



**ASSESSMENT OF STORMWATER DRAINAGE SYSTEM IN
HAWELA TULA SUBCITY, HAWASSA CITY**

MSC THESIS

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HAWASSA UNIVERSITY, HAWASSA, ETHIOPIA

Hawassa Ethiopia

May, 2023

**ASSESSMENT OF STORMWATER DRAINAGE SYSTEM IN HAWELA
TULA SUBCITY, HAWASSA CITY**

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**A THESIS SUBMITTED TO THE FACULTY OF BIO-SYSTEMS AND
WATER RESOURCE ENGINEERING DEPARTMENT OF WATER
RESOURCE AND IRRIGATION ENGINEERING**

**HAWASSA UNIVERSITY
INSTITUTE OF TECHNOLOGY
SCHOOL OF GRADUATE STUDIES
HAWASSA UNIVERSITY
HAWASSA, ETHIOPIA**

**IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE
DEGREE OF MASTER OF
SCIENCE IN WATER RESOURCE ENGINEERING AND
MANAGEMENT**



Hawassa, Ethiopia

May, 2023

EXAMINER’S APPROVAL SHEET
SCHOOL OF GRADUATE STUDIES

HAWASSA UNIVERSITY EXAMINERS’ APPROVAL SHEET

We, the undersigned, members of the Board of Examiners of the final open defense by Sintayehu Betiso Berasa have read and evaluated his thesis entitled “ASSESSMENT OF STORMWATER DRAINAGE SYSTEM IN HAWELA TULA SUBCITY, HAWASSA CITY”, and examined the candidate. This is, therefore, to certify that the thesis has been accepted in partial fulfillment of the requirements for the degree of master’s science.

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Declaration

I, Sintayehu Betiso, declare that this thesis, titled submitted to the school of Graduate studies of Hawassa University in Partial fulfillment of degree of Masters of Science in Water Resource Engineering and Management, is my original work and that it has not been presented and will not be presented, in whole or in part, by me to any other university for similar or any other degree award. Additionally, I ensure that the work is original and to the best of my knowledge and has not been taken from other sources except where such works have been cited and acknowledged within the text.

Sintayehu Betiso

Hawassa, Ethiopia

May, 2023

Acknowledgement

Firstly, I would like to thank the almighty GOD for his priceless help and protection during whole activity of my work.

It is also important and right to acknowledge the support and help of various people who had contributed for successful completion of this study. First and for most , I would like to express my genuine gratitude and appreciation to my Advisor Dr. Abraham Woldemichael whose encouragement, guidance and support from the start to the end enabled me to develop an understanding of the subject matter very well.

I would also like to thank Shebedino Woreda for sponsoring my Masters study in Hawassa University.

In the period of doing my thesis, I have also received significant supports and nice welcome while collection of necessary data from, National Meteorological Agency (NMA), Tula town municipality and Hawassa city administration. I admire fulfillment of their responsibility and keep to do so for the other researchers

Thank you to all my families, friends and colleagues who lend a hand, directly or indirectly, for the completeness of my thesis work.

Abstract

The major goal of this study was to evaluate Tula Town's current storm water drainage system and to provide ideas or engineering solutions that would be able to resolve the current issues. Storm Water Management Model (SWMM), which is a dynamic rainfall runoff simulation model widely applied for urban drainage, was chosen for assessing the triggering factors as well. Due to insufficient drainage network infrastructure and a rise in urban pavement on most urban sub-catchments, this problem developed. The Rainfall depths of sixteen years (2006-2021) of Tula meteorology station were obtained from National Metrological Agency. An intensity duration frequency 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 other method. The study made use of the SWMM5.2 model and the low impact development control. The calibration and validation of the SWMM5.2 model well-done and its performance was tested by the goodness of fit using the coefficient of determination (R^2) =0.98, the Nash –Sutcliffe coefficient (NSE) =0.91, and Relative error (RE) =24.13%. The simulated area of the study subdivide to two outlets of the total area of sub-catchment is 70.55ha joint to drainage system infrastructure of 37 nodes, 48 channels, by SWMM 5.2 model. According to the model's results, more than 50% of drainage infrastructure is flooded. At the outfall, total sub-catchment runoff has an average flow of $6.0785\text{m}^3/\text{s}$, a maximum flow of $11.39\text{m}^3/\text{s}$, and an overall volume of $222.228 \times 10^3\text{m}^3$ as predicted by the SWMM5.2 model. The town's ongoing drainage issue is helped by the varied implementation of low impact development techniques. In general, the Tula Town urban drainage system evaluation infrastructure overflow studied and regulated for the problem occurred using the best alternative mitigation measures by enhanced low impact development management for all study sites.

Key words: Drainage systems, LID, Storm water, SWMM5.2, Tula town.

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LIST OF ABBREVIATION AND ACRONYMS

DEM	Digital Elevation Model
ERA	Ethiopian road authority
GIS	Geographic Information System
GPS	Geographical Positioning System
IDF	Intensity Duration Frequency
LID	Low Impact Development
MOUSE	Modeling of Urban Sewers
NMSA	National Meteorology Service Agency
RAFT	Runoff Analysis and Flow Training Simulation
SUDS	Sustainable Urban Drainage System
SWDM	Storm Water Drainage Manual
SWMM	Storm Water Management Model
SC	Sub catchment
UH	Unit Hydrograph

1. INTRODUCTION

1.1 Background

Urban infrastructure is one of the vital elements in the process of urbanization and the emergence as well as for the continuity of urban growth (Dibaba, 2018). Urban drainages are parts of urban infrastructures which seeks to avoid interruption to the free movement of pedestrians and vehicles, material damage from torrential flows, and danger to health, also harms environment from rainfall in urban areas (Silveira, 2002). This infrastructure is imperative and essential for the needs of interactions between human's civil activities and the water cycle within the area and even beyond an urban area. There are two types of urban drainage systems, these are: wastewater drainage system and storm water drainage system. Wastewater and stormwater are the two basic types of drainage flow to be drained in urban area (Qiao et al., 2018).

With urbanization, impermeability increases because of the increase in impervious surfaces. This is the drainage pattern changes, increases overland flow resulting in flooding and related environmental problems. The effect of this is severe on spatial structures like road. This is because, flooding and its related environmental problems like sheet and gully erosion, surface flood tends to affect road services and its life span (Adisu and Hailemikael, 2017).

As human settlements rose higher and higher, they quickly began influencing the natural hydrological processes. Ditches dug, fields were under-drained, streams straightened and rivers embanked in order to quickly take water from the land to the sea (Abraha, 2018).

Drainage problems in urban areas include deterioration of roads, land degradation, flooding, sedimentation, water logging, blockage of drainage facilities, and others (Dagnachew, 2009).

Major cities of developing countries mainly suffer from the localized problem of flooding and water pollution due to presence of inefficient, unscientific and weakly maintained drainage system (Abraha, 2018). Traditionally surface water that occurs due to rainfall or other activities in a built-up area was removed using underground-fabricated pipe systems, which designed and constructed to prevent local flooding by flushing the water away immediately. These drainage systems are designed for specific flow rate of water and thus they are unable capture the fluctuations due to change in the volume of the water. As a result, pollutants from urban areas washed into rivers or the groundwater (Abraha, 2018).

1.2 Statement of problem

Tula town is one of Ethiopian towns that experience heavy rain in every rainy season in the year that is the main reason for the flood to disturb the lives of the residents by entering in to their house and washing out their property after the rainfall and clogging of the roads in the town. Frequent flood during the rainy season has damaged the streets, drainage canals, residential and institutional houses and market area. The flood has also created depressions and gullies in some parts of the town.

In Tula, flooding occurs in a number of locations due to severe rain and substantial runoff. During the rainy season, a sizable area of the city is vulnerable to floods. Significant losses in human life and property have been caused by this. In some drainage areas, there is also an issue with overtopping, blocking of drainage systems, and water logging, which has a major influence on the road's strength and accessibility. Rainy season runoff can carry wastes with a foul odor that travel along the drainage system and create an unwelcoming environment.

Effective particular study must be undertaken to understand the causes of the observed problems and suggest specific solutions leading to improvement of the system.

1.3 Objectives

1.3.1 General Objective

The general objective of this study is to assess the storm water drainage system in Tula town.

1.3.2 Specific Objectives

- ❖ To assess the current condition of the existing drainage system.
- ❖ To estimate discharge rate of stormwater drainage in the area.
- ❖ Indicate possible remedial engineering solution for the drainage problem.

1.3 Research Questions

- ❖ What is the current status of storm drainage system in the town?
- ❖ What is the discharge rate of stormwater drainage in the study area?
- ❖ What are the improvement engineering options?

1.4 Significance of the Study

In general, maintaining urban stormwater drainage systems is essential for effective environmental management since it prolongs the useful lives of urban infrastructures including roads, buildings, telephone lines, water supply lines, and existing rivers. As a result, the advantages that come from this research could support present efforts by governments and other relevant bodies to address the issue of drainage plans that improve service delivery. The findings of this study may also help to close data gaps by identifying issues with sustainability, introducing low impact development option, taking adequate stormwater drainage system design, and ensuring the operation of drainage schemes in the municipality. The cities will also consult it as a resource when creating their annual plans for the urban drainage system.

This study offers the following essential uses

To reduce the environmental degradation and health safety problems, to overcome the problems caused by the improper drainage structure and sustain the development of the town, Additionally to minimize the possible damage of road pavement by flooding, aesthetic of town, through proper sustainable drainage structure provisions the least but not the last is to put the documented data for another researcher in the town for the further study if needed.

1.5 Scope and Limitation of the Study

This thesis is limited to the evaluation of stormwater drainage system on Hawassa city, Hawela Tula sub city, Tula town. The intended purpose is to suggest reduction measures for problems that are found on the existing drainage structures. The research included both hydraulic and hydrologic analyses for drainage structures that are susceptible to failure.

2. LITERATURE REVIEW

2.1 General Description of Urban Drainage

Drainage generally defined as technical action to lessen the excess of water from rainwater, seepage, or irrigation excess from certain region/land. Moreover, drainage as an effort to controlling the groundwater quality in relation to salinity, drainage is not only concerned about surface water but also about groundwater (Goonetilleke et al., 2017).

Urban drainage systems are designed to remove the rainfall which accumulates on relatively impermeable surfaces in towns and cities. This increases the speed of runoff and reduces the natural attenuation of the land surface. The runoff from an urban area may have a significant effect on the hydrograph of a natural water course which is receiving the runoff, since the later may not have responded to the rainfall. More attention is now being paid to increasing the attenuation of urban drainage system by the use of tanks and reducing the flows by promoting infiltration (Adisu and Hailemikael, 2017).

Urban drainage is divided to include two types of water: wastewater and storm water (Burian Steven and Edwards Findlay, 2002).

2.1.1 Stormwater

Storm water (surface runoff) is the major urban flow of concern to the drainage engineering. Safe and efficient drainage of storm water is particularly important to maintain public health and safety (due to the potential impact of flooding on life and property) and to protect the receiving water environment. Reliable data on the quantity and quality of existing and projected storm water flows is a prerequisite for cost effective urban drainage design and analysis (Butler and Davies, 2004).

2.1.2 Wastewater

Wastewater is water that is released after use for life support, industrial processes, or life enhancement must be collected and disposed of appropriately to prevent nuisances and polluted conditions from developing in urban areas.

In most of least developed countries, the bulk of domestic and industrial wastewater is directly discharged into water streams without any treatment processes or after primary treatment only (Dhote and Ingole, 2012).

2.2 Historical Development of Drainage Systems in the World.

Drains in the streets are known since the early Mesopotamian Empire in Iraq (*ca.* 4000–2500 BC). However, well organized and operated sewerage and drainage systems were practiced for the first time in the history of humankind by the Minoan and Harappan civilizations in Crete, and in the greater Indus valley, respectively after *ca.* 3000 BC (de Feo et al., 2014).

In the Europe urban drainage started in nineteenth century. In order to carry surface storm runoff, industrial effluent and domestic foul sewage the system were initially built in the central areas of large towns and cities. These drainage systems were later followed by the development of sewage treatment works, which were introduced to improve the quality of wastewater discharged in to local watercourses. However, in many cases, the capacities of these sewers and sewage treatment facilities were not sufficient to meet the demands of these growing cities. This led to the introduction of in-sewer detention tanks, and overflow structures to discharge excessive storm flows directly into receiving waters, without having to pass through the treatment works (Andrew, 2002).

The relation of modern engineering to ancient engineering is difficult to understand considering that modern engineering is so highly specialized and technologically advanced. Design rules of thumb, physics, empirical equations, computer simulators, numerical methods, and other engineering tools taken for granted today were not available to ancient engineers. Despite the supreme technological advantage today's engineer has over the ancient engineer, fantastic engineering feats rivaling those of today were achieved throughout history. For example, several ancient civilizations built magnificent cities of brick, wood and equipped the cities with sophisticated infrastructure systems including roads, water supply and distribution systems, wastewater collection, and storm water drainage.

Both wastewater and storm water must be considered during urban drainage system planning.

2.3 Modern Urban Drainage Development

Technologies have been developing for handling urban drainage over a long period of time, even though design criteria have been relatively constant throughout the major urbanization era. Since the fifties, urban drainage recommendations have been to separate storm water which is caused by rain and snow melting and from wastewater which is caused by households in the sewer systems (Berggren, 2007).

Many urban sewer systems are often partly combined, especially in city centers where it is also more expensive to rebuild and replace pipes.

In surface water run-off, a watercourse, river, sea, or ground infiltration system, urban drainage has basic function which is removing the sewage with rainwater run-off from buildings, roads, and other impermeable areas, then transport it to every part of sewage treatment works (Zhou, 2014). Urban drainage presents a classic set of modern environmental challenges: the need for cost-effective and socially acceptable technical improvement in existing systems, the need for assessment of the impact of those systems, and the need to search for sustainable solutions (Butler and Davies, 2004). Butler and Davies (2004) stated there are two types of sewer that is accommodated by urban drainage, wastewater and storm-water.

2.4 Urban Storm Drainage System in Africa

The increasing urbanization of the world's population is constantly creating new challenges for stormwater management. Although rain is vital for both human beings and their environment (to replenish rivers, water points and groundwater, grow vegetation, etc.), rainfall events generate flows and volumes of water that can be difficult to control and that accumulate in the lowest parts of towns, flooding residential areas and creating pools of stagnant water (Burkhardt, 2013).

The population of many of sub-Saharan Africa's larger cities increased more than seven fold between 1950 and 1980 Nairobi, Dar es Salaam, Nouakchott, Lusaka, Lagos, and Kinshasa among them. During these same 30 years, populations in many Asian and Latin American cities (such as Seoul, Baghdad, Dhaka, Amman, Bombay, Jakarta, MexicoCity, Manila, Sao Paulo, Bogota, and Managua) tripled or quadrupled.

In such cities, net immigration has usually been greater contributor than natural increase to the population growth of recent decades (WCED, 1987).

In developing countries, urban storm water management is generally inadequate or nonexistent. Where they exist, storm water collection systems serve only the most central or wealthiest areas. Due to a chronic lack of care and maintenance, they are in extremely poor condition. In outlying neighbor hoods, water runs along natural ravines, but these are not able to evacuate all surface runoff during heavy rain. This water collects pollutants (wastewater, trash, sediment) and constitutes a significant public health, economic and environmental risk.

Stormwater management is a highly complex process, the effectiveness of which is influenced by a number of different factors (Burkhardt, 2013).

2.5 Urban Storm Drainage System in Ethiopia

In the case of Ethiopia, where watersheds of many urban centers receive significant amount of annual rainfall and where rainfall intensity is generally high, control of runoff at the source, flood protection, and safe disposal of the excess water/runoff through proper drainage facilities are very critical task (NUPI, 2000).

In Ethiopian urban areas drainage problems include flooding, deterioration of roads, land degradation, sedimentation, blockage of drainage facilities, water logging (Dagnachew, 2009).

Ethiopian cities and towns at large, are troubled with storm water leading into floods especially during the rainy season which is due to inadequate installation of drainage infrastructure, poor maintenance of existing drains. The problem is very crucial in highland cities like Addis Ababa, Adigrat; although it also exist in cities with flat & plain geographies like Hawassa, Assosa, Adama, Bahir Dar, Woliso. The provision of operational drainage system for flat and highland areas is important from different point of views (Kokeb, 2016).

Ethiopia is a country with poor urban stormwater drainage experience and the drainage management is poor as well. Some experience and realities in Ethiopia are mentioned below (Dagnachew, 2009).

- The sedimentation and the flooding problem in Hawassa Town and its lake resource are obvious problem which has been occurring due to inadequate urban storm water drainage provision and management.
- The flooding problems which is occurring in every year in various parts of Ethiopian cities especially Addis Ababa, which leads to congested traffic movement, property loss, soil and water degradation and other infrastructure, particularly around “Filwoha”, National bank of Ethiopia, Addis Ketema Comprehensive Secondary School, general vegetables market and so on.
- Soil erosion and the land sliding problem in Dessie and Adigrat Towns and the darkest and unforgettable time in Diredawa because of serious flooding event in 2006.

2.6 Problems of Urban Stormwater Drainage System

The well-known reasons for failure of stormwater drainage system are the inadequacy of drainage structures during the rainy season to pass the flood, poor quality construction, inappropriate site selection and improper alignment of some drainage structures with respect to the road alignment. These shortcomings cause damage to superstructures of drainage structures and stream crosscurrents. Improper alignment of drainage structures with respect to the natural channel and the roadway can greatly aggravate the magnitude of scour and failure. The storm water can be caused by natural as well as anthropogenic activities (Tadele, 2017).

The main reasons for the problem of urban storm water drainage systems are stated below Berggren, (2007) stated that, the urban water systems are somewhat complex, even though the design criteria are relatively straightforward.

ROADEX: - is an EU-funded collaboration of Northern European road organizations from Finland, Iceland, Ireland, Norway, Scotland and Sweden that aimed to improve the condition of rural road networks. The project started as a pilot in 1998 to share best practice, to research and develop new knowledge, and to implement and test new solutions; it is still active, as the ROADEX Network.

The ROADEX project has grouped the common drainage problems on low volume roads into three main categories; a) maintenance related, b) design related and c) other specific problems. Ground conditions, landscapes and climates vary significantly across the Northern Periphery area but the problems encountered are basically the same.

2.6.1 Maintenance Related Drainage Problems

The “maintenance related” category covers all of those drainage problems that can be avoided by good maintenance policies and practices. Currently most drainage maintenance works in the Northern Periphery are carried out by contractors and as a result drainage descriptions in the contract procurement documents have become highly critical issue.

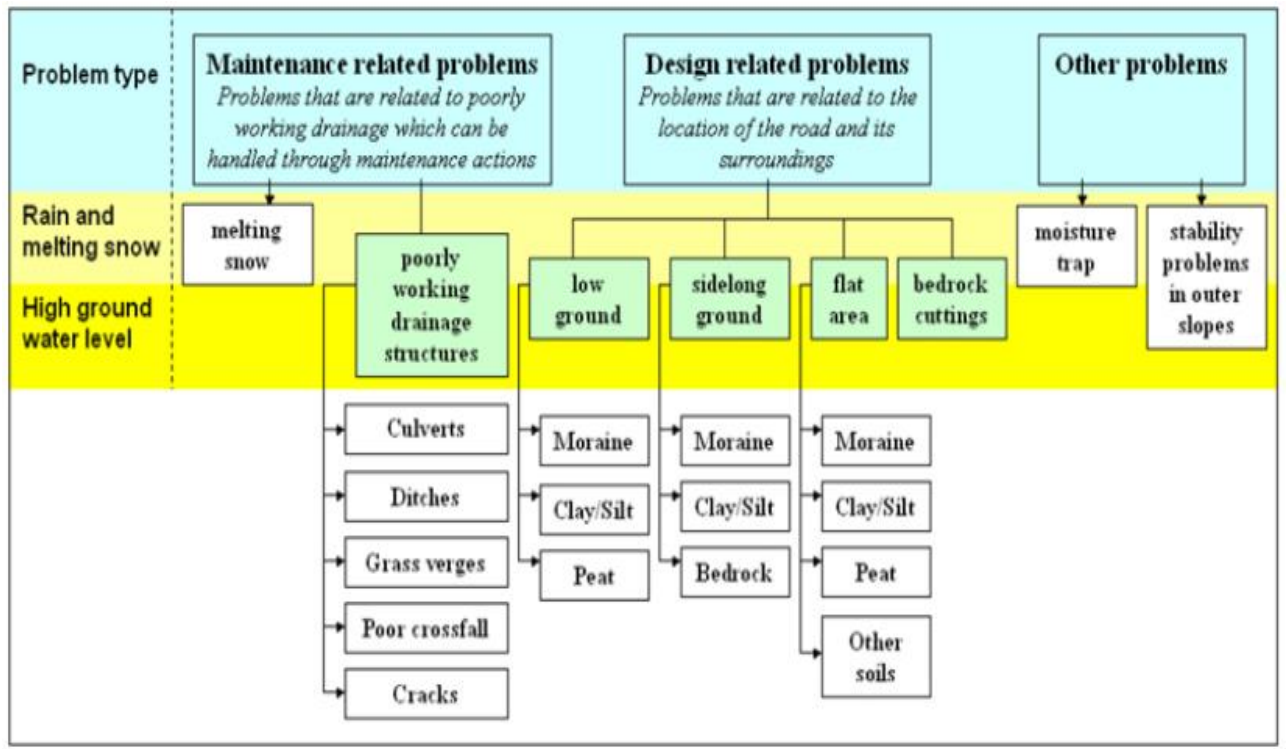


Figure 3.1: problems observed in drainage system

Poor drainage maintenance can have a major effect on the lifetime of a pavement and annual paving costs. It can also affect traffic safety. For this reason accurate definitions and requirements for each drainage maintenance task should be included in the contract documents, and followed up on site to ensure that the contractor fulfills its duties.

2.6.2 Seasonal Drainage Management

A number of drainage maintenance operations are directly linked to the seasons and especially to the winter and spring periods. These operation require good continuous monitoring of weather conditions, and a maintenance contractor with a good "touch" for operational timing to prevent road damages occurring, and prolong the lifetime of the pavement. Neglecting such maintenance works can have a major effect on annual paving costs. A good bonus system for contractors doing good work in these operations, as well as stiff sanctions for failing to deliver performance, would be a profitable option for road owners.

2.6.3 Maintenance Problems with Ditches and Sub drains

Wet soil around ditches is unstable and can flow to the bottom of the ditch raising its level. Vegetation likes the moist conditions in the bottom of a ditch and in time can fill the ditch if not removed early enough. One consequence of this is that the groundwater table will rise and moisture content will increase in the road structure. This will reduce the bearing capacity and

the road will begin to suffer from frost action. ROADEX research has shown that even a small blockage in the water flow in a side ditch can be reflected as differential frost heave in the adjacent road and further as shoulder deformation and pavement cracking.

The main issue in the maintenance of ditches and sub drains is the lack of cleaning. The policy in all ROADEX countries is that when the ditches are filled to a certain level (higher than 0.3 m from the bottom of road structures) the ditch must be cleaned out. However this operation usually has a low priority in road repair measures and often a new pavement has been the easier choice. Unfortunately this means that engineers have been dealing with the symptoms rather than addressing the underlying reason for the problems, with the result that the same problems appear after a few years. The need for regular ditch clearing is greatest in the areas of fine graded subgrade soils. These same soil types also have a low bearing capacity at high moisture content and it is therefore necessary to have a well working drainage system to keep them dry.

2.6.4 Runoff Maintenance, Keeping the Pavement Surface Dry

All of the water on the pavement surface should flow away from the road, and not infiltrate to the road structure beneath, to ensure a durable and long lasting pavement. This is achieved by providing a proper cross fall on the pavement surface and by keeping the bituminous pavement water impermeable. In gravel roads the wearing course should be as impermeable as possible, whilst ensuring that it does not turn plastic when it gets wet. The cross fall of road should be sufficient to ensure that surface water is run from the pavement during the rain. A well designed crossfall is essential for keeping the road surface dry. The crossfall of road should be sufficient to ensure that surface water will run from the pavement during the rain. A well designed crossfall is essential for keeping the road surface dry.

2.6.5 Topography Related Drainage Problems and Solutions

The ideal criteria for selection in this procedure is to determine the topography surrounding the road and then select and build the drainage solution based on that topography classification.

Designing and maintaining drainage in bedrock cuts can be difficult. The most major drainage issue with roads crossing a "flat and even" area is that the nearest natural drainage system is usually rather far away. One lane of a road built on side sloping terrain is generally in cutting, while the other lane is built on embankment. A wet and low lying valley is a special case, and a place where permanent deformation is often only one of many problems that have to be considered. Other issues which may arise include settlements, differential frost heaves

(especially around culverts) and rainy season maintenance considerations. Drainage design and maintenance in bedrock cuts is a challenging task. The greatest drainage issue with roads crossing a “flat and even” area is that there is usually a significant distance to the nearest natural drainage system.

2.6.6 Pollutants and Nutrients

Whose origin can be urban activities, industries, and farming, can cause problems in treatment processes and in the receiving waters, which also might affect the drinking water sources. These problems can be summarized as technical and environmental and they can also be intensified due to climate change, and more intense rainfall events (Berggren, 2007).

Socio-economic factors in developing countries make it more difficult to solve problems of urban drainage than in countries that are more advanced (Silveira, 2002).

2.7 Sustainable Urban Drainage Systems (SUDS)

2.7.1 Concepts of Sustainable Urban Drainage

For the past two decades, sustainable urban drainage system have become widely known when referring to urban runoff management because of the benefits they bring to the environment and quality of life in cities against low development impacts ((Zhou, (2014)) (Saurí and Palau-Rof, (2017); and (Ballard et al., (2015))).

Sustainable urban drainage systems are defined as surface water drainage systems inspired in the concept of sustainable development able to mitigate numerous adverse effects of urban stormwater runoff in cities (Ballard et al., 2015). The most convincing definition of sustainability, from the Brundtland Report WCED, (1987) states that “sustainable development is that which meet their own needs”. Therefore, sustainable solutions must include components of achieving goals “now” whilst not neglecting needs of “the future”. However, locational peculiarities and specific requirements associated with different stages of development make exact definitions difficult and generally inappropriate in most cases.

2.7.2 The Effect of Urbanization on SUDS

Alteration of natural drainage basin, either by the impact of urbanization, forestry, or agriculture, can impose radical changes in the movement and storage of water (Booth, 1991). 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. Due to

this impermeabilization, there is a reduction in rainwater infiltration, leading to a strong increase of rainwater runoff. It also increases the concentration per unit outlet.

Reduction in pervious area is one of the effects of urbanization, among others, which limits the natural infiltration and evaporation processes in urban areas. Urbanization coupled with population growth, climate change, and aging infrastructure has pressed the capacity of combined sewer systems to their limits (Nilsen et al., 2011).

2.7.3 Urban Stormwater Management

Stormwater management is concerned with the collection, conveyance, storage, treatment, and disposal of stormwater runoff in a manner that minimizes driven channel erosion, greater flood damage, and/or degradation of water quality, and in a manner that enhances and ensures public health, safety, and general welfare, according to the urban Stormwater drainage design manual, which shall include a system of vegetative or structural measures, or both, that control the increased volatilities (MoWUD, 2008).

Urban storm water management simply stated as everything done within a catchment to remedy existing storm water problems and to prevent the occurrence of new problems. This involves the development and implementation of a combination of structural and nonstructural measures to reconcile the conveyance and storage function of storm water systems, with space and related needs of expanding urban populations. It also involves the development and implementation of a range of measures or best management practices to improve the quality of urban storm water runoff prior to the discharge into receiving waters (Kokeb, 2016).

It has been argued that since it may never be possible to provide a completely perfect urban drainage system, resources would be better spent on identifying the least sustainable elements of conventional drainage systems and seeking to replace them with more operative alternatives. The benefits of such an 'incremental response' to the 'sustainability' question are that it avoids major economic, technical and logistical problems, which would otherwise be incurred by making dramatic changes to existing pieces of urban infrastructure (Butler and Parkinson 1997).

2.8 Urban Drainage System Modeling

Models are just simplifications of the real condition that is going to be simulated or analyzed. Modeling in urban drainage system serves various purposes such as the overall assessment of drainage area response as a part of strategic and master planning to the detailed network and

providing necessary support to primary activities such as elements design, assessment of pollution, operational management, real time control and analysis of interactions among subsystems (Siragi, 2019).

Urban storm water models have been applicable for usage such as operational tools. In spite of that, they are more frequently used as either planning or design tools. There are two basic modules of an urban storm water model which are in use as described by Zoppou (2001).

- i) **Rainfall runoff modeling** – The creation of surface and sub-surface runoff from rainfall, wash-off and build-up of pollutants from impervious surface.
- ii) **Transport modeling** - Routing of flows and pollutants through the storm water infrastructure, for example storages, open channel and pipe network

2.8.1 Hydraulic Model and Design

Urban flooding is two vital hydraulic ideas that arise: surcharge and backflow. A drainage structure is surcharged while its potential is passed, i.e. it gets extra extent of water than the gadget can carry. As an end result, the water stage rises upstream due to the community overloading Ericson, (2010).

Urban areas in rainy climates are commonly exposed to flood threats. In lowland areas urban flooding is often associated with an inadequate drainage system, in most cases due to inadequate inlet capacity. Urban flooding may cause material losses in the form of public and private property damage and even human casualties. To identify critical urban flooding areas so that appropriate measures for flow control and drainage can be taken, hydrologic and hydraulic modeling is often required. In many instances hydrologic and hydraulic modeling could involve large urban catchments making it necessary to increase computational efficiency without sacrificing accuracy. For a flood modeling tool to be practical, the flooding analysis needs to be done in a relatively short time frame so that flash flood warnings can be issued and traffic redirected to reduce emergencies and facilitate the work of first responders (Nanía et al., 2015).

2.8.2 Hydrologic Modeling

The term hydrology can be regarded as an essential topic for individuals and their surroundings. It deals with the occurrence, circulation, and distribution of earth's water, as well as its chemical and physical qualities, as well as its interaction with the environment, particularly its relationship with living organisms (Ray, 1975).

The hydrologic model transforms rainfall to runoff considering both impervious and pervious surfaces and simulates the infiltration process with the Green-Ampt method (Nanía et al., 2015).

2.8.3 Flood Frequency Analysis

The goal of frequency analysis is to examine the occurrence of a hydrologic variable using measured data and basin forecasts based on statistical laws within a statistical framework (ERA, 2013).

Standard statistical procedures for flood frequency analysis are based on the assumption of stationarity; that is, the extreme flood statistics do not change in time, and hence past observations can be considered as representative of future observations and thus used to estimate design flood events. In a changing environment the assumption of stationarity might not be applicable, and more advanced statistical methods are required that explicitly accounts for the non-stationarity of extreme flood characteristics (Center for Ecology and Hydrology, 2013).

2.8.4 Intensity Duration Frequency (IDF) Curve

The IDF-curves for rainfall tells the relationship between the rainfall intensity, the duration, or more precisely the aggregation time, and the frequency of an event. This information is required for urban storm drainage system design (ERA, 2013).

2.9 Models for Urban Stormwater Drainage Overview

An overview of some of the different urban Stormwater Drainage models is given below.

2.9.1 SWMM

According to Rossman (2010) the Environmental protection Agency (EPA) Storm Water Management Model (SWMM) is a dynamic rainfall-runoff simulation model that may be used for single-event or long-term (continuous) simulations of runoff quantity and quality in metropolitan environments. SWMM's runoff component works with a collection of sub catchment sites that receive precipitation and produce runoff and pollutant loads. SWMM's routing component transfers runoff through a network of pipelines, channels, storage/treatment devices, pumps, and regulators. During a simulation period comprised of numerous time steps, SWMM tracks the quantity and quality of runoff generated inside each sub catchment, as well as the flow velocity, flow depth, and quality of water in each pipe and channel.

2.9.2 PCSWMM

PCSWMM is a third-party interface for SWMM and is developed by Computational Hydraulics Inc. (CHI). It is a GIS integrated model that uses SWMM 5.0 as the model computational engine for hydrologic and hydraulic calculations. It is a stand-alone modeling tool with all necessary IS tools included and it has support for various CAD and GIS formats (Lind, 2015).

2.9.3 XPSWMM

XPSWMM is another hydraulic and hydrologic modeling tool from extreme programming (XP) solutions and has been used for analysis, design and simulations for over 25 years. Like XP Storm, XPSWMM includes stormwater and river systems/floodplains. Additionally, it also includes wastewater management. The model simulates 1D network flows in combination with 2D overland flows, LID structures and stormwater quality. The tool can be used for natural systems like for example ponds, rivers and lakes and manmade environments like pipes, conduits and streets (Lind, 2015).

2.9.4 RAFTS

RAFTS stand for Runoff Analysis and Flow Training Simulation and it has been used in Austria, simulates runoff hydrographs at defined points throughout the catchment for specific rainfall events (both observed and design). It is suitable for the modeling of catchments ranging from rural to urban. The model is capable of analyzing catchments comprising natural waterways, formalized channels, pipes, retarding and retention basins, and any combination of these. There are no specific limitations on the catchment size. It has been successfully used for on-site detention and on catchments up to 20,000 km² (Software, 1991).

2.9.5 HEC-HMS

Hydrology Engineering Concept – Hydrology Modeling System (HEC-HMS) – Version 4.0 is simulation software developed by HEC-corps of engineers of the United States Army to determine the behavior of a watershed for predefined rain events. This is single time event – deterministic model. The software has graphical user interface (GUI's), integrated with hydrological components and specific system of data storage system (DSS) and various management tools etc. In this model; infiltration, evaporation and interception processes determine from loss component while runoff processes are computed as the pure surface routing using transform component with various precipitation methods and their characteristics (Pujara et al., 2015).

2.9.6 MIKE URBAN

It is a versatile system for modeling and designing water distribution networks as well as waste water and stormwater collection systems. MIKE URBAN is built on a database that stores network and hydraulic modeling information. GIS-based model construction and management tool, integrated water quality, fire flow, real-time control, and water hammer simulation are all included in the software. It is possible to do thematic mapping and integrated dynamic result visualization (Butts and Graham, 2005).

2.10 Low Impact Development (LID) Controls

LID is essentially a stormwater management approach that involves improving the land. The basic goal of LID is to retain a site's predevelopment hydrology by decentralizing micro-scale controls, hence reducing the negative effects of precipitation flooding waters (Demisu, 2020). Instead of letting rainwater to run off into ditches and storm drains where it would otherwise contribute to flooding and pollution issues, LID practices often retain rainwater and encourage it to soak into the ground. The common LID techniques used to generate a functionally equitable hydrologic landscape include bio-retention, unseasoned roofs, permeable pavements, rain gardens, vegetative swales, and rain containers (Kong, 2017).

LID practices effectively reduce water-related problems through infiltration and evaporation of the storm water resulting environmental, social, and economic benefits. The common LID practices are bio-retention, green roofs, permeable pavements, rain gardens, vegetative swales, and rain cisterns (a.k.a. rain barrel) that are used to create a functionally equivalent hydrologic landscape (Eyosias, 2018). Few examples of LID for urban stormwater management are summarized below.

2.10.1 Infiltration Trench

A shallow, excavated trench that has been joined with a geotextile and backfilled with stone to create an underground reservoir is called an infiltration trench. Regular subsoil infiltration occurs as a result of stormwater runoff into the ditch. In the event of exceptionally heavy rains that surpass the reservoir's capacity, an overflow may be necessary. The permeability of the soil and the depth to the water table has a significant impact on the trench's overall performance. When used in conjunction with other source control techniques, infiltration trenches typically serve small catchment areas of up to 2-3 hectares. The more powerful they will be, the closer they may be to the runoff's source. By providing a filter strip, gully, or

sump pit to dispose of excessive solids as a pre-remedy for the inflow, the trench's operational viability may be improved. For the most effective pretreatment schemes, regular maintenance is necessary (Demisu, 2020).

2.10.2 Bio Retention Cell

Engineered, depressed landscape area designed to capture and filter or infiltrate the water quality capture volume (Adane, 2019). Water ponds most heavily at a height of 0.2 meters above the soil's surface. The building must have an additional freeboard of at least 5 centimeters over the water's maximum intensity, consistent with the same supply. Because there is not much room in Xiamen's strategic locations, vertical elements are intended to keep standing water in the bio-retention area (Laddimath et al., 2016).

2.10.3 Vegetative Swale

A vegetated Swale is a wide, shallow, trapezoidal or parabolic channel that is heavily vegetated with a variety of trees, shrubs, and/or grasses. It is intended to attenuate and, in some cases, infiltrate runoff volume from adjacent impervious surfaces, allowing some pollutants to settle out in the process. In steeper slope situations, check dams may be used to further enhance attenuation and infiltration opportunities (Pennsylvania Dept of Environment protection, 2006).

2.10.4 Green Roofs

The management of rainwater that collects on building rooftops can be decentralized with the use of green roofs. It can lessen the amount of contaminants transferred to water bodies through runoff, delay and lower peak stormwater runoff flow rates, and aid to minimize runoff volume. Green roof installation offers a decentralized urban stormwater management alternative in densely populated regions because it doesn't call for extra land. Additionally, green roofs improve urban quality of life visually (Zinabie, 2018).

2.10.5 Permeable Pavement Systems

A better alternative to normal paving is permeable pavement, which allows water to pass through the construction instead of dripping off. This feature needs to be taken into consideration when designing each floor as well as the sub-grade. Where conditions are right, water may be permitted to penetrate directly into the subsurface. Instead, it can be stored in the form of a reservoir beneath the paving for later use, infiltration, or discharge. The porous asphalt, grass Crete, gravel, or specially engineered concrete blocks may be used to create the permeable paving (Demisu, 2020).

3. MATERIALS AND METHODS

3.1 Description of the Area

3.1.1 Geographical Location

Hawassa city is found in the Sidama national regional state. The city is a regional administrative capital as well as an important business center for the region. Moreover, it is also an important tourist destination site in the region whereby a number of foreign and domestic tourists visited the city. The town is located at a distance of 273 km south of Addis Ababa. The geographic coordinates of the town are; approximately 7.05 latitude north and 38.48 longitudes east (Adane, 2019).

Specifically the study area Tula found in location of Latitude: 06⁰ 57' 28.9" North, Longitude: 038⁰ 28' 29.7" East, Altitude: 1883.00 m at the distance of 8 km south of Hawassa city.

In different literatures the area of the city is reported differently. According to Wondrade et al., (2014) the Hawassa City Administration covers an area of 16062 ha of which 6,465 ha is municipal boundary and is sub-divided into eight Sub-Cities and 32 “Kebeles”.

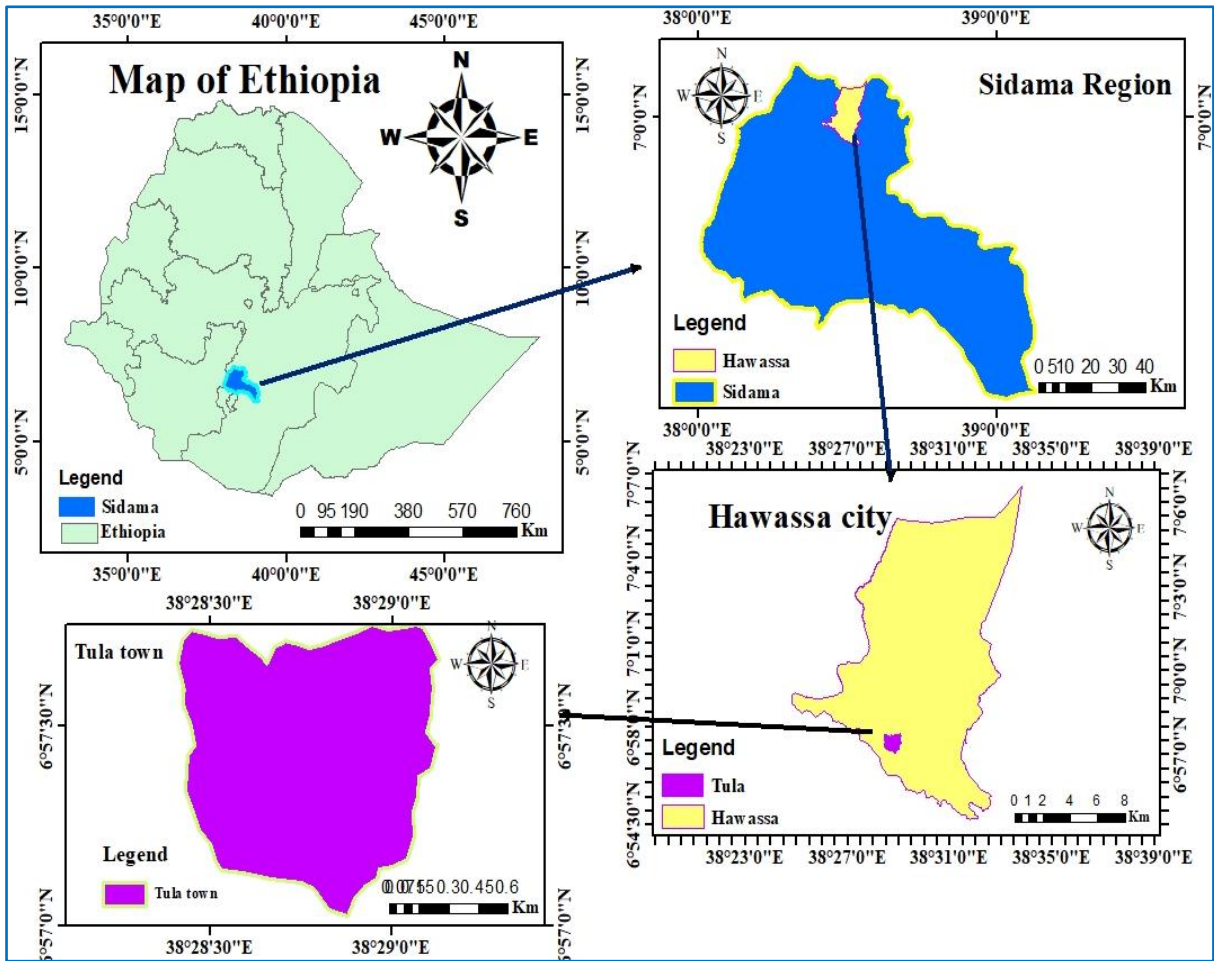


Figure 3.2: Location map of Hawassa city

3.1.2 Topography of the City

Hawassa city is situated at the eastern shore of Lake Hawassa close to eastern fault-belt of the central part and the great Ethiopian rift valley in a large volcano-tectonic collapse. The city is located on a plain between Lake Hawassa and Chelelaka wetland with general slope towards lake Hawassa. The average elevation of the town is 1700 a.m.s.l and that of the lake surface is 1680m above the sea level. The drainage of the town is towards the lake Hawassa (Adane, 2019).

3.1.2.1 Slope Analysis

The choice of a preferred landscape for urban growth and the determination of various urban applications in urban centers are both aided by slope analysis.

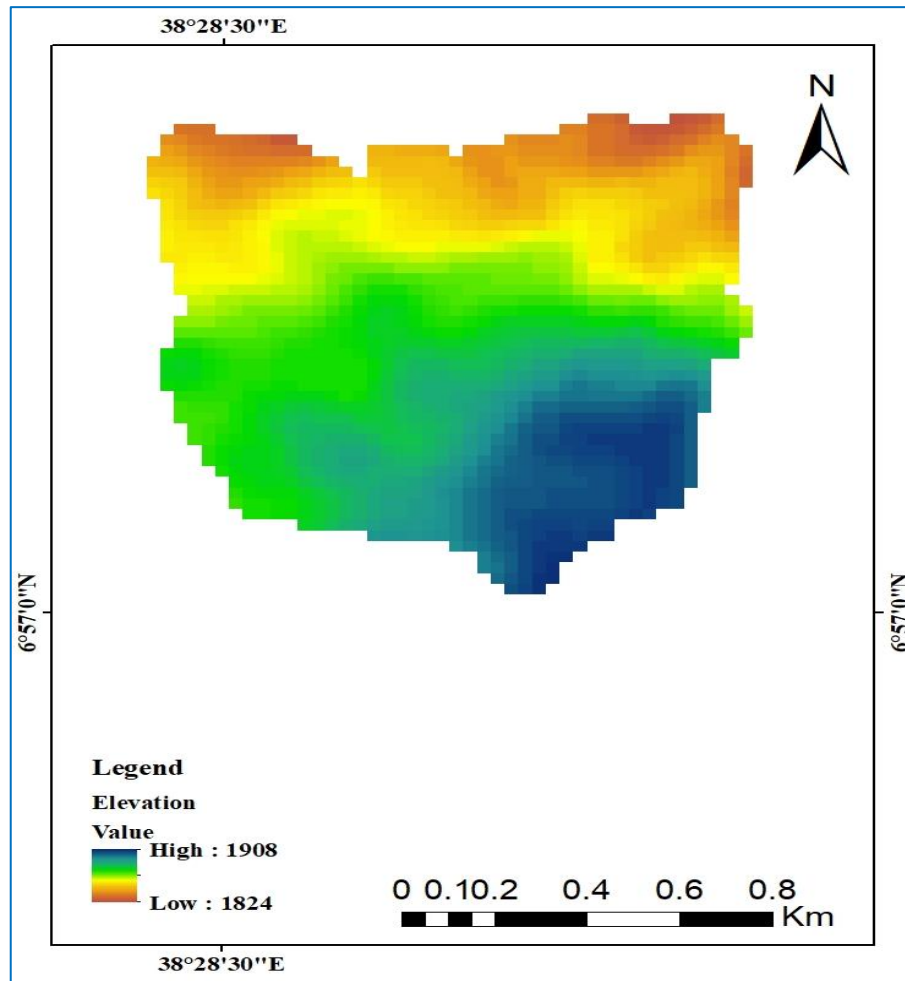


Figure 3. 3 Digital elevation model (DEM) of Tula town

Table 3.1 Slope classification and area covered

Slope classification (%)	m ²	Hectare	Percentage (%)
0-3	33340	33.34	27.8
3-6	61090	61.09	50.9
6-9	20870	20.87	17.4
>9	4500	4.5	3.7
Total	119800	119.8	100

3.1.3 Population

In 2017, Hawassa projected a total population of 457,231, with annual growth rates of 4.8 and 2.8 percent in urban and rural areas, respectively. The city's residents are heterogeneous both in ethnic and religion (Kinfu et al., 2019).

3.1.4 Land Use of the City

An alarming rate of urbanization is occurring in Tula. Since 2006, there has been an increase in imperviousness because of all the built-up regions. The constructed areas grew by almost 234.5% between the years of 1987 and 2011 according to a recent study (Wondrade et al., 2014).

3.1.5 Climate of the Study Area

The climate is warm and temperate in Hawassa. The summers are much rainier than the winters in Hawassa. The temperature here averages 19.2 °C. The rainfall here is around 1124 mm per year. The warmest month of the year is March, with an average temperature of 21.6 °C. The lowest average temperatures in the year occur in July, when it is around 17.8 °C.

3.1.5 Socio-economic Activities

There are large and diversified modern investments activities such as hotels and tourism, social services, real estate development, small and large industries including Hawassa industrial park and constructions in the town. There is one university with additional three independently located faculties, one teachers training institute, one Health College, two TVET centers, and a number of private health, engineering and other educational colleges; and schools (Wondrade et al., 2014).

3.2 Methodology

3.2.1 Data Collection Method

This study entails gathering both primary and secondary data, as well as information from relevant organizations. Primary data was collected from personal field observation/site investigation and Google earth data with the help of a base map and check list. Secondary data was collected through literature studies and document analysis. Several printed books, journals and manuals were used; in addition to that, internet was the major instrument to access government documents and different journals. The major collected secondary data includes.

Table 3.2: Data collection types and sources

Type of data	Data source
Meteorological data	National meteorological service agency SNNPR
Master plan and road network	Tula town municipality
Digital elevation model(DEM)	Ministry of water and irrigation and Electricity Addis Ababa

3.2.1.1 Data Collection and Material Used

- ✚ **ARC-GIS:** to analyze the spatial data and delineate the catchments of the study area.
- ✚ **SWMM:** is a dynamic rainfall-runoff simulation model used for single event or long-term (continuous) simulation of runoff quantity and quality from primarily urban areas.
- ✚ **Microsoft Excels:** to calculate the runoff on the excel spreadsheet and prepare a comparison graph between the capacity of the existing drainage system and estimated runoff.
- ✚ **Tape meter:** - to determine the length, depth, and width of existing stormwater drainage lines in order to determine the drainage system's capacity.
- ✚ **Google Earth:** - to determine the elevation and coordinates of drainage structure stations and to select alternative model areas.
- ✚ **Rainfall data:** - is gathered from Ethiopia's National Meteorology Service Agency (NMSA). The data recorded period is from 16 year (2006 – 2021 G.C) and used to analyze peak runoff.
- ✚ **DEM data:** - Spatial data such as Digital Elevation Model (DEM) of 20m*20m data is used as an input data for ARC-GIS software for catchment delineation and estimation of catchment characteristic including area, slope and length of the natural drainage networks' terrain.
- ✚ **Shape file data:** - is used as an input for ARC-GIS software for catchment delineation and estimation of catchment characteristic.

3.2.2 To Map the Current Status of Drainage System

A field survey have conducted to measure the dimensions of the existing drainage structures (depth, width, and slope) to determine the velocity of flow and the maximum discharge passed through the existing drainage system if there is no recorded designed data about the dimensions, location, and slope of the existing stormwater drainage systems. The manning roughness's, hydraulic radius, slope, and area of the channel is all played a role in determining flow velocity. Then, using Manning's formula, the discharge capacity of stormwater carried by the current drainage system is determined.

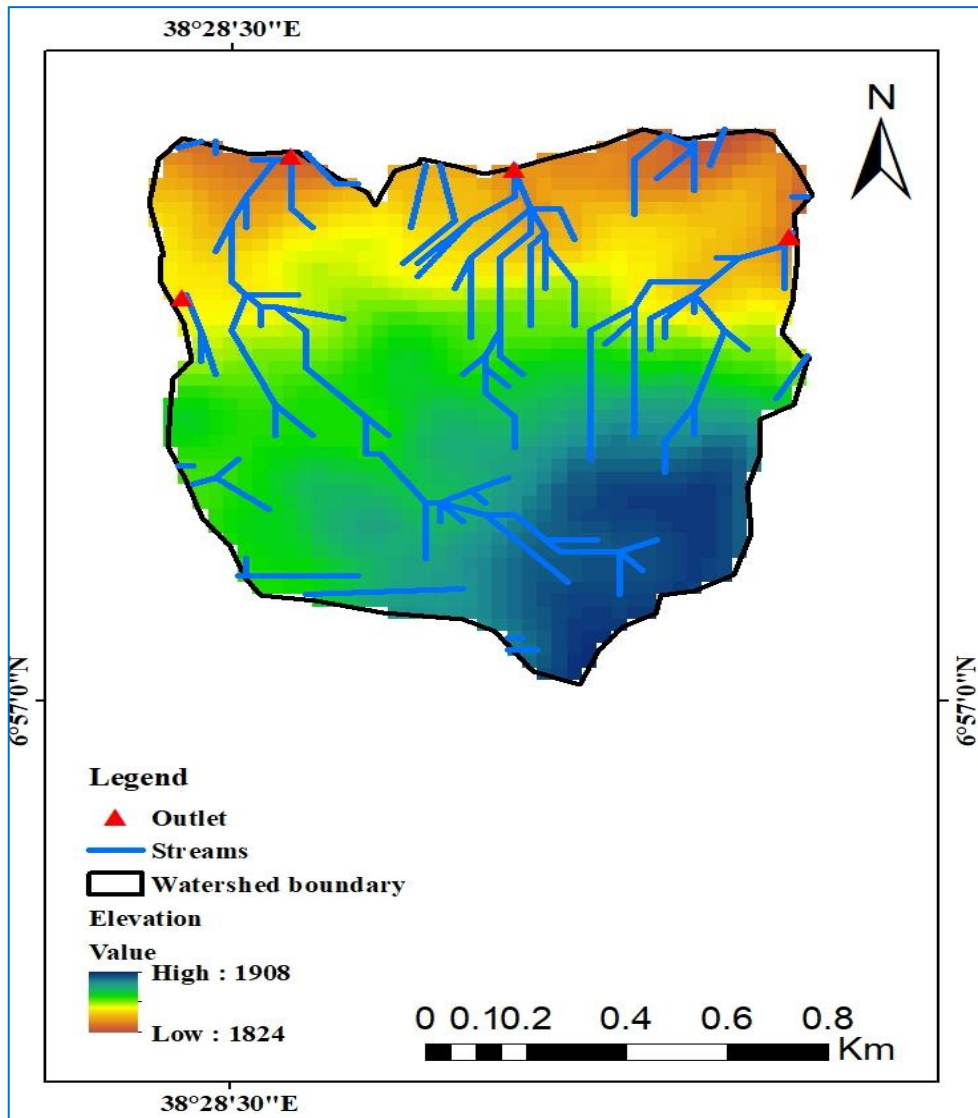


Figure 3. 4: The existing drainage networks of Tula town

3.2.3 To Estimate the Stormwater Rate in Selected Section of the Structure.

For the flow of a specific depth in a pipe of known diameter or stream channel cross-section, the Manning formula will be used to calculate the discharge of a known dimension of drainage structure cross-sectional area, wetted perimeter, and hydraulic radius. Equation 3.1 is used to calculate the hydraulic capacity of the open channel drainage structure.

$$Q = \frac{1}{n} AR^{2/3}S^{1/2} \dots\dots\dots\text{Equation 3:1}$$

Where Q= discharge passing through the channel (m³/s),

A = cross-sectional area (m²),

n= manning roughness coefficient

P= the wetted perimeter of the cross-sectional area of flow (m),

S = bottom slope of the channel (m/m),

R= hydraulic radius (m) = A/P

3.3 Methods of Data Analysis

3.3.1 Checking Consistency of Rainfall Data

The double-mass curve is a graphical approach for evaluating the trend and consistency of a given gauging station's rainfall data record over time in comparison to other stations. It is critical to evaluate the consistency of the data before performing any analysis with it, in order to eliminate bias caused by inconsistencies in the input data.

Independent rainfall data obtained from other 3 nearby stations Hawassa station, Wondogenet station and Morocho station for selected storm events were used.

3.3.2 Double Mass Curve

Double mass curve is employed to compare Tula station's hydrological data with data from other stations in order to assess the consistency of the data. (Hawassa station, Wondo Genet station and Morocho station) in the surrounding and that can used to adjust inconsistency precipitation data. The ratio between the variables is set by a correction factor of 1.052, which compares the annual rainfall total at Tula Station to the average annual rainfall total at nearby stations.

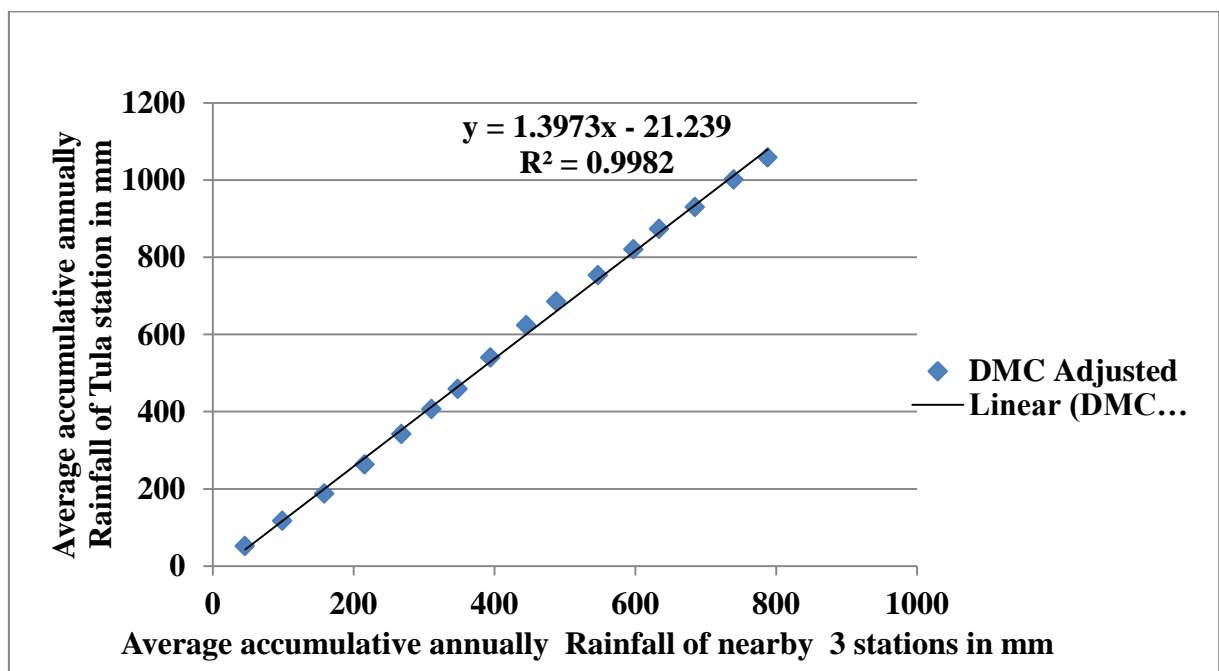


Figure 3.5 Double mass curve (Rain fall) of Tula town

3.3.3 Testing Relative Standard Error

The suitability of the rainfall data series should be assessed and realized before moving on to the other analysis. If the relative standard error is $\leq 10\%$, the data series is thus regarded as adequate.

$$X_m = 66.1751$$

$$\text{Standard deviation (Se)} = 2.589735$$

Relative standard error = 3.913456% which is 3.913456% <10% ok!

Therefore the data is reliable and adequate.

3.3.4 Testing Outlier

Outliers are data points that deviate from the general trend of the remaining data, as demonstrated in (appendix table 2) calculated the Skewness coefficient is between -0.4 and +0.4 our data recorded with respect to both outlier is appropriate range. Lower and higher outliers are screened for in this data.

3.3.4.1 Higher Outlier Test

$$K_n = 2.279 \text{ from table, when } n=16$$

$$Y_H = 1.9716$$

$$\text{Antilog } Y_H = 93.6758 \text{mm.}$$

The higher recorded daily maximum rainfall is >84.129mm in the 2013 which is lower than boundary value of higher outliers.

Hence there are no higher outlier's data recorded.

3.3.4.2 Lower Outlier

$$Y_L = 1.6597$$

$$\text{Antilog } Y_L = 45.6779 \text{ mm.}$$

The lowest recorded daily maximum rainfall is 51.73291 mm in the 2021 which higher than boundary of lower outliers. Hence the daily maximum rainfall data recorded with respect to lower outlier is within reasonable range, so there are no lower outliers in this sample data.

3.3.5 Estimating and Filling Missing Rainfall Data

Rainfall data records are sometimes incomplete due to the absence of an observer or equipment malfunction. In this scenario, the missing data might be estimated by comparing rainfall data from nearby stations. There are various methods for calculating missing rainfall data, depending on the effect of orography on rainfall, the distance between rainfall stations, and the range in rainfall amounts recorded at the stations.

Normal ratio method:

A normal ratio method is recommended to estimate missing rainfall data in regions where annual rainfall between stations differ by more than 10%. The formula for this method is presented in:-

$$P_x = \frac{1}{n} \left[P_1 \frac{N_x}{N_1} + P_2 \frac{N_x}{N_2} + P_n \frac{N_x}{N_n} \right] \dots \dots \dots \text{Equation 3:2}$$

Where

P_x = is the missing precipitation value for station X for a certain time period

Where $N_1, N_2,$ and N_3 = is the average annual rainfall at the different nearest gauge.

$P_1, P_2,$ and P_3 = rainfall data at the nearest different station (daily, monthly, yearly)

Arithmetic means method:

Arithmetic means method will be used to estimate missing rainfall data in regions where annual rainfall between stations differ by less than 10% of mean annual rainfall at missed station and estimated by using:-

$$P_x = \frac{\sum_{i=1}^n P_i}{n} \dots \dots \dots \text{Equation 3: 4}$$

Where: P_x - is the missing precipitation t station x.

3.3.6 Testing the Quality of Data by Relative Standard Error

It is important to confirm and understand the suitability of the rainfall data series before moving on to the other analysis. The data series should be considered and adequate if relative standard error $\leq 10\%$,

Where is the calculated as follows

$$\text{Average daily maximum annually rainfall of } \log x(y_m) = \sum X/N \dots \dots \dots 3.5$$

$$\text{Average daily maximum annually rainfall (xm)} = \sum X/N \dots \dots \dots 3.6$$

$$\text{Standard deviation (xi)} \delta n-1x = \sqrt{\sum (x-xm)^2} \dots \dots \dots 3.7$$

$$\text{Standard deviation (yi)} \delta n-1y = \sqrt{\sum (y-ym)^2} \dots \dots \dots 3.8$$

$$\text{Skewedness coefficient (Cs)} = \frac{N \sum (y-ym)}{(N-1)(N-2)S_y^3} \dots \dots \dots 3.9$$

$$\text{Relative standard error } (\delta e) \% = \frac{Se}{xm} * 100 \dots \dots \dots 3.10$$

$$Se = \frac{\delta e - 1x}{\sqrt{N}} \dots \dots \dots 3.11$$

Whereas N = Number of year recorded rainfall data

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

3.3.7 Testing outliers

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

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

Case 2; if Skewedness coefficient (Cs) $> +0.4$ check for higher outlier

Case3; if Skewedness coefficient (C_s $-0.4 < C_s < +0.4$ check for both outlier.

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

A, Lower outlier determination to detect the lower outlier the following equation is applied.

Lower outlier

$$Y_L = Y_m - K_n * s_y \dots\dots\dots 3.11$$

B, Higher outlier determination to detect the higher outlier the following equation are applied

Higher outlier

$$Y_H = Y_m + K_n * s_y \dots\dots\dots 3.12$$

Where Y_L and Y_H = antilog of value calculated Y_m = mean of data in log unit

K_N = read from table for sample size N (16 year) find K_N from (Chow., 1988).

s_y = for Skewedness coefficient (C_s) value calculated by above equation 3.8 find s_y from log transformation series garg book

3.5 Formulation of IDF Curve for Tula Town

The IDF curve is developed from a 24-hour rainfall data of 16 years' duration i.e. from 2006 to 2021, obtained from Ethiopian Meteorological Agency. 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 input to design hydraulic structures including urban drainage infrastructure. A number of techniques are available to formulate IDF curve. However Pearson Type III distribution which are the most commonly used in Ethiopia were employed for this research purpose using 16 years daily rainfall (24hr record) data.

The following equation were used to developed Rainfall in a given durations (hr).

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

Where:- R_t/R_{24} = rainfall ratio

R_t = rainfall in given duration (hr.)

R_{24} = rainfall in 24hr

n (constant) = 0.3

b (constant) =0.9 and

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

The relation adopted for IDF development at a given station, all probability distribution function were compared by chis-Square test of goodness fit and selecting the best probability function can be used for this is research is Pearson type III distribution method (Chow., 1988).

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

Where X_T =the magnitude computed runoff (mm)

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

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

K_T =the Frequency Factor

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

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

$$P = \frac{1}{T} \dots\dots\dots 3.17$$

Where Z=standard normal variation

W= an intermediate variable

P=probability function

T= return period (years)

When $p > 0.5$, $1-p$ is substituted for p in equation W and Z computed is given a negative sign. The error in this formula less than 0.00045 in Z (Chow., 1988).

3.6 Modeling Rainfall Using Environmental Protection Agency SWMM 5.2

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

3.6.1 Model Capability

SWMM takes into account a variety of hydrologic processes that result in runoff from metropolitan areas. By breaking up a research region into a number of smaller sub catchments areas, each of which has a different proportion of pervious and impervious sub-areas, spatial diversity in all of these processes is obtained. Runoff and external inflows are routed via the drainage system's network of channels, pipes, and storage unit structures using a versatile set of hydraulic modeling capabilities provided by the model.

Among them is the capacity for

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

It is crucial that rain gages be placed inside and close to the catchment for modeling accuracy to be more precise and for SWMM calibration to be successful (EPA SWMM 5.2 Manual).

3.6.2 Model Set up Procedure

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

3.6.3 Model Preparation Area

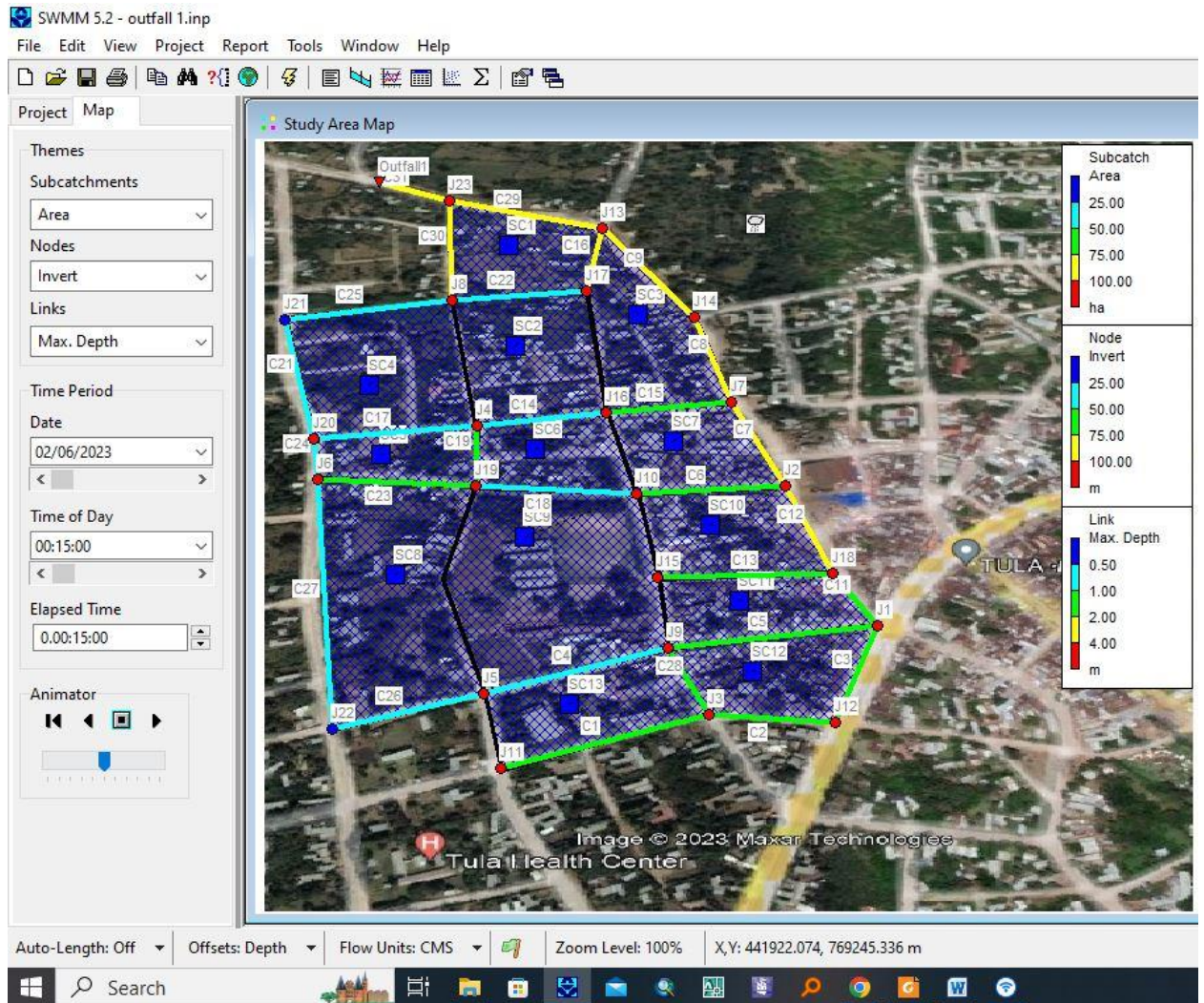


Figure 3.6 The outlet one map area prepared

The modeled area of outlet one starts from the highway near to Tula town municipality bureau to Tulo Woyo asphalt road. That simulated model of each 13 sub catchments connected into 23 nodes and 29 channels and flow routing outfalls into Tulo Woyo junction.

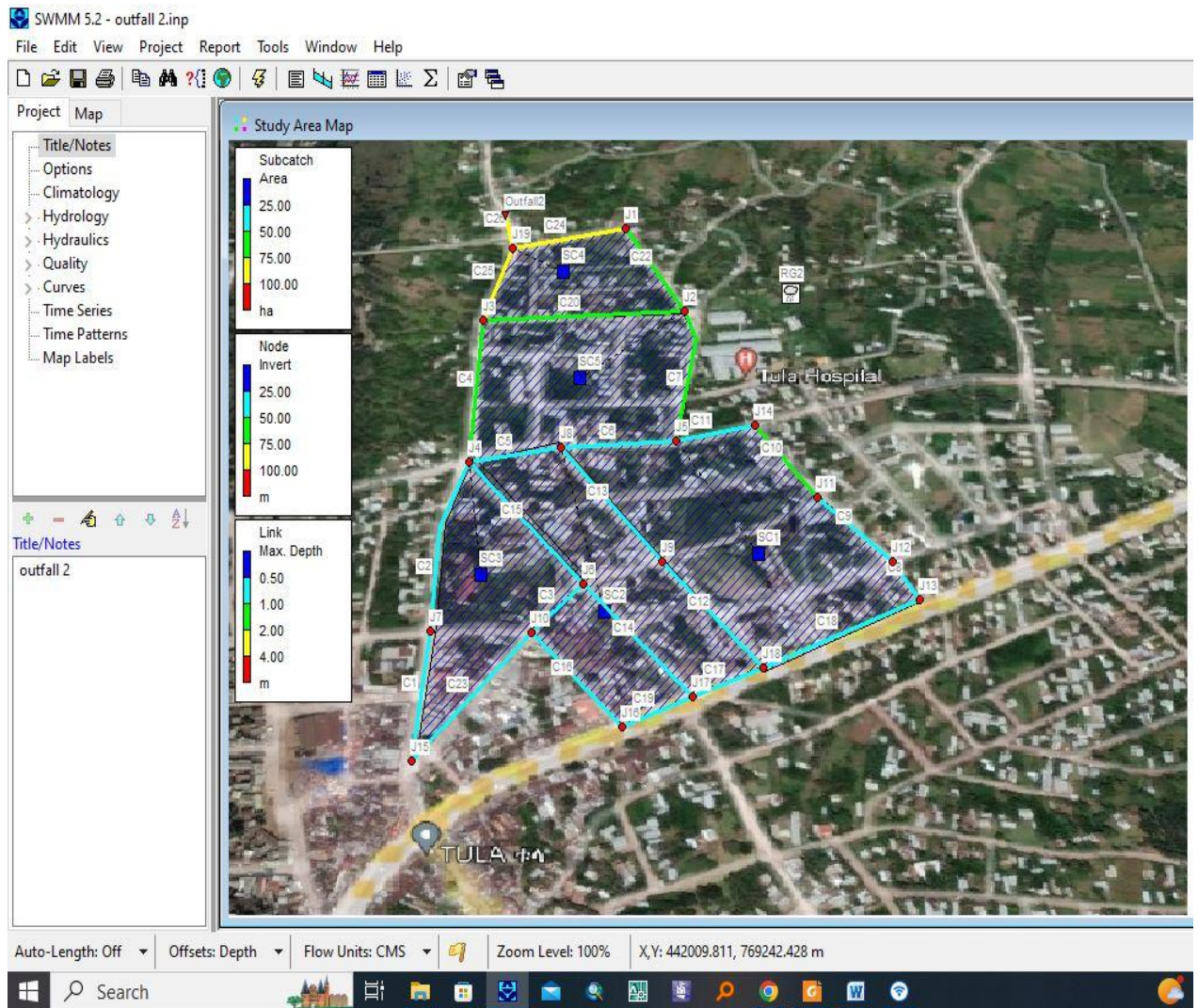


Figure 3.7: The outlet two map area prepared

The total model area prepared of Tula town drainage system networks consists of 70.55ha area from the whole 18 sub catchments 42 junctions and 55 channels and 2 outfalls which showed on the above figure 3.7, 3.8

The manholes and junctions were designed as rectangular channels with varying depths. It had expected that there would be no energy losses in the manholes. Each sub-relationship catchment's to the rainfall time-series was used to enter the precipitations into the model.

3.6.4 Model Governing Equations

According to SWMM, a sub catchment is conceptualized as a rectangle with a single outlet channel, a single width W , and a uniform slope S . This rectangle is depicted in Figure 3.9 Modeling the sub catchment as a nonlinear reservoir, as shown in the sketch, produces overland flow.

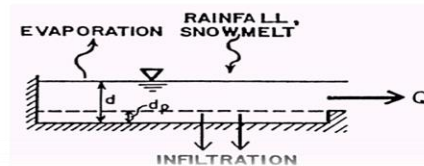
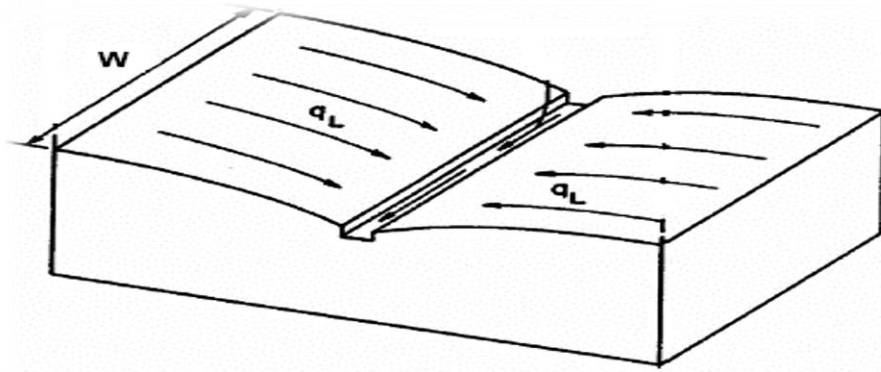


Figure 3.8 : Idealized and Nonlinear reservoir model of a sub-catchment

(Source SWMM model manual)

Nonlinear reservoir runoff method can be mathematically described by continuity and Manning's equations in Eq3.18, where d , q , and then $Q=q \times A$ can be solved at each time step with given i , e , and f . Parameters required to carry out nonlinear reservoir runoff method include drainage area A , overland flow Manning's n , depression storage d_s , width W , and sub catchment slope S which should be measured along the overland flow path.

$$\frac{\partial d}{\partial t} = (i - e - f - a(d - d_s))^{5/3} \dots\dots\dots \text{Eq 3.18}$$

where $a = \frac{1.49WS^{1/2}}{A n}$

i = rate of rainfall + snowmelt (m/s)

e = surface evaporation rate (m/s)

f = infiltration rate (m/s) and

q = runoff rate (m/s).

$$Q = \frac{1.49}{n} (AR^{2/3} S^{1/2}) \dots\dots\dots \text{Eq 3.19}$$

$$A = W * (d - d_s) \quad R = d - d_s$$

Substituting these expression into equation above

$$Q = \frac{1.49}{n} * W * (d - d_s)^2 * S^{1/2} \dots\dots\dots \text{Eq 3.2}$$

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

$$q = \frac{1.49WS^{1/2}}{A_n} (d - ds)^{5/3} \dots \dots \dots \text{Eq 3.3}$$

Some thoughts on SWMM nonlinear reservoir runoff method.

- ❖ Easy to build a model without the need of estimating Tc or selecting a UH transform method, however, not every regulating agency approves its use
- ❖ A subcatchment should NOT be too large in order to apply SWMM nonlinear reservoir runoff method. It is difficult to set up a certain upper limit, but it appears using several hectares (say 2.02ha or maybe even 4.04ha) as maximum allowable subcatchment areas is reasonable.
- ❖ How to appropriately calculate subcatchment width W is critical and tricky. EPA SWMM User’s Manual and Hydrology Reference Manual are good sources for width W estimation.
- ❖ Width W can be estimated as the quotient of drainage area divided by average maximum overland flow length. The maximum overland flow length is the length of the flow path from the remotest point to where the flow becomes channelized and it probably should be less than 30.48-304.8m (common) or 304.8m (uncommon).

3.6.5 Model Input Data

The catchment of the study area is classified into sub-catchments with the help of Google earth and Arc GIS according to the hydrologic unit of land whose topography and drainage system elements direct surface runoff to a single discharge point. Input parameters required for sub-catchment include: assigned land uses, tributary area, rainfall intensity, outlet nodes, manning coefficient and slope. Rainfall intensity is obtained from IDF curves for 2, 5, 10, 25, 50 and 100 years. This all parameters are necessary for the SWMM model for runoff simulation. The area of weighted percent of imperviousness is determined by summing the amount of impervious area of each sub-catchment and dividing this sum by the total catchment area. The slope of each sub-catchment is assumed to be equal with the flow path slope and can estimated as the elevation difference divided by the flow path length on the map.

Conduit, junction, and outfall are elements of urban stormwater drainage structures that used to transport the runoff from developed areas to outlet points. Conduits are drainage channels that transport stormwater runoff from one node to another in the conveyance system. The junction is an element of drainage structure nodes where links join together. Outfalls are

terminal nodes of the drainage system. The input parameter required for these drainage elements includes; invert elevation, the height of the ground surface, and cross-sectional geometry of conduits. These all data obtained through a field survey by using a tape meter. Typical values from the literature will be taken for Manning for pervious and impervious areas.

3.6.6 Model Performance Evaluation

The reliability of the flow monitoring data will be assessed as part of the model calibration and validation process. And also, model simulation errors will be quantified by measuring the difference between flow calculated by rational or/and SCS method and simulated flows. Results of all comparative studies to be analyzed with three error functions

$$\text{Coefficient of determination } (R^2) = \left[\frac{\sum_{t=1}^n (Q_{obs} - Q_{av.obs})(Q_{sim} - Q_{av.sim})}{\sqrt{\sum_{t=1}^n (Q_{obs} - Q_{av.obs})^2} \sqrt{\sum_{t=1}^n (Q_{sim} - Q_{av.sim})^2}} \right]^2 \dots \text{Equation 3:4}$$

$$\text{The Nash-sutcliffe efficiency (NSE)} = 1 - \frac{\sum_{t=1}^n (Q_{obs} - Q_{sim})^2}{\sum_{t=1}^n (Q_{obs} - Q_{ave.obs})^2} \dots \text{Equation 3:5}$$

$$\text{Relative error (RE)} = 1 - \frac{\sum_{t=1}^n |Q_{obs} - Q_{sim}|}{\sum_{t=1}^n Q_{obs}} \dots \text{Equation 3:6}$$

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

3.6.7 Steps in Using SWMM

One typically carries out the following steps when using EPA SWMM to model a study area:

- i. Specify a default set of options and object properties to use.
- ii. Draw a network representation of the physical components of the study area.
- iii. Edit the properties of the objects that make up the system.
- iv. Select a set of analysis options.
- v. Run a simulation.
- vi. View the results of the simulation.

For building larger systems from scratch, it will be more convenient to replace Step 2 by collecting study area data from various sources, such as CAD drawings or GIS files, and transferring these data into a SWMM input file (Rossman, 2015).

3.7 Selection of Model Area

After a two-week's visit to the town and discussions with the relevant Tula Municipality specialists, a 70.55 ha model area in the western part of the town was chosen. The reason to select that particular area as a model is it has been severely affected due to over flow of drains during the rainy season.

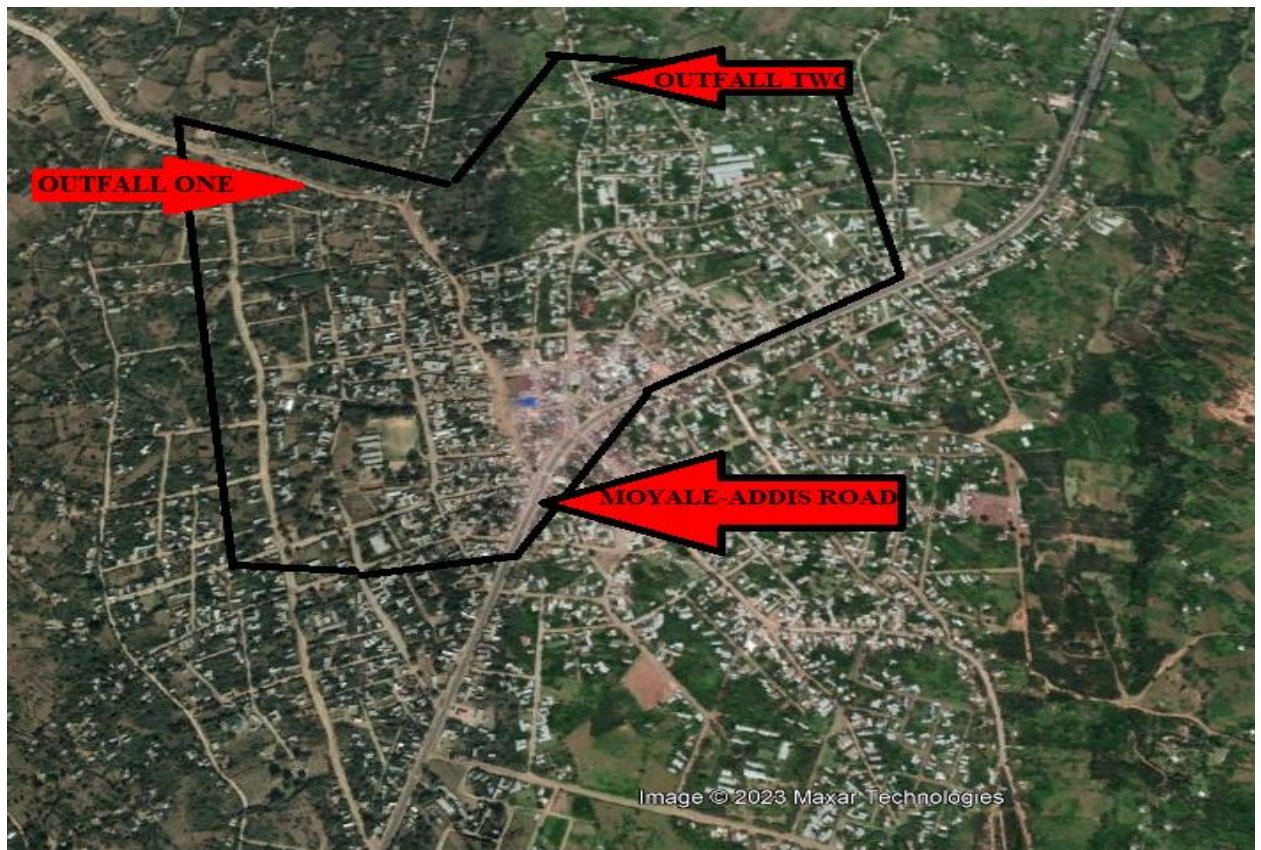


Figure 3.9 Watershed & location of selected model area

3.7.1 Physical Properties of Sub catchments

Table 3.3 Physical properties of sub catchments

Sub Catchment	Area(ha)	Width(m)	Av slope	Impervious %	N-imp	N-perv	Dstore imper	Dstore perv
SC1	2.53	213	0.5	26.43	0.01	0.1	1.5	0.05
SC2	2.95	236	2.94	25	0.01	0.1	1.3	0.05
SC3	2.87	381	6	37.76	0.015	0.5	2.5	7
SC4	3.12	233	3	10.89	0.015	0.5	1.5	7
SC5	1.22	152	0.1	32.78	0.015	0.5	2.5	7
SC6	1.51	136	0.1	47.68	0.015	0.5	0.05	7
SC7	1.77	157	3.35	88	0.015	0.5	2.5	7
SC8	4	358	3	31.37	0.015	0.5	0.05	7
SC9	4.33	297	1	28.22	0.015	0.5	1.3	7
SC10	1.95	150	0.1	93.4	0.015	0.5	2.5	7
SC11	1.33	180	1	96.54	0.015	0.5	2.5	0.05
SC12	1.55	180	0.1	93.54	0.015	0.5	2.5	7
SC13	1.47	173	1	29.25	0.015	0.5	2.5	7
Model Area 2								
SC1	5.35	367	0.5	39.82	0.015	0.5	2.5	7
SC2	4.42	386	0.5	57.43	0.015	0.5	2.5	7
SC3	2.73	380	0.5	49.25	0.015	0.5	2.5	7
SC4	2.65	224	0.5	28.5	0.015	0.5	2.5	7
SC5	3.8	236	5	52.5	0.015	0.5	2.5	7

3.8 Calibration and Validation of Model

The sensitivity analysis was carried out by changing each parameter while keeping the others constant and observing the changing in model output using the most recent SWMM5.2 obtained from the hydraulic model calibration and validation process were done for Tula town. The calibration model was able to predict the observed output with the reasonable accuracy (Demisu Tefera, 2020).

The simulated and observed runoff in the outlets was contrasted throughout the model calibration procedure, which was done manually through trial and error.

Table 3.4 some keys parameters used for sensitivity analysis

Parameter	Description	Allowable range of change
N-Imperv	Manning's roughness coefficient for impervious areas	0.005-0.05
N-perv	Manning's roughness coefficient for pervious areas	0.05-0.5
Dstore –Imperv	Depth of surface storage in impervious areas(mm)	1.3-2.5
Dstore-Perv	Depth of surface storage in pervious areas(mm)	2.5-7.6
Zero-Imperv	Impervious areas without surface storage (%)	3.76-89.11

3.8.1 Area of Simulation

It was necessary to calibrate and validate the SWMM model by checking the rainfall through the basin and recording the stream flow at the drainage system. It is challenging to gather data because there are no flow gauges installed in Tula town's drainage system and no documented data is available.

From Tula High School, through Moyale-Addis road ends to Tolo Woyo junction, around outlet one, flow depth data were recorded in an open rectangular trapezoidal channel in order to create an accurate model. In order to calibrate the sensitivity parameter and validate the SWMM 5.1 model for the region, this flow depth was measured for a period of 10 days, from June to the first quarter of September, at various intervals.

Table 3.5: Recorded flow depth for calibration and validation model

Gauged Area	Typical channel	Data of measured date	Recorded flow depth (m)	Method calculation flow discharge $Q = \frac{1}{n} AR^{2/3} S^{1/2}$
At outlet one for calibration used	Rectangular	June 15 2021	0.82	Listed in Result And Discussion
		July 12 2021	0.7	
		July 20 2021	0.43	
	Closed Rectangular	August 21 2021	0.93	
		September 11 2021	0.67	
at outlet one for validation used	Rectangular	June 6 2021	0.91	
		July 1 2021	0.73	
		July 13 2021	0.65	
	Closed Rectangular	August 10 2021	1.6	
		September 19 2021	0.96	

3.8.2 Acceptable Level of Calibration

The range 0–1 denotes models that are suitable. 1 denotes ideal models, 0 denotes poor models. Relative Error (RE) 30%, R2 is approaching one and suggests models that are suitable. The relationship between the observed and simulated hydrograph peak, volume, and time to peak is depicted on the graph. In the design and analysis of urban drainage systems, these three characteristics are crucial. In urban drainage design, peak discharge is necessary for sizing pipes, culverts, and bridges. For the construction and operation of flood control facilities like retarding basins, runoff volume is necessary. For the operation of control structures during storm events and flood predictions, time to peak discharge is necessary. Data on flows was adjusted for daily flows.

3.9 Evaluate Alternative Mitigation Actions for Drainage Overflow

Urban drainage regions that get too much storm water runoff are at risk of flooding, which causes serious economic losses, traffic jams, and even localized fatalities. In order to reduce the risk of flooding in urban areas, it is necessary to increase the drainage system's capacity, implement Sustainable Urban Drainage Systems (SUDS), add best management practices (BPMS), and use low impact development (LID) methods (Zhou, 2014).

3.9.1 Low Impact Development

Instead of using the conventional stormwater design, a low impact development (LID) method has been suggested for land development. LID's primary goal is to apply sound stormwater management techniques to lessen the impact of development on water-related issues (Shafique Muhammad and Kim Reeho, 2015). Owing to the town's severe flooding issues, two types of LID practices Bio-retention cells and Infiltration trenches were modeled as potential solutions to lessen the issues (Hawaz, 2020).

Bio retention Structure

Depressed areas in the landscape that are intended to receive stormwater are known as bio retention areas or rain gardens. They can be utilized in both residential and commercial settings, and are often planted with trees, shrubs, or perennials and coated with mulch made from shredded hardwood bark. Reduced surface runoff, enhanced groundwater recharge, and pollutant treatment through a variety of procedures are all advantages of bio retention areas (Prince George's County, 1999).

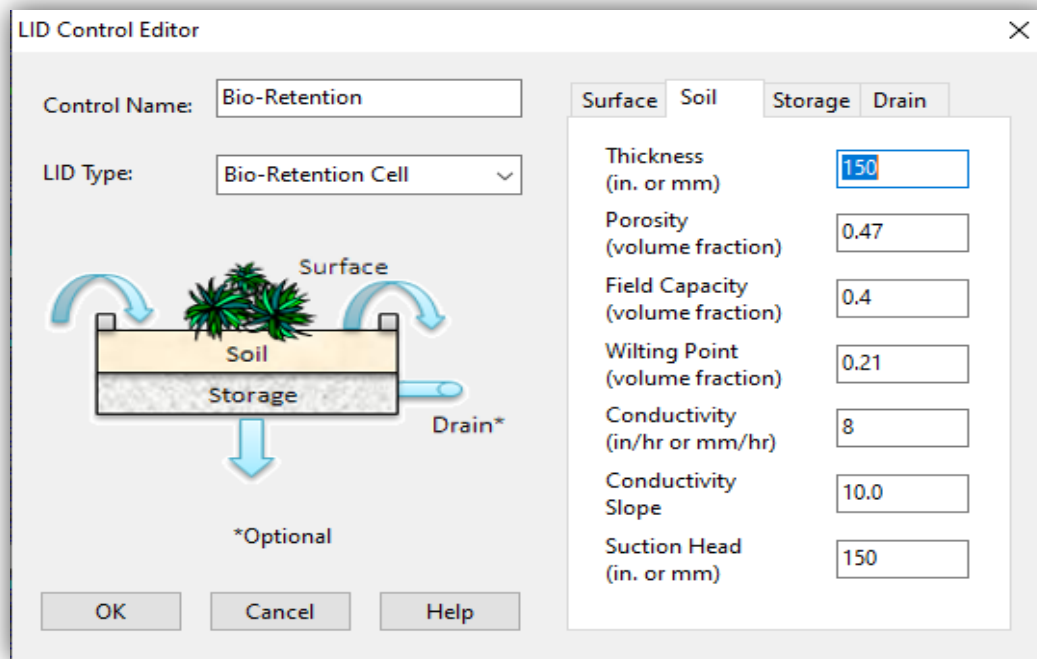


Figure 3.10 Conceptual model of a bio retention cell in SWMM

Table 3.6: Parameters of bio retention structure for the model

Surface	Parameters	Dimension
	Berm Height (mm)	300
	Vegetative Volume Fraction	0.1
	Surface Roughness	0.024
	Surface Slope (%)	1.5
Soil	Thickness (mm)	150
	Porosity (volume fraction)	0.47
	Field capacity (volume fraction)	0.4
	Wilting Point (volume fraction)	0.21
	Conductivity (mm/hour)	8
	Conductivity slope	10
	Suction head (mm)	150
Storage	Thickness (height) (mm)	350
	Void ratio (voids/solids)	0.75
	Seepage rate (mm/hour)	5
	Clogging Factor	0.1
Drain	Flow coefficient	5
	Flow exponent	0.5
	Offset height (mm)	500

Infiltration trench

Infiltration trench are constructed structures that store runoff and make it easier for water to seep into the ground. The majorities of the time, infiltration trenches are long, thin, and filled with aggregate. An infiltration trench in the LID area was used to channel runoff from the research area. They can be represented as totally permeable sub-catchments with depression storage depths that are equivalent to the depth of the trench's available pore space (Demisu Tefera, 2020).

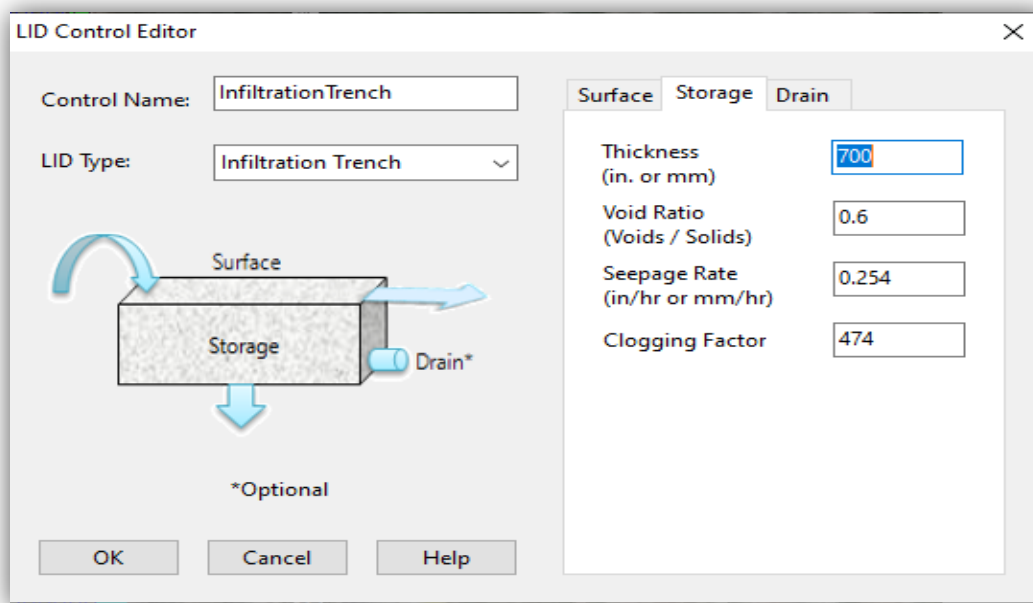


Figure 3.11 Conceptual Model of a bio retention cell in SWMM 5.2

Table 3.7 Parameters of infiltration trench for the model

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	Drain Flow coefficient	34
	Flow exponent	0.5
	Offset height (mm)	0.5

Vegetative Swale

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

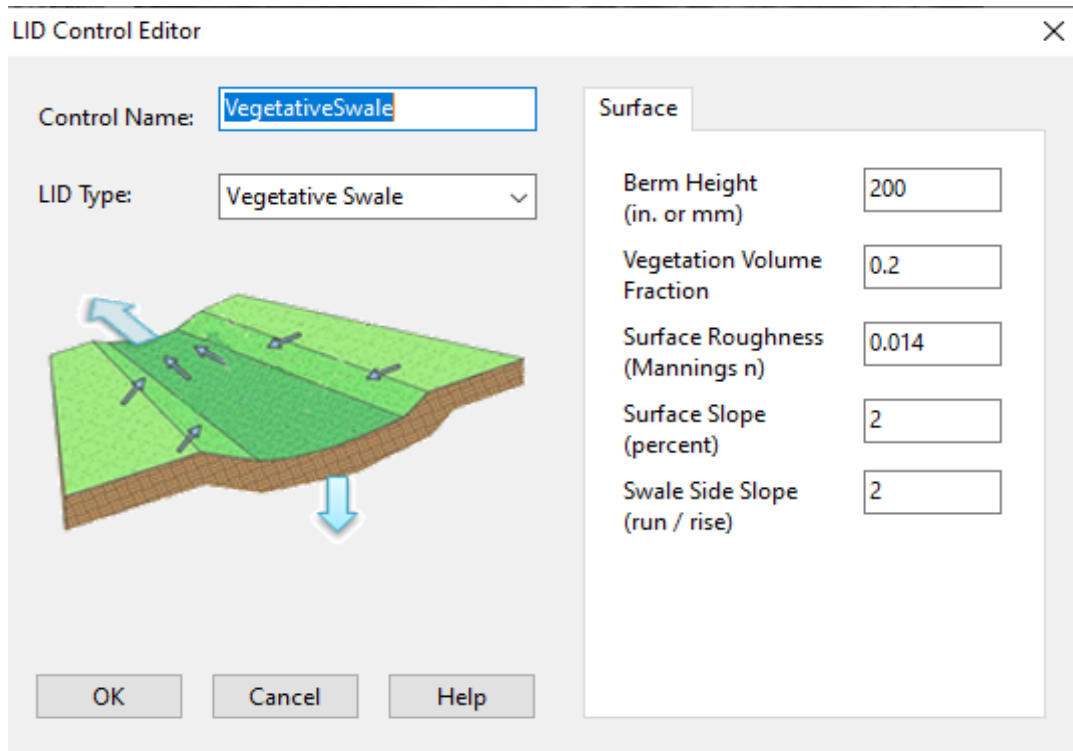


Figure 3.12 Conceptual Model of a Vegetative Swale in SWMM 5.2

Utilized LID controls editors in the SWMM tool to simulate the best alternate mitigation strategies to reduce the drainage overflow of urban drain systems in their gravel storage beds in order to transport extra captured runoff out of the site and stop the unit from flooding.

4. RESULTS AND DISCUSSIONS

4.1 IDF Curve Development

The intensity duration frequency curve developed from daily rainfall data of 16 years from 2006 to 2021 obtained in Ethiopia meteorological agency rainfall gauge station located in Hawassa city. For this study the rainfall intensity obtains from IDF curve is used to SWMM model as input parameter.

4.1.1 IDF Curve Development for Tula Town

Table 4.1 Tula IDF curve development

Duration (minute)	Duration (hour)	2 year Intensity mm/hr	5 year Intensity mm/hr	10 year Intensity mm/hr	25 year Intensity mm/hr	50 year Intensity mm/hr	100 year Intensity mm/hr
5	0.083333	77.16498	92.88139	100.5294	108.2879	113.0983	117.2634
10	0.166667	64.64472	77.8111	84.21822	90.71788	94.74772	98.23708
15	0.25	55.75871	67.11525	72.64166	78.24788	81.72378	84.7335
30	0.5	39.79772	47.90343	51.8479	55.84934	58.33026	60.47844
60	1	25.70929	30.94558	33.4937	36.07863	37.6813	39.06902
120	2	15.38453	18.51794	20.04274	21.58957	22.54861	23.37903
180	3	11.11672	13.38089	14.4827	15.60043	16.29342	16.89348
360	6	6.212017	7.477237	8.092928	8.717511	9.104758	9.440067
720	12	3.401924	4.094803	4.431978	4.774022	4.986092	5.169719
1440	24	1.843289	2.218717	2.401411	2.586743	2.701651	2.801147

The intensity duration frequency curve was created using daily rainfall data for 16 years, from 2006 to 2021, from the Hawassa city rainfall gauge station of the Ethiopian Meteorological Agency. The rainfall intensity obtained from the IDF curve is employed as an input parameter in the SWMM model for this study.

The chi-square test of goodness of fit was used to assess all probability distribution functions, and the function that produced the lowest chi-square value was chosen as the optimal probability function. The best fit probability is a normal distribution, which was utilized to build the IDF curve for Tula Town, as shown in figure 4.1, and which is given in depth analysis in appendix Table 3 to 9.

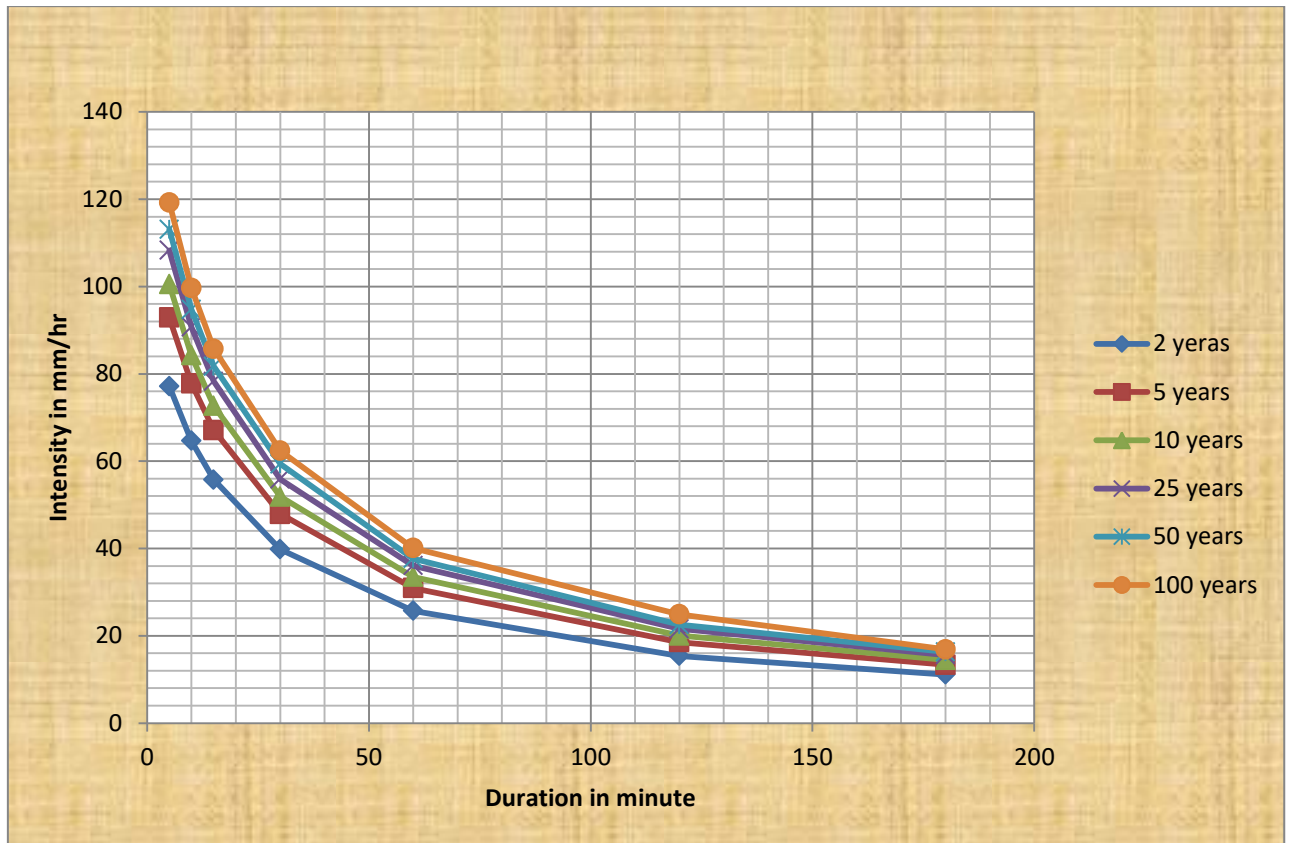


Figure 4.1 Developed IDF curve for Tula town

4.1.2 Relationship of Tula Town IDF Curve with ERA

Any probability distribution may be employed in the relationship established for IDF development at a specific station, but it was determined whether a given distribution was reliable and appropriate for the data set with ERA created. Any probability distribution is employed in the relationship established for IDF development at a specific station, but it was determined whether a given distribution was reliable and appropriate for the data set with ERA created.

Table 4.2 Comparison of IDF curve

Return period	Duration in min									
	10		30		60		120		180	
	Tula	ERA	Tula	ERA	Tula	ERA	Tula	ERA	Tula	ERA
2	64.64	75	39.79	48	25.7	30	15.38	19	11.11	12
5	77.81	96	47.9	59	30.94	36	18.51	23	13.38	17
10	84.21	110	51.84	70	33.49	44	20.04	28	14.44	19
25	90.71	125	55.84	76	36.07	50	21.58	30	15.6	22
50	94.74	140	58.33	85	37.68	55	22.54	33	16.29	24
100	98.23	152	60.47	94	39.06	60	23.37	36	16.89	25

4.2 Calibration and Validation of the Model

The SWMM model in this study must have its sensitive parameter values adjusted until the observational fit is comparable to the calibration, as determined by the goodness of fit as determined by the performance simulation model.

4.2.1 Calibration of the Model

For the modeled accuracy and effective calibration of SWMM, the calibration process has been verified by altering sensitive parameters to varied model parameter input and checking the output model result until it matches closely within the required range in comparison with the observed results.

Table 4.3 Sensitivity parameter used for the calibration of model



Parameters	Manning Description	Recommended value range	Initial used proceed sensitive parameter	Final used Sensitive Parameter
N-Impervious	Manning Roughness for Impervious Area	0.011- 0.015	0.013	0.015
N-Pervious	Manning Roughness For Pervious Area	0.05-0.8	0.4	0.5
Dstore-Impervious	Depth of Depression Storage on Impervious Area	0-3	2	2.5
Dstore-Pervious	Depth of Depression Storage For Pervious Area	3-10	4	7
Smooth concrete roughness	Manning Roughness Coefficient For Open Smooth Concrete	0.012	0.012	0.012

The input parameter's used in the above table 4.3 for the calibration of model the final output Calibrated results of peak discharge flow rate simulated by SWMM5.2 model is $5.27\text{m}^3/\text{s}$ and the observed flow depth at site is calculated peak discharge flow rate by manning equations is $5.1\text{m}^3/\text{s}$.

4.2.2 Validation of the Model

The velocity and discharge flow rate through the drainage systems are shown in table 4.4, demonstrating how well this model fits the data that were observed at the outlet's one roots.

Table 4.4 Validation flow depth observed result

Channel type	Recorded depth (m)	Manning roughness used(n)	Channel slope (s)	Width (m)	Area (m ²)	Perimeter (m)	Hydraulic Radius(m)	Velocity (m/s)	Discharge Q(m ³ /s)=ARS ^{1/2} /n
 Rectangular	0.91	0.013	0.0142	0.8	0.728	2.62	0.278	3.918	2.86
	0.73		0.0041	0.8	0.584	2.26	0.26	2	1.168
	0.65		0.0132	0.8	0.52	2.1	0.25	3.54	1.846
 Closed Rectangular	0.96	0.012	0.0032	1	0.96	2.92	0.33	2.251	2.16
	1.6		0.0083	1	1.6	4.2	0.38	3.983	6.3728

The channel velocity of 3.983 m/s and 3.91 m/s in result table 4.4 is a high number. The closed rectangular channel is made of stone masonry with floor stone pavement joined by concrete material and covered with reinforced precast. The rectangular open channel type material is made of high smooth quality concrete. Due to the great strength of the channel material used in construction, the high value of velocity above table 4.4 has no impact on the channel.

Table 4.5 Summary of the model result and observed values

Date of observed data	Recorded average flow depth (m)	Determine flow rate using manning equation Q(m ³ /s) = ARS ^{1/2} /n	Simulated flow rate using SWMM 5.2 (m ³ /s)
June 6 2021	0.91	2.86	3.45
July 1 2021	0.73	1.168	1.33
July 13 2021	0.65	1.846	2.29
August 10 2021	1.6	6.3728	6.61
September 19 2021	0.96	2.16	2.89

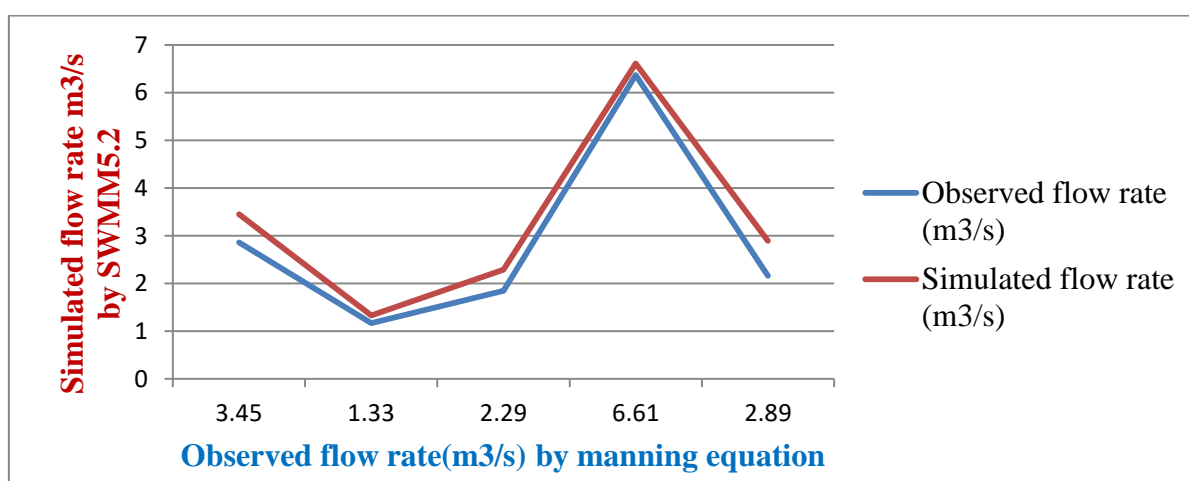


Figure 4.2 The Model Validation Flow chart

4.2.3 Location of Calibration and Validation Points in Model Area

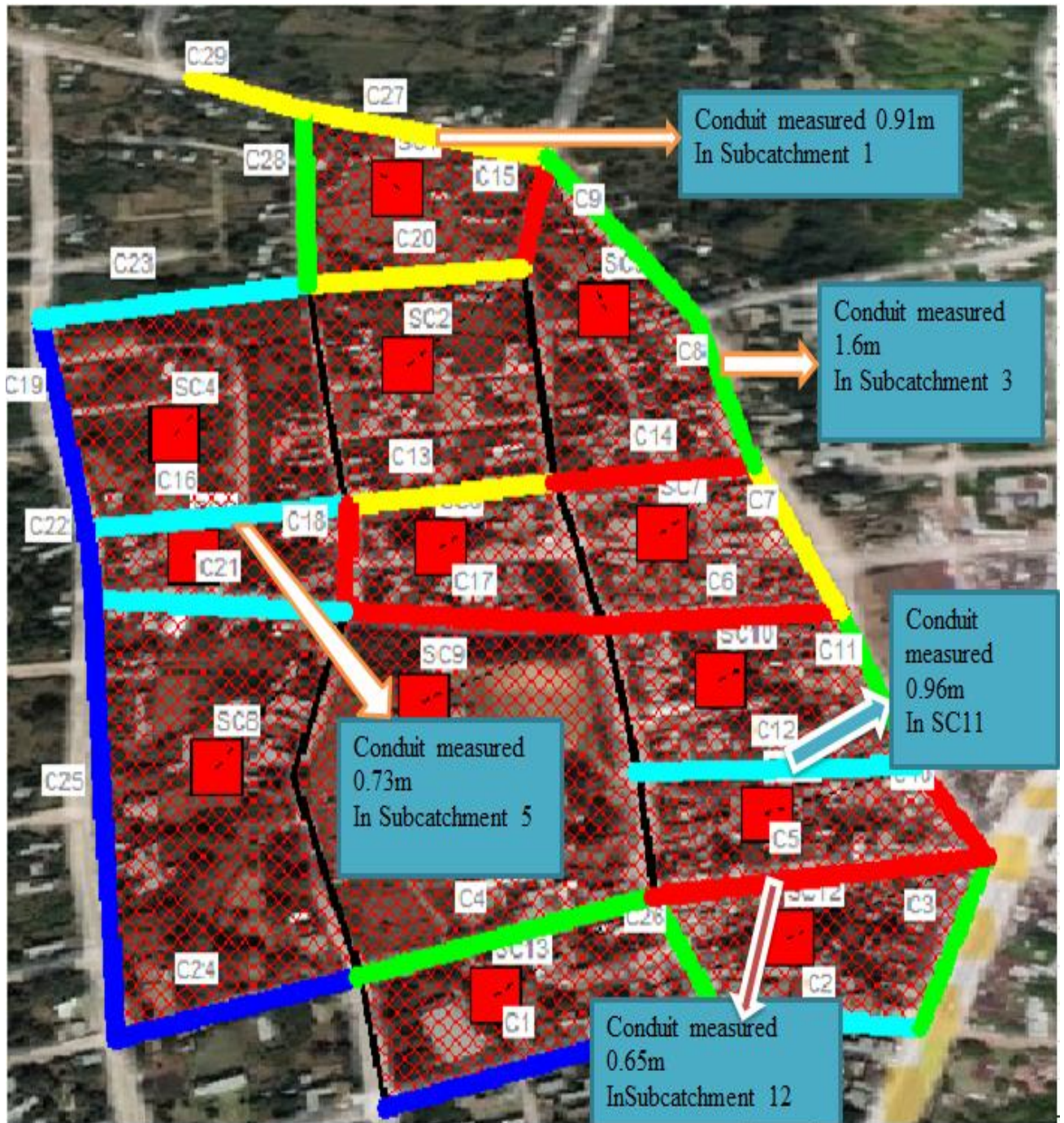


Figure 4.3 Places where calibration and validation undertaken

4.2.4 Model Performance Evaluation

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

Table 4.6 : Correctness function of the model performance

Recorded flow depth (m)	Simulated flow rate using SWMM 5.2 CMS	Calculated (observed flow rate using manning's) CMS	$q_t^{obs} - q_t^{avrg.obs}$	$q_t^{sim} - q_t^{avrg.sim}$	$(q_t^{obs} - q_t^{avrg.obs}) * (q_t^{sim} - q_t^{avrg.sim})$	$(q_t^{obs} - q_t^{avrg.obs})^2$	$(q_t^{sim} - q_t^{avrg.sim})^2$	$(q_t^{obs} - q_t^{sim})^2$	$ q_t^{obs} - q_t^{sim} $
0.91	3.41	2.86	0.02 136	0.37 74	0.0080 6	0.000456 2	0.142 430	0.3025	0.55
0.73	1.238	1.168	1.71 336	1.79 46	3.0747 95	2.935602 4	3.220 58	0.0049	0.07
0.65	2.01	1.846	1.03 536	1.02 26	1.0587 5	1.071970 3	1.045 710	0.0268 96	0.16 4
1.6	6.425	6.3728	3.49 144	3.39 24	11.844 36	12.19015 3	11.50 837	0.0027 2	0.05 22
0.96	2.08	2.16	0.72 136	0.95 26	0.6871 67	0.520360 2	0.907 446	0.0064	0.08
Total	15.163	14.406			16.657 02	16.71854 2	16.82 455	0.3434 20	0.24 13
Average	3.0326	2.8813				√	√		
						4.088831	4.101 774		

Coefficient of determination (R^2) = 0.98

Nash-Sutcliffe Efficiency (NSE) = 0.91

Relative Error (RE) = 0.2413

RE = 0.2413 = 24.13 % This is acceptable RE < 30%

Overall, the calibration resulted in a good fit with observed daily flows for the simulation period, with $R^2 = \mathbf{0.98}$, NSE = 0.91 and relative error (RE) = 24.13%.

This calibration and validation result indicated that the model structure and parameters matched the runoff producing pattern and the calibrated model was suitable for simulating runoff in the study area.

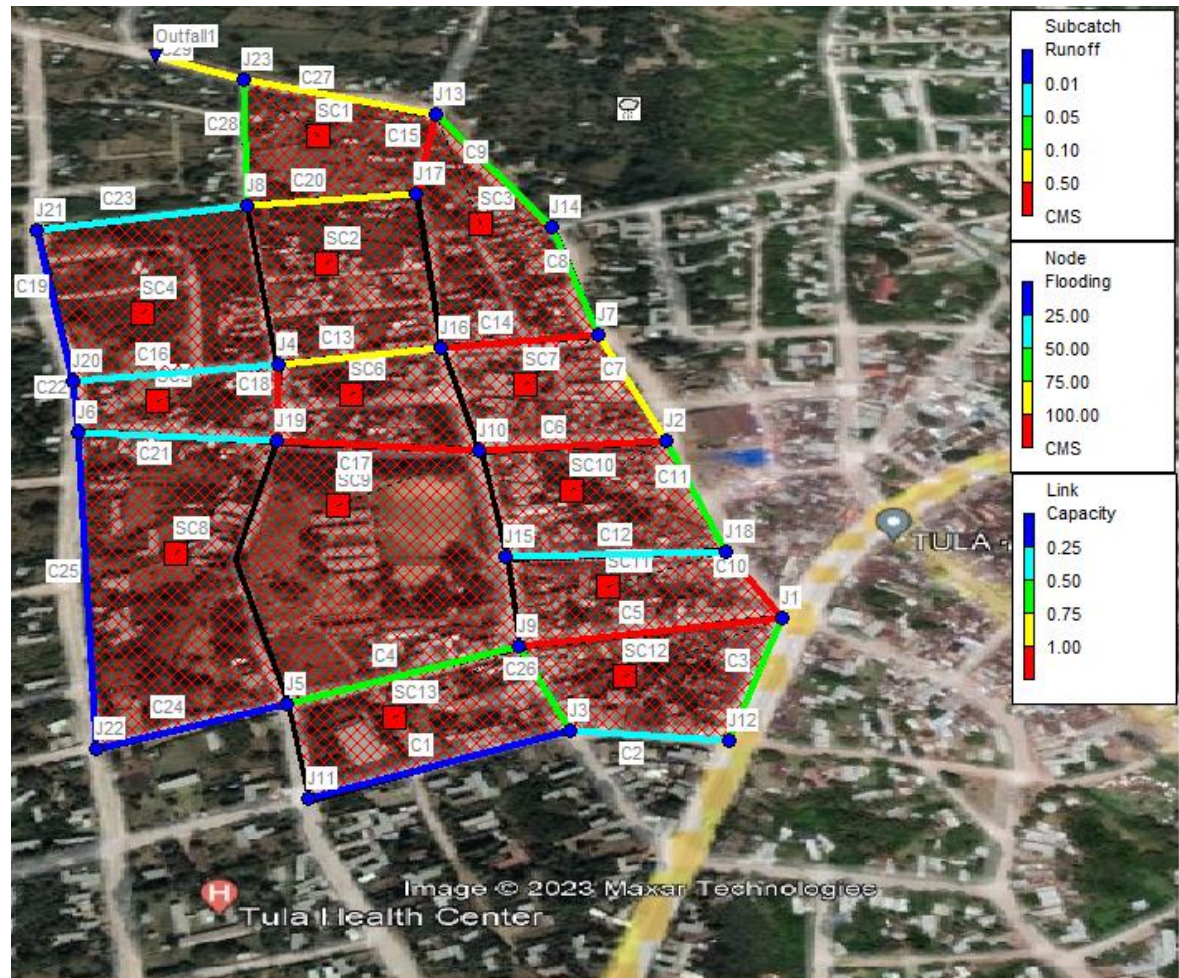
4.3 Model Result from SWMM5.2 Tool

In order to fully understand the performance of the town drainage system under various working conditions, a hydrological model was used in this study to first identify the critical condition and associated drainage system problem, then to evaluate the hydraulic performance of storm water drainage infrastructures, and finally to evaluate potential solutions for drainage problem mitigation measures.

The entire sub catchment in the research area is responsible for producing runoff from daily precipitation, which is calculated using rainfall data from 2006 to 2021 and taken maximum rainfall from each year. The sub catchment project that is utilized as the input for drainage systems connects through each node and simulates runoff in the network system of the SWMM5.2 model in each of the two outfall studies.

4.3.1 Result of Outlet One

4.3.1.1 Sub Catchments Runoff



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

Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Imperv Runoff mm	Perv Runoff mm	Total Runoff mm	Total Runoff 10 ⁶ ltr	Peak Runoff CMS	Runoff Coeff
SC9	1193.76	0.00	0.00	49.28	240.46	830.31	1070.77	40.80	3.79	0.897
SC10	1193.76	0.00	0.00	36.34	491.07	572.96	1064.03	41.50	4.26	0.891
SC11	1193.76	0.00	0.00	49.28	240.08	863.86	1103.94	36.43	3.85	0.925
SC12	1193.76	0.00	0.00	45.58	311.22	824.74	1135.96	8.86	1.23	0.952
SC13	1193.76	0.00	0.00	48.05	262.99	873.55	1136.53	17.16	2.58	0.952
SC8	1193.76	0.00	0.00	49.90	227.45	894.89	1122.34	44.89	5.61	0.940
SC5	1193.76	0.00	0.00	41.41	393.13	746.55	1139.68	9.92	1.32	0.955
SC6	1193.76	0.00	0.00	49.28	240.57	813.75	1054.32	31.63	2.77	0.883
SC7	1193.76	0.00	0.00	47.43	274.75	866.79	1141.54	11.42	1.98	0.956
SC3	1193.76	0.00	0.00	49.28	239.06	895.24	1134.30	27.22	4.02	0.950
SC2	1193.76	0.00	0.00	49.10	238.98	908.50	1147.48	18.36	3.55	0.961
SC1	1193.76	0.00	0.00	49.28	239.28	905.89	1145.17	31.61	5.50	0.959
SC4	1193.76	0.00	0.00	54.89	130.06	998.16	1128.22	35.20	5.27	0.945

Figure 4.4 Outlet one sub catchment runoff result

From figure 4.4 the simulation results the total area of the whole 13 sub catchments is 49.55 ha and produced the runoff from the area of all sub catchment is $315.42 \times 10^3 \text{ m}^3$ and the peak runoff generated at sub catchment (SC8) is $7.74 \text{ m}^3/\text{sec}$.

4.3.1.2 Junctions and channels flooded

Topic: Node Flooding <input type="button" value="v"/> 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
J1	1.65	1.722	0	00:20	2.967	0.000
J10	3.06	11.182	0	00:25	61.256	0.000
J13	3.06	11.831	0	00:22	60.828	0.000
J16	3.34	4.935	0	00:26	38.533	0.000
J17	0.32	1.338	0	00:20	1.026	0.000
J19	0.95	3.206	0	00:25	6.209	0.000
J7	3.04	5.284	0	00:15	22.795	0.000
J9	2.77	1.955	0	00:30	7.858	0.000
J23	1.02	3.211	0	00:26	6.897	0.000

Topic: Conduit Surcharge <input type="button" value="v"/> Click a column header to sort the column.					
Conduit	Hours Both Ends Full	Hours Upstream Full	Hours Downstream Full	Hours Above Normal Flow	Hours Capacity Limited
C10	0.57	3.11	0.57	3.06	0.57
C12	0.01	0.01	0.53	0.01	0.01
C13	0.01	0.01	3.44	0.01	0.01
C14	3.44	3.44	4.21	0.02	0.01
C15	0.32	0.32	3.64	0.01	0.01
C17	0.01	0.01	3.13	0.01	0.01
C18	1.67	2.31	1.67	2.10	1.67
C20	0.68	0.68	3.30	0.01	0.01
C27	1.02	3.06	1.02	2.23	0.01
C3	0.01	0.01	3.11	0.01	0.01
C28	0.01	0.01	3.05	0.01	0.01
C29	0.01	1.02	0.01	0.01	0.01
C4	0.01	0.01	3.08	0.01	0.01
C5	2.77	2.77	3.11	0.01	0.01
C6	3.13	3.13	3.43	0.03	0.01
C7	0.01	0.01	3.04	0.20	0.01
C8	0.01	3.04	0.01	3.30	0.01
C9	0.01	0.01	3.06	0.01	0.01

Figure 4.5 The outfall one flooded junctions and channels

Figure 4.5, in the network simulation status report, it was determined that the drainage system's channels and junctions frequently overflowed. Sections between junctions J1, J7, J9, J10, J13, J16, J17, J19, and J23 were found to be nodes of flooding, and links C10 to C11, C7

and C8, C9 and C27 overflowed because they had a limited capacity and were unable to convey flow inside the channels during high rainy season.

4.3.1.3 Water Profile and Outfall Charged

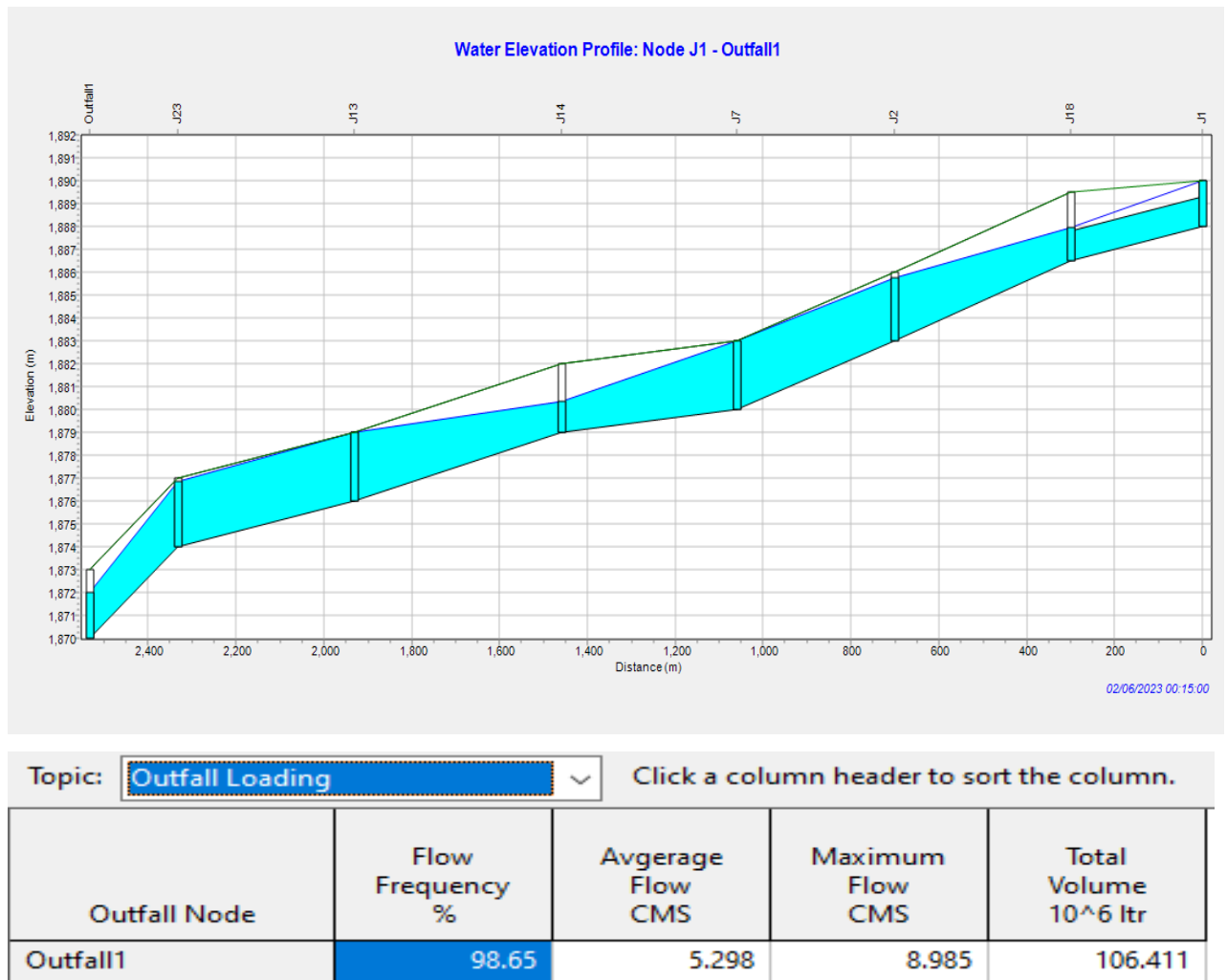


Figure 4.6 Outlet one water profile and discharge

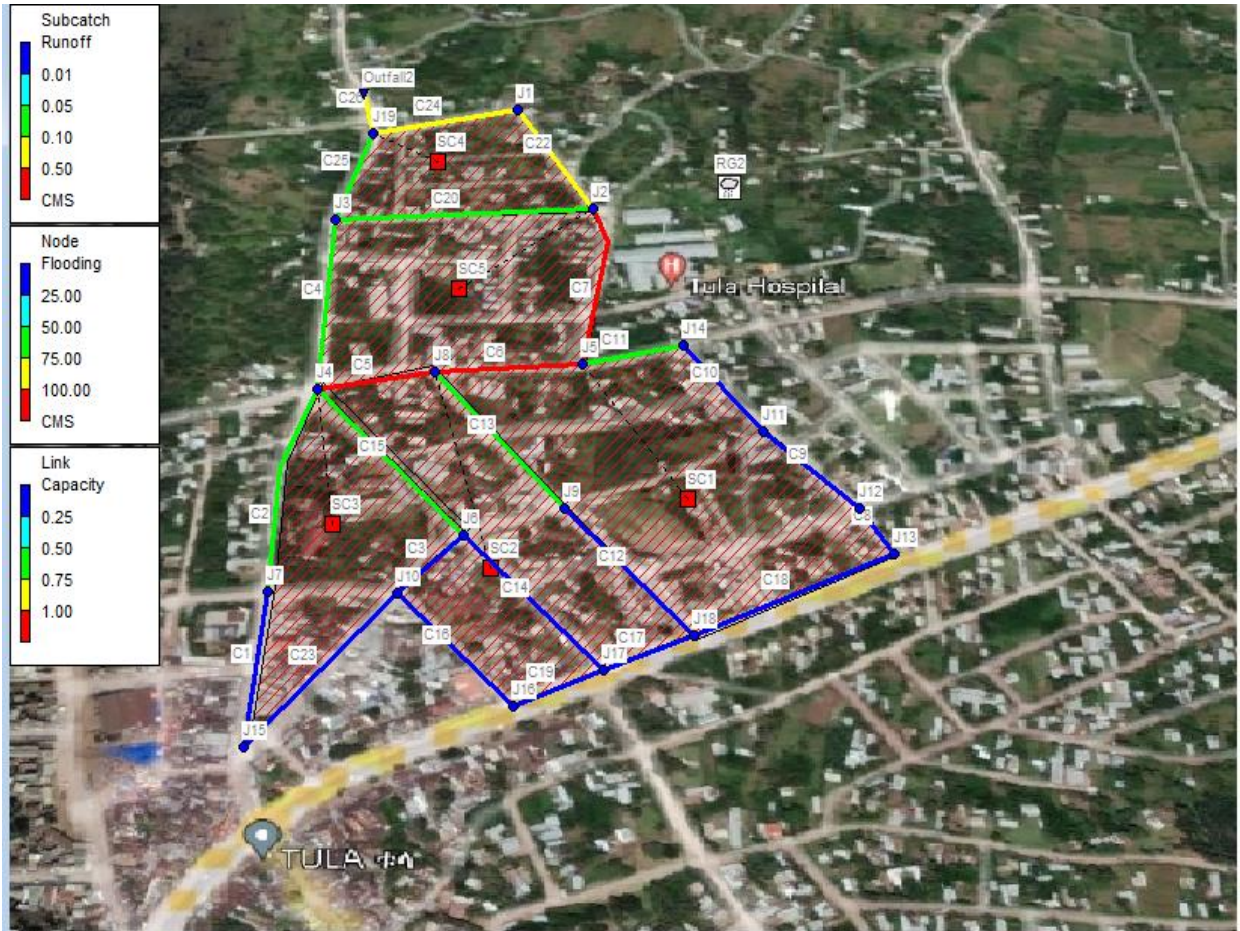
The figure 4.6 view of the each sub catchment used simulated with water profile plot is obtained. From this network profile of J1 to outlet one (O1) the water elevation is overflowed at J7, J13 and J23.

A sufficient junction or not flooded junction 18 and 14 are shown in figure 4.5 water elevation profile is sufficient hydraulic performance.

From the total simulated results of outfall node the average flow rate is 5.298m³/s, maximum flow rate at ends to outlet one is 8.985m³/s, and total volume of runoff is 106.411*10³ m³.

4.3.2 Result of Outlet Two

4.3.2.1 Sub Catchments Runoff



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

Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Imperv Runoff mm	Perv Runoff mm	Total Runoff mm	Total Runoff 10 ⁶ ltr	Peak Runoff CMS	Runoff Coeff
SC1	1164.76	0.00	0.00	37.06	467.25	651.47	1118.72	57.28	8.02	0.960
SC2	1164.76	0.00	0.00	26.22	674.07	464.91	1138.98	46.93	8.10	0.978
SC3	1164.76	0.00	0.00	31.26	576.86	555.35	1132.21	30.91	5.33	0.972
SC4	1164.76	0.00	0.00	44.04	333.75	773.90	1107.65	29.35	4.02	0.951
SC5	1164.76	0.00	0.00	29.25	614.23	521.25	1135.48	43.15	8.06	0.975

Figure 4.7 Sub catchment Runoff at outlet two

From the figure 4.7 simulation results 5 sub catchments of total area 21ha at outlet two studies. The runoff from whole area of sub catchment by SWMM5.2 model is 207.62m³ and the peak runoff generated at sub catchment (SC5) is 8.06m³/sec.

4.3.2.2 Junctions and Channels Flooded

From the figure 4.8 results, it was observed that the drainage system channels and nodes overflow in the network simulation status report shows that sections between junctions J2, J4, J5 and J8 are nodes flooded.

Topic: Node Flooding <input type="button" value="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
J2	2.20	5.628	0	00:20	16.391	0.000
J4	2.78	4.323	0	00:25	17.205	0.000
J5	3.05	7.353	0	00:30	44.959	0.000
J8	1.46	4.826	0	00:15	13.063	0.000

Topic: Conduit Surcharge <input type="button" value="Click a column header to sort the column."/>					
Conduit	Hours Both Ends Full	Hours Upstream Full	Hours Downstream Full	Hours Above Normal Flow	Hours Capacity Limited
C8	0.01	2.78	0.01	0.01	0.01
C5	1.74	1.74	3.04	0.01	0.01
C6	1.74	1.74	3.24	0.01	0.01
C12	0.01	2.21	0.01	0.01	0.01
C3	2.21	3.05	2.21	0.03	0.01
C4	0.01	2.21	0.01	0.01	0.01

Figure 4.8 Junction and Channels flooded at outfall two

The channels overflows occurred are C3, C4, C5, C6, C8 and C12 which are limited their capacity to convey flow in the channels infrastructure constructed

4.3.2.3 Water profile and outfall charged

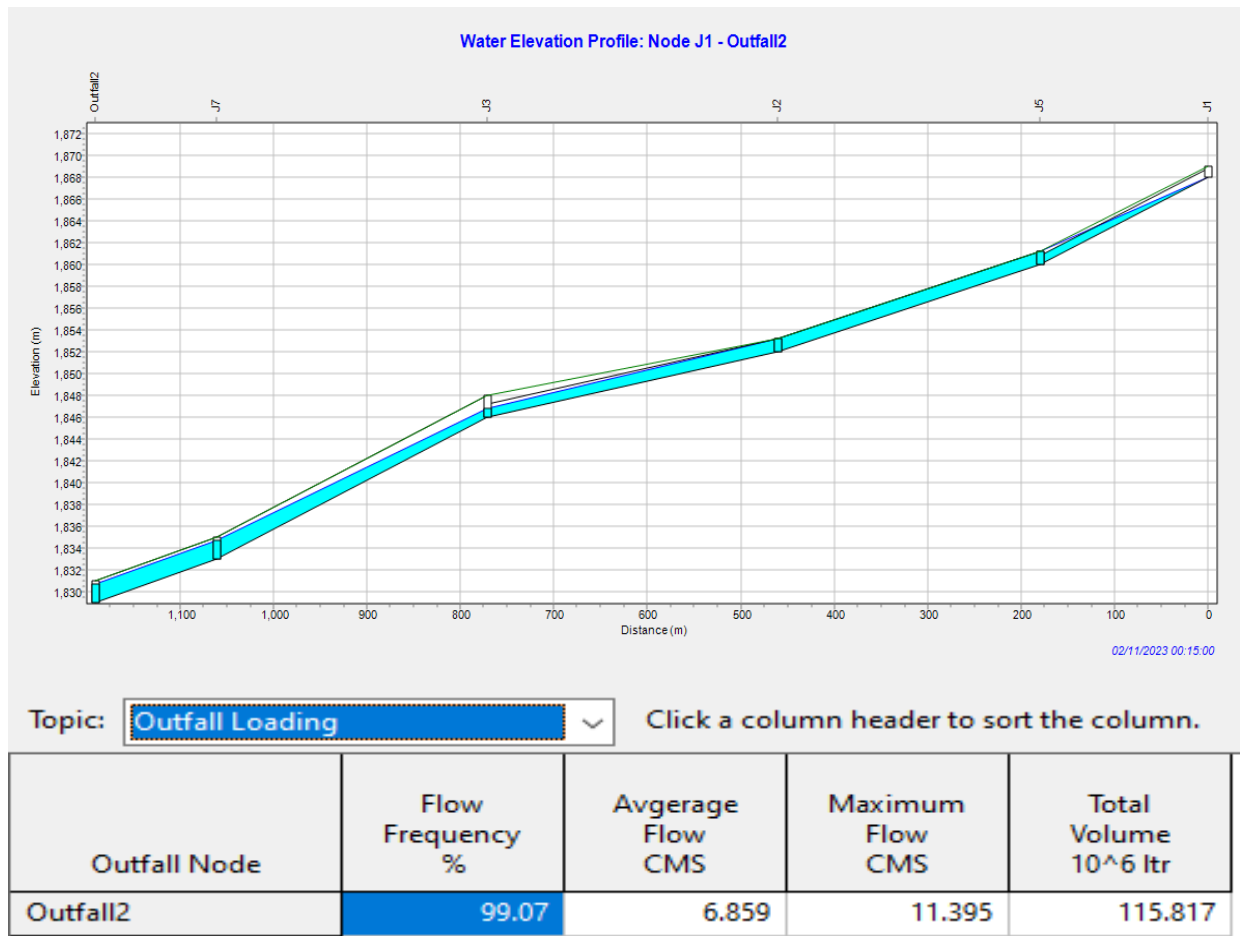


Figure 4.9 Outlet two water elevation profile and outfall discharge

From the figure 4.9 shows the water profile plot is obtained from J1 to outlet two (O2) the maximum flow rate at ends of this outlet is 11.395m³/s and total volume 115.817*10³m³.

By using the SWMM5.2 model, the hydraulic performance capacity from the complete drainage system is evaluated. The simulation of the runoff produced by each sub-catchment and flow routed through the network of channels and junction equipment built under various working conditions allows for a thorough evaluation of the results of a flooded drainage system.

Table 4.7: Outflow discharge and volume of runoff of each outfall

Outfall nodes Names	Average flow rate (m ³ /sec)	Maximum flow rate in m ³ /sec	Total volume to outlet *10 ³ m ³
Outlet one	5.298	8.985	106.411
Outlet two	6.859	11.395	115.817
Overall outfalls results	6.0785	11.395	222.228

The generally study simulated results in table 4.7 all two outlets the total simulation area of sub catchment is 70.55ha (0.7055km²), the combined drainage system are 37 nodes and 48 channels with total length canal flow routed through 15.48km.

Overall outfalls results in table 4.6 showed that 6.0785m³/sec average flow rate, 11.395m³/sec maximum flow rate and 222.228*10³m³ total volume runoff collective drainage systems involved each regular or irregular shape simulation applied.

4.3.3 Simulation of Low-Impact Development

Low impact development is regarded as one of the most efficient methods for ensuring proper urban storm water management in order to enhance the amount and quality of stormwater.

LID practices reduce the amount of stormwater reaching a surface water system thus helping to maintain natural stream channel functions and habitat. Groundwater recharge stormwater runoff that travels through pipes and drains into surface waters cannot soak into the ground.

LID basically aims to mimic the natural hydrological conditions of a specific location as a storm water management strategy. SWMM gives the chance to assess how various LID measures may affect a drainage system. The major goal of putting structural LID measures into place in the model area is to absorb a significant amount of stormwater runoff that is being produced by rainfall to infiltrate into the ground and the remaining flood to Tulo Woyo road ditches. Since the majority of the sources of excessive runoff originate in the upper part of the town, it is suggested that LID measures be built as soon as possible before stormwater runoff reaches religious, residential and commercial areas.

Three types of LID that is Bio retention, Infiltration trench and Vegetative swale measures were used in different sub catchments such as SC2, SC4, SC8 and SC9 in the model area one and SC1 and SC5 in model area two. These mentioned LIDs were chosen due to their unique ability to filter contaminants and significantly reduce stormwater runoff. The physical characteristics of the sub catchment have been used to specify the parameters of three LID constructions. Following model modeling, the effect of LID application on the drainage system is investigated.

4.3.4 Results of LID Development with SWMM 5.2

Table 4.8: SWMM5.2 results without and with LID for model area one

Item	Without LID	With LID					
		Bio-retention Cell		Infiltration trench		Vegetable Swales	
		Value	%	Value	%	Value	%
Peak runoff (m ³ /s)	8.985	8.62	4.23	8.76	2.56	8.81	1.98
Total volume runoff at outlet one (10 ³ m ³)	106.411	90.1	18.1	97.3	9.36	100.3	6.09

The above table 4.8 SC2, SC4, SC8, and SC9 reduced the total outlet one volume by 18.1% using bio-retention cells, 9.36% using infiltration trenches, and 6.09% using vegetable swales. Hence, for this study, choose the best LID control option to evaluate the drainage system problem mitigation methods that are implemented utilizing the appropriate LID control kinds. According to the findings in table 4.8, the Bio-retention cell and Infiltration trench are the most advised.

Table 4.9: SWMM5.2 results without and with LID for model area two

Item	Without LID	With LID					
		Bio-retention Cell		Infiltration trench		Vegetable Swales	
		Value	%	Value	%	Value	%
Peak runoff (m ³ /s)	11.395	10.2	1.17	9.3	22.5	9.86	15.56
Total volume runoff at outlet one (10 ³ m ³)	115.817	107.4	7.83	96.6	19.89	101.76	13.81

According to the findings in table 4.9 reduced the total outlet two area volume by 7.83% using bio-retention cells, 19.89% using infiltration trenches, and 13.81% using vegetable swales.

Therefore, the Infiltration trench and Vegetative swales are the most advised in model area 2

4.3.5 Modeled Area of LID Control

The two outfall networks of the LID control planned area are applied to the sub catchment applying bio retention, vegetative swales, and infiltration trench methods. The runoff flow results from each sub catchment in the system determine the sub catchments that are chosen.

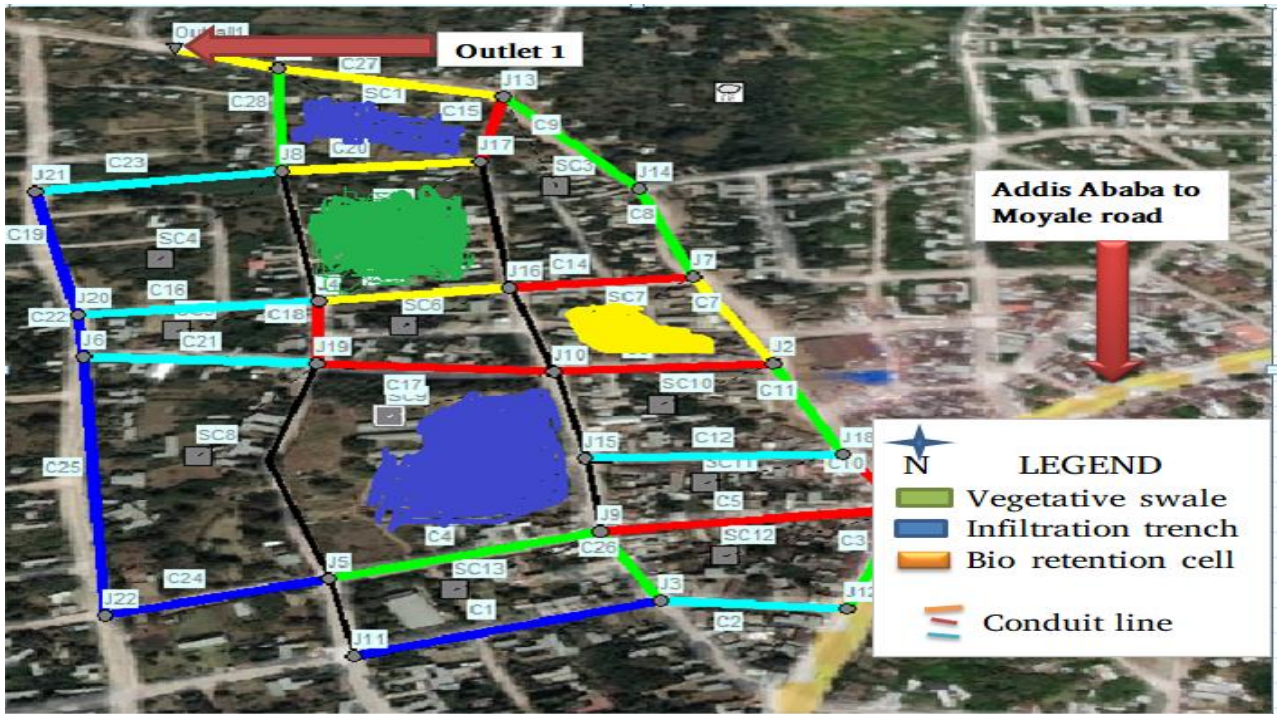


Figure 4.10 Outfall one study of LID control area modeled

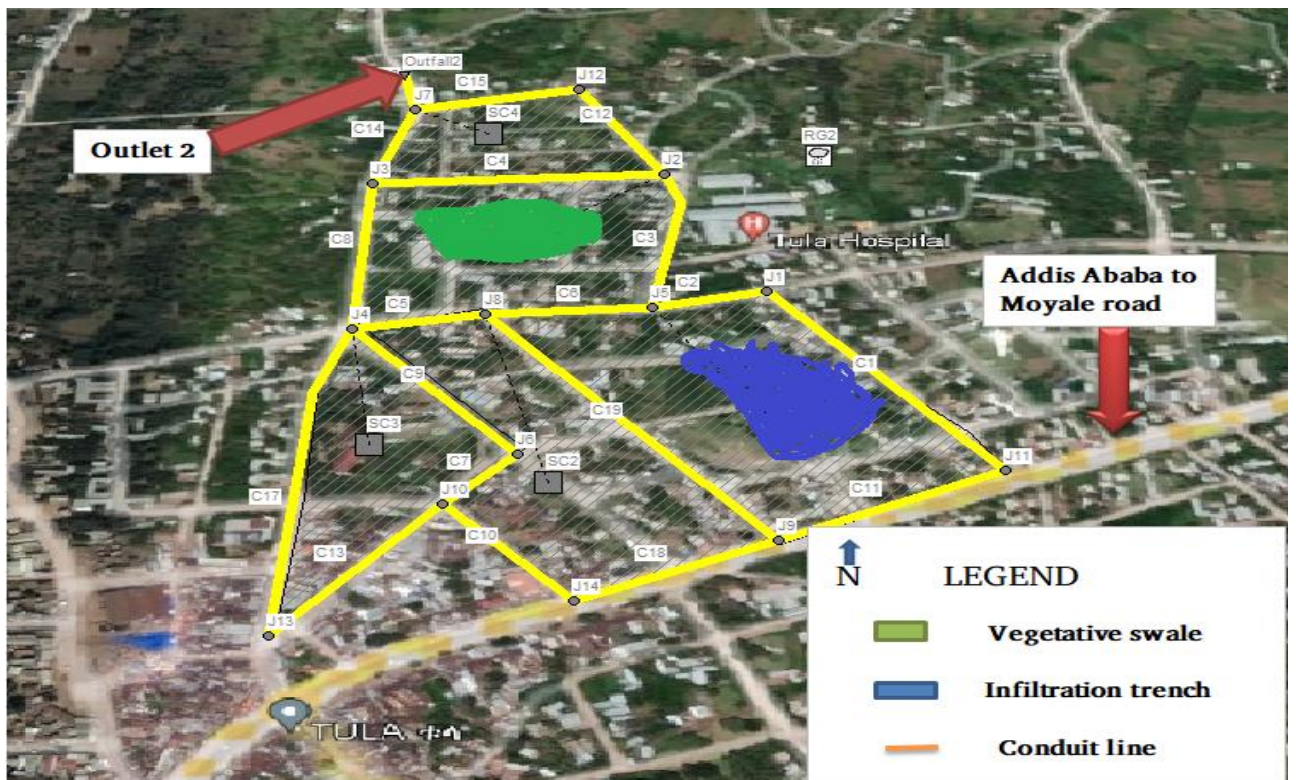


Figure 4.11 Outfall two study of LID control area modeled

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The study assessed and modeled the evaluation of Tula's urban drainage system using the SWMM5.2 model, which evaluated stormwater drainage infrastructures, assessed the evaluation of urban drainage systems, and also assessed alternatives for mitigating drainage issues. The calibration and validation were applied for 10 days of observed flow depth and the model result was compared. The hydraulic and hydrological evaluation of drainage was simulated using EPA SWMM5.2 with and without LID.

The flooding risk is very high due to the drainage system is undersized to cope with the current rainfall rates, but also is very limited to face the upcoming predicted rainfall. The most affected areas are the main line and ring road drainage system, although other singular spots are also likely to be affected by flooding.

For calibration 5 days flow depth data recorded is calculated by the manning equation is $5.1\text{m}^3/\text{s}$ and compared by adjusted sensitive parameters of model results $5.27\text{m}^3/\text{s}$ on the same site are almost equivalent. The other 5 days flow depth data used for validation without changed the sensitive parameters of model result is parallel with observed are verified.

The SWMM5.2 model performance for the selected area was tested by the goodness of fit using the coefficient of determination (R^2) =0.98, the Nash–Sutcliffe Efficiency (NSE) =0.91, and Relative error (RE) =24.13% which are simulated values indicated in an acceptable range.

The simulated area in this study is subdivided in to two outlets of the total area of sub catchment is 70.55ha the combined drainage system are 37 nodes and 48 channels with total length canal flow routed through 15.48 km. According to simulated result greater than 50% of drainage infrastructures are over flooded. From the result of SWMM5.2 simulated all total sub catchment average flow rate $6.0785\text{ m}^3/\text{s}$, maximum flow rate $11.39\text{m}^3/\text{s}$ and total volume of all outfall $222.228 \times 10^3\text{ m}^3$.

The LID control used in this study for reduced the peak runoff overflow and select the best alternative on the outlet one of the project in the same sub-catchments SC2, SC4, SC8, SC9, were the reduced the total runoff at outlet one by 18.1 % using bio-retention Cell, 9.36 % infiltration trench and 6.09 % vegetable swales for simulation area 1 and 7.83% using bio retention cell, 19.89 % using infiltration trench, 13.81% using vegetative swales. From all select highest value of runoff decreased by bio-retention cell and infiltration trench for total study applied.

5.2 Recommendations

The following suggestions are given for an improved and sustainable urban stormwater drainage system in order to address the issues that have been hindering drainage systems in this study area.

- ✓ Upgrading the drainage system layout as per to the master plan of Tula town and applying the updated IDF curve for town and urban stormwater drainage design manual of the country to assessing accurate measurement and improve storm drainage systems of the area
- ✓ Encourage the creation of parks, reserves and Special Areas of Environmental Interest indispensable for the preservation of flood and increase permeability.
- ✓ Create awareness within the community to use the drainage systems in a way that the drainage systems could be able to serve as their life span and the community should also know how to manage solid and liquid wastes.
- ✓ Detail feasibility and design study should be done before approval of the drainage design during early stage of the project.
- ✓ Supervision of drainage canals during construction should be mandatory to avoid wrong dimensions and quality of the work as a whole.
- ✓ This study properly considered only the effect of LID locations and sizes in the modeling processes by introducing the relative performance of each LID. Therefore when planning and designing LID systems considering the first count the cost and decide if it is difficult to use another alternative or if it is improvement connecting the LID techniques in the all drainage network to reduce the overflow runoff.
- ✓ A given conduit's end and another conduit's start cannot be linked in the number 90^0 . This is due to the fact that while water flows at a fast speed, it loses speed, clogging the drainage system and preventing stormwater from moving through. Other waste products such as silt, debris, and pieces of broken wood also become stuck in the water and obstruct the drainage system. The drainage system for the Tula town must therefore be designed with the necessary care and research.

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APPENDIX

Appendix Table 1: Double mass curve after consistency checked

Double Mass Curve Analysis											
Year	Annual RF of Tula Town in mm	Commulative annual RF OF Tula town	Hawassa station	Wondo genet station	Morocho station	Average RF of the neighboring stations	Average commulative RF of nnear stations(mm)	Adjusted RF of Tula station	Adjusted commulative RF of Tula station	Adjusted RF of Tula station	Adjusted commulative RF of Tula station
2021	30.4	30.4	26.4	61.1	50	45.83333333	45.83333	42.73292	42.73292	51.73291687	51.73292
2020	40.8	71.2	50.8	60.1	48.9	53.26666667	99.1	57.35207	100.085	65.35207264	117.085
2019	36.9	108.1	70.2	59.2	49.3	59.56666667	158.6667	51.86989	151.9549	70.86988923	187.9549
2018	29.7	137.8	51.8	70.4	49.4	57.2	215.8667	44.43982	196.3947	75.43982451	263.3947
2017	40.3	178.1	55.5	50.9	50.1	52.16666667	268.0333	60.3005	256.6952	78.30050262	341.6952
2016	48.4	226.5	49	43.4	36.5	42.96666667	311	54.97586	311.6711	64.97586047	406.6711
2015	46.1	272.6	48.3	29.1	34.4	37.26666667	348.2667	52.36337	364.0344	52.36337123	459.0344
2014	43	315.6	69.8	36.4	32.3	46.16666667	394.4333	48.84219	412.8766	80.84219009	539.8766
2013	88.8	404.4	43.3	76.3	33.7	51.1	445.5333	161.1299	574.0065	84.12987906	624.0065
2012	51	455.4	37.6	39.1	50.7	42.46666667	488	51	625.0065	61	685.0065

2011	56.6	512	33.7	91.7	51.9	59.1	547.1	56.6	681.6065	68.6	753.6065
2010	40.7	552.7	53.6	60.1	38.5	50.73333333	597.8333	40.91889	722.5254	66.91889153	820.5254
2009	33.3	586	39.6	25.2	43	35.93333333	633.7667	33.47909	756.0045	53.47909307	874.0045
2008	50.5	636.5	32.8	52.2	68.3	51.1	684.8667	50.7716	806.7761	55.7715976	929.7761
2007	32.2	668.7	71.5	41.9	51.1	54.83333333	739.7	51.84779	858.6239	71.84778947	1001.624
2006	29.3	698	57.6	40.8	47.3	48.56666667	788.2667	47.17827	905.8021	57.17826806	1058.802
									max	84.12987906	1058.802
									min	51.73291687	51.73292

Appendix Table 2 Data quality check by relative standard error

Year	X=Maximum rainfall	Y=log(X)	Y-Ym	(Y-Ym)^2	(Y-Ym)^3	(X-Xm)^2
2006	57.1783	1.75723	-0.0584	0.00341	-0.00020	80.9436
2007	71.8478	1.85641	0.04075	0.00166	0.00007	32.1790
2008	55.7716	1.74641	-0.06925	0.00480	-0.00033	108.2336
2009	53.4791	1.72818	-0.08748	0.00765	-0.00067	161.1895
2010	66.9189	1.82555	0.00988	0.00010	0.00000	0.5532
2011	68.6000	1.83632	0.02066	0.00043	0.00001	5.8800
2012	61.0000	1.78533	-0.03034	0.00092	-0.00003	26.7820
2013	84.1299	1.92495	0.10928	0.01194	0.00131	322.3729
2014	80.8422	1.90764	0.09197	0.00846	0.00078	215.1225
2015	52.3634	1.71903	-0.09664	0.00934	-0.00090	190.7648
2016	64.9759	1.81275	-0.00291	0.00001	0.00000	1.4383
2017	78.3005	1.89376	0.07810	0.00610	0.00048	147.0246
2018	75.4398	1.87760	0.06193	0.00384	0.00024	85.8345
2019	70.8699	1.85046	0.03480	0.00121	0.00004	22.0407
2020	65.3521	1.81526	-0.00041	0.00000	0.00000	0.6774
2021	51.7329	1.71377	-0.10190	0.01038	-0.00106	208.5776
Sum	1058.8021	29.05067	0.00000	0.07025	-0.00027	1609.6141
AVG	66.1751	1.81567				
Xm	66.1751					
Ym	1.8157					
Sd Xi(sx)	10.3589					
Sd Yi(sy)	0.0684					
Skewnes Cof(cs)	-0.0649					

Relative standard error = Se/Xm

$Se = \text{standard deviation} / (\text{numb of yr})^{0.5} = 10.3589 / (16^{0.5}) = 2.589735$ relative standard error = $(2.589735 / 66.1751) * 100 = 3.913456\% < 10\%$ ok

there for the data id reliable and adequate

Appendix Table 3: Frequency Analysis and IDF Development

Year	24hr max Annual RF	5min = 0.08hr	10min = 0.17hr	15min = 0.25hr	30min = 0.5hr	1hr	2hr	3hr	6hr	12hr	24hr
2006	57.17827	8.311229	13.92541	18.01686	25.71902	33.22893	39.7686	43.10465	48.17372	52.76331	57.17827
2007	71.84779	10.44354	17.49808	22.63922	32.31743	41.75407	49.97154	54.16348	60.53305	66.30014	71.84779
2008	55.7716	8.10676	13.58283	17.57361	25.08629	32.41145	38.79024	42.04421	46.98857	51.46526	55.7716
2009	53.47909	7.77353	13.0245	16.85125	24.05512	31.07917	37.19576	40.31597	45.0571	49.34976	53.47909
2010	66.91889	9.727091	16.29768	21.08612	30.10039	38.88966	46.5434	50.44776	56.38037	61.75182	66.91889
2011	68.6	9.971451	16.7071	21.61584	30.85656	39.86663	47.71264	51.71508	57.79673	63.30313	68.6
2012	61	8.866742	14.85617	19.22108	27.43805	35.44992	42.42669	45.98572	51.39359	56.28995	61
2013	84.12988	12.22882	20.48931	26.5093	37.84197	48.89176	58.51398	63.4225	70.88093	77.63388	84.12988
2014	80.84219	11.75093	19.68861	25.47335	36.36315	46.98113	56.22732	60.94403	68.111	74.60005	80.84219
2015	52.36337	7.611352	12.75277	16.49968	23.55326	30.43077	36.41975	39.47487	44.11708	48.32019	52.36337
2016	64.97586	9.444659	15.82447	20.47387	29.22641	37.76047	45.19198	48.98297	54.74333	59.95882	64.97586
2017	78.3005	11.38148	19.0696	24.67247	35.21989	45.50404	54.45953	59.02794	65.96958	72.25462	78.3005
2018	75.43982	10.96566	18.3729	23.77107	33.93314	43.84157	52.46987	56.87138	63.55941	69.61482	75.43982
2019	70.86989	10.30139	17.25992	22.33108	31.87757	41.18577	49.29139	53.42627	59.70915	65.39775	70.86989
2020	65.35207	9.499344	15.91609	20.59242	29.39563	37.97911	45.45364	49.26659	55.06029	60.30599	65.35207
2021	51.73292	7.519712	12.59923	16.30103	23.26968	30.06439	35.98126	38.99959	43.58591	47.73842	51.73292
SUM		153.9037	257.8647	333.6282	476.2536	615.3188	736.4176	798.193	892.0598	977.0479	1058.802
mean(Xm)		9.618981	16.11654	20.85177	29.76585	38.45743	46.0261	49.88706	55.75374	61.06549	66.17513
st.dev (Sx)		1.505738	2.522855	3.264098	4.659493	6.020058	7.204844	7.809233	8.727592	9.559085	10.35894
skewness coeff. (Cs)		-0.06494	-0.06494	-0.06494	-0.06494	-0.06494	-0.06494	-0.06494	-0.06494	-0.06494	-0.06494

Appendix Table 4 Reduction daily precipitation for each time t

given t	Rt	t in hr.
5 min	0.14535642	0.08333333
10 min	0.24354379	0.16666667
15 min	0.3150997	0.25
30 min	0.44980411	0.5
1 hr	0.58114618	1
2 hr	0.69551956	2
3 hr	0.75386419	3
6 hr	0.84251795	6
12 hr	0.92278611	12
24 hr	1	24

Where as

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

Rt is Rain fall Reduction formula

n = 0.9 and b = 0.3 is content

Probability distribution and IDF development

There are two options for this frequency analysis these are:

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

2, Use empirical plotting position approach to estimate the exceedance probabilities based on observations.

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

PROBABILITY DISTRIBUTION AND GOODENSE FIT DISTRIBUTION						
Distribution	2 year	5 year	10 year	25 year	50 year	100 year
Gumble	41.84	51.75	58.31	66.59	72.74	78.84
Normal	22.03	53.11	58.05	63.316	66.713	69.76
Log-normal	25.21	52.95	59.58	67.56	73.26	78.81
Log-pearson type III	42.84	53.13	58.99	65.60	70.06	74.17
Pearson	44.241	53.25	57.636	62.084	64.842	67.23
Weibull's formula	40.74	56.22	57.57	60.7	60.7	60.7

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

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

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

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

Appendix Table 6: Tested Goodness of Fit by Probability Distribution

						Observed RF
Return period(T)	Gumbel(EVI)	Normal	Lognormal	Log-Pearson	Pearson typeIII	Weibull's Formula:
2	64.47	66.17	65.41	65.52	66.28	66.08
5	73.62	74.89	74.68	74.72	74.92	76.95
10	79.68	79.45	80.05	79.957	79.37	81.42
25	87.33	84.31	86.19	85.88	84.07	84.12
50	93.01	87.45	90.41	89.90	87.08	84.12
100	98.65	90.27	94.38	93.66	89.77	84.12

Appendix Table 7: Chi - Square Test Result in dimensionless of X2 values

Return period(T)	Gumbel(EVI)	Normal	Lognormal	Log-Pearson	Pearson typeIII
2	0.040467105	0.000111	0.0069864	0.0048234	0.000604
5	0.15055459	0.056805	0.06876938	0.0666528	0.055116
10	0.037942163	0.04884	0.02340799	0.026837	0.052881
25	0.117891269	0.000404	0.04964062	0.0360612	0.00
50	0.84922626	0.126376	0.43696142	0.3715101	0.100302
100	2.138484846	0.418721	1.11398791	0.9706106	0.355014
Total Ch-square result X ²	3.334566233	0.651256	1.69975373	1.4764951	0.563947

All probability distribution functions were compared by chi square test of goodness of fit And then selecting the function that gave the smallest chi-square value determined the best probability function.

Therefore for this study the best fit probability distribution is **Pearson type III**

Appendix Table 8: Rainfall depth (mm) by Pearson type III at time t (yrs.)

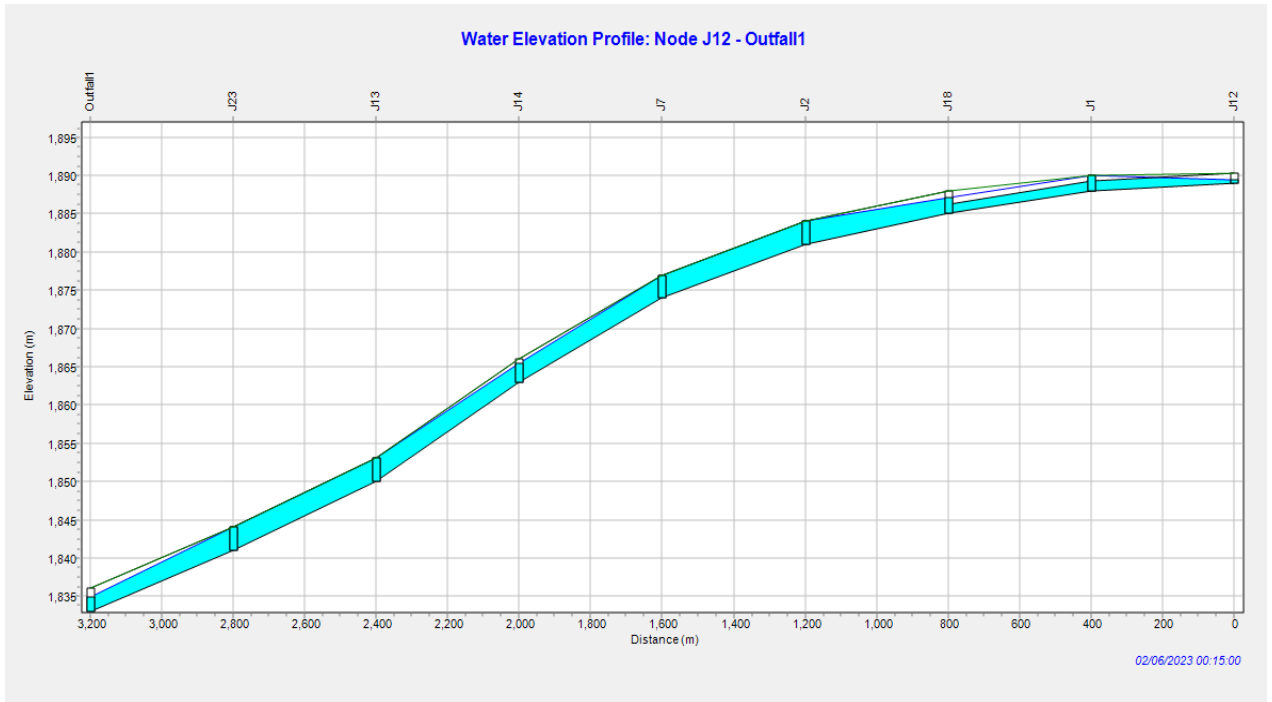
Rainfall depth (mm) by pearson type III distribution at time t (yrs.)							
Duration (min)	Duration (hr)	2 year	5 year	10 year	25 year	50 year	100 year
5	0.083333	6.43	7.74	8.38	9.02	9.42	9.77
10	0.166667	10.77	12.97	14.04	15.12	15.79	16.37
15	0.25	13.94	16.78	18.16	19.56	20.43	21.18
30	0.5	19.90	23.95	25.92	27.92	29.17	30.24
60	1	25.71	30.95	33.49	36.08	37.68	39.07
120	2	30.77	37.04	40.09	43.18	45.10	46.76
180	3	33.35	40.14	43.45	46.80	48.88	50.68
360	6	37.27	44.86	48.56	52.31	54.63	56.64
720	12	40.82	49.14	53.18	57.29	59.83	62.04
1440	24	44.24	53.25	57.63	62.08	64.84	67.23

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

Duration (min)	Duration (hr)	2 year	5 year	10 year	25 year	50 year	100 year
5	0.083333	77.16	92.88	100.52	108.28	113.09	117.2634
10	0.166667	64.64	77.81	84.21	90.71	94.74	98.23708
15	0.25	55.75	67.11	72.64	78.24	81.72	84.73
30	0.5	39.79	47.90	51.84	55.84	58.33	60.47
60	1	25.70	30.94	33.49	36.07	37.68	39.06
120	2	15.38	18.51	20.04	21.58	22.54	23.37
180	3	11.11	13.38	14.48	15.60	16.29	16.89
360	6	6.21	7.47	8.09	8.71	9.10	9.44
720	12	3.40	4.09	4.43	4.77	4.98	5.16
1440	24	1.84	2.21	2.40	2.58	2.70	2.80

The IDF developed in appendix 9 for given return period is used in SWMM 5.2 as input data

Appendix Table 10: Flow profile for flooding junction



Appendix Table 11: Summary result sub catchment runoff

Topic: Click a column header to sort the column.

Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Imperv Runoff mm	Perv Runoff mm	Total Runoff mm	Total Runoff 10 ^{^6} ltr	Peak Runoff CMS	Runoff Coeff
SC9	1193.76	0.00	0.00	49.28	240.46	830.31	1070.77	40.80	3.79	0.897
SC10	1193.76	0.00	0.00	36.34	491.07	572.96	1064.03	41.50	4.26	0.891
SC11	1193.76	0.00	0.00	49.28	240.08	863.86	1103.94	36.43	3.85	0.925
SC12	1193.76	0.00	0.00	45.58	311.22	824.74	1135.96	8.86	1.23	0.952
SC13	1193.76	0.00	0.00	48.05	262.99	873.55	1136.53	17.16	2.58	0.952
SC8	1193.76	0.00	0.00	49.90	227.45	894.89	1122.34	44.89	5.61	0.940
SC5	1193.76	0.00	0.00	41.41	393.13	746.55	1139.68	9.92	1.32	0.955
SC6	1193.76	0.00	0.00	49.28	240.57	813.75	1054.32	31.63	2.77	0.883
SC7	1193.76	0.00	0.00	47.43	274.75	866.79	1141.54	11.42	1.98	0.956
SC3	1193.76	0.00	0.00	49.28	239.06	895.24	1134.30	27.22	4.02	0.950
SC2	1193.76	0.00	0.00	49.10	238.98	908.50	1147.48	18.36	3.55	0.961
SC1	1193.76	0.00	0.00	49.28	239.28	905.89	1145.17	31.61	5.50	0.959
SC4	1193.76	0.00	0.00	54.89	130.06	998.16	1128.22	35.20	5.27	0.945

Appendix Table 12: Summary Result of link flow at each junction

Topic: Link Flow		Click a column header to sort the column.					
Link	Type	Maximum [Flow] CMS	Day of Maximum Flow	Hour of Maximum Flow	Maximum [Velocity] m/sec	Max / Full Flow	Max / Full Depth
C1	CONDUIT	0.000	0	00:00	0.00	0.00	0.10
C2	CONDUIT	0.181	0	00:27	0.41	0.14	0.55
C3	CONDUIT	0.579	0	00:14	1.04	0.51	0.95
C4	CONDUIT	0.150	0	01:07	0.52	0.21	0.51
C5	CONDUIT	0.704	0	01:08	0.92	0.55	1.00
C11	CONDUIT	1.545	0	01:32	1.72	1.11	1.00
C13	CONDUIT	0.000	0	00:00	0.00	0.00	0.50
C6	CONDUIT	2.293	0	00:10	2.84	1.03	1.00
C12	CONDUIT	4.882	0	00:17	1.93	0.75	0.86
C7	CONDUIT	8.885	0	00:42	3.04	1.03	1.00
C8	CONDUIT	10.911	0	01:09	4.50	1.01	0.93
C9	CONDUIT	10.850	0	01:10	3.89	0.93	0.93
C22	CONDUIT	0.200	0	00:30	0.58	0.14	0.61
C16	CONDUIT	7.007	0	00:17	4.36	0.56	0.80
C15	CONDUIT	1.833	0	00:11	2.08	0.82	1.00
C14	CONDUIT	0.795	0	03:14	1.44	1.09	1.00
C18	CONDUIT	0.609	0	00:33	1.09	0.84	1.00
C19	CONDUIT	4.426	0	00:12	5.72	1.07	1.00
C21	CONDUIT	0.000	0	00:00	0.00	0.00	0.50
C17	CONDUIT	0.000	0	00:00	0.00	0.00	0.50
C23	CONDUIT	0.000	0	00:00	0.00	0.00	0.50
C24	CONDUIT	0.000	0	00:00	0.00	0.00	0.00
C25	CONDUIT	4.310	0	00:30	12.99	0.14	0.61
C26	CONDUIT	0.150	0	00:31	9.89	0.00	0.51

Appendix Table 13: Summary Result of node flow at each junction

Topic: Node Inflow		Click a column header to sort the column.						
Node	Type	Maximum Lateral Inflow CMS	Maximum Total Inflow CMS	Day of Maximum Inflow	Hour of Maximum Inflow	Lateral Inflow Volume 10 [^] 6 ltr	Total Inflow Volume 10 [^] 6 ltr	Flow Balance Error %
J1	JUNCTION	4.200	4.784	0	00:15	18.6	29	-0.057
J10	JUNCTION	6.786	7.395	0	00:35	49.3	54.3	0.008
J11	JUNCTION	0.000	0.000	0	00:00	0	0	0.000
J12	JUNCTION	0.000	0.595	0	00:16	0	2.02	0.152
J13	JUNCTION	6.396	24.062	0	00:20	33.1	183	-0.029
J14	JUNCTION	0.000	10.911	0	01:09	0	114	0.022
J15	JUNCTION	0.000	0.000	0	00:00	0	0	0.000
J16	JUNCTION	2.291	2.951	0	00:10	17.3	26.7	0.029
J17	JUNCTION	6.909	7.025	0	00:15	34	35.3	0.024
J18	JUNCTION	4.294	5.746	0	00:10	15.9	34	0.053
J2	JUNCTION	5.027	11.293	0	00:16	23.3	81.1	0.015
J19	JUNCTION	7.735	7.735	0	00:25	45.9	45.9	-0.017
J20	JUNCTION	0.000	0.000	0	00:00	0	0	0.000
J21	JUNCTION	0.000	4.310	0	00:30	0	28.8	0.776
J22	JUNCTION	0.000	0.150	0	00:31	0	1.55	16.708
J3	JUNCTION	0.000	0.315	0	00:17	0	1.75	0.134
J4	JUNCTION	1.699	5.858	0	01:09	13.8	49.1	-0.053
J5	JUNCTION	0.000	0.150	0	01:07	0	1.55	-0.022
J6	JUNCTION	0.000	0.000	0	00:00	0	0	0.000
J7	JUNCTION	5.437	15.274	0	00:14	21.1	121	0.042
J8	JUNCTION	5.274	5.274	0	00:30	35.2	35.2	0.019
J9	JUNCTION	2.643	2.643	0	00:25	16.8	16.8	-0.016
J23	JUNCTION	5.377	15.775	0	00:25	29.1	155	0.044
Outfall1	OUTFALL	0.000	8.793	0	00:13	0	110	0.000

Appendix Table 14 Measured Input Parameter of Model in Outfall 1 and 2

Name of junction code	Invert elevation of nodes	Name of canal code	Inlet node of canal	Outlet node of canal	Elevation b/n nodes	Channel demission parameters					Channel slope= Elev/length	Roughness depend on material (N)	
						Type of channel	Bottom width (m)	Top width (m)	Max depth (m)	Side slope H/V			Length conduit (m)
J1	1888	C10	J1	J18	1.5	Closed Rectangular	1	1	1.3		300	0.005	0.018
J2	1883	C7	J2	J7	3	Closed Rectangular	1	1	3		360	0.0083	0.018
J3	1891.5	C2	J3	J12	2.5	Rectangular	0.8	0.8	0.8		190	0.0132	0.02
J4	1885	C13	J4	J16	3	Trapezoidal	0.7	1	0.8	0.18	204	0.0147	0.022
J5	1890	C4	J5	J9	1	Rectangular	0.7	0.7	0.8		430	0.0023	0.02
J6	1888.5	C21	J6	J19	3.5	Trapezoidal	0.7	1	0.8	0.18	380	0.0092	0.022
J7	1880	C8	J7	J14	1	Closed Rectangular	1	1	3		400	0.0025	0.018
J8	1880	C28	J8	J23	6	Rectangular	1	1	2		300	0.02	0.018
J9	1889	C5	J9	J1	1	Rectangular	0.7	0.7	1.3		407	0.0024	0.018
J10	1884	C6	J10	J2	1	Trapezoidal	0.7	1	0.8	0.18	380	0.0026	0.022
J11	1892	C1	J11	J3	0.5	Rectangular	0.7	0.7	1.3		391	0.00127	0.025
J12	1889	C3	J12	J1	1	Closed Rectangular	0.7	0.7	1.3		310	0.0032	0.024
J13	1876	C27	J13	J23	2	Closed Rectangular	1	1	3		403	0.0049	0.024
J14	1879	C9	J14	J13	3	Closed Rectangular	1	1	3		470	0.0063	0.018
J15	1888	C12	J15	J18	1.5	Trapezoidal	0.8	1.2	0.7	0.285	396	0.0037	0.022
J16	1882	C14	J16	J7	2	Rectangular	0.7	0.7	0.8		350	0.0057	0.02
J17	1878	C15	J17	J13	2	Rectangular	1	1	2		140	0.0143	0.02
J18	1886.5	C11	J18	J2	3.5	Closed Rectangular	1	1	3		400	0.0087	0.024
J19	1885.5	C18	J19	J4	0.5	Trapezoidal	0.8	1.2	0.6	0.33	110	0.0045	0.024

J20	1888	C19	J20	J21	2	Rectangular	0.6	0.6	0.8		460	0.0043	0.02
J21	1886	C23	J21	J8	6	Rectangular	0.7	0.7	0.8		420	0.0142	0.025
J22	1891	C25	J22	J6	2.5	Rectangular	0.7	0.7	0.8		600	0.0041	0.025
J23	1874	C29	J23	OF	4	Rectangular	1	1	3		200	0.02	0.02
		C16	J20	J4	3	Rectangular	0.7	0.7	0.8		387	0.0077	0.025

Name of junction code	Invert elevation of nodes	Name of canal code	Inlet node of canal	Outlet node of canal	Elevation b/n nodes	Channel demission parameters						Channel slope= Elev/length	Roughness depend on material (N)
						Type of channel	Bottom width (m)	Top width (m)	Max depth (m)	Side slope H/V	Length conduit (m)		
J1	1841	C15	J1	J7	8	Rectangular	1	1	2		200	0.005	0.018
J2	1852	C12	J2	J1	11	Closed Rectangular	1	1	3		360	0.0083	0.018
J3	1846	C14	J3	J7	13	Rectangular	0.8	0.8	0.8		190	0.0132	0.02
J4	1862	C8	J4	J3	16	Trapezoidal	0.7	1	0.8	0.18	204	0.0147	0.022
J5	1860	C3	J5	J2	8	Rectangular	0.7	0.7	0.8		430	0.0023	0.02
J6	1876	C9	J6	J4	14	Trapezoidal	0.7	1	0.8	0.18	380	0.0092	0.022
J7	1833	C16	J7	OF	4	Closed Rectangular	1	1	3		400	0.0025	0.018
J8	1863.5	C5	J8	J4	1.5	Rectangular	1	1	2		300	0.02	0.018
J9	1879.5	C19	J9	J8	16	Rectangular	0.7	0.7	1.3		407	0.0024	0.018
J10	1879	C7	J10	J6	3	Trapezoidal	0.7	1	0.8	0.18	380	0.0026	0.022
J11	1874	C1	J11	J12	6	Rectangular	0.7	0.7	1.3		391	0.00127	0.025
J12	1868	C2	J12	J5	8	Closed Rectangular	0.7	0.7	1.3		310	0.0032	0.024
J13	1882	C17	J13	J4	20	Closed Rectangular	1	1	3		403	0.0049	0.024
J14	1884	C10	J14	J10	5	Closed Rectangular	1	1	3		470	0.0063	0.018
OF	1829												