses lead faster rainfall to runoff transformation. (R.Aryal et al., 2009). A sustainable urban water system, or stormwater system, is not only a question of problems and avoiding unwanted content in the water, it can also be a question of its potential usability as a water resource in society. Stormwater drainage may not only be considered as systems to divert undesired water from urban areas, but also as a valuable element for landscaping the surrounding of buildings and roads. (Gogate & Rawal, 2012).

2.2.1.1. Stormwater Quantity

Rain falling over an urban watershed will strike either a pervious surface or an impervious surface. On pervious surface most of the rainfall infiltrated to the subsurface and some remains runoff as overland flow and depression storage. Depression storages are small pores on land surfaces which temporarily store water. Some portion of the runoff water may be evaporated (R. Aryal et al., 2009). It has long been recognized that covering land with impervious surfaces, such as roofs and roads, reduces both the volume of water infiltrating into soils and the volume of water lost to the air through evapotranspiration, thus increasing the volume of runoff following rain events (L.B. Leopold, 1968).

2.2.1.1.1. Runoff Generation

Rain falling over an urban watershed will strike either a pervious surface or an impervious surface. On pervious surface most of the rainfall infiltrated to the subsurface and some remains runoff as over land flow and depression storage. Depression storages are small pores on land surfaces which temporarily store water. Some portion of the runoff water may be evaporated.(R. Aryal et al., 2009).Impervious covers, such as asphalt, concrete or a bare soil highly consolidated by traffic, contribute to high runoff rates because of low hydrologic losses (UNESCO, 1987). The water that was formerly able to infiltrate soil now flows off surfaces, creating a greater volume of runoff, which ends up converging to regions of a lower topographic level, thus generating areas of flooding. The increase of surface runoff may also lead to overflow of streams, gullies, ditches, and rivers, creating problems of riverside flooding. Flooding results in risks to people's health and quality of life, in addition to social and economic losses (C. Poletto & R. Tassi, 2012).

2.2.1.2. Storm Water Quality

Sources of urban storm water runoff are related to untreated wastewater, households, road network, fuel stations, parking lots, and parks. However, some other factors also play a crucial role in urban run-off pollution: atmospheric pollutants deriving from urban transport and deposited in urban surface; pollutants deposited in surfaces like roads, roofs and parking lots; pollution deriving from construction fields; the use of pesticides in parks and green space; and deicing materials used in roads during winter. The most important pollutants, included in urban runoff, are sediments, nutrients (nitrogen and phosphorus), chlorides, heavy metals, diesel hydrocarbons, microbial pollution, organic compounds and litter (Yannopoulos S.I. et al , 2013) .Based on a study (Amsalu A., 2018) regarding the impact of urbanization on surface water quality in Gonder Town, Keba is labeled as a river highly polluted due to anthropogenic activities. Land use change has great impact

on Keha River so that water quality has decreased in the past 15 years. The quality of river water decreased from upstream to downstream.

Based on wet and dry season sampling of water from three rivers in the Little Akaki catchment in Addis Ababa, the rivers have a poor water status when compared to international standards. This includes low oxygenation, too high salt concentrations and tendency to too alkaline pH, high nutrient status of both N and especially P, and severe heavy metal contamination with Cr and Cu. Even when compared to the less strict Ethiopian standards phosphate is still problematic, and in two of the three rivers also Cr and Mn. (Dagnachew Adugna et al., 2018). A research (Amde E.G. et al., 2016) with an objective to determine the pollution profile of Mojo River, it was found that there was change on the physico-chemical parameters from upstream to downstream areas, which indicates an introduction of pollution load from industrial wastewater and recommended to enforce on all industries to install facilities and treat wastewater effectively.

According to the study (Berehanu Behailu, 2007) the analysis result of water sample collected from the Mojo major stream show that Mojo River is highly polluted. The high degree of pollution is shown by high organic pollutant, which is characterized by very high BOD (535mg/l), very high COD(542 mg/l), high coliform bacteria count (8×10^7), high turbidity, high suspended solids, high dissolved solids, phosphate(13.2 mg/l), NH_4 (44.8 mg/l), Cl(511mg/l) and high conductivity value.

2.2.2. Effects of Urbanization on Urban Drainage

There are a large number of human and non-human activities that can destabilize the hydrologic cycle. Among human activities that have the greatest impact on the natural hydrologic cycle is the urbanization process. Along with urbanization, new engineering works arise such as buildings, paving of streets, sidewalks, and consequent removal of original plant cover from the environment, which causes a change in the natural permeability of these areas (C. Poleto & R. Tassi, 2012) .

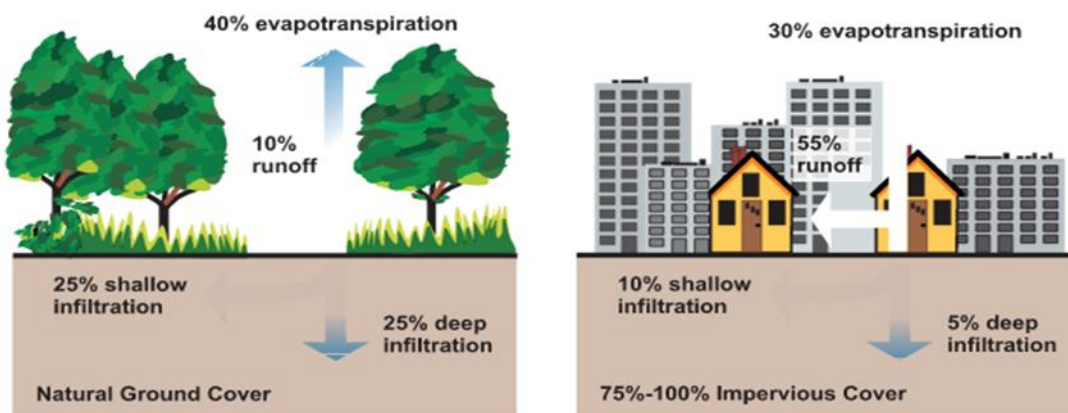


Figure 4. Relationship between Impervious Cover and Surface Runoff (Source: U.S. EPA)

The combined effects of population growth, urbanization and industrializing change adversely the hydrological response of the affected area and cause various environmental impacts. The natural environment is changed to a man-made one, with concomitant quantitative and qualitative changes of the water cycle. Urban drainage is part of this cycle and is related to the urban hydrological system in a very complex way (UNESCO, 1987). Changes in the local hydrologic cycle and its consequences due to the urbanization process, directly affect the environmental balance and, thus, the quality of life of the resident population. Problems arise both due to the inflows to the system, which in future will be less certain, and increasing downstream hydraulic and regulatory constraints on outflows (R.Ashley et al., 2007).

The quantitative evidence from the analysis of the three period maps of the Addis Ababa indicated that expansion of built environment is significant and continues at an alarming rate of modifying the flood regime of the city. The mean runoff transformation potential (runoff coefficient) of the city has increased from 28% in 1984 to 45% in 2002. The mean increase in flood volume between 1984 and 2002 was more than 60%. The peak flood discharge of selected watersheds exceeds from 180% to 300% in some part of Addis Ababa (Semu A.M et al., 2019). A typical impact of urban development on recipient waters is increased amounts of water entering them, and a lower time of concentration. The combination of these two events can result in altered stream paths due to erosion of streambeds and stream banks, and more rapid runoff. Rapid runoff and impermeable surfaces also prevent infiltration to soil and aquifer recharge, which can lead to lower sustained Stream flows, especially during dry periods (J.F.Coles et al., 2012). Urban runoff ending in surface water bodies (lakes, rivers, etc.) may cause biological integrity elements' degradation. It is possible to provoke negative impacts to entire ecosystems, including water quality, bottom sediments, aquatic life and habitats(R. Pitt & M. Bozeman, 1982).

2.2.3. Flood

Floods are extreme natural event and can occur in a wider geographical scale like river basin or in a smaller scale such as catchment and watershed (Ochoa-Rodriguez et al., 2015). Flooding is typically coupled to water moving faster than normal, in part because of the weight of an increased amount of water upstream, leading to an increase in the pressure gradient that drives the flow (C.A.Doswell, 2003). In contrast to rural flood, urban floods can have both area-wide and localized origin. Impacts of urbanization could entail a substantial increase in frequency and magnitude of flooding(Zhou, 2014). In case of inland cities far away from shores, urban floods have become more severe and occurs in the built-up areas as a result of heavy rain and subsequent rainfall runoff

(Paquier et al., 2015). They are further exacerbated by the high intensity rainfall in cities combined with inappropriate sewer system and diverse land cover (L.Salvan et al., 2016).

Floods produce damage through the immense power of moving water and through the deposition of dirt and debris when flood waters finally recede. The energy of that moving water goes up as the square of its speed; when the speed doubles, the energy associated with it increases by a factor of four. In most cases, the damage potential of the flood is magnified by the debris that the waters carry: trees, vehicles, boulders, buildings, etc. When the waters move fast enough, they can sweep away all before them, leaving behind scenes of terrible destruction (C.A.Doswell, 2003).

According to a study (Daniel Alemayehu D., 2007), flood event in Dire Dawa occurred during in April 1981, 1994 and in May, 2005. The May 2005 has caused loss of 35 human lives as well as an estimated amount of 10 million birr damages to property. Moreover, according to the study (Ephrem, 2006) the flood history is changed and the most devastating flood in the history of Dire Dawa occurred on the fifth day of August, 2006. This flood is the worst of its kind which resulted for the loss of more than 240 human lives. Property damage is also more than ever before.

2.2.4. Runoff collection and Component of Drainage system

Roadway runoff is collected in different ways based on the edge treatment, either curbed or uncurbed. Runoff collection and conveyance for a curbed roadway is typically provided by a system of inlets and pipe, respectively. Runoff from an uncurbed roadway typically referred to as "an umbrella section", proceeds overland away from the roadway infill sections or to roadside swales or ditches in roadway cut sections. (Ministry of Works & Urban Development, 2008). The urban drainage system comprises two main subsystems: micro-drainage and macro drainage. The micro-drainage system is essentially defined by the layout of the streets in urban areas, acting in collecting rainfall from urban surfaces. The macro-drainage is intended to receive and provide the final discharge of the surface runoff brought by the micro-drainage net. Macro-drainage corresponds to the main drainage network, consisting of rivers and complementary works, such as artificial canals, storm drains, dikes and other constructed structures (M. G. Miguez et al., 2016) .

2.3. Hydrology

2.3.1. Definition

Hydrology is the physical science which treat the waters of the earth, their occurrence, circulation and distribution, their chemical and physical properties, and their reaction with the environment, including their relation to living things (Unesco, 1991). Moreover, hydrology deals with understanding the underlying physical and stochastic processes involved and estimating the quantity and quality of water in the various phases. A number of physical and statistical laws are

applied, mathematical models are developed, and various state and input and output variables are measured at various points in time and space (Salas & Govindaraju, 2014). Hydrology also deals with estimating flood magnitudes as the result of precipitation. In the design of roadway drainage structures, floods are usually considered in terms of peak runoff or discharge in cubic meters per second (m^3/s) and hydrographs as discharge per time. For drainage facilities, which are designed to control volume of runoff, like detention facilities, or where flood routing through culverts is used, then the entire discharge hydrograph will be of interest (Ministry of Works & Urban Development, 2008). Of all human activities, urbanization is that which produces the greatest local changes in the processes of the Earth's hydrologic cycle. The most rapidly changed factor is the runoff coefficient, which is the ratio between the volume of surface water runoff and the volume of rainwater. Various problems are caused by change in the runoff coefficient. These problems go beyond erosion of the drainage area; they cause imbalances in the rainwater channel (erosion and silting) and the ever so frequent urban flooding (C. Poletto & R. Tassi, 2012).

2.3.2. Intensity Duration Curve

IDF curves are commonly used in the design of hydrologic, hydraulic, and water resource systems. IDF curves are obtained through frequency analysis of rainfall observations.

The basic principle behind the IDF relationship is finding how to relate the intensity of a rainfall event and its duration to its expected frequency. Precipitation intensity is the depth rainfall per unit of time, and is usually expressed in the units of mm/hr and in/hr. The average intensity is frequently applied, which is expressed as the rainfall depth for a particular precipitation event divided by the duration of that event. The frequency is often described in terms of return period, which is the average period of elapsed time between rainfall events that are equal to or more than the magnitude of design. The return period is the inverse of the annual probability of exceedance of an event. The return periods usually represented are for 2, 5, 10, 25, 50 and 100 years (I.H.Elsebaie, 2012). Once a particular return period has been selected for design and a time of concentration calculated for the catchment area, the rainfall intensity can be determined from Rainfall-Intensity-Duration curves.

2.3.2.1. Estimation of Short Duration Rainfall

Design and analysis of drainage structures require rainfall intensity of shorter duration. However, the available rainfall depth is for 24 hr duration so that it is mandatory to convert it to shorter duration. Short-duration rainfall is critical for the small catchments and urban drainage systems. There is always a shortage of short-duration rainfall data as it requires automatic rain gauges to

record such data. On the contrary, daily rainfall values are generally available due to the use of cheap manual Instruments (Al Abdullah et al., 2018).

According to (Ethiopian Road Authority, 2013), it is recommended to use the following formula for short duration conversion.

$$R_{Rt} = \frac{t(b+24)^n}{24(b+c)^n} \dots\dots\dots \text{Eq. 1}$$

Where R_{Rt} is rainfall depth ratio R_t : R_{24}

R_t is rainfall depth in a given duration t

R_{24} is 24 hr rainfall depth

Coefficients $b=0.3$ & $n=0.78-1.09$

After rearrangement, the above formula has the following form.

$$R_t = \frac{t(b+24)^n}{24(b+t)^n} \cdot R_{24} \dots\dots\dots \text{Eq. 2}$$

After substituting intensity ($I_t=R_t/t$), the above formula has the following form.

$$I_t = \frac{R_{24}(b+24)^n}{24(b+t)^n} \dots\dots\dots \text{Eq. 3}$$

2.3.2.2. Probability Distribution

Since most hydrologic events are represented by continuous random variables, their density functions denote the probability distribution of the magnitudes. Some of the frequently used density functions in hydrologic analysis are given below:

- ✓ Normal distribution
- ✓ Lognormal distribution
- ✓ Extreme value distribution
 - Extreme value type I distribution (Gumbel distribution, 1941)
 - Extreme value type II distribution (Frechert, 1927)
 - Extreme value type III distribution (Weibull, 1939)
- ✓ Log Pearson's Type-III distribution
- ✓ Logarithmic Pearson Type-III distribution

2.3.2.3. Gumbel Distribution Method

According to (I. H.Elsebaie, 2012), the Gumbel distribution is the most widely used distribution for IDF analysis owing to its suitability for modeling maximum. It is relatively simple and uses only extreme events (maximum values or peak rainfalls). The Gumbel distribution calculates the 2, 5, 10, 25, 50 and 100 years return intervals for each duration period and requires several calculations. Frequency precipitation P (in mm) for each duration with a specified return period Tr (in year) is given by

$$P_T = P_{av} + K_T S \dots\dots\dots \text{Eq. 4}$$

Where K_T is Gumbel frequency factor given by:

$$K_T = \frac{-\sqrt{6}}{\pi} \left\{ 0.5772 + \ln \left[\ln \left(\frac{T_r}{T_r - 1} \right) \right] \right\} \dots\dots\dots \text{Eq. 5}$$

and P_{av} is the average of the maximum precipitation corresponding to a specific duration. In utilizing Gumbel's distribution, the arithmetic average is used:

$$P_{av} = \frac{1}{n} \sum_{i=1}^n P_{(t)} \dots\dots\dots \text{Eq. 6}$$

Where; n is the number of events or years of record and the standard deviation S is calculated by:

$$S = \sqrt{\frac{(\bar{P} - P_{ave})^2}{n-1}} \dots\dots\dots \text{Eq. 7}$$

Where; P Maximum precipitation depth corresponding to a specific duration. Then the rainfall intensity I_T (mm/h) for return period T is obtained from:

$$I_T = P_T / T_d \dots\dots\dots \text{Eq. 8}$$

Where: T_d is duration in hours.

2.3.2.4. Log Pearson Type III distribution

According to (Abdo K. et al., 2017), this distribution is suitable for both the annual non extreme series and the extreme flood frequency analysis. The recommended technique for fitting a log Pearson Type III distribution to observe the annual peaks is to compute the base 10 logarithms of the rainfall at selected probability, P, by the following equation:

$$\log \chi_T = \bar{x} + ks \dots\dots\dots \text{Eq. 9}$$

$$x_T = \text{anti log}(\log x_T) \dots\dots\dots \text{Eq. 10}$$

Where:-

X_T = design storm for selected return period

\bar{x} = mean logarithm

S = standard deviation of the logarithm

K = a factor that is a function of the skew coefficient (G) and desired recurrence interval

selected probability

The mean, standard deviation and the skew coefficient of the station data used in Log Pearson Type III distribution are computed using the following equation:

$$\bar{x} = \sum_{i=1}^n \log x_i \dots\dots\dots \text{Eq. 11}$$

$$S = \sqrt{\frac{\sum_{i=1}^n (\log x_i - \log \bar{x})^2}{n-1}} \dots\dots\dots \text{Eq. 12}$$

$$G = \frac{n \sum_{i=1}^n (\log x_i - \log \bar{x})^3}{(n-1)(n-2)S^3} \dots\dots\dots \text{Eq.13}$$

Where: G=skew coefficient of logarithm
n=number of items in the data

2.3.3. Runoff Estimation

The rational formula estimates the peak rate of runoff at any location in a catchment area as a function of the catchment area; runoff coefficient; and mean rainfall intensity, for a duration equal to the time of concentration. The rational formula is expressed as:

$$Q = 0.00278CIA \dots\dots\dots \text{Eq. 14}$$

Where: Q = maximum rate of runoff, m³/s
C = runoff coefficient representing a ratio of runoff to rainfall
I = average rainfall intensity for a duration equal to the time of concentration, for a selected return period, mm/hr
A=catchment area tributary to the design location, ha (ERA, 2013)

2.4. Hydraulics of stormwater Drainage system

Hydraulic design of storm drainage systems requires an understanding of basic hydraulic concepts and principles. Important hydraulic principles include flow classification, conservation of mass, conservation of momentum, and conservation of energy (US High Way Administration, 2001). Inlets are the receptors for surface water collected in ditches and gutters, and serve as the mechanism whereby surface water enters storm drains. When located along the shoulder of the roadway, storm drain inlets are sized and located to limit the spread of surface water on to travel lanes. Drainage inlet locations are often established by the roadway geometries as well as by the intent to reduce the spread of water onto the roadway surface. Generally, inlets are placed at low points in the gutter grade, intersections, crosswalks, cross-slope reversals, and on side streets to prevent the water from flowing onto the main road (J.Ahern, 2017).

2.4.1. Hydraulic Capacity

The hydraulic capacity of a drainage channel is dependent on the size, shape, slope and roughness of the channel section. For a given channel, the hydraulic capacity becomes greater as the grade or depth of flow increases. The channel capacity decreases as the channel surface becomes rougher. A rough channel can sometimes be an advantage on steep slopes where it is desirable to keep flow velocities from becoming excessively high. A good open channel design minimizes the effect on existing water surface profiles (Ethiopian Road Authority, 2013). A study was conducted in Addis Ababa city administration in two “Woreda”s to represent the new & old city with a purpose to evaluate hydraulic capacity of the drainage system. The results showed that 14% of the drains in

new city parts and 28% in old city parts were in conditions inadequate for removal of stormwater, resulting in flash flooding and infrastructure degradation in the associated watersheds. Further, although more than 72% of the surveyed drains were oversized, stormwater overtopping reoccur as a season-to-season problem, ascribed to illegal dumping of waste into drains (Dagnachew A. et al., 2019).

The most widely used formula for determining the hydraulic capacity of storm drains for gravity and pressure flows is the Manning's formula and it is expressed by the following equation.

$$V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} \dots\dots\dots \text{Eq. 16}$$

- Where: V = Mean velocity of flow, m/s
- n = Manning's roughness coefficient
- R = hydraulic radius
- S = the slope of the energy grade line, m/m

Continuity Equation– The continuity equation is the statement of mass in fluid mechanics. For the special case of one dimensional, steady flow of an incompressible fluid, It assumes the form:

$$V_1 A_1 = V_2 A_2 \quad (Q_1 = Q_2) \dots\dots\dots \text{Eq. 17}$$

- Where Q = discharge, m³/s
- A = cross-sectional area of flow, m²
- V = mean cross-sectional velocity, m/s

Based on the combination of the above two formulas, discharge is

$$Q = VA = \frac{1}{n} AR^{\frac{2}{3}} S^{\frac{1}{2}} \dots\dots\dots \text{Eq. 18}$$

2.4.2. Hydraulic Grade Line /Energy Grade line

The energy grade line (EGL) is an imaginary line that represents the total energy along a channel or conduit carrying water. Total energy includes elevation head, velocity head and pressure head. The calculation of the EGL for the full length of the system is critical to the evaluation of a storm drain. In order to develop the EGL, it is necessary to calculate all of the losses through the system. (U.S. Federal High Way Administration, 2001). The hydraulic grade line (HGL) is the last important feature to be established relating to the hydraulic design of storm drains. This grade line aids the designer in determining the acceptability of the proposed system by establishing the elevations along the system to which the water will rise when the system is operating from a flood of design frequency (Ethiopian Road Authority, 2013).

2.4.4. Head Loss

According (T. M. Walski et al ., 2003), Fluids possess energy in three forms. The amount of energy depends on the fluid's movement (kinetic energy), elevation (potential energy), and pressure

(pressure energy). In a hydraulic system, a fluid can have all three types of energy associated with it simultaneously. The total energy associated with a fluid per unit weight of the fluid is called head. The kinetic energy is called velocity head ($V/2g$), the potential energy is called elevation head (Z), and the internal pressure energy is called pressure head (P/γ). While typical units for energy are foot-pounds (Joules), the units of total head are feet (meters).

$$H=z+p/\gamma+V^2/2g \dots\dots\dots \text{Eq. 19}$$

- where :- H= total head (L)
Z = elevation above datum (L)
P =pressure (M/L/T²)
 γ = fluid specific weight (M/L²/T²)
V = velocity (L/T)
g = gravitational acceleration constant (L/T²)

Each point in the system has a unique head associated with it. A line plotted of total head versus distance through a system is called the energy grade line (EGL). The sum of the elevation head and pressure head yields the hydraulic grade line (HGL). Head losses, are generally the result of two mechanisms:

- ✓ Friction along the pipe walls
- ✓ Turbulence due to changes in streamlines through fittings and appurtenances.

Head losses along the pipe wall are called friction losses / head losses/Major Loss due to friction, while losses due to turbulence within the bulk fluid are called Minor losses.

2.5. Best Management Practice (BMP)

The traditional method of managing stormwater has been to remove runoff as quickly as possible from the area through large pipes and concrete channels. An alternative to this strategy, known as low-impact development, establishes economic and land-use planning policies aimed at reducing stormwater runoff in combination with stormwater best management practices (BMPs) that treat stormwater on site. Urban BMPs, such as bio retention systems, porous pavements, permeable patios, rain barrels/cisterns, green roofs, wet ponds, and dry ponds, are common practices implemented in urban areas to treat stormwater runoff quantity and quality (Liu et al., 2015).

There are two broad types of BMPs: structural and non-structural. Structural BMPs are defined as engineered and constructed systems that allow in-situ water quality improvement. Non-structural BMPs comprise institutional and pollution prevention strategies to minimize the transport of pollutants in storm water runoff and/or reduce the volume of runoff generated (A. Taylor & Wong, 2002). BMPs installed are capable of mitigating an even smaller share of urban impacts, primarily because of inadequacies in design standards. Even with these shortcomings, though, results showed that structural BMPs help to sustain aquatic biological communities, especially at

moderately high urbanization levels, where space limits non-structural options (Horner & May, 2002).

2.5.1. Structural Best Management Systems

Structural BMPs are designed to enhance the natural processes of the nutrient cycle by employing systems based on physical, chemical or biological attenuation of nutrients. Structural BMPs are widely used in urban and agricultural environments (O.Barron et al., 2010). There are six typical storm water structural BMPs: dry and wet ponds, wetlands, filtering and infiltration practices, and swales (R. Pennington et al., 2003). The successful selection of structural BMPs appropriate for an urban catchment relies on the understanding of catchment-specific issues, including water quality parameters of concern, the landscape, hydrology and hydrogeology of the catchment, BMP maintenance requirements, land availability and construction and maintenance costs (O.Barron et al., 2010). Structural BMPs are not able to achieve water quality objectives by themselves, especially in urbanized areas. Therefore, additional program elements, including public education, outreach, and involvement, warrant inclusion in communities' watershed protection activities are suggested. Additional nonstructural best management practices that might be considered include: water sensitive urban design, planning controls, legislative controls, and municipal maintenance controls (R. Pennington et al., 2003).

2.5.2. Non-Structural Best Management Systems

The non-structural BMPs involve preventive actions that do not include structural solutions, but rules and regulations, which aim to prevent or restrict pollutants' load in storm water runoff (A. Taylor & Wong, 2002). Nonstructural BMPs are usually more economically efficient, as they are strongly to prevention and not mitigation. However, it is preferable in most urban runoff management plans to use a combination of structural and non-structural BMPs to achieve better results (Yannopoulos S.I. et al., 2013). They do not involve fixed, permanent facilities and they usually work by changing behavior through government regulation (e.g. planning and environmental laws), persuasion and/or economic instruments (A. Taylor & Wong, 2002).

2.6. Urban Stormwater Drainage Models

A model is a schematic description of a system, theory, or phenomenon that accounts for its known or inferred properties and may be used for further study of its characteristics. Every model is a simplification of the real system. The applicability of a given model depends on the degree of simplification that can be applied while still representing the system in a meaningful way to solve a problem (S. Dent, 2005).

A runoff model can be defined as a set of equations that helps in the estimation of runoff as a function of various parameters used for describing watershed characteristics. The two important inputs required for all models are rainfall data and drainage area. Along with these, water shed characteristics like soil properties, vegetation cover, watershed topography, soil moisture content, characteristics of ground water aquifer are also considered. Hydrological models are now a day considered as an important and necessary tool for water and environment resource management (G. K. Devi et al., 2015). There are several computer soft wares that are currently available for urban storm water drainage modelling. Since, the urban drainage system has hydrological & hydraulic attributes, it is better to use a model that has combined hydrological /hydraulics components to get a better representation of the flows and some of them are discussed as follows.

2.6.1 SWMM

It is a dynamic rainfall-runoff simulation model used for a single rainfall event or for long-term simulation of runoff quantity and quality of water in predominantly urbanized areas. The model monitors the quantity and quality of runoff, flow-rate, water depth and quality in each sub catchment area (P. Hlстик, 2017). It conceptualizes a drainage system as a series of water and material flows between several major environmental compartments. These compartments include the atmosphere compartment, the land surface compartment, the groundwater compartment and the transport compartment. In general, the channel and pipe flow routings are governed by the Saint-Venant equations. The Saint-Venant equations consist of mass Conservation equation (Equation (20)) and momentum Conservation equation (Equation (21)) for gradually varied, unsteady flow. In the transport compartment, SWMM solves the Saint-Venant equation using an explicit finite difference method and successive approximation.

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \dots\dots\dots \text{Eq. 20}$$

$$\frac{\partial h}{\partial x} + \frac{v}{g} \frac{\partial v}{\partial x} + \frac{1}{g} \frac{\partial v}{\partial t} = S_o - S_f \dots\dots\dots \text{Eq. 21}$$

Where : Q is discharge, A is area of flow cross section, h is depth of water, x is longitudinal distance, t denotes time, v is velocity, g is gravitational acceleration, S₀ is channel slope term and S_f is friction term (W. Chen & G. Huang, 2017). The simulation time in individual channels and pipelines consists of multiple time steps. The results can be displayed as river basin maps, time series charts and tables and as statistical frequency analyses. The run-off process is based on sub-catchments with rainfalls with resulting surface runoff and its pollution. Furthermore, the runoff is conveyed into a system of conduits, manholes and storm water tanks, pumping stations, and treatment controls can also be included. The SWMM monitors the quantity and quality of runoff

within each area. It also monitors the flow-rate, depth, and water quality in the individual pipe and channel sections during the simulation which consists of several time steps (P. Hlustik, 2017).

2.6.2. SEWERCAD

Sewer CAD is a program for the design, analysis and planning of gravity flow and pressure low through pipe networks. The operational behavior of various gravity and pressure network elements (manholes, outlets, junction chambers, pressure junctions, wet wells and pumping stations) can be simulated. Prototype elements can be defined with default characteristics, reducing data entry requirements if a group of network elements share common data (G.Freni et al.,2003). Sewer CAD software is used to analyses the pipe networks, design the flow, velocity, and surface features topography in somehow to delineate the paths and drawn the profiles to show the ground surface, elevations (Amiri et al., 2016).Sewer CAD can run both Steady State Analyses, modelling a single instant in time, and extended period analyses, modelling a network over a specified duration of time; moreover, the program allows to automatically design gravity piping and structures, specifying the elements to be designed, from a single pipe size to the entire system, or anything in between, intending the program's design only as a preliminary step (G. Freni et al., 2003).

2.6.3. MIKE URBAN

MIKE URBAN is formulated by Danish Hydraulic Institute to perform 1D flood model. It is GIS based software system which performs numeric modelling (1D) and provides platform for analysis of urban storm water collection and drainage, and waste water systems for both combined and separate sewer collection system. (M. Basnet, 2017). It is a module software. The main modules are : Model Manger ,Water Distribution ,Collection system(CS) and 2D Overland flow. The Model Manger is a the base module of the interface and uses the computational engine SWMM for stormwater and Sewer systems(J. Lind, 2015)

2.6.4. Storm CAD

Storage, Treatment, Overflow, Runoff Model (STORM) is a modelling software that has been developed by US Corps of Engineers. This software is capable to simulate the runoff in the urban or rural catchment area in response to precipitation. STORM not only can be used for single event but it also can be used to simulate continuous model by using hourly time steps. Runoff of every sub catchment will simply accumulate in one catchment by using hourly precipitation. To calculate hourly runoff unit hydrograph method, coefficient method and soil-complex-cover method can be chosen to use. Runoff is a linear relationship between runoff and the precipitation minus rainfall interception (C. Zoppou, 1999). Storm CAD is used for the design and analysis of storm sewer systems. The model assumes uniform flow and can account for supercritical and subcritical flow.

The model considers all gradually varied flow profiles: horizontal, mild, critical, steep and adverse slopes, in combination with the subcritical, critical, and supercritical types of flow. The model provides additional design functionality through a constraint-based design. With this option the model can automatically design a system based on constraint parameters provided by the user (J. Lind, 2015).

2.6.5. XP STORM

XP STORM is one of many types of software of XP solutions that offers numerous software technologies and professional solutions worldwide to government agencies, engineering and environmental management organizations to plan, design, simulate and manage the physical and social environment (F. Akram et al.,2014). It is an integrated hydraulic and hydrologic model used for storm water and river Systems/floodplain management. Areas of use regarding storm water analysis, design and planning includes among others Storm water master plan design, LID structures, Detention pond optimization, 1D and 2D urban flooding. The program has GIS and CAD integrations and can import and export several GIS and CAD formats. SWMM 5.0 formats can also be imported and exported. The simulation results can be presented as result documents, tables, profile plots, flood mapping and animations. (J. Lind, 2015).

Chapter Three: Material and Methods

3.1. Description of the Area

Mojo is one of the towns in the country that had emerged along the Railway line from Djibouti to Addis Ababa, constructed between the years 1894 and 1917. It is one of the pre- Italian periods of the country that had been considered as from their early period of formation as satellite towns along railway lines east of the capital. It comprises most important industrial establishments and also hosts one of the dry stations (port) of the country. Mojo was recognized as a town since 1941 and it gained a municipal status in 1944 (Mojo Municipality, 2015).

3.1.1. Location

Mojo town is located in east Shewa zone of Oromia regional state at Lume wereda (district) some 70 Km away from the capital city Addis Ababa toward the South east direction with central coordinates of $8^{\circ} 35' 25''N$ and $39^{\circ} 07' 46''E$ within the range of 1694-1884 meter elevation above sea level. It has an area of 5180.14ha (51.80 Sq. Km).The town is divided in two kebeles.

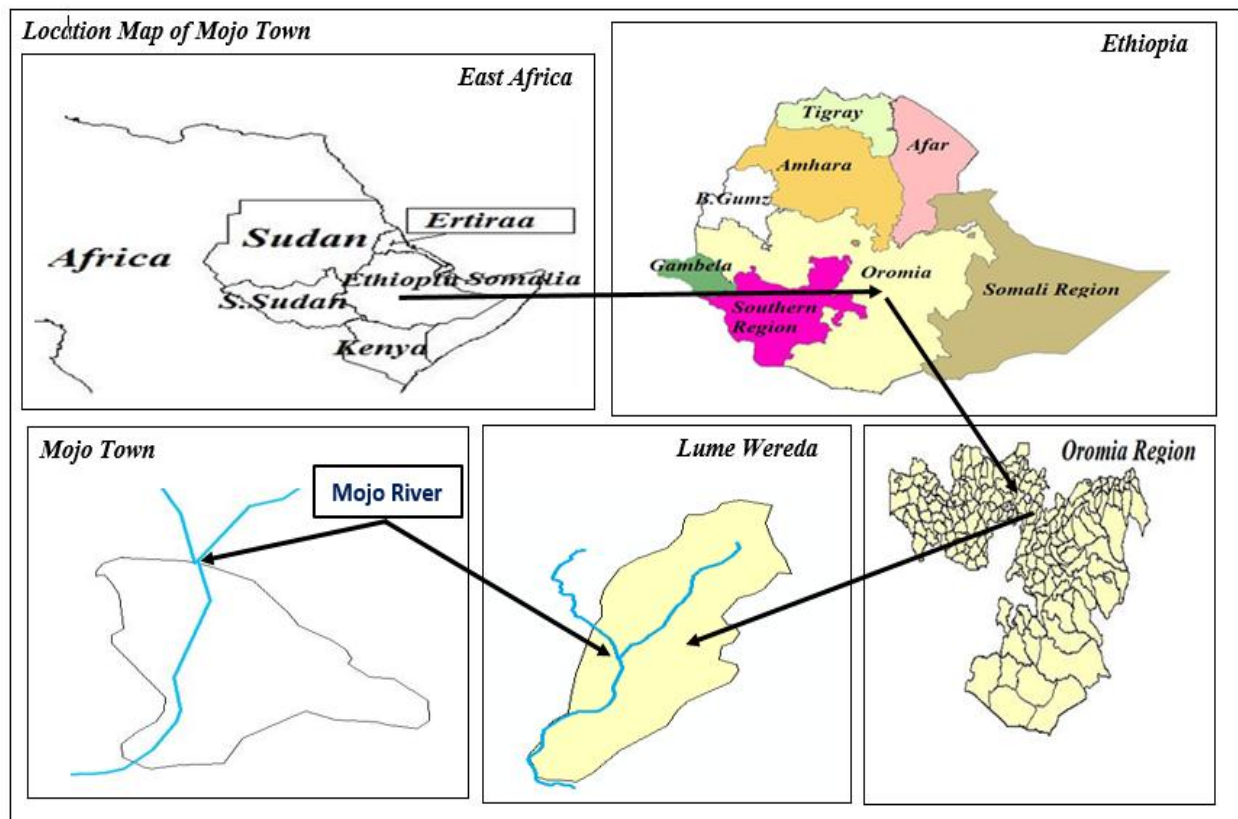


Figure 5. Mojo Town Location Map.

3.1.2. Population

Total population of the town is 29,547 out which 14,355 male and 15192 females(CSA, 2007). According to CSA (2007) report, the population growth rate was 3 % and based on this rate the

existing total population at this time is estimated to be 42127. Whereas, based on Mojo municipality estimation the total population is 100,000. Due to the establishment of industries and the job opportunity created by government and individual investment projects, a number of job seekers massively came to the town from different areas. Hence, this might be the reason for the drastic population increase of the town according to the population estimation of the municipality.

3.1.3. Topography

Majority of Mojo town is characterized by gentle slope topography however out of the town particularly at northern direction there are undulating type of hills extended from east to west direction. Elevation increases from west to east and from south to north and vice versa.

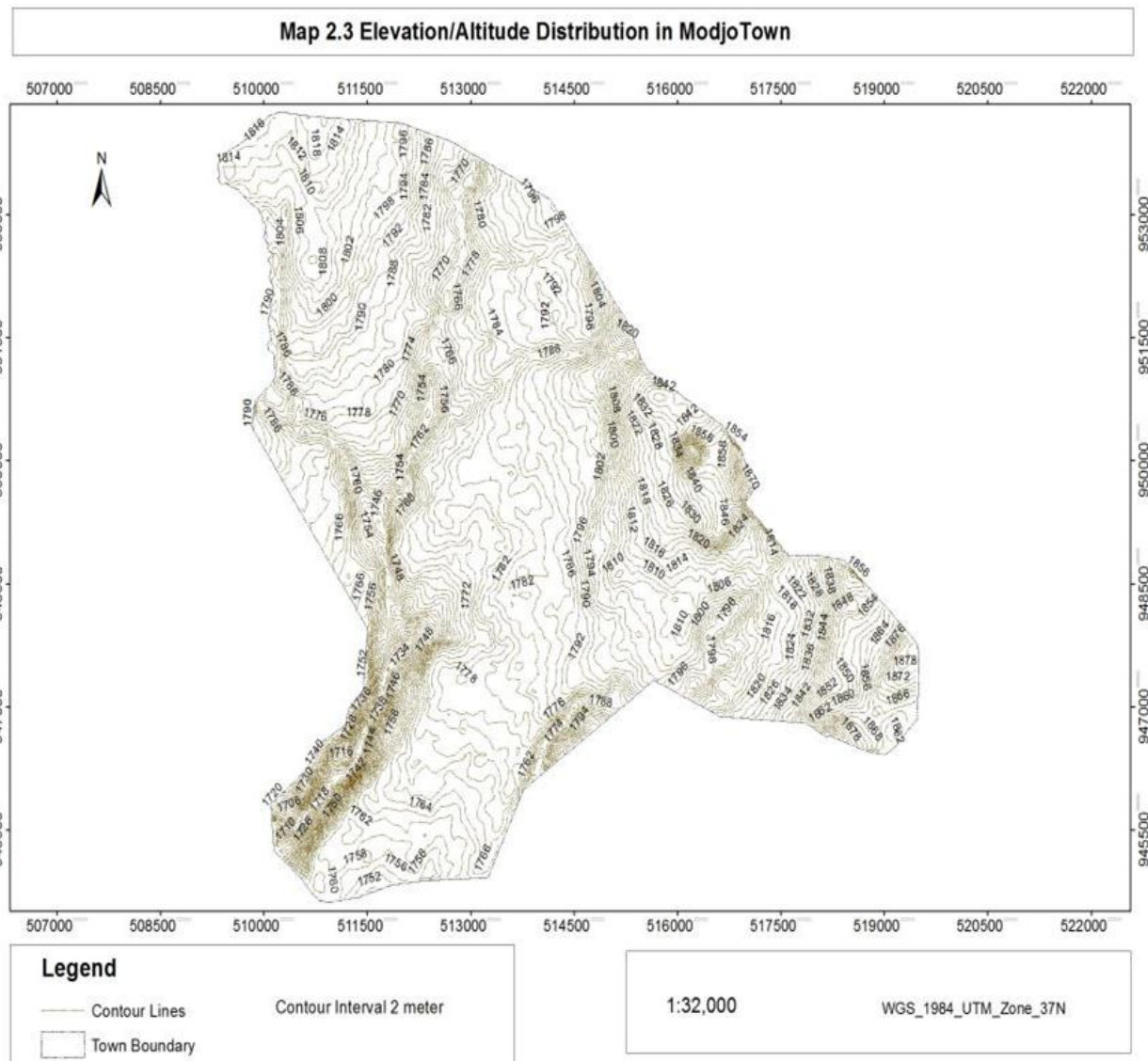


Figure 6. Topographic Map of Mojo Town (Source: Mojo Socio economic study report, 2015).

According to socio economic study for the preparation of Mojo town master plan (2015), slope classification of the town is indicated in table 1.

Table 1. Slope classification of Mojo Town.

Slope Classification (%)	Area in ha	%
0-1	594.9	11.6
1-2	1390.80	27.2
2-5	2213.4	43.3
5-10	711.0	13.9
10-15	134.20	2.6.
15-20	54.5	1.1
>20	8.9	0.2
Total	5108.1	

Most part which is nearly half of the town is categorized as gentle slope due to this reason it is not challenging to construct urban infrastructures like urban drainage, road network, and water supply lines e.t.c. However, it is important to note that construction of urban drainage requires appropriate slope selection or provision in order to attain cleansing flow velocity that helps to have smooth storm water movement.

3.1.4. Climate

The town is classified as “Dry Woina-Dega” and the mean annual rainfall is about 863 mm. The mean annual maximum and means annual minimum is 27.7 °C and 12.7°C respectively. Whereas the mean annual temperature is 20.05° C.

Based on 35 years (1986-2018) daily temperature data from Ethiopian Meteorology Agency, the mean annual monthly temperature graph of the town is depicted in fig. 7.

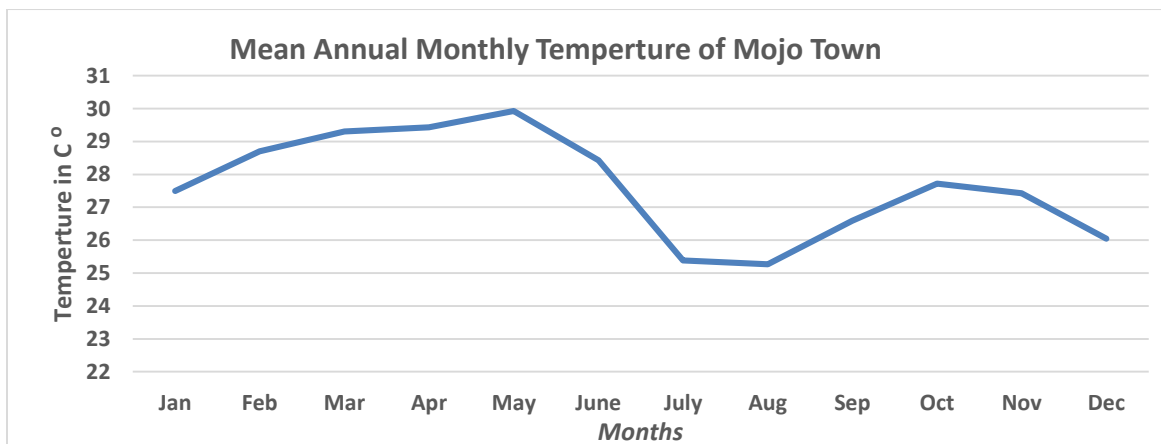


Figure 7. Mean annual monthly temperature graph

In similar fashion, mean annual monthly rainfall graph of the town is shown in fig. 8.

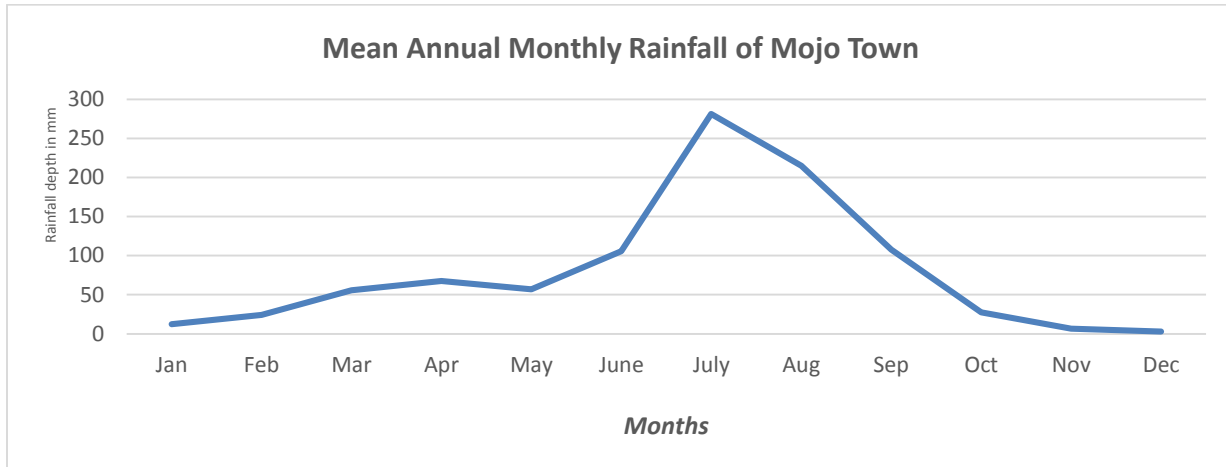


Figure 8. Mean Annual Monthly rainfall graph of Mojo Town

The monthly rainfall distribution indicated that the highest rainfall occurs during summer in July and August. Therefore, it is expected to have more stormwater that might be a challenge for the drainage system of the town.

3.1.5. Soil and Geology

3.1.5.1. Soil

According to (Oromia Urban Planning Institute, 2009), there are three types of soils observed in the town and its surrounding: -

1. **Vertisols (black cotton)** found in the central, south and south western part of the town.
2. **Sandy and Silty soils:** - these soils are observed in different colours in the areas of the town i.e. light brown, dark brown and red. Although clay soils are found in these category Silt clays, sandy silt and loam soil which is the combination of clay and silt are the main components of these soils. These soils are found in north eastern, eastern and central part of the town.
3. **Alluvium:** - the main compositions of these soils are sandy silt, sandy clay and flood plain. These deposits are found in the southern, northern and eastern part of the town.

Based on the information collected from Ministry of Agriculture, the soil texture of the model area is dominantly clay with Vertisols and Andosols soil types. Naturally, clay is the most impermeable soil which could be a barrier to have more storm water infiltration in the town that might be one of the causes to have more surface runoff.

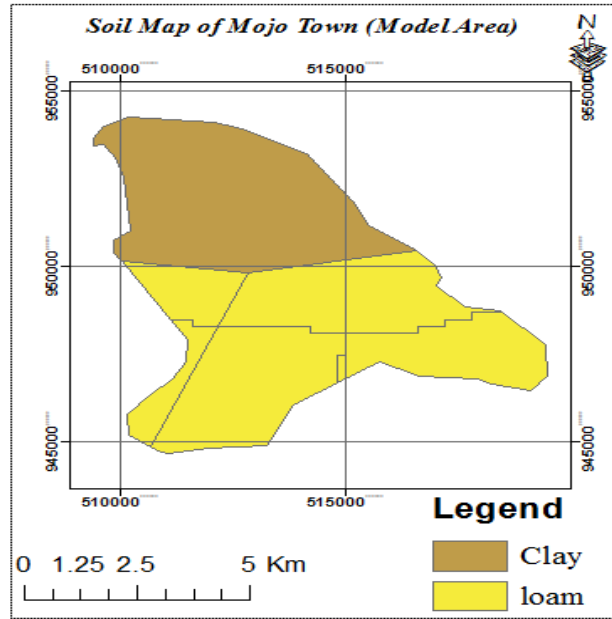


Figure 9. Soil map of mojo town

3.1.5.2. Geology

Mojo and its surrounding areas are supposed to have been covered by the ancestral lake during the pluvial period of the Quaternary. Currently in the areas of Mojo Town the lacustrine sedimentation is found. These lacustrine sediments are the redeposit of volcanic sands, silt stone, sand stone, and diatomite with intercalations of water-laid tuff. (Oromia Urban Planning Institute, 2009). According to USGS, the town particularly the model area is identified as Tertiary (undivided) & Quaternary (Extrusive & intrusive rock) based on geological time scale as illustrated in Fig.10.

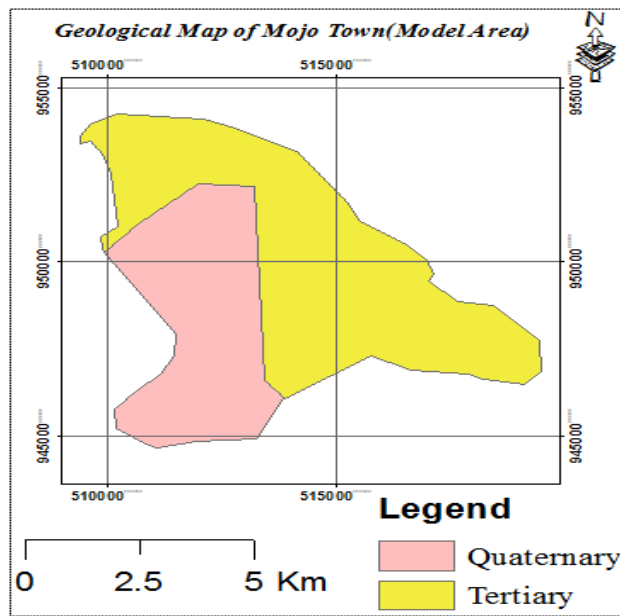


Figure 10. Geological map of Mojo Town (Model Area)

Tuff, pumices, ignimbrite, scoria, basalt, pyroclastic flows etc. are the Quaternary volcanic rocks in the area of the town. In the northern and southern areas of the town scoria cone and basaltic rocky areas are found. The other volcanic rock found abundantly in the area of the town is ignimbrite. It is a good construction material and found in Bowwa stream valley in combination with lacustrine deposits. In the town this stone is used for construction of walls and stone slabs (Mojo Municipality, 2015). Deterioration of river banks and land slide is a serious problem in some parts of the towns due to excessive extraction of construction materials coupled with high storm runoff.

3.1.6. Land Use

The town is labelled as a second grade City and at the time being it has its own Structural plan for a period of 10 years that has been implemented since 2014. According to the Structural plan, the town is classified for seven major classes. Those are for Housing, Administration, Business & Commerce, Social Service, Manufacturing & Storage, Green, Recreation & Environment, Road Network and Transport Service.

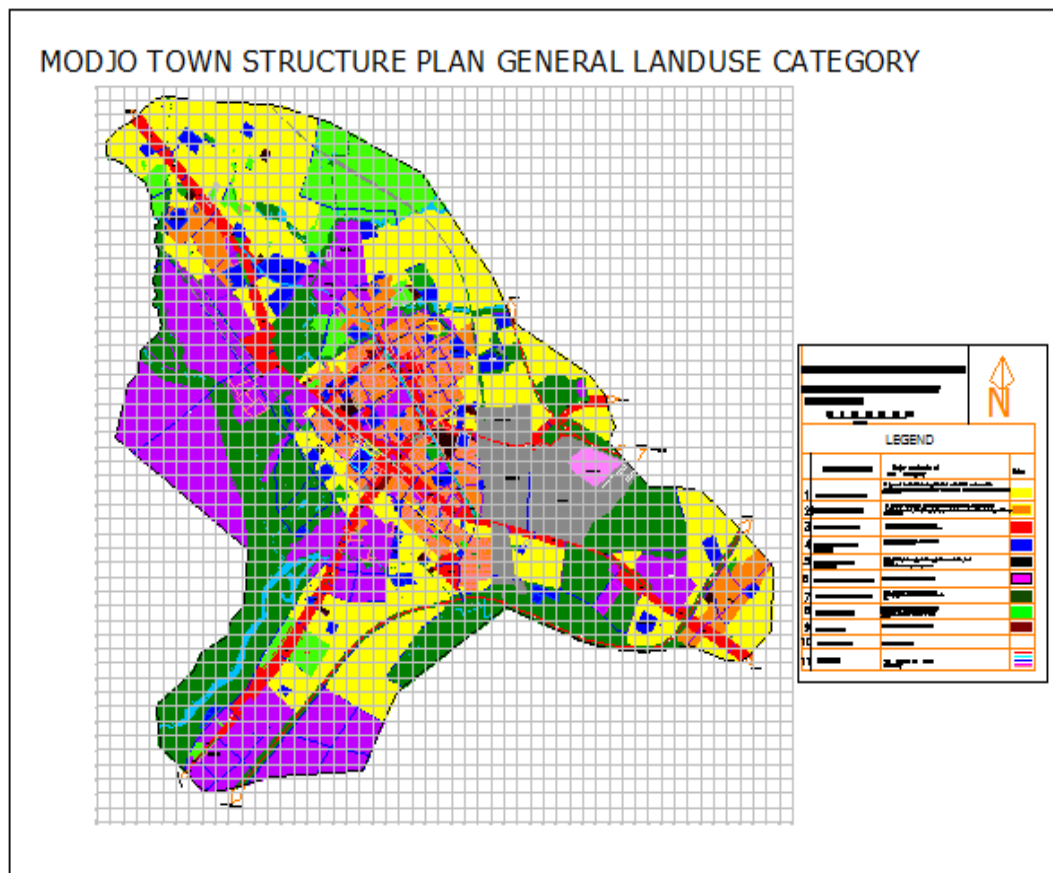









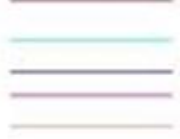


Figure 11. Structural Plan of Mojo Town (Mojo Socio economic study report, 2015).

Table 2. Legend for Mojo town structure plan general land use category

	Land Use Category	Major Contents of the category	Color
1	Proposed Mixed Housing	All types of Residential development Educational service, Health service: clinic, Neighborhood market /Gulits / Shopping, medium & Manufacturing & storage Administration.	
2	Existing Mixed Housing	All types of Residential development Educational service, Health service: clinic, Neighborhood market /Gulits / Shopping, medium & Manufacturing & storage Administration.	
3	Commercial Activities	Shops ,market ,hotels ,different hierarchy of commercial activities and bank & insurances	
4	Social & Municipal services	All social services (education ,health, warships ,community facilities) and Municipalizes services	
5	Infrastructure & Transport	Public Utilities / water supply ,electric supply, telecommunication / Road networks Terminals for major transportation	
6	Manufacturing & Storage	Industry, Manufacturing warehouses, Storage	
7	Open Space & Environment	Open space for outdoor recreation, playground play fields, stadium ,buffer area forests e t c	
8	Urban Agriculture	Including farming & animal rearing such as horticulture, field cropping, livestock fattening ,rearing & other activities in urban areas	
9	Administration	All Administration such as offices	
10	Special function	Gorges & Rivers	
11	Road Centers	The center of road hierarchy	

According to the master plan of the town, 3630.5 ha is allocated for built up area with the proportion of 70 % of the town coverage. If the master plan is properly implemented provided that good planning & management of urban drainage infrastructure and green development are implemented, the prospect is good to avoid storm water induced problems.

3.1.7. Road Net Work and Urban Drainage

Based on the study (Mojo Municipality, 2015), the total distance of different types of roads is described in the table 2.

Table 3. *Length, Width and Area of Road Net work*

No	Road Type	Width(m)	Length (K.m)	%	Area(ha)	%
1	Express Way	90	13.98	5.12	65.65	13.1
2	Primary & Sub Arterial Street	40	27.33	10.02	85.81	17.1
3	Collector Streets	15-30	139.32	51.07	256.66	51.2
4	Local Streets	10-15	92.17	33.79	93.68	18.7
	Total		272.8	100	501.8	100

The drainage system that was constructed in the town is intended to entertain only stormwater flow. Domestic waste water from individuals is being disposed to the open environment. Moreover, industries, with the possible pollution causing effluents, are expected to release their waste water to the nearby water bodies after having treatment by their own treatment plant based on EPA standard. However, most of the industries have not been performing based on their responsibility as it is indicated on different studies and reports.

Table 4. *Length of urban drainage infrastructure*

No	Urban Drainage Type	Length (K.m)	Percent (%)
1	Concrete	2.25	2.4
2	Masonry	22.11	23.7
3	Masonry-trachytic Stone	43.44	46.6
4	Earthen	25.44	27.3
	Total	93.24	100

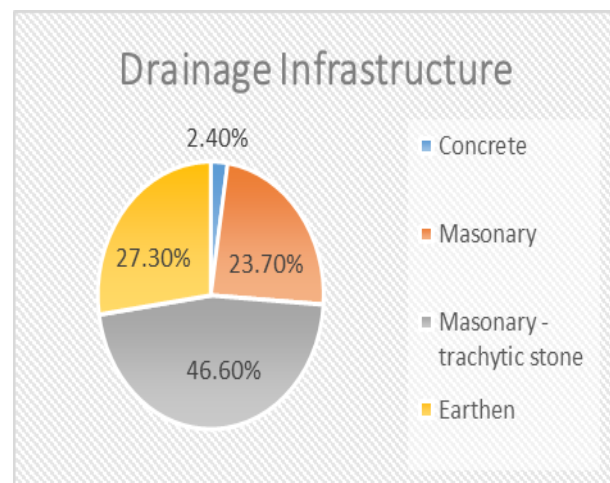


Figure 12. Drainage Infrastructures of Mojo Town



Figure 13. Some pictures of drainage infrastructures of the town

The existing drainage system of the towns is facing a number of challenges that have been a challenge to have a proper drainage system performance. Those are:

- ✓ Absence of drainage master plan
- ✓ Absence of well networked drainage infrastructures
- ✓ Poor waste management system particularly solid waste



Figure 14. Dumping of Solid wastes in the urban drainage infrastructures

3.1.8. Environment

The town environment has been highly threatened due to the fact that there are several types of industries like tannery, abattoir, chemical etc. functioning in and near by the town provided that most of solid and liquid wastes are being emitted to the environment without treatment.

Thus, hazardous industrial wastes, which are harmful for living things, are emitted to the environment which could be a cause for the pollution of water bodies, soil and air. In similar fashions, because of lack of proper waste (Solid & Liquid) management infrastructures, different types of wastes from domestic sources are being released to open field.

Due to all these reasons, components of the environment like water, soil and are highly being degraded as the time goes on. Mojo, similar to other rift-valley towns has felt on different natural environmental hazardous/ problems because of their location. Erosion, land cracking, land slide and the like are some of the major problems of these common environmental problems which are particular to these towns. Erosion is taken to be the major problem than the other environmental problems.



Figure 15. Pictures of Soil erosion around Moshe area

This can be practically seen from the current physical observation of the soil types of the locality particularly at the bank of Mojo River and Bowa gorge/seasonal Rive as well as Malka lammi seasonal river and the bare land at Xadde Village locality that occurred due to over utilization of resources.(Mojo Municipality, 2015).

3.1.8.1. Solid & liquid waste Management

There is no an organized and systematic way to collect solid wastes from the sources in the town. Absence of modern landfill, which helps to dispose the solid waste safely, is also another problem. Dumping of solid waste in open area from domestic and non-domestic sources is a common practices in the town. The solid waste is not timely and entirely collected. As it is common in other towns, people are forced to burn solid waste to get rid of the wastes from their vicinity. In similar

manner, there is no a system to collect & transport liquid waste from the sources. Therefore, liquid wastes is simply discharged to open environment or the nearest water body. The nature of the liquid waste is hazardous to human being and environment due to the fact that a number of industries like tannery, abattoir, and textile e.t.c massively working in the town.

3.2. Research Inputs

Different types of primary and secondary data as well as different equipment, software packages, and computer applications have been used as an input for the research.

3.2.1. Data Collection

Relevant data and information were collected directly from the research area as primary data and from the concerned offices & individuals as a secondary data.

3.2.1.1. Primary data

Neither soft nor hard copy of an organized existing drainage network and the corresponding dimension data were available in Mojo municipality. Due to this reason, relevant data of the drainage network was collected from the model area which is important input for the model.

Table 5.Types of urban drainage infrastructural data collected

No	Details
1	Types of Conduit & Junctions (Material)
2	Shape of Conduit & Junctions
3	Geometrical Dimension of Conduit & Junctions Like Length, Depth, Width
4	Location of Conduit & Junctions (X,Y,Z coordinates)

Moreover, field visit were frequently made so as to make physical observation about the existing status of the drainage system.

3.2.1.2. Secondary data

For this research, a number of secondary data were required out of which some were collected that are listed in table 5.

Table 6. List of secondary data collected.

<i>Data type</i>	<i>Source</i>	<i>Purpose</i>
<i>Metrological data</i> ➤ Daily Rainfall data (1997-2019) ➤ Daily Temperature data (1986-2018)	National Metrology Agency	To formulate IDF curve.
<i>Land surface data</i> ➤ Digital Elevation Model of the area (Resolution of 30 meter X 30 meter)	Satellite image from USGS (United States Geological Survey)	To make watershed & topographic map of the area.
<i>Master plan of the town</i> ✓ Structural plan of the town 10 yrs plan from 2015-2014	Mojo Municipality	To examine the existing land use with respect to the structural plan
<i>Soil data for the town (texture)</i>	Ministry of Agriculture	To use as an input for the model.

3.2.2. Equipment

In the process, Digital-Camera, Global Positioning System (GPS) and meter were frequently used to collect data from the field and to take photographs in some important locations. GPS was used to collect data about location(X & Y coordinates) of drainage infrastructure as well as elevation (Z).The GPS was working with $\pm 3m$ accuracy. The meter was used to measure dimensions of drainage infrastructures without a significant accuracy problem.

3.2.3. Software Packages

This research is completely dependent on computer software and applications. Software and computer application which have been used in the research process are listed in following table 6.

Table 7. List of Software and computer applications.

<i>Software/Application</i>	<i>Used for</i>
Google Earth	✓ To select alternative model areas. ✓ To do previous & existing land use of the model area. ✓ To draw road/drainage network of the study area.
Arc-Gis (10.5 version)	✓ To identify stream and to do flow direction & watershed of the area. ✓ To do topographic map of the study area. ✓ To do land use classification of the study area.
Digital Elevation Model (DEM)	✓ An input for Arc-GIS to do watershed and related issues.
Global mapper	✓ To do contour map of the study area.
Microsoft excel, Microsoft word	✓ Large data manipulation, data analysis and manuscript writing.
SWMM	✓ Storm water quantity simulation

3.3. Methods

3.3.1. Rain fall data analysis

Twenty-three years (1997 -2019) daily rainfall data of Mojo town (metrological station) were collected from National Metrological Agency. Meanwhile, out of the total rain fall data about 3% was not available. Particularly, the missing is more during rainy season which was 5 %.On the other hand, missing of recent years (2018 & 2019) data were more than 65% which was difficult to use it for research purpose. Additional data from the surrounding three metrological stations (Adama, Bishoftu, and Koka) were collected in order to fill the missing data between 1997 -2017 to insure data continuity provided that 2018 & 2019 were disregarded. Hence, totally 21 years rainfall data were used.

3.3.1.1. Filling Rainfall Data

There are several techniques to fill rainfall missing data out of which *Inverse Distance Weighting* (IDW) is selected and employed for this research purpose.

According to (Chen et al., 2014), in case of the weight for each station is assumed to be inversely proportional to its squared distance of the target station from the neighbouring station with data.

$$P_x = \frac{\sum_{i=1}^m \frac{1}{d_i^2} P_i}{\sum_{i=1}^m \frac{1}{d_i^2}} \dots\dots\dots \text{Eq. 22}$$

Where P_x : Average perception

P_i : perception of nearby rain gauge stations

d : distance between rain gauge stations

Every missed daily rainfall data from 1997-2017 is estimated using the nearby two metrological stations (Adama, Bishoftu, and Koka).

3.3.1.2. Identification of Outliers

Outliers are part of the data which are found significantly different from the trend of the whole data due to different reasons. It may be caused due to measuring equipment error, lack of curiosity induced by the data collectors' etc. It is important to identify those data and reject out of the total in order to reduce their negative impact that might cause deviation from the actual fact. Therefore, the following equations in logarithmic scale were used to identify outliers among the yearly maximum daily rainfall data .

$$X_H = \bar{X} + K_N S , \text{ for high outliers } \dots\dots\dots \text{Eq. 23}$$

$$X_L = \bar{X} - K_N S , \text{ for lower outliers } \dots\dots\dots \text{Eq.24}$$

Where:

X_H = higher outlier threshold in log unit

X_L = lower outlier threshold in log unit

\bar{X} = Mean (logarithm)

S= Standard deviation of logarithm

K_N = K Value from table for sample size n

Mean, Standard deviation were computed using Microsoft excel software whereas K_N value is obtained from outlier test K_N table (Annex 1).Outliers were not found in the list of yearly maximum daily rain fall data ,after calculation is made using the formulae. The result is displayed in table attached as Annex 2. Therefore, all the data stated in the table were used for the IDF formulation purpose.

3.3.1.3. Data Consistency

Double-mass curve is graphical method which is used to evaluate the trend and consistency of rainfall data record of a particular gauging station with respect to other stations through time. Before doing any analysis using the data at hand, it is very important to check its consistency in order to avoid biasness due to lack of input data consistency. Based on this fact, average rainfall data (yearly) obtained from other 2 nearby stations (Adama and Koka) and Mojo station were used to construct the curve as illustrated below. The detail data is attached in table as Annex 3.

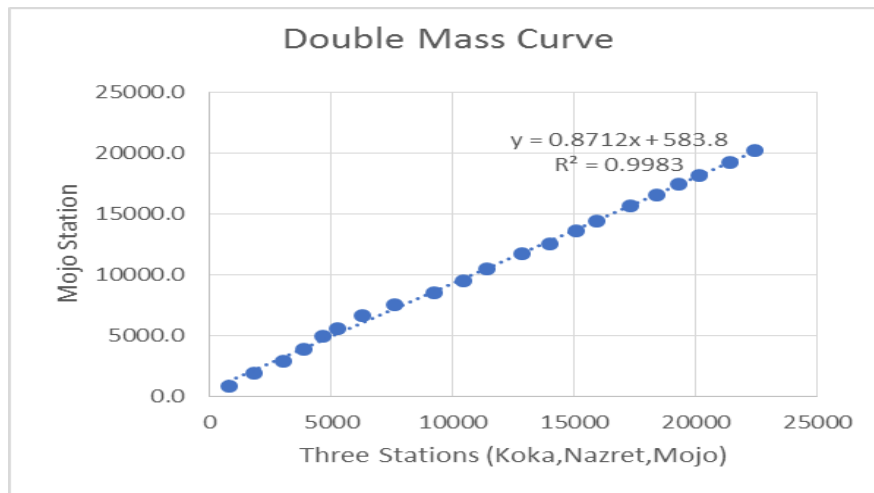


Figure 16. Double mass curve (Rain fall) of mojo town

According to the R^2 (Coefficient of Determination) value on the graph, the data is so consistent that does not require adjustment.

3.3.1.4. Formulation of IDF curve

Rainfall intensity-duration-frequency (IDF) curve is a graphical representation of the amount of rainfall that falls within a given period of time for particular area. An IDF curve helps to know the expected rainfall intensity of a given duration of storm taking the desired return period in to account. The curve is an important in put to design hydraulic structures including urban drainage

infrastructure. A number of techniques are available to formulate IDF curve. However two types (Extreme value type I distribution [Gumbel] & Logarithmic Pearson Type-III distribution) which are the most commonly used in Ethiopia were employed for this research purpose using twenty one years daily rain fall (24 hr record) data.

A. Annual Maximum Rainfall Serious

With the help of Microsoft excel software, the maximum rainfall records were selected for each year.

Table 8. Annual maximum rainfall in daily (mm/24 hr) basis for 1997-2017

Year	0	1	2	3	4	5	6	7	8	9
1990								54.2	83	92
2000	44.8	42.5	46.8	85.1	47.2	56.3	44.5	38.8	94.9	73.2
2010	56.1	44.3	81.4	62.8	61.8	44.8	91.6	64.4		

B. Short period rainfall computation

Based on the annual maximum rain fall record (24 hr) of each year, short period rainfall intensities for different durations (10-180minutes) were computed using rainfall intensity-duration-frequency relationship formula (Eq. 3) that is recommended by Drainage Design Manual of ERA (2013). An average value of the range (0.78-1.09) is used for n that is 0.94. The result is tabulated and attached as Annex 4.

C. Probability distribution

Extreme value type I distribution (Gumbel distribution) and Log Pearson Type III distribution are commonly used for extreme hydrological series (Abdo K. et al., 2017).

1. Extreme value type I distribution (Gumbel distribution)

- ✓ Based on an estimated rain fall for each short period duration, average (P_{av}) and standard deviation (s) is computed using Microsoft excel software. The obtained result is attached as Annex 5 in tabular form.
- ✓ Gumbel frequency factor (K_T) is computed using a formula (Eq. 5) for each return period.

Table 9. Gumbel frequency factor (K_T) value

Return period	K_T value
2	-0.16427
5	0.719457
10	1.304563
25	2.043846
50	2.592288
100	3.136681

- ✓ Based on av. rainfall (P_{av}), standard deviation (s) and Gumbel frequency factor (K_T) parameters using a formula (Eq. 4), rainfall intensity for the 2, 5, 10, 25, 50 and 100 return periods is computed for each duration. The computed value is attached as Annex 6.

2. Log Pearson Type-III distribution

- ✓ The logarithmic value of estimated rain fall (for each short period duration) is computed. The values are organized in tabular form and attached as Annex 7.
- ✓ Based on the rainfall logarithmic value, the average (\bar{x}), standard deviation (s) and skew coefficient (G) are computed using Microsoft software. The result in tabular form is attached as Annex 8.
- ✓ K value is computed using skew coefficient (G) and return period from K value table that is attached as Annex 9.
- ✓ Using the average (\bar{x}), standard deviation (S) & K value, intensity is computed using the formula (Eq. 9) in logarithmic scale. Ultimately, the antilogarithm of the values were computed that enable to obtain the intensity duration frequency curve. The values are displayed in table attached as Annex 10.

3.3.2. Selection of Model Area

After having three days visit through the town and consultation with the concerned Mojo municipality experts, a model area which has an area of 354 ha was selected at the eastern part of the town. The rational to select that particular area as a model is it has been severely affected due to over flow of drains and its adverse consequences particularly after the construction of the two mega projects: Addis Ababa-Djibouti Rail way (new) and Addis Ababa-Adama Express way Road.

3.3.2.1. Catchment Delineation

The catchment was initially demarcated based on DEM (30m resolution) combined with drainage network using the Arc GIS and modified according to the site observation of stormwater lines and out falls. The catchment was divided into 10 watershed sub catchments.

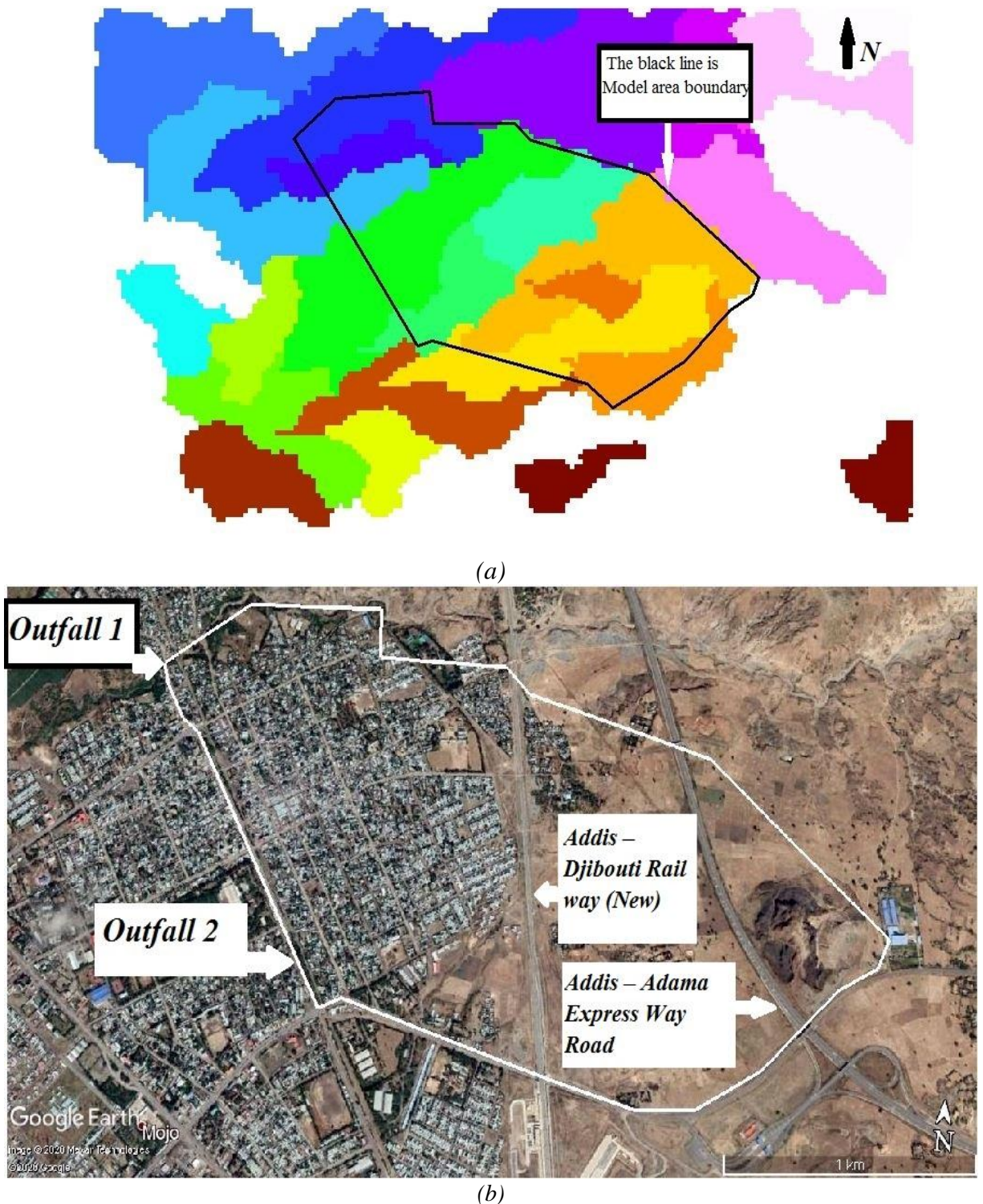


Figure 17. Watershed & location of selected model area (a & b)

The model area is being used for business, commercial, churches, mosques, residential, manufacturing & storage functions. Moreover, other major infrastructures like road network & transport, social services are existing. The model area is vulnerable for flood during rainy season.

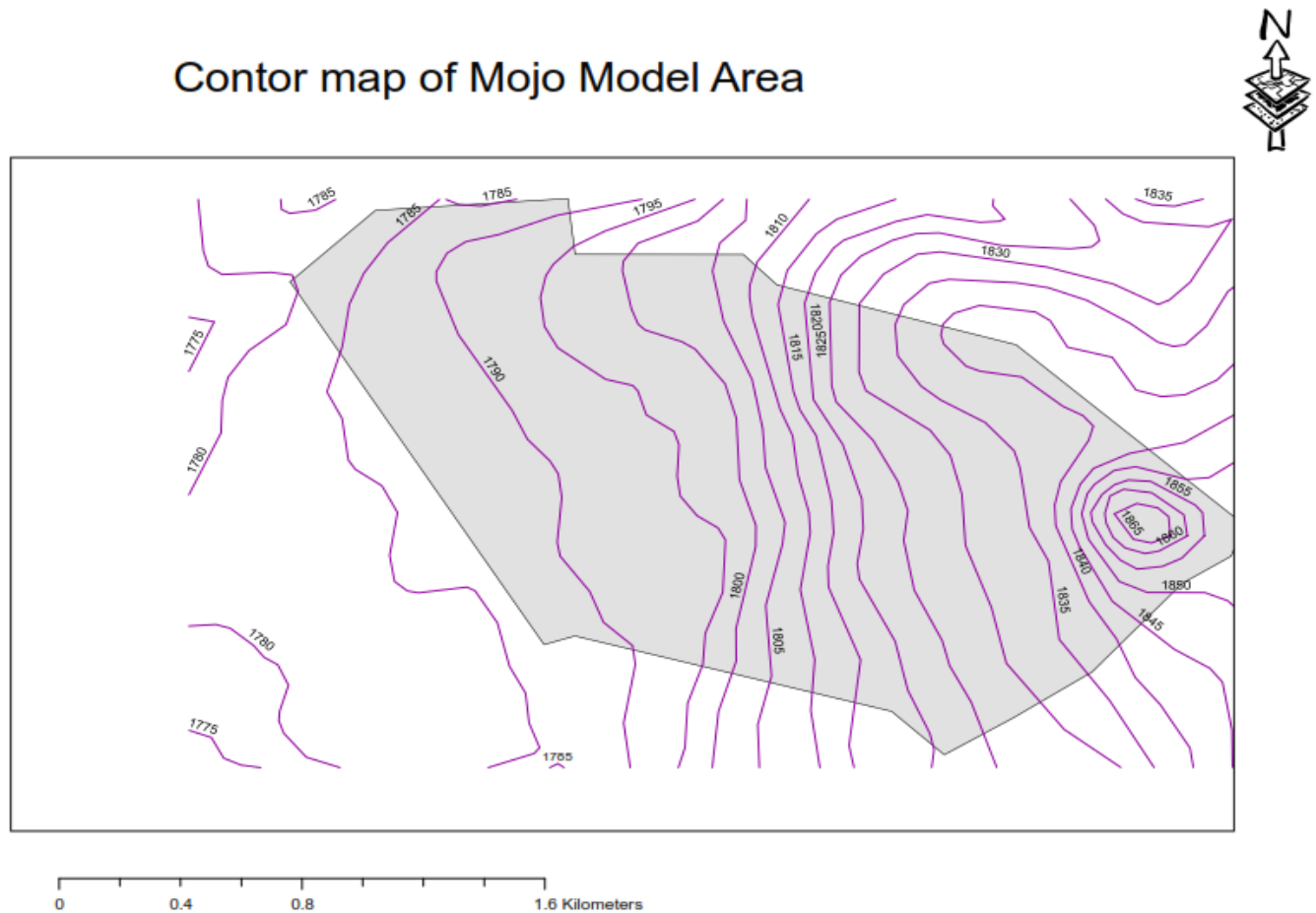


Figure 18. Contour map of Mojo model area

3.3.2.2. Land Cover/Land Use

Land Use Land Cover (LULC) refers to two separate terminologies that are often used interchangeably. Land Cover can be defined as the physical characteristics of the earth's surface which involve vegetation, water, soil, and other physical features created through human activities like settlements, while Land Use refers to land used by humans for habitats concerning economic activities (J.S. Rawat, 2015). In urban areas, there has been a great tendency of change from previous areas like vegetated areas to impervious areas like building, road which induces more runoff and flooding. This is a common type of practise in most of cities & towns of Ethiopia.

Initially, it was intended to examine the land use change of the model area using satellite image of 30 m resolution by making land use classification using Arc-GIS software. Due to the fact that the model area is very small and the resolution of the image is coarse, it was difficult to make land use classification by satellite image. Using digitation technique, Google earth image of 2009, 2014 & 2019 were used to classify the land use of the model area in order to observe the land use change through 11 years' time.

3.4. Stormwater Modelling

The EPA Stormwater Management Model (SWMM 5) is selected to simulate the hydrological and hydraulic response of the area with the purpose to evaluate the urban drainage system of the town based on single event simulation for stormwater quantity. Despite its minor limitations, SWMM has several advantages to use it for urban drainage evaluation. The model is freely available & stable programme. Moreover it has the ability to simulate low impact development (LID) in urban areas. Due to all these reasons, SWMM5 is found preferable than others to use for this study. On the other hand, absence of direct GIS interface and problem to use the model for large area are considered as a limitation of the model.

3.4.1. Model Description

The EPA Storm Water Management Model, SWMM first developed in 1969-71, was one of the first models. It has been continually maintained and updated and is perhaps the best known and most widely used of the available urban runoff quantity/quality models (A. Rossman & C. Huber, 2016). SWMM converts excess precipitation (rainfall, infiltration, evaporation, and initial abstraction) into surface runoff (overland flow). Because SWMM is a distributed model it allows a study area to be subdivided into any number of irregularly shaped sub catchment areas to best capture the effect that spatial variability in topography, drainage pathways, land cover, and soil characteristics have on runoff generation. Generation of runoff is therefore computed on a sub catchment by sub catchment basis (EPA, 2016). In SWMM, sub catchments are represented mathematically as spatially lumped, nonlinear reservoirs, and their outflows are routed through the channel/pipe. Sub catchments are subdivided into three subareas, impervious area with and without depression storage and pervious areas with depression storage. Flow from one subarea is not routed over another subarea. Overland flow is generated from each of the three subareas by approximating them as nonlinear reservoirs (C. Huber & E. Dickinson, 1992).

The runoff component of SWMM operates on a collection of sub catchment areas that receive precipitation and generate runoff and pollutant loads. The routing portion of SWMM transports this runoff through a system of pipes, channels, storage/treatment devices, pumps, and regulators. SWMM tracks the quantity and quality of runoff generated within each sub catchment, and the flow rate, flow depth, and quality of water in each pipe and channel during a simulation period comprised of multiple time steps (J. Larry et al., 2009).

3.4.2. Model Parameterization

3.4.2.1. Runoff Quantity

SWMM requires three major information for runoff quantity modelling. (1) Physical catchment characteristics, (2) rainfall, and (3) infiltration. The physical catchment data are total catchment

area (A), percentage of impervious area (%Imp), catchment width (W), average slope (So), surface depression storage and surface roughness (M.F.Chow et al ., 2012).

3.4.2.1.1. Building the Model

Based on the characteristic of the study area, the catchment is divided in to 45 sub catchments with 74 junctions, 94 conduits and 2 outfalls.

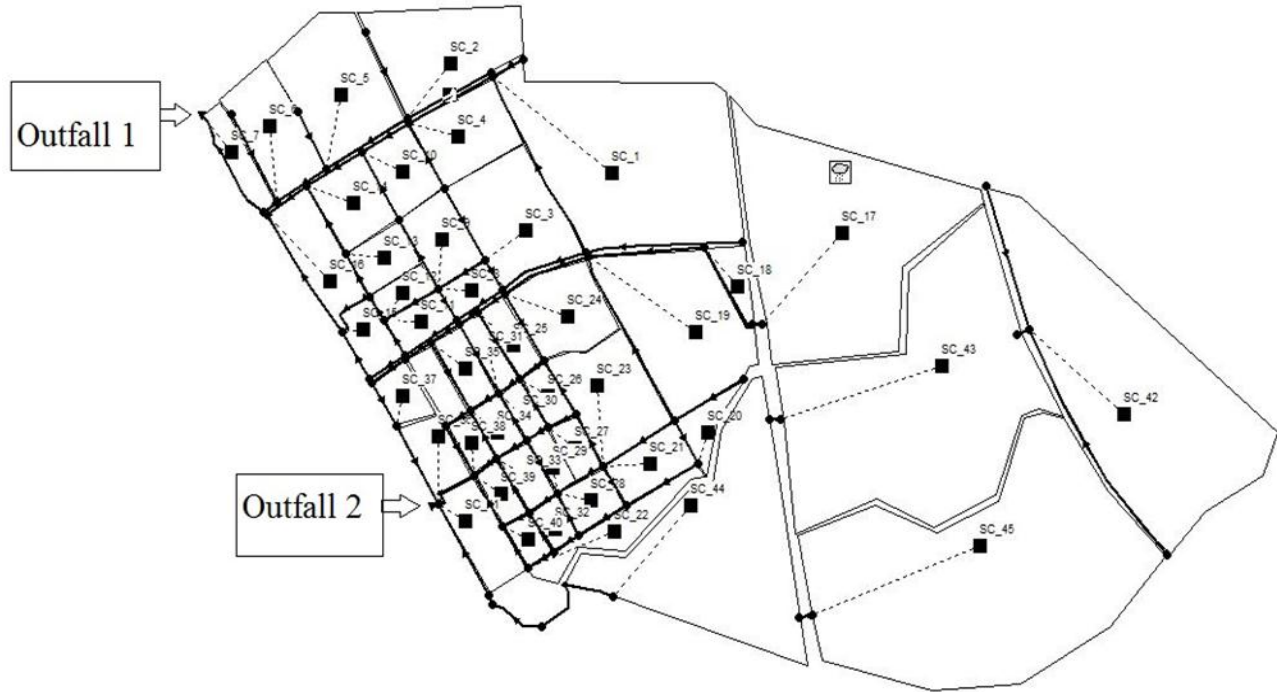


Figure 19. SWMM model for the area

Most of the information regarding physical catchment characteristics was obtained from the existing land use map that was derived from Google earth through digitization. Frequent field visit was made to ratify the surface and subsurface drainage configurations so as to accurately discretize the sub catchment areas. The proportion of imperviousness area was determined by dividing its area to the total sub catchment area. Whereas, sub catchment slope was computed as the elevation difference divided by the flow path length. Due to the fact that SWMM assumes a sub catchments as rectangular surface with uniform slop, width was computed by diving sub catchment area to flow path length. Surface roughness value is obtained from Manning's roughness coefficient for overland flow table (Annex 11) based on surface characteristic of the sub catchment and surface depression storage (d) is computed with the following formula.

$$d = K/\sqrt{s}$$

k= coefficient depending on surface type (0.07 for impervious surfaces & 0.28 for pervious surfaces) (mm)

s= ground slop

Table 10 .Physical properties of Sub catchments

Sub Catch.	Area(ha)	Width (m)	Imperviousness (%)	N - Imp	N - Perv	Av.slope (%)	Dstore- Imperv	Dstore- Perv
SC_1	26.40	437.2	34	0.013	0.095	3	0.35	1.4
SC_2	9.33	307.5	31	0.013	0.095	1	0.7	2.8
SC_3	8.51	371.5	59	0.013	0.095	1	0.7	2.8
SC_4	5.35	222.3	87	0.013	0.095	1	0.7	2.8
SC_5	9.84	330.6	37	0.013	0.095	1	0.7	2.8
SC_6	5.83	222.8	49	0.013	0.095	1	0.7	2.8
SC_7	2.21	102.6	64	0.013	0.095	3	0.4	1.6
SC_8	1.54	119	70	0.013	0.095	1	0.70	2.8
SC_9	3.54	181	61	0.013	0.095	2	0.50	2.0
SC_10	3.22	172.1	61	0.013	0.095	1	0.7	2.8
SC_11	1.86	131.7	88	0.013	0.095	1	0.7	2.8
SC_12	1.42	114.6	75	0.013	0.095	2	0.5	2
SC_13	2.38	160.8	70	0.013	0.095	2	0.5	2
SC_14	3.87	199.4	66	0.013	0.095	1	0.7	2.8
SC_15	2.28	167.9	49	0.013	0.095	2	0.5	2
SC_16	4.68	183.6	33	0.013	0.095	2	0.5	2
SC_17	29.63	217.9	11	0.013	0.095	5	0.3	1.7
SC_18	1.48	89.7	1.60	0.013	0.095	8	0.25	1
SC_19	13.59	348.4	54	0.013	0.095	2	0.5	2
SC_20	1.67	94.1	94	0.013	0.095	4	0.35	1.4
SC_21	3.2	166.3	49	0.013	0.095	1	0.7	2.8
SC_22	3.38	82.3	75	0.013	0.095	1	0.7	2.8
SC_23	7.27	282.6	52	0.013	0.095	1	0.7	2.8
SC_24	5.53	232.7	71	0.013	0.095	1	0.7	2.8
SC_25	1.65	101.3	100	0.013	0.095	2	0.5	2
SC_26	1.29	97.1	100	0.013	0.095	2	0.5	2
SC_27	1.41	100.1	100	0.013	0.095	2	0.5	2
SC_28	1.87	126.1	25	0.013	0.095	1	0.7	2.8
SC_29	1.09	86.8	100	0.013	0.095	0.1	0.5	2
SC_30	1.03	87.8	100	0.013	0.095	2	0.5	2
SC_31	1.43	90.5	69	0.013	0.095	1	0.7	2.8
SC_32	1.03	89	100	0.013	0.095	1	0.7	2.8
SC_33	1.49	110.5	100	0.013	0.095	1	0.7	2.8
SC_34	1.32	101.4	100	0.013	0.095	1	0.7	2.8
SC_35	1.89	111.5	83	0.013	0.095	1	0.7	2.8
SC_36	4.03	130.6	60	0.013	0.095	1	0.7	2.8
SC_37	1.72	128.1	69	0.013	0.095	1	0.7	2.8
SC_38	1.06	86.1	55	0.013	0.095	1	0.7	2.8
SC_39	1.21	93.2	72	0.013	0.095	3	0.4	1.6
SC_40	0.95	79.4	89	0.013	0.095	3	0.4	1.6
SC_41	3.64	166.9	62	0.013	0.095	2	0.5	2
SC_42	34.02	217.9	4	0.013	0.095	5	0.3	0.021
SC_43	39.64	152.5	0.6	0.013	0.095	3	0.4	1.6
SC_44	23.06	217.9	10	0.013	0.095	3	0.4	1.6
SC_45	47.12	196.1	0	0.013	0.095	4	0.35	1.4

IDF curve formulated for the town was used as rainfall input for the model where by 10 years return period is employed.

Meanwhile, waste water that is expected to be generated from the area is incorporated in the model as 51liter/c /day (85% of 60lit/c/day) based on the growth and transformation plan 2 (GTP 2) of water supply service level standard. In GTP 2, towns like Mojo having a population of 50,000-100,000 labeled as 3rd category and deserved to have water supply provision of 60 liter /c/day.

In order to determine the proportion of wastewater generated from the town, urban drainage book (David Butler & John W. Davies, 2011) was used as bench mark as depicted in tab.10. The proportion of Middle East, poor housing, category was selected for the analysis due to the fact that the society has a resemblance at a certain level in terms of social, economic and environmental aspect than the other categories.

Table 11. Percentage of Water discharged as waste water

Country	%
UK	95
Middle East	
✓ Poor housing	85
✓ Good housing	75
USA	60

3.4.2.1.2. Soil Infiltration

In the Model, there are three possible alternatives for infiltration: SCS, Horton model & Green Ampted equation .SCS curve number is one of the reliable technique to get soil infiltration rate of a given area based on the hydrological soil group category of the area. Detail Land use and soil infiltration attribute of the area are required to fix the curve number which is the most important parameter to determine the infiltration rate. Due to the absence such type detail information of the model area, it was not possible to implement SCS curve number. Horton model is applicable in a situation where rainfall intensity exceeds the infiltration capacity. However, what is prevailing on the ground is different from Horton’s assumption. Green Ampted equation is a physically based model, which take in to account mechanical property of a soil so that it is believed to recognize the infiltration process in better manner.

The basic concept of the Green-Ampt method is that water infiltrates through soil of some porosity along a “wetted front.” These are the average capillary suction head, S_u (mm), at the wetting front, the initial moisture deficit (IMD) in mm/mm and the saturated hydraulic conductivity, K_s , of the soil (mm/h) (Z.Asghar &B. Garg , 2018). Therefore, Green-Ampt Equation was designated to model the infiltration process among the three methods.

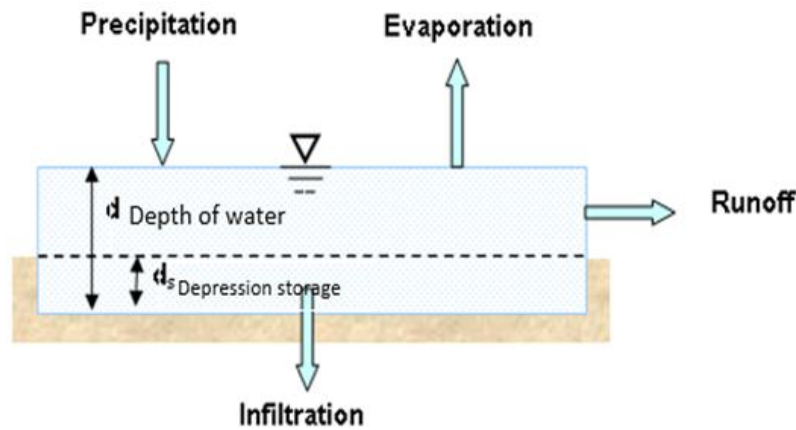


Figure 20. Non-linear Reservoir model of a sub catchment (EPA, 2015)

3.4.2.1.3. Flow routing

There are three routing options in the model based on flow routing algorithm of SWMM: Steady, Kinematic and dynamic routing. Steady flow routing is the simplest type that assumes flow as uniform & steady. Steady flow routing is not favored because it does not comply with the actual situation of the model area flow characteristic.

Kinematic routing method solves the continuity equation along with a simplified form of the momentum equation in each conduit. The latter requires that the slope of the water surface equal the slope of the conduit. The maximum flow that can be conveyed through a conduit is the full normal flow value. Any flow in excess of this entering the inlet node is either lost from the system or can pond atop the inlet node and be re-introduced into the conduit as capacity becomes available (W. James et al., 2010). On the other hand, Kinematic routing method is eligible under uniform unsteady flow that might be difficult to categorize the flow characteristic of the town drainage system under the aforementioned flow category so that it is not preferred.

Dynamic Wave routing solves the complete one-dimensional Saint-Venant flow equations and therefore produces the most theoretically accurate results. These equations consist of the continuity and momentum equations for conduits and a volume continuity equation at nodes. Dynamic wave routing can account for channel storage, backwater, entrance/exit losses and flow reversal (W. James et al., 2010). This method is selected for the model due to the fact that it is eligible for non-uniform unsteady flow moreover it allows to simulate flow parameters within shorter period of time.

Routing step of 30 seconds as well as Hazen Williams equation for pressure drop due to friction were employed for model simulation.

3.4.2.3. Model Calibration and Validation

Model calibration and validation are necessary and critical steps in any model application. For most models, calibration is an iterative procedure of parameter evaluation and refinement, as a result of comparing simulated and observed values of interest. Model validation is in reality an extension of the calibration process. Its purpose is to assure that the calibrated model properly assesses all the variables and conditions which can affect model results, and demonstrate the ability to predict field observations for periods separate from the calibration effort (A.S. Donigian et al., 2012). Sensitivity analysis is carried out to test the level of significance of the parameters associated with runoff quantity (Akdogan Z.& Guven B, 2016) so that these parameters supposed to be used for calibration. According to study conducted by (Li et al., 2014), the parameters used for sensitivity analysis and their allowable range of change was proposed as indicated in table 11. Table 12. Sensitivity analysis key parameters and their allowable range(Li et al., 2014).

Parameters	Description	Allowed range of change
N-Imperv	Manning's roughness coefficient for impervious area	0.005-0.05
N-Perv	Manning's roughness coefficient for pervious area	0.05-0.5
D store-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

3.4.2.3.1. Data Collection for Model Calibration and Validation

Flow data is required in order to calibrate and validate the model. However, flow data was not available due to the fact that flow gages were not installed to measure the quantity of flow generated from the drainage system of the town. Three conduits were selected for this purpose. Those were D_48 (Outfall 2), which has an open rectangular channel with 260.0 ha catchment area coverage that is 73.4 % of the total model area, D_89 and D_90 which are box culverts linked to Addis Ababa-Djibouti new railway line.

Flow measurements, using depth and velocity, were taken for ten days in order to calibrate sensitive parameters and validate the model. Depth of flow is measured with meter at one hour difference right after the start of the rain until the flow ends from the channel. At the same moment, floating method using leaf is used within specified distance of channel in order to get the time taken to cover the distance which ultimately helps to compute the flow velocity.

The measurement of velocity is expected to have some error due to the reasons mentioned below.

- ✓ The velocity of the leaf cannot be exactly the same with storm water.

- ✓ The measurement was taken at the top of the flow which expected to be much faster than at the middle and lower profile of the flow.

Hence, the discharge was computed with continuity equation. Moreover, ten days rainfall data which are parallel to the data collection date were collected from Ethiopian Metrological Agency. Out of the ten flow measurement data, only four day’s flow data which are labelled as extreme rain fall events were used for the purpose: three days(July 15, April 1 & 07/2020) flow data for calibration and the remaining one day(April 21/2020) for validation.

3.4.2.4. Model Performance Evaluation

The reliability of the model was tested using Coefficient of Determination (R²) and Nash–Sutcliffe coefficient (NSC).

$$\text{Coefficient of Determination (R}^2\text{)} = \frac{\sum_{t=1}^n (q_t^{obs} - q_t^{Av.obs})(q_t^{sim} - q_t^{Av.sim})}{\sqrt{\sum_{t=1}^n (q_t^{obs} - q_t^{Av.obs})^2} \sqrt{\sum_{t=1}^n (q_t^{sim} - q_t^{Av.sim})^2}} \quad \dots \text{Eq.25}$$

The range of R² lies between 0 and 1 which describes how much of the observed dispersion is explained by the prediction. A value of zero means no correlation at all whereas a value of 1 means that the dispersion of the prediction is equal to that of the observation(J.Y. Wu et al., 2013) .

$$\text{Nash–Sutcliffe coefficient (NSC)} = 1 - \frac{\sum_{t=1}^n (q_t^{obs} - q_t^{sim})^2}{\sum_{t=1}^n (q_t^{obs} - q_t^{Av.obs})^2} \dots \text{Eq. 26}$$

The value of NSC ranges from 1 to - ∞.When NSC value is 1, the model performance is perfect. If NSE value is negative, it implies that the performance of the model is poor.

Notations : q_t^{obs} is observed flow, $q_t^{Av.obs}$ is an average observed flow , q_t^{sim} is simulated flow, $q_t^{Av.sim}$ is an average simulated flow ,t is time and n is the total number of time steps.

Level of performance of the model is characterized according to tab. 12

Table 13.Performance evaluation criteria for both methods (Moriassi et al., 2015)

Method	Output Response	Temporal Scale	Performance Evaluation Criteria			
			Very Good	Good	Satisfactory	Not Satisfactory
R ²	Flow	Day/Month /Year	R ² > 0.85	0.75 < R ² ≤ 0.85	0.60 < R ² ≤ 0.75	R ² ≤ 0.60
NSC	Flow	Day/Month /Year	NSC > 0.85	0.75 < NSC ≤ 0.85	0.60 < NSC ≤ 0.75	R ² ≤ 0.60

3.4.2.5. Low Impact Development

A low impact development (LID) is an alternative land development approach for storm water that has been recommended instead of the traditional stormwater design. The main purpose of LID is to reduce the impact of development on water related problems through the use of sound stormwater management practices (Muhammad Shafique & Reeho KIM, 2015). LID may benefit all types of development, it is best suited for new, suburban residential development. Moreover, the LID practices and technologies are best integrated into a developer's existing land development process and practices (NAHR Rech. Cent., 2016) .Due to the fact that the town is highly being affected by flooding problems, two types of LID practises were modelled that could be a solution to alleviate the problems: Bio-retention cell and Infiltration trench.



Figure 21. Proposed location of Low Impact Development structures

Bio retention Structure

Bio retention areas or rain gardens are depressed areas in the landscape that are designed to accept stormwater. They can be used in residential and commercial settings and are typically planted with shrubs, perennials, or trees, and covered with shredded hardwood bark mulch. The benefits of bio retention areas include decreased surface runoff, increased groundwater recharge, and pollutant treatment through a variety of processes (Prince George Country, 1999).

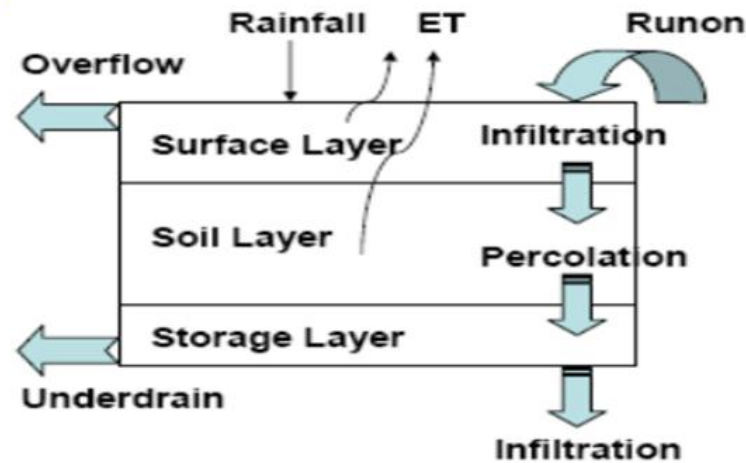


Figure 22. Conceptual Model of a bio retention cell in SWMM (L.A. Rossman, 2015)
Fifteen bio retention cells, having an area of 2000 m² for each, have been modelled based on design elements as indicated in table 13.

Table 14. Parameters of bio retention cells for the Model

Layer	Parameters	Dimension
Surface	Berm Height (mm)	300
	Vegetative Volume Fraction	0.20
	Surface Roughness	0.13
	Surface Slope (%)	0.04
Soil	Thickness (mm)	700
	Porosity (volume fraction)	0.475
	Field capacity (volume fraction)	0.378
	Wilting Point (volume fraction)	0.265
	Conductivity (mm/hour)	0.254
	Conductivity slope	45
	Suction head (mm)	12.6
Storage	Thickness (height) (mm)	350
	Void ratio (voids/solids)	0.60
	Seepage rate (mm/hour)	0.254
	Clogging Factor	474
Drain	Flow coefficient	5
	Flow exponent	0.5
	Offset height (mm)	500

Infiltration trench

An infiltration trench is an excavated trench that has been back-filled with stone to form a subsurface basin. Stormwater runoff is diverted into the trench and is stored until it can be infiltrated in to the soil (Fuss & O’Neill, 2013). One infiltration trench with an area of 4000 m² have been modelled with the following design elements.

Table 15. Parameters of infiltration trench for the Model

Layer	Parameters	Dimension
Surface	Berm Height (mm)	500
	Vegetative Volume Fraction	0.20
	Surface Roughness	0.13
	Surface Slope (%)	0.04
Storage	Thickness (height) (mm)	700
	Void ratio (voids/solids)	0.6
	Seepage rate (mm/hour)	0.254
	Clogging Factor	474
Drain	Flow coefficient	34
	Flow exponent	0.5
	Offset height (mm)	0.5

Chapter Four: Results & Discussions

4.1. Formulation of IDF curve for Mojo station

IDF curves were formulated using both Gumbel & Logarithmic Pearson Type-III distribution methods.

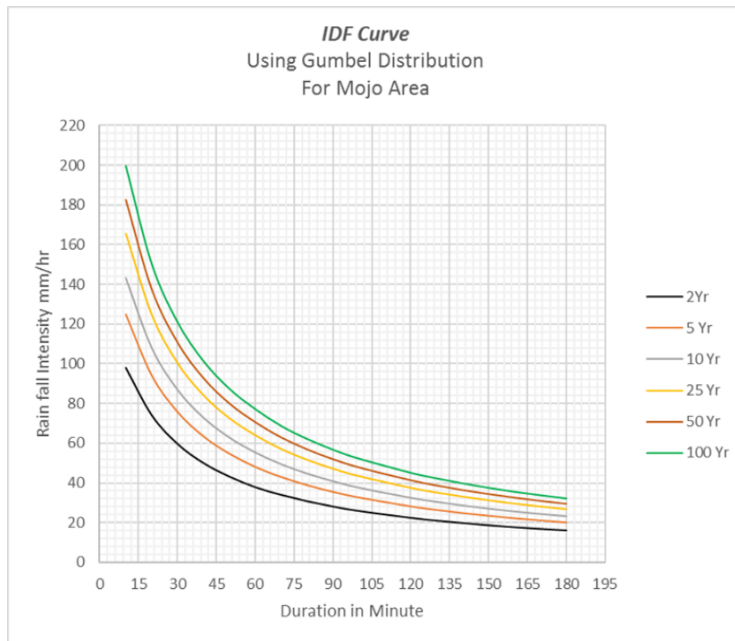


Figure 23. IDF curve by Gumbel method

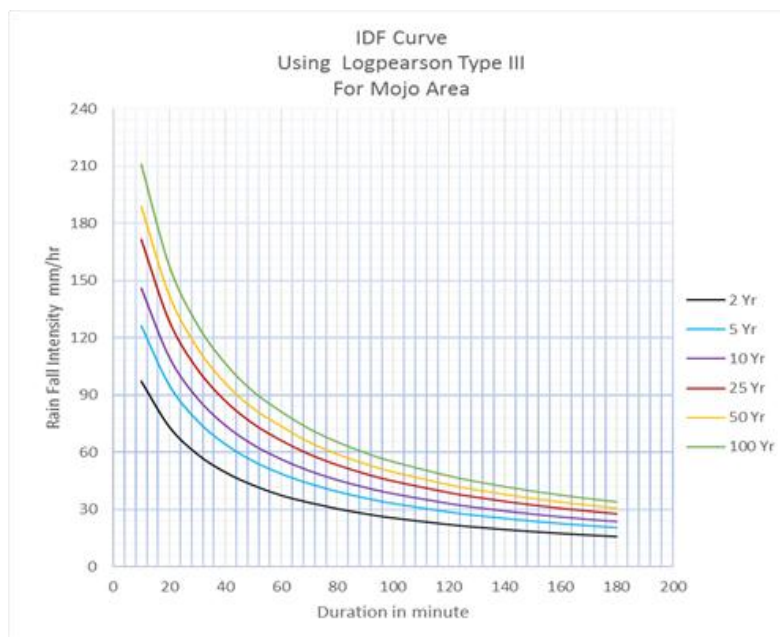


Figure 24 . IDF curve by Log Pearson type III

4.1.1. Goodness of Fit test

Chi square method was employed to test the fitness of the yearly maximum rain fall data (1997-2017) records with respect to both Gumbel's and Log Pearson type III probability distribution methods. Details of Maximum rain fall data is attached as annex-24.

Gumbel Distribution

Table 16. Chi square test table (1)

Roll No	Interval	Probability (P) in %	Mean	Stand. Deviation	Return Period	Reduced Variation Y_t	Frequency Factor K	Rainfall Depth in (mm)
1	38-52.9	20	62.405	18.77	5	1.499	1.009	81.34
2	53-67.9	40	62.405	18.77	2.5	0.672	0.181	65.79
3	68-82.9	60	62.405	18.77	1.67	0.091	-4.400	54.89
4	83-97.9	80	62.405	18.77	1.25	-0.476	-0.967	44.26

Table 17. Chi square test table (2)

Interval of Class		Observed (O)	Expected(E)	$(E-O)^2/E$
0	44.26	2	4.2	1.152
44.26	54.89	7	4.2	1.867
54.89	65.8	5	4.2	0.152
65.8	81.34	1	4.2	2.438
>81.34		6	4.2	0.771
Total (Chi square calculated value)				6.381

According to 99 % confidence limit ($\alpha: 0.01$), the chi square critical value is 7.01 provided that degree of freedom is 18($21-2-1=18$). Since the calculated chi square value is less than the critical value, the data could be fit with Gumbel distribution at 99% confidence limit .

Log Pearson type III distribution

Table 17. Chi square test table (1)

Roll. No.	Probability (P) in %	Mean	Stand. Deviation	Skew. Coefficient	Frequency Factor K	Rainfall Depth	
						Log	in mm
1	20	1.777	0.128	0.255	0.8267	1.88282	76.3515
2	40	1.777	0.128	0.255	0.247	1.80862	64.36
3	60	1.777	0.128	0.255	-0.43	1.72196	52.718
4	80	1.777	0.128	0.255	-0.858	1.66718	46.4704

Table 18. Chi square test table (2)

Interval of Class		Observed (O)	Expected(E)	(E-O) ² /E
0	46.471	6	4.2	0.777
46.471	52.718	7	4.2	1.152
52.718	64.36	5	4.2	0.152
65.36	76.351	1	4.2	1.152
> 76.351		6	4.2	1.152
Total (Chi square calculated value)			4.00	

Based on 99 % confidence limit ($\alpha: 0.01$), the chi square critical value is 7.01 provided that degree of freedom is 18(21-2-1=18). Since the calculated chi square value is less than the critical value, the data could be fit with Log Pearson type III distribution at 99% confidence limit.

Table 19. Comparison of Gumbel & Log Pearson type III distribution Methods.

Distribution Methods	Chi-square test results (at 99% Confidence limit)		
	χ^2 Calculated	χ^2 Critical	Decision
Gumbel	6.381	7.01	Accepted
Log Pearson type III	4.0	7.01	Accepted

Due to the fact that Log Pearson Type III has a smaller chi square calculated value, it is selected for further analysis.

Comparison Between Self-made and ERA IDF

Based on ERA (2013) design drainage manual, Mojo town is categorized in A2 rainfall region. Both rainfall region categorization map and IDF curves designated for A2 rainfall region are attached as annex 12 & 13 respectively. The IDF curve formulated for Mojo town is examined with respect to the IDF curve of ERA (2013) as depicted in table 19.

Table 20. Comparison of IDF curves

Ret. Period	Duration in min											
	10		30		60		90		120		180	
	Self-made	ERA	Self-made	ERA	Self-made	ERA	Self-made	ERA	Self-made	ERA	Self-made	ERA
2	97.2	75	59.1	48	37.4	30	27.7	23	22.1	19	15.8	12
5	126.3	96	76.6	59	48.8	36	36.0	28	28.7	23	20.1	17
10	146.0	110	88.4	70	56.4	44	41.6	34	33.1	28	23.7	19
25	171.5	125	103.7	76	66.2	50	48.8	46	38.8	30	27.8	22
50	188.8	140	114.8	85	73.7	55	54.0	50	43.0	33	30.7	24
100	211.0	152	127.1	94	81.3	60	59.8	54	47.6	36	34.0	25

The difference between ERA (2013) and self-made IDF curves is not significant for longer rain fall duration above 60 minutes particularly for short length return periods like 2,5 and 10 years.

Whereas the difference in rain fall intensity for short rain fall duration particularly for long length return periods (25, 50, 100 years) is significant. From economic point of view, ERA IDF curve is preferable because of its lower intensity of rain fall which does not requires higher investment cost for construction of water infrastructures compared to the self-made one. However, Self-made IDF curve seems appropriate form design safety point of view.

For this research purpose, self-made IDF curves was chosen for further analysis because it is made for that particular area .Although self-made IDF curve has a negative impact in terms of economic cost , it is overlooked due to the fact that economic analysis is not the major concern of this study.

The possible causes for the difference between the IDF curves.

- ✓ The A2 rainfall region covers very large area in Ethiopia which includes two large regional states (Amhara & Oromia) and Addis Ababa city administration. Therefore, the IDF curve is prepared in such a way to represent the rainfall variability all over these areas. Whereas the self-made one only represents Mojo area which is very small compared to the A2 rainfall region coverage.
- ✓ The rainfall data that was used as an input for IDF formulation for A2 rainfall region is until 2010. Whereas for self-made, IDF the rainfall data until 2017 was used where by recent rain fall events were taken in to account.

4.2. Land Cover/Land Use of the Model Area

As an industrial town, Mojo is expected to have massive land use change. In order to examine the land use change, three successive Google earth images were used as indicated below.



2009 (a)



2014 (b)



2019(c)

Figure 25. Google Earth Image 2009(a), 2014(b), 2019(c)

After digitizing three successive (2009, 2014 and 2019) Google Earth images, the following results were obtained.

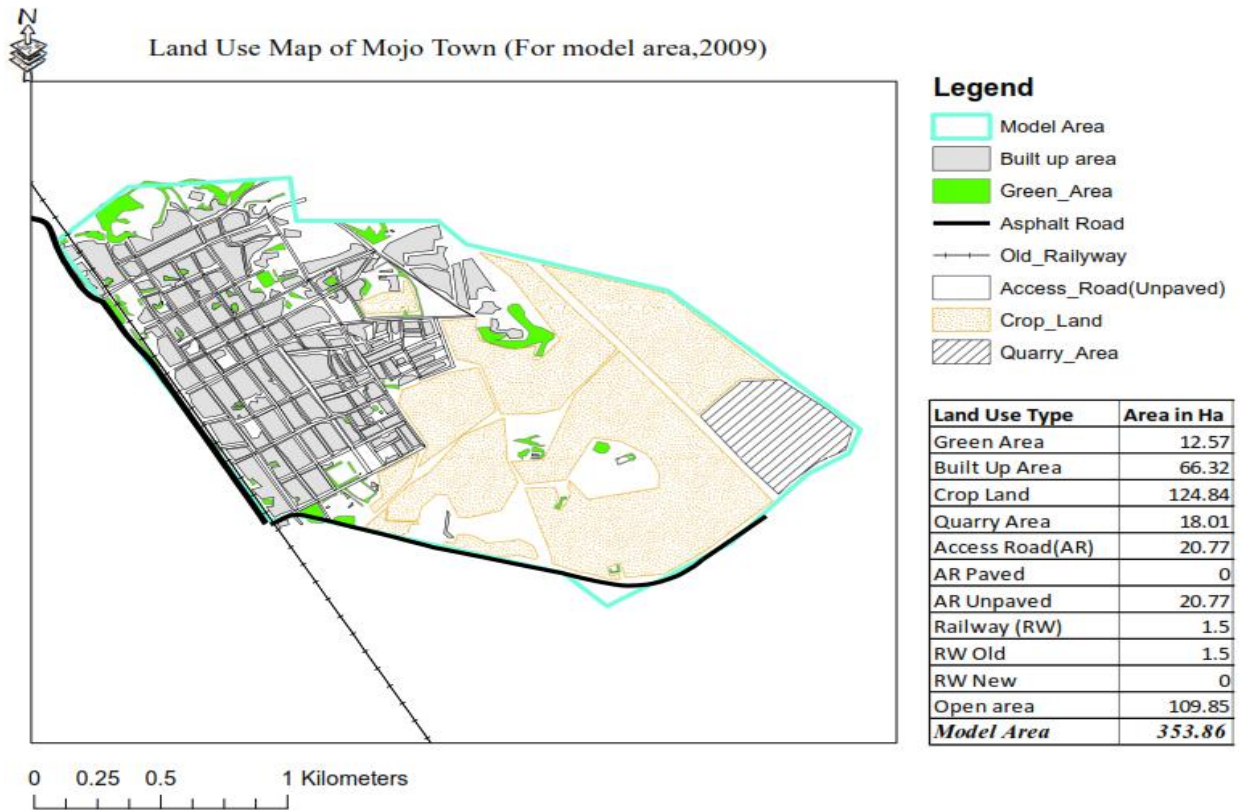


Figure 26.Land Use of the Model Area (2009)

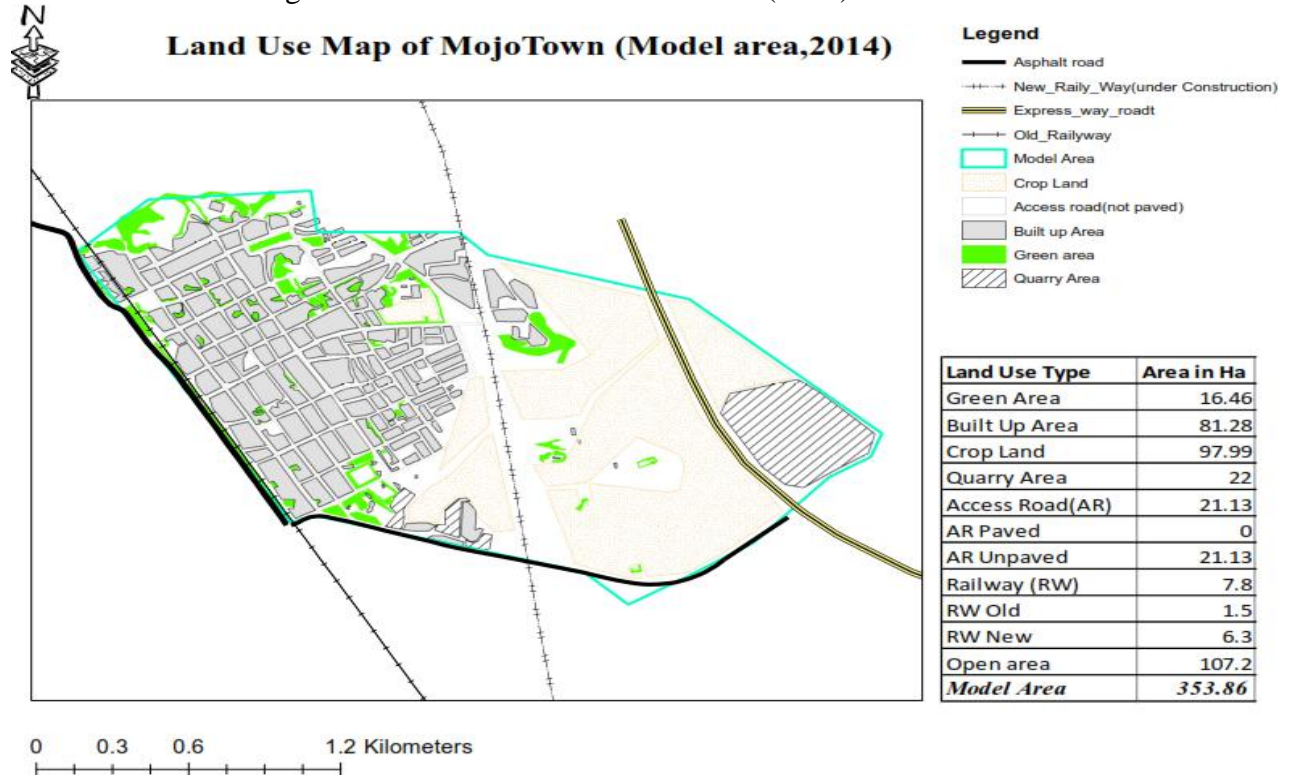


Figure 27.Land Use of the Model Area (2014)

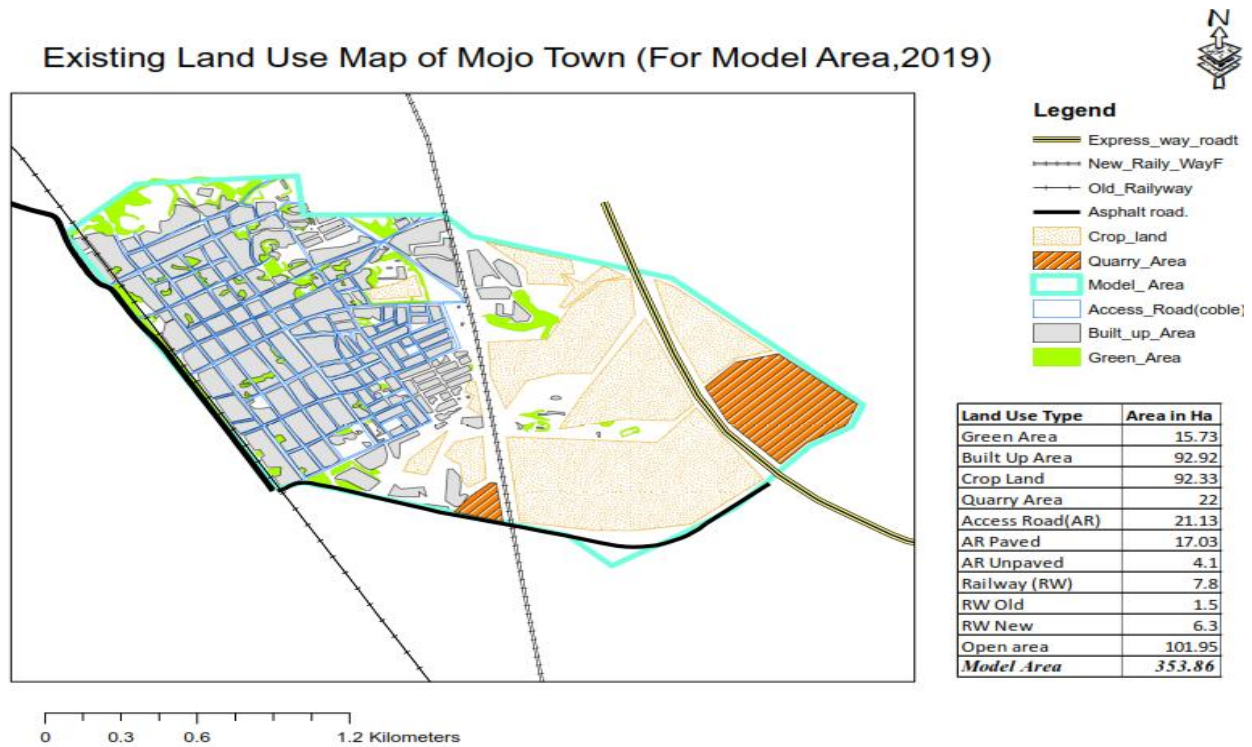


Figure 28.Land Use of the Model Area (2019)

Eleven years land use land cover change observed in the model area is summarized in table 20.

Table 21.Land use change in the model area from 2009-2019

No.	Land Use Type	2009 (Ha)	2014 in Ha	Difference in %	2019 in Ha	Difference in %	Total change (%)
1	Built up area	66.32	81.32	+22.5	92.92	+14.3	+40.1
2	Green area and Parks	12.57	16.46	+30.90	15.73	-4.4	+25.1
3	Crop Land	124.84	97.99	-21.5	92.33	-5.8	-26.0
4	Express Way	0	6.0		6.0	0	-
5	Quarry Area	18.01	22.0	22.2	22	0	+22.2
6	Railways						
6.1	Old	1.5	1.5	0	1.5	0	0
6.2	New	0	UC*		6.3	-	-
	Sub total	1.5	1.5		7.8	+420	420
7	Access Road						
7.1	Paved				17.01	-	-
7.2	Unpaved	20.77	21.33	+1.7	4.32	-	-79.2

UC*: Under Construction

4.2.1. Built Up Area

Residential, business, commercial, manufacturing areas and other social service areas like schools, churches, mosques etc. are included in the built up area category. The rate of increase as indicated in the table with respect to build up area with in the last eleven years is so big that fosters to have

more impervious areas. Therefore, built up area increase observed in the town could be considered as one of the main causes for the flooding incidents that has been observed in the town.

4.2.2. Green Area and Parks

There are a number of places where the existing land use is crop-land but proposed to be green according to the structural plan of the town. Generally, the emphasis given for green area development is poor compared to other economic development activities. If green area development had been done in a much better extent in the town, the flooding problem would have been reduced to a significant manner.

4.2.3. Crop land

There is a large crop land area in the model near by the two mega projects (Express & Railway Projects). That area is environmentally fragile on the top of that it is exposed to high flow discharge emanated from different drainage infrastructures of the two mega project drainage systems. Due to this reason, different forms of land degradation like soil erosion, gully formation is observed in many places.



*Figure 29.*Gully formation

The above photograph shows storm water discharges to the town from a bridge located at eastern part of the town which is a segment of Addis Ababa–Adama express way road causing gully (Av. Width 3 m * Av. depth 2 m) formation. This is one of the indications by which the mega projects drainage systems were not designed taking the actual conditions of the towns in to account located along the road.

4.3. Result of parameter Sensitivity Analysis

Local sensitivity test is used and the impact on runoff was examined with respect to six different parameters: Imperviousness, depression storage both for pervious & impervious surface, catchment width, and Manning coefficient both for impervious and pervious areas. The value of each selected parameter were examined by 10% increase /decrease within a range -20% to +20%. Sensitivity is determined, after computing total runoff in terms of each parameters.

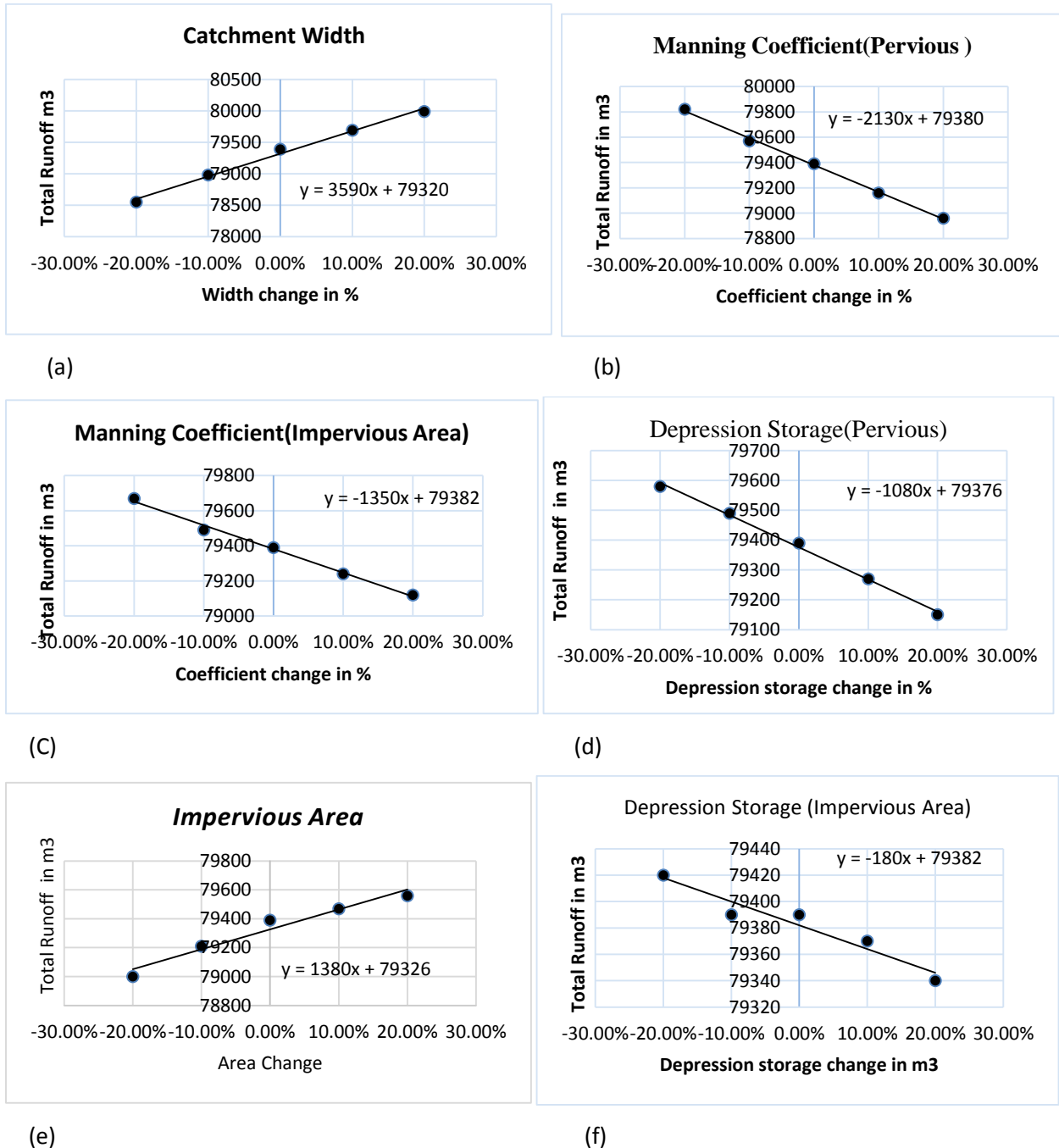


Figure 30 .Sensitivity analysis for the model (a-f)

The parameters that generate more total runoff for the same percentage of increase or decrease considered to be more sensitive. The increment of percentage refers to parameters like catchment width and impervious area on the other hand the decrease refers to the other four parameters. The parameter ranking is describe in table 21.

Table 22. Parameter Sensitivity Ranking

Parameters	Ranking
Catchment width	1 st
Manning’s roughness coefficient for pervious areas	2 nd
Manning’s roughness coefficient for impervious areas	3 rd
Depression storage (pervious)	4 th
Imperviousness	5 th
Depression storage (impervious)	6 th

4.4. Model Calibration and Validation

4.4.1. Hydrological & Hydraulic Parameters

For calibration and validation purpose, ten days flow measurements within one hour interval were collected at three conduits (D_48, D_89 & D_90) and only those data related to four extreme rainfall were used: July 15, August 01, 07& 21/2020.

Table 23: Rain fall data

Month/2020	July 15		August 01		August 07		August 21	
	Daily	hourly	Daily	hourly	Daily	hourly	Daily	hourly
Rainfall in mm	4.5	2.5	13.7	2.0	34	7.4	38.8	2.86

Source: Daily data from NMA & hourly from CHRS

Table 24 .Details of the three conduits

List of conduits	Shape	Type of channel	Height in m.	Width in m	Top width in m	Bottom width in m	Remark
D_48	Trapezoidal	Masonry	1.1		1.30	0.85	
D_89	Rectangular	Box Culvert	2.0	1.50			K90+628.64
D_90	Rectangular	Box Culvert	3.50	3.0			K91+179.64

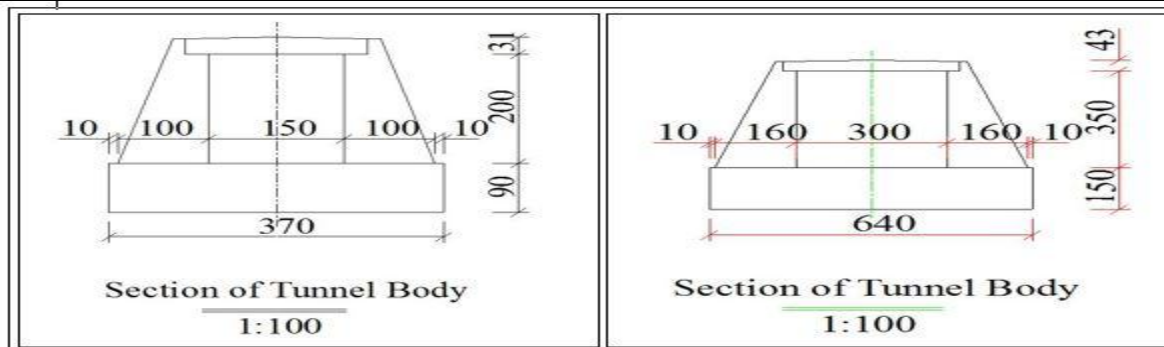


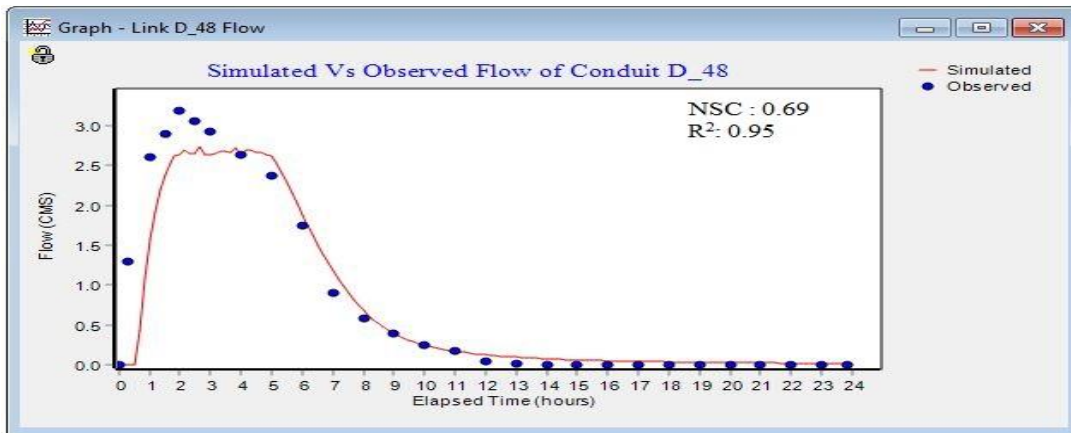
Figure 31. Cross-sectional view of D_89 & D_90 respectively (Source: Ethiopia Railways Corp.)

The detail flow data of the three conduits is attached as Annex 17-19. Based on the sensitivity analysis results for the model, the two most sensitive parameters (Catchment width & Manning roughness coefficient) were adjusted until the simulated and observed values reached at acceptable level. The final values of model parameters used for calibration are presented in table 24.

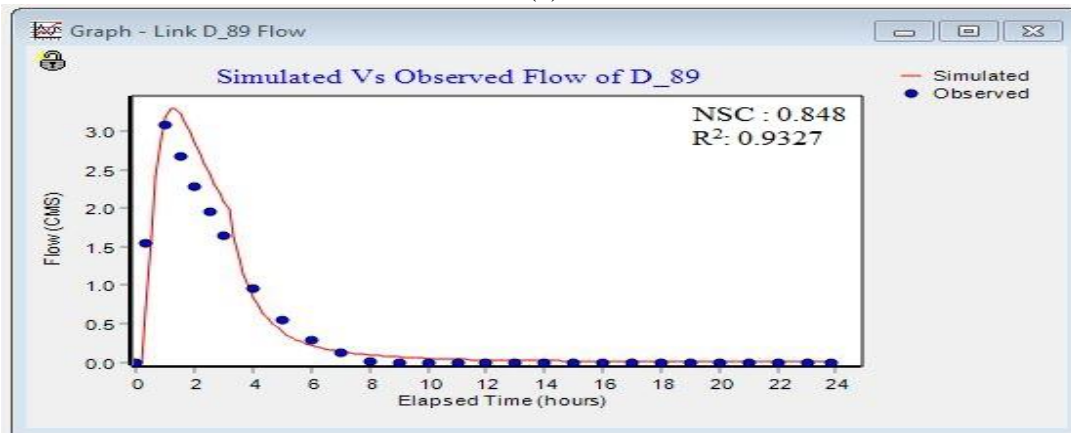
Table 25. Range of values for calibrated model parameters

Parameters	Range of Recommended value	Range of Used values	Remark
Width	$0.2 < K > 5$	0.24-1.27	Width= $K \cdot \sqrt{A}$, Where K: flow coefficient & A: catchment area.
Manning roughness coefficient for pervious surface (n)	0.05-0.5	0.05-0.095	

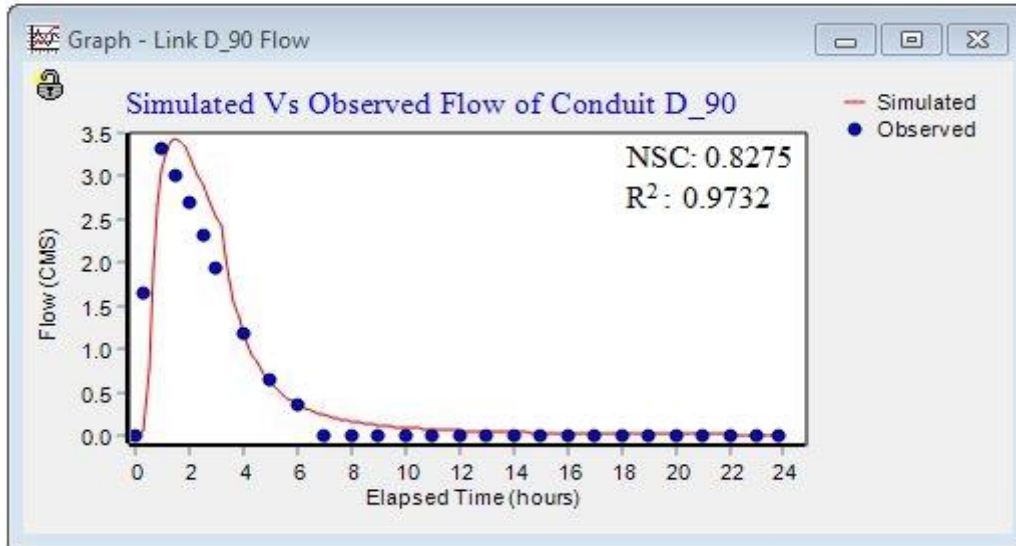
Three steps of model calibration were made based on the observed flow data collected at July 15, August 01, 07 at each conduit. Curves obtained at the end of the calibration process, which illustrate the relationship between the simulated and observed flow values for three conduits (D_48, D_89 & D_90), are plotted as indicated as in Fig 32(a-c).



(a)

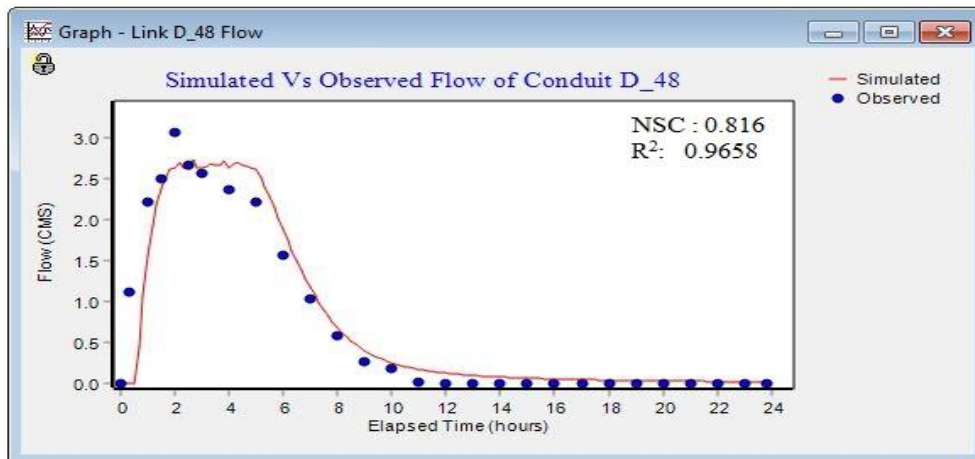


(b)

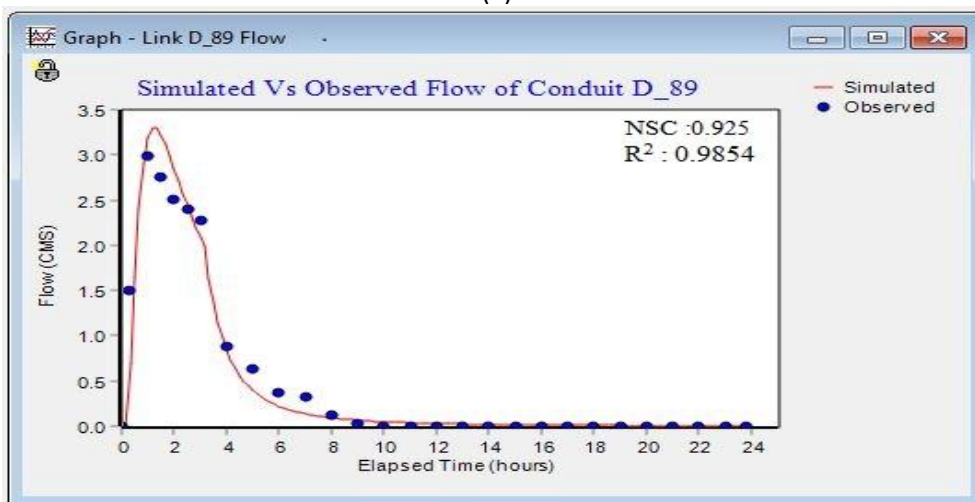


(c)

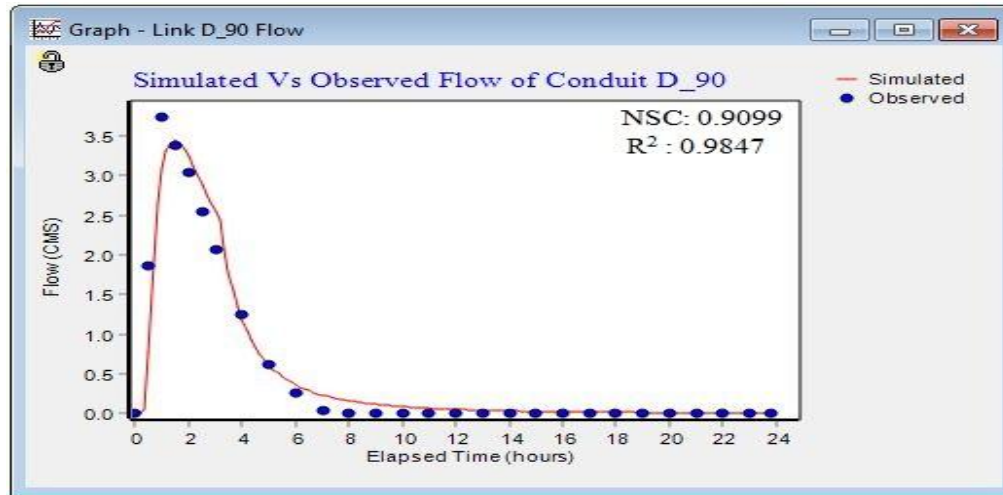
Figure 32. Calibration results of three conduits based on three rain fall events (a-c).



(a)



(b)



(c)

Figure 33. Validation results of three conduits based on one rain fall event (a-c).

The results obtained from calibration of model parameters are within the range of acceptable limit. With regard to calibration, the performance of the model using Nash–Sutcliffe model efficiency (NSC) for each conduits (D_48, D_89, and D_90) is 0.69, 0.848 and 0.8275 respectively. It is 0.95, 0.9327, and 0.9732 respectively using coefficient of determination (R2). Referring to the Validation, the performance of the model using Nash–Sutcliffe model efficiency (NSC) for the aforementioned each conduits is 0.816, 0.925, 0.9099 and it is 0.9658, 0.9854, 0.9847 using coefficient of determination (R2) respectively. The calibration and validation results indicated that the model simulation outputs represent the storm water runoff being generated in the study area.

4.5. Drainage System Capacity Analysis

As a major objective of the study, the existing urban drainage system capacity analysis of the town and identification of overflow locations are assessed. For simulation, ten years return period is used as per urban storm water drainage design manual of Ministry of works & urban development (2008), 3 hours rain fall duration is also considered. SWMM evaluates the quality of the simulation using continuity error (surface runoff, flow routing & stormwater quality). The value of continuity error is expected to be less than 10 % in order to be valid. As per Water Management Model User’s Manual Version 5.1 (2015), simulation of a model is low quality if the continuity error is > 10%. The quality of the simulation results is valid due to the fact that the continuity error for surface runoff and flow routing are -0.26 and -0.20 % respectively. As it was discussed in chapter 3, model parameters were derived from sub catchments, junctions, conduits, rain gages and the model area was divided into 45 sub-catchments, 74 junctions, 94 conduits 2 outfalls and one rainfall station. According to the output of model simulation, 25 conduits and 28 junctions were flooded which is 27.0 % and 37.8% respectively out of the total drainage system coverage of the town. This

indicates that the drainage system capacity is not adequate to accommodate the storm water which is being generated by the town so that the drainage system needs upgrading in order to avoid flood induced threats. Details of those conduits and junctions are presented in table 25.

Table 26. List of overflowing conduits and Junctions

Conduits	Junctions
D_2,D_3,D_4,D_10,D_11,D_12,D_13, ,D_21,D_22,D_25,D_28,D_29,D_30, D_33,D_34,D_37,D_38,D_39,D_40, D_48,D_54,D_60,D_63,D_72, D_92.	J_3,J_9,J_10,J_11,J_12,J_13, J_18,J_20,J_21,J_22,J_28,J_29,J_30, J_31,J_32,J_41,J_45,J_46,J_53,J_54,J_56 J_60,J_61,J_62,J_67,J_69,J_71,J_72

As shown in the map Fig. 34, the overflowing of conduits at the moment of 1:00:00 time simulation that means the situation of the drainage system after one hour of rain fall start is depicted in red colour and the other colours are considered to be safe from overflowing problem.

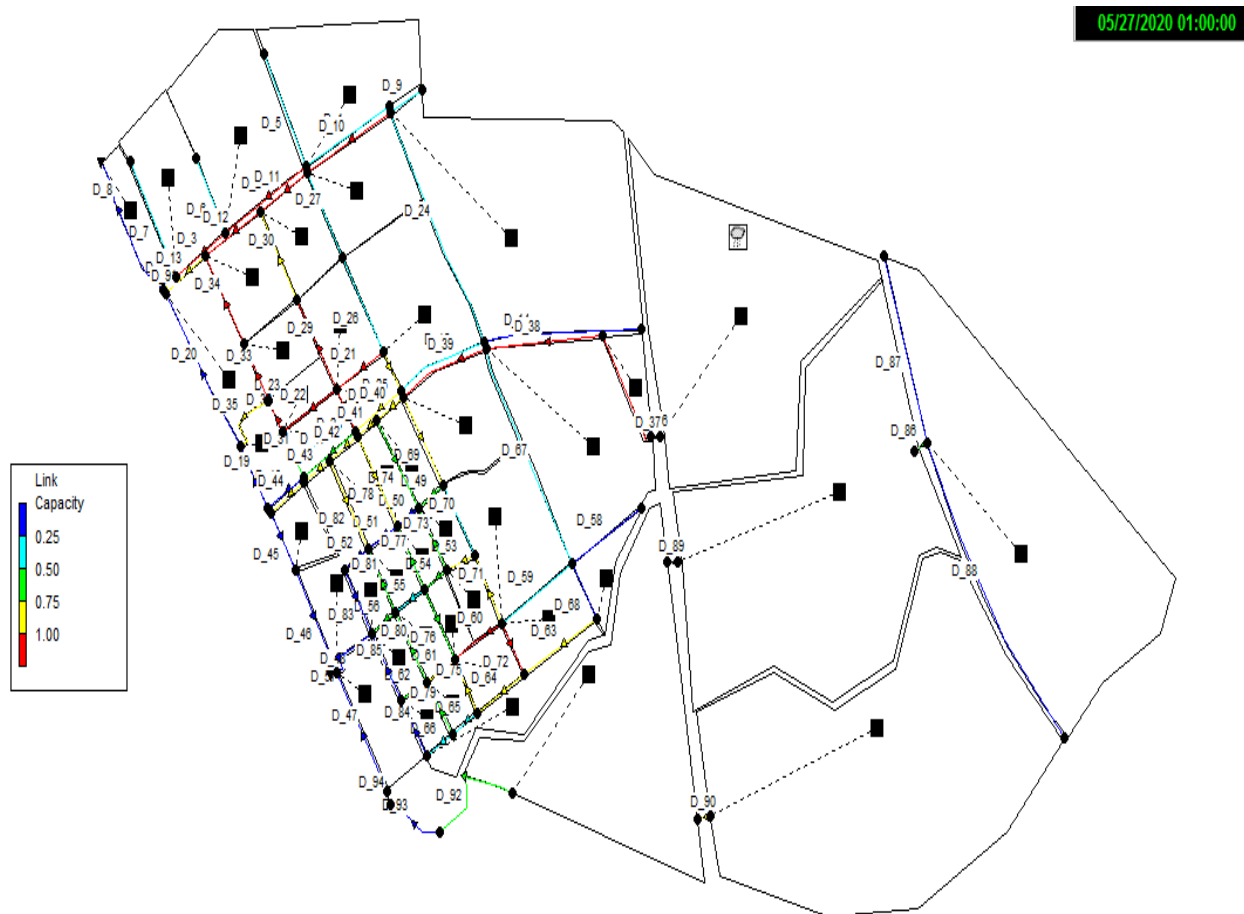


Figure 34. Maps of overflowing conduits

After simulation of the model, flooding of junction at the moment of 1:00:00 time is illustrated in fig 35. Moreover, list of flooded junctions are listed as annex 22.

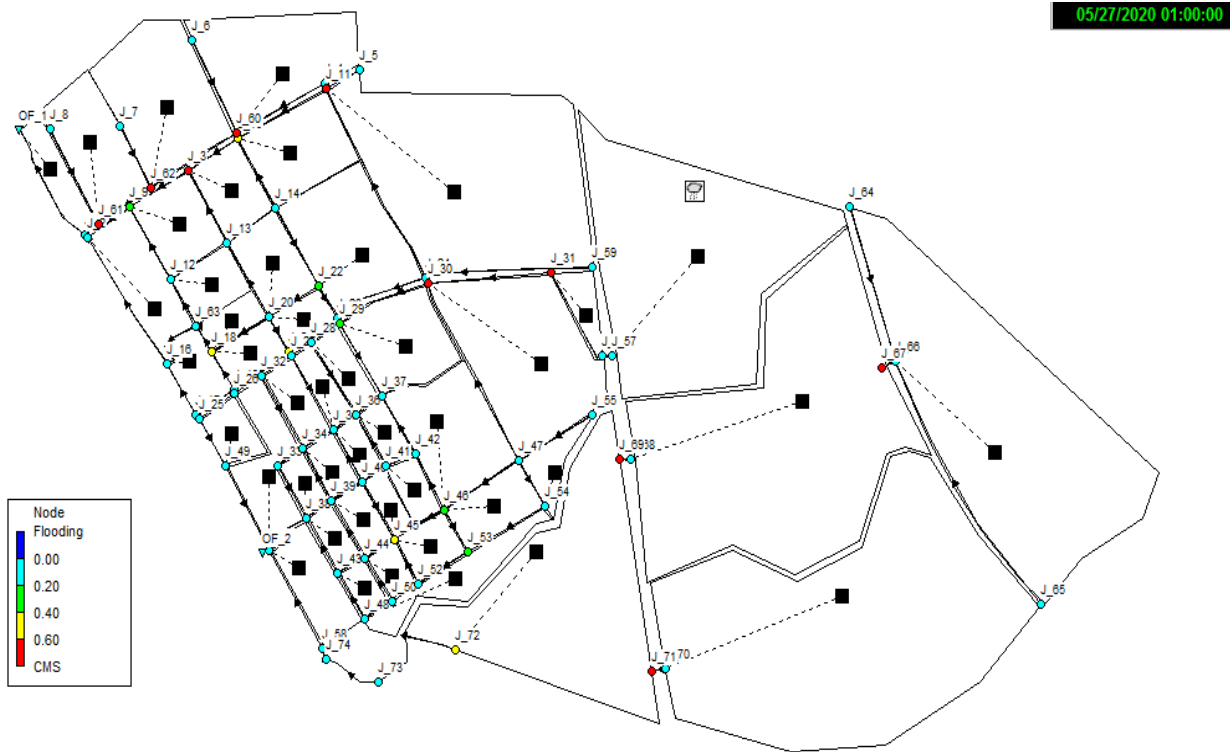


Figure 35. Maps of flooded Junctions

In fig. 36, both drainage capacity and flooded junction are shown at 1:00:00 simulation time.

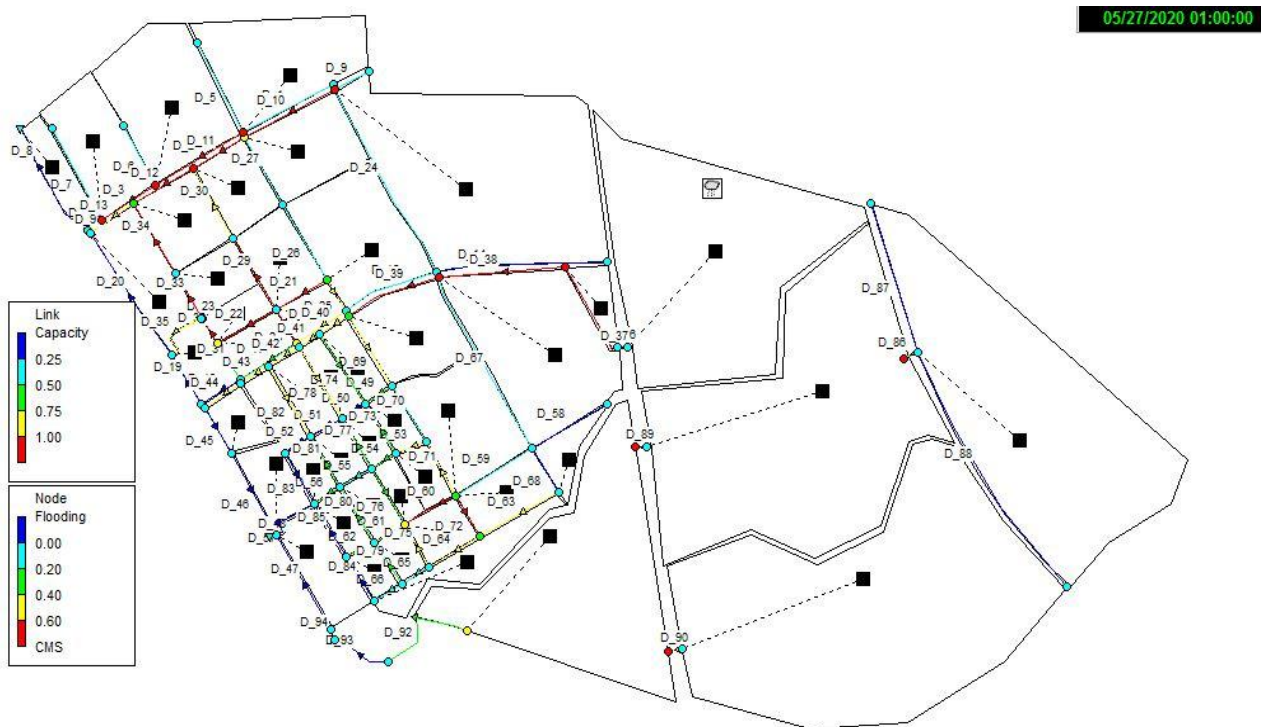


Figure 36. Map of drainage capacity and node flooding

From the map it can be deduced that overflowing drains and flooded junction are located in most of the same area.

The major source of the storm water in this study is the two mega projects located at the upland of the town which could be a cause for an adverse impact on the performance of the drainage infrastructures. This fact is inferred by the high rate peak discharge manifested due to LULC change on most of the conduits (D_20, D_36, D_37, D_50, D_51, D_52, D_56, D_57, D_81, D_83, D_89, and D_90) that have a connection with the two mega projects drainage infrastructures.

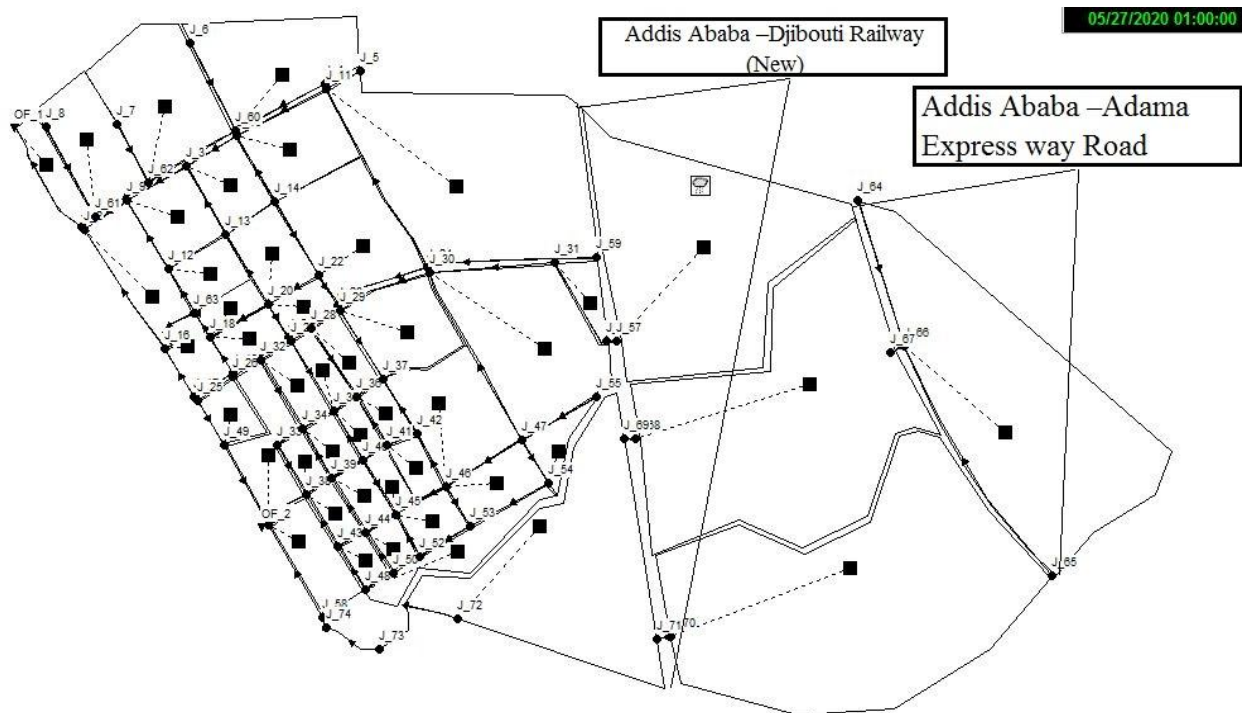


Figure 37. Map of Conduits influenced by mega projects

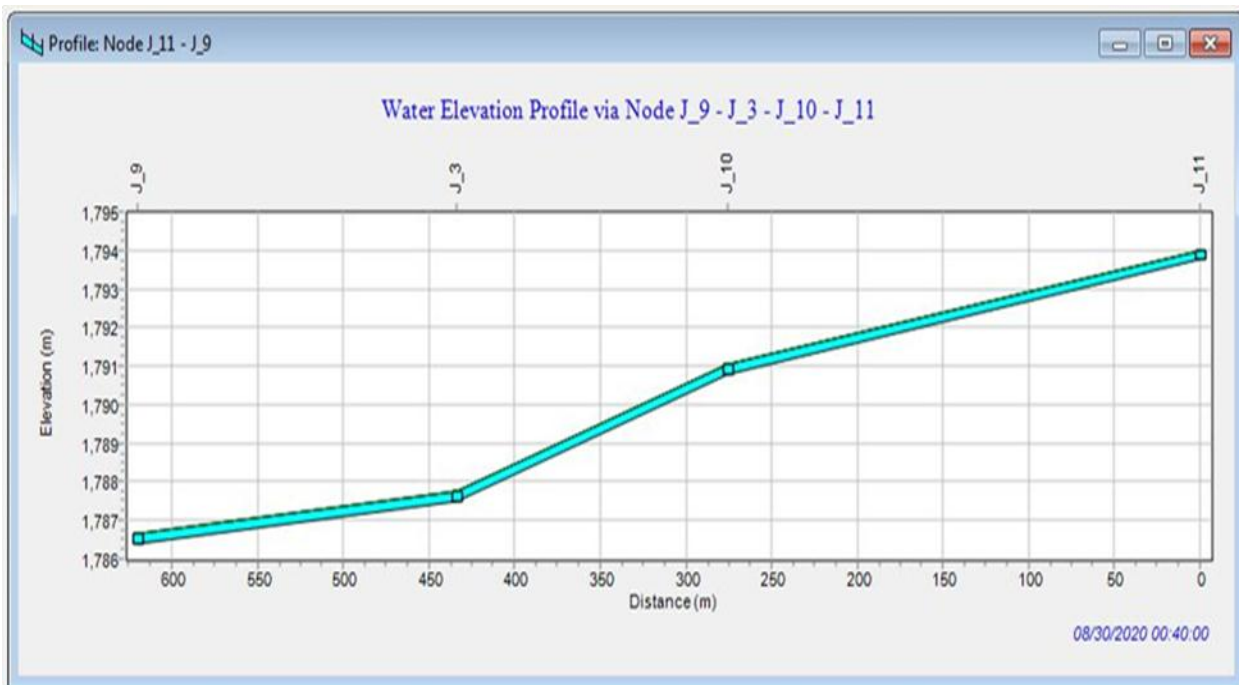
In some place of the study area, it is observed that protection measures are made by individuals so as to protect themselves from immediate flooding problem. One of the photographs taken at conduit D_66 (fig 38) is an evidence that shows the prevailing storm water management problems which need sustainable and long lasting solution.



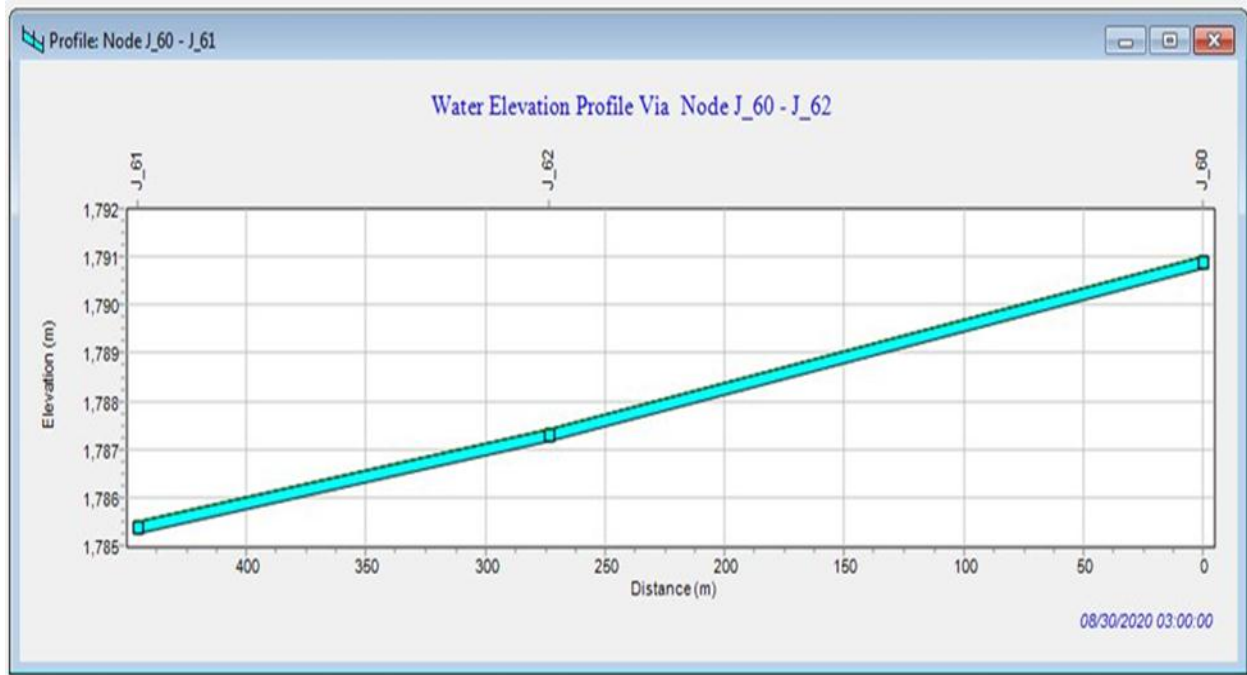
Figure 38. Conduit D_66

4.5.1. Profile plot

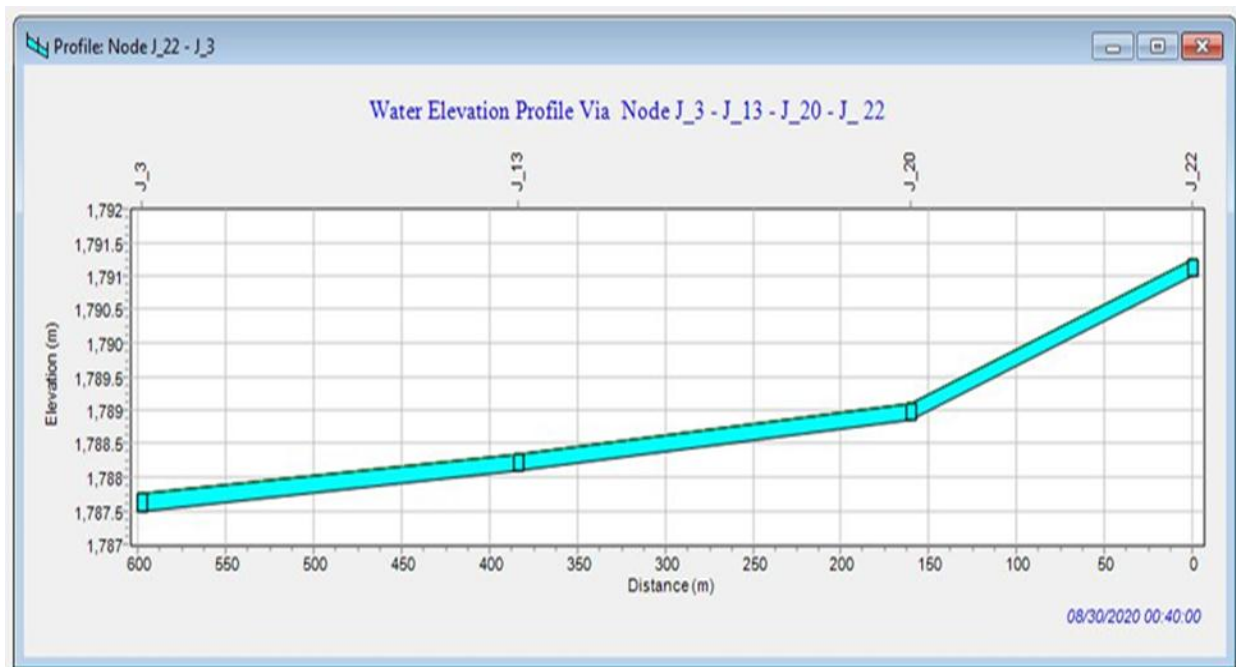
Another important graphical demonstration generated by SWMM is a profile plot which shows the water surface depth variation along the longitudinal section of the conduits with respect to time. The stormwater plot profile of three locations are displayed in fig 39(a-c): J_9, J_3, J_10, J_11 & J_60 - J_60 & J_3, J_13, J_20, and J_22.



(a)



(b)



(c)

Figure 39. Water surface profile at different Junctions (a-c).

As indicated in the profile plot, drains like conduit 2(J_60-J_62), conduit 10 (J_10-J_11.), conduit 11 (J_3-J_10), conduit 12 (J_3-J_9), conduit 21 (J_20-J_22), conduit 29 (J_13-J_20) and conduit 30 (J_3-J_13) were over flooded right after the start of 40 minutes of the rain fall. On the other

hand, drains like conduit 2 (J_60-J_62) was over flooded after the start of 3 hours of the rain fall. In the model, most of the overflowing of drains were observed during early minute and hour after the rain fall is started. This implies that most of the rain fall become surface runoff instead of infiltrating in to the sub surface soil.

The rain fall event on July 15/2020 has caused overflow of drains nearby the Ethio-Japan textile factory which has a direct connection to the model area. The rainfall intensity of the day as it was collected from National Metrology Agency was 4.5mm/day. Hourly intensity is 2.94mm/hr based on the ERA's formula for estimation of short duration rainfall. According to CHRS (Centre for hydro metrology & remote sensing) the intensity during the rainfall period is 2.5mm/hr. The hourly rainfall intensity is not a kind of high intensity rainfall that could be categorized even for the lowest 2 years return period for that particular area. The cause for the overflow of drains might be under estimated channel design together with blockage of drains with solid wastes.

Some photographs were taken at the moment of flooding .Hence, those photographs help to relate the profile plots results and to visualize the drainage system problem of the town. Some photographs are presented in fig 40.



Figure 40. Pictures of drains before & at the moment of flooding.

4.5.2. Flow Velocity

Flow velocity in the drain is one of the very important characteristics of storm water that has a contribution to determine the drainage system performance. If the velocity is below the minimum threshold, there exists siltation of solid material in the drainage system that would be a barrier for a better system performance. On the other hand if the velocity is beyond the maximum threshold, the stormwater causes damage on infrastructure. Therefore, there should be range of minimum and maximum flow velocity for a drainage system in order to maintain proper system performance.



Figure 41. Map of Flow velocity at the time of 4:00:00 hour.

A common design criterion is to specify a minimum velocity when the pipe runs full. This method has the advantage of simplicity in computation, but lacks precision. The other common approach is to specify self-cleansing velocities at some specified depth of flow [e.g. 0.75 m/s at $d/D = 0.75$](David Butler & John W.Davies, 2011). Flow velocity with an assumption of below 0.9 m/s is considered to be minimum threshold in this study. Based on the model simulation, several drains about 14.9 % of the system are categorized below the cleansing velocity. According to the study (C.S Adaba & J. C. Agunwamba, 2014) conducted in small urban water shade in Nigeria, 8 channels in the system have flow velocities less than 1 m/s. A flow velocity less than minimum requirement of 1m/s caused deposition of sediments in the drain. Field observation of these drains showed that the drains had been silted by sediments. A redesign of inadequate drainage channels was done and the result showed that, their velocities ranges from 1.21m/s to 1.54m/s.

In order to improve the minimum velocity in the system, redesigning in different forms is essential for those drains which are categorized below cleansing velocity.

Although it is desirable to transport flows in sewer systems rather swiftly, harmful high velocities leading to erosion of sewers must be avoided. In storm sewers, flows are intermittent and the maximum design velocities could be allowed to exceed the limit of 3 m/s (Unesco, 1991). Conduits (D₃₇ & D₉₂) which have a velocity of 3.27m/s and 3.09 m/s respectively at 00:40:00 simulation time are categorized under high velocity. Based on the output of model simulation, maximum flow velocity is not a critical issue that would negatively affect the drainage system performance.

On the other hand, there has been intensive soil erosion from the farm lands located adjacent to the two mega projects because of high flow runoff velocity. The storm water transports the soil through the drainage infrastructures in the meantime certain amount is being deposited in some places due to the minimum flow velocity. This problem negatively affects the drainage system performance which is being aggravated due to poor solid waste management and high sediment content of the storm water. This situation has been often observed in frequent field visits of the town.

4.5.3. Peak rate of Runoff

Drainage systems should be designed to convey based on a predetermined peak runoff magnitude so as to avoid significant flood hazards. Based on the simulation results of SWMM, peak rate of runoff & peak runoff time (Peak time) for different return periods is obtained for each sub catchment as indicated in table 26.

Peak run off time is the time that is needed to reach at peak runoff right after the start of the rain. As impervious area increases, the peak runoff time become shorter and shorter. On the other hand, peak run of time increases, with an increased area of green infrastructure. The concept is similar to time of concentration in ration formula which is used to compute peak discharge of a given catchment.

Table 27. Peak runoff of Sub catchments.

Sub Catch.	Return period, Yrs											
	2		5		10		25		50		100	
	Peak time	Q in cms	Peak time	Q in cms	Peak time	Q in cms	Peak time	Q in cms	Peak time	Q in cms	Peak time	Q in cms
SC_1	0:40:00	1.81	0:40:00	2.50	0:40:00	2.98	0:40:00	3.62	0:40:00	4.09	0:40:00	4.63
SC_2	0:50:00	0.63	0:40:00	0.87	0:40:00	1.04	0:40:00	1.27	0:40:00	1.44	0:40:00	1.64
SC_3	0:30:00	0.82	0:30:00	1.13	0:30:00	1.33	0:30:00	1.60	0:30:00	1.79	0:30:00	2.02
SC_4	0:30:00	0.60	0:30:00	0.81	0:30:00	0.95	0:30:00	1.13	0:30:00	1.26	0:30:00	1.40
SC_5	0:40:00	0.71	0:30:00	0.98	0:40:00	1.17	0:40:00	1.42	0:40:00	1.60	0:40:00	1.81
SC_6	0:30:00	0.50	0:30:00	0.68	0:30:00	0.81	0:30:00	0.98	0:30:00	1.11	0:30:00	1.25
SC_7	0:30:00	0.25	0:30:00	0.33	0:20:00	0.41	0:20:00	0.50	0:20:00	0.56	0:20:00	0.63
SC_8	0:30:00	0.18	0:20:00	0.24	0:20:00	0.29	0:20:00	0.35	0:20:00	0.39	0:20:00	0.45
SC_9	0:30:00	0.39	0:30:00	0.52	0:20:00	0.61	0:20:00	0.75	0:20:00	0.84	0:20:00	0.96
SC_10	0:30:00	0.33	0:30:00	0.45	0:30:00	0.53	0:30:00	0.63	0:30:00	0.71	0:20:00	0.79
SC_11	0:30:00	0.23	0:20:00	0.30	0:20:00	0.36	0:20:00	0.44	0:20:00	0.50	0:20:00	0.57
SC_12	0:20:00	0.19	0:20:00	0.26	0:20:00	0.30	0:20:00	0.37	0:20:00	0.41	0:20:00	0.46
SC_13	0:20:00	0.29	0:20:00	0.40	0:20:00	0.48	0:20:00	0.58	0:20:00	0.64	0:20:00	0.73
SC_14	0:30:00	0.41	0:30:00	0.55	0:30:00	0.65	0:30:00	0.78	0:30:00	0.85	0:20:00	0.98
SC_15	0:30:00	0.24	0:30:00	0.33	0:20:00	0.39	0:20:00	0.47	0:20:00	0.53	0:20:00	0.61
SC_16	0:40:00	0.38	0:40:00	0.52	0:40:00	0.62	0:30:00	0.75	0:30:00	0.85	0:30:00	0.96
SC_17	1:20:00	1.38	1:20:00	1.94	1:10:00	2.32	1:10:00	2.85	1:10:00	3.24	1:10:00	3.68
SC_18	0:40:00	0.14	0:30:00	0.19	0:30:00	0.23	0:30:00	0.28	0:30:00	0.31	0:30:00	0.35
SC_19	0:30:00	1.22	0:30:00	1.67	0:30:00	1.98	0:30:00	2.39	0:20:00	2.68	0:30:00	3.03
SC_20	0:20:00	0.23	0:20:00	0.31	0:20:00	0.37	0:20:00	0.44	0:20:00	0.49	0:20:00	0.56
SC_21	0:30:00	0.29	0:30:00	0.40	0:30:00	0.48	0:30:00	0.57	0:30:00	0.65	0:30:00	0.73
SC_22	0:40:00	0.32	0:30:00	0.44	0:30:00	0.52	0:30:00	0.63	0:30:00	0.71	0:30:00	0.80
SC_23	0:30:00	0.64	0:30:00	0.88	0:30:00	1.04	0:30:00	1.26	0:30:00	1.42	0:30:00	1.60
SC_24	0:30:00	0.58	0:30:00	0.79	0:30:00	0.93	0:30:00	1.11	0:30:00	1.24	0:30:00	1.39
SC_25	0:20:00	0.20	0:20:00	0.28	0:20:00	0.34	0:20:00	0.41	0:20:00	0.46	0:20:00	0.52
SC_26	0:20:00	0.17	0:20:00	0.23	0:20:00	0.28	0:20:00	0.33	0:20:00	0.37	0:20:00	0.42
SC_27	0:20:00	0.18	0:20:00	0.25	0:20:00	0.29	0:20:00	0.36	0:20:00	0.40	0:20:00	0.45
SC_28	0:40:00	0.15	0:30:00	0.19	0:40:00	0.25	0:30:00	0.29	0:30:00	0.33	0:30:00	0.38
SC_29	1:40:00	0.03	1:30:00	0.05	1:20:00	0.06	1:20:00	0.08	1:20:00	0.09	1:20:00	0.11
SC_30	0:20:00	0.14	0:20:00	0.19	0:20:00	0.23	0:20:00	0.27	0:20:00	0.30	0:20:00	0.34
SC_31	0:30:00	0.16	0:30:00	0.21	0:20:00	0.25	0:20:00	0.30	0:20:00	0.34	0:20:00	0.39
SC_32	0:30:00	0.13	0:20:00	0.17	0:20:00	0.21	0:20:00	0.25	0:20:00	0.28	0:20:00	0.32
SC_33	0:30:00	0.18	0:20:00	0.24	0:20:00	0.29	0:20:00	0.35	0:20:00	0.39	0:20:00	0.45
SC_34	0:30:00	0.16	0:20:00	0.21	0:20:00	0.26	0:20:00	0.31	0:20:00	0.35	0:20:00	0.40
SC_35	0:30:00	0.22	0:30:00	0.30	0:20:00	0.34	0:20:00	0.42	0:20:00	0.48	0:20:00	0.55
SC_36	0:40:00	0.36	0:30:00	0.50	0:30:00	0.60	0:30:00	0.72	0:30:00	0.81	0:30:00	0.92
SC_37	0:30:00	0.20	0:20:00	0.26	0:20:00	0.31	0:20:00	0.38	0:20:00	0.43	0:20:00	0.49
SC_38	0:30:00	0.11	0:30:00	0.15	0:30:00	0.18	0:20:00	0.21	0:30:00	0.24	0:20:00	0.28
SC_39	0:20:00	0.16	0:20:00	0.22	0:20:00	0.26	0:20:00	0.32	0:20:00	0.35	0:20:00	0.40
SC_40	0:20:00	0.14	0:20:00	0.19	0:20:00	0.22	0:20:00	0.26	0:30:00	0.29	0:20:00	0.33
SC_41	0:30:00	0.40	0:30:00	0.53	0:30:00	0.62	0:20:00	0.75	0:30:00	0.84	0:20:00	0.97
SC_42	1:00:00	2.09	1:00:00	2.90	1:10:00	3.45	0:50:00	4.18	0:50:00	4.74	0:50:00	5.39
SC_43	1:30:00	1.97	1:20:00	2.76	1:10:00	3.30	1:10:00	4.05	1:10:00	4.6	1:10:00	5.22
SC_44	1:20:00	1.06	1:20:00	1.49	1:20:00	1.79	1:10:00	2.19	1:10:00	2.47	1:10:00	2.84
SC_45	1:30:00	2.04	1:30:00	2.86	1:30:00	3.44	1:20:00	4.21	1:20:00	4.80	1:20:00	5.47

Based on the figures tabulated in the table 26, most of the peak runoffs are observed in early minutes and early hours after the start of the rain. This situation might be induced due to high rate of urbanization resulting very low infiltration because of large area of surface imperviousness and poor permeability characteristic of clay soil of the model area. Hence, as the area of impervious increases the drainage system is not capable to entertain the increased stormwater. Moreover, this type of circumstance is a serious threat for the urban infrastructures, human life and property due to flood induced problems.

4.6. Impact of LULC change on surface runoff

Within eleven years period (2009-2019), 38.92 ha of land was used for built up in the model area including the two mega projects which is an increment of 58.7%. This implies that there has been significant urban development in the area. Based on the output of the model, peak surface runoff, total runoff of each sub-catchments and peak discharge of each conduit were organized in tabular form in order to evaluate the change due to the land use/land cover change. Details of the output of the model is attached as annex 20 & 21. According to the result of the model simulation for the year 2009 & 2019 independently, a total of 1884 m³ of surplus surface runoff is generated from the model area with in the specified 11 years period.

The output of the model indicated that the minimum and maximum increase in terms of peak runoff is 0.86% and 79.31 %. Regarding total runoff, the minimum and maximum increase is 0.05 % and 44.44%. Moreover, the minimum and maximum increase in terms of peak discharge is 0.04% and 59.2 % respectively. The pattern of the simulation output data clearly indicated that as the LULC change increases ,the rate of increase of all the aforementioned three parameters are supposed to be high.

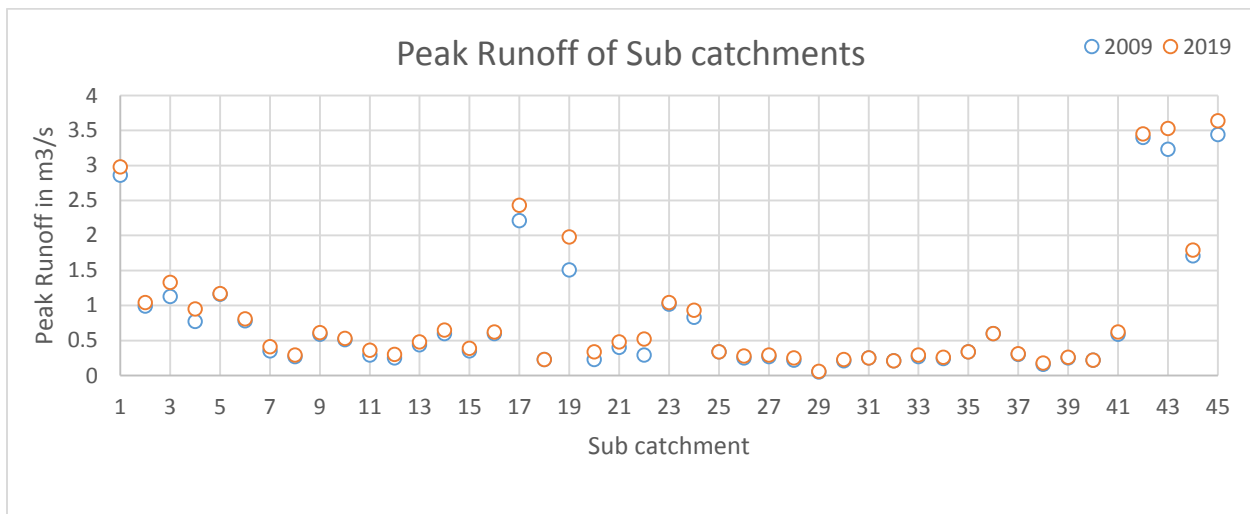


Fig –Sub Catchment Peak Runoff at 2009 & 2019.

The pattern of the simulation output data clearly indicated that as the LULC change increases, all the aforementioned three parameters increase as well as. Complete overlapping of points in the graph implies that there was not LULC change within the given period of time.

4.7. Low Impact Development Simulation

Low impact development is considered as one of the effective measures to have proper urban storm water management in order to improve stormwater quantity and quality. Basically, LID is a storm water management approach that tries to simulate the natural hydrological condition of a given area. Despite the fact that the existing situation allows to apply LID measures, most of the towns in Ethiopia are not familiar with it. SWMM provides an opportunity to evaluate the impacts of different types of LID measures for a drainage system.

The overall objective of implementing the structural LID measures in the model area is to capture a big amount of stormwater runoff which is being generated from the two mega projects and enhance some storm water to infiltrate in to the ground and the remaining particularly in case of high flows to divert to the nearby river which is one of the tributaries of Mojo River. Moreover, the structures also help to filter pollutants that are coming from the farm lands due to the use fertilizers and hazardous chemical released from those vehicles using the express way road.

As most of the sources of excessive runoff is coming from upland of the town where the mega projects are located, the LID measures are proposed to be constructed in the farm land immediately below the projects in order to capture and treat the stormwater runoff before it reaches residential and commercial areas.

In the model, two type of LID (Bio retention & Infiltration trench) measures were applied in different sub catchments. The aforementioned measures are selected due to its peculiar characteristic to obstruct large amount of stormwater runoff and to filter pollutants as well. Parameters of both structures are specified based on different physical characteristic of the sub catchment. SWMM manual provides tables, formulas e.t.c to compute values for each parameters. Berm height & area of structures were computed based on the expected amount of stormwater to be accumulated and diverted. After the simulation of the model, the impact of LID application induced on the drainage system is examined.

Table 28. Details of LID measures

List of Sub Catch.	Bio retention				Infiltration trench			
	Pcs	Length (m)	Width (m)	Total Area (m ²)	Pcs	Length (m)	Width (m)	Total Area (m ²)
SC_17	2	200	10	4000	1	300	2.67	801.0
SC_18	1	200	10	2000	-			
SC_43	5	200	10	10000	1	700	2.67	1869.0
SC_44	3	200	10	6000	-			
SC_45	4	200	10	8000	1	498	2.67	1330.0
Total				30,000				4000.0

4.7.1. Comparison of the exiting flow & LID Scenario

In order to evaluate the change before and after the application of LID measures, model simulation outputs of peak runoff from sub catchments and peak discharge from conduits along with the peak discharge time were compared in areas where all LID measures are assumed to be applied. 10 yrs & 25 yrs return period were taken in to consideration. For 10 yrs return period, continuity Error for surface runoff and flow routing results were 1.80% & -0.23 respectively. Whereas for 25 yrs return period, continuity Error for surface runoff and flow routing results were 1.96% & -0.20 respectively.

Table 29. Comparison of Peak runoff.

Sub Catch.	Existing Peak Runoff				LID Scenario				Peak Runoff reduction (%)	
	10 yrs return period		25 yrs return period		10 yrs return period		25 yrs return period		10 yr R. period	25 yr R. period
	Peak time	Q (m ³ /s)	Peak Time	Q (m ³ /s)	Peak time	Q (m ³ /s)	Peak time	Q (m ³ /s)		
SC_17	1:10:00	2.32	1:10:00	2.85	1:30:00	1.94	1:20:00	2.62	16.4	8.7
SC_18	0:30:00	0.23	0:30:00	0.28	1:10:00	0.15	1:00:00	0.21	34.8	25
SC_43	1:10:00	3.30	1:10:00	4.05	1:50:00	2.63	1:40:00	3.23	20.3	20.2
SC_44	1:20:00	1.79	1:10:00	2.19	1:40:00	1.58	1:30:00	1.54	11.7	29.7
SC_45	1:30:00	3.44	1:20:00	4.21	1:50:00	3.03	1:50:00	3.46	11.9	17.8

Based on the analysis associated to 10 yrs return period, LID application helps to decrease the magnitude of peak run off with the minimum of 11.9% and maximum of 34.8 %. Moreover, peak runoff time is increased by a minimum of 22.2 % and a maximum of 133.3 %. This situation enables significantly to improve the drainage system performance of the town if it is done in extensive and proper manner. Regarding 25 yrs return period, magnitude of peak run off is decreased by minimum of 8.1% and maximum of 26.7 %. Moreover, peak runoff time is increased by a minimum of 14.3 % and a maximum of 100 %.

Table 30. Comparison of Peak Discharge

Conduit	Existing Peak Discharge				LID Scenario				Peak Discharge reduction (%)	
	10 yrs return period		25 yrs return period		10 yrs return period		25 yrs return period		10 yr return Period	25 yr return Period
	Peak time	Q (m ³ /s)	Peak Time	Q (m ³ /s)	Peak time	Q (m ³ /s)	Peak time	Q (m ³ /s)		
D_36	1:10:00	2.31	1:10:00	2.85	1:40:00	1.95	1:20:00	2.62	31.6	8.1
D_37	1:20:00	2.29	0:50:00	2.38	1:40:00	1.97	1:20:00	2.38	17.2	0
D_89	1:10:00	3.30	1:10:00	4.05	1:50:00	2.63	1:40:00	3.23	35.1	20.2
D_90	1:50:00	3.49	1:30:00	4.49	1:50:00	3.04	1:50:00	3.47	32.3	22.7
D_92	0:40:00	1.34	0:30:00	1.34	2:50:00	1.32	0:50:00	1.30	1.5	3.0

Regarding 10 yrs return period, the magnitude of peak discharge is decreased with the minimum of 1.5% and maximum of 35.1 %. In addition, peak discharge time is increased by a minimum of 0 % and a maximum of 325.0 %. With respect to 25 yrs return period, magnitude of peak discharge is decreased by a minimum of 0 % and a maximum of 22.7 %. Moreover, peak discharge time is increased by a minimum of 14.3 % and a maximum of 66.7 %.

Chapter Five: Conclusion and Recommendation

5.1. Conclusion

In the study a storm water drainage network performance of Mojo town is examined using Storm water Management model. Thus, it is hoped that the study results are helpful for a better planning and management of urban drainage system of the town as well as for other towns with similar situations. From the study the following conclusions are deduced as general and specific to the Mojo town.

General

- ✓ Urban drainage infrastructure development and management were not given due attention as of other public infrastructures and public utilities. Absence of urban drainage master plan and an organized existing drainage data are evidences for the reluctance being observed in the sector.

Specific

- ✓ There has been an intense land use/ land cover change due to urbanization that has led to an increased peak runoff flows and volume because of a significant increase of impervious surface of the town. Within eleven years period, 58.7% of the model area is used for built up so that peak discharge, peak run off and total runoff are increased up to 59.2%, 79.31 & 44.4 %, respectively.
- ✓ According to simulation of SWMM for 10 years return period, the existing drainage system of the town is insufficient to accommodate the runoff being generated in the town. Out of the total drainage system, 27.0 % and 37.8% of conduits and junctions do not provide proper service which could be a cause for flood induced problems. As the return period increases, the malfunctioning proportion of conduits, junctions & other related drainage infrastructures supposed to be increased.
- ✓ Result of the existing system simulation particularly after 3:00:00 hours indicated that flow velocity of most of the conduits categorized below the minimum flow velocity (0.9m/s).This implies that redesigning of drains and related drainage infrastructure is critically required in such a way to meet the minimum limit of velocity and other important parameters as well.
- ✓ With respect to 10 years return period, LID application helps to decrease the magnitude of peak run off with the minimum of 11.9% and maximum of 34.8 %.Moreover, peak runoff time is increased by a minimum of 22.2 % and a maximum of 133.3 %.Regarding 25 yrs return period, magnitude of peak run off is decreased by minimum

- of 8.1% and maximum of 26.7 %.Moreover, peak runoff time is increased by a minimum of 14.3 % and a maximum of 100 %. This situation is enables significantly to improve the drainage system performance of the town.
- ✓ Green areas and parks are well known to enhance high infiltration of rainfall. According to the master plan of Mojo town, 30 % of the area is allocated for green area & park development. However, based on the model area the green area coverage of the town so far is only 2.1% which is very insignificant. Most of the areas allocated for green development are utilized for other land uses due to this reason the areas are subjected to an increased runoff and land degradation.

5.2. Recommendation

Based on the findings of the study, the following recommendations are given as general and specific to the Mojo town.

General

- ✓ As a Municipality, Mojo town is expected to implement a paramount urban drainage infrastructure development along with other urban infrastructures. Proper implementation of the town master plan helps to regulate reasonable land use land cover change. Moreover, the town needs to have drainage master plan as soon as possible that enhances proper urban drainage management and green infrastructure development.

Specific

- ✓ Redesigning of drains and associated structures is required.
 - ✚ Increasing the size of conduits where flow velocity is beyond $3\text{m}^2/\text{s}$ and overflow has happened as well.
 - ✚ Increasing of conduit slope where it was difficult to attain cleansing velocity.
- ✓ There are locations where additional drainage network is required. The drainage of Addis Ababa-Adama Express way Road and Addis Ababa-Djibouti Railway are not interconnected with the town drainage system. Therefore, Sub catchments (SC_17, SC_42, SC_43, SC_44, SC_45) need drainage network that should cover the whole sub catchment incorporating the two mega projects. In some areas (SC_1 & SC_19) micro drainages are required that help to collect storm water from the source and supply to the main channels.
- ✓ Due to the fact that the major source of flood in the town is from the two mega projects, it is recommended to do LID structural measures in the farm land which is located near by the projects so as to capture the storm water before it reaches to residential and commercial areas. Bio retention & Infiltration trenches majorly recommended structures in order to do in eight sub catchments (SC_1, SC_17, SC_19, SC_37, SC_42, SC_43, SC_44, SC_45).

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Annex

Annex 1.Outlier test K_N value table

Sample Size, n	K_N	Sample Size, n	K_N	Sample Size, n	K_N	Sample Size, n	K_N
10	2.036	24	2.467	38	2.661	60	2.837
11	2.088	25	2.486	39	2.671	65	2.866
12	2.134	26	2.502	40	2.682	70	2.893
13	2.175	27	2.519	41	2.692	75	2.917
14	2.213	28	2.534	42	2.7	80	2.940
15	2.247	29	2.549	43	2.710	85	2.961
16	2.279	30	2.563	44	2.719	90	2.981
17	2.309	31	2.577	45	2.727	95	3.000
18	2.335	32	2.591	46	2.736	100	3.017
19	2.361	33	2.604	47	2.744	110	3.049
20	2.385	34	2.616	48	2.753	120	3.078
21	2.408	35	2.628	49	2.76	130	3.104
22	2.429	36	2.639	50	2.768	140	3.129
23	2.448	37	2.65	55	2.804	141	3.131

Annex 2. Table that depicts threshold of outliers

Sample No	Year	Max Rainfall (RF)	Log(RF)
1	1997	54.2	1.7339993
2	1998	83	1.9190781
3	1999	92	1.9637878
4	2000	44.8	1.651278
5	2001	42.5	1.6283889
6	2002	46.8	1.6702459
7	2003	85.1	1.9299296
8	2004	47.2	1.673942
9	2005	56.3	1.7505084
10	2006	44.5	1.64836
11	2007	38.8	1.5888317
12	2008	94.9	1.9772662
13	2009	73.2	1.8645111
14	2010	56.1	1.7489629
15	2011	44.3	1.6464037
16	2012	81.4	1.9106244
17	2013	62.8	1.7979596
18	2014	61.8	1.7909885
19	2015	44.8	1.651278
20	2016	91.6	1.9618955
21	2017	64.6	1.8102325
	Skewness	0.522870129	0.2910708
	Av.	62.41428571	1.7770701
	St	18.77064958	0.1280765
	K _N	2.408	2.408

Threshold	Higher	107.61	2.0854783
	lower	17.2	1.4686619

Annex 3. Cumulative rainfall data of Mojo, Koka and Adama stations for Double mass curve development

Year	Mojo Station	Koka Station	Nazret Station	Cumulative			Average
				Mojo Station	Koka Station	Adama Station	
1997	760.3	875.5	876.6	760.3	875.5	876.6	837.5
1998	1033.9	1193.7	961.6	1794.2	2069.2	1838.2	1900.5
1999	1216	814.6	849.3	3010.2	2883.8	2687.5	2860.5
2000	852.1	1164.7	1062.7	3862.3	4048.5	3750.2	3887
2001	788.1	1612	849.6	4650.4	5660.5	4599.8	4970.2
2002	564.1	615.6	614.4	5214.5	6276.1	5214.2	5568.3
2003	1060.1	1007.2	1126.7	6274.6	7283.3	6340.9	6632.9
2004	1300	776.4	719.6	7574.6	8059.7	7060.5	7564.9
2005	1632	533.2	720.5	9206.6	8592.9	7781	8526.8
2006	1199.1	833.2	912.2	10405.7	9426.1	8693.2	9508.3
2007	1001.6	832.9	1107.2	11407.3	10259	9800.4	10488.9
2008	1441.4	1122.1	1092.2	12848.7	11381.1	10892.6	11707.5
2009	1108.3	689.5	588.7	13957	12070.6	11481.3	12503.0
2010	1110.4	1120.1	1035.5	15067.4	13190.7	12516.8	13591.6
2011	854.6	951	744.7	15922	14141.7	13261.5	14441.7
2012	1370.14	1018.9	1213.7	17292.14	15160.6	14475.2	15642.6
2013	1096.4	764.7	904.2	18388.54	15925.3	15379.4	16564.4
2014	875.1	748.5	847.6	19263.64	16673.8	16227	17388.1
2015	859.4	582.1	747.5	20123.04	17255.9	16974.5	18117.8
2016	1275.8	893.9	1005.6	21398.84	18149.8	17980.1	19176.2
2017	1036.4	1019.7	1031	22435.24	19169.5	19011.1	20205.3

Annex 4. Estimation of rain fall (mm/hr.) for short duration

Year	Time in min.																
	10	20	30	40	50	60	70	80	90	100	120	130	140	150	160	170	180
	Time in hour																
	0.167	0.333	0.5	0.667	0.833	1	1.167	1.333	1.5	1.667	2	2.167	2.333	2.5	2.667	2.833	3
1997	89.10	67.15	54.02	45.29	39.08	34.39	30.74	27.82	25.41	23.40	20.23	18.95	17.84	16.85	15.97	15.17	14.46
1998	146.11	110.27	88.70	74.36	64.18	56.49	50.47	45.68	41.73	38.42	33.22	31.12	29.29	27.66	26.21	24.92	23.75
1999	151.26	113.99	91.68	76.87	66.34	58.37	52.17	47.22	43.13	39.71	34.34	32.17	30.28	28.60	27.10	25.76	24.55
2000	73.65	55.50	44.64	37.44	32.30	28.43	25.41	22.99	21.04	19.34	16.72	15.67	14.74	13.92	13.19	12.55	11.95
2001	69.88	52.67	42.36	35.50	30.65	26.97	24.10	21.82	19.93	18.34	15.87	14.86	13.99	13.21	12.52	11.90	11.34
2002	76.95	57.99	46.64	39.10	33.75	26.69	26.54	24.02	21.94	20.20	17.47	16.36	15.41	14.55	13.78	13.10	12.49
2003	139.94	105.44	84.82	71.11	61.36	54.00	48.25	43.68	39.89	36.74	31.77	29.76	28.01	26.45	25.07	23.83	22.67
2004	77.60	58.50	47.04	39.43	34.03	29.95	26.76	24.22	22.13	20.38	17.62	16.51	15.53	14.67	13.90	13.22	12.59
2005	87.72	69.76	56.10	47.05	40.60	35.72	31.93	28.90	26.39	24.30	21.02	19.69	18.53	17.50	16.58	15.76	15.02
2006	73.17	55.14	44.34	37.18	32.09	28.24	25.24	22.84	20.86	19.21	16.61	15.56	14.65	13.83	13.00	12.46	11.87
2007	63.77	48.08	38.66	32.41	27.97	24.62	22.01	19.92	18.19	16.75	14.48	13.57	12.77	12.06	11.43	10.86	10.35
2008	156.05	117.60	94.58	79.30	68.43	60.21	53.81	48.71	44.49	40.97	35.42	33.18	31.23	29.50	27.95	26.57	25.32
2009	120.36	90.69	72.96	61.15	52.79	46.45	41.51	37.57	34.32	31.60	27.32	25.60	24.09	22.75	21.40	20.50	19.53
2010	92.22	69.52	55.92	46.87	40.46	35.60	31.81	28.79	26.30	24.22	20.94	19.62	18.47	17.44	16.52	15.71	14.97
2011	72.81	55.50	44.16	37.02	31.94	28.11	25.12	22.74	20.77	19.12	16.54	15.49	14.58	13.77	13.05	12.40	11.82
2012	133.83	100.87	81.16	68.01	58.69	51.65	46.16	41.78	38.16	35.14	30.38	28.46	26.79	25.30	23.97	22.79	21.72
2013	103.23	77.81	62.58	52.47	45.28	39.85	35.61	32.24	29.44	27.11	23.44	21.96	20.67	19.52	18.50	17.59	16.75
2014	101.62	76.58	61.60	51.63	44.56	39.21	35.05	31.72	28.97	26.68	23.07	21.61	20.34	19.21	18.20	17.30	16.49
2015	73.65	55.50	44.64	37.44	32.30	28.43	25.41	22.99	21.00	19.34	16.72	15.67	14.74	13.92	13.19	12.55	11.95
2016	150.60	113.51	91.30	76.54	66.05	58.12	51.95	46.99	42.94	39.54	34.19	32.03	30.15	28.47	26.98	25.65	24.44
2017	105.87	79.79	64.18	53.81	46.43	40.86	36.52	33.05	30.19	27.80	24.04	22.52	21.20	20.02	18.97	18.03	17.18

Annex 5. Average rain fall (mm/hr) and standard deviation for each durations

Year	Time in min.																
	10	20	30	40	50	60	70	80	90	100	120	130	140	150	160	170	180
	Time in hour																
	0.167	0.333	0.5	0.667	0.833	1	1.167	1.333	1.5	1.667	2	2.167	2.333	2.5	2.667	2.833	3
Av.	102.83	77.71	62.48	52.38	45.20	39.64	35.55	32.17	29.39	27.06	23.40	21.92	20.63	19.49	18.45	17.55	16.72
Std.	30.80	23.12	18.62	15.61	13.47	11.99	10.59	9.59	8.76	8.06	6.97	6.53	6.15	5.81	5.50	5.23	4.98

Annex 6. Rainfall intensity in mm/hr for short rainfall duration (Gumbel Distribution)

Duration		Return Period					
		2	5	10	25	50	100
10	0.167	97.7694	124.986	143.01	165.77	182.67	199.4
20	0.333	73.9086	94.339	107.87	124.96	137.64	150.2
30	0.5	59.4213	75.8762	86.771	100.54	110.75	120.9
40	0.667	49.8156	63.6087	72.741	84.279	92.839	101.3
50	0.833	42.9912	54.894	62.775	72.732	80.119	87.45
60	1	37.6665	48.2627	55.278	64.143	70.719	77.25
70	1.167	33.8099	43.1704	49.368	57.198	63.008	68.77
80	1.333	30.6002	39.0711	44.68	51.766	57.023	62.24
90	1.5	27.953	35.6912	40.814	47.288	52.09	56.86
100	1.667	25.7376	32.8639	37.582	43.544	47.966	52.36
120	2	22.2535	28.415	32.494	37.649	41.473	45.27
130	2.167	20.8491	26.6215	30.443	35.272	38.855	42.41
140	2.333	19.6242	25.0577	28.655	33.201	36.573	39.92
150	2.5	18.5331	23.6642	27.061	31.354	34.538	37.7
160	2.667	17.5482	22.4112	25.631	29.699	32.717	35.71
170	2.833	16.6941	21.3163	24.377	28.243	31.112	33.96
180	3	15.9061	20.3086	23.223	26.906	29.638	32.35

Annex 7. Logarithmic value of Estimated rain fall (mm/hr) for short duration

Year	Time in min.																	
	10	20	30	40	50	60	70	80	90	100	120	130	140	150	160	170	180	
	Time in hour																	
	0.167	0.333	0.5	0.667	0.833	1	1.167	1.333	1.5	1.667	2	2.167	2.333	2.5	2.667	2.833	3	
1997	1.950754	1.826593	1.732555	1.656242	1.591732	1.536432	1.487785	1.444201	1.404948	1.369216	1.305996	1.277732	1.251325	1.226548	1.203237	1.18107	1.160164	
1998	2.165541	2.042024	1.947924	1.871573	1.80721	1.751972	1.703169	1.659607	1.620414	1.58467	1.5214	1.493125	1.466698	1.441915	1.418571	1.396507	1.375598	
1999	2.180585	2.056447	1.962275	1.885955	1.821592	1.76619	1.717528	1.673988	1.634813	1.599009	1.5358	1.507565	1.481135	1.456305	1.432989	1.410906	1.389988	
2000	1.868056	1.743823	1.649724	1.57351	1.509095	1.453777	1.405078	1.361492	1.323046	1.286546	1.223236	1.195047	1.168581	1.143764	1.120451	1.09842	1.077485	
2001	1.845222	1.721151	1.626956	1.550473	1.486232	1.430881	1.382069	1.338656	1.299435	1.263589	1.20044	1.172198	1.145773	1.120969	1.097648	1.075461	1.054609	
2002	1.887054	1.762904	1.668759	1.592399	1.528042	1.426349	1.423994	1.380483	1.341237	1.305523	1.242293	1.213946	1.187601	1.162803	1.139407	1.11731	1.096442	
2003	2.14681	2.022552	1.928498	1.852144	1.787687	1.732394	1.683639	1.640133	1.6009	1.565186	1.501949	1.4737	1.447268	1.422459	1.399133	1.377006	1.355447	
2004	1.890756	1.76671	1.672467	1.596047	1.531734	1.476397	1.427625	1.384129	1.344916	1.309247	1.245883	1.217726	1.191211	1.166489	1.143171	1.121038	1.100136	
2005	1.943989	1.84317	1.748963	1.672744	1.608355	1.552911	1.504296	1.460748	1.421494	1.385713	1.322529	1.294228	1.267808	1.243038	1.219748	1.197621	1.176666	
2006	1.865222	1.740994	1.646796	1.570543	1.50618	1.450865	1.402139	1.358506	1.319314	1.283573	1.22037	1.192095	1.165668	1.140885	1.11386	1.095477	1.074568	
2007	1.805501	1.681513	1.587262	1.510947	1.446537	1.391288	1.342648	1.299126	1.259753	1.224067	1.160769	1.132555	1.106093	1.081347	1.058046	1.035971	1.015076	
2008	2.194126	2.069964	1.975799	1.899465	1.835056	1.779669	1.731013	1.687507	1.648295	1.612508	1.549249	1.520997	1.494572	1.469822	1.446479	1.424382	1.403459	
2009	2.081347	1.957128	1.863085	1.786645	1.722338	1.666986	1.618257	1.574726	1.535547	1.499797	1.436481	1.408266	1.381836	1.35702	1.330439	1.311654	1.290698	
2010	1.965672	1.841672	1.747567	1.67108	1.606811	1.55145	1.502661	1.459166	1.419956	1.38421	1.320977	1.292699	1.266298	1.241546	1.218174	1.196062	1.175121	
2011	1.863085	1.743823	1.645029	1.568612	1.504226	1.448861	1.400217	1.356647	1.317367	1.28167	1.218404	1.190159	1.163757	1.138997	1.115611	1.093504	1.072613	
2012	2.127429	2.003331	1.909342	1.832764	1.768401	1.71307	1.6644	1.620838	1.581608	1.545901	1.482588	1.454352	1.427972	1.403189	1.379804	1.357733	1.336789	
2013	2.014689	1.890589	1.796436	1.720159	1.655753	1.600428	1.551729	1.508227	1.468938	1.433194	1.369958	1.341723	1.31525	1.29048	1.267142	1.245106	1.224097	
2014	2.007833	1.883661	1.789581	1.713154	1.648789	1.593397	1.544777	1.501196	1.461998	1.426218	1.362953	1.334732	1.308351	1.283573	1.260221	1.238075	1.217129	
2015	1.868056	1.743823	1.649724	1.57351	1.509095	1.453777	1.405078	1.361492	1.322219	1.286546	1.223236	1.195047	1.168581	1.143764	1.120451	1.09842	1.077485	
2016	2.178689	2.054613	1.960471	1.884087	1.819702	1.764326	1.715669	1.671913	1.632862	1.597168	1.533899	1.50563	1.479225	1.454418	1.431122	1.408998	1.388097	
2017	2.025634	1.901513	1.8074	1.731065	1.666668	1.611298	1.562667	1.519106	1.479911	1.444107	1.380844	1.352627	1.326189	1.301464	1.278096	1.255953	1.235103	

Annex 8 .Average (\bar{X}), standard deviation (s) and skew coefficient (G) values

Year	Time in min.																	
	10	20	30	40	50	60	70	80	90	100	120	130	140	150	160	170	180	
	Time in hour																	
	0.167	0.333	0.5	0.667	0.833	1	1.167	1.333	1.5	1.667	2	2.167	2.333	2.5	2.667	2.833	3	
Average	1.994098	1.871333	1.776981	1.700625	1.636249	1.578701	1.532211	1.488661	1.449475	1.413698	1.350441	1.322198	1.295771	1.27099	1.247324	1.225556	1.204608	
Stand dv.	0.130228	0.129665	0.129926	0.129905	0.129898	0.132193	0.129893	0.129879	0.129876	0.129903	0.129902	0.129896	0.129903	0.129897	0.129978	0.129898	0.129852	
G	0.266646	0.247165	0.264355	0.264509	0.264495	0.248522	0.264676	0.264606	0.264778	0.264692	0.264639	0.264692	0.264589	0.264481	0.265965	0.264749	0.26492	

Annex 9. K value for Log Pearson Type III distribution

Skew Coefficient (G)	Recurrence interval						
	2	5	10	25	50	100	200
	Per cent chance (annual probability of occurrence p(%))						
	50	20	10	4	2	1	0.50
3.5	-0.396	0.420	1.18	2.278	3.152	4.051	4.97
2.5	-0.36	0.518	1.25	2.162	3.048	3.845	4.652
2	-0.307	0.609	1.302	2.219	2.912	3.605	4.298
1.8	-0.282	0.643	1.318	2.193	2.848	3.499	4.147
1.6	-0.254	0.675	1.329	2.163	2.78	3.388	3.99
1.4	-0.225	0.705	1.337	2.128	2.706	3.271	3.828
1.2	-0.195	0.732	1.34	2.087	2.626	3.149	3.661
1.0	-0.164	0.758	1.34	2.043	2.542	3.022	3.489
0.9	-0.148	0.769	1.339	2.018	2.498	2.957	3.401
0.8	-0.132	0.78	1.336	1.993	2.453	2.891	3.312
0.7	-0.116	0.79	1.333	1.967	2.407	2.824	3.223
0.6	-0.099	0.8	1.328	1.939	2.359	2.755	3.132
0.5	-0.083	0.808	1.323	1.91	2.311	2.686	3.041
0.4	-0.066	0.816	1.317	1.88	2.261	2.615	2.949
0.3	-0.050	0.823	1.309	1.849	2.211	2.544	2.86
0.2	-0.033	0.830	1.301	1.818	2.159	2.472	2.763
0.1	-0.017	0.836	1.292	1.785	2.107	2.400	2.67
0	0.000	0.841	1.282	1.751	2.054	2.326	2.576
-0.1	0.017	0.846	1.27	1.716	2	2.252	2.482
-0.2	0.033	0.85	1.258	1.680	1.945	2.178	2.388
-0.3	0.050	0.853	1.245	1.643	1.89	2.104	2.294
-0.4	0.066	0.855	1.231	1.606	1.834	2.029	2.201
-0.5	0.083	0.86	1.216	1.567	1.777	1.955	2.108
-0.6	0.099	0.857	1.2	1.528	1.72	1.88	2.016
-0.7	0.115	0.857	1.183	1.488	1.663	1.806	1.926
-0.8	0.132	0.856	1.166	1.448	1.606	1.733	1.837
-0.9	0.148	0.854	1.147	1.407	1.549	1.66	1.749
-1.0	0.164	0.852	1.128	1.366	1.492	1.588	1.664
-1.2	0.195	0.843	1.086	1.282	1.379	1.449	1.501
-1.4	0.225	0.832	1.041	1.198	1.27	1.318	1.351
-1.6	0.254	0.817	0.994	1.116	1.166	1.197	1.216
-1.8	0.282	0.799	0.945	1.035	1.069	1.087	1.097
-2.0	0.307	0.777	0.895	0.959	0.98	0.99	0.995
-2.5	0.360	0.710	0.771	0.793	0.798	0.799	0.80
-3.0	0.396	0.636	0.660	0.660	0.666	0.667	0.667

Annex 10. Rainfall intensity in mm/hr for short rainfall duration (Log Pearson III distribution)

Duration in mint.	Return Period					
	2	5	10	25	50	100
10	97.2436	126.3288	146.0284	171.5494	188.8422	210.9776
20	73.38145	95.15627	109.8294	128.7484	142.4075	157.7915
30	59.05715	76.61576	88.44712	103.6999	114.7887	127.1095
40	49.53512	64.26037	74.18256	86.97416	96.26925	106.6066
50	42.71088	55.40666	63.96122	74.98957	83.00365	91.91555
60	37.43243	48.7569	56.38933	66.22408	73.72995	81.30176
70	33.61218	43.60317	50.33541	59.0147	65.31835	72.33597
80	30.40526	39.44185	45.53077	53.38046	59.08269	65.42797
90	27.78169	36.03852	41.60219	48.77496	53.98278	59.78401
100	25.58484	33.19045	38.31545	44.92277	49.7214	55.06409
120	22.11701	28.69163	33.12185	38.83337	42.98206	47.59962
130	20.72442	26.88479	31.03587	36.38754	40.2742	44.60152
140	19.50105	25.29807	29.20428	34.2402	37.8987	41.96943
150	18.41958	23.89474	27.58396	32.33999	35.79624	39.63937
160	17.44122	22.63033	26.12827	30.63946	33.90362	37.5666
170	16.58973	21.52122	24.84429	29.12852	32.23945	35.7043
180	15.80847	20.50593	23.67119	27.75188	30.71337	34.01512

Annex 11. Manning's n –Overland Flow

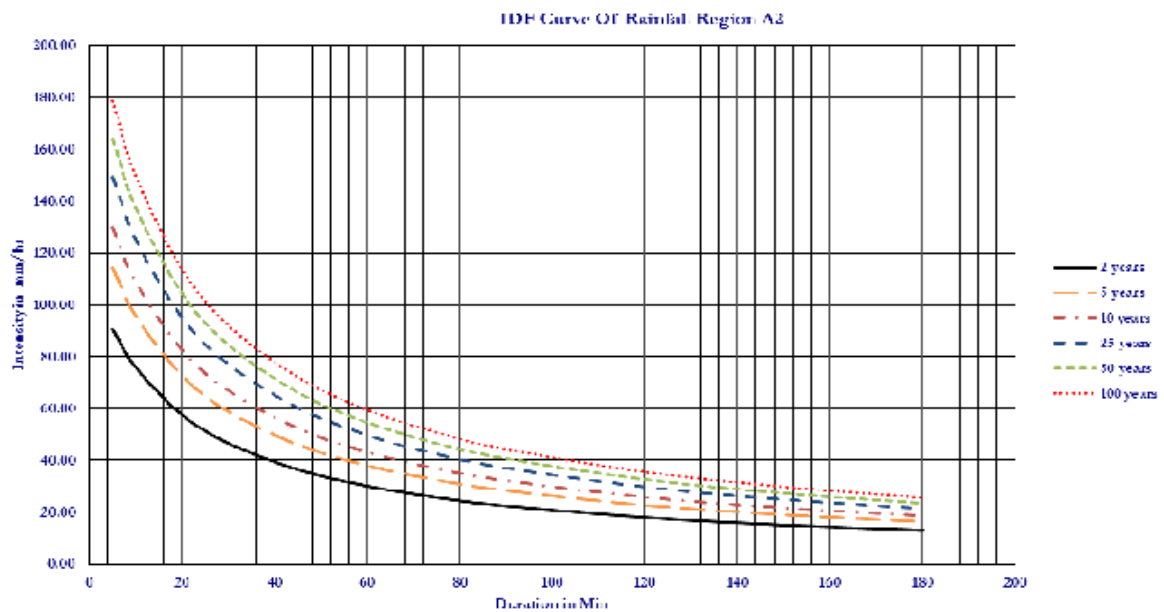
Surface	n
Smooth asphalt	0.011
Smooth concrete	0.012
Ordinary concrete line	0.013
Good wood	0.014
Brick with cement mortar	0.014
Vitrified clay	0.015
Cast iron	0.015
Corrugated metal pipe	0.024
Cement rubble surface	0.024
Fallow soil (no residue)	0.05
Cultivated	
Residue cover < 20 %	0.06
Residue cover > 20 %	0.017
Range (natural)	0.013
Grass	
Short prairie	0.15
Dense	0.24
Bermuda grass	0.41
Wood	
Light under bush	0.40
Dense under bush	0.80

Source: McCuen, R. et al. (1996), *Hydrology*, FHWA-SA-96-067, Federal Highway Administration, Washington, DC

Annex 12. Rainfall Region Categories of Ethiopia (ERA,2013)



Annex 13. IDF curve of Rainfall Region A2



Annex 14. Manning's n for Channels (Chow, 1959).

Type of Channel and Description	Minimum	Normal	Maximum
1. Lined or Constructed Channels			
a. Cement			
1. neat surface	0.010	0.011	0.013
2. mortar	0.011	0.013	0.015
b. Wood			
1. planed, untreated	0.010	0.012	0.014
2. planed, creosoted	0.011	0.012	0.015
3. un planed	0.011	0.013	0.015
4. plank with battens	0.012	0.015	0.018
5. lined with roofing paper	0.010	0.014	0.017
c. Concrete			
1. trowel finish	0.011	0.013	0.015
2. float finish	0.013	0.015	0.016
3. finished, with gravel on bottom	0.015	0.017	0.020
4. unfinished	0.014	0.017	0.020
5. gunite, good section	0.016	0.019	0.023
6. gunite, wavy section	0.018	0.022	0.025
7. on good excavated rock	0.017	0.020	
8. on irregular excavated rock	0.022	0.027	
d. Concrete bottom float finish with sides of:			
1. dressed stone in mortar	0.015	0.017	0.020
2. random stone in mortar	0.017	0.020	0.024
3. cement rubble masonry, plastered	0.016	0.020	0.024
4. cement rubble masonry	0.020	0.025	0.030
5. dry rubble or riprap	0.020	0.030	0.035
e. Gravel bottom with sides of:			
1. formed concrete	0.017	0.020	0.025
2. random stone mortar	0.020	0.023	0.026
3. dry rubble or riprap	0.023	0.033	0.036
f. Brick			
1. glazed	0.011	0.013	0.015
2. in cement mortar	0.012	0.015	0.018
g. Masonry			
1. cemented rubble	0.017	0.025	0.030
2. dry rubble	0.023	0.032	0.035
h. Dressed ashlar/stone paving	0.013	0.015	0.017
i. Asphalt			
1. smooth	0.013	0.013	
2. rough	0.016	0.016	
j. Vegetal lining	0.030		0.500
h. Excavated or Dredged Channels			
Earth, straight, and uniform			
1. clean, recently completed	0.016	0.018	0.020
2. clean, after weathering	0.018	0.022	0.025
3. gravel, uniform section, clean	0.022	0.025	0.030
4. with short grass, few weeds	0.022	0.027	0.033

Annex 15. Depression storage values

Impervious surface	0.05-0.10 inches
Lawns	0.10-0.20 inches
Pasture	0.20 inches
Forest Litter	0.3 inches

(Source: ASCE (1992), Design & construction of stormwater Management Systems, New York, NY).

Annex 16. Soil physical characteristic value table

Soil Texture Class	Saturated Hydraulic Conductivity (in/hr.)	Suction Head (in.)	Porosity (fraction)	Field Capacity (fraction)	Wilting Point (fraction)
Sand	4.74	1.93	0.437	0.062	0.024
Loamy Sand	1.18	2.40	0.437	0.105	0.047
Sandy Loam	0.43	4.33	0.453	0.190	0.085
Loam	0.13	3.50	0.463	0.232	0.116
Silt Loam	0.26	6.69	0.501	0.284	0.135
Sandy Clay Loam	0.06	8.66	0.398	0.244	0.136
Clay Loam	0.04	8.27	0.464	0.310	0.187
Silty Clay Loam	0.04	10.63	0.471	0.342	0.210
Sandy Clay	0.02	9.45	0.430	0.321	0.221
Silty Clay	0.02	11.42	0.479	0.371	0.251
Clay	0.01	12.60	0.475	0.378	0.265

(Source: EPA SWMM 5)

Annex 17. Observed discharges of D_48

Part 1

Observed discharge records of D_48 at July 15/2020 in Mojo town								
Date :July 15/20 Recording time	Time in hours	Depth in meter	Time in second for 3 meter distance	Velocity (m/s)	Top width in meter	Area in m ³	Discharge in m ³ /sec.	
	7:15:00 AM	0:00	0	0	0	0	0	
	8:15:00 AM	1:00	1.06	1.65	1.818	1.28	1.38	2.505
	9:15:00 AM	2:00	1.1	1.3	2.308	1.30	1.43	3.300
	10:15:00 AM	3:00	0.99	1.65	1.818	1.26	1.29	2.340
	11:15:00 AM	4:00	0.97	1.7	1.765	1.25	1.26	2.225
	12:15:00 PM	5:00	0.96	1.7	1.765	1.24	1.25	2.202
	1:15:00 PM	6:00	0.85	1.8	1.667	1.20	1.11	1.842
	2:15:00 PM	7:00	0.71	2.3	1.304	1.14	0.92	1.204
	3:15:00 PM	8:00	0.55	3.1	0.968	1.08	0.72	0.692
	4:15:00 PM	9:00	0.39	4.3	0.698	1.01	0.51	0.354
	5:15:00 PM	10:00	0.21	10.3	0.291	0.94	0.27	0.080
	6:15:00 PM	11:00	0.14	15.4	0.195	0.91	0.18	0.035
	7:15:00 PM	12:00	0.06	26.7	0.112	0.87	0.08	0.009

Observed discharge records of D_48 at August 1/2020 in Mojo town								
Date :August 01/20 Recording time	Time in hours	Depth in meter	Time in second for 3 meter distance	Velocity (m/s)	Top width in meter	Area in m ³	Discharge in m ³ /sec.	
	1:00:00 PM	0:00	0	0	0	0	0	
	2:00:00 PM	1:00	1	1.55	1.935	1.26	1.30	2.516
	3:00:00 PM	2:00	1.1	1.35	2.222	1.30	1.43	3.178
	4:00:00 PM	3:00	1.05	1.5	2.000	1.28	1.37	2.730
	5:00:00 PM	4:00	0.99	1.53	1.961	1.26	1.29	2.524
	6:00:00 PM	5:00	0.91	1.7	1.765	1.22	1.18	2.088
	7:00:00 PM	6:00	0.84	1.9	1.579	1.19	1.09	1.724
	8:00:00 PM	7:00	0.71	2.22	1.351	1.14	0.92	1.247
	9:00:00 PM	8:00	0.6	2.6	1.154	1.10	0.78	0.900
	10:00:00 PM	9:00	0.46	4	0.750	1.04	0.60	0.449
	11:00:00 PM	10:00	0.32	5.3	0.566	0.98	0.42	0.235
	12:00:00 AM	11:00	0.19	10.4	0.288	0.93	0.25	0.071
	1:00:00 AM	12:00	0.09	22.1	0.136	0.89	0.12	0.016
	2:00:00 AM	13:00	0	0				0
	3:00:00 AM	14:00	0	0				0

Part 2

Observed discharge records of D_48 at August 07/2020 in Mojo town

Date :August 07/20 Recording time	Time in hours	Depth in meter	Time in second for 3 meter distance	Velocity (m/s)	Top width in meter	Area in m3	Discharge in m3/sec.
1:00:00 PM	0:00	0	0	0	0	0	0
2:00:00 PM	1:00	1	1.5	2.000	1.26	1.30	2.600
3:00:00 PM	2:00	1.1	1.35	2.222	1.30	1.43	3.178
4:00:00 PM	3:00	1.05	1.4	2.143	1.28	1.37	2.925
5:00:00 PM	4:00	1.01	1.5	2.000	1.26	1.31	2.626
6:00:00 PM	5:00	0.97	1.6	1.875	1.25	1.26	2.364
7:00:00 PM	6:00	0.85	1.9	1.579	1.20	1.11	1.745
8:00:00 PM	7:00	0.69	3	1.000	1.13	0.90	0.897
9:00:00 PM	8:00	0.5	3.3	0.909	1.05	0.65	0.591
10:00:00 PM	9:00	0.4	4	0.750	1.01	0.52	0.390
11:00:00 PM	10:00	0.28	4.3	0.698	0.96	0.36	0.254
12:00:00 AM	11:00	0.2	4.5	0.667	0.93	0.26	0.173
1:00:00 AM	12:00	0.15	11	0.273	0.91	0.20	0.053
2:00:00 AM	13:00	0.06	20.3	0.148	0.87	0.078	0.012
3:00:00 AM	14:00	0				0	0

Observed discharge records of D_48 at August 21/2020 in Mojo town

Date :August 21/20 Recording time	Time in hours	Depth in meter	Time in second for 3 meter distance	Velocity (m/s)	Top width in meter	Area in m3	Discharge in m3/sec.
7:30:00 AM	0:00	0	0	0	0	0	0
8:30:00 AM	1:00	0.94	1.65	1.818	1.23	1.22	2.222
9:30:00 PM	2:00	1.1	1.55	1.935	1.30	1.43	2.768
10:30:00 PM	3:00	1.05	1.6	1.875	1.28	1.37	2.559
11:00:00 AM	4:00	1	1.65	1.818	1.26	1.30	2.364
12:30:00 PM	5:00	0.97	1.7	1.765	1.25	1.26	2.225
1:30:00 PM	6:00	0.88	2.2	1.364	1.21	1.14	1.560
2:30:00 PM	7:00	0.71	2.7	1.111	1.14	0.92	1.026
3:30:00 PM	8:00	0.53	3.5	0.857	1.07	0.69	0.591
4:30:00 PM	9:00	0.31	4.4	0.682	0.98	0.40	0.275
5:00:00 PM	10:00	0.24	5.2	0.577	0.95	0.31	0.180
6:00:00 PM	11:00	0.1	20.5	0.146	0.89	0.13	0.019
7:30:00 AM	12:00	0.01	42	0.071	0.85	0.01	0.001
8:30:00 AM	13:00	0				0	0.000

Annex 18: Observed discharges of D_89

Part 1

Observed discharge records of D_89 at July 15/2020 in Mojo town								
Date :July 15/20	Recording time	Time in hours	Depth in meter	Time in second for 32.60 meter distance	Velocity (m/s)	Area in m ³	Discharge in m ³ /sec.	Remark
	7:15:00 AM	0:00	0					
	8:15:00 AM	1:00	0.85	20.6	1.583	1.28	2.02	
	9:15:00 AM	2:00	0.84	21.1	1.545	1.26	1.95	
	10:15:00 AM	3:00	0.82	22	1.482	1.23	1.82	
	11:15:00 AM	4:00	0.69	24.5	1.331	1.04	1.38	
	12:15:00 PM	5:00	0.54	30.1	1.083	0.81	0.88	
	1:15:00 PM	6:00	0.31	49.3	0.661	0.47	0.31	
	2:15:00 PM	7:00	0.18	80.7	0.404	0.27	0.11	
	3:15:00 PM	8:00	0.05	149.4	0.218	0.08	0.02	
	4:15:00 PM	9:00	0			0.00	0.00	

Observed discharge records of D_89 at August 1/2020								
Date :August 1/20	Recording time	Time in hours	Depth in meter	Time in second for 32.6 meter distance	Velocity (m/s)	Area in m ³	Discharge in m ³ /sec.	Remark
	1:00:00 PM	0:00	0	0	0	0	0	
	2:00:00 PM	1:00	0.98	17	1.918	1.47	2.82	
	3:00:00 PM	2:00	0.88	20.1	1.622	1.32	2.14	
	4:00:00 PM	3:00	0.8	21.6	1.509	1.20	1.81	
	5:00:00 PM	4:00	0.55	29.1	1.120	0.83	0.92	
	6:00:00 PM	5:00	0.43	36.5	0.893	0.65	0.58	
	7:00:00 PM	6:00	0.3	46.5	0.701	0.45	0.32	
	8:00:00 PM	7:00	0.19	67.2	0.485	0.29	0.14	
	9:00:00 PM	8:00	0.08	78.5	0.415	0.12	0.05	
	10:00:00 PM	9:00	0	0		0.00	0.00	
	11:00:00 PM	10:00	0			0.00	0.00	

Part 2

Observed discharge records of D_89 at August 07/2020 in Mojo town

Date : August 07/20 Recording time	Time in hours	Depth in meter	Time in second for 30.60mete r distance	Velocity (m/s)	Discharge in m3/sec	Remark
1:00:00 PM	0:00	0				
2:00:00 PM	1:00	1.1	16.4	1.866	3.08	
3:00:00 PM	2:00	1	20.1	1.522	2.28	
4:00:00 PM	3:00	0.86	24.1	1.270	1.64	
5:00:00 PM	4:00	0.7	33.7	0.908	0.95	
6:00:00 PM	5:00	0.57	39	0.785	0.67	
7:00:00 PM	6:00	0.3	48.2	0.635	0.29	
8:00:00 PM	7:00	0.19	71.1	0.430	0.12	
9:00:00 PM	8:00	0.021	69.4	0.441	0.01	
10:00:00 PM	9:00		0		0.00	
11:00:00 PM	10:00	0	0	0.000	0.00	

Observed discharge records of D_89 at August 21/2020 in Mojo town

Date : August 21/20 Recording time	Time in hours	Depth in meter	Time in second for 30.60me ter distance	Velocity (m/s)	Area in m3	Discharge in m3/sec
7:30:00 AM	0:00	0				
8:30:00 AM	1:00	1.12	17.2	1.779	1.68	2.99
9:30:00 AM	2:00	1.1	20.1	1.522	1.65	2.51
10:30:00 AM	3:00	1.05	21.1	1.450	1.58	2.28
11:30:00 AM	4:00	0.7	36.7	0.834	1.05	0.88
12:30:00 PM	5:00	0.55	40.3	0.759	0.83	0.63
1:30:00 PM	6:00	0.4	49.2	0.622	0.60	0.37
2:30:00 PM	7:00	0.37	51.1	0.599	0.56	0.33
3:30:00 PM	8:00	0.19	72.4	0.423	0.29	0.12
4:30:00 PM	9:00	0.06	95.7	0.320	0.09	0.03
5:30:00 PM	10:00	0	0	0.000	0.00	0.00

Annex 19: Observed discharge of D₉₀
Part 1

Observed discharge records of D₉₀ at July 15/2020							
Date :July 15/20 Recording time	Time in hours	Depth in meter	Time in second for 39.10 meter distance	Velocity (m/s)	Area in m3	Discharge in m3/sec.	Remark
7:15:00 AM	0:00	0					
8:15:00 AM	1:00	0.71	28.2	1.387	2.13	2.95	
9:15:00 AM	2:00	0.69	29.9	1.308	2.07	2.71	
10:15:00 AM	3:00	0.64	32.2	1.214	1.92	2.33	
11:15:00 AM	4:00	0.51	42.9	0.911	1.53	1.39	
12:15:00 PM	5:00	0.36	66	0.592	1.08	0.64	
1:15:00 PM	6:00	0.25	80.5	0.486	0.75	0.36	
2:15:00 PM	7:00	0.16	161.6	0.242	0.48	0.12	
3:15:00 PM	8:00	0.03	180.1	0.217	0.09	0.02	
4:15:00 PM	9:00	0	0		0.00	0.00	

Observed discharge records of D₉₀ at August 1/2020							
Date :August 1/20 Recording time	Time in hours	Depth in meter	Time in second for 39.1 meter distance	Velocity (m/s)	Area in m3	Discharge in m3/sec.	Remark
1:00:00 PM	0:00	0	0	0	0	0	
2:00:00 PM	1:00	0.97	35.3	1.108	2.91	3.22	
3:00:00 PM	2:00	0.89	33.2	1.178	2.67	3.14	
4:00:00 PM	3:00	0.71	35.1	1.114	2.13	2.37	
5:00:00 PM	4:00	0.49	59.8	0.654	1.47	0.96	
6:00:00 PM	5:00	0.35	63.3	0.618	1.05	0.65	
7:00:00 PM	6:00	0.29	74.1	0.528	0.87	0.46	
8:00:00 PM	7:00	0.18	81.2	0.482	0.54	0.26	
9:00:00 PM	8:00	0.08	95.7	0.409	0.24	0.10	
10:00:00 PM	9:00	0				0.00	

Part 2

Observed discharge records of D_90 at August 07/2020							
Date :August 07/20			Time in second for 39.1 meter distance	Velocity (m/s)	Area in m3	Discharge in m3/sec.	
Recording time	Time in hours	Depth in meter					
1:00:00 PM	0:00	0	0	0	0	0	
2:00:00 PM	1:00	0.8	29.3	1.334	2.40	3.20	
3:00:00 PM	2:00	0.79	30.1	1.299	2.37	3.08	
4:00:00 PM	3:00	0.61	36.7	1.065	1.83	1.95	
5:00:00 PM	4:00	0.46	45.8	0.854	1.38	1.18	
6:00:00 PM	5:00	0.35	63	0.621	1.05	0.65	
7:00:00 PM	6:00	0.24	78.1	0.501	0.72	0.36	
8:00:00 PM	7:00	0.135	158.2	0.247	0.41	0.10	
9:00:00 PM	8:00	0.01	192.7	0.203	0.03	0.01	
10:00:00 PM	9:00	0				0.00	

Observed discharge records of D_90 at August 21/2020							
Date :August 21 /2020			Time in second for 39.1 meter distance	Velocity (m/s)	Area in m3	Discharge in m3/sec.	
Recording time	Time in hours	Depth in meter					
7:30:00 AM	0:00	0	0	0	0	0	
8:30:00 AM	1:00	0.87	27.3	1.432	2.61	3.74	
9:30:00 AM	2:00	0.83	32.1	1.218	2.49	3.03	
10:30:00 AM	3:00	0.7	39.7	0.985	2.10	2.07	
11:30:00 AM	4:00	0.51	48.8	0.801	1.53	1.23	
12:30:00 PM	5:00	0.33	62.3	0.628	0.99	0.62	
1:30:00 PM	6:00	0.2	88.1	0.444	0.60	0.27	
2:30:00 PM	7:00	0.06	188.2	0.208	0.18	0.04	
3:30:00 PM	8:00	0	192.7	0.203	0.00	0.00	
4:30:00 PM	9:00	0				0.00	

Annex 20: Peak Runoff and Total Runoff of each sub-catchment for the year 2009 & 2019

Sub. Cat.	LULC in %	Peak Runoff in CMS (2009-2019)				Total Runoff in M ³ (2009-2019)			
		2009	2019	Change in Peak Runoff	Change in Peak Runoff (%)	2009	2019	Change in Total Runoff	Change in Total Runoff in %
1	3.8	2.86	2.98	0.12	4.20	24560	24590	30	0.12
2	7.5	0.99	1.04	0.05	5.05	8560	8590	30	0.35
3	16.9	1.13	1.33	0.20	17.7	7880	7920	40	0.52
4	36	0.77	0.95	0.18	23.38	4970	5030	60	1.21
5	1.2	1.16	1.17	0.01	0.86	9080	9080	0	0
6	4.1	0.78	0.81	0.03	3.85	5400	5410	10	0.19
7	21.9	0.35	0.41	0.06	17.14	2070	2080	10	0.48
8	9	0.27	0.29	0.02	7.41	1440	1440	0	0
9	5.5	0.59	0.61	0.02	3.39	3320	3320	0	0
10	4.8	0.51	0.53	0.02	3.92	3000	3000	20	1.16
11	42.8	0.29	0.36	0.07	24.14	1730	1750	10	0.75
12	26.8	0.25	0.3	0.05	20.00	1330	1340	10	0.45
13	10.3	0.44	0.48	0.04	9.09	2230	2240	10	0.28
14	12.8	0.6	0.65	0.05	8.33	3600	3610	10	0.28
15	19.2	0.35	0.39	0.04	11.43	2120	2130	10	0.47
16	5.4	0.6	0.62	0.02	3.33	4340	4350	10	0.23
17	16.2	2.21	2.43	0.22	10.0	27110	27340	230	0.85
18	1.6	0.23	0.23	0.00	0.00	1380	1380	0	0
19	28.7	1.51	1.98	0.47	31.13	12570	12690	120	0.95
20	94	0.23	0.34	0.11	47.8	1550	1580	30	1.94
21	19	0.4	0.48	0.08	20.00	2950	2970	20	0.68
22	72	0.29	0.52	0.23	79.31	3060	3160	100	3.27
23	2.3	1.02	1.04	0.02	1.96	6750	6750	0	0
24	16.2	0.83	0.93	0.10	12.05	5140	5170	30	0.58
25	17.9	0.34	0.34	0.00	0.00	1560	1560	0	0
26	35.7	0.25	0.28	0.03	12.00	1210	1220	10	0.83
27	37	0.27	0.29	0.02	7.41	1300	1310	10	0.77
28	21.1	0.22	0.25	0.03	13.64	1710	1720	10	0.58
29	33	0.05	0.06	0.01	20.00	540	780	240	44.44
30	36.9	0.21	0.23	0.02	9.52	970	980	10	1.03
31	0	0.25	0.25	0.00	0.00	1340	1340	0	0
32	17.5	0.21	0.21	0.00	0.00	970	970	0	0
33	30.9	0.27	0.29	0.02	7.41	1400	1410	10	0.71
34	30.3	0.24	0.26	0.02	8.33	1240	1250	10	0.81
35	6.8	0.34	0.34	0.00	0.00	1770	1780	10	0.56
36	0	0.6	0.60	0.00	0.00	3750	3750	0	0
37	5.6	0.3	0.31	0.01	3.33	1610	1610	0	0
38	16.3	0.16	0.18	0.02	12.50	980	990	10	1.02
39	5.9	0.25	0.26	0.01	4.00	1140	1140	0	0
40	10.1	0.22	0.22	0.00	0.00	900	900	0	0
41	7.6	0.59	0.62	0.03	5.08	3410	3410	0	0
42	4	3.4	3.45	0.05	1.47	31830	31860	30	0.09
43	13.2	3.23	3.53	0.3	9.3	36290	36500	204	0.05
44	9.3	1.71	1.79	0.08	4.68	21010	21300	290	1.38
45	10.6	3.44	3.64	0.20	5.81	42990	43240	250	0.60
							G. Total	1884	

Annex 21: Peak discharge of each conduit for the year 2009 & 2019

Conduit	Peak discharge in CMS			
	2009	2019	Change	Change in %
D_1	0	0	0	0
D_2	0.352	0.352	0	0
D_3	0.323	0.323	0	0
D_4	0.319	0.319	0	0
D_5	0	0	0	0
D_6	0	0	0	0
D_7	0	0	0	0
D_8	1.475	1.466	0.009	0.62
D_9	0	0	0	0
D_10	0.318	0.318	0	0
D_11	0.442	0.442	0	0
D_12	0.236	0.238	0.002	0.85
D_13	0.484	0.484	0	0
D_14	0	0	0	0
D_15	0	0	0	0
D_16	0.256	0.256	0	0
D_17	0.039	0.039	0	0
D_18	0.02	0.02	0	0
D_19	0.02	0.02	0	0
D_20	0.335	0.351	0.016	4.78
D_21	0.355	0.355	0	0
D_22	0.31	0.31	0	0
D_23	0	0	0	0
D_24	0	0	0	0
D_25	0.256	0.256	0	0
D_26	0	0	0	0
D_27	0	0	0	0
D_28	0.327	0.367	0	0
D_29	0.267	0.267	0	0
D_30	0.244	0.244	0	0
D_31	0.019	0.019	0	0
D_32	0.193	0.193	0	0
D_33	0.107	0.107	0	0
D_34	0.123	0.123	0	0
D_35	0.044	0.044	0	0
D_36	2.21	3.189	0.979	44.30
D_37	2.202	2.298	0.096	4.36
D_38	0.266	0.266	0	0
D_39	0.297	0.297	0	0
D_40	0.405	0.405	0	0
D_41	0.4	0.4	0	0
D_42	0.205	0.205	0	0
D_43	0.23	0.23	0	0
D_44	0.23	0.23	0	0
D_45	0.23	0.23	0	0

Conduit	Peak discharge in CMS			
	2009	2019	Change	Change in %
D_46	0.63	0.641	0.001	0.16
D_47	1.295	1.297	0.002	0.15
D_48	2.82	2.821	0.001	0.04
D_49	0.3	0.3	0	0
D_50	1.01	1.032	0.022	2.18
D_51	1.656	1.674	0.018	1.09
D_52	2.189	2.222	0.033	1.51
D_53	0.275	0.275	0	0
D_54	0.109	0.109	0	0
D_55	0.049	0.049	0	0
D_56	0.23	0.237	0.007	3.04
D_57	3.442	3.849	0.407	11.82
D_58	0	0	0	0
D_59	0	0	0	0
D_60	0.357	0.357	0	0
D_61	0.219	0.219	0	0
D_62	0.4	0.404	0.004	1
D_63	0.229	0.313	0.084	36.68
D_64	0.209	0.209	0	0
D_65	0.121	0.121	0.001	0.83
D_66	0.089	0.089	0	0
D_67	0	0	0	0
D_68	0	0	0	0
D_69	0.3	0.3	0	0
D_70	0	0	0	0
D_71	0.275	0.275	0	0
D_72	0.318	0.318	0	0
D_73	0.23	0.23	0	0
D_74	0.243	0.244	0.001	0.41
D_75	0.089	0.089	0	0
D_76	0.017	0.017	0	0
D_77	0.042	0.042	0	0
D_78	0.201	0.201	0	0
D_79	0.033	0.033	0	0
D_80	0.058	0.058	0	0
D_81	0.157	0.164	0.007	4.46
D_82	0.183	0.183	0	0
D_84	2.236	2.284	0.048	2.16
D_84	0.375	0.597	0.222	59.2
D_85	0.785	1.138	0.353	44.97
D_86	3.4	3.453	0.053	1.56
D_87	0	0	0	0
D_88	0	0	0	0
D_89	3.291	3.534	0.243	7.38
D_90	3.44	3.636	0.196	5.70
D_91	1.138	1.147	0.009	0.79
D_92	1.357	1.357	0	0
D_93	1.332	1.332	0	0
D_94	1.447	1.449	0	0

Annex 22. List of flooded Junctions

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 Poned Depth Meters
J_3	4.14	0.979	0	00:30	7.735	0.000
J_9	2.95	0.528	0	00:30	2.099	0.000
J_10	3.03	0.827	0	00:30	3.462	0.000
J_11	3.68	2.664	0	00:40	18.878	0.000
J_12	3.18	0.458	0	00:20	1.991	0.000
J_13	0.78	0.031	0	00:20	0.058	0.000
J_18	3.07	0.837	0	00:20	4.024	0.000
J_20	0.64	0.310	0	00:20	0.412	0.000
J_21	3.65	0.584	0	00:27	5.014	0.000
J_22	1.49	0.720	0	00:30	1.801	0.000
J_28	0.42	0.099	0	00:20	0.078	0.000
J_29	1.43	0.525	0	00:30	1.210	0.000
J_30	5.87	1.953	0	00:30	12.279	0.000
J_31	4.74	2.261	0	01:12	21.430	0.000
J_32	0.60	0.145	0	00:20	0.180	0.000
J_41	1.34	0.225	0	00:20	0.498	0.000
J_45	3.12	0.507	0	00:50	3.890	0.000
J_46	1.03	0.569	0	00:30	1.143	0.000
J_53	2.92	0.422	0	00:27	1.924	0.000
J_54	0.15	0.059	0	00:20	0.013	0.000
J_56	0.02	0.710	0	01:12	0.016	0.000
J_60	2.94	0.688	0	00:40	3.812	0.000
J_61	3.51	0.817	0	00:30	5.227	0.000
J_62	3.35	1.198	0	00:40	8.668	0.000
J_67	23.72	3.453	0	01:00	31.732	0.000
J_69	23.75	3.534	0	01:00	36.450	0.000
J_71	23.66	3.636	0	01:20	42.880	0.000

Annex 23. Yearly Maximum Rainfall data (1997-2017)

No.	Year	Rainfall (RF) max. in mm	Log Rf
1	2007	38.8	1.588831726
2	2001	42.5	1.62838893
3	2011	44.3	1.646403726
4	2006	44.5	1.648360011
5	2000	44.8	1.651278014
6	2015	44.8	1.651278014
7	2002	46.8	1.670245853
8	2004	47.2	1.673941999
9	1997	54.2	1.733999287
10	2010	56.1	1.748962861
11	2005	56.3	1.750508395
12	2014	61.8	1.790988475
13	2013	62.8	1.797959644
14	2017	64.4	1.808885867
15	2009	73.2	1.864511081
16	2012	81.4	1.910624405
17	1998	83	1.919078092
18	2003	85.1	1.92992956
19	2016	91.6	1.961895474
20	1999	92	1.963787827
21	2008	94.9	1.977266212
	Average	62.405	1.777005974
	Standard Deviation	18.76953585	0.12805939
	Skewness		0.255324134