PERFORMANCE ASSESSMENT OF URBAN DRAINAGE SYSTEMS OF YIRBA TOWN SIDAMA REGIONAL STATE, ETHIOPIA

M.Sc. THESIS

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HAWASSA UNIVERSITY, HAWASSA, ETHIOPIA

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A THESIS SUBMITTED TO THE DEPARTMENT OF HYDRAULIC AND
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ADVISORS’ APPROVAL SHEET

This is to certify that the thesis entitled “Performance Assessment of urban drainage systems of Yirba town Sidama Regional State, Ethiopia” submitted in partial fulfillment of the requirements for the degree of Master's with specialization in Hydraulic Engineering, the Graduate Program of the Department/School of Hydraulics and Water Resource Engineering, and has been carried out by Sitota Mathewos Rikba ID. No. GPHydrR0014/13, under our supervision. Therefore we recommend that the student has fulfilled the requirements and hence hereby can submit the thesis to the department.

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We, the undersigned, members of the Board of Examiners of the final open defense by Sitota Mathewos Rikba have read and evaluated his thesis entitled "Performance Assessment of urban drainage systems of Yirba town Sidama Regional State, Ethiopia", 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 of Science in Hydraulic Engineering.

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DECLARATION

I hereby declare that this MSc thesis is my original work and has not been presented for a degree in any other university, and all sources of material used for this thesis have been duly acknowledged.

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List of Abbreviations

BMPs	Best management practices
CAD	Computer aided design
CHI	Computational Hydraulics international
CSA	Central Statistical Agency
DEM	Digital Elevation Model
ERA	Ethiopian road authority
GIS	Geographic Information System
GPS	Global positioning system
IDF	Intensity-Duration-Frequency
IDW	Inverse Distance Weighting
LID	Low Impact Development
NMA	National Metrological Agency
NSE	Nash–Sutcliffe model efficiency
PCSWMM	Personal Computer stormwater management model
R^2	Coefficient of determination
STORM Model	Storage, Treatment, Overflow, Runoff Model
SUDs	Sustainable drainage systems
SWAT	Soil and Water Assessment Tool
SWMM5.2	Stormwater management model version5.2
U.S EPA	United State Environmental Protection Agency
UD and FC	Urban Drainage and Flood Control District
USGS	United State Geological Survey
XPSWMM	Stormwater and Wastewater Management Model

ABSTRACT

*Developing countries experience accelerated urbanization without adequate investment in infrastructure. Yirba town, like other towns of Ethiopia, have a lot of problems including inadequacy and poor-quality drainage infrastructure. The objective of this study is to assess the hydraulic performance of stormwater drainage systems of the study area, to identify stormwater management issues and existing urban drainage system problems, and to identify the best management practices for drainage problem mitigation measures and to achieve these specific objective SWMM5.2 model and LID structural measures were used in the study. Rainfall, infiltration, and physical characteristics of the catchment area were organized as three main inputs for the model. Rainfall depths of thirty one years (1992-2022) of Yirba station were obtained from National Meteorological Agency. According to chi-square test of the best fit probability is Log Pearson type III distribution and that is used to developed IDF curve. Infiltration of the model area is represented by Curve number method. Physical characteristics of the sub catchments including topography of the model area was analyzed using Goggle Earth and ArcGIS software. The calibration and validation of the SWMM5.2 model were done and its performance was tested by the goodness of fit using the coefficient of determination (R2) =0.97 and Nash-Sutcliffe Efficiency Coefficient (NSE) =0.90. The total simulation area of sub catchment is 80 ha; the model area is divided in to 45 sub catchments with a drainage network of 38 conduits, 38 junctions and 2 outfalls. From model result 50% of conduits and 55% of junctions are flooded; at the outfall total sub-catchment runoff is 4.35m^3/s average flow rate, 10.84m^3/s maximum flow rate and 91.10*103 m^3 total volumes of both outfalls. Among the various low impact development (LID) structural measures, infiltration trench and vegetative swale are incorporated and simulated in the model. Based on the simulation of LID's 10-year return period, improvements in terms of reduced peak discharges and increased peak discharge times were observed in sites where LID was used. In terms of a 10-year period, the peak runoff from particular sub catchments of drains is reduced by a minimum of 1.82% and a maximum of 19.91%. In general, the Yirba town urban drainage system performance infrastructure overflow was assessed and managed for the problem occurring using the possible mitigation techniques by improved LID control for all study areas.*

Keywords:- *Yirba town, Drainage systems, Stormwater, SWMM5.2, LID control*

CHAPTER 1 INTRODUCTION

1.1 Background

Infrastructure development in Africa is appalling, falling behind the rest of the globe in terms of quality, quantity, and accessibility. The situation is worse in developing nations, where a wide range of issues have caused the provision of physical infrastructure and services to consistently lag behind the growing rate of urban population. Rapid urbanization occurs in developing nations without appropriate infrastructure investment and against a background of inadequate public service for water treatment, collection and treatment of foul sewage, waste collection, urban drainage, transport and health. Urban concentrations have environmental consequences in the form of urban flooding and pollution of water courses, soil and air (Silveira, 2002). Settlements are established in unsuitable areas, such as areas that were initially designated for environmental protection, on steep hillsides, and in flood-prone areas.

In the Ethiopian context, where watersheds of many urban centers receive significant amounts of annual rainfall and where rainfall intensity is typically high, controlling runoff at the source, protecting against flooding, and safely disposing of excess water/runoff through proper drainage facilities become crucial. Flooding, deteriorating roads, soil degradation, sedimentation, obstruction of drainage facilities, and water logging are some of the drainage problems Ethiopian urban areas confront. (Asfaw, 2016)

Inadequate management of urban stormwater drainage represent one of the most frequent complaints from residents in many Ethiopian towns and this problem which is just getting worse as the country continues to see a rapid rate of urbanization (Alemu, 2017). When urbanization progresses more quickly, drains become more and more overloaded and unable to handle significant rainfall. Most developing urban areas in Ethiopia experience flooding as a result of rapid urbanization, which also has an impact on the performance of the drainage systems due to its impermeable structures and poor design. (Belete, 2011). The inadequate integration between the road and drainage networks is the other challenge in urban areas, which prevents the safe discharge of runoff produced in a given metropolitan area into the ultimate receiving system. As a result, this will be the cause of environmental problems such over topping, erosion, pollution, traffic barriers, and other problems. (Dagnachew, 2009)

This thesis report is based on a comprehensive urban drainage study that was carried out using the SWWM model in a selected model area of Yirba town in order to identify problems with stormwater management and the current urban drainage system as well as possible solutions.

1.2 Statement of the problem

Inadequate stormwater drainage management is one of the most common concerns from residents in many Ethiopian urban centres; this issue is made worse by the nation's quick urbanization. The increase of urbanization leads to the construction of more roads, sidewalks, and buildings. This results in reducing natural permeable surface that can infiltrate water into the ground. As a result, the impermeable surface will create more stormwater runoff.

Yirba town is like other towns of Ethiopia have a lot of problems including inadequacy and poor-quality drainage infrastructure. Several areas of Yirba town were inundated by the flood that often arises from the town's slope during the rainy season. When observed study area of drainage systems problems like floods cause soil erosion, gullies form in various locations, overflow onto the road surface, erode the roadways in the overflow area, drainage facilities become blocked, and water accumulates in some drainage sites. This study aims will be to identify the major stormwater drainage problems and to propose possible solutions to mitigate the drainage issues.

1.3 Objectives of the study

1.3.1 General objective

The main objective of this study is to assess the performance of urban drainage system of Yirba town Sidama Regional State, Ethiopia.

1.3.2 Specific objective

- To assess the hydraulic performance of stormwater drainage systems of the study area.
- To identify stormwater management system problems.
- To identify the best management practices for drainage problem mitigation measures.

1.4 Research Questions

- How well do stormwater drainage systems perform hydraulically?
- What are the existing stormwater management system problems?
- What are the possible best management practices to mitigate the drainage problems of the study area?

1.5 Significance of the Study

This study helps the area in developing future stormwater drainage infrastructures by assessing the performance of the urban stormwater drainage infrastructure that is currently in place and suggesting mitigation measure to avoid undesirable functioning. Additionally, it assists in identifying both engineering and non-engineering solutions that the town should use to solve its existing challenges.

This study's significance may also help to fill gaps by identifying issues with sustainability, carrying out accurate modeling of the stormwater drainage system, and ensuring appropriate operation of the town's drainage schemes. It can be used as a guide for proper urban drainage system design, implementation, and maintenance by the concerned body and organizations that serve the area. In general, Yirba is the part of which facing the drainage problems, so further investigation were contribute the solution for stormwater drainage problem and sustainable drainage system for future use in the area.

1.6 Scope and limitation of the study

The scope of the study is focused to assessing the performance of existing stormwater drainage structures and recommending mitigation solutions. The main focus of this study includes: investigate the functionality of the existing stormwater drainage systems, identify the serious situation and related problem with drainage system of the study area, land use land cover change effects on the stormwater runoff volume and identify the alternative effective stormwater measures as well as proposing best management practice for the existing problem.

The study is only focused on the crucial and related issue with the drainage system in the Yirba town, excluding the agricultural area of the town. Additionally, this study only models the types and dimensions of significant drainage structures; it does not address the detailed structural design of drainage infrastructures.

1.7 Thesis outline

The thesis is organized into five chapters from introduction part to the conclusion and recommendation. Introduction, statement of the problem, research question, objectives of the study, significance, study scope, and study limitation are all covered in the first chapter. Second chapter review the literatures that related to study with stormwater and drainage system performance. The third chapter's materials and methods start with a description of the study area, the materials used for the research, and the methods applied to achieve the objectives. Results and discussion sections are included in the fourth chapter. Fifth chapter conclusion and recommendation with the possible mitigation measures and the last reference and appendix are included in this study.

CHAPTER 2 LITERATURE REVIEW

2.1 Historical perspective of the urban drainage system.

According to historical records of ancient civilizations (Mesopotamians, the Minoans and the Greeks), urban drainage systems may have been constructed with great care and with the objectives of collecting rainwater, preventing nuisance floods, and conveying wastes (Burian and Edwards, 2002). The systems that eventually met their objectives likely did so after trial-and-error modifications. In general, planning and design were limited. Few numerical standards existed for urban drainage and engineering calculations were not used during design. Despite the lack of optimization and the use of trial-and-error construction methods, numerous ancient urban drainage systems can be rated very successful. Lewis Mumford summarized the state of ancient urban infrastructure when he stated that ancient sewer systems were an uneconomic combination of refined technical devices and primitive social planning (Mumford, 1968). Significant improvements in urban drainage systems were established during the Roman Empire Age. (Burian and Edwards, 2002)

2.1.1 Development of modern urban drainage system

The beginning of modern urban drainage practices was initiated in European cities during the nineteenth century. One critical turning point in urban drainage occurred during the middle of the nineteenth century. During the first half of the nineteenth century sanitary wastes were discharged from buildings to privy vaults and cesspools (Metcalf, 2003). Most sewers were designed exclusively for stormwater drainage. Sanitary wastes accumulated in privy vaults and cesspools and were periodically collected by scavengers and transported to a suitable disposal location (e.g., farm, dump outside city). As the nineteenth century progressed the concept of urban drainage changed with the incorporation of water-carriage sanitary waste collection into the urban drainage systems. Sanitary connections to the sewers were made legal and new sewers were constructed to drain stormwater and sanitary wastewater.

The perspective of urban drainage also changed from a design standpoint during the nineteenth century. Most sewers constructed before the nineteenth century were not planned or designed by an engineer using numerical calculations. Instead a trial-and-error process was executed, which in some cases eventually produced well-functioning systems (Belete, 2011).

Urban drainage in the early parts of the twentieth century was firmly established as a vital public works system. Engineers continued to improve design concepts and methods. Regulations, monitoring, computer modeling, and environmental concerns have altered the perspective of urban drainage from a public health and nuisance flooding concern during the first half of the twentieth century into a public health and nuisance flooding with additional concerns for ecosystem protection and urban sustainability. (UD and FCD, 2011)

Methods to design and construct sustainable urban drainage systems are currently being researched and tested. Alternative development concepts (e.g., low-impact development) are influencing development practices to minimize the impacts of development on urban drainage. Urban drainage systems has indeed expanded significantly during the past few decades beyond a technical challenge to drain the urban area expeditiously to include the consideration of social, economic, political, environmental, and regulatory factors.
(Graham, 2012).

2.2 Urban drainage

Urban drainage includes the removal of all unwanted water from urban areas through man made and artificial structures. It includes waste water like sewage, grey water and storm water. Grey water, sometimes called sullage is domestic wastewater predominantly from kitchens, baths, basins and washing machines. The unwanted water may or may not be used for other purposes with or without treatment. Indeed, it is part of the philosophy of sustainable that there is ultimately no waste; all ‘waste’ from one process should be input for another (McDonough and Braungrat, 2002).

2.2.1 Urban drainage system

Urban drainage systems have been regarded as a dynamic natural resource, a discarded transit method, and a flooding challenge. Climate, terrain, geology, clinical expertise, engineering and construction skills, common values and other factors have affected how locals evaluate urban drainage for the purpose of building cities. These factors have also directed and constrained the development of urban drainage solutions. Many exciting and unique urban drainage solutions are attracted by historical accounts (Asfaw, 2016).

2.2.2 Effects of urbanization on urban drainage

Urban drainage replaces one part of the natural water cycle and, as with any artificial system that takes the place of a natural one. In nature, when rainwater falls on a natural surface some water returns to the atmosphere through evaporation or transpiration by plants; some infiltrates the surface and becomes groundwater; and some runs off the surface. The relative proportions depend on the nature of the surface and vary with time during the storm. Surface runoff tends to increase as the ground becomes saturated. Both groundwater and surface runoff are likely to find their way to a river, but surface runoff arrives much faster. (David et al, 2018)

Development of an urban area involving covering the ground with artificial surfaces has a significant effect on the water cycle processes. The artificial surfaces increase the amount of surface runoff in relation to infiltration and therefore increase the total volume of water flow in the urban areas. Surface runoff travels quicker over hard surfaces and through sewers than it does over natural surfaces. Generally, effects of urbanization on drainage or the effects of replacing natural drainage by urban drainage are to produce higher and more sudden peaks in flow to introduce pollutants and to create the need for artificial wastewater treatment (David et al, 2018).

2.3 Urban stormwater drainage system and its management

2.3.1 Stormwater

Stormwater is the water that drains from a site as a result of rain that falls on the ground and everything it takes with it. In an urban environment, stormwater is produced by rain runoff from roofs, roads, driveways, footpaths, and other impermeable structures or hard surfaces (Zhou, 2014).

2.3.2 Stormwater drainage system

In order to reduce the impact of floods on people's lives and property, it is essential to make sure that wastewater and stormwater are properly drained. Maintaining public safety and health is an important societal obligation. In addition, protecting receiving waterways from toxins that might be attracted by water pouring at the surface during heavy rain events is part of today's environmental conscience (Alemu, 2017).

The separated system, which consists of two distinct stormwater and waste pipelines, prevents environmental pollution from entering the environment while preventing flooding in low-lying dwelling basements and floors during extreme rainfalls (UD and FCD, 2011). Stormwater is typically less contaminated than sewage water, thus it can be directed to retention basins or watercourses with less energy and expense, whereas wastewater needs a more thorough treatment.

2.3.3 Component of stormwater drainage system

In order to ensure that the substructure is adequately drained a drainage system must have all the necessary components. This may include open ditches, closed ditches with pipe drains, drainage through stormwater drainage pipes, channels, and culverts. (Asfaw, 2016)

2.3.4 Function of stormwater drainage system

The purpose of the drainage system is to collect surface water and/or groundwater and direct it elsewhere, keeping the bed's strength drained. Along with preventing erosion of the substructure of drainage system, the drainage system must also promote public health, safety, environmental preservation, and sustainable development. The impact of drain and sewer systems on the receiving waterways must also comply with all applicable municipal, state, and federal standards. (Asfaw, 2016)

2.3.5 Urban stormwater drainage problems

Urban drainage practices in developing countries face more severe issues than those in developed countries because urban development takes place in more challenging socioeconomic, technological and climatic contexts. Accelerated urbanization occurs in developing countries without appropriate infrastructure investment and against a backdrop of inadequate public services for transport, health, water treatment, waste collection, and foul sewage collection and treatment. Urban concentrations have environmental consequences in the form of urban flooding and pollution of water courses, soil and air.

Settlements are established in unsuitable areas, such as areas that were initially designated for environmental protection, on steep hillsides, and in flood-prone areas. (Hassen, 2016)

The following are some of the specific problems that are preventing developing countries from modernizing urban drainage, mostly through the infiltration and retention of storm runoff:

a) Uncontrolled urban settlement

The main causes of urban floods related to traditional urban settlement, pursued without consideration for the environment, are impermeable surfaces and the building of drains for quick stormwater drainage. These patterns of urbanization make it difficult to manage urban drainage since it can have an impact on concerns further downstream in addition to local flooding. The amount of impermeable cover is highly associated with both population density and runoff coefficients, therefore these two variables go hand in hand. Due to this, comparing the two variables is a poor method of evaluating how urbanization affects drainage. There is evidence from around the world that rising urban population density frequently leads to increased storm-water generation (Asfaw, 2016), yet many urban planners fail to account for this significant effect and ignore the overall costs of their stormwater control strategies.

Contrary to the theory of higher population density, modern urban drainage requires for retention and infiltration spaces. Many cities in developing countries have a density index that already creates serious drainage problems. Along with issues with control in legal settlements, socioeconomic issues cause slums to expand on public areas and develop, with high rates of impermeable soil surface and population density (Hassen, 2016).

b) Absence of active community participation

The absence of community involvement in the quest for long-term solutions to urban drainage issues is one of the largest obstacles to the effectiveness of contemporary storm runoff management approaches, whether through structural or non-structural techniques (Mulualem, B.and K. Naga, 2018). In the majority of developing countries, this has proven a problem for efficient stormwater drainage management. Because the community is not involved, drainage problems are repeatedly fixed wrongly, harming the reputation of government action and revealing a lack of concern for environmental issues (Asfaw, 2016). It can also bring about low investment in urban infrastructures.

c) Excess sediment and garbage

Urban areas in developing countries contain a sizable part of exposed soil that is subject to erosion and produces a lot of sediment. Building sites typically lack controls for erosion prevention or for retaining sediment to keep it out of the streets, storm drains, and urban rivers, whether they are in areas where the city is growing or within the built urban area. It is not overstated to suggest that between 10 and 15 percent of the urbanized area in emerging nations significantly contributes to the generation and transportation of sediment. For developing countries, the rate of garbage accumulation in the streets is certainly higher, since in some parts of the cities the stormwater network is used for garbage disposal. With these high sediment and garbage loads, no modern solution to urban drainage is viable without special retention structures upstream or rigorous maintenance procedures with dredging or mechanical removal of the large volumes carried after every storm (Alemu, 2017).

d) Lack of appropriate technology

In order for the environmental approach to be effective, a shift in technical culture is needed. This can be accomplished through training (capacity building at all levels, including district engineers and urban planners) and public environmental education (Mukherjee, 2016). Academic institutions can play a significant role in taking on the duty of distributing information through recurrent seminars and technical-scientific meetings that help professionals in the field of storm-water drainage to get more knowledge about the subject. According to Mukherja, the confidence that grows in these interactions between researchers and technicians opens up communication channels and encourages municipal and university cooperation in technical support services for upgrading urban drainage practices.

2.3.6 Urban stormwater management

Urban stormwater management can be stated as anything done within a catchment to solve the current stormwater problems and prevent the formation of future problems (Getachew,

K. W.& Tamene, A. D., 2015). This entails the development and application of a variety of structural and nonstructural methods to integrate the space and associated needs of increasing urban populations with the conveyance and storage function of stormwater systems. In addition, it entails the development and application of a number of best management practices or measures to enhance the quality of urban stormwater runoff before it is discharged into receiving waterways. It is vital to understand the type, characteristics, and function of the stormwater drainage system before analyzing the stormwater management system.

2.4 Flooding

Flooding, by its very nature, frequently results from both hydrologic and meteorological strategies; the characteristics of a flood are dictated by both the nature of the situation in which the occasion is likely to occur and by the actual conduct of the precipitation (soil situations, quantity of antecedent rainfall, and so on). Even if it is genuinely well inside our capabilities to predict the possibility of the most of flood events, it is unlikely that exactly unique forecasts of flooding events will ever be possible.

The following variables will be considered for all hydrologic analyses and taken into account if they significantly affect the results:-

- Drainage basin characteristics including length, shape, slope, land use, geology, soil type, surface infiltration, and storage;
- Channel characteristics including geometry and configuration, natural and artificial controls, channel modification, aggradation/degradation, and particles;
- Flood plain characteristics; and
- Meteorological factors such as precipitation volumes and types (rain, hail, or combinations of both), hurricane mobile characteristics, storm path, and precipitation time charge (Ethiopian Road Authority, 2013).

2.5 Hydrologic consideration

The hydrologic effects of urban development often are greatest in small stream basins where, prior to development, much of the precipitation falling on the basin would have become subsurface flow, recharging aquifers or discharging to the stream network further downstream. Moreover, urban development can completely transform the landscape in a small stream basin, unlike in larger river basins where areas with natural vegetation and soil are likely to be retained (Asfaw, 2016).

Streams are fed by runoff from rainfall and snowmelt moving as overland or subsurface flow. Floods occur when large volumes of runoff flow quickly into streams and rivers. The peak discharge of a flood is influenced by many factors, including the intensity and duration of storms and snowmelt, the topography and geology of stream basins, vegetation, and the hydrologic conditions preceding storm and snowmelt events (Urgessa, 2016).

2.5.1 Intensity Duration Frequency curve

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

2.6 Hydraulic considerations

Hydraulic design of storm drainage systems requires an understanding of basic hydraulic concepts and principles. Important hydraulic principles include flow classification, conservation of mass, conservation of momentum and conservation of energy concepts (U.S. Department of Transportation, 2001). Urban flooding is two vital hydraulic ideas that arise: surcharge and backflow. A drainage gadget is surcharged while its potential is passed, i.e. it gets extra extent of water than the gadget can carry. As an end result, the water stage rises upstream due to the community overloading (Zhou, 2014), if the power line reaches higher points downstream than upstream, water may alternate the normal waft direction leading to backflow.

2.6.1 Hydraulic capacity

The hydraulic capacity of a storm drain is managed by its length, type, slope, and friction resistance. Numerous slide friction formulas have been superior which outlines the connection among flow capability and these parameters. The maximum extensively used components for gravity and stress flow in storm drains is Manning's Equation (David et al, 2018).

2.7 Model Justification

SWMM (U.S. Environmental Protection Agency, 1992) is a comprehensive computer model for simulating urban runoff quantity and quality in storm and combined sewer systems. SWMM 5.2 stands for Storm Water Management Model version 5.2. All urban hydrologic characteristics include, floor runoff conveyance via the drainage system and storage. Like most hydrologic fashions, SWMM5.2 subdivides the general catchment into sub-catchments predicting runoff from sub-catchments on the basis of their individual properties and combining their outflows using a float routing scheme. Storm Water Management Model version 5.2 can also simulate backwater outcome. In SWMM5.2, sub-catchments are represented mathematically as spatially lumped, nonlinear reservoirs and their outflows are routed via the channel/pipe. Sub catchments are subdivided into 3 subareas, impervious area with and without depression storage, and pervious areas with depression storage. Go with the flow from one subarea isn't always routed over another subarea. Overland water is generated from each of the three subareas by way of approximating them as nonlinear reservoirs. This nonlinear reservoir is hooked up by means of combining the continuity equation with Manning's equation. Infiltration from pervious regions can be computed through all Horton, Modified-Ampt and Curve number method equation. Flow routing in channel/pipes is also carried out via a nonlinear reservoir by combining the continuity equation with Manning's equation (Rossman, L. and M. Simon, 2022).

2.7.1 Model selection

There are several methods for assessing stormwater runoff that are based on the water balance equation, empirical equation, and practical models like CivilStorm, STORM model, XPSWMM, PCSWMM and SWAT by such strategies calculations for penetration, overflow of surface, routing of flow, and slacking of surface overflow have been re arranged to permit simulation of flow with a hydraulic and Hydrologic Simulation Program For each one of those number of techniques they have some limitations fixing urban drainage systems networks (Sidek L,et al., 2011).

2.7.1.1 CivilStorm

Civil Storm is a fully dynamic, multi-platform, hydraulic modeling solution developed for the analysis of complex stormwater systems. It is used to analyze these systems using built-in hydraulic and hydrology tools and a variety of wet-weather calibration methods. It is applied in master planning, water quality study, and modeling the impact of LID structures. (J.Lind, 2015)

2.7.1.2 STORM model

The U.S. Army Corp of Engineers (1977) developed Stormwater Runoff Model (STORM) to analyze quantity and quality of runoff from urban and nonurban catchments. STORM was primarily developed to evaluate the storm water storage and treatment capacity required to reduce untreated overflows below specified values. Computations of treatment, storage and overflow proceed in an hourly basis by simple runoff volume and pollutant mass balance for the entire catchment. Since this model runs on hourly time step, this model is not suitable for small catchments where time of concentration is less than one hour. STORM is a continuous simulation model. Since this model is primarily used for planning, it cannot be used for precise quantity or quality modeling. (J.Lind, 2015)

2.7.1.3 XPSWMM

Another hydraulic and hydrologic modeling tool from XP-solutions, XPSWMM has been used for analysis, design, and simulations for more than 25 years. Similar to XP Storm, XPSWMM includes river systems and 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 (J.Lind, 2015).

2.7.1.4 PCSWMM

Developed by Computational Hydraulics Inc. (CHI), PCSWMM is a third-party interface for SWMM. It is a GIS integrated model that uses SWMM as the model computational engine for hydrologic and hydraulic calculations. It is an independent modelling programme that includes with the required GIS tools and supports a number of CAD and GIS formats. (J.Lind, 2015).

In urban areas, EPA SWMM5.2 is used for flood modelling and analysis to determine the drainage size by considering both pervious and impervious areas into consideration. So that modeling in urban drained the SWMM software program is more convenient for Yirba drainage systems Hydraulic and Hydrologic Simulation tool for this study's aims. The SWMM5.2 has often been used to estimate town runoff. (Kong F,et.al , 2017)

2.8 Best Management Practice (BMP)

Stormwater management has traditionally consisted of removing runoff from the area as quickly as possible using huge pipes and concrete channels. Low impact development (LID), an alternative to this strategy, provides economic and land-use planning guidelines with the goal of reducing stormwater runoff when combined with stormwater best management practises (BMPs) that treat stormwater on site. LID is effectively a land re-improvement strategy for stormwater management. The basic goal of LID is to maintain a site's predevelopment hydrology by decentralizing micro-scale controls, hence reducing the negative effects of precipitation flooding waters. Through the infiltration and evaporation of stormwater, LID methods successfully alleviate water-related problems, resulting in positive environmental, social, and economic effects. The common LID practices are bio-retention, inexperienced roofs, permeable pavements, rain gardens, vegetative swales, and rain containers that are used to create a functionally equal hydrologic landscape. (Kong F,et.al , 2017)

2.8.1 Selection criteria of LID techniques

There are several factors to take into account while selecting SUDS in order to use the most appropriate technique and implement SUDS effectively. Such elements encompass website online suitability, available land area, value, maintenance troubles, and network popularity (Graham, 2012).

There are three possible techniques to model overland flow from LID controls (Eyosias, 2018). The first technique is to route impervious sub-catchments to pervious sub-catchments and then to receiving nodes because the features of pervious areas should fit the LID control scheme. The sub catchment's pervious area serves as the LID control. This technique does not produce precise results and is not realistic.

The second technique is creating a separate sub catchment for LID and routing the primary sub catchment to the receiving node of the LID sub catchment. The sub catchment properties are to be matched with the LID design. The original pervious or impervious area must be used to extract the LID area.

The final technique is to include a LID in the original sub catchment and to direct runoff through it before adding a receiving node from the LID region to the original pervious or impervious area.

There are both pervious and impervious areas in the sub catchment of the YIRBA town, and no area is used separately as a LID receiving node to manage stormwater. This means that the area where LID control was applied is considered as being a part of the original sub catchment. Therefore, the second alternative is chosen to send runoff through LID before reaching the receiving node.

2.8.1.1 Infiltration Trench

An infiltration trench is an excavated trench that has been back-filled with stone to form a subsurface basin. Stormwater runoff is diverted into the trench and is stored until it can be infiltrated in to the soil (Rossman, L. and M. Simon, 2022). The overall performance of the trench relies largely on the permeability of the soil and the depth to the water table. Infiltration trenches are commonly used in combination with other source control techniques to service small catchment areas of up to 2-3 hectares. They will be more powerful the closer they are to the runoff's source. By providing a pre-remedy for the inflow, such as a filter strip, gully, or sump pit, to dispose of excessive solids, the trench's operational viability may be improved. For maximum pretreatment designs, routine maintenance may be necessary.

a) Asphalt road b) Gravel road

Figure 2-1: Infiltration trench structural view on side road
Source: (Collins, 2019)

2.8.1.2 Bio retention

Bio-retention Cells are depressions that contain vegetation grown in an engineered soil mixture placed above a gravel drainage bed. They provide storage, infiltration and evaporation of both direct rainfall and runoff captured from surrounding areas. Bio retention areas or rain gardens are depressed areas in the landscape that are designed to accept stormwater. They can be used in residential and commercial settings and are typically planted with shrubs, perennials, or trees, and covered with shredded hardwood bark mulch. The benefits of bio retention areas include decreased surface runoff, increased groundwater recharge, and pollutant treatment through a variety of processes (Rossman, L. and M. Simon, 2022).

a) Side road b) Open space

Figure 2-2: The construction practice of bio retention method on the side roads and open space.
Source: (Sundberg, 2019)

2.8.1.3 Vegetative Swales

Vegetative swales are low pitched trapezoidal or triangular ditches that are heavily planted and intended to move runoff slowly. Due to their wide cross sections and low longitudinal slopes, grass swales allow for slow, shallow flow conveyance, promoting sedimentation and filtration (straining) while preventing erosion. To slow down flow and promote settling and infiltration, berms or check dams can be included into vegetative swales (Rossman, L. and M. Simon, 2022) .

a) Center street b) Side road

Figure 2-3: Vegetative swale structural view on side road and Center Street.

Source: (Sujit Ekka and Bill Hunt, 2020)

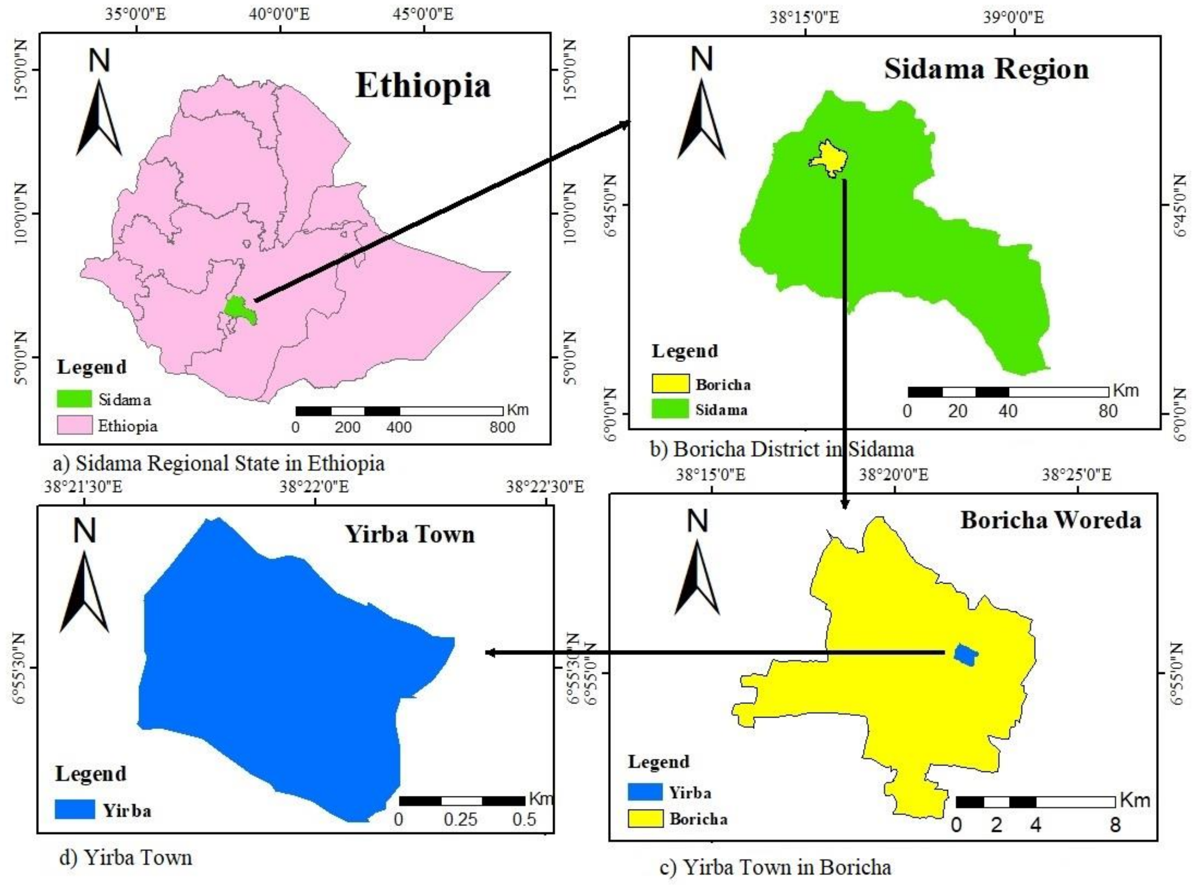
CHAPTER 3 MATERIALS AND METHODS

3.1 Description of the Study Area

3.1.1 Location and Area

Yirba town is located in Boricha district which is one of districts in the National Regional State of Sidama. It is found at a distance 32 km south of the Regional Capital, Hawassa, 309 km south of Addis Ababa. The town coordinates are 6° 54' 49.3'' N to 6° 56' 15'' N and 38° 21' 30'' E to 38° 22' 30'' E.

The study area existing built up area of the town that is developed for different urban land use was about 400 hectare. A total area of 1700 hectare is delineated for the town in collaboration with the district administration.


a) Sidama Regional State in Ethiopia

b) Boricha District in Sidama

d) Yirba Town

c) Yirba Town in Boricha

Figure 3-1: Location maps of the study area

3.1.2 Topography

The topography of Yirba town has mixed characteristics. It consists of hills relatively flat and sloppy areas. To conclude the town in general has undulating topography, which would be costly for development of road, drainage and housing, while on the other hand has beautiful landscape. Yirba town is a part and parcel of the Great East African Rift Valley.

A. Altitude/Elevation

Altitude analysis in the town indicates that elevation generally decreases from west to center and from east to center of the town and increases slightly from south to north. The elevation ranges from **1,966 to 2,014** meters above sea level

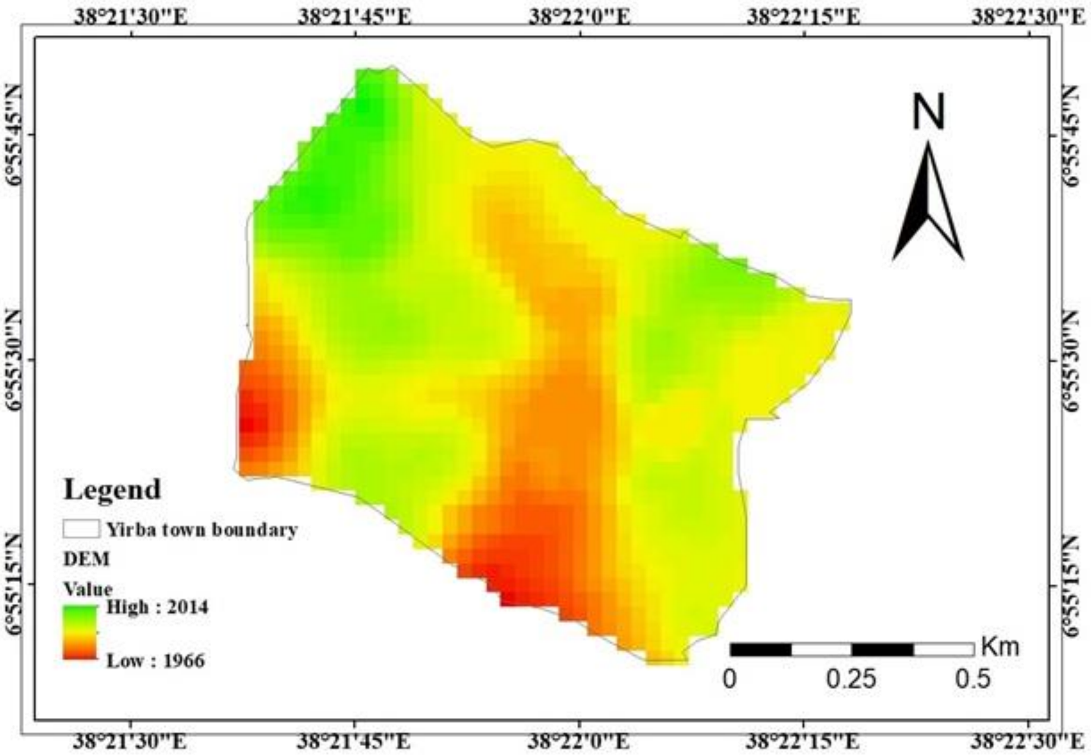


Figure 3-2: Digital elevation model (DEM) of Yirba town

B. Slope Analysis

Slope analysis facilitates in the selection of suitable landscape for urban expansion and the determination of various urban applications in urban centers. The slope of the town is analyzed using xyz coordinates or digital elevation model (DEM) through GIS tools. As shown on Table 3.1 below, the slope analysis of the town consists of a large area with a slope of 1%-9%. These are floodable areas so they are exclusive for different constructions and also they are exposed for storing stagnate water that may negatively affect the health and safe movement of the town dwellers. However, it is important to note that construction of urban drainage requires appropriate slope selection or provision in order to attain cleansing flow velocity that helps to have smooth storm water movement.

Table 3-1 Slope Classification and Area Covered

Slope classification (%)	m^2	Hectare	Percentage (%)
0-3	213721.99	21.37	26.71
3-6	316031.66	31.60	39.50
6-9	215469.47	21.55	26.93
9-12	51551.35	5.16	6.44
>12	3266.29	0.33	0.41
Total	**800040.76**	**80.00**	**100**

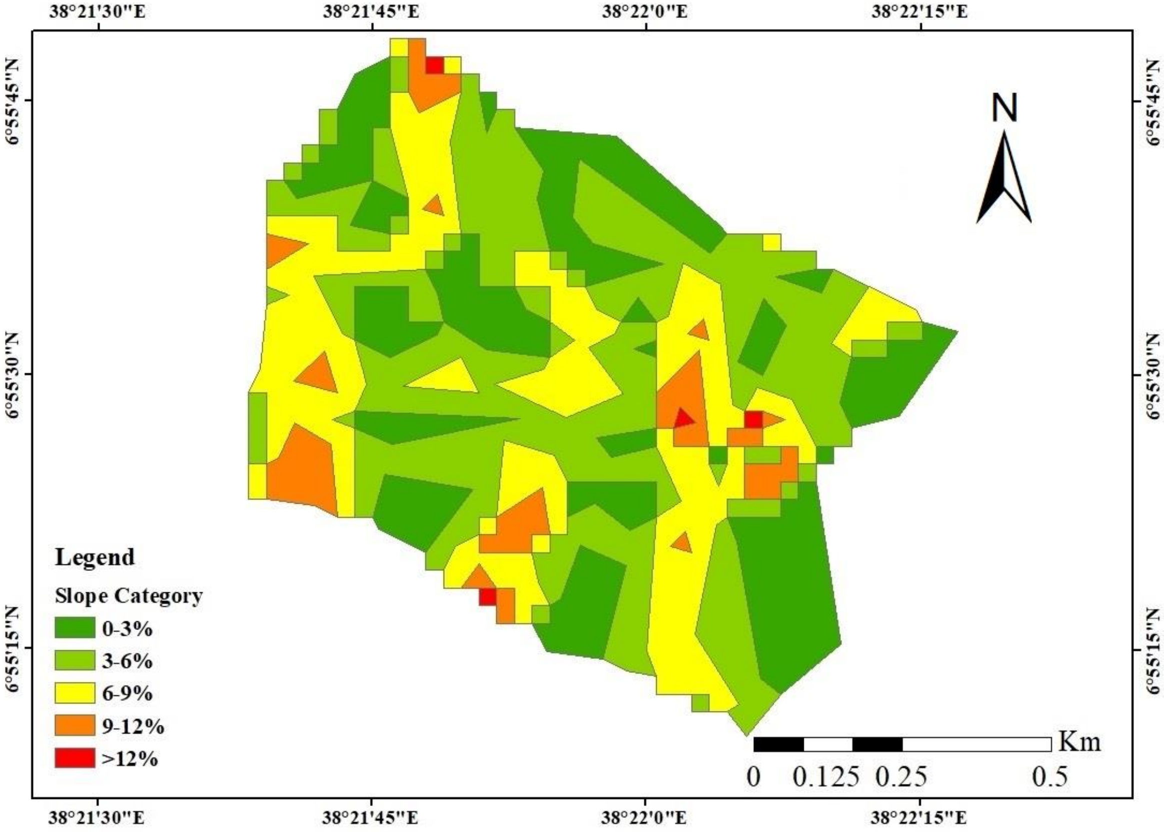


Figure 3-3: Slope distribution of Yirba town

3.1.3 Climate

The climatic condition of Yirba is Woina Dega zone, mean annual minimum and mean annual maximum temperature is 14ºc and 26ºc respectively. That means the average annual temperature is 20ºc. The highest temperature is experienced between January and March and lowest temperature record is between July and September. (Yirba_Municipality, 2012)

Based on 31 years (1992-2022) daily rainfall data from Ethiopian Meteorology Agency, the mean annual monthly rainfall chart of the town is depicted in figure 3.4. The mean annual rainfall is about 76.90 mm.

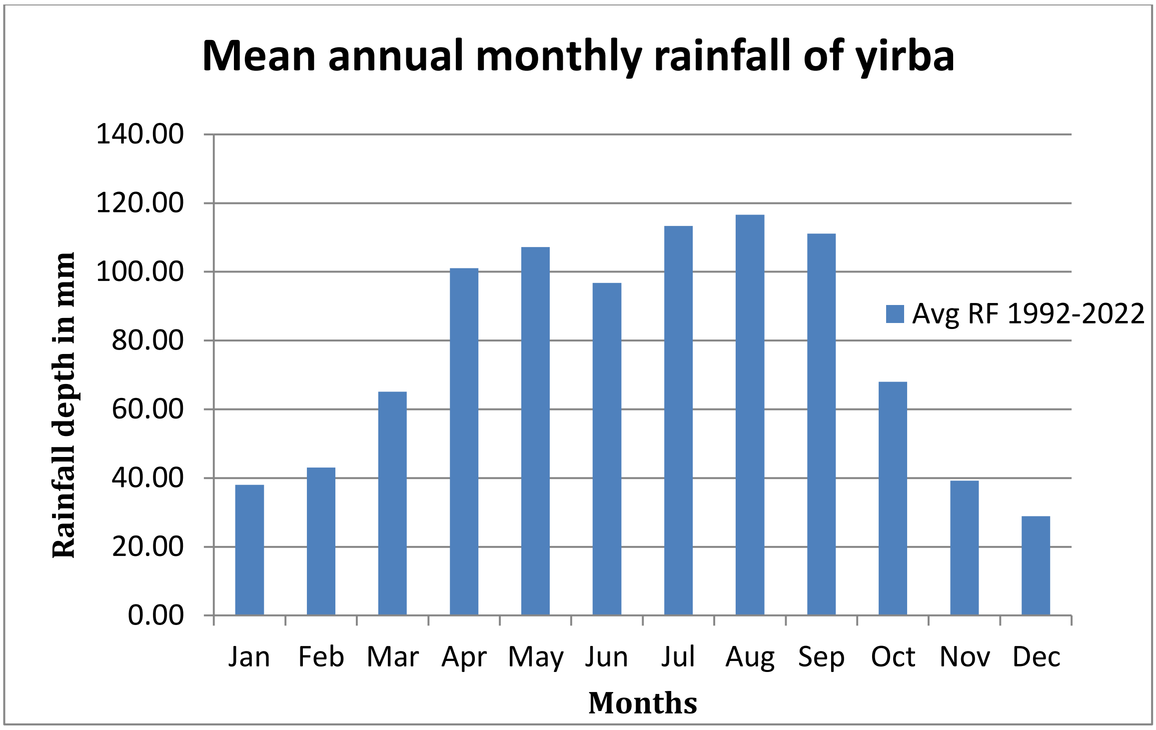


Figure 3-4: Mean monthly rainfall over the period 1992-2022 in Yirba

The above figure 3.4 presents the long-term mean monthly rainfall over the period 1992-2022 in Yirba. Similar to other areas of country the highest rainfall is recorded between July and September. Therefore, it is expected to have more stormwater that might be a challenge for the drainage system of the town.

3.1.4 Land Use Land Cover, Soil and Geology of the study area

3.1.4.1 Land use land cover

The land use land cover (LULC) data was used as a primary input to construct the runoff coefficient in the SWMM5.2 model, which determines runoff.

There are trees and vegetation in the town that covers 2.2% of the total are of the town. The main types of vegetation in the town are Eucalyptus, Tid and different kind of bushes. Eucalyptus tree is preferred for its commercial value. However in most of the areas where such trees are planted the land is found to be degraded. As municipality file in the landuse share and percentage eroded areas in the town cover 1.87% of the total area of the town. These areas need to be rehabilitated by planting indigenous trees that could mend the land. (Yirba_Municipality, 2012)

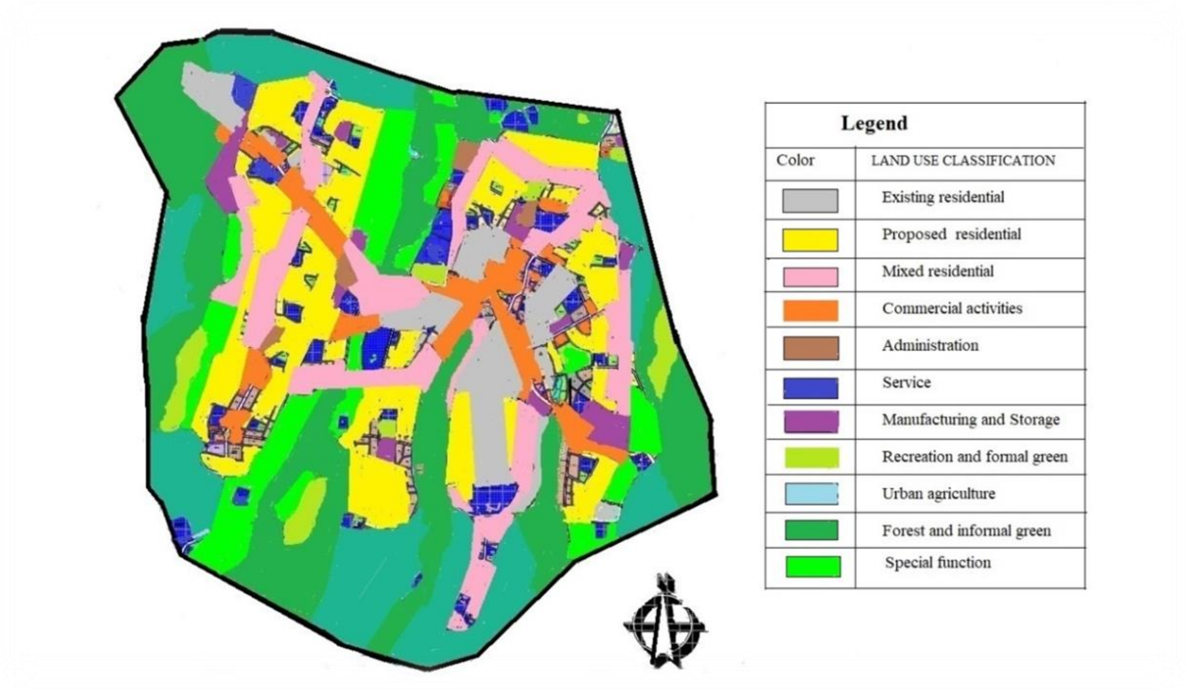


Figure 3-5: The current Yirba town structural Land use land cover.
(Source: Yirba master plan, 2012)

3.1.4.2 Soil and Geology

Due to undulating topographic condition of the town the top soil from the escarpment and hilly areas of the town is washed and deposited in the lower parts of the town. In these areas the color of the soil is black and this soil type is Mollic Andosols. In other parts of the town predominant soil type is red soil (Vitric Andosols). The Reference Soil Group of the Andosols includes the soils developed in volcanic materials with andic properties. Translocations of weathering products and accumulation of short-range-order minerals and stable organo-mineral complexes are essential processes in the formation of the characteristic soils of volcanic regions (Andosols). The soil in the Yirba town is the clay loam/ sandy clay loam is dominant texture. (Yirba_Municipality, 2012)

The geologic origin of the study area is volcanic rocks and ingenious intrusions. Boricha Rhyolite and different types of volcanic rocks observed around the town. Boricha Rhyolite usually occurs in the form of white to bluish-gray lava flows, columnar jointing and also as glassy brecciated rhyolite lava, which is mostly confined to the Boricha dome. A large reserve quarry site is located south of Yirba town. The low lying areas are characterized by deposits of soil washed down from the surrounding areas. (Yirba_Municipality, 2012)

3.1.5 Existing drainage networks of the town

The existing storm drainage systems in Yirba Town can be divided into closed and open drainage lines. There are some areas with closed drainage systems, particularly along the major routes connecting Hawassa to Wolayta Sodo via Morocho Road. Open drainage channels, constructed by masonry are found along sub-mains and local roads. The stormwater drainage system collects the runoff and discharges to neighborhood through south and south west directions.

According to observations from the field survey, the key problems with the existing drainage channel are:

- The drainage systems are not connected well;
- There are no drainage systems in several areas of the town;
- Most ditches have not an appropriate slope;
- The drainage systems overflow as a result of being filled with solid waste and soil; and insufficient capacity of the drainage infrastructures.

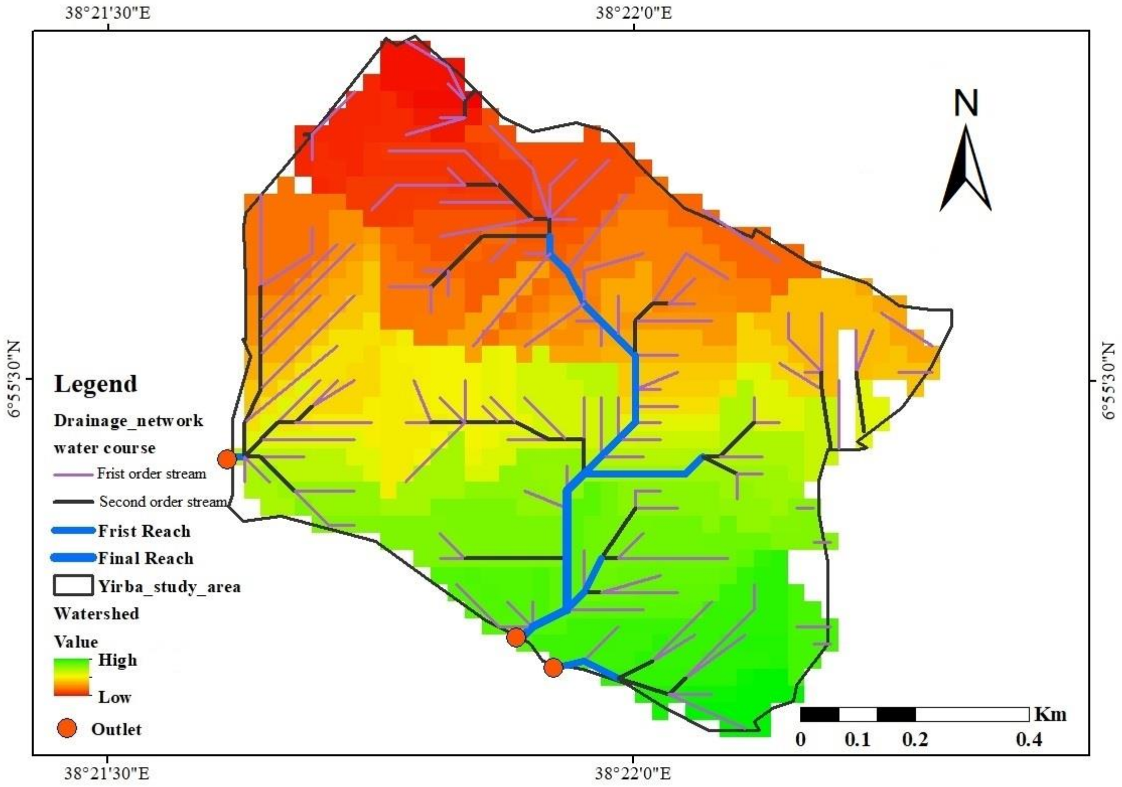


Figure 3-6: The existing natural drainage networks of Yirba town

3.2 Data collection and Material used

3.2.1 Primary data

In the municipality of Yirba, neither a soft copy nor a hard copy of an organized existing drainage network with the corresponding dimension data available. Due to this, important data on the drainage system was gathered from the study area, which is a crucial input for the model. Additionally, frequent field visits were undertaken in order to make physical observations about the existing status of the drainage system.

Table 3-2: Types of urban drainage infrastructural data collected

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

3.2.2 Secondary data

A variety of secondary data were necessary for this research, some of which were gathered and are listed in table 3.3.

Table 3-3 List of secondary data collected

Data type	Source	Purpose
Meteorological data ❖ Daily Rainfall data (1992-2022)	National Meteorological agency	To formulate IDF curves
Land surface data ❖ Digital Elevation Model of the area (Resolution of 20 meter X 20 meter)	Satellite image from USGS (United States Geological Survey)	To make watershed & topographic map of the area.
Master plan of the town	Yirba Municipality	To examine the existing land use with respect to the structural plan
Soil data for the town (texture)	Yirba master plan Socio economic report	To use as an input for the model

3.2.3 Material used

All rainfall data for daily intervals were gathered from the National Meteorological Agency. The data recording period was 31 years (1992 – 2022). In general, the following information was used for each study's data collection and supporting materials:

- ❖ Input shape file data is utilized to define catchments and estimate catchment characteristics using ArcGIS software.
- ❖ ArcGIS were used to gather geographic data, hydrological and physical parameters, and watershed catchment characteristics for the study area.
- ❖ Use Google Earth map software to confirm the catchment divides and water shed in the research region.
- ❖ Hydrological and hydraulic data were used as input for the SWMM tool.
- ❖ To determine the node and drainage dimension and elevations, GPS and a TAPE meter are used.
- ❖ Using as a reference various journals, thesis, books, design documents, and manuals.

3.3 Method of the study

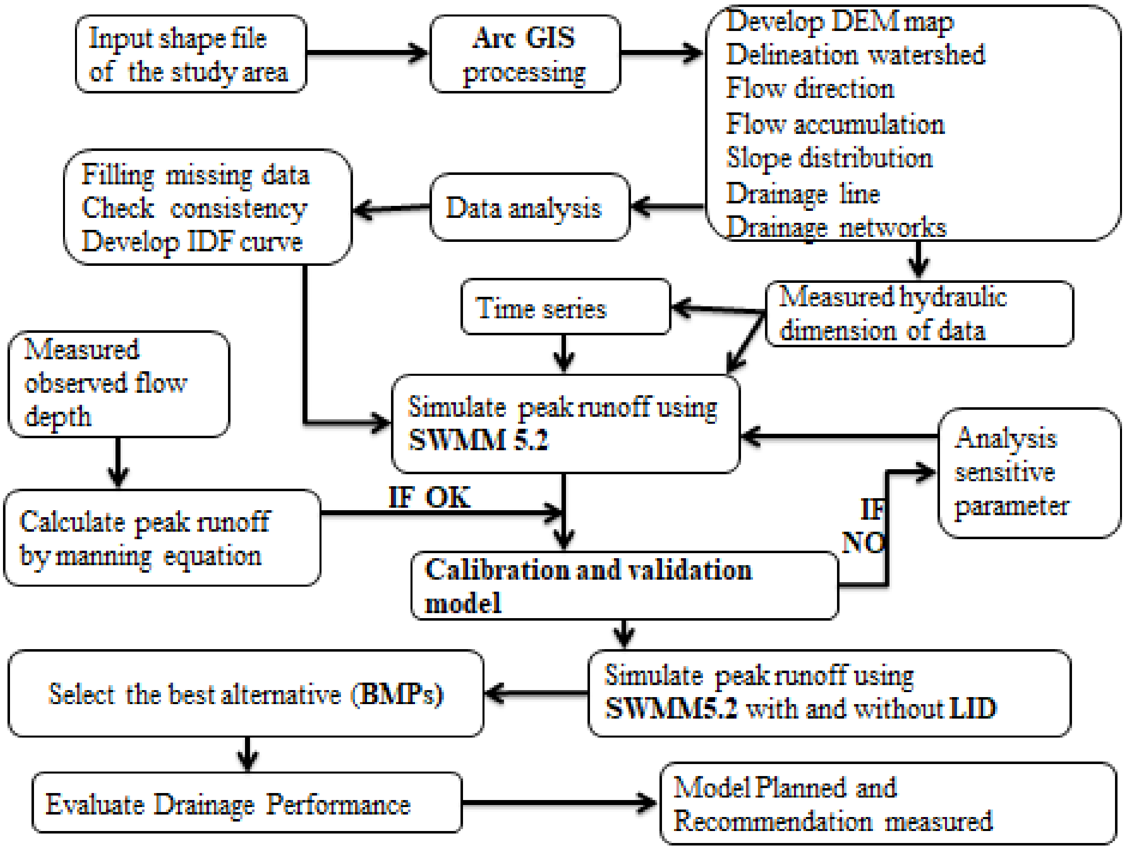


Figure 3-7: Overall framework methodology

3.4 Rain fall data analysis

Rainfall surrounds the missing values which can be attributed to a variety of causes, including equipment failures, weathering, or human data input errors. The first step in the data processing procedure is to estimate the value of any missing rainfall. Four stations are taken into account for this study; one target station and three (3) nearby stations within a radius of 5km to 20 km. In order to produce more accurate estimations, the data consists of daily rainfall from 1992 to 2022 G.C (31 Years'). (Kang H. and Yusof F., 2012)

3.4.1 Filling of Missing Data

There are a number of ways to fill in missing rainfall data, but Inverse Distance Weighting (IDW) has been chosen and used in this study. According to (Chen F.and Liu C., 2012), it is the most acceptable method and is widely used for determining the missing rainfall data and mostly climate data spatially varies with distance, in case of the weight for each station is assumed to be inversely proportional to its squared distance of the target station from the neighbouring station with data.

$$P_x = \frac{\sum_{i=1}^{m} \frac{1}{d^2} P_i}{\sum_{i=1}^{m} \frac{1}{d^2}} \text{.........} 3.1$$

Where: P_x = Average precipitation

P_i = Precipitation of nearby rain gauge stations

d = Distance between rain gauge stations

Every missed daily rainfall data from 1992-2022 is estimated using the nearby three metrological stations (Morocho, Derara, and Balela).

3.4.2 Checking the accuracy of the data

3.4.2.1 Test for consistency of record

The rainfall data from that station would become inconsistent. One of the most frequent reasons for record inconsistency is the relocation of a rain gauge station, a noticeable change in the station's neighborhood, a change in the ecosystem as a result of disasters like forest fires and landslides, and the occurrence of observational error starting on a specific date. Testing for inconsistency of record is done by the double mass curve technique. The procedure is that accumulated rainfall at the gauge station whose record is in doubt is plotted as ordinate versus the average concurrent accumulated average rainfall of nearby station of rainfall data are available (Subramanya, 2008).

The following equation 3.2 relation is used to adjust the precipitation data at station x after the period of change in order to resolve a break in the slope of the resulting plot that indicates a shift in the station's precipitation regime.

$$P_{Cx} = P_x \left(\frac{S_c}{S_o}\right) \text{.........} 3.2$$

Where P_{CX} = corrected precipitation at any time period t1 at station x

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

S_C = corrected slope of double mass curve

S_O = original slope of the double mass curve

The double mass curve is used to evaluate the consistency of the hydrological data and can be used to correct inconsistent precipitation data by comparing the hydrological data from the Yirba station to those from nearby stations (Morocho, Derara, and Balela). In the graph 3.8, which compares the cumulative annual rainfall at the Yirba station with the cumulative annual rainfall at the average neighboring stations, the ratio between the variables is fixed by the correction factor 1.245769. (Appendix table 4)

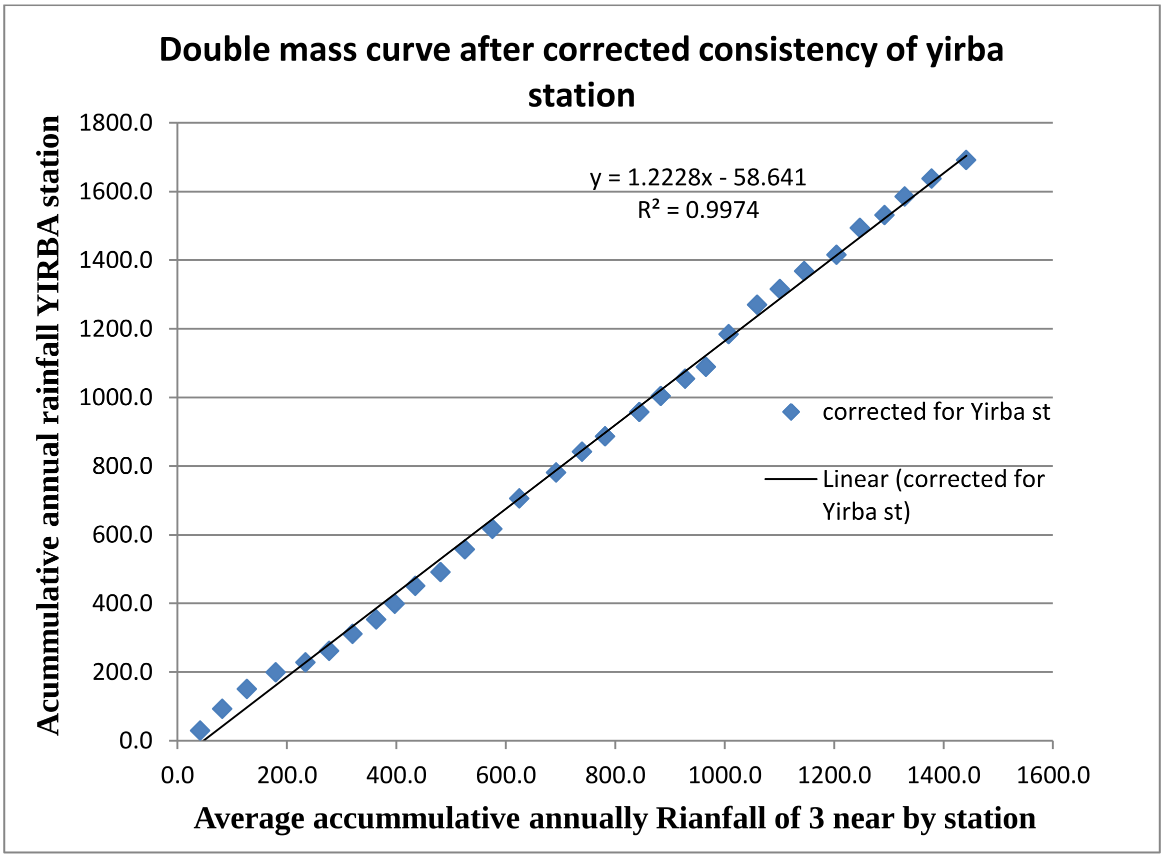


Figure 3-8: Double mass curve after corrected inconsistency of Yirba rainfall station

According to the R^2 (Coefficient of Determination) value on the graph, the data is so consistent that it doesn't need to be adjusted.

3.4.2.2 Testing the quality of data by relative standard error

The adequacy of the rainfall data series should be verified and realized before moving on to the other analysis. The data series should be considered and adequate if relative standard error ≤10%, where is the calculated as follows:

$$\text{Average daily maximum annualy rainfall of logx(ym)} = \sum \text{Y}/\text{N} \text{.............................} 3.3$$

$$\text{Average daily maximum annualy rainfall (xm)} = \sum \text{X}/\text{N} \text{.......................................} 3.4$$

$$\text{Standard deviation(xi), Sx} = \frac{\sqrt{\sum(\text{x}-\text{xm})^2}}{\text{N}-1} \text{...} 3.5$$

$$\text{Standard deviation(yi), Sy} = \frac{\sqrt{\sum(\text{y}-\text{ym})^2}}{\text{N}-1} \text{...} 3.6$$

$$\text{Skweness coefficient(Cs)} = \frac{\text{N}\sum(\text{y}-\text{ym})^3}{(\text{N}-1)(\text{N}-2)\text{Sy}^3} \text{...} 3.7$$

$$\text{relative standard error } (\delta\text{e})\% = \frac{\text{se}}{\text{xm}} * 100\% \text{..} 3.8$$

$$\text{se} = \frac{\text{Sx}}{\sqrt{\text{N}}} \text{...} 3.9$$

Whereas N= Number of year recorded rainfall data

$$se = \frac{17.0}{31^{\ 0.5}} = 3.05$$

$$\delta e = (3.05/54.58) * 100 = 5.59\%$$

5.59% ≤ 10%......ok! Therefore the data is adequate and reliable. (Appendix table 5)

3.4.2.3 Testing outlier

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

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

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

Case3; if Skweness coefficient (Cs) -0.4< Cs <+0.4 check for both outlier.

Since it is stated that the Skweness coefficient (Cs) is between -0.4 and +0.4 our recorded with respect to both outlier is within reasonable range. The data points referred to as outliers depart from the trend of the remaining data, as shown in (appendix table 5). Thus the data is checked for both lower and higher outlier.

I. Lower outlier determination

To detect the lower outlier the following equation are applied.

$$\text{Lower outlier } \mathrm{Y_L} = \mathrm{ym} - \mathrm{Kn} * \mathrm{Sy} \text{...} 3.10$$

Where: - Y_L= antilog of value calculated

ym= mean of data in log unit

S_y= for Skweness coefficient (Cs) value calculated by above equation 3.7, find Sy from log transformation series (garg book).

From table sample size for N (31year) rainfall data read Kn = 2.577 (Ven Te chow table 12.5.3 outlier test Kn values).

$$Y_L = 1.72 - (2.577 * 0.134) = 1.371$$

$$Antilog Y_L = \mathbf{23.499mm}$$

The lowest recorded daily maximum rainfall is **28.46 mm** in the 2018 (Appendix table 5) 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.

II. Higher outlier determination

To detect the higher outlier the following equation are applied.

$$\text{Higher outlier } \mathrm{Y_H} = \mathrm{ym} + \mathrm{Kn} * \mathrm{Sy} \quad \text{……………………………………………………………………}3.11$$

$$Y_H = 1.72 + (2.577 * 0.134) = 2.063$$

$$AntilogY_H = \mathbf{115.633\ mm}$$

The higher recorded daily maximum rainfall is **95.17 mm** in the 2001 (Appendix table 5) which lower than boundary value of higher outliers. Hence there are no higher outlier's data recorded.

3.5 Development of IDF curve for Yirba town

Intensity duration frequency (IDF) curve describe as the relationship between rainfall intensity, rainfall duration and return period. This curve is widely used in hydrologic, hydraulic, and water resource system design. Through analysis of daily rainfall frequency, IDF curves are produced. The development of IDF curve requires the frequency analysis is be performed of each set of annual maximum rainfall. Determine the exceedance probability distribution of rainfall intensity for each duration using the frequency analysis (M.L. Waikar and Undegaonkar Namita U, 2015). There are two options for this frequency analysis, the first one use an empirical plotting position approach to estimate the exceedance probabilities based on the observation and fit a theoretical extreme value distribution(Gumbel EVT-1, Normal, Lognormal, Log Pearson type III distribution and Pearson type III) to estimate the rainfall associated with given exceedance probabilities. Because of absence extreme rainfall values for periods less than 24hr (12hr, 6hr, 3hr…) in Yirba town station IDF curve develop to obtain the depth and intensity of 5 minute interval that used input for swmm 5.2 model. The following equation adopted for IDF development of rainfall in a given durations (M.L. Waikar and Undegaonkar Namita U, 2015).

$$\mathrm{R_{Rt}} = \frac{\mathrm{t}}{24}\frac{(\mathrm{b}+24)^{\mathrm{n}}}{(\mathrm{b}+\mathrm{t})^{\mathrm{n}}} \quad \text{………………………………………………………………………………}3.12$$

Where: $\mathrm{R_{Rt}}$= Rainfall depth Ratio Rt: R24

Rt = Rainfall depth in a given duration (hr.)

R24= 24 hr rainfall depth

b (constant)=0.3 and n(constant) =0.9

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 chi-Square test of goodness fit and selecting the best probability function can be used for this is research is **Log Pearson type III distribution method** (Ven Te Chow, 2012) and detail analysis in appendix Table 6 to 11 is specified.

$$Y_T = \mathrm{ym} + K_T \mathrm{Sy} \quad \text{......} 3.13$$

Where: X_T= (Antilog Y_T) the magnitude computed runoff (mm)

ym= mean logarithm

Sy = standard deviation of the logarithm

K_T = the frequency factor

The values of (K_T), read from table of Pearson type III (appendix table 16) Cs (Skweness coefficient) with respect to T (return period) with interpolation.

3.6 Stormwater Modelling

The EPA Stormwater Management Model (SWMM 5.2) is used to model the hydrological and hydraulic response of the area with the purpose of assessing the town's urban drainage system based on single event simulation for stormwater quantity. SWMM provides a number of benefits to utilize it for evaluating urban drainage despite its minor limitations. The model is freely available & stable programmed. Moreover, it has the ability to simulate low impact development (LID) in urban areas. Due to all these reasons, SWMM5 is found preferable than others to use for this study. On the other hand, absence of direct

GIS interface and problem to use the model for large area are considered as a limitation of the model (Rossman, L. and M. Simon, 2022).

3.6.1 Model description and capability

SWMM is a dynamic rainfall-runoff simulation model used for single event or long-term (continuous) simulation of runoff quantity and quality from primarily urban areas. The runoff component of SWMM operates on a collection of sub catchment areas that receive precipitation and generate runoff and pollutant loads. The routing portion of SWMM transports this runoff through a system of pipes, channels, storage/treatment devices, pumps, and regulators. The quantity of runoff generated within each sub catchment, and the flow rate, flow depth, and channel during a simulation period includes multiple time steps even for small catchments, runoff and consequent model predictions (Rossman, L. and M. Simon, 2022).

SWMM requires three major information for runoff quantity modeling; Physical catchment characteristics, rainfall, and infiltration. The physical catchment data are total catchment area (A), percentage of impervious area (%Imp), catchment width (W), average slope (So), surface depression storage and surface roughness (M.F.Chow, Z.Yosop, M. E. T., 2012).

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

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

For modeling accuracy to more specifically and successful calibration of SWMM essential rain gages are located within and adjacent to the catchment (Rossman, L. and M. Simon, 2022).

3.6.2 Model set up procedure

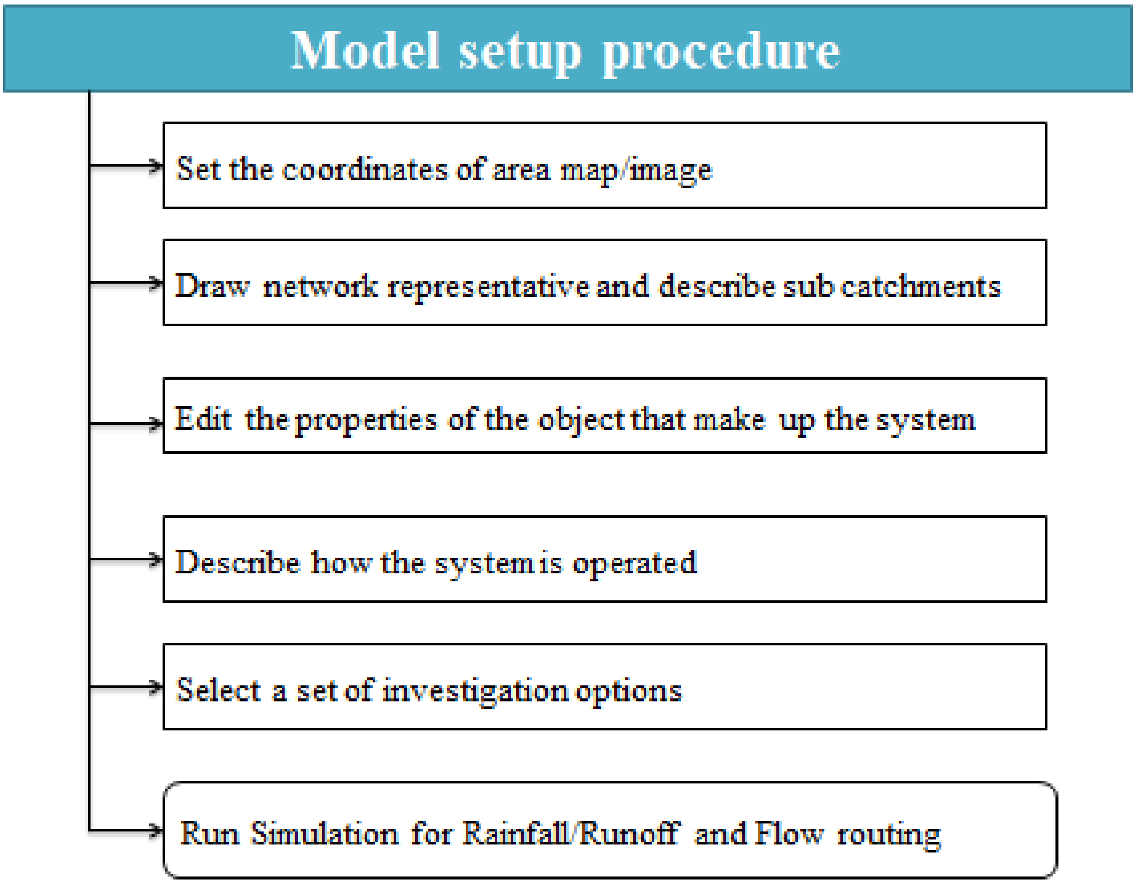
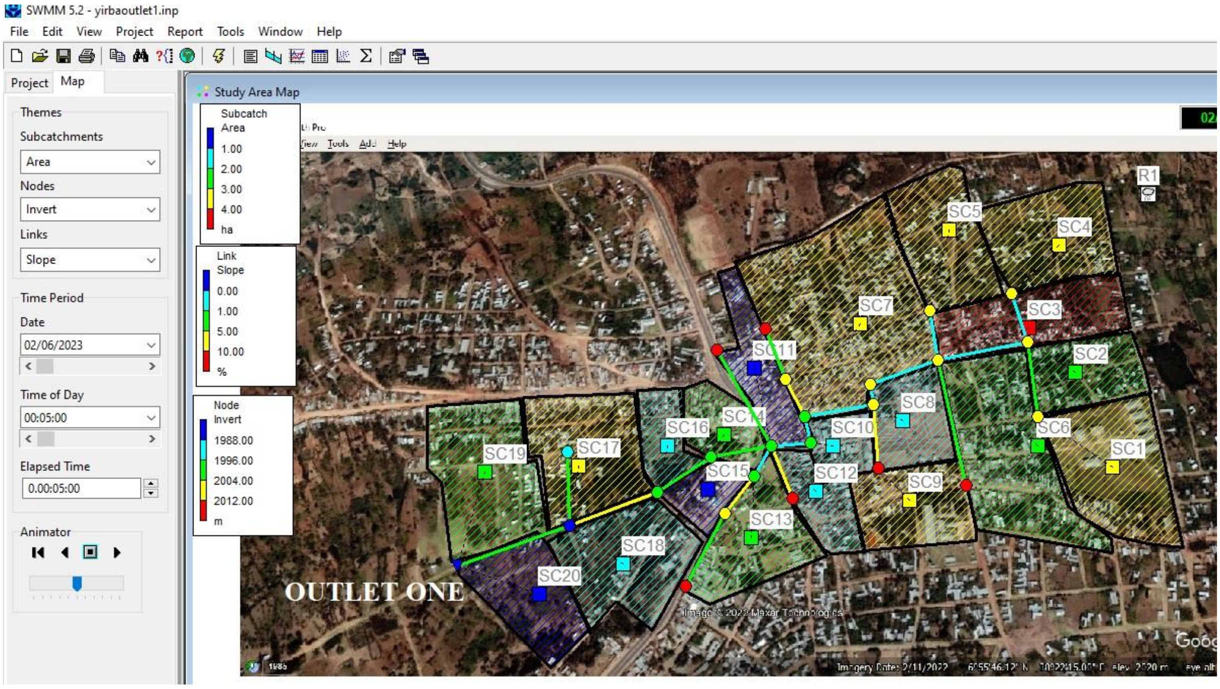


Figure 3-9: Steps of model setup procedure

3.6.3 Model preparation area

The existing land use map which was derived from Google Earth through digitization provided the majority of the data describing the physical catchment characteristics. In order to accurately delineate the sub catchment areas, it was necessary to frequently visit the field to assess the surface and subsurface drainage configurations.

The below figure 3.10 shows modeled area of outlet one starts from Yirba TVET to high school. That simulated model of each 20 sub catchments connected into 23 nodes and 23 channel and flow routing outfalls into neighbor kebele.

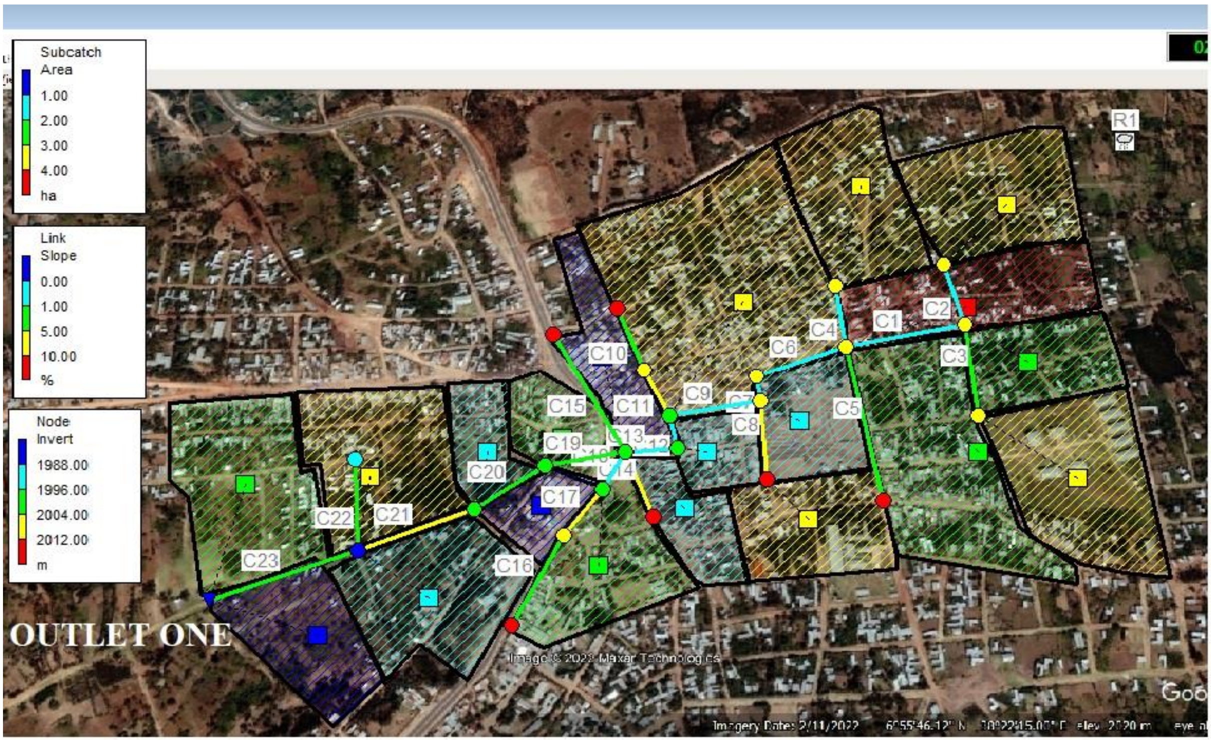


SWMM 5.2 - yirbaoutlet1.inp

Themes
Subcatchments: Area
Nodes: Invert
Links: Slope

Time Period
Date: 02/06/2023
Time of Day: 00:05:00
Elapsed Time: 0.00:05:00

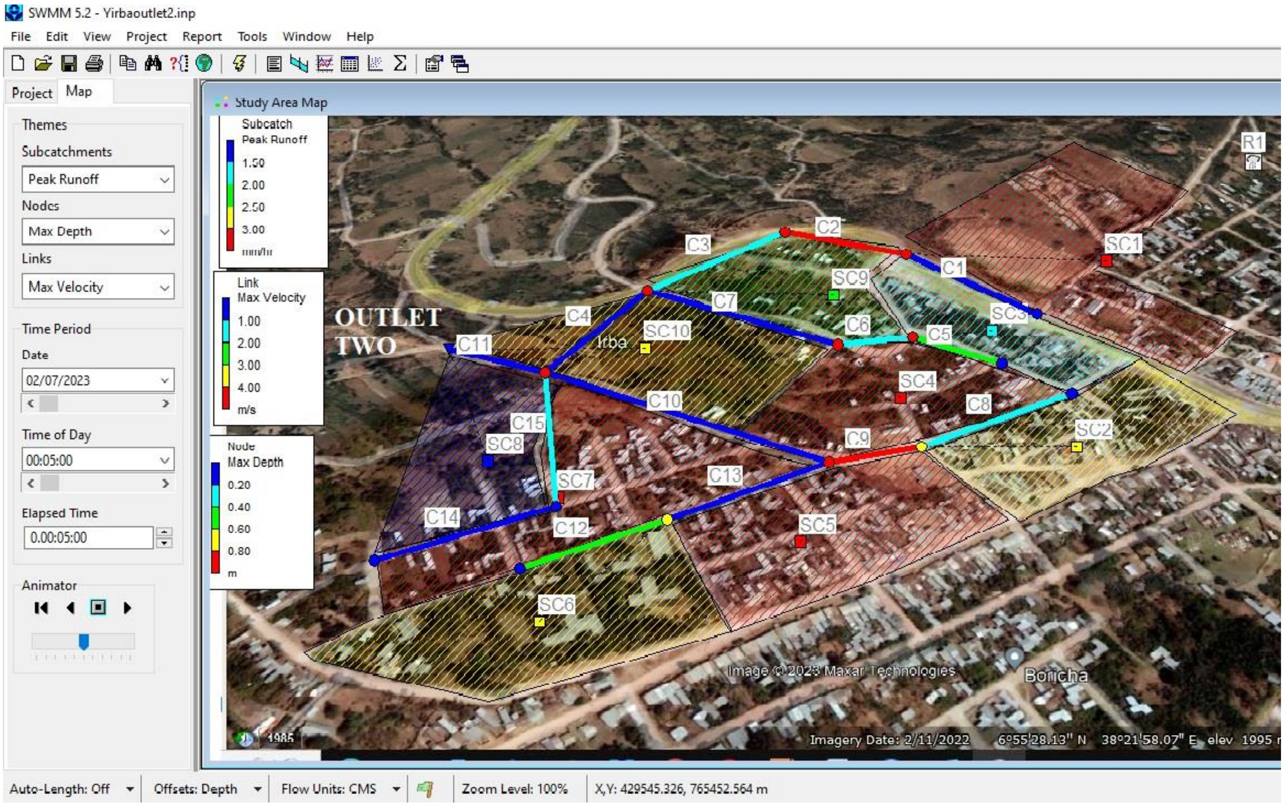
a) With sub catchments



b) With links (channels)

Figure 3-10: The outlet one 1 map area prepared with sub catchments and channels

The below figure 3.11 illustrates the outlet two modeled area which extends from the Yirba Yanase Elementary School to the Wolayta Sodo Exit via the Balela Main Road. That simulated model of each 10 sub catchments connected into 15 nodes and 15 channels and flow routing outfalls into neighbor kebele.



SWMM 5.2 - Yirbaoutlet2.inp

Themes
Subcatchments: Peak Runoff
Nodes: Max Depth
Links: Max Velocity

Time Period
Date: 02/07/2023
Time of Day: 00:05:00
Elapsed Time: 0.00:05:00

Auto-Length: Off
Offsets: Depth
Flow Units: CMS
Zoom Level: 100%
X,Y: 429545.326, 765452.564 m

Figure 3-11: The outlet two 2 map area prepared

The total model area prepared of Yirba town drainage system networks consists of 80ha area from the whole 30 sub catchments, 38 junctions and 38 channels in all two outfalls which shown on the above figure 3.10 and 3.11.

The manholes/junctions were represented as rectangular channels with varying depths in the modeling. It had assumed that there are no energy losses in the manholes. The precipitations were introduced into the model by associating each sub-catchment to the rainfall time-series.

3.7 Calibration and validation of model

The sensitivity analysis was conducted 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 Yirba town. The calibration model was able to predict the observed output with a reasonable degree of accuracy. The most parameters used for sensitivity analysis and allowable range of change proposed by (Weaver, 2019). The simulated and observed runoffs in the outlets were compared 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 (%)	50-80

3.7.1 Calibration and validation data

Flow data is required in order to calibrate and validate the model. However, flow data was not available due to the fact that flow gages were not installed to measure the quantity of flow generated from the drainage system of the town. Five conduits were selected for this purpose. Those were C9 and C13 which have an open trapezoidal channel with SC10 and SC11 sub catchments; C14, C19 and C20 which have open rectangular channel with SC12, SC14 and SC15 sub catchments respectively. This flow depth was recorded at different interval for ten (10) days from July to September to calibrate of sensitive parameters and validate SWMM 5.2 model for the area.

Table 3-5: Recorded flow depth for calibration and validation model

Gauged area	Channel type	Date of observed data	Recorded average flow depth (m)	Method calculation flow discharge using manning equation (m^3/s)
For calibration used	Rectangular	July 12 2022	0.85	Each values are filling in below calibration and validation of model section
		Sept. 01 2022	0.64	
		August 19 2022	0.90	
	Trapezoidal	August 10 2022	0.86	
		July 14 2022	0.77	
for validation used	Rectangular	Sept. 06 2022	0.68	
		Sept. 12 2022	0.56	
		August 12 2022	0.82	
	Trapezoidal	August 03 2022	0.90	
		July 21 2022	0.72	

The model was calibrated using the maximum flow depth (m) obtained from the previous table 3.5 for five days of rainfall data, and the remaining five days were used to validate the model.

The manning equation is used as a calculating tool for calibrating and validating observed data.

$$Q = \frac{1}{n} * A * R^{2/3} * S^{1/2} \ldots\ldots 3.14$$

Whereas: - Q=flow discharge in m^3/s

n=manning roughness coefficient

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

R (A/P) =hydraulic radius in m

P=wetted perimeter in m

S=channel slope in m/m fraction

3.7.2 Model calibration

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 3-6: Sensitivity parameter used to calibrate the model

Parameters	Manning Description	Recommended Value Range	Initial Used Proceed Sensitive Parameters	Final used Sensitive Parameters
N-Imperv	Manning roughness for impervious area	0.011- 0.015	0.012	0.013
N-perv	Manning roughness for pervious area	0.05-0.8	0.2	0.5
Dstore-Imperv	Depth of depression storage on impervious area	0-3	1.5	3
Dstore-Perv	Depth of depression storage for pervious area	3-10	3	7.6
Smooth concrete roughness	Manning roughness coefficient for open smooth concrete	0.0122	0.0122	0.0122

The input parameters needed to calibrate the model's final output are listed in table 3.6 above. Peak discharge flow rate determined using manning equations is $5.86m^3/s$ but peak discharge flow rate simulated using calibration values from the SWMM model is $5.81m^3/s$.

3.7.3 Model validation

The table 3.7 below lists the velocity and the discharge flow rate in the drainage systems along with a validation of this model's fit to the observed at the outlet one drainage lines.

Table 3-7: Validation flow depth observed result

Channel type	Record depth (m)	Manning roughness used (n)	channel slope (S)	Side Slope (m)	Width (m)	Area (m^2)	Perimeter (m)	Hydraulic radius (m)	Velocity (m/s)	Discharge (m^3/s)
Rectangular	0.68	0.013	0.0122		1.5	1.02	2.86	0.36	4.3	4.36
	0.56		0.0894		0.6	0.34	1.72	0.20	7.7	2.60
	0.82		0.0041		1.2	0.98	2.84	0.35	2.4	2.39
Trapezoidal	0.90	0.022	0.0030	0.25	1.0	1.10	2.86	0.39	1.3	1.46
	0.72		0.0270	0.20	0.8	0.68	2.27	0.30	3.3	2.27

According to result table 3.7, the channel velocity of 4.3 m/s and 7.7 m/s is high values. The material for the trapezoidal open channel is made of stone masonry with the floor stone pavement joined by concrete material. The material for the rectangular open channel is made of high smooth quality concrete reinforced and the channel has a high channel slope. Due to the great strength of the channel material used in construction, the high value of velocity shown in table 3.7 has no impact on the channel.

Table 3-8: Summary of the model result and observed values

Date of observed data	Recorded average flow depth (m)	Flow rate using manning equation(m^3/s)	Flow rate using SWMM5.2 (m^3/s)
Sept. 12 2022	0.56	2.60	2.32
Sept. 06 2022	0.68	4.36	4.54
July 21 2022	0.72	2.27	2.37
August 12 2022	0.82	2.39	2.68
August 03 2022	0.9	1.46	1.44

The validation of the model with the observed flow rate calculated by manning equation and simulated by SWMM 5.2 used at the same site network of channel is validated showed the below figure 3.12 charts

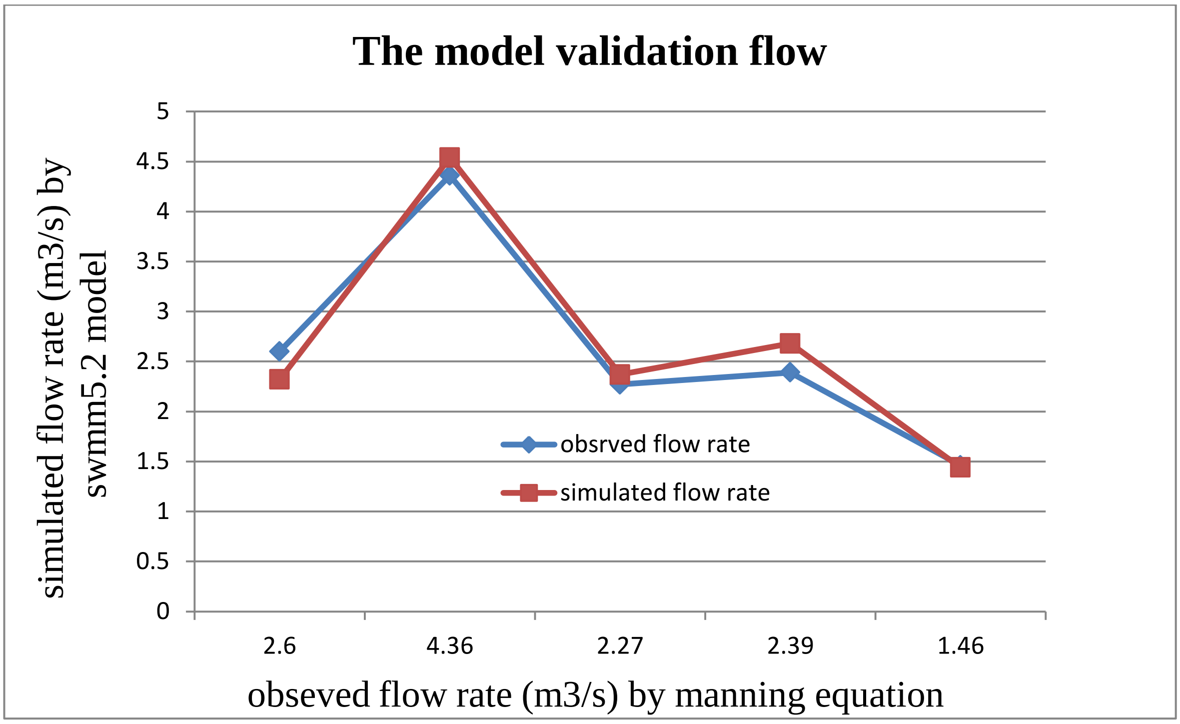


Figure 3-12: The Model Validation Flow chart

3.7.4 Criteria of evaluation Model performance

The reliability of the model was tested using Coefficient of Determination (R^2) and Nash–Sutcliffe efficiency coefficient (NSE).

$$\text{Coefficient of determination}(\mathrm{R}^2) = \left[\frac{\sum_{t=1}^{n}(q_t^{obs}-q_t^{Av.obs})(q_t^{sim}-q_t^{Av.sim})}{\sqrt{\sum_{t=1}^{n}(q_t^{obs}-q_t^{Av.obs})^2}\sqrt{\sum_{t=1}^{n}(q_t^{sim}-q_t^{Av.sim})^2}}\right]^2 \text{..3.15}$$

The R2 value, which ranges from 0 to 1, indicates how much of the observed dispersion is explained by the prediction. A value of zero indicates that there is absolutely no correlation, while a value of one indicates that the dispersion of the prediction and the observation is the same (J.Y. Wu et al., 2013).

$$\text{Nash} - \text{sutcliffe coefficient(NSE)} = 1 - \frac{\sum_{t=1}^{n}(q_t^{obs}-q_t^{sim})^2}{\sqrt{\sum_{t=1}^{n}(q_t^{obs}-q_t^{Av.obs})^2}} \text{...................................3.16}$$

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

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

$q_t^{sim}$and $q_t^{Av.sim}$ = are the simulated and average flow respectively

t=at time t and

n= is the total number of time steps

Level of performance of the model is characterized according to table 3.9

Table 3-9: Performance evaluation criteria for both methods (Moriasi, 2015).

Method	Output Response	Temporal Scale	Performance Evaluation Criteria			
			Very Good	Good	Satisfactory	Not satisfactory
R^2	Flow	Day/Month/ Year	$R^2>0.85$	$0.75<R^2\leq0.85$	$0.60< R^2\leq0.75$	$R^2\leq0.60$
NSE	Flow	Day/Month/ Year	NSE>0.85	0.75<NSE≤0.85	0.60<NSE≤0.75	NSE ≤0.60

These two attributes play a crucial role in the planning and study of urban drainage systems. 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. Flow data was calibrated for daily flows (Eyosias, 2018).

3.7.5 Model performance evaluation

With the help of the methods of Coefficient of Determination (R^2) and Nash-Sutcliffe Efficiency Coefficient (NSE) in the detailed evaluation shown in Table 3.10 below, the model performance was evaluated by comparing the observed and simulated data.

Table 3-10: model performance evaluation

Recorded average flow depth(m)	Flow rate using manning equation (m^3/s)	Flow rate using SWMM5.2 (m^3/s)	$q_t^{obs} - q_t^{Av.obs}$	$q_t^{sim} - q_t^{Av.sim}$	$(q_t^{obs} - q_t^{Av.obs})*(q_t^{sim} - q_t^{Av.sim})$	$(q_t^{obs} - q_t^{Av.obs})^2$	$(q_t^{sim} - q_t^{Av.sim})^2$	$(q_t^{obs} - q_t^{sim})^2$
0.56	2.60	2.32	-0.02	-0.350	0.0056	0.0003	0.1225	0.0784
0.68	4.36	4.54	1.74	1.870	3.2613	3.0415	3.4969	0.0324
0.72	2.27	2.37	-0.35	-0.300	0.1038	0.1197	0.0900	0.0100
0.82	2.39	2.68	-0.23	0.010	-0.0023	0.0511	0.0001	0.0841
0.90	1.46	1.44	-1.16	-1.230	1.4219	1.3363	1.5129	0.0004
Total	13.08	13.35			4.7903	4.5489	5.2224	0.2053
Average	2.62	2.67						

$$\text{Coefficient of Determination } (\mathrm{R}^2) = \left(\frac{4.7903}{\sqrt{4.5489}\sqrt{5.2224}}\right)^2$$

$$\mathrm{R}^2 = \mathbf{0.97}$$

$$\text{Nash} - \text{Sutcliffe Efficiency (NSE)} = 1 - \left(\frac{0.2053}{\sqrt{4.5489}}\right)$$

$$\mathrm{NSE} = \mathbf{0.90}$$

Overall, the calibration achieved an excellent fit with observed daily flows for the simulation period, with $\mathbf{R^2 = 0.97}$ and **NSE = 0.90.** This calibration and validation result showed that the model structure and parameters matched the runoff production pattern, and the calibrated model was adequate for simulating runoff in the research area.

3.8 Evaluate alternatives mitigation measures for drainage problems

Urban drainage areas that get too much stormwater runoff are at risk of flooding, which causes serious economic losses, traffic jams, and even localized fatalities. So that is need to decrease this flood risk from urban area are by improving existing drainage system capacity, evolution Sustainable Urban Drainage Systems (SUDS) and adding best management practices (BPMs) or low impact development(LID) techniques in the drainage system. (Zhou, 2014).

For this study, the natural system of storm water management which is Low impact development (LID) technique used to reduce the negative influence of water quantity. LID is redeveloped using the best management practices (BMPs) of reduce peak flows and attentiveness with sustainable urban drainage system (SUDS). By boosting infiltration, LID measures primarily minimize the flood peak and volume. This study only focused on decrease the quantity of surface runoff, and the appropriate practice measure was the use of vegetative swales and infiltration trenches. In this study, the goals of minimizing peak runoff and runoff volume were quantified using various LID approaches and compared to the state of the environment using the results from the EPA SWMM5.2.

Urban flood management is a primary goal of urban planning techniques that control storm water flow at its source. One of the most common strategies for effective storm water management and flood control in urban areas is the use of LID. Urban development and flood management with LID is smarter and more sustainable. LID reduces the risk of flood through several non-structural and structural measures. Some of these measures include Green Infrastructure like as Bio retention cells, rooftop disconnection, Infiltration trench, vegetative swales and rain garden.

3.8.1 LID Improvement Methods

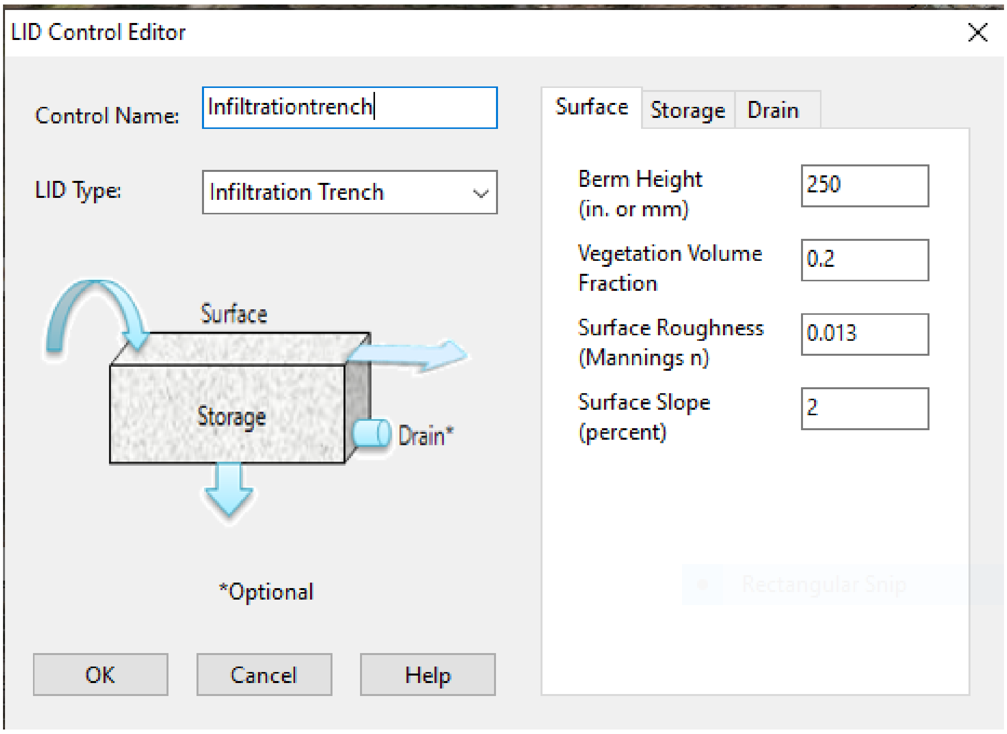
The low impact development techniques created to absorb surface runoff and provide it some combination of detention, infiltration, and evapotranspiration. They are considered as being part of a particular sub catchment (Rossman, L. and M. Simon, 2022).

The design features of each relevant layer, such as thickness, void volume, hydraulic conductivity, drain characteristics, etc., are used by the LID Control Editor when a user adds a particular type of LID control object to a SWMM project. By changing the sub catchment's LID Controls property, these LID objects can then be added to selected sub catchments at any desired sizing (or areal coverage) (Eyosias, 2018).

The surface, soil, storage, and under drain portions of a LID unit are represented by a number of interconnected, fully mixed layers in LID models. The storage in each of the layers is dynamically controlled by infiltration, drainage, and overflow. SWMM can explicitly model bio retention cells, infiltration trenches, porous pavement, rain gardens, vegetated swales, and rooftop disconnection. (Niazi, M. et al, 2017)

3.8.1.1 Infiltration trenches

The subsurface infiltration of runoff is facilitated by infiltration trenches, which are engineered structures that provide storage. Typically, infiltration trenches are long, narrow, and aggregate-filled. A trench used for infiltration was used to direct runoff from the study area into the LID area. They can be modeled as totally permeable sub-catchments with storage depths equal to the corresponding depth of the trench's pore space. Infiltration Trenches are small, gravel-filled ditches used to catch runoff from impermeable surfaces further up a slope. They give storage space and more time for runoff to saturate the underlying native soil (Acharya, 2018).



LID Control Editor

Control Name: Infiltrationtrench

LID Type: Infiltration Trench

Berm Height (in. or mm): 250

Vegetation Volume Fraction: 0.2

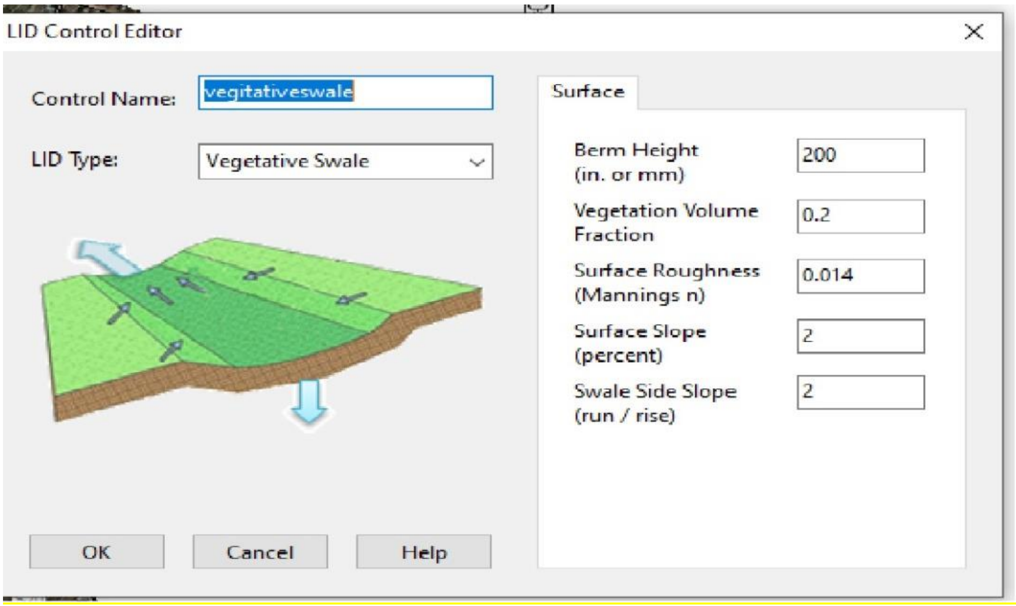
Surface Roughness (Mannings n): 0.013

Surface Slope (percent): 2

Figure 3-13: LID editor practices of Infiltration trenches within swmm5.2

3.8.1.2 Vegetative Swales

Vegetative Swales are channels or depressed areas with sloping sides that are covered in grass and other vegetation. They slow down the flow of collected runoff, giving it more time to infiltrate the topsoil. (Rossman, L. and M. Simon, 2022)



Control Name: vegitativeswale

LID Type: Vegetative Swale

Berm Height (in. or mm): 200

Vegetation Volume Fraction: 0.2

Surface Roughness (Mannings n): 0.014

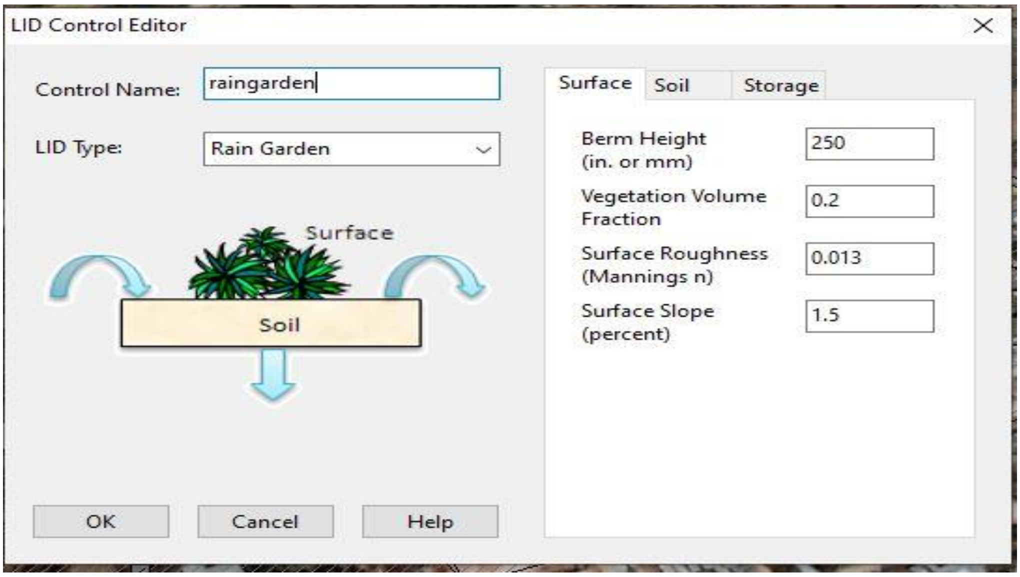
Surface Slope (percent): 2

Swale Side Slope (run / rise): 2

Figure 3-14: LID editor practices of Vegetative Swales within swmm5.2

3.8.1.3 Rain garden

Rain gardens are a type of bio-retention cell consisting of just the engineered soil layer with no gravel bed below it (Rossman, L. and M. Simon, 2022). The purpose of a rain garden is to collect rainfall that flows off the grass, roof, and driveway by replacing a portion of the lawn with maintained features. This shallow depression has thick, loose soil that naturally filters and absorbs runoff, keeping it out of the storm drain system and consequently into of the waterways.



Control Name: raingarden

LID Type: Rain Garden

Berm Height (in. or mm): 250

Vegetation Volume Fraction: 0.2

Surface Roughness (Mannings n): 0.013

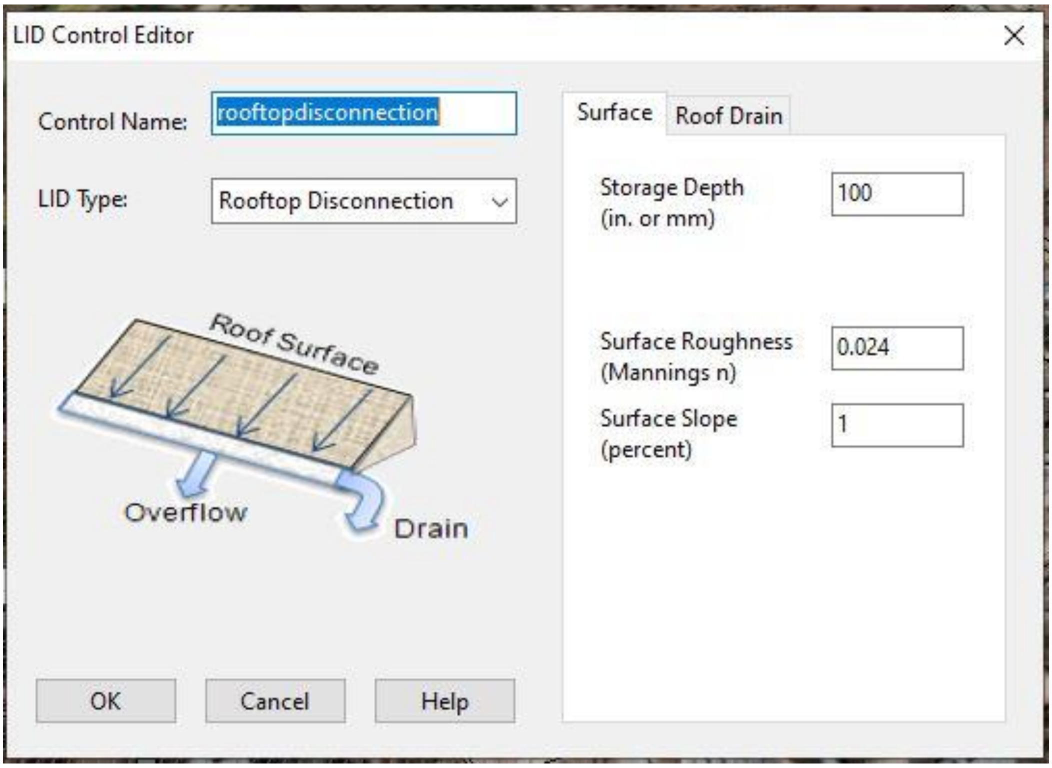
Surface Slope (percent): 1.5

Figure 3-15: LID editor practices of Rain garden within swmm5.2

This study used LID control editor in Rain garden surface parameter value shows in figure 3.15 above are used 250mm berm height, 0.2 vegetative volume fraction, 0.013 surface roughness and 1.5 surface slope in percent; all value depends on surface nature of the town.

3.8.1.4 Rooftop Disconnection

Rooftop Disconnection has downspouts discharge to pervious landscaped areas and lawns instead of directly into storm drains. It can also model roofs with directly connected drains that overflow onto pervious areas. (Rossman, L. and M. Simon, 2022)



LID Control Editor

Control Name: rooftopdisconnection

LID Type: Rooftop Disconnection

Storage Depth (in. or mm) 100

Surface Roughness (Mannings n) 0.024

Surface Slope (percent) 1

Figure 3-16: LID editor practices of Rooftop disconnection within swmm5.2

In the aforementioned figures 3.13, 3.14, 3.15, and 3.16, LID controls editors are used in the SWMM tool to model the best alternative mitigation operations to reduce drainage overflow of urban drain systems in their storage beds in order to transport extra captured runoff off of the site and prevent the unit from flooding.

Table 3-11: Designing and modeling parameters of LID types

Designing and modeling LID	Layer	Properties	LID types			
			Infiltration trench	Rain garden	Vegetative swale	Rooftop disconnection
Designing parameters of LID units	Surface	Berm Height (mm)	250	250	200	
		Vegetative Volume Fraction	0.2	0.2	0.2	
		Surface Roughness	0.013	0.024	0.013	0.024
		Surface Slope (%)	2	1.5	2	1
		Swale side slope(run/rise)			2	
		Storage depth (mm)				100
	Storage	Thickness (height) (mm)	450	0		
		Void ratio (voids/solids)	0.75	0.75		
		Seepage rate (mm/hour)	0.35	0.35		
		Clogging Factor	0.1	0		
	Soil	Thickness(mm)		150		
		Porosity(volume fraction)		0.5		
		Field capacity(volume fraction)		0.2		
		Wilting points(volume fraction)		0.1		
		Conductivity (mm/hr.)		0.04		
		Conductivity slope		10		
		Suction head(mm)		3.5		
	Drain	Flow coefficient	0	0		
		Flow exponent	0	0		
		Offset height (mm)	4	0		
		Flow capacity (mm/hr.)				1
Modeling LID parameters	Areas of each unit from total (%)		20	20	20	20
	Number of units		2	2	2	2
	Surface width per(m) unit		2.42	2.42	2.42	1
	Initially saturated (%)		1.5	1.5	1.5	1.5
	Impervious area treated (%)		75	80	85	85

Table 3.11 above illustrates the modeling and designing of the surface, storage, and soil layers for the LID control editor. The EPA SWMM5.2 tool manual's recommended values were incorporated for setting the parameters for each type of LID and applied to the drainage systems.

3.8.2 LID measures in the model area

Two best alternative LID (Vegetative Swales & Infiltration Trench) measures were used in the model area in various sub catchments. The aforementioned actions were chosen due to their unique ability to filter particles and significantly reduce stormwater runoff. The physical characteristics of the sub catchment have been used to specify the parameters of both structures.

The overall goal of applying structural LID measures in the model area is to absorb a sizable proportion of stormwater runoff that is generated from the town's slope area and improve some stormwater infiltration into the ground. In addition, the structures help to reduce soil erosion near drainage outfalls.

Table 3-12: LID measures in detail

Outfall names	List of Sub Catchment	Infiltration trench				Vegetative swale			
		Pcs	Length (m)	Width (m)	Total area(m^2)	Pcs	Length (m)	Width (m)	Total area(m^2)
Outlet one (O1)	SC6	1	200	2.42	484.0	-			
	SC8	2	250	2.42	1210.0	-			
	SC10	2	200	2.42	968.0	-			
	SC12	1	160	2.42	387.0	-			
	SC13	1	160	2.42	387.0	-			
	SC15	-				1	140	2.56	358.4
	SC18	-				1	200	2.56	512.0
	SC20	-				1	200	2.56	512.0
Outlet two (O2)	SC7	1	308	2.42	745.4	-			
	SC8	-				1	160	2.56	409.6
	SC9	1	250	2.42	605.0	-			
Total					**4786.4**				**1792.0**

Table 3.12 shows that infiltration trenches and vegetative swales, each with an area of 4786.4 m^2 and 1792 m^2, respectively, have been modeled using the design parameters.

CHAPTER 4 RESULTS AND DISCUSSIONS

4.1 IDF curve formulation

4.1.1 Formulation of IDF curve for Yirba station

The intensity duration frequency curve was formed using daily rainfall data over thirty one (31) years, from 1992 to 2022 G.C, collected from the Yirba town rainfall gauge station of the Ethiopian Meteorological Agency (NMA). The SWMM model uses the rainfall intensity determined by the IDF curve as an input parameter for this study. Detail analysis in appendix Table 6 to 12 is specified.

Table 4-1: Yirba IDF curve development

Duration (min)	Duration (hr)	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	0.083	91.04	117.96	135.02	155.85	170.89	185.60
10	0.167	76.27	98.82	113.12	130.56	143.17	155.49
15	0.25	65.78	85.24	97.57	112.62	123.49	134.11
30	0.5	46.95	60.84	69.64	80.38	88.14	95.72
60	1	30.33	39.30	44.99	51.93	56.94	61.84
120	2	18.15	23.52	26.92	31.07	34.07	37.00
180	3	13.12	16.99	19.45	22.45	24.62	26.74
360	6	7.33	9.50	10.87	12.55	13.76	14.94
720	12	4.01	5.20	5.95	6.87	7.53	8.18
1440	24	2.17	2.82	3.23	3.72	4.08	4.43

The best probability function was established by comparing all of the probability distribution functions using the chi-square test of goodness of fit. The function that produced the least chi-square value was chosen as the best probability function. From the final result table 4.1 the best fit probability is Log Pearson type III distribution and that used to developed IDF curve of YIRBA town as shown figure below 4.1 and detail analysis in appendix Table 6 to 12 is specified.

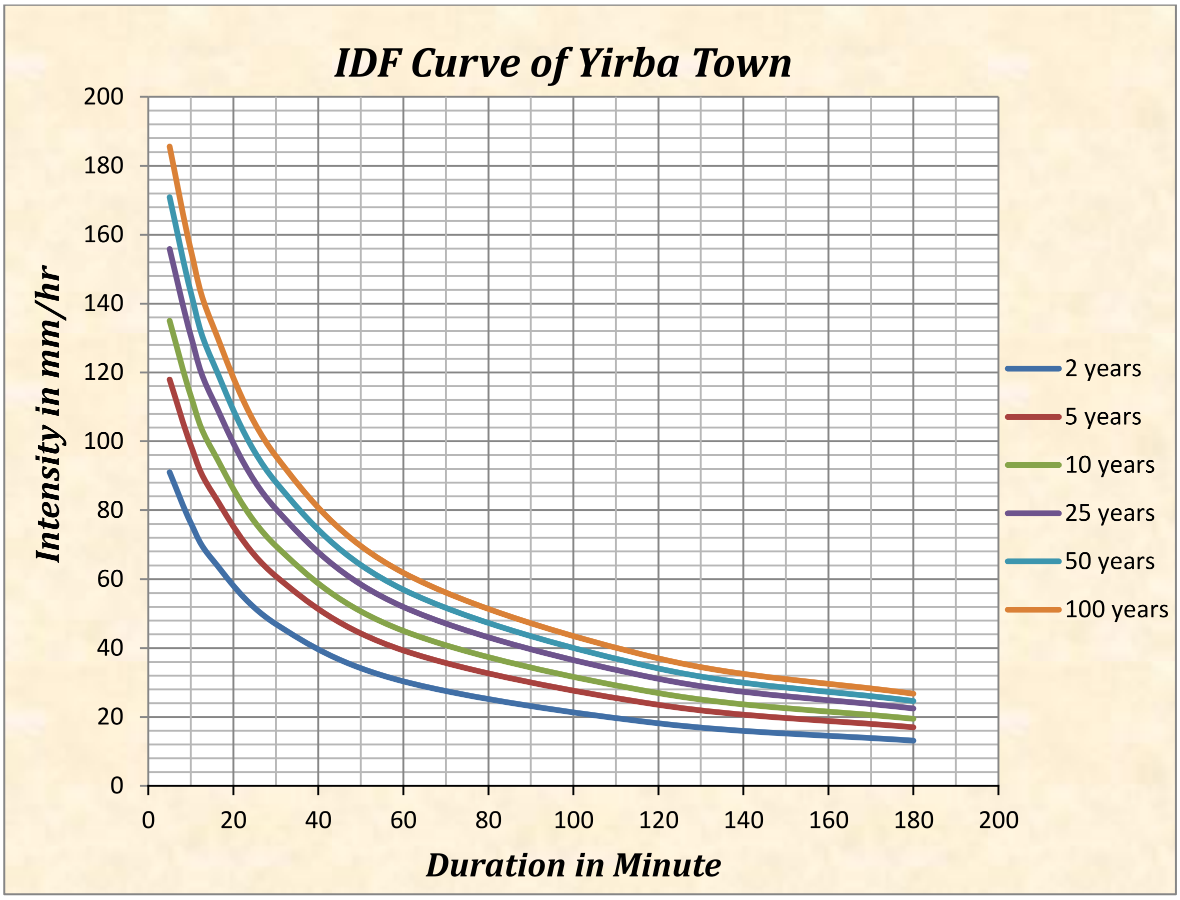


Figure 4-1: Developed IDF curve for Yirba town

4.1.2 Comparison between Yirba and ERA IDF Curve

Any probability distribution can be used 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 developed. According to the ERA (2013) design drainage manual, the town of Yirba is categorized to the B2 rainfall region. As shown below table 4.2, the IDF curve developed for Yirba town is compared to the IDF curve from ERA (2013). For this study IDF curve from hydrological section of Yirba town is applicable, but the developed IDF curve is necessary to interrelated calculated IDF curve with ERA drainage design manual even the values of rainfall intensity are difference. In ERA drainage design manual Ethiopia is divided in to several hydrological regions which display similar rainfall patterns. Yirba town falls in region B2 of this division.

Table 4-2: Comparison of IDF curves

Return period	**Duration in min.**											
	10		**30**		**60**		**90**		**120**		**180**	
	Yirba	ERA	Yirba	ERA	Yirba	ERA	Yirba	ERA	Yirba	ERA	Yirba	ERA
2	76.27	78	46.95	48	30.33	31	23	25	18.15	21	13.12	15
5	98.82	100	60.84	61	39.30	40	29	31	23.62	26	16.99	18
10	113.12	114	69.64	70	44.99	45	34	35	26.92	30	19.45	21
25	130.56	131	80.38	81	51.93	52	40	41	31.07	34	22.45	25
50	143.17	145	88.14	89	56.94	57	44	45	34.07	36	24.62	26
100	155.47	157	95.72	97	61.84	66	48	49	37.00	40	26.74	30

The difference between ERA (2013) and self-made IDF curves is not significant or the values of rain fall intensity are all most the same. For this research purpose, a self-made (Yirba town) IDF curve was chosen for further analysis because it is made for that particular area.

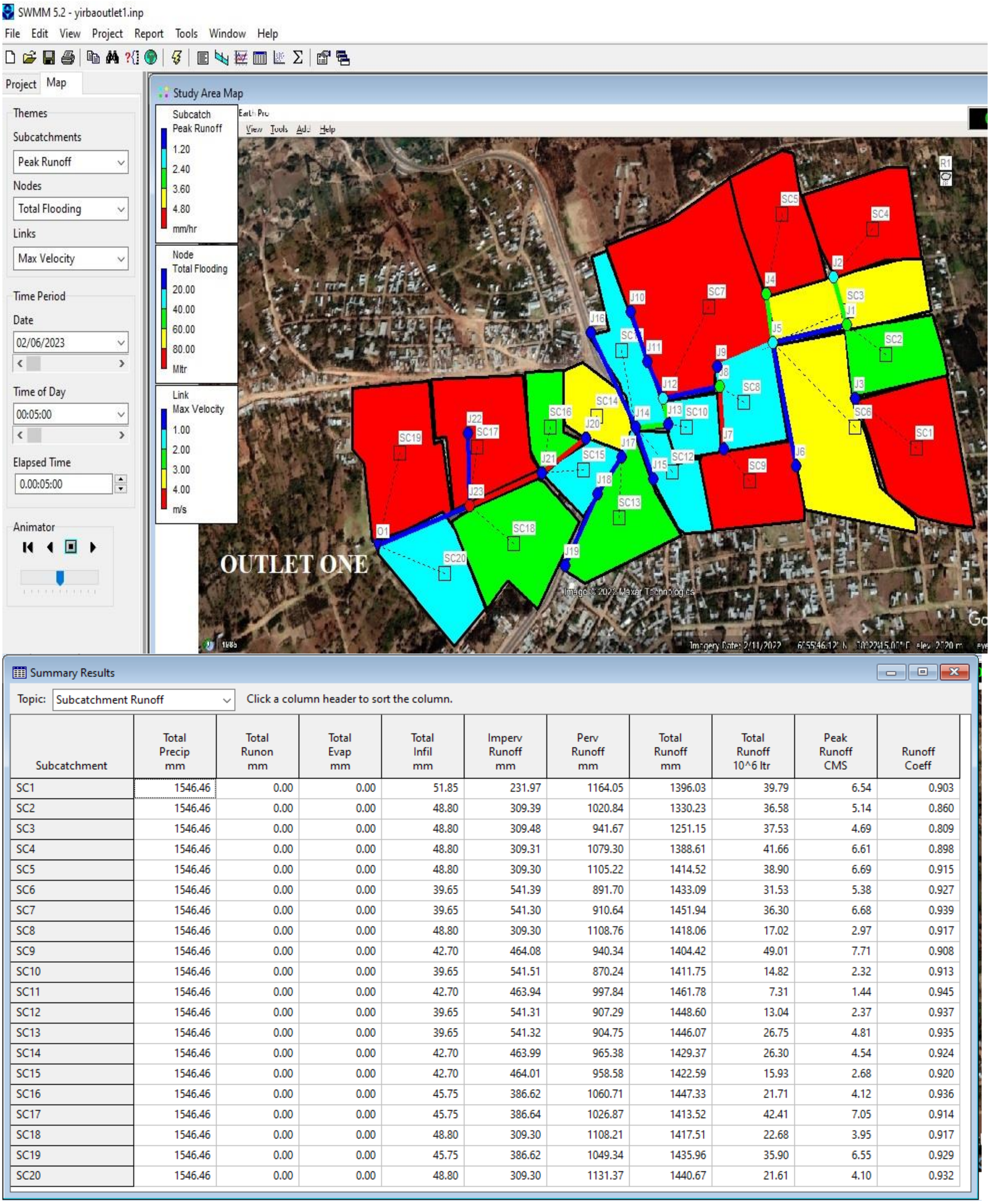
4.2 Model result of from SWMM5.2 tool

This study result of the model fully understanding of the town drainage system performance under multiple working condition one to identify stormwater management issues and existing urban drainage, secondly to assess the hydraulic performance of storm water drainage infrastructures and finally to evaluate alternatives for drainage problem mitigation measures by using hydrological model.

The total sub catchments of the study area are generating runoff from daily precipitation taken as extreme events from each year from 1992-2022 rainfall data. The project of sub catchment used as input for drainage systems connect through each nodes and simulate runoff in the network system of swmm5.2 model in a outfall study.

4.2.1 Result of Outlet one study

4.2.1.1 Runoff from sub catchments



Themes

Subcatchments: Peak Runoff

Nodes: Total Flooding

Links: Max Velocity

Time Period

Date: 02/06/2023

Time of Day: 00:05:00

Elapsed Time: 0.00:05:00

Summary Results

Topic: Subcatchment Runoff

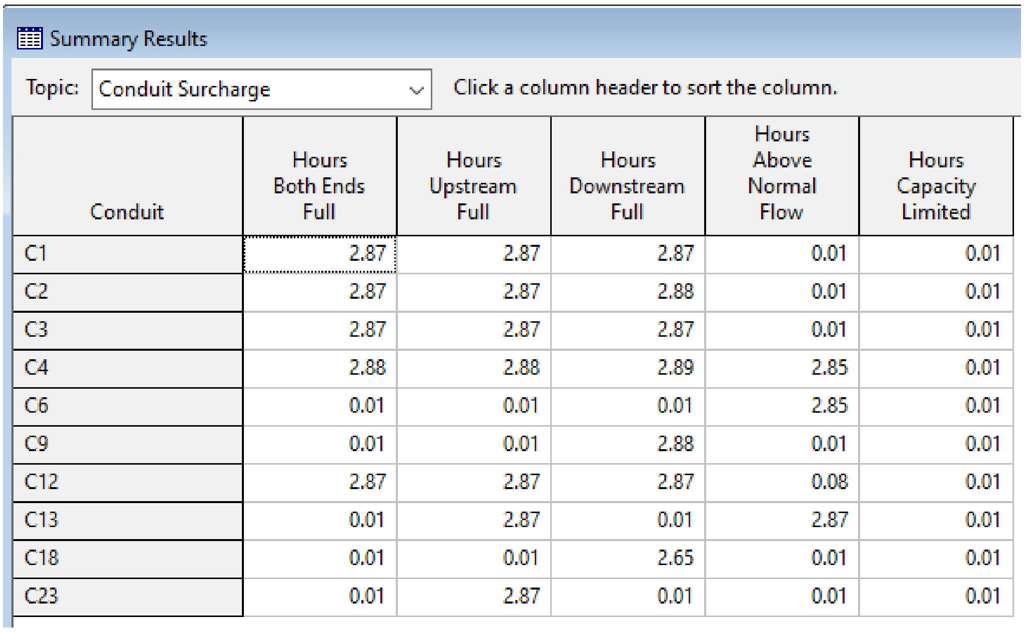
Click a column header to sort the column.

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	1546.46	0.00	0.00	51.85	231.97	1164.05	1396.03	39.79	6.54	0.903
SC2	1546.46	0.00	0.00	48.80	309.39	1020.84	1330.23	36.58	5.14	0.860
SC3	1546.46	0.00	0.00	48.80	309.48	941.67	1251.15	37.53	4.69	0.809
SC4	1546.46	0.00	0.00	48.80	309.31	1079.30	1388.61	41.66	6.61	0.898
SC5	1546.46	0.00	0.00	48.80	309.30	1105.22	1414.52	38.90	6.69	0.915
SC6	1546.46	0.00	0.00	39.65	541.39	891.70	1433.09	31.53	5.38	0.927
SC7	1546.46	0.00	0.00	39.65	541.30	910.64	1451.94	36.30	6.68	0.939
SC8	1546.46	0.00	0.00	48.80	309.30	1108.76	1418.06	17.02	2.97	0.917
SC9	1546.46	0.00	0.00	42.70	464.08	940.34	1404.42	49.01	7.71	0.908
SC10	1546.46	0.00	0.00	39.65	541.51	870.24	1411.75	14.82	2.32	0.913
SC11	1546.46	0.00	0.00	42.70	463.94	997.84	1461.78	7.31	1.44	0.945
SC12	1546.46	0.00	0.00	39.65	541.31	907.29	1448.60	13.04	2.37	0.937
SC13	1546.46	0.00	0.00	39.65	541.32	904.75	1446.07	26.75	4.81	0.935
SC14	1546.46	0.00	0.00	42.70	463.99	965.38	1429.37	26.30	4.54	0.924
SC15	1546.46	0.00	0.00	42.70	464.01	958.58	1422.59	15.93	2.68	0.920
SC16	1546.46	0.00	0.00	45.75	386.62	1060.71	1447.33	21.71	4.12	0.936
SC17	1546.46	0.00	0.00	45.75	386.64	1026.87	1413.52	42.41	7.05	0.914
SC18	1546.46	0.00	0.00	48.80	309.30	1108.21	1417.51	22.68	3.95	0.917
SC19	1546.46	0.00	0.00	45.75	386.62	1049.34	1435.96	35.90	6.55	0.929
SC20	1546.46	0.00	0.00	48.80	309.30	1131.37	1440.67	21.61	4.10	0.932

Figure 4-2: Outlet one sub catchment runoff result

According to figure 4.2's simulation results, all 20 sub catchments together have a total area of 60 hectares. The total runoff from all 20 sub catchments is 96.34 m^3/sec, with sub catchment (S9) generating the highest peak runoff at 7.71 m^3/sec.

4.2.1.2 Channels and junctions flooded



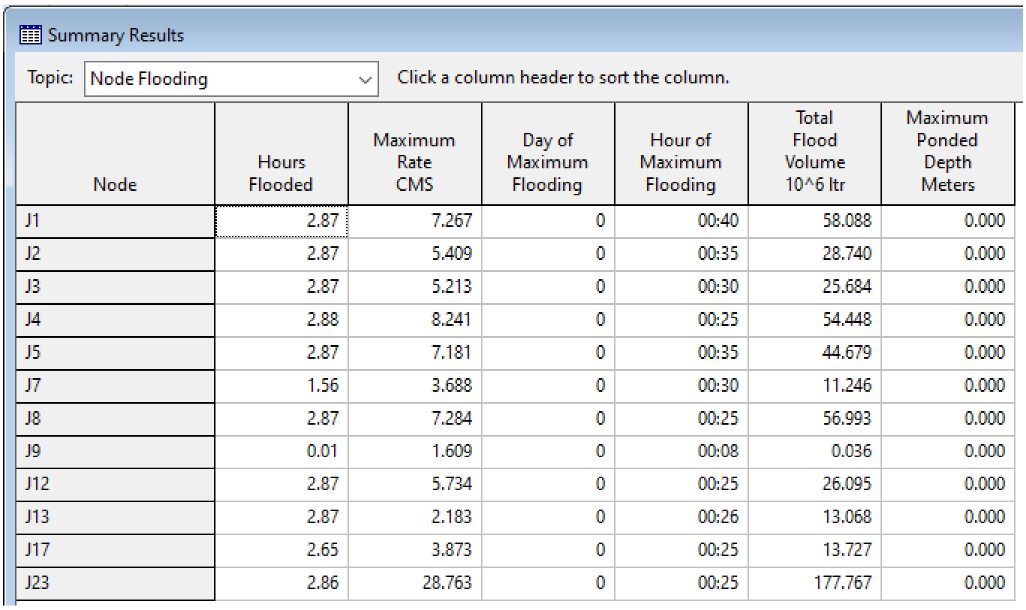
Topic: Conduit Surcharge

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
C1	2.87	2.87	2.87	0.01	0.01
C2	2.87	2.87	2.88	0.01	0.01
C3	2.87	2.87	2.87	0.01	0.01
C4	2.88	2.88	2.89	2.85	0.01
C6	0.01	0.01	0.01	2.85	0.01
C9	0.01	0.01	2.88	0.01	0.01
C12	2.87	2.87	2.87	0.08	0.01
C13	0.01	2.87	0.01	2.87	0.01
C18	0.01	0.01	2.65	0.01	0.01
C23	0.01	2.87	0.01	0.01	0.01

Figure 4-3: The outfall one flooded channels

According to the results of table 4.3, there have been overflows in the following channels: C1 through C4, C6, C9, C12, C13, C18, and C23, which have limited the capacity of the constructed channel infrastructure to convey flow.



Topic: Node Flooding

Click a column header to sort the column.

Node	Hours Flooded	Maximum Rate CMS	Day of Maximum Flooding	Hour of Maximum Flooding	Total Flood Volume 10^6 ltr	Maximum Ponded Depth Meters
J1	2.87	7.267	0	00:40	58.088	0.000
J2	2.87	5.409	0	00:35	28.740	0.000
J3	2.87	5.213	0	00:30	25.684	0.000
J4	2.88	8.241	0	00:25	54.448	0.000
J5	2.87	7.181	0	00:35	44.679	0.000
J7	1.56	3.688	0	00:30	11.246	0.000
J8	2.87	7.284	0	00:25	56.993	0.000
J9	0.01	1.609	0	00:08	0.036	0.000
J12	2.87	5.734	0	00:25	26.095	0.000
J13	2.87	2.183	0	00:26	13.068	0.000
J17	2.65	3.873	0	00:25	13.727	0.000
J23	2.86	28.763	0	00:25	177.767	0.000

Figure 4-4: The outfall one flooded junctions

The results of Table 4.4 show that nodes flood in the sections between Junctions J1 and J5, J7 to J9, J12, J13, J17, and J23. The drainage system's nodes and channels are overflowing as a result.

4.2.1.3 Water Elevation profile and outfall charged

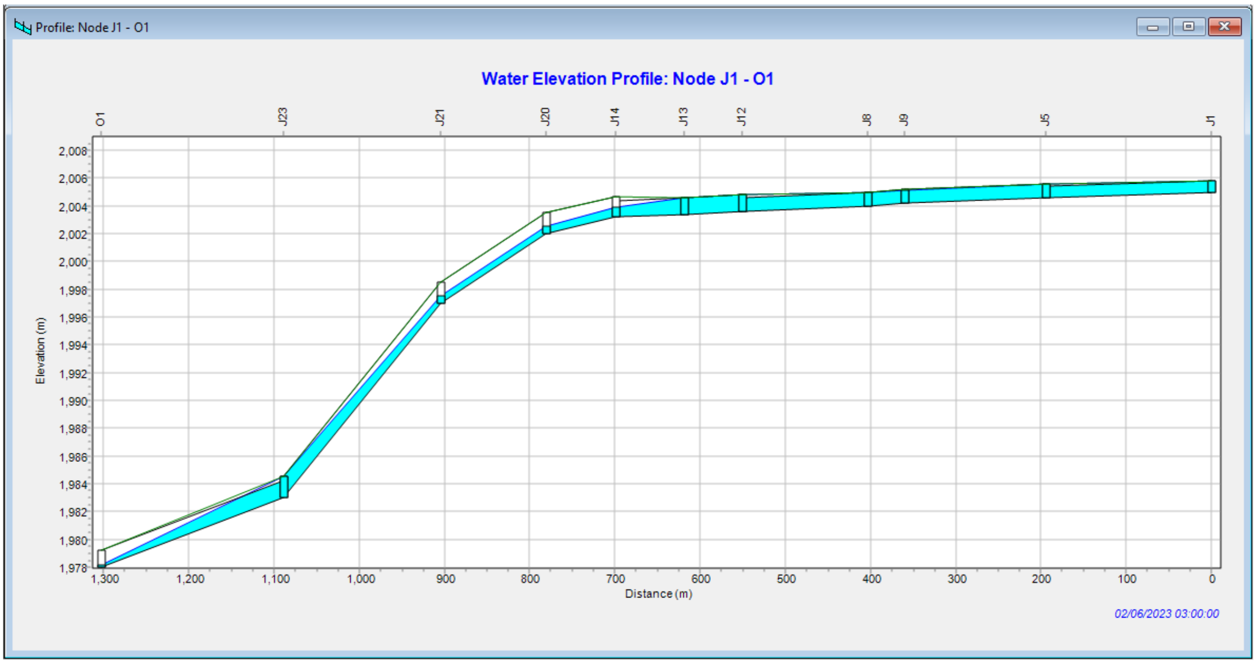
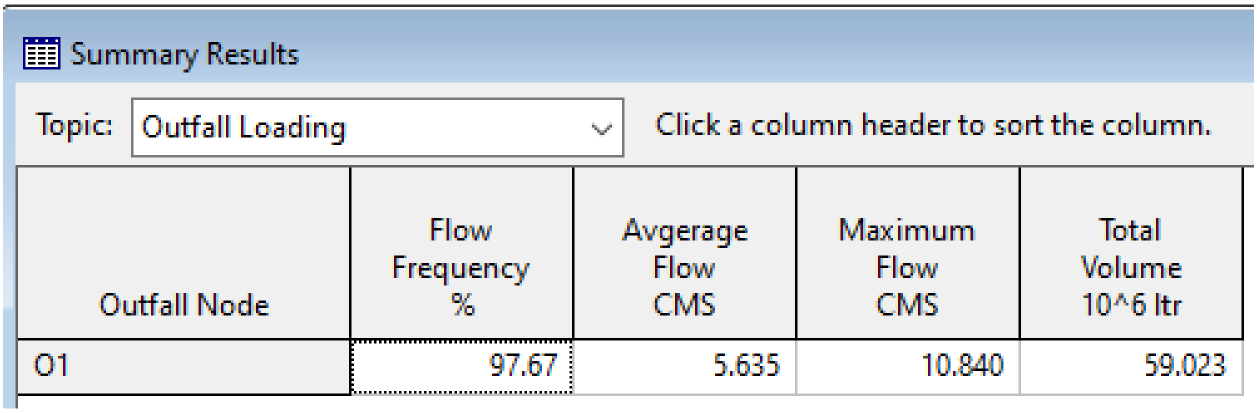


Figure 4-5: Water elevation profile from junction J1 to outfall one

The water profile plot in figure 4.5 gives a view of each sub-catchment that was used for simulation. Water elevation overflows at junction one (J1), J5, J8, J9, J12, J13 and J23 from this network profile of J1 to outlet one (O1). In the aforementioned figure 4.5 water elevation profiles, the junctions fourteen, twenty and twenty one are shown as adequate or not flooded junctions with hydraulic performance.



Summary Results

Topic: Outfall Loading

Click a column header to sort the column.

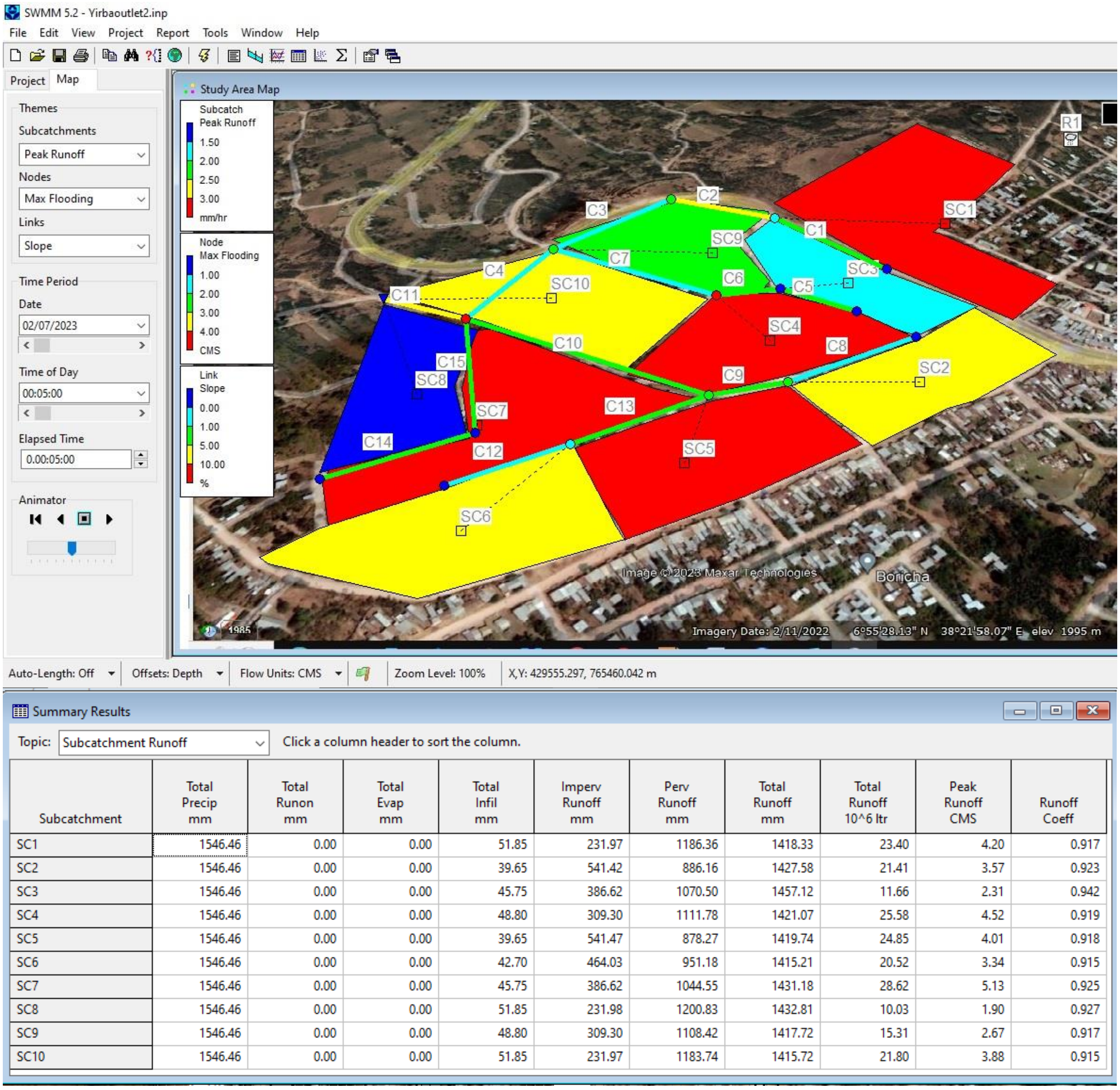
Outfall Node	Flow Frequency %	Avgerage Flow CMS	Maximum Flow CMS	Total Volume 10^6 ltr
O1	97.67	5.635	10.840	59.023

Figure 4-6: Summary result of outfall of outlet one

According to the figure 4.6 result of the simulation for the outfall one (1) node, the average flow rate is 5.635m³/s, the maximum flow rate is 10.840m³/s at the ends of outlet one, and the total runoff volume is 59.023*10³ m³.

4.2.2 Result of the Outlet Two study

4.2.2.1 Runoff from sub catchments



Themes
Subcatchments: Peak Runoff
Nodes: Max Flooding
Links: Slope
Time Period
Date: 02/07/2023
Time of Day: 00:05:00
Elapsed Time: 0.00:05:00

Auto-Length: Off | Offsets: Depth | Flow Units: CMS | Zoom Level: 100% | X,Y: 429555.297, 765460.042 m

Summary Results

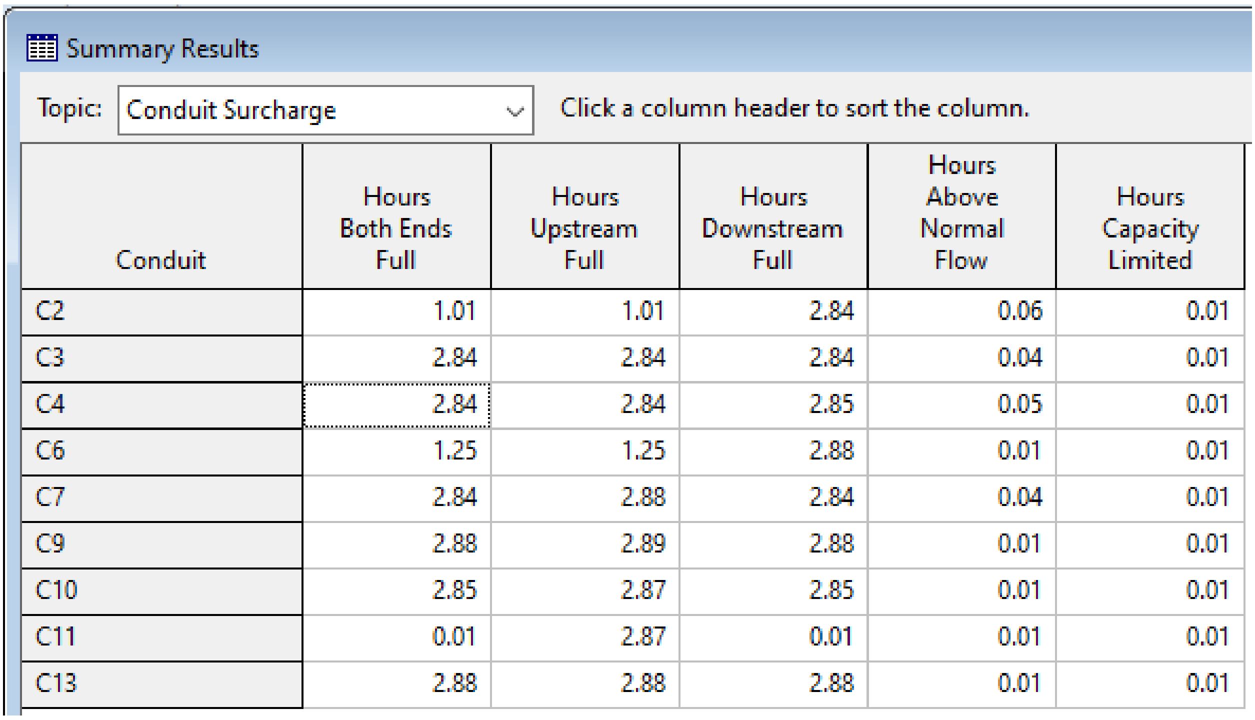
Topic: Subcatchment Runoff — Click a column header to sort the column.

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	1546.46	0.00	0.00	51.85	231.97	1186.36	1418.33	23.40	4.20	0.917
SC2	1546.46	0.00	0.00	39.65	541.42	886.16	1427.58	21.41	3.57	0.923
SC3	1546.46	0.00	0.00	45.75	386.62	1070.50	1457.12	11.66	2.31	0.942
SC4	1546.46	0.00	0.00	48.80	309.30	1111.78	1421.07	25.58	4.52	0.919
SC5	1546.46	0.00	0.00	39.65	541.47	878.27	1419.74	24.85	4.01	0.918
SC6	1546.46	0.00	0.00	42.70	464.03	951.18	1415.21	20.52	3.34	0.915
SC7	1546.46	0.00	0.00	45.75	386.62	1044.55	1431.18	28.62	5.13	0.925
SC8	1546.46	0.00	0.00	51.85	231.98	1200.83	1432.81	10.03	1.90	0.927
SC9	1546.46	0.00	0.00	48.80	309.30	1108.42	1417.72	15.31	2.67	0.917
SC10	1546.46	0.00	0.00	51.85	231.97	1183.74	1415.72	21.80	3.88	0.915

Figure 4-7: Outlet two sub catchment runoff results

Figure 4.7 shows the simulated results for 10 sub catchments totaling 20 hectares at the outlet two 2 study. The total runoff created by the SWMM5.2 model is 35.53 m³/sec, with the highest peak runoff generated at sub catchment (SC7) being 5.13m³/sec.

4.2.2.2 Channels and junctions flooded



Summary Results

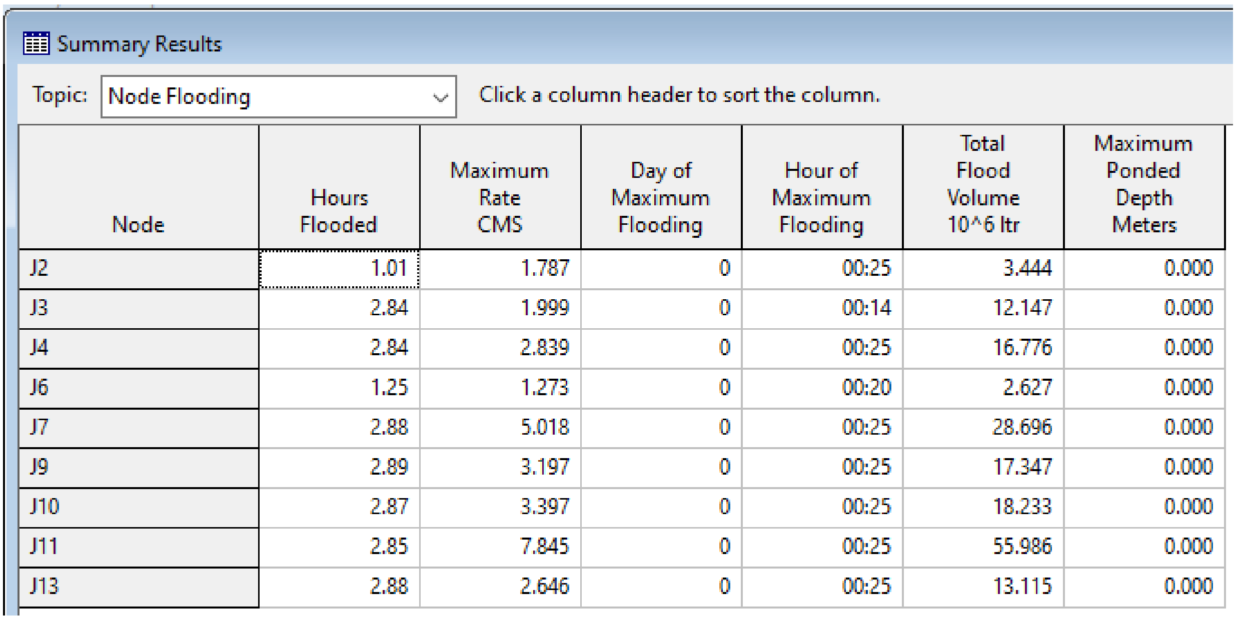
Topic: Conduit Surcharge

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
C2	1.01	1.01	2.84	0.06	0.01
C3	2.84	2.84	2.84	0.04	0.01
C4	2.84	2.84	2.85	0.05	0.01
C6	1.25	1.25	2.88	0.01	0.01
C7	2.84	2.88	2.84	0.04	0.01
C9	2.88	2.89	2.88	0.01	0.01
C10	2.85	2.87	2.85	0.01	0.01
C11	0.01	2.87	0.01	0.01	0.01
C13	2.88	2.88	2.88	0.01	0.01

Figure 4-8: The outfall two flooded channels

From the above table 4.8 results, links (channels) overflows are C2 to C4, C6, C7, C9 to C11, and C13 have limited capacity and are unable to carry flow within the channels during high rain seasons.



Summary Results

Topic: Node Flooding

Click a column header to sort the column.

Node	Hours Flooded	Maximum Rate CMS	Day of Maximum Flooding	Hour of Maximum Flooding	Total Flood Volume 10^6 ltr	Maximum Ponded Depth Meters
J2	1.01	1.787	0	00:25	3.444	0.000
J3	2.84	1.999	0	00:14	12.147	0.000
J4	2.84	2.839	0	00:25	16.776	0.000
J6	1.25	1.273	0	00:20	2.627	0.000
J7	2.88	5.018	0	00:25	28.696	0.000
J9	2.89	3.197	0	00:25	17.347	0.000
J10	2.87	3.397	0	00:25	18.233	0.000
J11	2.85	7.845	0	00:25	55.986	0.000
J13	2.88	2.646	0	00:25	13.115	0.000

Figure 4-9: The outfall two flooded junctions

The drainage system junctions’ overflow in the network simulation status report, according to the above table 4.9 results, shows that the sections between junctions J2, J3, J4, J6, J7, J9, J10, J11, and J13 are nodes flooded.

4.2.2.3 Water Elevation profile and outfall charged

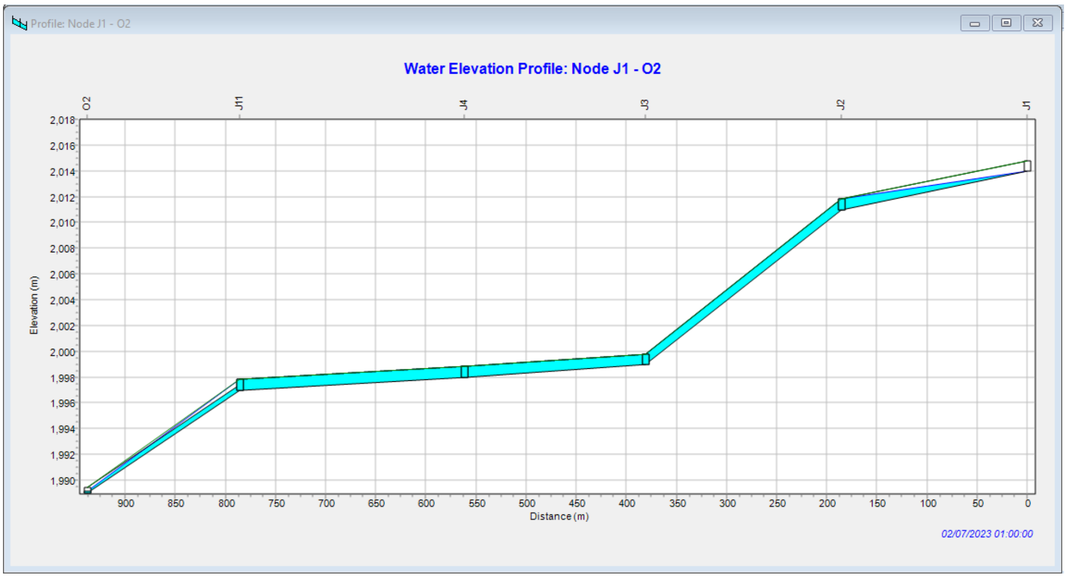


Figure 4-10: Water elevation profile from junction J1 to outfall two

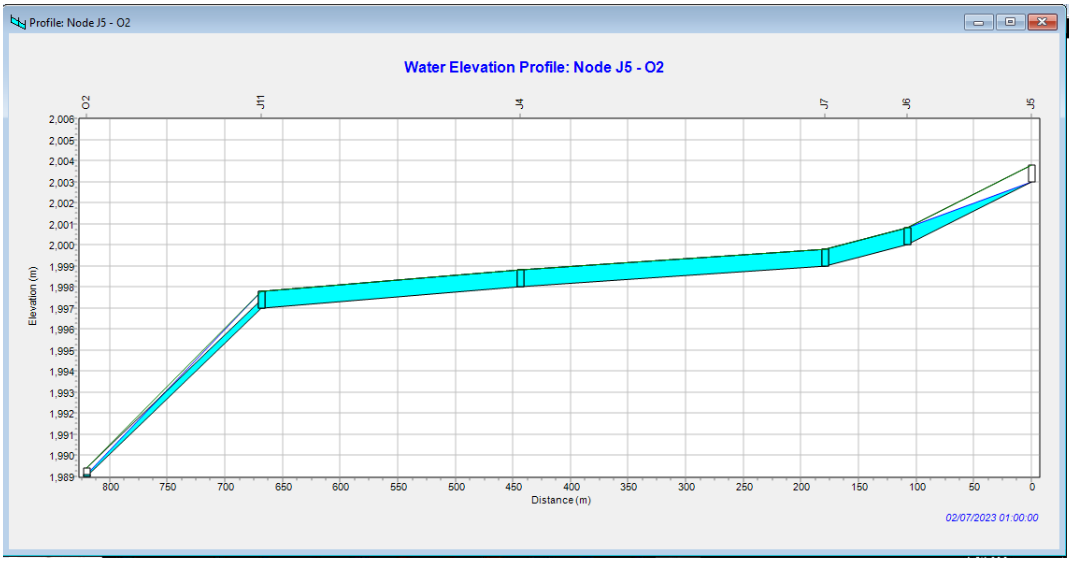
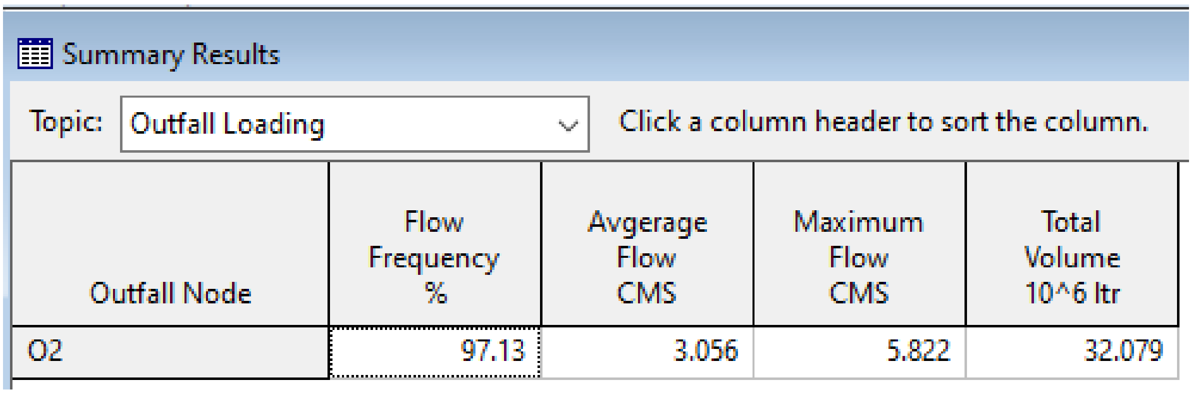


Figure 4-11: Water elevation profile from junction J5 to outfall two

Figures 4.10 and 4.11's water elevation profile plot provides an overview of each sub-catchment that was used for the simulation. Water elevation overflows at junction two (J2), J3, J4, J6, J7 and J11 from this network profile of J1 to outlet two (O2).

In the aforementioned figures 4.10 and 4.11 water elevation profiles, the junctions one and five are shown as adequate or not flooded junctions with hydraulic performance.



Outfall Node	Flow Frequency %	Avgerage Flow CMS	Maximum Flow CMS	Total Volume 10^6 ltr
O2	97.13	3.056	5.822	32.079

Figure 4-12: Outlet two water elevation profile and discharge

According to figure 4.12 results of the simulation for the outfall two (2) node, the average flow rate is 3.056m³/s, the maximum flow rate is 5.822m³/s at the ends of outlet two, and the total runoff volume is 32.079*10³ m³.

The hydraulic performance capacity of the entire drainage system is assessed using the swmm5.2 model. The systems simulate the runoff generated from each sub catchment and flow routed via the network of channels and junction infrastructure constructed under varied operating situations the outcomes of flooded drainage system are well evaluated.

Table 4-3: Outflow discharge and volume of runoff for both outfalls

Names of outfall nodes	Average flow rate (m^3/s)	Maximum flow rate (m^3/s)	Total volume to outlet ($*10^3$ m^3)
Outlet one	5.635	10.840	59.023
Outlet two	3.056	5.822	32.079
Outfalls result	4.346	10.840	91.102

The overall study simulated results in table 4.3 both outlets the total simulation area of sub catchment is 80 ha, the combined drainage system is 38 nodes and 38 channels with a total length channel flow routed through 5.865 km and 30 sub catchments in entire drainage system.

Both outfalls results in table 4.3 showed that 4.346m³/s average flow rate, 10.840m³/s maximum flow rate and 91.102*10³ m^3 total volume runoff collective drainage systems involved each regular or irregular shape simulation used.

4.3 The alternate method of managing drainage systems

The table below compares the values with and without the LID technique for infiltration trenches, rain gardens, rooftop disconnections, and vegetative swales in SWMM5.2 while modeling the drainage system network.

Table 4-4: SWMM5.2 results without and with LID

Item	swmm5.2 results without LID (Existing condition)	swmm5.2 results with low impact development							
		Infiltration trench		Rain garden		Vegetative swale		Rooftop disconnection	
		Value	%	value	%	value	%	value	%
Peak runoff (m^3/s)	5.822	5.34	8.24	5.63	3.26	5.51	5.36	5.75	1.24
Total volume runoff at outlet two (*10^3 m^3)	32.079	24.03	**25.08**	29.14	9.15	29.10	**9.27**	31.02	3.30

The runoff reduction in these two outfall projects is compared in the above table 4.4 to select the best alternative. For outlet two of the project on the same sub catchment, the total volume outfall two was reduced by 3.30% using rooftop disconnection, **25.08%** using an infiltration trench, 9.15 using a rain garden, and **9.27**% using a vegetative swale.

Therefore for this study select the best alternative of LID control to Evaluate the Yirba town drainage system problem mitigation measures is solved using the acceptable LID control types **Infiltration trench and Vegetative swale** are the most recommended from result observed in table 4.4.

4.3.1 Comparison of the current flow and LID Scenario

In areas where all LID measures are assumed to be applied, the outputs of model simulation of total volume runoff and peak runoff from sub catchments and peak discharge along with the peak discharge time from conduits were compared in order to assess the change before and after the application of LID measures. When the return period of 10 years was considered, the continuity error for surface runoff and flow routing were **1.96% and 0.11%**, respectively for outlet one simulation with SWMM5.2. Additionally, the continuity error for surface runoff and flow routing were **2.85% and 0.14%**, respectively for outlet two simulations with LID alternatives.

Table 4-5: Comparison of Peak runoff and total volume runoff

Outfall names	List of sub catch.	Existing Peak Runoff		LID Scenario		Change (%)		Description
		Total volume runoff (*10 m^3)	Peak runoff (m^3/sec)	Total volume Runoff (*10 m^3)	Peak runoff (m^3/sec)	Total volume runoff	Peak runoff	
Outlet one	SC6	31.53	5.38	24.69	4.31	21.68	19.84	Infiltration trench LID alternative used
	SC8	17.02	2.97	13.78	2.40	19.02	19.26	
	SC10	14.82	2.32	11.48	1.88	22.56	19.01	
	SC12	13.04	2.37	10.19	1.90	21.86	19.82	
	SC13	26.75	4.81	21.00	3.85	21.51	19.91	
	SC15	15.93	2.68	11.43	2.53	28.24	5.67	Vegetative swale LID used
	SC18	22.68	3.95	19.48	3.83	14.13	3.14	
	SC20	21.61	4.10	15.39	4.03	28.78	1.82	
Outlet two	SC7	28.13	5.13	21.95	4.14	21.97	19.28	Infiltration trench used
	SC9	15.31	2.67	11.86	2.19	22.53	18.02	
	SC8	10.03	1.90	8.21	1.71	18.12	9.92	Vegetative swale used

According to the analysis related to a 10-year return period, the use of infiltration trench LIDs unit reduces peak runoff magnitude from sub catchment by a minimum of 18.02% and a maximum of 19.91%. Additionally, vegetative swale LIDs in SWMM5.2 reduced the peak runoff from the sub catchment by a minimum of 1.82% and a maximum of 9.92%. Infiltration trench have more effect than vegetative swale for reduce peak discharge because of an infiltration trench has three layers (surface, storage and drain layer) but vegetative swale only has a surface layer.

The magnitude of total volume runoff is reduced over a 10 year return period in SWMM5.2 with LIDs alternate simulation by a minimum of 14.13% and a maximum of 28.78%. If it is managed extensively and properly, this scenario offers a significant opportunity to improve the town's drainage system performance.

Table 4-6.Comparison of Peak Discharge

Outfall names	Conduits	Existing Peak Runoff		LID Scenario		Change (%)	
		Peak time	Q (m^3/sec)	Peak time	Q (m^3/sec)	Peak time (%)	Peak discharge reduction (%)
Outlet one	C1	00:51	0.488	01:05	0.347	27.45	28.84
	C6	01:10	1.259	02:16	1.154	94.29	8.32
	C9	01:18	1.082	01:36	0.957	23.08	11.54
	C13	01:26	2.070	02:22	1.281	65.12	38.13
Outlet two	C7	00:35	1.688	00:58	1.347	65.71	20.20
	C10	00:30	0.544	00:54	0.404	80.00	25.72

The peak discharge magnitude is reduced by a minimum of 8.32% and a maximum of 38.13% throughout the span of a 10-year return period. Additionally, the peak discharge time is increased by a minimum of 23.08% and a maximum of 94.29%. If it is managed effectively and properly, this circumstance offers a significant opportunity to improve the Yirba town drainage system performance.

4.3.2 Modeled area of LID control

The LID control proposed area is applied to sub catchment using infiltration trench and vegetative swale method to the two outfall networks. The runoff flow results from each sub catchment in the system are taken into account while choosing the sub catchments.

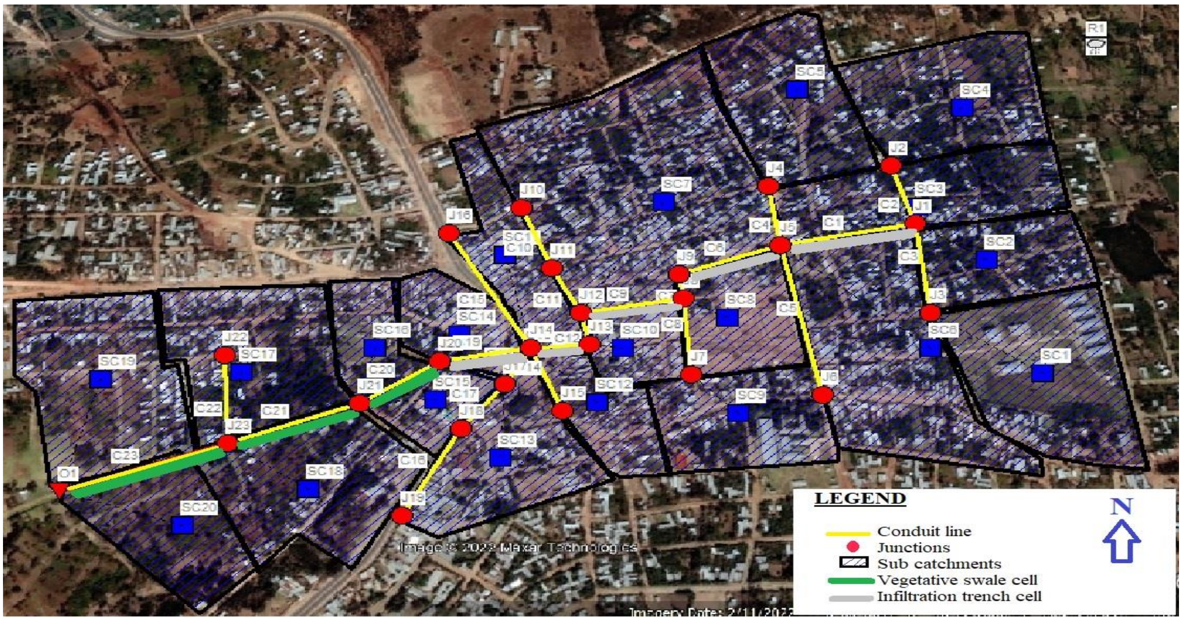


Figure 4-13: Outfall one study of LID control area modeled

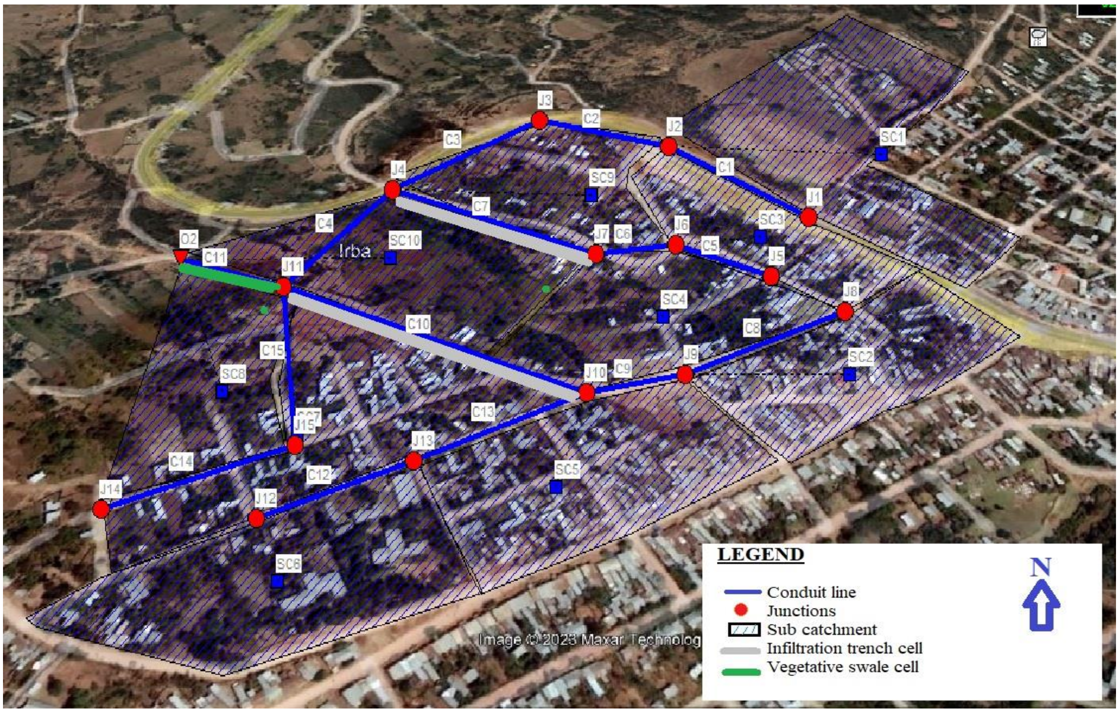


Figure 4-14: Outfall two study of LID control area modeled

CHAPTER 5 CONCLUSION AND RECOMMENDATION

5.1. Conclusion

This study used the swmm5.2 model to assess the performance of urban drainage system in Yirba town and has identified sormwater management issues and existing urban drainage system problems, assessed the hydraulic performance of stormwater drainage systems, and also proposed the possible solutions for the drainage problems.

The hydraulic and hydrological performance of urban drainage was simulated by EPA SWMM 5.2 with and without LID. The calibration and validation were applied for 10 days of observed flow depth and the model result was compared. For calibration five days of flow depth data were collected and Manning's equation was used to calculate the flow, which is 5.86m^3/s. When the sensitive parameters for the model were adjusted, the result at the same site reached 5.81m^3/s, which is almost similar. The model results parallel with observed are validated using the other five days of flow depth data that were used for validation without altering the sensitive parameters.

The performance of the SWMM5.2 model and goodness of fit for the certified area were evaluated using the simulated values of the Nash-Sutcliffe efficiency coefficient (NSE) =0.90 and the coefficient of determination (R^2) =0.97, which are in an acceptable range. Hence, the SWMM5.2 model is an effective tool for assessing the performance of urban drainage systems and also manages the overflow drainage infrastructure for this study.

According to a SWMM simulation for a 10 years return period, the town existing drainage system is unable to control the runoff being generated in the town. The drainage system as a whole has 50% and 55% of conduits and junctions that do not provide enough service, which could be a reason for problems of flooding. Moreover, solid and liquid wastes were directly disposed into the drainage system which could be a cause for flood induced problems.

Over a 10 years return period, LID application contributes to peak runoff magnitude reductions of at minimum 8.32% and a maximum of 38.13%, respectively. In addition, the peak runoff time is increased by a minimum of 23.08% and a maximum of 94.29%.

This condition allows for the ability to dramatically improve the town's drainage system performance.

Overall, the performance of these stormwater drainage systems was poor. In order to provide the best service, it is recognized that its capacity has generated lesser results requiring some adjustment or improvement. It also requires considerable ongoing maintenance, as well as drainage networking for places lacking drainage systems.

5.2 Recommendation

The following recommendations are provided for an improved and sustainable urban stormwater drainage system in order to solve the problems that have been hindering drainage systems in this study area.

- Yirba Town is expected to develop a fundamental urban drainage system together with other urban infrastructures as a Municipality. The town also needs to prepare a drainage master plan as soon as possible to support effective urban drainage management and the development of green infrastructure.
- Develop a culture of knowledge within the community on how to manage solid and liquid wastes, as well as how to use the drainage systems so that their lifespans last as long as possible.
- The current drainage network lines are silted up, thus I advise that the silt be removed from the channel, the failing infrastructures be rebuilt and the existing drainage system be properly regulated using LID technique awareness is provided for any relevant stakeholders in this area.
- To assess the viability of predictable and LID solutions for the achievement of sustainability goals, the upcoming study should focus on model parameters of LID practices and the predictable measures further with cost-benefit studies must be included in the design process.

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APPENDIX

Appendix table 1: Manning‘s n –Overland Flow

Surface	Manning's n
Smooth asphalt	0.011
Smooth concrete	0.012
Ordinary concrete lining	0.013
Good wood	0.014
Brick with cement mortar	0.014
Vitrified clay	0.015
Cast iron	0.015
Corrugated metal pipes	0.024
Cement rubble surface	0.024
Fallow soils (no residue)	0.05
Residue cover < 20%	0.06
Residue cover > 20%	0.17
Range (natural)	0.13
Short, prairie	0.15
Dense	0.24
Bermuda grass	0.41
Light underbrush	0.40
Dense underbrush	0.80

Source: (U.S. Department of Transportation, 2001)

Appendix table 2: Depression storage values

Lawns	0.10 - 0.20 inches
Impervious surfaces	0.05 - 0.10 inches
Pasture	0.20 inches
Forest litter	0.30 inches

Source: (U.S. Department of Transportation, 2001)

Appendix table 3: Manning’s n- closed conduit and open channel

Conduit Material	**Manning's n**
Asbestos-cement pipe	0.011 - 0.015
Brick	0.013 - 0.017
Cast iron pipe	
- Cement-lined & seal coated	0.011 - 0.015
Concrete (monolithic)	
- Smooth forms	0.012 - 0.014
- Rough forms	0.015 - 0.017
Concrete pipe	0.011 - 0.015
Corrugated-metal pipe	
- Plain	0.022 - 0.026
- Paved invert	0.018 - 0.022
- Spun asphalt lined	0.011 - 0.015
Plastic pipe (smooth)	0.011 - 0.015
Pipes	0.011 - 0.015
- Liner plates	0.013 - 0.017
Channel Type	**Manning's n**
Lined Channels	
- Asphalt	0.013 - 0.017
- Brick	0.012 - 0.018
- Concrete	0.011 - 0.020
- Rubble or riprap	0.020 - 0.035
- Vegetal	0.030 - 0.40
Excavated or dredged	
- Earth, straight and uniform	0.020 - 0.030
- Earth, winding, fairly uniform	0.025 - 0.040
- Rock	0.030 - 0.045
- Unmaintained	0.050 - 0.140
- Fairly regular section	0.030 - 0.070
- Irregular section with pools	0.040 - 0.100

Source: (U.S. Department of Transportation, 2001)

Appendix Table 4: Double mass curve after consistency checked

Year	Annually RF (mm) of YIRBA station	Accumulative annual rainfall YIRBA station (mm)	MOROCHO station (mm)	DERARA station (mm)	BALELA station (mm)	Average RF of the 3 Neighboring stations	Average Accumulative Rainfall of nearby station(mm)
2022	28.7	28.7	44.3	45.0	34.8	41.4	41.4
2021	63.6	92.3	50.0	31.9	40.0	40.6	82.0
2020	58.6	150.9	48.9	45.4	40.2	44.8	126.8
2019	48.5	199.4	49.3	49.4	60.0	52.9	179.7
2018	28.5	227.9	49.4	37.2	76.8	54.5	234.2
2017	33.2	261.0	50.1	39.6	40.0	43.2	277.4
2016	50.1	311.1	36.5	41.7	50.0	42.7	320.2
2015	42.2	353.3	34.4	37.7	58.0	43.4	363.5
2014	45.0	398.3	32.3	29.7	40.0	34.0	397.5
2013	52.2	450.5	33.7	37.7	40.0	37.1	434.7
2012	40.0	490.5	50.7	59.9	29.0	46.5	481.2
2011	66.2	556.7	51.9	51.2	30.0	44.4	525.6
2010	60.6	617.3	38.5	51.2	60.2	50.0	575.5
2009	87.8	705.1	43.0	39.3	66.0	49.4	625.0
2008	75.5	780.7	68.3	49.8	84.0	67.4	692.3
2007	60.6	841.3	51.1	52.1	37.8	47.0	739.3
2006	45.1	886.4	47.3	45.0	34.8	42.4	781.7
2005	71.0	957.3	96.1	48.0	43.8	62.6	844.3
2004	46.9	1004.2	51.7	33.4	31.3	38.8	883.1
2003	50.4	1054.7	39.3	52.7	42.9	45.0	928.1
2002	34.0	1088.7	33.7	38.0	41.5	37.7	965.8
2001	95.2	1183.8	40.7	47.4	36.3	41.5	1007.3
2000	85.7	1269.6	54.9	48.5	53.0	52.1	1059.4
1999	45.8	1315.4	50.6	31.5	42.5	41.5	1101.0
1998	52.2	1367.6	54.9	40.6	38.5	44.7	1145.6
1997	47.8	1415.3	99.9	38.4	38.1	58.8	1204.4
1996	78.5	1493.9	35.6	45.2	48.2	43.0	1247.4
1995	37.0	1530.9	48.6	48.8	37.5	45.0	1292.4
1994	54.6	1585.5	36.8	45.9	28.4	37.0	1329.4
1993	52.0	1637.5	58.2	50.6	38.5	49.1	1378.5
1992	54.5	1692.0	59.8	90.1	39.4	63.1	1441.6

From the above table 4.1 the consistency check is more closely related to the original data in the result and discussion section; however, the rainfall data for the Yirba station needs to be corrected for the years 1992-1995,1995-1997,1997-2001,2001-2003,2003-2006,2006-2009 2012-2014, 2014-2016, 20016-2018,2018-2020,and 2020-2022 by correction values of 1.356, 1.138, 1.473, 1.001, 1.281, 1.679, 0.801, 1.407, 0.761, 0.758, and 0.850 respectively. The slope of the rainfall data for the years 2009 to 2012 is equal to 1.246 as the correction value.

Appendix Table 5: Data quality by relative standard error

Year	24hr max annual rainfall (mm) YIRBA station (X)	Y=LogX	Y-Ym	(Y-Ym)^2	(Y-Ym)^3	(X-Xm)^2
1992	54.52	1.737	0.019	0.00038	0.00001	0.004
1993	52.00	1.716	-0.001	0.00000	0.00000	6.660
1994	54.60	1.737	0.020	0.00041	0.00001	0.000
1995	37.02	1.568	-0.149	0.02208	-0.00328	308.311
1996	78.52	1.895	0.178	0.03165	0.00563	572.963
1997	47.79	1.679	-0.038	0.00142	-0.00005	46.071
1998	52.15	1.717	0.000	0.00000	0.00000	5.902
1999	45.82	1.661	-0.056	0.00314	-0.00018	76.811
2000	85.74	1.933	0.216	0.04671	0.01009	970.918
2001	**95.17**	1.978	0.261	0.06835	0.01787	1647.389
2002	34.02	1.532	-0.185	0.03434	-0.00636	422.645
2003	50.43	1.703	-0.014	0.00021	0.00000	17.202
2004	46.88	1.671	-0.046	0.00213	-0.00010	59.347
2005	70.96	1.851	0.134	0.01794	0.00240	268.146
2006	45.08	1.654	-0.063	0.00397	-0.00025	90.190
2007	60.61	1.783	0.065	0.00429	0.00028	36.322
2008	75.55	1.878	0.161	0.02598	0.00419	439.690
2009	87.81	1.944	0.226	0.05128	0.01161	1103.873
2010	60.60	1.782	0.065	0.00428	0.00028	36.232
2011	66.20	1.821	0.104	0.01077	0.00112	135.008
2012	40.00	1.602	-0.115	0.01323	-0.00152	212.597
2013	52.20	1.718	0.001	0.00000	0.00000	5.668
2014	45.00	1.653	-0.064	0.00408	-0.00026	91.790
2015	42.20	1.625	-0.092	0.00842	-0.00077	153.282
2016	50.10	1.700	-0.017	0.00030	-0.00001	20.077
2017	33.18	1.521	-0.196	0.03849	-0.00755	457.990
2018	**28.46**	1.454	-0.263	0.06906	-0.01815	682.200
2019	48.52	1.686	-0.031	0.00097	-0.00003	36.745
2020	58.60	1.768	0.051	0.00259	0.00013	16.169
2021	63.56	1.803	0.086	0.00742	0.00064	80.614
2022	28.72	1.458	-0.259	0.06701	-0.01735	668.744
SUM	**1692.00**	**53.230**		**0.54000**	**-0.00160**	**8669.561**
Average(Xm)	**54.58**					
Average(Ym)	**1.72**					
St,dev Xi(Sx)	**17.00**					
St,dev Yi(Sy)	**0.13**					
Skweness coff(cs)	**-0.02**					

Whereas relative standard error =se/xm

se = standard deviation/ (number of year)$^{0.5}$=16.9996/(31$^{0.5}$)= 3.0532

Relative standard error = (3.0532/54.58)*100 = 5.59% ≤10% ok! Therefore the data is reliable and adequate.

Appendix Table 6: - Frequency analysis and development of the IDF curve

Year	24hr max annual rainfall (mm) YIRBA station (X)	5min = 0.08hr	10min = 0.17hr	15min = 0.25hr	30min = 0.5hr	1hr	2hr	3hr	6hr	12hr	24hr
1992	54.52	7.92	13.28	17.18	24.52	31.68	37.92	41.10	45.93	50.31	54.52
1993	52.00	7.56	12.66	16.39	23.39	30.22	36.17	39.20	43.81	47.98	52.00
1994	54.60	7.94	13.30	17.20	24.56	31.73	37.98	41.16	46.00	50.38	54.60
1995	37.02	5.38	9.02	11.67	16.65	21.52	25.75	27.91	31.19	34.16	37.02
1996	78.52	11.41	19.12	24.74	35.32	45.63	54.61	59.19	66.15	72.45	78.52
1997	47.79	6.95	11.64	15.06	21.50	27.77	33.24	36.03	40.27	44.10	47.79
1998	52.15	7.58	12.70	16.43	23.46	30.31	36.27	39.31	43.94	48.12	52.15
1999	45.82	6.66	11.16	14.44	20.61	26.63	31.87	34.54	38.60	42.28	45.82
2000	85.74	12.46	20.88	27.02	38.57	49.83	59.63	64.64	72.24	79.12	85.74
2001	95.17	13.83	23.18	29.99	42.81	55.31	66.19	71.74	80.18	87.82	95.17
2002	34.02	4.95	8.29	10.72	15.30	19.77	23.66	25.65	28.66	31.40	34.02
2003	50.43	7.33	12.28	15.89	22.69	29.31	35.08	38.02	42.49	46.54	50.43
2004	46.88	6.81	11.42	14.77	21.09	27.24	32.60	35.34	39.49	43.26	46.88
2005	70.96	10.31	17.28	22.36	31.92	41.24	49.35	53.49	59.78	65.48	70.96
2006	45.08	6.55	10.98	14.21	20.28	26.20	31.36	33.99	37.98	41.60	45.08
2007	60.61	8.81	14.76	19.10	27.26	35.22	42.15	45.69	51.06	55.93	60.61
2008	75.55	10.98	18.40	23.81	33.98	43.91	52.55	56.95	63.65	69.72	75.55
2009	87.81	12.76	21.38	27.67	39.50	51.03	61.07	66.19	73.98	81.03	87.81
2010	60.60	8.81	14.76	19.10	27.26	35.22	42.15	45.68	51.06	55.92	60.60
2011	66.20	9.62	16.12	20.86	29.78	38.47	46.04	49.91	55.77	61.09	66.20
2012	40.00	5.81	9.74	12.60	17.99	23.25	27.82	30.15	33.70	36.91	40.00
2013	52.20	7.59	12.71	16.45	23.48	30.34	36.31	39.35	43.98	48.17	52.20
2014	45.00	6.54	10.96	14.18	20.24	26.15	31.30	33.92	37.91	41.53	45.00
2015	42.20	6.13	10.28	13.30	18.98	24.52	29.35	31.81	35.55	38.94	42.20
2016	50.10	7.28	12.20	15.79	22.54	29.12	34.85	37.77	42.21	46.23	50.10
2017	33.18	4.82	8.08	10.46	14.92	19.28	23.08	25.01	27.95	30.62	33.18
2018	28.46	4.14	6.93	8.97	12.80	16.54	19.80	21.46	23.98	26.26	28.46
2019	48.52	7.05	11.82	15.29	21.82	28.20	33.75	36.58	40.88	44.77	48.52
2020	58.60	8.52	14.27	18.47	26.36	34.06	40.76	44.18	49.37	54.08	58.60
2021	63.56	9.24	15.48	20.03	28.59	36.94	44.21	47.92	53.55	58.65	63.56
2022	28.72	4.17	6.99	9.05	12.92	16.69	19.98	21.65	24.20	26.50	28.72
SUM		**245.94**	**412.08**	**533.15**	**761.07**	**983.30**	**1176.8**	**1275.5**	**1425.5**	**1561.4**	**1692.0**
Mean (xm)		**7.93**	**13.29**	**17.20**	**24.55**	**31.72**	**37.96**	**41.15**	**45.99**	**50.37**	**54.58**
St.dev (Sx)		**2.47**	**4.14**	**5.36**	**7.65**	**9.88**	**11.82**	**12.82**	**14.32**	**15.69**	**17.00**
Skweness coff(Cs)		**-0.02**	**-0.02**	**-0.02**	**-0.02**	**-0.02**	**-0.02**	**-0.02**	**-0.02**	**-0.02**	**-0.02**

Table 6 in the Appendix shows the average annual rainfall (mm) and 1 volume for the short durations with appendix table 7, reduce the daily precipitation data to shorter durations

Appendix Table 7: Reduction daily precipitation for each time t

given t	Rt.	t in hr.
5min	0.14536	0.0833333
10min	0.24354	0.1666667
15min	0.31510	0.25
30min	0.44980	0.5
1hr	0.58115	1
2hr	0.69552	2
3hr	0.75386	3
6hr	0.84252	6
12hr	0.92279	12
24hr	1	24

Whereas $R_t = \frac{t(b+24)^n}{24(b+t)^n} * R_{24}$ = Rainfall Reduction formula

n = 0.9 and b = 0.3 is constant (M.L. Waikar and Undegaonkar Namita U, 2015)

Probability distributions and IDF curve development

There are two possibilities for this frequency analysis and they are as follows:

1) A theoretical **Extreme Value (EV)** distribution (such as, Gumbel Type I) can be fitted to the observations and the corresponding rainfall events with given exceedance probability can then be estimated using the theoretical distribution.
2) Estimate the exceedance probability based on the observations using **an empirical plotting position** approach.

Appendix Table 8: Probability distribution of extreme value XT in (mm)

Probability distribution	2 year	5 year	10 year	25 year	50 year	100 year
Gumbel	51.79	66.82	76.78	89.34	98.67	107.93
Normal	54.58	68.88	76.37	84.35	89.50	94.13
Lognormal	52.13	67.62	77.47	89.57	98.37	107.02
Log-Pearson	52.19	67.63	77.41	89.35	97.97	106.40
Pearson typeIII	54.65	68.89	76.32	84.21	89.28	93.82
Weibull’s Formulla	52.00	68.89	83.93	91.95	95.17	95.17

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

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

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

Where, O_i is the observed rainfall and E_i is the expected rainfall.

Appendix Table 9: Tested Goodness of Fit by Probability Distribution

Return period(T)	Expected rainfall(mm)= E_i					Observed rainfall (mm)= O_i
	Gumbel	**Normal**	**Lognormal**	**Log-Pearson**	**Pearson typeIII**	**Weibull's Formula:**
2	51.79	54.58	52.13	52.19	54.65	52.00
5	66.82	68.88	67.62	67.63	68.89	68.89
10	76.78	76.37	77.47	77.41	76.32	83.93
25	89.34	84.35	89.57	89.35	84.21	91.95
50	98.67	89.50	98.37	97.97	89.28	95.17
100	107.93	94.13	107.02	106.40	93.82	95.17

Appendix table 10: Chi - Square Test Result in dimensionless of X^2 values.

Return period(T)	**Gumbel(EVI)**	**Normal**	**Lognormal**	**Log-Pearson**	**Pearson typeIII**
2	0.001	0.1220	0.0003	0.0007	0.1284
5	0.064	0.0000	0.0240	0.0236	0.0000
10	0.666	0.7485	0.5378	0.5492	0.7581
25	0.076	0.6851	0.0630	0.0756	0.7112
50	0.124	0.3591	0.1044	0.0803	0.3890
100	1.509	0.0114	1.3130	1.1862	0.0195
Total chi-square(X^2)	2.440	1.926	2.0426	**1.9156**	2.0063

Chi - Square Test Result in different return period of probability distributions. 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 **Log Pearson type III distribution.**

Appendix table 11: Rainfall depth (mm) by Log 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	0.0833	7.59	9.83	11.25	12.99	14.24	15.47
10	0.1667	12.71	16.47	18.85	21.76	23.86	25.91
15	0.25	16.45	21.31	24.39	28.15	30.87	33.53
30	0.5	23.48	30.42	34.82	40.19	44.07	47.86
60	1	30.33	39.30	44.99	51.93	56.94	61.84
120	2	36.30	47.04	53.84	62.15	68.14	74.01
180	3	39.35	50.98	58.36	67.36	73.86	80.21
360	6	43.97	56.98	65.22	75.28	82.54	89.65
720	12	48.16	62.41	71.43	82.45	90.41	98.19
1440	24	52.19	67.63	77.41	89.35	97.97	106.40

Appendix table 12: YIRBA IDF Curve (mm/hr.) developed

Duration (min)	Duration (hr.)	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	**0.0833**	91.04	117.96	135.02	155.85	170.89	185.60
10	**0.1667**	76.27	98.82	113.12	130.56	143.17	155.49
15	**0.25**	65.78	85.24	97.57	112.62	123.49	134.11
30	**0.5**	46.95	60.84	69.64	80.38	88.14	95.72
60	**1**	30.33	39.30	44.99	51.93	56.94	61.84
120	**2**	18.15	23.52	26.92	31.07	34.07	37.00
180	**3**	13.12	16.99	19.45	22.45	24.62	26.74
360	**6**	7.33	9.50	10.87	12.55	13.76	14.94
720	**12**	4.01	5.20	5.95	6.87	7.53	8.18
1440	**24**	2.17	2.82	3.23	3.72	4.08	4.43

The IDF developed in appendix 12 for given return period is used in swmm5.2 as input data.

Appendix Table 13: Measured input parameters of model in outfall one (1)

Name of junction code	Invert elevation of nodes	Name of Conduit code	Inlet node of conduit	Outlet node of conduit	Elevation b/n nodes	Channel dimension parameters						channel slope= Elvn/length	roughness depend on material (n)
						Types of channel	Bottom width (m)	Top width (m)	Max depth (m)	Side slope (S)	conduit length (m)		
J1	2005.0	C1	J1	J5	0.4	Trap.	0.6	1.0	0.8	0.25	194	0.0021	0.022
J2	2005.8	C2	J2	J1	0.8	Irreg.			0.4		124	0.0065	0.450
J3	2009.0	C3	J3	J1	4.0	Rect-o	0.6	0.6	0.8		173	0.0231	0.020
J4	2004.8	C4	J4	J5	0.2	Irreg.			0.4		104	0.0019	0.450
J5	2004.6	C5	J6	J5	13.0	Rect-o	0.7	0.7	1.0		286	0.0468	0.020
J6	2018.0	C6	J5	J9	0.4	Trapz.	0.8	1.2	1.0	0.20	166	0.0024	0.022
J7	2015.0	C7	J9	J8	0.2	Trapz.	0.8	1.2	1.0	0.20	43	0.0468	0.022
J8	2004.0	C8	J7	J8	11.0	Rect-o	0.7	0.7	1.0		138	0.0024	0.020
J9	2004.2	C9	J8	J12	0.4	Trapz	0.8	1.2	1.0	0.20	148	0.0027	0.022
J10	20015.0	C10	J10	J11	5.0	Rect-o	0.6	0.6	0.8		119	0.0420	0.020
J11	2010.0	C11	J11	J12	6.4	Rect-o	0.7	0.7	1.0		98	0.0653	0.020
J12	2003.6	C12	J12	J13	0.2	Trapz.	0.8	1.4	1.2	0.25	68	0.0029	0.022
J13	2003.4	C13	J13	J14	0.2	Trapz.	0.8	1.4	1.2	0.25	80	0.0250	0.022
J14	2003.2	C14	J15	J14	11.0	Rect-o	0.6	0.6	0.8		132	0.0894	0.020
J15	2015.0	C15	J16	J14	11.0	Rect-c	1.0	1.0	0.8		260	0.0454	0.018
J16	2015.0	C16	J19	J18	8.0	Rect-c	1.0	1.0	0.8		166	0.0482	0.018
J17	2003.0	C17	J18	J17	7.0	Rect-c	1.0	1.0	0.8		112	0.0625	0.018
J18	2010.0	C18	J14	J17	0.2	Rect-c	1.0	1.0	1.2		69	0.0029	0.018
J19	2018.0	C19	J17	J20	1.0	Rect-o	2.0	2.0	1.5		82	0.0122	0.020
J20	2002.0	C20	J20	J21	5.0	Rect-o	2.0	2.0	1.5		123	0.0410	0.020
J21	1997.0	C21	J21	J23	14.0	Rect-o	2.0	2.0	1.5		185	0.0760	0.020
J22	1990.0	C22	J22	J23	7.0	Irreg.			0.4		160	0.0440	0.450
J23	1983.0	C23	J23	O1	5.0	Rect-o	1.5	1.5	1.2		214	0.0234	0.020
O1	1978.0												

Appendix Table 14: Measured input parameters of model in outfall two (2)

Name of junction code	Invert elevation of nodes	Name of Conduit code	Inlet node of conduit	Outlet node of conduit	Elevation b/n nodes	Channel dimension parameters					channel slope=Elvn/length	roughness depend on material (n)
						Types of channel	Bottom width (m)	Top width (m)	Max depth (m)	conduit length (m)		
J1	2014	C1	J1	J2	3	Rect-c	0.68	0.68	0.8	185	0.0162	0.018
J2	2011	C2	J2	J3	12	Rect-c	0.68	0.68	0.8	196	0.0162	0.018
J3	1999	C3	J3	J4	22	Rect-c	0.68	0.68	0.8	181	0.1215	0.018
J4	1998	C4	J4	J11	3	Rect-c	1.00	1.00	0.8	224	0.0133	0.018
J5	2003	C5	J5	J6	5	Rect-o	0.60	0.60	0.8	108	0.0463	0.02
J6	2000	C6	J6	J7	5	Rect-o	0.60	0.60	0.8	71	0.0704	0.02
J7	1999	C7	J7	J4	16	Rect-o	0.60	0.60	0.8	265	0.0604	0.02
J8	2011	C8	J8	J9	1	Rect-o	0.42	0.42	0.6	174	0.0058	0.02
J9	2010	C9	J9	J10	1	Rect-o	0.42	0.42	0.6	77	0.0138	0.02
J10	2009	C10	J10	J11	6	Rect-o	0.60	0.60	0.8	323	0.0186	0.02
J11	1997	C11	J11	O2	29	Irreg.			0.4	152	0.1910	0.45
J12	2017	C12	J12	J13	1	Rect-o	0.42	0.42	0.6	121	0.0083	0.02
J13	2016	C13	J13	J10	7	Rect-o	0.42	0.42	0.6	188	0.0372	0.02
J14	2016	C14	J14	J15	2	Rect-o	0.42	0.42	0.6	147	0.0136	0.02
J15	2014	C15	J15	O2	40	Irreg.			0.4	209	0.1910	0.45
O2	1989											

- Open rectangular channel(Rect-o)
- Closed rectangular conduit(Rect-c)
- Irregular natural channel(Irreg.)
- Open trapezoidal channel (Trapz.)

Appendix table 15: Input data in swmm5.2 at outlet 1 and 2 sub catchments parameters

Subcatchment (SC)	Outlet of SC	Area of SC (ha)	width	% of slope	% of Imper	N-imp	N-perv	Dstore-imprev	Dstore-perv	% zero imp rev
SC1	J3	5.43	338	3.5	15	0.013	0.5	2.5	7.6	85
SC2	J1	2.75	234	0.5	20	0.013	0.5	2.5	7.6	80
SC3	J5	3.58	427	0.5	20	0.013	0.5	2.5	7.6	80
SC4	J2	4.41	292	3	20	0.013	0.5	2.5	7.6	80
SC5	J4	4.67	340	4	20	0.013	0.5	2.5	7.6	80
SC6	J5	6.35	474	3	35	0.013	0.5	2.5	7.6	65
SC7	J12	8.36	406	5	35	0.013	0.5	2.5	7.6	65
SC8	J8	3.38	300	2	20	0.013	0.5	2.5	7.6	80
SC9	J7	3.02	225	2	30	0.013	0.5	2.5	7.6	70
SC10	J13	1.76	146	4	35	0.013	0.5	2.5	7.6	65
SC11	J14	3.3	460	2.6	30	0.013	0.5	2.5	7.6	70
SC12	J14	2.14	230	3.3	35	0.013	0.5	2.5	7.6	65
SC13	J17	4.1	340	3	35	0.013	0.5	2.5	7.6	65
SC14	J20	2.6	182	3.2	30	0.013	0.5	2.5	7.6	70
SC15	J21	1.46	119	4.6	30	0.013	0.5	2.5	7.6	70
SC16	J21	2.33	257	4.7	20	0.013	0.5	2.5	7.6	80
SC17	J23	4.69	300	5	25	0.013	0.5	2.5	7.6	75
SC18	J23	4.44	248	3	20	0.013	0.5	2.5	7.6	80
SC19	**O1**	6.2	300	5	25	0.013	0.5	2.5	7.6	75
SC20	**O1**	5.03	300	5	20	0.013	0.5	2.5	7.6	80

SC1	J2	3.25	152	3.75	15	0.013	0.5	2.5	7.6	85
SC2	J9	1.29	150	0.5	35	0.013	0.5	2.5	7.6	65
SC3	J6	1.04	120	2.5	25	0.013	0.5	2.5	7.6	75
SC4	J7	2	142	3.3	20	0.013	0.5	2.5	7.6	80
SC5	J10	1.2	152	0.5	35	0.013	0.5	2.5	7.6	65
SC6	J13	1.9	148	0.5	30	0.013	0.5	2.5	7.6	70
SC7	J11	2.02	152	2.6	25	0.013	0.5	2.5	7.6	75
SC8	**O2**	2.3	132	1.5	15	0.013	0.5	2.5	7.6	85
SC9	J4	2.14	122	3.3	20	0.013	0.5	2.5	7.6	80
SC10	**O2**	2.86	151	2	15	0.013	0.5	2.5	7.6	85

Appendix table 16: K_T value for Pearson Type III distribution

Skew coefficient (C_S)	Return period in years						
	2	5	10	25	50	100	200
	Exceedence probability						
	0.5	0.2	0.1	0.04	0.02	0.01	0.005
3.0	-0.396	0.420	1.180	2.278	3.152	4.051	4.970
2.5	-0.360	0.518	1.250	2.262	3.048	3.845	4.652
2.0	-0.307	0.609	1.302	2.219	2.912	3.605	4.298
1.8	-0.282	0.643	1.318	2.193	2.848	3.499	4.147
1.6	-0.254	0.675	1.329	2.163	2.780	3.388	3.990
1.4	-0.225	0.705	1.337	2.128	2.706	3.271	3.828
1.2	-0.195	0.732	1.340	2.087	2.626	3.149	3.661
1.0	-0.164	0.758	1.340	2.043	2.542	3.022	3.489
0.9	-0.148	0.769	1.339	2.018	2.498	2.957	3.401
0.8	-0.132	0.780	1.336	1.993	2.453	2.891	3.312
0.7	-0.116	0.790	1.333	1.967	2.407	2.824	3.223
0.4	-0.066	0.816	1.317	1.880	2.261	2.615	2.949
0.3	-0.050	0.823	1.309	1.849	2.211	2.544	2.586
0.2	-0.033	0.830	1.301	1.818	2.159	2.472	2.763
0.1	-0.017	0.836	1.292	1.785	2.107	2.400	2.670
0.0	0.000	0.841	1.282	1.751	2.054	2.326	2.576
-0.1	0.017	0.846	1.270	1.716	2.000	2.252	2.482
-0.2	0.033	0.850	1.258	1.680	1.945	2.178	2.388
-0.3	0.050	0.853	1.245	1.643	1.890	2.104	2.294
-0.4	0.066	0.855	1.231	1.606	1.834	2.029	2.201
-0.7	0.115	0.857	1.183	1.488	1.663	1.806	1.926
-0.8	0.132	0.856	1.166	1.448	1.606	1.733	1.837
-0.9	0.148	0.854	1.147	1.407	1.549	1.660	1.749
-1.0	0.164	0.852	1.128	1.366	1.492	1.588	1.664
-1.2	0.195	0.843	1.086	1.282	1.379	1.449	1.501
-1.6	0.254	0.817	0.994	1.116	1.166	1.197	1.216
-2.0	0.307	0.777	0.895	0.959	0.980	0.990	0.995
-2.5	0.360	0.710	0.771	0.793	0.798	0.799	0.800
-3.0	0.396	0.636	0.660	0.660	0.666	0.667	0.667