PERFORMANCE ASSESSMENT OF URBAN STORM WATER DRAINAGE SYSTEM OF BUTAJIRA TOWN, SNNPR, ETHIOPIA (USING SWMM MODEL)

M.SC. THESIS

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ADVISOR'S THESIS SUBMISSION APPROVAL SHEET

This is to certify that the thesis entitled "PERFORMANCE ASSESSMENT OF URBAN STORM WATER DRAINAGE SYSTEM OF BUTAJIRA TOWNSNNPR, ETHIOPIA (USING SWMM5.1 MODEL)" submitted in partial fulfillment of the requirement for the degree of Master's with specialization in WATER RESOURCE ENGINEERING AND MANAGEMENT, The graduate program of the school of Bio system and water resource Engineering, Institute of Technology, Hawassa university and has been carried out by Ms. EDEN TADELE, Id No GPREMW/0005/12 under my supervision. Therefore I recommend that the student has fulfilled the requirements and hence hereby can submit the thesis to the department for defense.

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We, the undersigned, members of the Board of Examiners of the final open defense by EDEN TADELE have read and evaluated her thesis entitled " PERFORMANCE ASSESSMENT OF URBAN STORM WATER DRAINAGE SYSTEM OF BUTAJIRA TOWN, SNNPR, ETHIOPIA (USING SWMMMODL)", and examined the candidate. This is, therefore, to certify that the thesis has been accepted in partial fulfilment of the requirement for the degree .

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DECLARETION

I, EDEN TADELE, declare that the content of this thesis is entirely my own work with the exception of such quotations or references which have been attributed to their authors or sources.

I also declare that all photographs are made or drawn by me except where I have acknowledged another as the author. This thesis has not been previously submitted to this or any other university for a degree requirement.

EDENE TADELE

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ABSTRACT

Storm water drainage problem is a major challenge facing most of the Cities and Towns in Ethiopia including Butajira Town. Drainage problem in Butajira Town cause the worst problem of over flooding and over toping drainage system during high rainfall season in a town. The objective of this study is to assess the performance of drainage system of Butajira town. The study employed both primary and secondary data collection. To achieve the specific objective SWMM5 model and LID control used as method. The SWMM was used to simulate the storm drainage flow. The calibration and validation of the SWMM5.1 model was done and its performance was tested by the goodness of fit using the coefficient of determination R^2 = 0.98, the Nash –Sutcliffe coefficient NSE =0.87 and Relative error RE =19%. The total simulated sub catchment in this study is 173.83ha joint to drainage system infrastructure of 12 junction, 12 channels, are simulated by SWMM5.1, From model result greater than 25% of drainage infrastructure is flooded, at the outfall total sub-catchment runoff is 1.904m^3/s average flow, 2.011 m^3/s maximum flow and 33.95*10^3 m^3 total volume of all outfall. Finally, from simulation results, most of junction's canal dimensions are insufficient to drain the generated runoff from sub-catchments. To alleviate these problems, applying LID and construction of additional drainage structures with proper dimension, design and construction of well-connected structures, adopting the culture of clearing waste materials and periodic maintenance of drainage structures before failure were the remedial measures to be taken to solve the problem of highly flooded study area specially sub catchment S8 and S9.

Key words: urban drainage, SWMM5, model calibration and verification, LID control, and alternative mitigation

LIST OF ACRONYMS AND ABBREVIATION

a.s.l. ……………….at sea level

CSA……………….Central statistical Agency

DEM ……………...Digital Elevation Model

EMA……………....Ethiopian Mapping Agency

ERA……………….Ethiopian Road Authority

EPA……………….Environmental Protection Agency

GIS ……………….Geographic Information System

GPS……………….Global Positioning System

IDF………………..Intensity duration frequency

LID……………….Low Impact Development

LULC……………..Land Use / Land Cover

NMA…………….. National Meteorological Agency

NSE………………Nash Sutcliffe simulation Efficiency

RE………………..Relative Error

R^2……………………Coefficient of Determination

SNNPRS………….Southern Nation Nationality and People Regional States

SWMM…………. Storm Water Management Model

SWAT……………Soil & Water Assessment Tool

USWD……………Urban Storm water drainage

UTM……………. Universal Transverse Mercator

1 INTRODUCTION

1.1 General background

Urban storm water drainage facilities are part of the urban infrastructure elements and design of these facilities require due attention. In Ethiopian context, where watersheds of many urban centers receive significant amount of annual rainfall and where rainfall intensity is generally high, control of runoff at source, flood protection, and safe disposal of excess water/ runoff though proper drainage facilities becomes essential (AASHTO, 1991).

Urban floods are caused due to increase in population density, development of urban infrastructure without paying due consideration to drainage aspects and increase in paved surfaces. Unplanned growth of urban area is affecting the natural drainage surface (Ahmed, 2013). Urbanization replaces permeable surface with impermeable ones, streets, parking lots, building etc. Due to high intensity rainfall events which are unpredictable in nature, occurring in urban catchments is becoming a major challenges these days due to lack of proper drainage systems and climate change factors (Ahmed, 2013).

Depicts the major causes of flooding as the blockage of urban storm water drainage lines along with inadequate/poor integration between the road and urban storm water drainage infrastructure. Also, with the rapid expansion of urbanization, impermeability increases with the increase in impervious surfaces, drainage pattern changes, the overland flow gets faster flooding, and environmental problems such as land degradation increases (Belete, 2011).

As there are lot of advancements in technology, there are number of models readily available to design and monitor urban runoff, including SWMM (Rossman, 2010). Keeping the necessity to model urban floods several modeling software's are developed namely, Hec-HMs, Hec- Ras, STORM, RAFTS and others (David, 1999). SWWM model can be used for design of drainage system and to simulate dynamic of single events or for modeling on a continuous basis (Delleur, 2003). Generally, runoff is diverted using pipes, channels, pumps and outlets. It is a full dynamic wave's simulation model used for single event or long-term events (Waikar, 2015).

Flood modeling is mainly designed to explore all the characteristics and the nature of flood in the urban area with respect to the impact of heavy rainfall on the runoff of the urban sub-catchments and the various socio-economic aspects of flood (Lee, 2018). Several researchers have made their efforts to improve the existing EPA tool kit but no such efforts have been seen so far in the case of SWMM (Banik, 2014). The model tries to study and understand the design and monitoring of drainages systems (Barco, 2008). Unlike other types of flooding, urban flooding is a direct, quick and localized consequence of rainfall (Awakimjan, 2015).

Storm water drain networks in cities are usually designed to effectively collect and convey excess surface runoff in order to avert urban flooding (Gouri, 2015). But, often most of them face reduction of functionality and capacity for transferring the runoff flow and their level of service reduces due to degradation in time, improper maintenance, inappropriate design, sedimentation and siltation, increase in materials roughness and structural deterioration. In addition, urban development and climate change exacerbate the situation. Such phenomena are followed by increase in runoff volume and peak flow rates (Barreto, 2012).

Hydrological and hydrodynamic modeling plays a key role for the hydraulic, structural and environmental assessment. Sustainable approaches, oriented to the control of runoff volumes from the beginning of the rainfall are preferable than methodologies based on conveyance. These sustainable approaches are also oriented to keep environment, social and economic values in balance (Barreto, 2012). Such infrastructure play a key role in preventing urban floods and their performance should be assessed after being quantified (Negin, 2016).

The main goal of this study was investigated the performance of the existing urban drainage network of Butajira town in conveying runoff from typically rainfall, without inundation. The appropriate performance of urban drainage systems plays a key role in preventing storm water over flowing. There are no studies done before for performance assessment of urban storm water drainage system in Butajira town. Many studies done on drainage system are reviewed to learn a gap regarding to the storm water drainage system problem and the sustainable measurement. Almost all research was done on the performance assessment of storm drainage system by using the rational method, SCS method and SWMM model to determine the runoff occurring at the area for comparison. In this study the storm flood was modeled with SWMM5.1 model, mitigation measures to minimize the run off occurrence at the Butajira town were done using Low Impact Development (LID) that compatible with SWMM5 model.

1.2 Statement of the problem

Heavy rains with high runoff in Butajira town causes storm water drainage problems of over flowing that cause damages to houses close to the banks of the streams. This has been the cause for substantial loss of property during the rainy season. Majority of the road pavements are red ash and compacted earth which lacks proper drainage system. As a result, flooding is the main Causes of structural damage to the roads during rainy season. The road pavements are deteriorating at a faster rate than they can be maintained. It is important to note that this problem will increase further as the increase in population and population density, the increase in paved areas, create an increase in storm water runoff. Drains in the majority of cities discharge into gully which are eroded badly. There are rarely exit structures from the drains into watercourses. There are no retention or detention ponds.

The problem of drainage such as overtopping and Flooding of urban storm water may be due to either small drainage dimension, an incremental of rainfall, the increasing of pavements in the overall catchment or urbanization in a town, and /or improper management of the sewage system like, using drainage system as solid waste removal. Because of no maintenance some of the top element of the drainage ditches cover has been broken, most of the town constructed drainage diches are open and exposed for easily entrance of suspended materials. Moreover, they are treats to life of Children's and elders. The flood risk increase due to the entrance of the sewerage to the drainage system and also most of open earthen channels provided to drainage system. This problems are more pronounced in Erenzaf sub city around condominium and Stadium area to Erenzaf River of Butajira Town.

1.3 Objective of the study

1.3.1 General objective

The general objective of this thesis work was to assess the performance of urban storm water drainage system of Butajira town using SWMM model.

1.3.2 Specific objectives

- To assess the existing condition and identify major problems related to storm water drainage system of Butajira town
- To assess the hydraulic performance of storm water drainage system of study area
- To test some best management practices in the study area using LID (low Impact development) with SWMM5 to minimize flood and to recommend appropriate measures

1.4 Research questions

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

1) What is the status of current storm water drainage system and major problem related to their in the town?
2) What is the hydraulic performance of storm water drainage system of study area?
3) What kind of measure has been taken to minimize runoff occurrence by using low Impact development with SWMM5 model?

1.5 Scope of the study

The scope of this study is relay on assessing and modeling of drainage system performance using SWMM tool to compare the results with actual measurement for calibrating the two results and also for sustainability in Drainage schemes that contribute for better services of Butajira town drainage system performance. This study specifically focused on Condominium sefer and Stadium area in Erezaf sub city of the town to assessing of drainage system, to identify major problems and evaluating alternatives for drainage problem mitigation and to recommend appropriate measures to solve the problem. Moreover, the study aims to identify existing conditions and problems inhibiting the adoption of modern solutions to storm water drainage systems in the town. Also the study simulates the drainage networks of each sub catchments.

1.6 Significance and limitation of the study

This study benefited to the area for future storm water drainage structures construction assessing the performances of the existing storm water and proposing mitigation measure to avoid improper functioning. The findings from this research provide valuable information that can be used as an input or reference for further studies in this area.

Also it is beneficial for academicians and researchers who conduct similar researches on other storm water drainage structures, urban drainage, storm water management, local street drainage and land use /land cover effects on runoff. The result of this study helps in filling the gaps by identifying problems to sustainability, taking proper designing of storm water drainage system and proper functioning of drainage schemes in Butajira town.

This study doesn`t include structural design of all part of drainage system of the town and only mentioned part of the town i.e. focusing flood affected area and describing of the town existing condition. In other way round lack of recorded data was the challenge of my study and working plan drainage system drawing are not found in municipal regarding to drainage network.

2 LITERATURE REVIEW

2.1 Overview of storm water discharges

Population growth and urban development can create potentially severe problems in urban water management. One of the most important facilities in preserving and improving the urban water environment is an adequate and properly functioning storm water drainage system construction of houses, commercial buildings, parking lots, paved roads and streets increases the impervious cover in a watershed and reduces infiltration. Also, with urbanization, the spatial pattern of flow in a watershed is altered and there is an increase in the hydraulic efficiency of flow through artificial channels, curbing, gutters and storm drainage and collection systems. These factors increase the volume and velocity of runoff and produce large peak flood discharges from urbanized watersheds than occurred in the pre-urbanized condition. Many urban drainage urbanization and have inadequate capacity (Manish, 2015).

Storm water is the part of precipitation that accumulates on earth's surface, ditch of road side, in culvert, on pavement and generally in drainage system. During rain storms, water that falls on to impervious surfaces flows to the nearest storm drain or local water body. This water can come from events other than a rain storm, such as a snow melt or street wash water, all of which are defined as storm water (Jared, 2011).

In urban areas, rain that falls on the roof of our house, or collects on paved areas like driveways, roads and footpaths is carried away through a system of open channel that separated or mixed from sewerage system. Unlike sewage, storm water is not treated. If storm water will be not drained properly, it would cause inconvenience, damage, flooding and further health risks (Davies, 2000).

2.2 Previous studies on Ethiopia

Some studies that had conducted in different parts of Ethiopia shown that there were storm water drainage problems. Among those studies some are: (Mulualem and Naga, 2018) has studied the Performance Assessment of Road Drainage Systems of Burayu Town and found that runoffs runover road surfaces due to soil erosion, lack of drainage lines along the roads in the town lack of appropriate maintenance of existing drainage facilities, poor waste management system and the drainage channels are filled with or blocked by silt and garbage.

The study of the urban drainage system in Addis Abeba, Yeka sub-city (Adugna, 2009) found that Effect of deforestation of Yeka Mountain and pavements of structures are the major problems. A research by (Asfaw, 2016) assessed storm water drainage systems in Kemise Town, and found that drainage system is inadequate at different parts of the study area and improper construction alignment problem.

(Wagari and Moltot, 2020) studied on performance of urban drainage of Holeta town using SWMM5 model and found that the storm water drainage management of the town is not efficient as a result of managing problems and drainage systems are not well connected. Moreover, System lacks the capacity to carry large amounts of water, hence resulting in overflowing. (Hassen, 2016) studied on modeling and analysis of urban flooding in Bole sub-city system performance and evaluation of possible improvements using EPA SWMM5 and found that road surface drainage of the study area was inadequate due to insufficient road profile, insufficient drainage structures provision, improper maintenance and lack of proper interconnection between the road and drainage infrastructures. All of these previous reviews provide valuable background on the concepts, features, objectives, techniques and tools for sustainable drainage design, with a specific focus on one of the components.

2.3 urban storm water and management

2.3.1 Storm water

Storm water is the water drainage off a site from a site during the rain that falls on the land and everything it carries with it. In an urban area, storm water is generated by rain runoff from the roof, roads, driveways, footpath and other impervious or hard surfaces (Ericsion, 2010).

Storm water is rain water with other impurities within it that might be percolating in to the ground, retained on surface and/ or runoff. In urban environment, because of imperviousness storm water cannot easily infiltrate to the ground as it does in natural landscape. This creates effects such as flooding and water pollution (Schueler, 1994).

2.4 Storm water management system

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

Storm water management, whether structural or non-structural, on or off-site, must fit into the natural environment, be functional, safe and aesthetically acceptable. Re vegetation with native, non-invasive grasses, shrubs and possibly trees, creating low impact development may be required to achieve compatibility with the surrounding urban storm water (Manual, 2002).

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

2.4.1 Solid and Liquid waste management

Solid and liquid wastes generated from both animal and domestic sources can significantly impair drinking, irrigation, recreational water and other water sources in rural and urban areas. Waste as a management issue has been evident for over four millennia. Disposal of waste to the biosphere has given way to thinking about, and trying to implement, an integrated waste management approach. Solid waste management is a separate discipline dealing with the control of generation, storage, collection, transfer and transport, processing, and disposal of solid wastes in a manner that is in accordance with the best principles of public health, economics, engineering, conservation, aesthetics, and other environmental considerations, and that also is responsive to public attitudes. In its scope, solid waste management includes all administrative, financial, legal, planning, and engineering functions involved in the whole spectrum of solutions to problems of solid wastes thrust upon the community by its inhabitants (Syed, 20006).

Generally solid and liquid waste management affects the urban stormwater drainage system of the study area.

2.4.2 Storm water drainage system

Storm drainage is the process of drainage excess water from streets, sidewalks, roofs, buildings, and other areas. The systems used to drain storm water are often referred to as storm drains. Many cities and towns have carefully planned storm water drainage systems that consist of inlets, outlets and pipes. The inlets of storm drains are often covered by protective gates that help to ensure that large items don′t fall in while water can enter freely. Since it′s important for the large amount of water to flow in to these drains, the bars of the grates must be spaced some distance from each other.

2.5 Functions of storm water drainage system

The system′s of drainage functionality are to collect surface water or groundwater and direct it away, thereby keeping the strength of the bed drained. It also manipulates the flood produced in appropriate and place to dispose in rivers or place where prepared for ground recharge minimizing the damage which tends to occur. The drainage system must also protect the substructure from flooding, erosion, public health and safety, environmental protection and sustainable development. Additionally, the impact of drain and sewer systems on the receiving waters shall meet the requirements of any national or local regulations or relevant authority (Manual, 2002).

2.5 1 Urban drainage system

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

2.6 Sustainable urban drainage system

Sustainable Urban Drainage System (SUDS) is used with its main focus on maintaining good public health, protecting valuable water resources from pollution and preserving biological diversity and natural resources for future needs (Willems, 2012)**.**

Indications for sustainable urban drainage system (SUDS) is that in the potential way to develop the natural system of storm water handling, in order to reduce peak flows and deliver treatment for the storm water on to the recipients. The effects of high peak flows, which arrive quickly after the storm starts. Since the drain systems in the town normally was’t designed to handle these infrequent peak flows, flooding is often the result. With the introduction of SUDS, the water late on it′s way downstream, in similarity to nature′s way of handling storm water runoff (ERA, 2013).

SUDS is known as Low-Impact Development (LID) in the United States and Canada, which describes an approach promoting the interaction of natural processes with the urban environment to preserve and recreate ecosystems for water management (Coffman, 1998). LID puts the emphasis on conserving and using natural features in combination with small-scale hydrological controls to mitigate adverse impacts of urbanization (Elliott, 2007).

2.7 Hydrological Modelling

When choosing a suitable model, it should first be considered if it is possible to use the model in respect of investments in time and money. It should then be considered whether the model gives the desired output data required for the project and if the input data required is possible to obtain within a reasonable amount of time and price (Beven, 2012). A complex model often requires more input data than a simple model, while a simple model with fewer input data instead may not be specific enough for the current study. It is therefore important to select a good combination of model complexity and available input data (Shamsi, 2005). Finally, it should be considered whether there are limitations in the model that will be affect the results (Beven, 2012).

There are several hydrologic-hydraulic models, including Storm Water Management Model (SWMM),Western Washington Hydrology Model(WWHM2012), System for Urban Storm water Treatment and Analysis Integration(SUSTAIN),Urban Drainage and Sewer Model (MOUSE), Distributed Routing Rainfall-Runoff Model(DR3M), Storage, Treatment, Overflow, Runoff Model (STORM), and Long-Term Hydrologic Impact Assessment-Low Impact Development (L-THEA-LID), which are common only used to simulate the effects of LID on hydrology and water quality(Ackerman &Li, 2019).Among them, the SWMM model has been extensively used in the U.S. and many other countries worldwide (Elliott & Wang, 2017). SWMM can simulate the peak discharge, runoff volume, and water quality for short- and long-term rainfall events, making it suitable for designing and implementing several LIDs (Wang & Soleimani, 2016).

2.8 Model Justification

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

2.8.1 Selection criteria of SWMM model

There are various number of techniques for evaluation of storm water runoff on bases of water balance equation, empirical equation and viable models like STORM model, CIVIL CAD, RAFTS, MOUSE and SWAT by such strategies calculations for penetration, overflow of surface, routing of flow, and slacking of surface overflow have been rearranged 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, 2011).

2.8.2 STORM Model

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

2.8.3 MOUSE

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

2.8.4 RAFTS

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

2.8.5 SWMM Model

It is used for planning, analysis, and design related to storm water runoff, combined sewers, sanitary sewers, and other drainage systems in urban areas. It is a dynamic rainfall-runoff simulation model used for single event or long-term simulation of runoff quantity and quality from the primary area. It can simulate the rainfall – runoff, evaporation, infiltration, and groundwater connection for root, street, and grassed areas rain gardens and ditches and pipes. The runoff component of SWMM operates on a collection of sub-catchment area that receives precipitation and generates runoff and pollutant loads. The runoff water flow that can be routed through drainage system like pipes, channels, and outlets are identified (Suriya, 2012). SWMM offers four choices for modeling infiltration, modified Horton method, Green –Ampt method, infiltration method, curve number method. EP SWMM5 is utilized in urban areas flood modeling and analyses to fix the drainage size by way of considering the pervious and impervious areas. So that modeling in urban drained the SWMM software program is more comfortable for Butajira drainage systems Hydraulic and Hydrologic Simulation program for this study objectives, the SWMM5 has been favored for city runoff estimation.(Kong , 2017).

For this study, SWMM was selected due to its ability to model dynamic rainfall-runoff properties in urban environment and its wide applicability for planning, analysis and design related to drainage systems in urban areas. SWMM can simulate the peak discharge, runoff volume, and water quality for short- and long-term rainfall events, making it suitable for designing and implementing several LIDs(Wang, 2017 & Soleimani , 2016).

2.9 Low Impact Development (LID) concept

Low impact development (LID) is a new, innovative approach for storm water management that seeks to mitigate the adverse effects of urbanization by maintaining the pre-development natural hydrology of a site using decentralized, micro-scale control measures (NDM, 2010 & Guo 2009) by achieving water balance (Davis, 2009). LID emphasizes the use of small scale, natural drainage features integrated throughout the urban area to slow, clean, infiltrate and capture the urban runoff and precipitation, thus reducing water pollution, replenishing local aquifers, and increasing water reuse.

Low Impact Development (LID) is an urban planning technique with the primary objective of urban flood management through source control of storm water runoff. Using LID in urban areas is becoming one of the most popular methods for sustainable storm water management and flood mitigation (Ahmed, 2013). LID is a smarter and more sustainable urban development and flood management technique (Su, 2021). LID reduces the risk of flood through several non-structural and structural measures (Douglas, 2011). Some of these measures include reducing imperviousness, conserving natural resources and ecosystems, as well as constructing Green Infrastructure (GI). GIs are infrastructure which is implemented for LID purposes; some well-known examples of GIs are bio-retention cells, green roofs and permeable pavements (Papalexiou &Booth, 2019).

Low Impact Development (LID) controls Basically, LID is a land re-development approach to manage storm water. The main goal of LID is to reduce the negative effects of precipitation flooding waters by maintaining the pre-development hydrology of a site by decentralizing micro-scale controls (Coffman, 2002). LID practices effectively reduce water-related problems through infiltration and evaporation of the storm water resulting environmental, social, and economic benefits. The common LID practices are bio-retention, green roofs, permeable pavements, rain gardens, vegetative swales, and rain cisterns (a.k.a. rain barrel) that are used to create a functionally equivalent hydrologic landscape (Coffman, 2002).

2.9.1 Low Impact Development (LID) controls

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

2.9.2 Selection criteria of LID techniques

A range of things to be considered while choosing such that the maximum suitable method is followed and SUDS are successfully applied. Such elements encompass website online suitability, available land area, value, maintenance troubles, and network popularity. (Square and Place, 2010), Overland flow from LID controls can be modeled in three ways. (Eyosias, 2018).

The first approach is to route impervious sub catchment to pervious sub catchment to receiving node as pervious area properties are to be matched to LID control design. The pervious area of the sub catchment acts as LID control. This approach is not realistic and does not give accurate Results.

The second approach is to create LID sub catchment as a separate sub catchment and to route the original sub catchment to the LID sub catchment to receiving node. The LID design is to be matched to sub catchment properties. LID area is to be extracted from Original pervious or impervious area.

The last approach is to create LID as part of original sub catchment and to route runoff through LID prior to receiving node from LID area is to be added to original pervious or impervious area. (USEPA., 2000)

If multiple LID units are placed in a sub catchment, then the LID units take the impervious area runoff of a sub catchment. Different capture ratios can be given to different LID units. The options for routing the surface flow and under drain flow of the LID units are as follows: a) both surface overflow and under drain flow is routed to the sub catchment's outlet; and b) Under drain flow can be routed to a separate outlet other than its sub catchment pervious/impervious area.(USEPA., 2000). Sub catchment of Butajira town are contain both pervious and impervious and no area used separately for LID be for receiving node to control the storm water. Because of this the area were LID control applied are considered as part of original sub catchment. So the second alternative is selected to route runoff through LID prior to receiving node.

A. Permeable pavements

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

Figure 1: View of permeable pavements (EPA, 2015)

B. Infiltration trench

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

Figure 2: View of infiltration trench structural (EPA, 2015)

C. Bio retention

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

Figure 3: View of construction practice of bio retention method (EPA, 2015)

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

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

2.10 Urban drainage problem mitigation measure

The extent of impervious land covering the landscape is an important indicator of storm water quantity and quality and the health of urban watersheds. Impervious land coverage is a fundamental characteristic of the urban and sub urban environment rooftops, parking's areas and other impenetrable surfaces cover soils that before development, allowed rainwater to infiltrate (Eskade, 2013). The achievement of runoff reduction starts by recognizing that developing or redeveloping land within a watershed inherently increases the imperviousness of the areas and therefore the volume and rate of runoff and the associated pollutant load and outlines various approaches to reduce this impact through planning and design techniques.

Improving Low impact development (LID) to mitigate urban flood, is a new, innovative storm water management approach for the land management and development because it has ability to reduce runoff, soil loss and improve the water quality (Rezaei, 2019)and now it has become popular around the world (Papalexiou, 2019).

The technics that is related to reduce of urban run-off is encompasses manage watershed impervious area, minimize directly connected impervious areas to storm drainage system and consider coefficient of Runoff reduction areas. Although some LID practices can also provide significant pollutant reduction benefits, at this time SWMM only models the reduction in runoff mass load resulting from the reduction in runoff flow volume. (EPA SWMM. 2015).

Figure 1: Liquid wastes in to storm drainage (site visiting time)

Figure 2: Drainage lines filled by grass, sediment and have no road drainage structure on both sides (site visiting time)

Figure 3: Flooding and its impacts around Condominium and Stadium sefer in Butajira Town (Butajira. municipal)

Figure 4: Satellite Map Showing Flooding Risk Areas in Butajira Town (Butajira Municipal)

3 MATRIALS AND METHODS

3.1 Description of the study area

3.1.1 Location and size of the town

Butajira town located in the Southern Nations, Nationalities and people′s Region (SNNPR) south of Addis Ababa. Geographically, the town is roughly located 08^0 07' 21.4'' N latitude of equator at 038^0 22' 33.4'' E longitudes. Butajira is located to the southern part of the federal capital city of Addis Ababa at distance of 135km and at a distance of 163km from the Hawassa city.

The town is divided in to sub-cities (Erinzaf & Ersha) and five kebele administrative units with 53 village's covers an area of 1805 hectare of land with average elevation 2100 – 2400 meter a. s. l. The town sits astride the main north-south highway coming from Addis Ababa and south in the Hossana. Among 22 reform towns of southern region, the town has got third grade status.

The overall population density is medium, SNNPR, with over 500 person /km^2. Within a 30km radius of Butajira, no less than six small towns. Most of which are on or near to the north, south and south east highway. Some of them are Buei, kella, Enseno, Koshe, Kibet, Worabe etc.

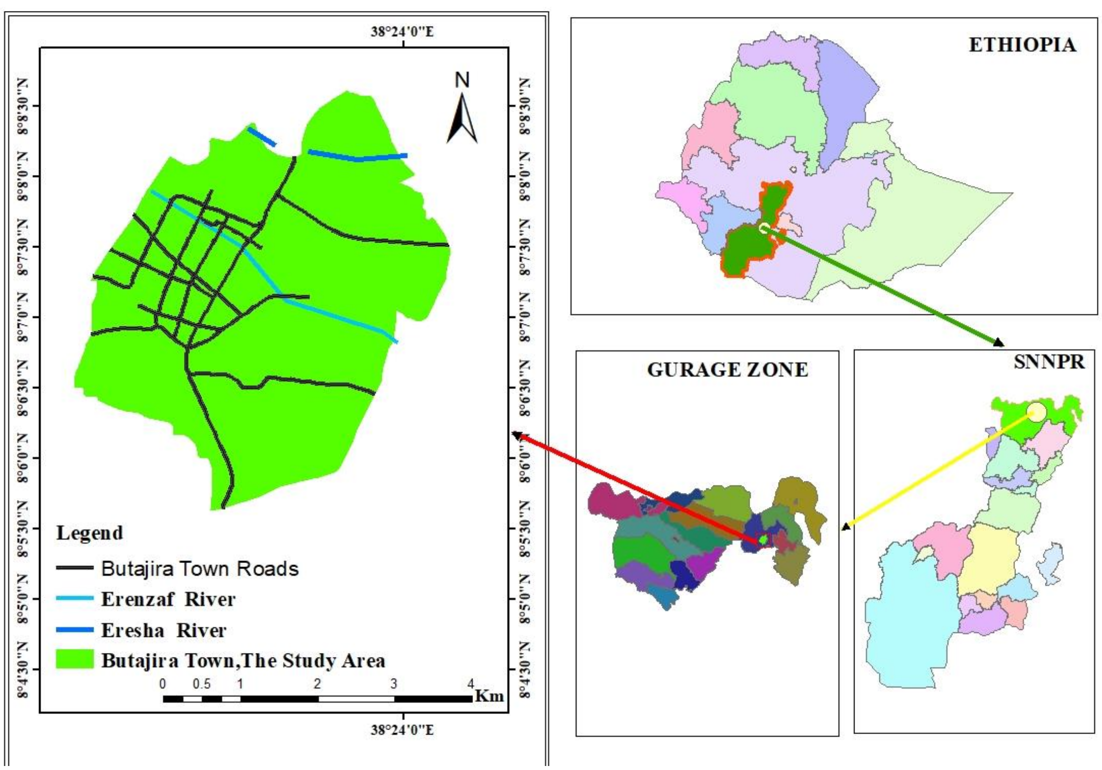


Figure 5**: Area Map of Butajira Town (Arc GIS)**

3.2 Topography and drainage pattern of the town

The relief of the town is characterized by plain, hilly areas (rugged), and gorges along river side. However, the land form of the town is dominated by gentle slope. The height of the town slightly increases from east to west forming gorges along the two main rivers in the town. The height of the town ranges from 2100-2400 meters a. s. l having an average elevation of 2131 meters a.s.l.

Erinzaf and Erasha rivers merge together after a short distance from the city and discharge in Meki River (in oromiya region). The surface run-off from the sides in the immediate neighborhood of the town passes though the town exerting pressure in it. The drain flow is generally east to west to join Meki- River.

Erinzaf and Eresha rivers are started from the same source and they flow separately to the east and south direction bounding the town from north east and south east. At last they join again and flow to Meki – River.

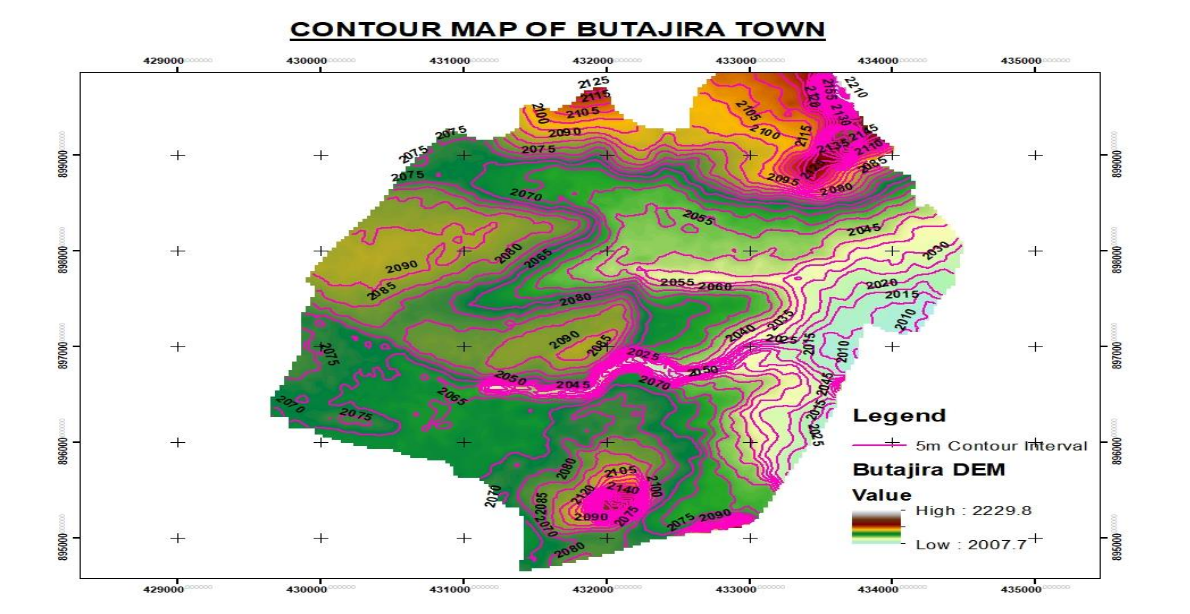


Figure 6: Contour Map of Butajira Town (Arc GIS)

3.2.1 Slope of Butajira town

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

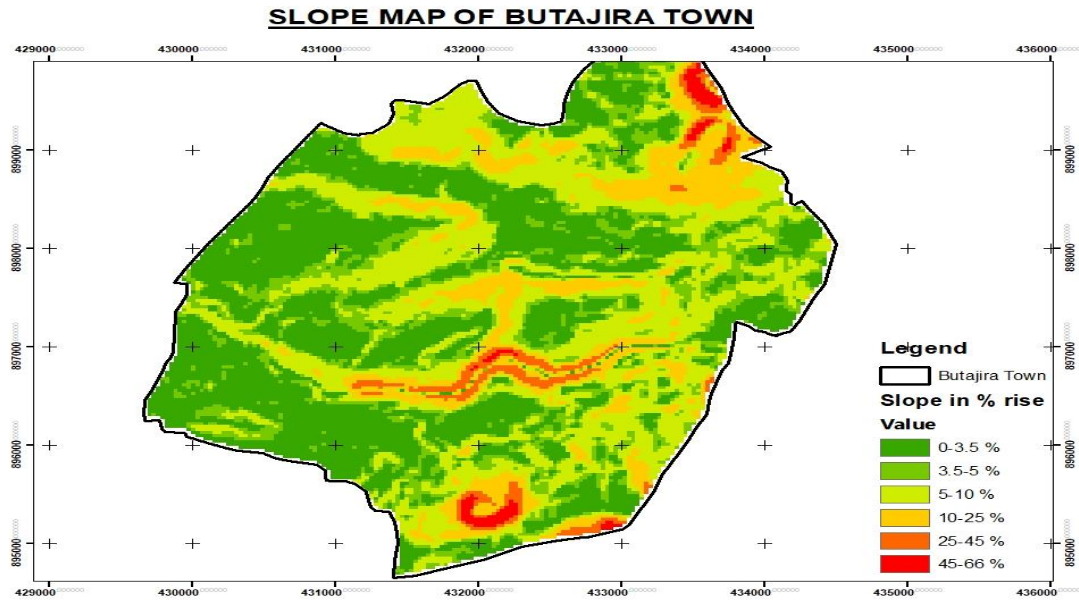


Figure 7: Slope Map of Butajira Town (Arc GIS)

The slope of the town is analyzed using xyz coordinates through GIS tools. In the town plain slope which is less than 3.5 % occupies 38 % of the total area. These are floodable areas so they are exclusive for different constructions and also they are exposed for storing stagnate water that may negatively affect the health and safe movement of the town dwellers.

Table 1: slope classification and area covered

Slope classification in %	Area		% age Area cover
	in m2	in ha	
0 _ 3.5	6,089,225.28	608.92	38%
3.5 _ 5	2,621,176.98	262.12	16%
5 _ 10	4,890,370.09	489.04	30%
10 _ 25	1,911,454.71	191.15	12%
25 _ 45	453,646.80	45.36	3%
45 _ 66	161,126.14	16.11	1%

3.2.2 Land use Land cover

The land use land cover of the surrounding of the town is a very rich agricultural area with dense habitation of small farms which are supported by very productive field crops, fruit, and even chat trees. But based on the land use/ land cover and CSA result in Butajira town currently urbanization highly increased because of land use modifications associated with urbanization such as the removal of vegetation, replacement of pervious areas with impervious surface this increased runoff peak, runoff volume and reduced time of peak. The land use land cover data is an essential input for the calculation of run off coefficient in the determination runoff by SWMM5.1 model.

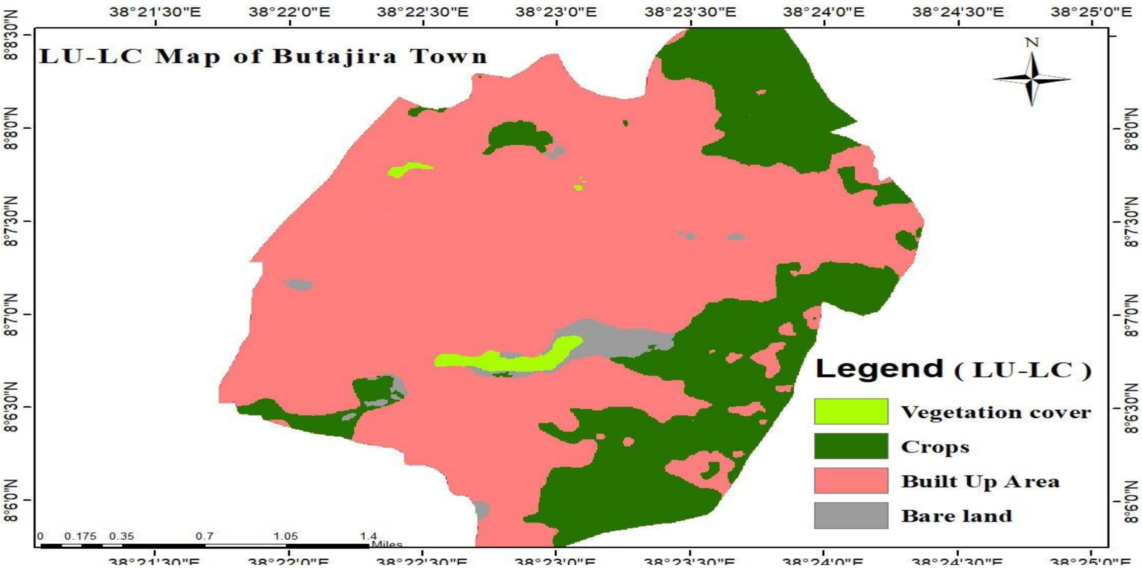


Figure 8: Land use Land Cover Map of Butajira Town (Arc GIS)

The natural vegetation surrounding the town is depleted and causes land degradation. Most of the project area was covered with built up areas, crop lands, shrubs cover areas, Tree cover areas, grassland, & bare areas in some compounds of individuals, religious and government institutions. Planting these natural vegetation's are very important for rehabilitation of degraded land within and surrounding the town and this contributes for modifying micro climate of the area.

3.3 Soil type

According the Soil description of SNNP Regional Atlas of soil map of the southern region Haplic Phaeozems of volcanic origin are the major soil types of Butajira town. With a deeper profile the soil is suitable for agricultural activities, whereas the agricultural value is limited in steep slopes and highlands where soil depth is often limited by hard rock at shallow depth. According to Mesqan Wereda Rural Development and Butajira town socio-economic and demographic information (March 2006), the soil has a textural classes of sandy, clay and loam.

The development of soils depends primarily on geologic and climatic conditions. The FAO Soil Map of Ethiopia classifies 19 soil units. For this research, since the FAO classification system is recent, the FAO classification (FAO, 1998) is selected, about 2 major types of soils are found.

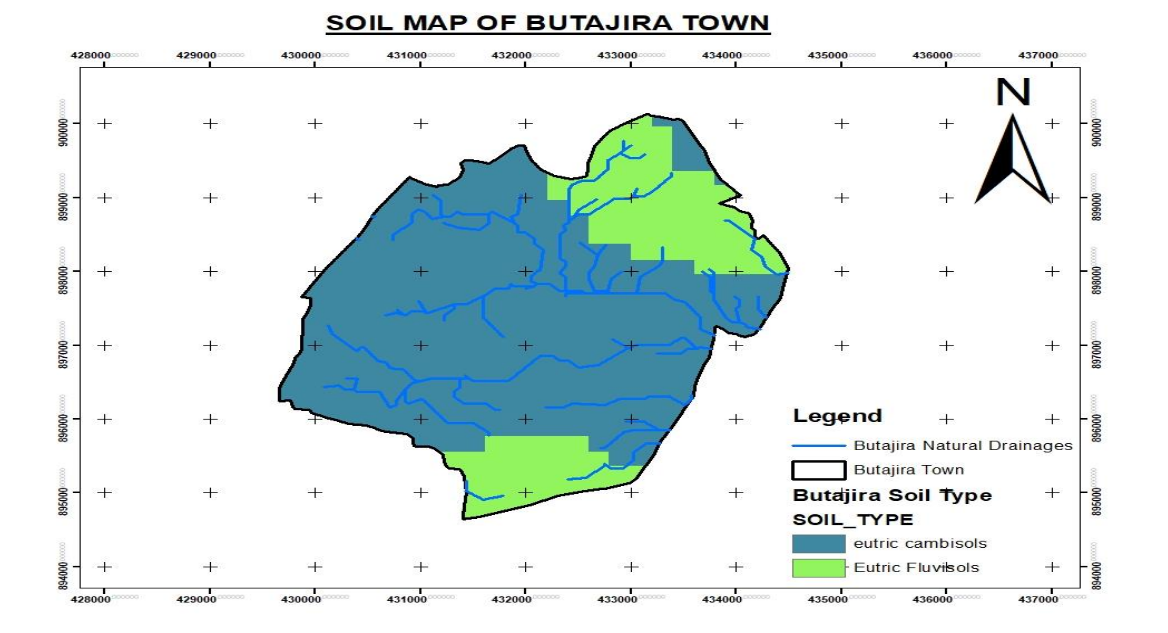


Figure 9: Soil Map of Butajira Town (Arc GIS)

3.4 Natural and artificial drainage networks of town

Butajira town and its vicinity is part of the areas which are found in Rift valley. This means Butajira town and its surrounding are areas which are drained to Meki River and its tributaries. When we consider specifically the drainage pattern of Butajira town and its neighborhood type of drainage pattern is observed. This shows that there are optional natural drainage channels to dispose the runoff concentrated of the town.

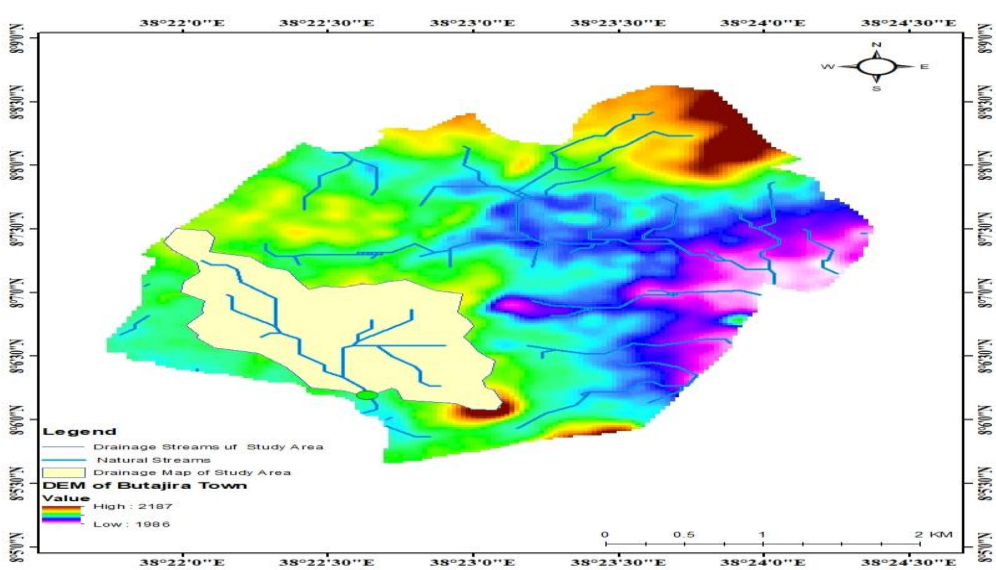


Figure 10 : Drainage Map of Butajira Town (Arc GIS)

3.4.1 Drainage network of Butajira town

There are no proper drainage channels as well as well-designed drainage systems in most part of the city. This is mainly attributed due to the poor road network system of the city. The existing roads are not properly planned and designed and have no sufficient width for accommodation of other infrastructures. Up until now there was limited proper design document of the drainage system as well as proper plan of road network system of the city. The existing roads are not properly planned and designed and have no sufficient width for accommodation of other infrastructures. The previous master plan (2012) didn't recommend anything about drainage system and network for the town. In connection to this limited plan design drawings are found in the municipality regarding the existing drainage network.

Existing storm drainage facilities in Butajira town are generally classified in to closed and open drainage lines. Closed drainage lines are found main roads of Butajira to Addis Ababa and Butajira to Welikite road. Open drainage channels, constructed by masonry are found along sub-mains and local roads. In many localities, access roads serve as wide open channels with severe erosion and flooding problems. The current open dich drainage channel has minimized the size and creates inconvenience for the pedestrian walkway.

In general, from the overall observation and assessment, the main challenges and problems of the Existing drainage channel of study area are;-

- Most of drainage systems have no proper start and end point to be terminated and hence dead ends observed on most of the drainage structures construct.
- Most ditches have not proper slope
- The coverage is still at its low level
- The open ditch drainage facilities are creating problem for pedestrians like children and elderly people in particular
- Most of local and collector road have no road drainage structure on both side. Some of cobble stone road have only one side drainage structure
- The drainage systems are fill by solid, liquid, grasses and sand hence resulting in overflowing

3.5 Climate and hydrology

3.5.1 Climate

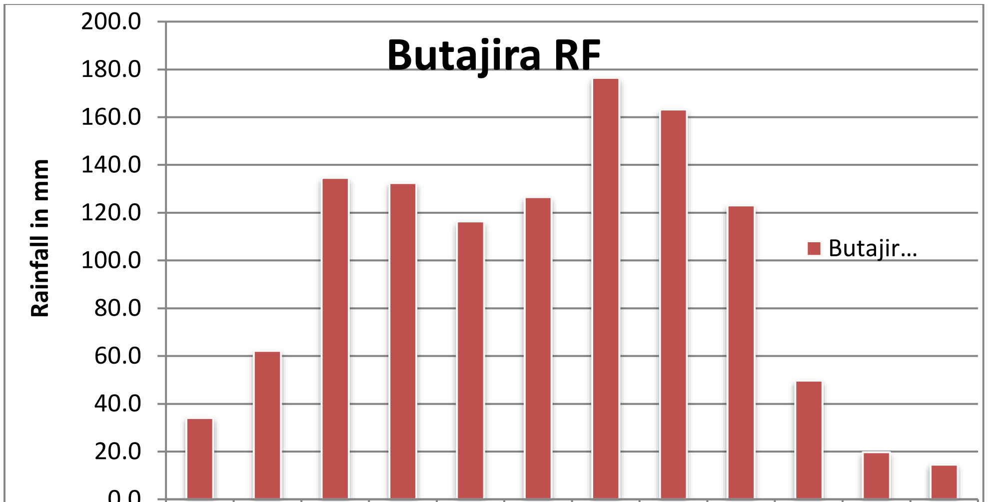
Climatic condition of the city is uniform and suitable for life. Butajira climatic condition is characterized by relatively moderate. Average temperature of the city is 23^{0}c with annual rain fall of 1152 mm. Majority of the road pavements are red ash and compacted earth which lacks the proper drainage system. As a result, flooding is the main causes of structural damage to the roads during rainy season and dust is a problem in the dry season. The road pavements are deteriorating at a faster rate than they can be maintained. It is important to note that this problem will increase further as the increase in population and population density, the increase in paved areas, create an increase in storm water runoff. Drainages in the majority of town discharge into gully which are eroded badly. There are rarely exit structures from the drains into watercourses.

- **Rainfall**

Since the town is situated at border of the Rift Valley, its rainfall categorized under summer major rainfall area. The mean annual rain for the station situated in Butajira area is 1267.07mm.

The rainfall distribution of the town can be summarized in to the following seasons.

- Summer (June, July and August). This season is the main rainy season of the town, during summer the town gets 40.9 % of the total annual rainfall.
- Autumn (September, October, and November). Meher (autumn) is the third largest rainy season of the town. It takes 20.5% of the total annual rain.
- Winter: - it is the driest month of the town like all parts of the country. Nevertheless the town gets 7.5%of the total annual rain during this season.
- Belg (March, April and May), this is the second rainy season for the town following summer. During this season the town receives 30.1% of the total annual rain.



	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Butajira RF	34.0	62.1	134.6	132.5	116.3	126.6	176.4	163.2	123.0	49.7	19.7	14.4

Figure 11: Mean monthly Rainfall of Butajira Station

- **Temperature of Butajira town**

Temperature of any place is influenced by relief, the presence of water bodies, the existence of natural vegetation and cloud cover. As far as the town is concerned it is found within altitude 2100 to 2400 meter a.s.l. Generally as altitude decreases, temperature increases and the reverse is also true. Therefore the temperature of the town is somewhat higher when compared with towns that are established at higher elevation. The temperature of the town ranges from 18.2^{0}c- 19.85^{0}c. The average temperature of the town is 18.70c.

3.6 Hydrology

- Hydrological Distribution, Occurrence and Circulation

Probability distributions are used in hydrology to determine the probability of occurrence of extreme events: maximum flows, maximum volumes and maximum precipitation. There are two types of probability distributions: Discrete probability distributions and Continuous probability distributions. For a discrete distribution, probabilities can be assigned to the values in the distribution – for example, "the probability that the web page will have 12 clicks in an hour is 0.15." In contrast, a continuous distribution has an infinite number of possible values, and the probability associated with any particular value of a continuous distribution is null. Therefore, continuous distributions are normally described in terms of probability density, which can be converted into the probability that a value will fall within a certain range. In this section, the analyzed probability distributions are presented.

The probability density function, f(x); the complementary cumulative distribution function, F(x), and quantile function (inverse function), x(p) are presented below.

- **Population Size of the Town**

According to the recent population and housing census results (2013 CSA preliminary), the total population size of the city is 47,978, but now using arithmetic mean method (2014 E.C) its population is reached 66,827 from which male account 50.1% and, remains are female . The estimate population growth rate is 4.8% precent. Regarding age structure, 36 % of the total population of the city falls within the age bracket of 0-14 years. The majority of the population (61%) belongs to the working age group (between 15- 64), and the group with 65 years and above represent only 3 %. From this the dependency ratio of the town is 0.39.

- **Local Geology and Structure**

The study area is found between the eastern escarpment of the south west Ethiopian Highlands and the main Ethiopian Rift valley in the east. It is dominantly covered by volcanic formations. The rock that covers Butajira town and its surroundings is Trap series that include Magdalla

Volcanic groups and basaltic flows and related spatter cones. This formation is associated with construction materials like basaltic rock which has dark colour and red ash of volcanic origin.

According to Mesqan Wereda Rural Development and Butajira town socio-economic and demographic information (March 2006), the soil has a textural classes of sandy, clay and loam.

3.7 Data Collection and material used

For this study, information and data collection was obtained via two sources which include: primary and secondary sources. Primary sources study area observations: pictures of different drainage status were taken to show the true state of things in the study area. Observation was also made to identify the status of the drainage that properly functioning, partially blocked and fully blocked. A measure flow depth which used for model calibration and validation was done.

Secondary sources also physical data of the catchments and their storm water drainage systems was also collected for modeled of study catchments. Drainage shape and land use layout catchments areas, Invert Elevation and dimensions, slope and roughness parameters of drainage conduits and slope of the drainage profile was collected from as built gained from Ethiopia road authority Butajira town municipality. Rainfall data of 32 years was used for IDF curve developed was collected from

Ethiopia metrological agency and sensitive parameter that use as input for SWMM5 model was collected from books, journals, manuals etc.

A. Primary Data collection

- Site observation the overall of drainage system condition and booked camera
- Measured the drainage dimensions using GPS and Tape material for the input of SWMM model simulation.
- Measured the flow depth in drainage system that was used for calibration and validation of model.

B. Secondary Data collection

Table 2: Data and there sources

No	Types of data	Data source
1	Metrological Data	From the National Meteorological Services Agency collected weathered data of rainfall in daily from different stations of Butajira, Worabe, Bui and Ziway Observatory gauged station.
	Topography and shape file	From Ministry of water, Irrigation and energy use shape file of Rift Valley Lake basin.
2	Master plan and road network	Butajira Town Municipal
3	DEM (30m x 30m)	USGS/united states geological station/ Ministry of water, Irrigation and Electricity Addis Abeba Ethiopia
4	Land use /land cover data	Ministry of Agriculture, Addis Abeba, Ethiopia
5	Soil data	Ministry of Agriculture, Addis Abeba, Ethiopia
6	Population	CSA

3.7.1 Material used

All rainfall data recorded daily intervals was, collected from national meteorological agency. All rainfall data recorded daily intervals were, collected from National meteorological Agency. The data recorded used was of 32 years (1989 – 2020 G.C) record. Generally the data collection and material used for each results of study were as follows:

- Shape file data is used as an input for ARC-GIS software for catchment delineation and estimation of catchment characteristic
- ARC-GIS to obtain hydrological and physical parameters and spatial information of the catchments of the study area.
- Google Earth 2021 Software to verify watershed and divides of catchments of the study area.
- Hydrological and hydraulic data where used as input for SWMM tool
- GPS, total station and TAPE meter to measure elevation of nodes and drainage demission that input for SWMM tool.
- Storm water management model to determine the peak runoff
- All the primary and secondary data organized used the study
- Referring different journals/thesis, books, design documents and manuals used as guideline

3.8 METHOD

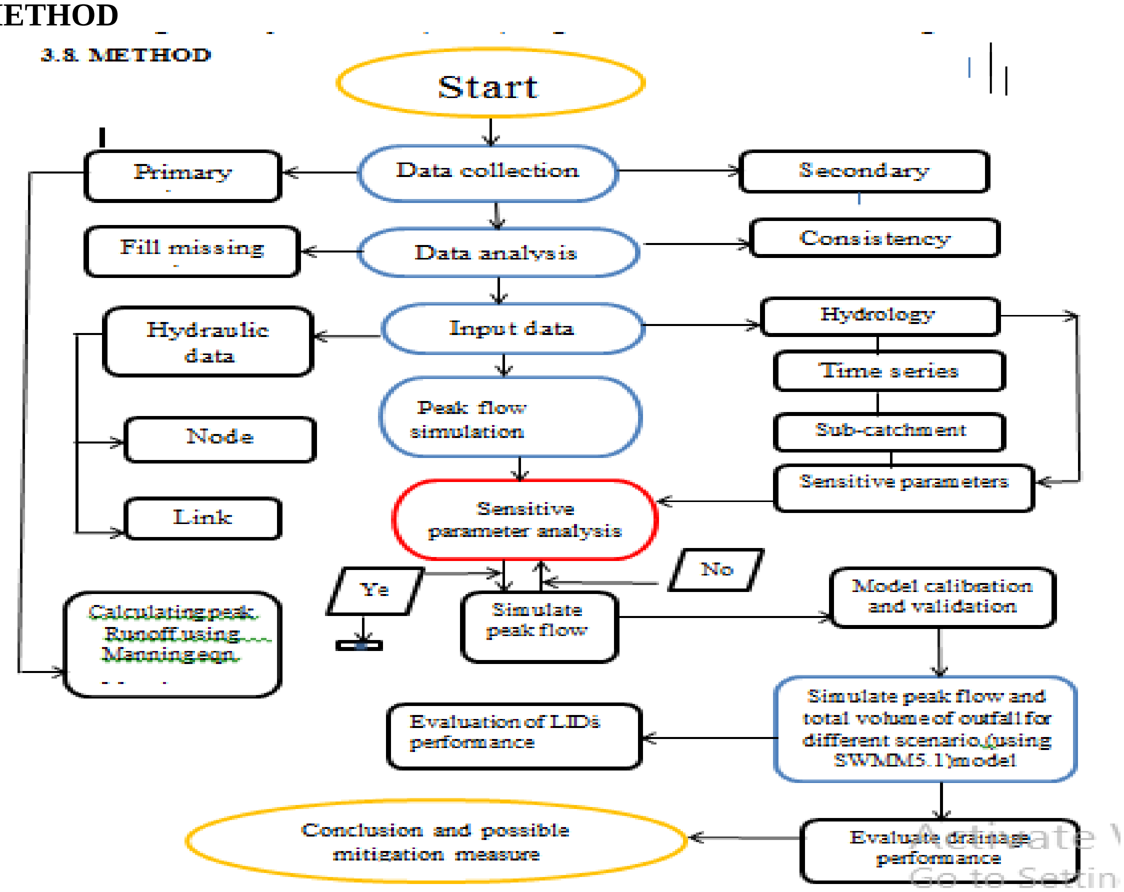


Figure 12 : Framework of the procedure followed

3.9 Data analysis

3.9.1 Filling of missing data

Precipitation is part of atmospheric moisture, which reaches the surface in several forms. Hydrologists start working when the precipitation reaches the surface. This connects hydrology with meteorology. Rainfall data plays a central role in developing rainfall runoff models. Measured precipitation data are important for several problems in hydrologic analysis and style purposes. The most problem in hydrological analysis is that these data′s are sometimes having partly missing records (Amin, 2016). Among different methods to treat the missing observations for this study normal ratio method was utilized.

- **Normal ratio method**

If the annual precipitations vary considerably by more than 10%, the missing record is estimated by the Normal Ratio Method, by weighing the precipitation at the neighboring stations by the ratios of normal annual precipitations (Chow, 19980).

$$Px = \frac{Nx}{m}\left(\frac{P1}{N1} + \frac{P2}{N2} + \frac{P3}{N3} \dots \dots \dots \frac{Pm}{Nm}\right) \dots \dots 3.1$$

Where:

Nx = Annual-average precipitation at the gage with missing values

m = Number of neighboring stations

P1, P2… Pm = Precipitation records at the neighboring stations

N1, N2…..Nm are annual rainfall of known stations

Whenever possible, the collected rainfall data of Butajira were checked against fill the missing data normal ratio method were used because of the normal annual precipitation of the index stations lies exceeds ±10% of normal annual precipitation of interpolation station.

Independent rainfall data obtained from other 3 nearby stations Buie station, Worabe station and welkite station for selected storm event were used.

- **Rainfall Data**

Rainfall data were obtained from Ethiopian Meteorological Services Agency which is nearby Butajira station located at an average distance of 5 km from the area. Available rainfall data on this station has been collected and analyzed in order to prepare the necessary depth or intensity input data for peak discharges computation. For this study, 32 year historic observation data is used to calculate the peak drainage storm water of the Butajira town.

3.10 Tests for consistency of data record

3.10.1 Outlier identification, retention, modification, deletion

Check on outliers has been undertaken on the recoded rainfall and flow data to identify any low or high outliers. Outliers are data points, which depart significantly from the trend of the remaining data. The retention, modification, deletion of these outliers can significantly affect the statistical parameters computed from the data, especially for small samples. All procedures for treating outliers ultimately require judgment involving both mathematical and hydrologic considerations. The procedure followed for detection and treatment of high and low outliers for this project are summarized in the next section based on statistical technique described here under: For this study I have used two methods to check the quality of data's.

- **Outlier Test**

Higher outlier $YH = \bar{Y} + KN*Sy$ KN values are obtained from tables

Maximum rainfall $X = 10^{YH}$

Lower outlier $YL = \bar{Y} + KN*Sy$ KN values are obtained from tables

Minimum rainfall $X = 10^{Yl}$

Where $\bar{Y}$ is mean Sy is standard deviation

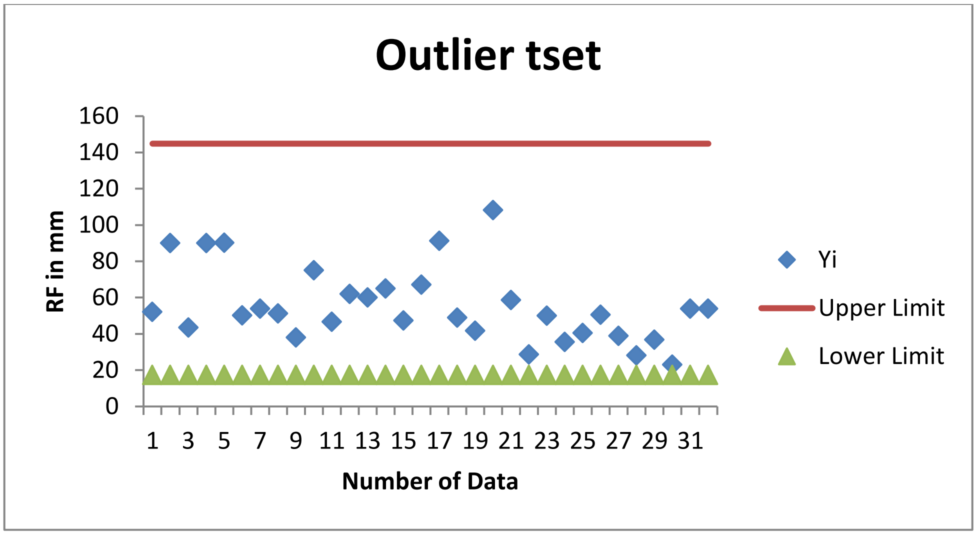


Figure 13: Outlier Test for Butajira station

Rain fall data used for input is should be detected by using outliers test. As the daily maximum rainfall data is detected by outliers test by help of range value. As it is examine the value of higher outliers rain fall depth is 144.85mm and the value of lower outlier's 17.36mm.

3.11 The hydraulic capacity of the existing storm water drainage system

Hydraulic properties and design of the existing drainage systems of Butajira town was collected from secondary data and field measurement. The following method was used to estimate the hydraulic capacity of the existing drainage of the town and to generate the data.

- **Manning formula**

Manning's formula was used for calculating the cross-sectional area, wetted perimeter, and hydraulic radius for the flow of a specified depth in a canal of known diameter. Manning's equation is applicable for a constant flow rate of water through a channel with constant slope, size & shape, and roughness.

$$Q = \frac{AR^{2/3}\sqrt{S}}{n} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.2$$

Where,

Q = the volumetric flow rate passing through the channel reach in m3/sec.

A = the cross-sectional area of flow normal to the flow direction in

S = the bottom slope of the channel in m/m (dimensionless).

n = a dimensionless empirical constant called the Manning roughness coefficient.

R = the hydraulic radius = A/P.

P = the wetted perimeter of the cross-sectional area of flow in m.

For storm drains flowing full, the above equations become:

$$Q=\frac{0.312D^{\frac{8}{3}}S^{\frac{1}{2}}}{n} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.3$$

$$V=\frac{0.397D^{2/3}S^{1/2}}{n} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.4$$

Where: D= diameter of pipe, m

3.12 Urban storm water drainage lines

There are two types of urban drainage lines in Butajira town. Namely: concrete and masonry drains, which are constructed by the municipality and Ethiopian Roads Authority in order to safely discharge the flood generated within the town in the process of urbanization. But, as Observed during data collection through field survey with the help of road net-work, these drains becomes causes of flooding by collecting rain water from various parts of the drainage system. These drains totally concentrated on asphalt and on non-asphalt roads. The asphalt roads provided with manmade drainage ditches of closed types either in both sides or in one side. The gravel roads provided with manmade drainage ditches either closed or open rectangular masonry. Moreover, Cobble stone side drains also provided with the cobble stone road. During field survey the side drains which are provided at both sides of the cobble stone roads cannot carry flood other than the flood which is generated on the cobble road surface and it is observed that in some cobblestone roads even no side drain is provided the existed drainage structure is shown in figure14.

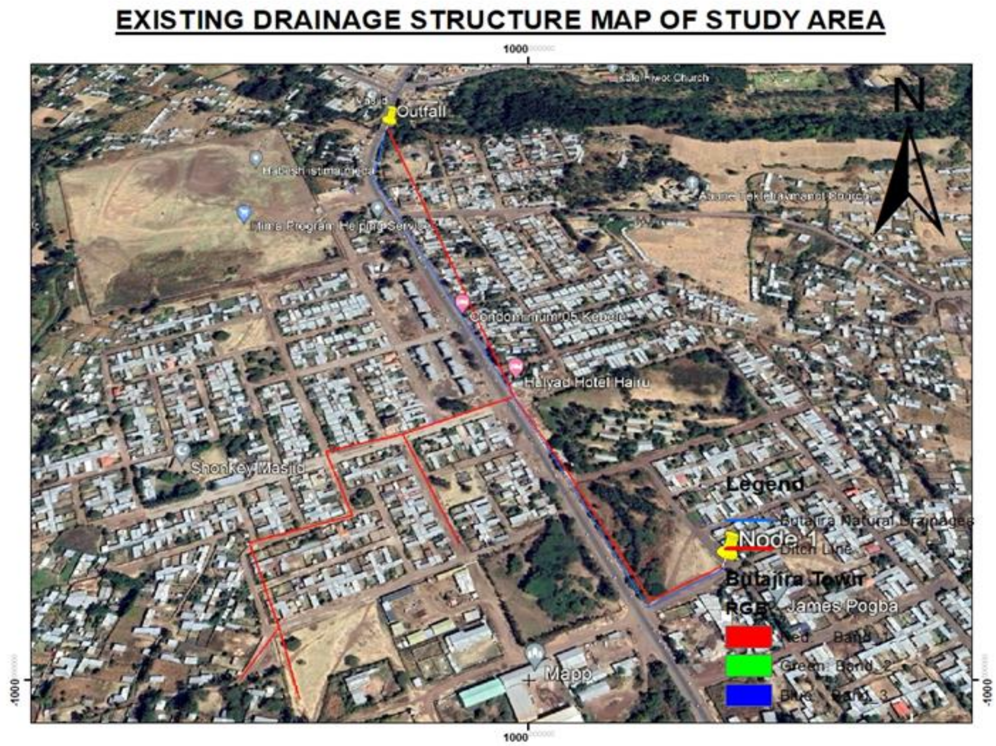


Figure 14: Existing drainage structure of Butajira town (Google earth, 2021)

3.13 The intensity duration frequency curve for the various return periods

The IDF relationships are used when designing drainage works for any engineering project, and allow the engineer to design safe and economical flood control measures.

Rainfall intensities of various frequencies and durations are important parameters for the hydrologic design of storm sewers, culverts, and other hydraulic structures. This can be achieved by the rainfall Intensity-Duration-Frequency (IDF) relationship, which is determined through rainfall frequency analysis. The order statistics approach is applied for the determination of distributional parameters to estimate rainfall and develop IDF relationships for a set of selected return periods of 2, 5, 10, 25, 50, and 100 years. Data like rainfall, rainfall intensity, return period, and rainfall duration are required to develop this curve.

Goodness- of -Fit (GoF) tests employed for checking the adequacy of fitting the distributions to the recorded data. Model Performance Indicator (MPI) such as the root mean square error and the correlation coefficient was used to analyses the performance of IDF relationships given by the probability distribution function.

3.13.1 Design rainfall of shorter duration

The rainfall depths obtained from gauging station are of 24hr duration depth. Design and analysis of drainage structures require rainfall intensity duration relationship of shorter duration. Because rainfall data of shorter duration is unavailable, appropriate IDF derivation for shorter duration is required and calculated by using a reduction formula, which is suggested by the Ethiopian Road Authority (ERA, 2013). Drainage Design Manual of 2013 suggests the following Equation (3.6) for calculation of shorter duration rainfall from 24-hour duration rainfall was used.

$$\frac{Rt}{R24} = \frac{t(b+24)^n}{24(b+t)^n} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots 3.5$$

Where:

Rt: R24 = Rainfall depth ratio

Rt = Rainfall depth in a given duration t

R24 = 24hr rainfall depth

t= time (hr)

Coefficients b = 0.3 and n = 0.78 - 1.09 are constants

Rearranging the above Equation:

$$Rt = \frac{t(b+24)^n}{24(b+t)^n} \times R24 \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots 3.6$$

The methods employed to develop IDF curve for the shorter duration events using the above equations are as follows. Among many frequency analysis Log Pearson type III better value, so for this study Log Pearson type III distribution was selected Using the trend line equation obtained from Log Pearson type III distribution method of frequency analysis, i.e. y= 0. 5456.x+68.84 where y is 24-hour rainfall depth (R24) of a return period x under consideration, R24 is calculated for 2, 5, 10, 25, 50 and 100 year return period. Substituting Intensity (mm/hr.) in the above equation

$$It = \frac{Rt}{t} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots 3.7$$

$$It = \frac{R24(b+24)^n}{24(b+t)^n} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots 3.8$$

Using b = 0.3 and n = 0.92 as suggested by ERA manual and results are tabulated for rainfall durations 5,10,15,20,25,30,60,90,120,150,and 180 minutes.

- **Return Period**

Return period, also called recurrence interval is a term commonly used in hydrology. It is the average time interval between the occurrence of storms and floods of a given magnitude. The selection of the design return period depends on economic balance between the costs of periodic repair or replacement of the structure, potential flood hazard to property, expected level of service, budgetary constraints as well as the magnitude and risk associated with damage from larger flood events (ERA,2008). Based on the above criterions for this study 25 year return period was used.

- **Peak Discharge Estimation**

The hydrological study is the basic step that should be done carefully in every flood management and road drainage facilities. The hydrological investigation is dealt with by analyzing rainfall data, high water mark observations, topographical maps, satellite image, and aerial photographs for the estimation of design discharge. The investigation is supplemented by field inspection of the watershed area. It is known that there are many methods used for peak flood calculation, but their applicability depends mainly on the availability of hydrological data.

3.14 Rational Method

Mulvaney developed the rational method, in 1851 for peak flow estimation in small to medium watersheds with no significant storage. It takes into account the steady-state condition to estimate peak runoff discharge resulting from a rainfall event of specific recurrence interval, which is usually in the range of 2-10 years and occasionally up to 25 years (Watt et al., 2003). The rational method has been the most common because it is simple, and also because urban drainage designs typically requires peak discharge data only (Methods and Durrans, 2003).

According to ERA Drainage Design Manual 2013, this method estimates the peak runoff rate for small urban and rural watersheds of less than 50ha. The rational formula estimates the peak rate of runoff at any location in a catchment area as a function of the catchment area; runoff coefficient; and mean rainfall intensity, for duration equal to the time of concentration. The peak runoff given by the following expression:

$$Q_p = 0.00278 * C * I * A \qquad \text{......} 3.10$$

Where:

QP _ Discharge at outlet (m3/s)

C – Rainfall-Runoff Coefficient

I – Maximum probable rainfall Intensity (mm/hr)

A– Catchment Area (hectares)

The storm water drainage outlet usually have been delineated tributary catchment area/ watershed, designated with variable area in the above equation, that contributes runoff to it, the size of which would be easily determined from photogrammetric or topographic survey. The main input variable used in rational method will be; Rainfall intensity, Rainfall duration, Rainfall frequency, Catchment area, Hydrologic abstractions, Runoff Concentration, Run-off Diffusion But, the peak discharge was the product of; Runoff coefficient, Rainfall intensity and Catchment area.

A. Determination of peak discharge

The procedures in rational method to determine peak discharge was conduct by Collecting the necessary information for each sub area or catchment such as; Drainage area, Land use, Soil types (its permeability/highly permeable or impermeable), Distance from the farthest point of the drainage area to the point of discharge, Difference elevation from the farthest point of the drainage area to the point of discharge. The second one is determining the time of concentration for the selected recurrence interval with duration was equal to the time of concentration. The third step is determine the rainfall intensity for the selected return period then Selecting the appropriate runoff coefficient(C) the last step could be Compute the design flow.

((Qp= 0.00278CIA)).

B. Delineation of Catchment Area (A)

The catchment area determined considering land cover with runoff coefficient depending on land types of the study area from topographic maps that contribute to the canal. For large catchment areas, it is necessary to divide the area in to sub-catchment areas to account for major land-use changes, obtain analysis results at different points within the catchment area, and for locating storm water drainage systems and assess their effects on the flood flows.

The catchment of this study area was divided into sub-catchments to make it suitable for the estimation of peak discharge and delineated by using ArcGIS 10.3.1.From the total area of Butajira town, the areas were selected for modeling and runoff estimation. The reason behind this area was more affected by storm water runoff and the problem is very serious than other parts of the town. This area is also more developed and this cause the runoff generated in the study area is with a high rate for the infiltration rate in this area is very less with such an increased present of impervious surface. Particularly, the flooding and drainage problem seems serious around the Erenzaf sub city, o5 kebele/condominium sefer/ and stadium area of the town. For this reason, this study was mainly focused on analysis of the drainage condition in those mentioned areas.

C. Time of Concentration (Tc)

The time of concentration (Tc) used in the Rational Method to determine the critical rainfall duration, which could then be combined with an appropriate rainfall intensity duration frequency (IDF) relation to establish the required design rainfall intensity. Tc is the time required for water to flow from the most remote point of the basin to the location being analyzed (ERA, 2013). Runoff is assumed to reach a peak at the time of concentration Tc when the Entire watershed is contributing to flow at the outlet. The time of concentration to any point in a storm drainage system is the sum of the inlet time (the time it takes for flow from the remotest point to reach the sewer inlet), and the flow time ft. in the upstream sewers connected to the outer point. The velocity of flow depends on the catchment characteristics and slope of the water course by using manning equation. Many empirical equations are available for calculating time of concentration for a watershed (Addis Ababa City Road Authority, 2004).

D. Rainfall Intensity (I)

The rainfall intensity (I) is the average rainfall rate in mm per hour for a particular drainage basin or sub basin the design duration is equal to the time of concentration for the drainage area under consideration. The intensity can be selected on the basis of the design Rainfall duration and return period. Once a particular return period was selected for design and time of concentration calculated for the drainage area, the rainfall intensity can be determined from Rainfall-Intensity-Duration data.

E. Runoff Coefficient

The runoff coefficient is the most important variable in the rational method of rainfall to runoff transformation. A weight age method is employed to obtain the representative runoff coefficient i.e.

the individual areas multiplied by their specific runoff coefficient and their values added together and divided by the cumulative area (Zewdu, 2015).

3.15 Hydraulic and hydrological modelling using SWMM model

Description of SWMM5 model SWMM (U.S. Environmental Protection Agency, 1992) is a comprehensive computer model for simulation of urban runoff quantity and quality in storm and combined sewer systems.

SWMM stands for Storm Water Management Model. All aspects of the urban hydrologic and quality cycles are simulated, including surface runoff, transport through the drainage network, storage and treatment. Like most hydrologic models, SWMM subdivides the overall catchment into sub catchments, predicting runoff from the sub catchments on the basis of their individual properties, and combining their outflows using a flow routing scheme. SWMM can also simulate backwater effects (EPA, 1992).

In SWMM, sub catchments are represented mathematically as spatially lumped, nonlinear reservoirs, and their outflows are routed via the channel/pipe. Sub catchments are subdivided into three subareas, impervious area with and without depression storage and pervious areas with depression storage. Flow from one subarea is not routed over another subarea. Overland flow is generated from each of the three subareas by approximating them as nonlinear reservoirs. This nonlinear reservoir is established by combining the continuity equation with Manning's equation. Infiltration from pervious areas can be computed by either Horton or Green-Ampt equation. Flow routing in channel/pipes is also performed through a nonlinear reservoir by combining the continuity equation with manning's equation (EPA, 1992). Before any analysis work could be conducted, a hydrologic model of the Duck Pond watershed needed to be constructed. No one had previously assembled a detailed model of the entire watershed, and no single database existed that contained all of the necessary information. Data had to be gathered from numerous sources and compiled together into a coherent body of information. The program selected to perform the model SWMM5.

- **Model capability**

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
- 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. (SWMM 5.1 Manual)

Model setup procedure

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

3.16 Urban Runoff Model

The storm water management model (5.1) is a comprehensive computer model for simulation of urban runoff quantity and quality in storm and combined sewer systems from urban areas. The SWMM model has been used in numerous aspects to assess the effects of storm water management based on traditional drainage systems. To apply SWMM in urban areas, depth of depression storage on impervious and pervious area parameters and Manning roughness coefficient has been extracted from the values suggested by ASCE (1992) and McCuen et al (1996) and ASCE (1982) manuals. Manning's roughness coefficients for the impervious area (N-Imperv) is 0.012, the pervious area (N-Perv) is 0.1. To simulate the generation of runoff, other parameters are determined with Arc GIS and from the field survey. The required input data to define the properties of the sub catchments is set by the choice of one hydrological model. Curve Number method and Kinematic Wave are selected for infiltration and routing model. The flow conditions of the drainage systems are performed by solving the Saint Venant equations in many points of the pipes and manholes.

$$\frac{\partial Q}{\partial x}+\frac{\partial A}{\partial t}=0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.11$$

Where: Q= the discharge (m^3/s),

A=the cross sectional area of flow (m^2),

x=the distance along the channel (m), and

t=the (h).

The momentum equation is given by:

$$\frac{1}{g}\frac{\partial V}{\partial t}+\frac{V}{g}\frac{\partial V}{\partial x}+\frac{\partial h}{\partial x}+(S_f - S_o)=0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.12$$

Where: g=acceleration due to gravity (m/s^2),

V=the velocity (m/s),

h=the depth of flow (m),

S_f and S_o=friction and bed slopes (m/m) respectively.

The rainfall is initial abstract in surface ponding, interception by flat roofs and vegetation, and surface wetting, but the remaining is changed to surface runoff. The flow across the sub catchment's surface behaves as if it were uniform flow within a rectangular channel of width; the Manning equation can be used to express the runoff's volumetric flow rate Q (cms) as:

$$Q=\frac{AR^{2/3}\sqrt{S}}{n} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.13$$

Where: -

n -is the surface roughness coefficient, S the average slope of the sub catchment (m/m),

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

R- is the hydraulic radius (m).

- **Sub catchment.**

Based on urban drainage, the sub-catchment was identified. The study area was divided into sub catchment based on Topographic map, building blocks, direction of flow in drainage. The surface runoff, water elevation profile was obtained by simulating the SWMM5. The modeled area was divided in to 9 different regions called as sub-catchments (S). Each sub catchment is designed with storm water lines by providing proper slope at intermediate junctions by connecting with conduits. The overall runoff which was delivered from all the sub- catchments is discharging to outfalls through conduits with required slope. The modeled area of Butajira Town drainage network consists of 12 nodes means 12 junctions and 1outfall. See Figure 19.

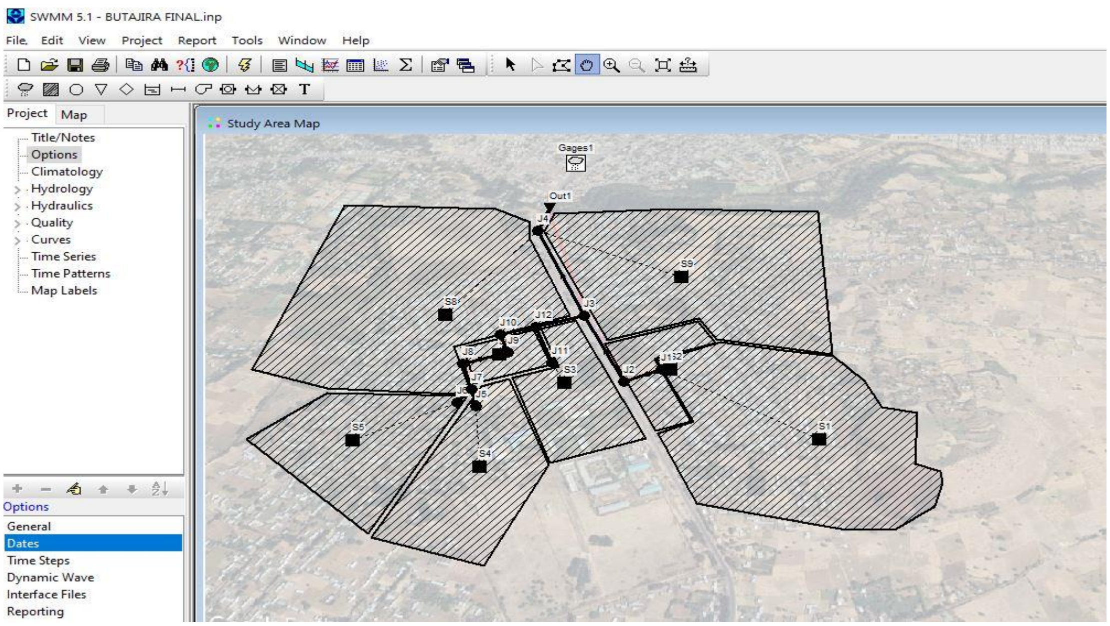


Figure 15: Sub catchments

3.17 Model Calibration and Validation

SWMM model calibration able to predict the observed output with the reasonable accuracy; the sensitivity analysis was perform by changing each parameter while keeping all others constants and observing the changing in model output using the recent SWMM5.1 obtained from the hydraulic model calibration and validation process done for Butajira town. The most parameters used for sensitivity analysis and allowable range of change proposed by (weaver, 2018).

For this study I was used five (5) days rainfall data observed for calibration the model with flow calculating using maximum flow depth (m) record and other left five (5) days also used for validation of the model. The observed data was used in model calibration, a process made by a manual trial and error method and the simulated and the observed runoff in the outlets was compared.

- **Calibration and validation data**

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

Figure 16: The recorded flow depth at site

In the figure above (16), records of flow depth data in open rectangular channel from Butajira Faro, through the main road which go to Worabe and ends in Erinzaf River were collected for the purpose of model calibration. This flow depth was recorded at different interval for 10 days from July to August to calibration of sensitivity parameter and validate SWMM5.1 model for the area.

Table 3: Recorded flow depth for calibration and validation model

Gauged Area	Channel type	Date of measured data	Recorded flow depth(m)
@Condiut3 for Calibration	Rectangular	July22 2022	0.8
		July26 2022	0.75
		July28 2022	0.9
		Aug5 2022	1
		Aug18 2022	1
@Conduit 7 for Validation	Rectangular	Julay20 2022	0.6
		Julay24 2022	0.7
		Julay30 2022	0.75
		Aug12 2022	0.8
		Aug22 2022	0.8

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

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

Where as

Q = flow discharge in m3/s

n = manning roughness coefficient

A= Cross-sectional area in m2 that derived from recorded flow depth (m) with given drainage dimensionless

R (A/P) = hydraulic radius in m

P = wetted parameter in m

S=channel slope in m/m fraction

3.17.1 Criteria of Model performance evaluation

The performance of a model must be evaluated on the extent of its accuracy, consistency and adaptability (Negasa, 2013). Assessing storm rainfall runoff modeling requires subjective and /or objective estimates of the closeness of the simulated behavior of the model to observations. In this thesis work, the model performance in simulating observed from each sub catchment's discharge has been evaluated during calibration and validation through using Nash and Sutcliffe efficiency criteria (NSE) , coefficient of determination (R2) and Relative error (RE). The consistency of the flow checking data is evaluated to calibration and validation process. The model simulation errors were quantified by measuring the difference between observed and simulated hydrographs. Analyzed the results as following equations.

Nash-Sutcliffe efficiency (NSE):- is a measure of efficiency that relates the goodness of fit of the model to the variance of measured data. NSE value between 0.9 and 1 indicate that the model performs very well while values between 0.6 and 0.8 indicate the model performs well (Abeyou, 2008). The NSE efficiency, proposed by Nash and Sutcliffe (Nash, 1970), is defined as:

The Nash Sutcliffe Efficiency(NSE) = 1- $\frac{\sum_{t=1}^{n}(q_t{}^{obs}-q_t{}^{sim})^2}{\sqrt{\sum_{t=1}^{n}(qt^{obs}\ -\ qt^{avgobs})^2}}$………………3.15

Where, q_t^{obs} = is measured discharge,

q_t^{sim} = simulated flow,

q_t^{avrobs} = average of measured flow

Coefficient of determination (R^2):- is defined as the squared value of the coefficient of correlation. Estimated as:

$$R^2 = \frac{\{\sum_{i=1}^{n}(Q_s-\overline{Q}_s)(Q_o-\overline{Q}_o)\}^2}{\{\sum_{i=1}^{n}(Q_s-\overline{Q}_s)^2\}\{\sum_{i=1}^{n}(Q_o-\overline{Q}_o)^2\}}$$

…………………………………3.16

Where, Qo = measured discharge ,

Qs = simulated flow,

$\overline{Q}_o$ = average of measured discharge and

Qs = average of simulated flow

Relative error (RE)

RE= $\frac{\sum_{t=1}^{n} | q_t{}^{obs}-q_t{}^{sim} |}{\sqrt{\sum_{t=1}^{n} q_t{}^{obs}}}$ ………………………..……..3.17

Where

q_t^{obs} and q_t^{avrobs} = are the calculated and average flow respectively,

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

t = at time t and n = is the total number of time steps.

Acceptable level of calibration

NSE value between 0 and 1 shows acceptable model

- IF < 0 indicates poor models,
- = 1 perfect models
- = 0 Model is no better than using as an Estimator

R^2- Approach to one and indicates adequate models. RE < 30%

These three attributes are important in design and analysis of urban drainage systems. Peak discharge is required in urban drainage design for sizing pipes, culvert and bridges. Runoff volume is required for design and operation of flood control structures such as retarding basins.

Time to peak discharge is required for flood forecasting and operation of control structures during storm events. Flow data was calibrated for daily flows (“Eyosaisb, 2018).

3.18 Evaluate alternatives mitigation actions for drainage overflow

Surplus storm water runoff increase in urban drainage areas overflow risks and leads to significant economic losses, traffic disturbances even occur death on community. So that is need to decrease this flood risk from urban area are by improving existing drainage system capacity, evaluation sustainable Urban Drainage System (SUDS), adding best management practices (BMPs) and develop Low Impact Development (LID) techniques in the drainage system (Zhou,2014).

3.19 LID Improvement Systems

The low impact development practices designed to capture surface runoff and provide some combination of detention, infiltration and evapotranspiration to it. They are considered as properties of a given sub catchment (States, 2015). When a user adds a specific type of LID control object to a SWMM project the LID control editor is used the design properties of each relevant layer (such as thickness, void volume, hydraulic conductivity, drain characteristics, etc.). These LID objects can then be placed within selected sub catchments at any desired sizing or areal coverage by editing the sub catchments's LID control property (Eyosias, 20180).

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

3.19.1 LID modeling techniques

The main purpose of LID is to reduce or minimize the altered areas of the post development hydrograph, by reducing the peak discharge rate, volume and duration of flow through the use of site design and storm water quality control measures. The benefits of reduced storm water runoff volume including, reducing pollutant loadings groundwater recharge and evapotranspiration rates. The storm water management model (SWMM) has also widely used to model SUDS through its Low Impact Development (LID) model (Burszta-adamiak and Mrowiec). LID practices are designed to capture surface runoff, providing detention, infiltration, evapotranspiration, or some combination of the three. SWMM LID features are attributes of individual sub-catchments. SWMM allows the user for placing LID control:

- Create a new sub catchment dedicating exclusively to a single LID control; or
- Place one or more LID controls within an existing sub catchment, displacing an equal

Five common types of LID (bio retention cell, vegetative swales, rain barrel and porous pavement and infiltration trenches) are programed in SWMM and are accessed through simple dialog boxes. The LID technologies were programmed using algorithms that already existed in the SWMM engine and generic LID unit is represented by a number of vertical layers (Rossman, 210).

I. Infiltration trenches

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

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

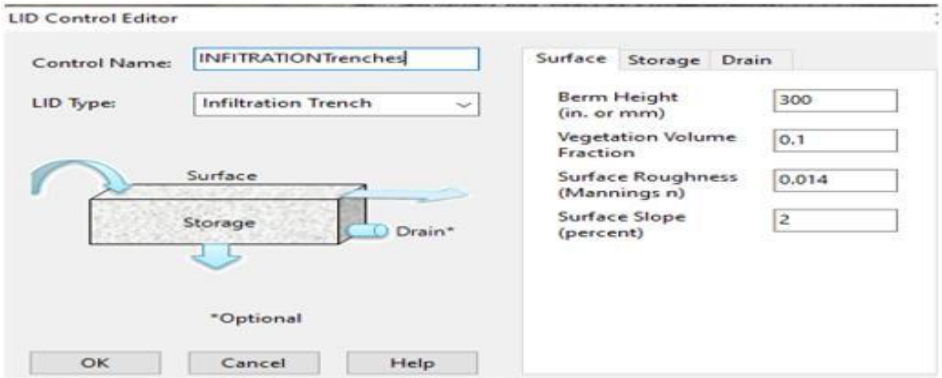


Figure 17: LID editor techniques of infiltration trenches within swmm5.1

II. Bio-retention Cells

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

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

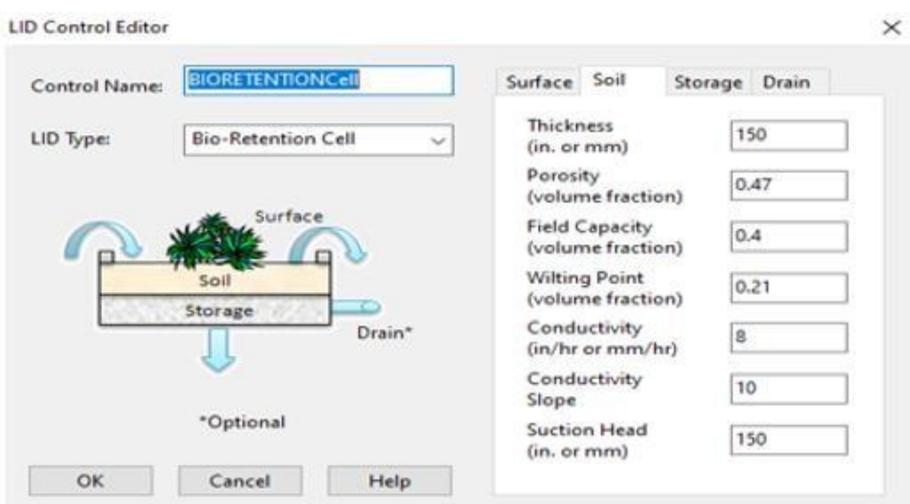


Figure 18: LID editor of bio retention within swmm5.1

This study used LID control editor in Bio-retention soil parameter value shows in figure 18 above are used 150mm soil thickness, 0.47 porosity in volume fraction, 0.4 field capacity in volume fraction, 0.21 wilting point in volume fraction, 8mm/hr. hydraulic conductivity, 10 conductivity slope and 150mm suction head of the soil moisture all value depends on soil texture class of town.

III. Permeable Pavement

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

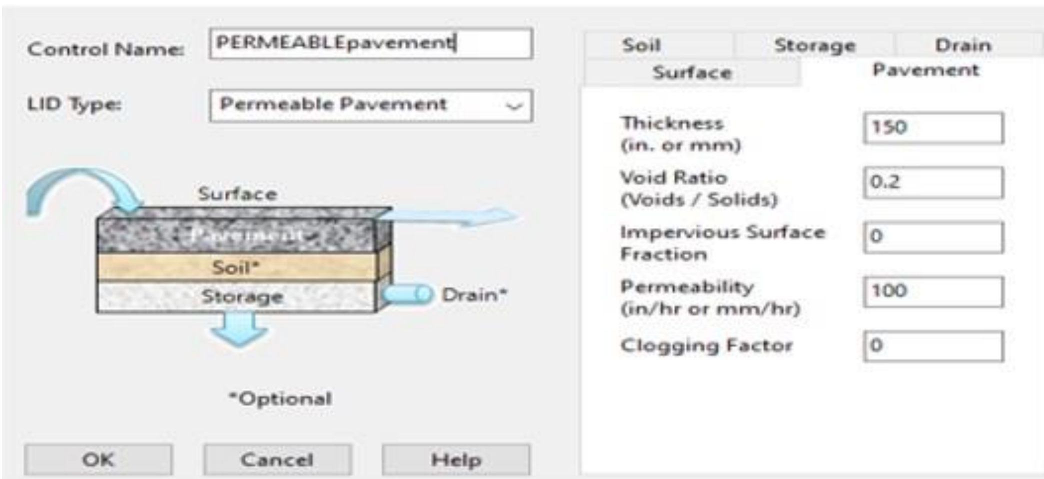


Figure 19: LID editor of permeable pavement within swmm5.1

IV. Vegetative Swales

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

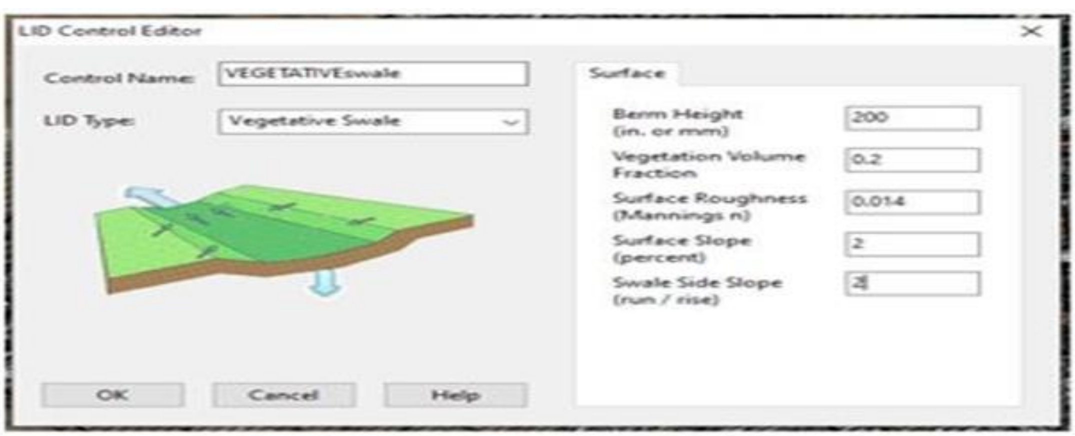


Figure 20: LID Editor of Vegetative Swales with in Swmm5.1

The above figures 17, 18, 19 and 20 are used LID controls editors within SWMM tool for modeling the best alternative mitigation activities to decreasing the drainage overflow of urban drain systems in their gravel storage beds to convey excess captured runoff of the site and prevent the unit from flooding.

Table 4: Designing and Modelling Parameters of LId Types

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

In The Table 4 the designing and modeling of Lid control editor's layer of surface, storage, soil, and pavements parameters for each types of LID developed with the recommended value From SWMM5.1 tool manual are in the drainage system.

4 RESULTS AND DISCUSSIONS

4.1 IDF Curves of Butajira Town

Rainfall intensity duration frequency (IDF) curves are graphical representations of the amount of rainfall that falls within a given period for a specified frequency in a catchment. The intensity duration frequency curve developed from daily rainfall data of 32 years from 1989 to 2020 obtained in Ethiopia meteorological agency rainfall gauge station located in Butajira town. For this study the rainfall intensity obtains from IDF curve is used to SWMM model as input parameter. All the probability distribution functions were compared by chi-square test of goodness of fit and the selecting the function that gave the smallest chi-square value determined the best probability function. From the final result the best fit probability is Pearson distribution and that used to developed IDF curve of Butajira town as shown figure 21.

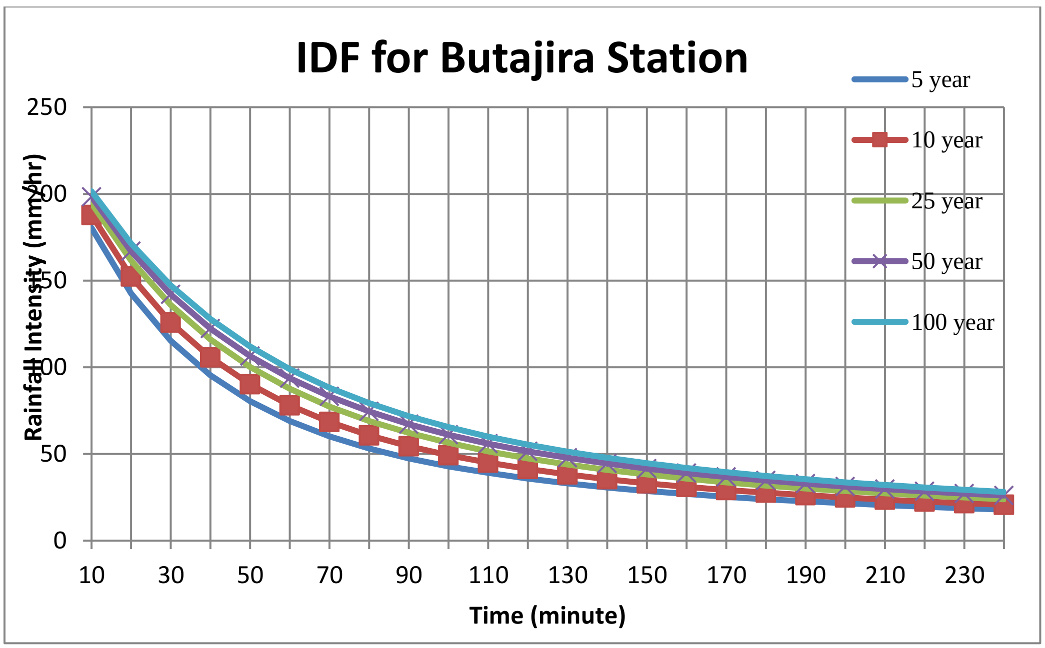


Figure 21: Butajira IDF Curve

4.2 Sensitivity analysis

For model accuracy and successful calibration the most sensitive parameters are selected by changing their value calibration proceed and among the sensitive parameter only three parameters are more sensitive parameter that significantly change the result are used for calibration. The most parameters used for sensitivity analysis and allowable range of change proposed by (Weaver and Nachabe, 2018).

Table 5: Sensitivity parameter used for the calibration of model

No	Parameters	Description	Recommended Value Range	Rank
1	N-Impervious	Manning roughness for impervious area	0.011-0.015	1
	N-Pervious	Manning roughness for pervious area	0.08-0.8	
2	D-store Impervious	Depth of depression storage on storage on impervious area	0-3	3
	D-store Pervious	Depth of depression storage on pervious area	3_10	
3	Smooth concrete roughness	Manning roughness coefficient for open smooth concrete	0.012	2
4	Infiltration Method	Suction	3.5	
		Conductivity	0.5	
		Initial Deficit	0.25-0.26	

4.3 Calibration and validation of the model

Calibration is tuning of model parameters based on checking results against observations to ensure the same response over time (Haleyesus, 2011). The calibration and validation of SWMM model in this study depends on the sensitive parameter value adjusting until to fit the observation is comparable to that achieved in the calibration by using goodness of fit measured with performance simulation model.

4.3.1 Model calibration

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

Table 6: Calibration parameter used for the Calibration of model

Parameters	Description	Recommended Value Range	Initial used proceed Sensitive Parameters	Final used proceed Sensitive Parameters
N-Impervious	Manning roughness for impervious area	0.011-0.015	0.013	0.015
N-Pervious	Manning roughness for pervious area	0.08-0.8	0.3	0.5
D-store Impervious	Depth of depression storage on storage on impervious area	0-3	2	2.5
D-store Pervious	Depth of depression storage on pervious area	3_10	4	7
Smooth concrete roughness	Manning roughness coefficient for open smooth concrete	0.012	0.012	0.012
Infiltration Method	Suction	3.5	3.5	3.5
	Conductivity	0.5	0.5	0.5
	Initial Deficit	0.25-0.26	0.26	0.26

The input parameter's used in the above table for the calibration of model the final output Calibrated results of peak discharge flow rate simulated by swmm5.1 model is 1.904 m^3/s and the observed flow depth at Outfall is calculated peak discharge flow rate by manning equations is 1.76 m^3/s.

4.3.2 Model validation

The validation of this model fit to the observed at the Conduit 7 root is verified the velocity and the discharge flow rate in the drainage systems is listed in appendix 9.

As appendix 9 shown that channel velocity 2.02 m/s and 2.06 m/s are high value. The rectangular open channel type material is constructed from high smooth quality concrete reinforced material.

Table 7: Summary of the model result and observed values

Date of Observed data	Recorded average flow depth (m)	Flow rate using manning equations	Simulated flow rate using SWMM
July 20 2022	0.6	1.14	1.08
July 24 2022	0.7	1.39	1.31
July 30 2022	0.75	1.52	1.41
Aug 12 2022	0.8	1.65	1.52
Aug 22 2022	0.8	1.65	1.52

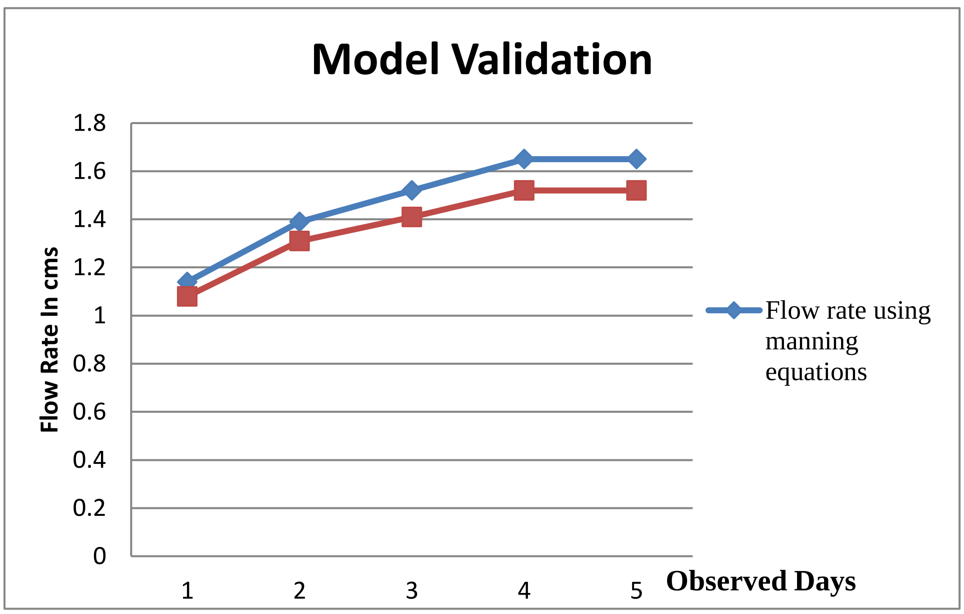


Figure 22: Observed flow rate (calc. flow rate) with Simulated (modeled) flow rate

The validation of the model with the observed flow rate calculated by manning equation and simulated by SWMM 5.1 used at the same site network of channel is validated showed the above figure22.

4.4 SWMM performance evaluation

The Model performance evaluation were quantified by the difference between observed and simulated using method of Coefficient of Determination (R2), Nash-Sutcliffe Efficiency (NSE) and Relative Error (RE) in the detail Evaluation in appendix10.

Overall, the Validation resulted in a good of fit with observed daily flows for the simulation period, with **R2 = 0.98**, **NSE = 0.87** and relative error **(RE) =19 % .**This calibration and validation result indicated that the model structure and parameters matched the runoff producing pattern and the calibrated model was suitable for simulating runoff in the study area.

4.5 Simulated flow (using SWMM 5.1)

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

The total sub catchment of the study area is generating runoff from daily precipitation taken as extreme events from each year from 1989-2020 rainfall data. The project of sub catchment used as input for drainage systems connect through each nodes and simulate runoff in the network system of swmm5.1 model in all outfalls.

- **Spatial Setting, Land Use and Land Cover map**

Land uses in Butajira towns are defined as spatially specific different organized human activities in an urban context. Like many other middle level Ethiopian towns and urban centers, Butajira existing settlement pattern also significantly proportioned built-up area uses as its major urban activity as shown in the figure 8.

4.6 Urban runoff

Different case scenarios have been considered in this study to obtain a fully understanding of the system performance under multiple working conditions. Firstly, the model had been run with the continuous rainfall events to analyze the current performance, as shown in table 8.

Table 8 : Peak runoff at each sub catchment

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	Total Runoff mm	Total Runoff 10^6 ltr	Peak Runoff CMS	Runoff Coeff
S1	95.50	0.00	0.00	18.30	63.42	22.77	2.36	0.664
S2	95.50	0.00	0.00	15.25	73.02	4.02	0.43	0.765
S3	95.50	0.00	0.00	19.83	67.64	4.13	0.45	0.708
S4	95.50	0.00	0.00	21.35	60.89	8.10	0.85	0.638
S5	95.50	0.00	0.00	15.25	72.47	11.02	1.19	0.759
S6	95.50	0.00	0.00	12.20	80.40	2.51	0.28	0.842
S8	95.50	0.00	0.00	21.35	53.46	32.99	3.28	0.560
S9	95.50	0.00	0.00	18.30	63.40	20.92	2.17	0.664

Moreover, different local improvements in the system nearby the properties which have been flooded in previous years were modeled and analyzed. The above table showed the calculation of peak runoff for the sub catchments which are the most runoff producers. Those sub catchments are the main source of the runoff to become the drainage network flooded.

4.7 Network simulation

The drainage systems modeled to cope with a 25 years return period rainfall in terms of water level below the surface in drainage systems. This implies that the flooding risk must be verified in the nodes (manholes) in the systems, whereas water level at each manhole is checked independently with a longitudinal profile in drainage systems which are connected. The general network performance is determined by maximum water flow production. The water elevation profile in the manhole was over flooded as shown in the figures (23) below.

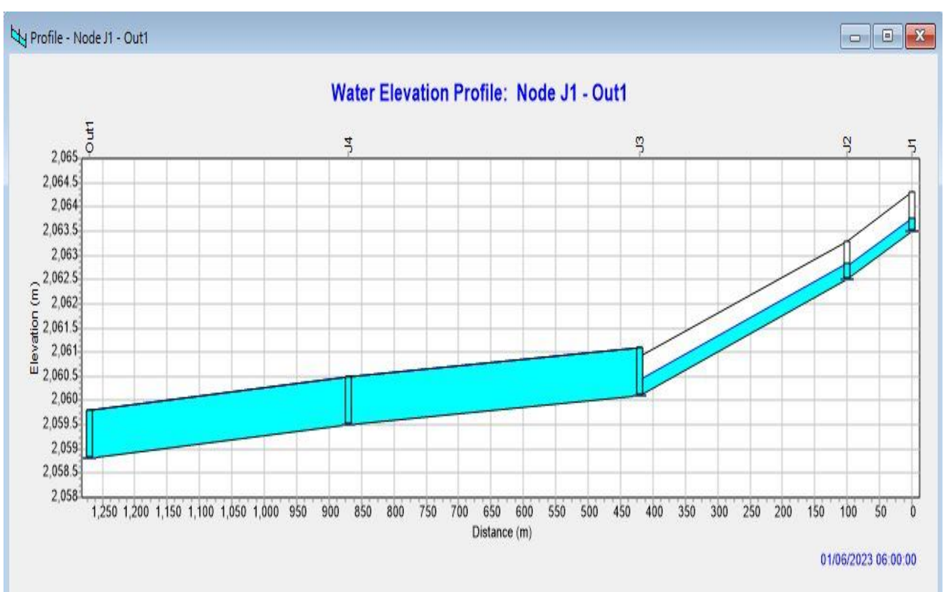


Figure 23: Water elevation profiles of the drainage network sections Node J1-Out1

As is seen in the figure 23, water elevation profile: Node J1-Out1 chain age from 450m-1250m the downstream study area had a severe flooding that means (J3 and J4) are also insufficient for holding high runoff. The upstream part of the drainage channels as simulation result indicates the junction (J1 and J2) are sufficient to carry the generated runoff and there is no flooding as shown figure 23 above.

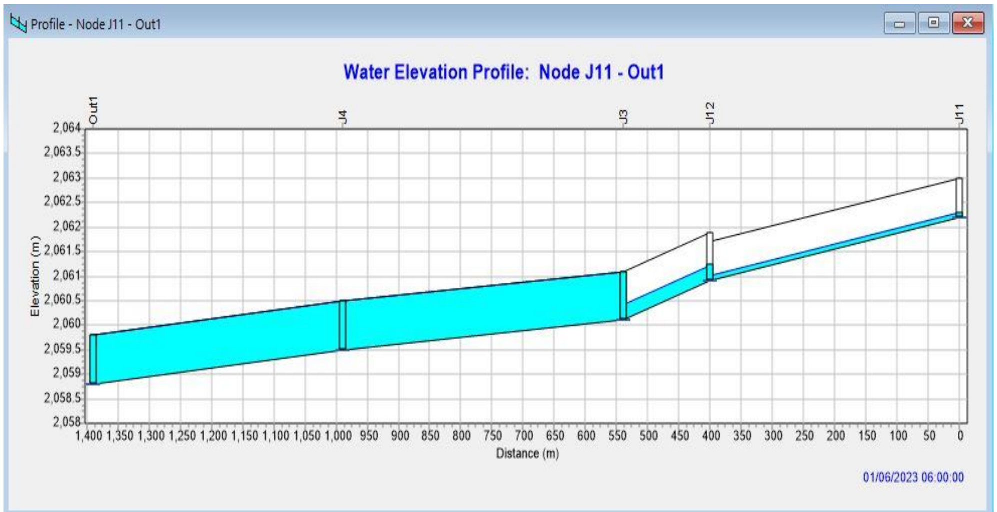


Figure 24: Water elevation profiles of the drainage network sections Node J11-Out1

Also Figure 24 indicate water elevation profile: Node J11-Out1 chain age from 550m-1400m the drainage canal found upstream part of channel as simulation result indicate the junction (J11 and J12) are sufficient to carry the generated runoff . But the downstream of the link junctions are insufficient. That means flooding from overtopping and lack of sufficient dimension of drainage structures capacity and entering waste materials in to diches that occur overtopping and flooding in to the area**.**

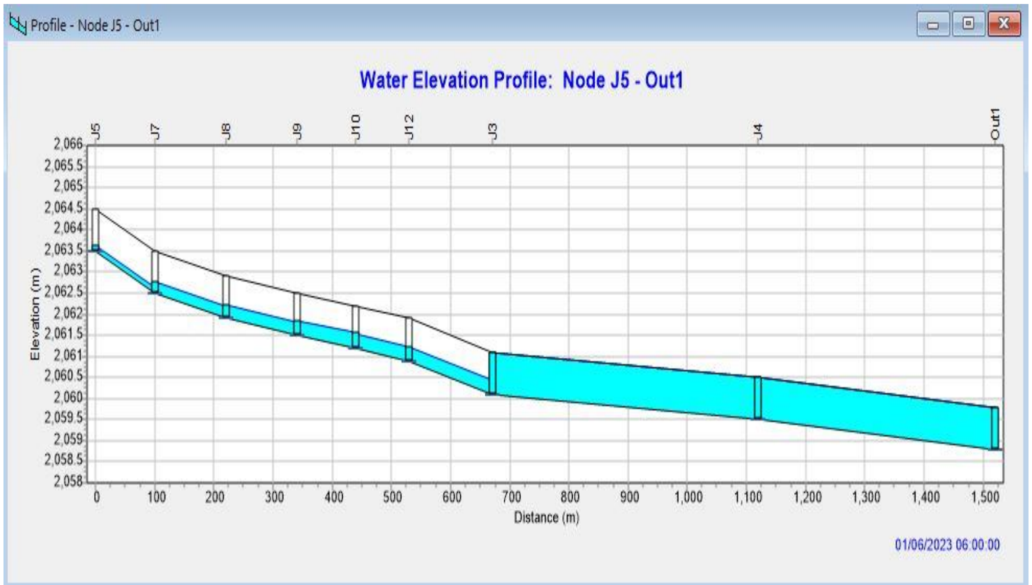


Figure 25: Water elevation profiles of the drainage network sections Node J5-Out1

As is seen in the above figure (25), water elevation profile: Node J5-Out1 chain age from 660m-1500m the simulation result indicates that both junctions (J3 and J4) are flooded due to insufficient design water depth. Thus, the channel is busy at this time and overflow of runoff has happened.

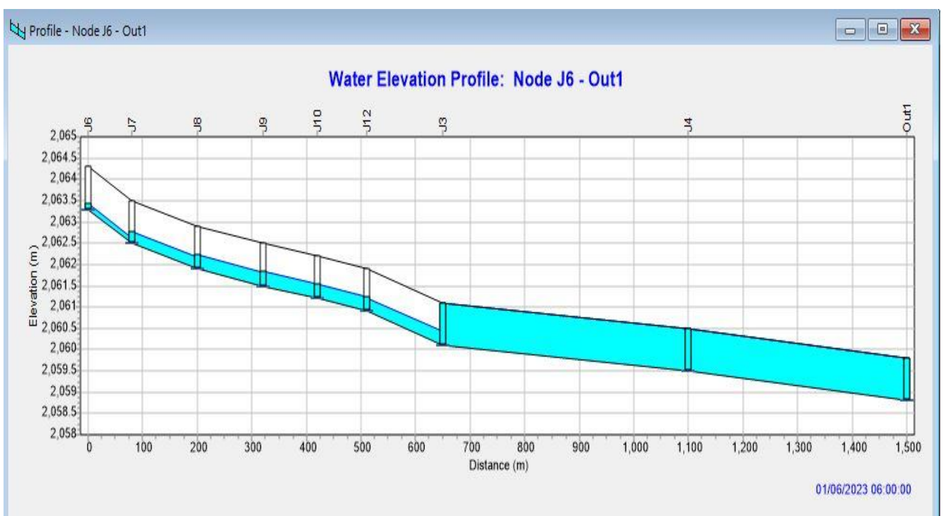


Figure 26**: Water elevation profiles of the drainage network sections Node J6-Out1**

Figure 26, show that water elevation profile: Node J6-Out1 chain age from 670m-1500m the study area had a severe flooding from overtopping and lack of sufficient drainage structures capacity in which the street and nearby houses are flooded. In this area, the houses, roads and residential suffered from flooding and excess runoff.

We conclude that, problem of sizing of all drainage systems by the same dimension, lack of connected drainage system inside the rode, lack of LID control system, problem of miss management of diches and lack of proper maintenance of drainage system on time are the main cause during heavy rainy season this also occur large impact in Community. And also total average flow 1.904 m3 /s and total maximum peak flow to outfall 2.011 m3 /s were occurred from all 9 sub catchment as shown in appendix11.

- **Flooded Junctions and conduits of Butajira town in Erinzaf sub city**

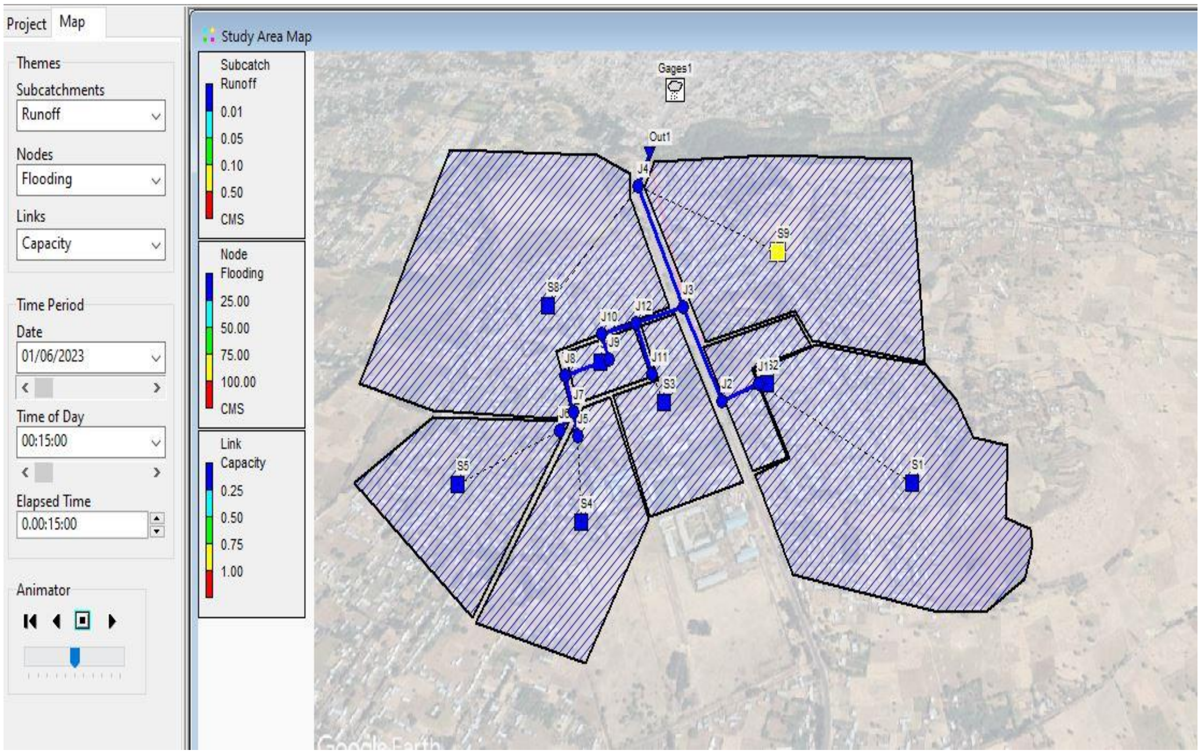


Figure 27: Flooded Junctions and conduits of Butajira town @ time of 00:15:00

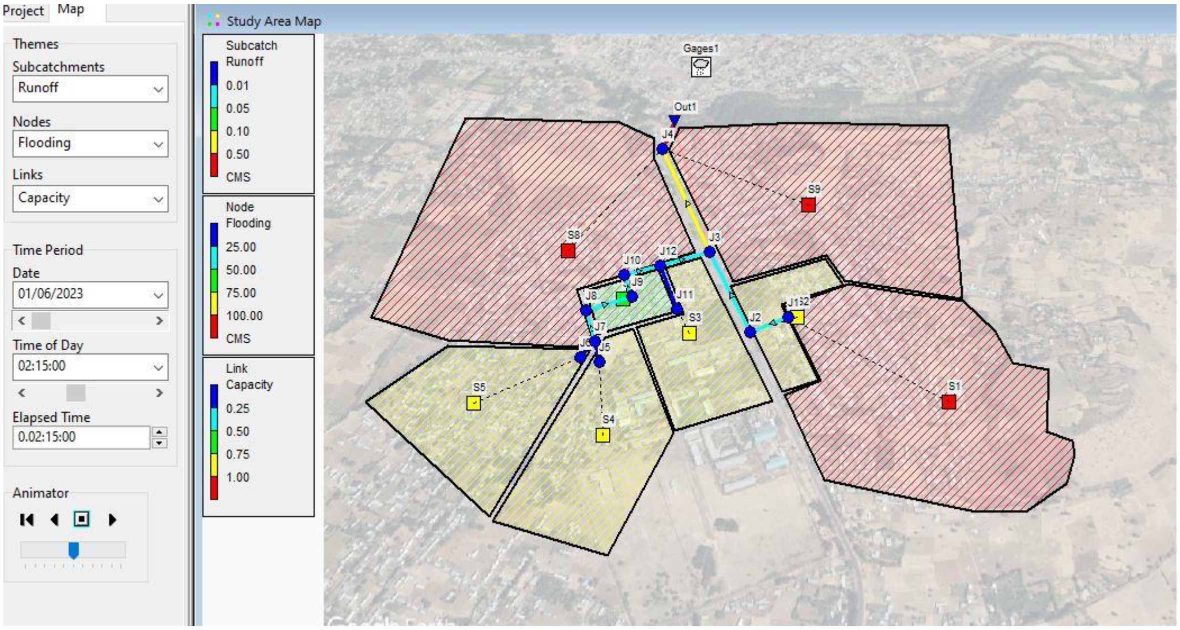


Figure 28: Flooded Junctions and conduits of Butajira town @ time of 02:15:00

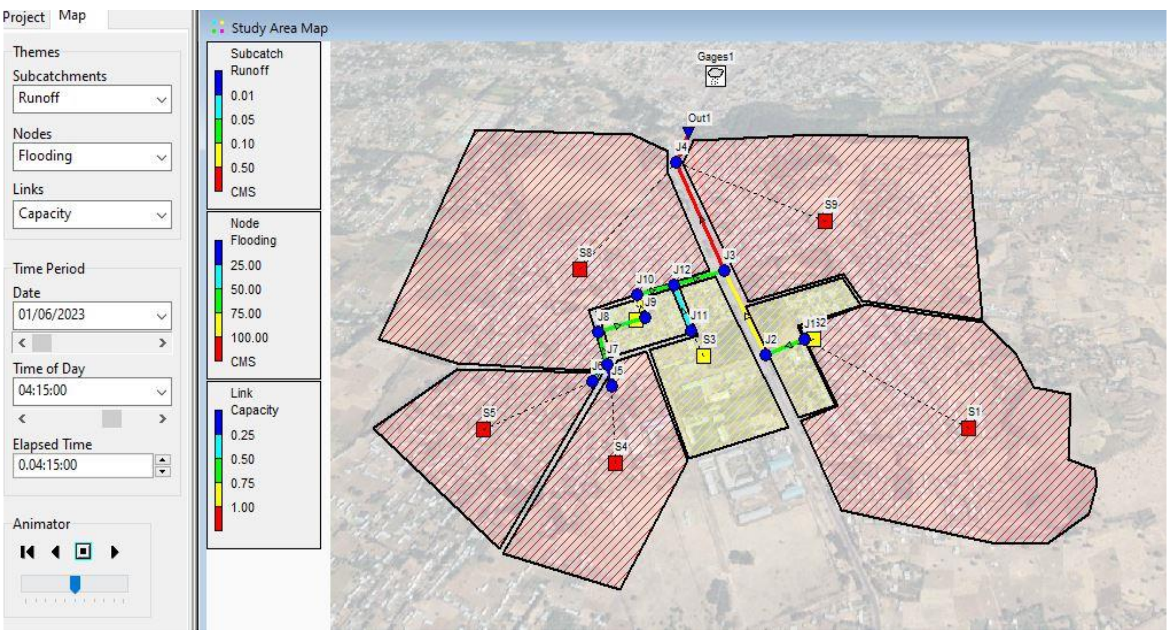


Figure 29: Flooded Junctions and conduits of Butajira town @ time of 04:15:00

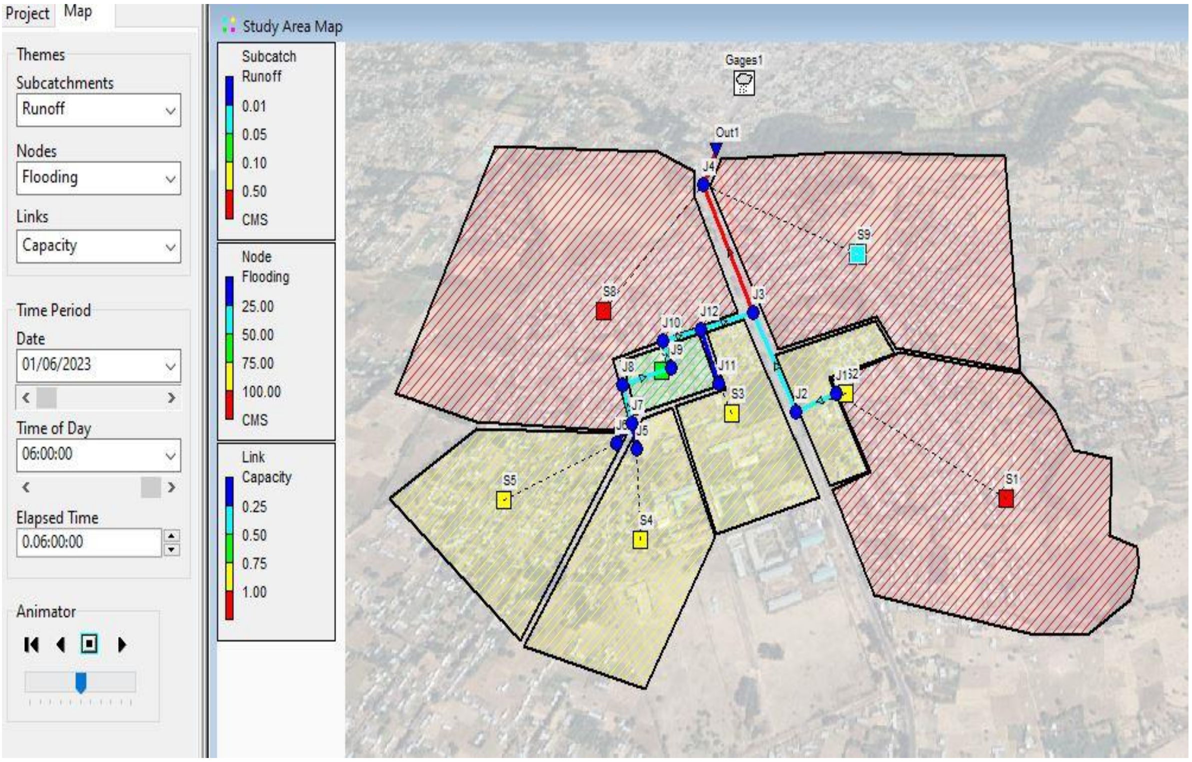


Figure 30: Flooded Junctions and conduits of Butajira town @ time of 06:00:00

As is seen in the above figures 27, 28, 29 and 30, the study area had a severe flooding from overtopping and lack of sufficient drainage structures in which the street and nearby houses are flooded. In this area, the houses, roads and stadiums suffered from flooding. When we conclude that, the flooding is much due to under sizing and lack of sufficient drainage structures, the drainage open channel and uniform slopes of the consecutive nodes, accumulation of silt along the channel, and due to unmanageable entrance of solid waste materials to the manhole.

Therefore, the catchment result shows that mitigation measure require to minimize and solve `the problem of runoff that directly enter in to the channel should be controlled by LID ,resizing of drainage ditches and close follow up of constructed ditches to protect from dry disposal.

Table 9: Water flow profile of flooded junctions

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 Volume 1000 m3
J3	3.81	3.763	0	04:02	24.008	0.000
J4	4.24	5.184	0	04:00	46.663	0.000

The water elevation profile and tabulated is obtained for the junction in between J3 and J4 to outlet 1 as shown in the table 9 above. The simulation status shows that sections between these junctions are the most surcharged (flooded). That means the performance of J3 and J4 which deliver downstream of the channel under capacitated and upstream, right and lest channels at the inlet are the reasons for flooding and overtopping. Upstream inlet connections should be avoided during re designing and other new junctions and intake sub main channels shall be provided to dispose the storm smoothly in appropriate nearby revers after treating the storm before entering in to the rivers. This shows small number of junction points are provided in the existing system and over connection of junction points result short life span of structures running with damage and rehabilitation .This defines that existing drainage structures are not follow scientific procedure and exposed redundant financial planning for reconstruction.

Generally: sizing, location of junction points, treating and nearby disposal of storm water by drainage channels identified as main target point for detailed design of the town drainage structure.

4.8 The alternative measures of drainage system management

The best practice to reduce peak runoff by developing sustainable urban drainage system (SUDS). Like, Built-up proper drainage structure, by improving good management practices of drainage system used, on time maintenance of structure and developing LID control system are the main measures for good drainage system. The LID techniques are capable of mitigating the impact of imperviousness on both horology and water quality of urban storm water runoff. In other words, the key principle of LID measures is to ensure that new urban developments do not make the existing hydrologic regime flashier and increase flooding in the catchment. Advantage of LID techniques are for runoff volume reduction, infiltration improvement, peak flow reduction, extending the lag time, pollutant loads reduction and baseflow increase.

In this study the targets of reducing peak runoff and runoff volume quantity using different Techniques of the LID practices and compared with the existing condition using SWMM5.1 results.

4.9 LID Development with SWMM 5.1

The LID technique results for infiltration trenches, bio retention, Permeable pavement, and vegetative swales are compared the value with and without LID technique in SWMM5.1 as shown appendix 12 modeling drainage system network.

As appendix12 shown that the effect of respective measures to mitigate the flood problem. For selecting the best alternative to evaluate the runoff decrease in the out fall projects, the comparison was done at the outlets of the project on the sub catchment S1,S2,S3,S4,S5,S6,S8 and S9.

It was found that the flood is reduced at the total outlet 1 by total outfall volume by **26.82% and peak runoff** 7.23% using Bio-retention Cell**,** 24.22% total outfall and peak runoff 4.26**%** using infiltration trench, **24.51% total outfall and peak runoff** 6.25**%** using Permeable pavement and also on the same sub catchment reduced total volume outfall 1 by 2.02 % using vegetable swales.

Previous studies also investigated the impact of other types of LID controls on urban runoff, flooding and water quality.

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

Davis (2008) observed that bio-retention can reduce the peak flow from 66% to 44% depending on site conditions such as soil and slope with sustainably delayed time- to- peak. Therefore, this two scenario what options may be acceptable in Butajira town drainage system. The result confirmed that LID could replace the conventional stormwater management system. Based on this, this study select the best alternative of LID control to evaluate the Butajira town drainage system problem mitigation measures is solved using the acceptable LID control types. **Bio-retention Cell and Permeable pavement** are the most recommended mitigation measures out of the tested LID (in table 10).

Table 10: summary results of swmm5.1 with and without lid

Outfall name	Before LID use total volume of runoff	After LID control used Bio-retention Cell and Permeable pavement	
		Value	total volume of runoff Reduced by%
outfall total volume (103)m3	33.96	25.75	26.28

At ends of results in table 10 the total volume of runoff in outlet is reduced by 26.28 percentage volume and outfall before LID control develop in the drainage system total volume of runoff 33.96*10^3 m^3. After LID control developed in the drainage systems used Bio-retention Cell and Permeable pavement in the sub catchments the first total volume results reduced to 25.75*10^3 m^3 which is a flood reduction by 26.28%.

5. CONCLUSION AND RECOMMENDATION

5.1 Conclusion

This thesis assessed and modeled urban drainage system performance on Butajira town using Swmm5.1 has totally identified the critical problem with the existing drainage systems. It also to assess the hydraulic performance of storm water drainage system of study area and also evaluated the alternative for mitigation measures to drainage problem. The performance of hydraulic and hydrology of drainage was simulated by Swmm with and without LID. For calibration and validation a 10 days of observed flow depth were used to be compared with the model result. For calibration the 5 days flow depth data recorded is calculated the flow by the manning equation is 1.76m^3/s was compared peak discharge flow rate simulated by swmm5.1 model is 1.904 m^3/s .The other 5 days flow depth data used for validation without changed the sensitive parameters of model results parallel with observed are verified. The result show Swmm5.1 model performance for certified area was tested by goodness of fit using the coefficient of determination (R^2) =0.98, the Nash –Sutcliffe coefficient (NSE) =0.87, and Relative error (RE) =19% which are simulated values indicated in an acceptable range. So that the SWMM5.1 model is powerful application for assessed and modeled urban drainage system performance and also controls the overflow drainage infrastructure for this study.

The area simulated in this study subdivide to sub catchment is 173.83ha, the combined drainage system are 12 Junction with total length canal flow routed through 2.42km. According to simulated result greater than 25% of drainage infrastructure are over flooded, From the result of SWMM5.1 simulated all total sub catchment average flow rate 1.904m^3/sec, maximum flow rate 2.011m^3/sec and total volume of all outfall 33.95*10^3 $m^{3.}$

In this study the LID control used to reduce the peak runoff overflow. The selected best alternative on the outlet of the project in the same sub catchments S1, S2, S3, S4, S5, S6, S8, and S9 were reduced the total runoff at outlet by 26.86% using bio-retention Cell, 24.20% infiltration trench, 24.51% permeable pavement and 2.02 % vegetable swales. From all select highest value of runoff decreased by Bio-retention Cell and permeable pavement for total study applied and after selected two LID control types to the total volume of outfall is reduced from 33.96*10^3 m^3 to 25.75*10^3 m^3 by 26.28% from the study. Finally, the Butajira town urban drainage system performance infrastructure's over flow assessed and controlled for the problem occurred the usage of the excellent alternative mitigation measures by improved LID control for minimizing runoff and have aesthetic values to improve environmental attraction.

5.2 Recommendation

Based on analysis of this study the following appropriate mitigation measures are recommended;

1. The existing drainage network not properly connected to natural drainage carryover the flood produced so, the natural drainage channels flood caring capacity should identified properly and the existing artificial drainage network shall be reconnected to formulate appropriate flood flow.

2. Creating awareness for community concerned the effect of disposing solid materials in to the drainage infrastructure by concerned body and it is better to document organized data in softcopy and hardcopy of design document of storm drainage system of the town.

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

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

5. At J3 and J4 runoff contribution for the town is high, so to avoid this problem well construct, maintained drainage and develop the LID to minimize the effect.

6. Appling this research study results for future plans of the urban drainage systems in Butajira town in order to standardize all sub- catchment of drainage networks in a sustainable way.

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

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Year	Yi	KN	2.5911	Year	Yi	Upper Limit	Lower Limit
1989	52.10	XH	144.973	1989	52.10	144.97	17.36
1990	90.00	XL	17.360	1990	90.00	144.97	17.36
1991	43.50			1991	43.50	144.97	17.36
1992	90.00			1992	90.00	144.97	17.36
1993	90.20			1993	90.20	144.97	17.36
1994	50.20			1994	50.20	144.97	17.36
1995	54.00			1995	54.00	144.97	17.36
1996	51.30			1996	51.30	144.97	17.36
1997	38.00			1997	38.00	144.97	17.36
1998	75.00			1998	75.00	144.97	17.36
1999	46.60			1999	46.60	144.97	17.36
2000	62.10			2000	62.10	144.97	17.36
2001	60.00			2001	60.00	144.97	17.36
2002	65.00			2002	65.00	144.97	17.36
2003	47.30			2003	47.30	144.97	17.36
2004	67.20			2004	67.20	144.97	17.36
2005	91.30			2005	91.30	144.97	17.36
2006	48.90			2006	48.90	144.97	17.36
2007	41.80			2007	41.80	144.97	17.36
2008	108.20			2008	108.20	144.97	17.36
2009	58.70			2009	58.70	144.97	17.36
2010	28.7			2010	28.700	144.97	17.36
2011	50			2011	50.000	144.97	17.36
2012	35.5			2012	35.500	144.97	17.36
2013	40.5			2013	40.500	144.97	17.36
2014	50.6			2014	50.600	144.97	17.36
2015	39			2015	39.000	144.97	17.36
2016	28.2			2016	28.200	144.97	17.36
2017	36.8			2017	36.8	144.97	17.36
2018	23			2018	23	144.97	17.36

2019	54			2019	54	144.97	17.36
2020	55			2020	55	144.97	17.36

Appendix

Appendix 1 Outlier test

Appendix 2: Butajira IDF

		5 year	10 year	25 year	50 year	100 year
	24hr RF	71.78	82.88	95.55	104.21	112.31
	0.5hr MRF	0.804	0.759	0.712	0.682	0.655
10	0.167	180.37	187.76	194.50	198.32	201.45
20	0.333	142.60	152.32	161.51	166.87	171.34
30	0.500	115.37	125.79	135.98	142.07	147.23
40	0.667	95.38	105.67	116.01	122.33	127.77
50	0.833	80.42	90.17	100.22	106.49	111.95
60	1.000	69.01	78.06	87.60	93.65	98.98
70	1.167	60.15	68.47	77.40	83.14	88.25
80	1.333	53.13	60.76	69.06	74.46	79.32
90	1.500	47.49	54.48	62.17	67.23	71.81
100	1.667	42.88	49.30	56.42	61.15	65.45
110	1.833	39.05	44.96	51.57	55.98	60.03
120	2.000	35.84	41.30	47.45	51.57	55.36
130	2.167	33.10	38.17	43.90	47.76	51.32
140	2.333	30.75	35.48	40.83	44.45	47.80
150	2.500	28.70	33.13	38.15	41.55	44.71
160	2.667	26.91	31.07	35.79	38.99	41.97
170	2.833	25.33	29.24	33.70	36.72	39.54
180	3.000	23.92	27.62	31.83	34.70	37.37
190	3.167	22.67	26.17	30.16	32.88	35.42
200	3.333	21.53	24.86	28.66	31.25	33.66
210	3.500	20.51	23.68	27.30	29.76	32.07
220	3.667	19.58	22.60	26.06	28.41	30.62
230	3.833	18.72	21.62	24.93	27.18	29.29

240	4.000	17.94	20.72	23.89	26.05	28.07

Appendex-3 Statistical analysis

	Year	Yi	logYi	Rank	$(Yi-\bar{Y})^2$	$(Yi-\bar{Y})^3$	$(Yi-\bar{Y})^4$	$(logYi-\bar{Y})^2$	$(logYi-\bar{Y})^3$	Fi	1-Fi	$(1-Fi)^2$	$(1-Fi)^3$	(1-Fi)Yi	$(1-Fi)^2Yi$	$(1-Fi)^3Yi$		Normal	Log
	1989	52.10	3.95	3.00	4.88	-10.78	23.83	0.00	0.00	0.08	0.92	0.84	0.77	47.79	43.83	40.20	N	32.00	32.00
	1990	90.00	4.50	4.00	1273.82	45463.46	1622619.21	0.34	0.20	0.11	0.89	0.78	0.70	79.73	70.64	62.58	$\bar{Y}$	54.31	3.92
	1991	43.50	3.77	5.00	116.84	-1263.00	13652.19	0.02	0.00	0.15	0.85	0.73	0.62	37.18	31.78	27.16	m1	54.31	3.92
	1992	90.00	4.50	6.00	1273.82	45463.46	1622619.21	0.34	0.20	0.18	0.82	0.68	0.56	74.11	61.02	50.25	m2	467.55	0.17
	1993	90.20	4.50	7.00	1288.14	46232.04	1659296.84	0.34	0.20	0.21	0.79	0.63	0.50	71.46	56.61	44.84	m3	7257.96	-0.03
	1994	50.20	3.92	8.00	16.89	-69.39	285.17	0.00	0.00	0.24	0.76	0.58	0.44	38.20	29.07	22.12	m4	750500.94	-0.66
	1995	54.00	3.99	9.00	0.10	-0.03	0.01	0.01	0.00	0.27	0.73	0.53	0.39	39.40	28.75	20.98	σ	21.62	0.41
	1996	16.50	2.80	10.00	1429.55	-54050.35	2043609.88	1.24	-1.38	0.30	0.70	0.49	0.34	11.52	8.05	5.62	Cv	0.40	0.04
	1997	38.00	3.64	11.00	266.00	-4338.22	70753.72	0.08	-0.02	0.33	0.67	0.45	0.30	25.35	16.92	11.29	Cs	0.49	-0.47
	1998	75.00	4.32	12.00	428.10	8857.70	183271.29	0.16	0.07	0.36	0.64	0.40	0.26	47.70	30.33	19.29	Ck	3.43	
	1999	46.60	3.84	13.00	59.43	-458.20	3532.46	0.01	0.00	0.40	0.60	0.37	0.22	28.18	17.04	10.30	α_0	54.31	
	2000	62.10	4.13	14.00	60.69	472.84	3683.74	0.05	0.01	0.43	0.57	0.33	0.19	35.61	20.42	11.71	α_1	25.70	
	2001	60.00	4.09	15.00	32.38	184.28	1048.67	0.03	0.01	0.46	0.54	0.29	0.16	32.53	17.64	9.56	α_2	16.47	
	2002	65.00	4.17	16.00	114.29	1221.83	13062.08	0.07	0.02	0.49	0.51	0.26	0.13	33.21	16.97	8.67	α_3	11.69	
	2003	47.30	3.86	17.00	49.13	-344.38	2413.89	0.00	0.00	0.52	0.48	0.23	0.11	22.69	10.88	5.22	λ_1	54.31	
	2004	67.20	4.21	18.00	166.17	2142.01	27611.87	0.09	0.02	0.55	0.45	0.20	0.09	30.14	13.51	6.06	λ_2	2.91	
	2005	91.30	4.51	19.00	1368.31	50614.51	1872262.23	0.36	0.21	0.58	0.42	0.17	0.07	38.09	15.89	6.63	λ_3	-1.07	
	2006	48.90	3.89	20.00	29.26	-158.29	856.23	0.00	0.00	0.61	0.39	0.15	0.06	18.87	7.28	2.81	λ_4	6.27	
	2007	41.80	3.73	21.00	156.48	-1957.52	24487.39	0.03	-0.01	0.65	0.35	0.13	0.04	14.83	5.26	1.87	τ	0.05	
	2008	####	4.68	22.00	2904.20	156509.12	8434374.52	0.59	0.45	0.68	0.32	0.10	0.03	35.00	11.32	3.66	τ_3	0.03	
	2009	58.70	4.07	23.00	19.28	84.64	371.63	0.02	0.00	0.71	0.29	0.09	0.02	17.15	5.01	1.46	τ_4	0.18	
	2010	28.70	3.36	24.00	655.84	-16795.65	430126.22	0.31	-0.17	0.74	0.26	0.07	0.02	7.49	1.95	0.51	Y_{min}	16.50	
	2011	50.00	3.91	25.00	18.57	-80.03	344.87	0.00	0.00	0.77	0.23	0.05	0.01	11.48	2.64	0.61			
	2012	35.50	3.57	26.00	353.79	-6654.62	125169.20	0.12	-0.04	0.80	0.20	0.04	0.01	7.04	1.40	0.28			
	2013	40.50	3.70	27.00	190.70	-2633.43	36366.05	0.05	-0.01	0.83	0.17	0.03	0.00	6.77	1.13	0.19			
	2014	50.60	3.92	28.00	13.76	-51.04	189.32	0.00	0.00	0.86	0.14	0.02	0.00	6.88	0.94	0.13			
	2015	39.00	3.66	29.00	234.38	-3588.16	54932.56	0.06	-0.02	0.90	0.10	0.01	0.00	4.08	0.43	0.04			
	2016	28.20	3.34	30.00	681.70	-17798.75	464714.16	0.33	-0.19	0.93	0.07	0.01	0.00	2.07	0.15	0.01			
	2017	36.80	3.61	31.00	306.58	-5367.99	93990.20	0.10	-0.03	0.96	0.04	0.00	0.00	1.55	0.07	0.00			
	2018	23.00	3.14	32.00	980.28	-30691.86	960942.92	0.61	-0.47	0.99	0.01	0.00	0.00	0.25	0.00	0.00			
	2019	54.00	3.99	33.00	0.10	-0.03	0.01	0.01	0.00	1.02	-0.02	0.00	0.00	-1.10	0.02	0.00			
	2020	55.00	4.01	34.00	0.48	0.33	0.23	0.01	0.00	1.05	-0.05	0.00	0.00	-2.84	0.15	-0.01			

Appendix 4 : Analysis of statistical parameters

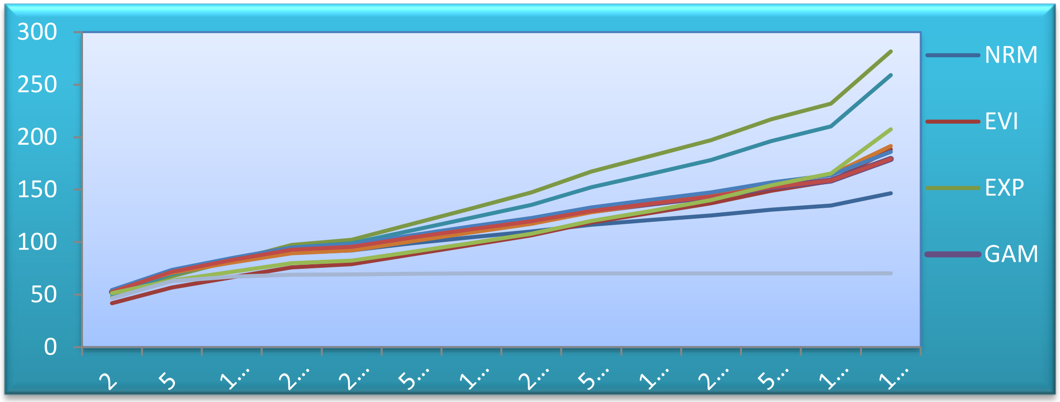
					EV1	E(2)	LN(2)		G2		LoG	GPAR		UT/LT			
T	F	P	W	U_T	KT	KT	KT	δT	KT	δKT/δCs	KT	δx/δα	δx/δκ	KT	q	Ku	KL
2	0.5	0.5	1.1774	-0.0000001	-0.1643	-0.3069	-0.18825	0.898340024	-0.0812	-0.1633	0.00000	0.4623	-9.7728	0.082299	-0.07781	4.195142	-0.42486
5	0.8	0.2	1.7941	0.8414567	0.7195	0.6094	0.69712	1.573704924	0.8079	-0.0878	0.76430	0.6893	-26.9934	0.855602	0.64747	38.81467	0.382096
10	0.9	0.1	2.1460	1.2817288	1.3046	1.3026	1.29724	2.169948768	1.3215	0.0547	1.21139	0.7501	-35.2581	1.216442	1.395147	55.14804	0.579479
20	0.95	0.05	2.4477	1.6452114	1.8658	1.9957	1.88109	2.780930079	1.7718	0.2301	1.62335	0.7756	-39.9645	1.492756	2.143735	67.66023	0.725748
25	0.96	0.04	2.5373	1.7510765	2.0438	2.2189	2.06814	2.980038747	1.9075	0.2914	1.75215	0.7800	-40.9514	1.569662	2.379252	71.14312	0.766048
50	0.98	0.02	2.7971	2.0541886	2.5923	2.9120	2.65079	3.606576686	2.3076	0.4934	2.14567	0.7880	-42.9582	1.78116	3.087946	80.72207	0.876245
100	0.99	0.01	3.0349	2.3267853	3.1367	3.6052	3.24028	4.246823284	2.6823	0.7094	2.53342	0.7914	-43.9615	1.960595	3.759347	88.84937	0.969184
200	0.995	0.005	3.2552	2.5762361	3.6791	4.2983	3.84052	4.902748526	3.0378	0.9365	2.91835	0.7928	-44.4495	2.116095	4.393272	95.89289	1.049422
500	0.998	0.002	3.5255	2.8785061	4.3947	5.2146	4.65503	5.79681743	3.4849	1.2508	3.42519	0.7935	-44.7267	2.293687	5.176416	103.9374	1.14079
1000	0.999	0.001	3.7169	3.0905222	4.9355	5.9078	5.28958	6.495440657	3.8095	1.4975	3.80789	0.7937	-44.8120	2.411355	5.730047	109.2675	1.2012
2000	0.9995	0.0005	3.8989	3.2907605	5.4762	6.6009	5.94165	7.214646577	4.1245	1.7509	4.19032	0.7937	-44.8514	2.517395	6.252694	114.0711	1.255567
5000	0.9998	0.0002	4.1273	3.5402445	6.1907	7.5172	6.8326	8.198827077	4.5285	2.0951	4.69566	0.7938	-44.8728	2.642761	6.8996	119.7501	1.319764
10000	0.9999	0.0001	4.2919	3.7191243	6.7312	8.2103	7.52983	8.969892819	4.8262	2.3617	5.07787	0.7938	-44.8792	2.728143	7.358178	123.6179	1.363443
100000	0.99999	0.00001	4.7985	4.2648446	8.5266	10.5129	10.0017	11.70739491	5.7767	3.2811	6.34740	0.7938	-44.8840	2.966232	8.713946	134.4036	1.485088

C_0	C_1	C_2	d_1	d_2	d_3	b_1	b_2	b_3	b_4	b_5	b_0	b_2	b_4	b_6	b_8	b_{10}
2.515517	0.802853	0.010328	1.432788	0.18927	0.00131	0.3194	-0.35656	1.7815	-1.8213	1.3303	2.50524	1.2831	0.22647	0.1306469	0.02025	0.00391

w LN(3)m	0.987	B	103.076	β	0.9	P (UT/LT)	0.043657	var(a)	440.07
z LN(3)	0.009	C	190.939605	α	0.05	k (UT/LT)	0.081752	var(k)	0.23
B	6.838	w(Ln(3))	0.9870	wa	2.44775	Z (Ln(2)	0.398141	cov(a,k)	9.88
C	0.261			za	1.64521	u(Ln(3))	-14.0029		
A	0.214			Z(Ln(3))	0.16208	1/b (WEI)	0.447878		

Appendix 5: Quantile test statistics for the most commonly used distributions

	T	L_T	NRM	EVI	EXP	GAM	LN2	LOG	LN3	PE3	LP3	GPAR	U_T
	2	45.12	54.31	41.81	47.67	52.55	50.46	54.31	54.30	52.55	51.34	46.68	145.02
	5	62.57	72.50	56.71	67.49	**71.78**	69.68	70.84	73.65	71.78	63.46	62.84	893.59
	10	66.84	82.02	66.57	82.47	82.88	82.50	80.50	84.88	82.88	71.73	67.18	1246.77
	20	70.00	89.88	76.03	97.46	92.62	94.84	89.41	94.76	92.62	79.85	68.99	1517.31
	25	70.87	92.17	79.04	102.29	**95.55**	98.77	92.20	97.75	95.55	82.48	69.31	1592.62
	50	**73.26**	**98.73**	**88.28**	**117.28**	**104.21**	**110.95**	**100.70**	**106.60**	**104.21**	**90.73**	**69.88**	**1799.75**
	100	75.27	104.62	97.46	132.26	112.31	123.18	109.09	114.93	112.31	99.21	70.12	1975.48
	200	77.00	110.01	106.61	147.25	119.99	135.55	117.41	122.88	119.99	107.98	70.22	2127.78
	500	78.98	116.55	118.67	167.06	129.66	152.21	128.37	132.96	129.66	120.13	70.27	2301.73
	1000	80.28	121.14	127.79	182.05	136.68	165.11	136.65	140.32	136.68	129.80	70.28	2416.98
	2000	81.46	125.46	136.90	197.04	143.49	178.29	144.92	147.51	143.49	139.92	70.28	2520.85
	5000	82.85	130.86	148.95	216.85	152.23	196.20	155.84	156.80	152.23	154.06	70.29	2643.64
	10000	83.79	134.73	158.06	231.84	158.67	210.13	164.11	163.69	158.67	165.39	70.29	2727.28
	100000	86.42	146.53	188.33	281.63	179.22	259.06	191.56	186.00	179.22	207.45	70.29	2960.49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EXP
GAM

Appendix 6: Reduction Daily precipitations for Each Time

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

Whereas $R_t = \frac{t}{24}\frac{(b+24)^n}{(b+t)^n} * R_{24}$= Rain fall Redaction formula n = 0.9 and b = 0.3 is content

Appendix 7:Calculated value for calibration and validation

Gauged Area	Channel Type	Date of Measured Data	Recorded Depth	Manning Roughness used (n)	Channel Slope (S)	Side Slope H/V(m)	Width (m)	Area (m2)	Parameter (m)	Hydraulic Radius (m)	Velocity (m/s)	Discharge Q (m3/s) $Q = \frac{1}{n} * A *$
@ Conduit 3 for Calibration	RECTANGULAR	July 22 2021	0.8	0.013	0.003		1	0.8	2.6	0.31	1.92	1.54
		July 26 2021	0.75	0.013	0.003		1	0.75	2.5	0.30	1.89	1.42
		July 28 2021	0.9	0.013	0.003		1	0.9	2.8	0.32	1.98	1.78
		Aug 5 2021	1	0.013	0.003		1	1	3	0.33	2.03	2.03
		Aug 18 2021	1	0.013	0.003		1	1	3	0.33	2.03	2.03
@ Conduit 7 for Validation		July 20 2021	0.6	0.014	0.004		1	0.6	2.2	0.27	1.90	1.14
		July 24 2021	0.7	0.014	0.004		1	0.7	2.4	0.29	1.99	1.39
		July 30 2021	0.75	0.014	0.004		1	0.75	2.5	0.30	2.02	1.52
		Aug 12 2021	0.8	0.014	0.004		1	0.8	2.6	0.31	2.06	1.65
		Aug 22 2021	0.8	0.014	0.004		1	0.8	2.6	0.31	2.06	1.65

Appendix 8: Maximum annual daily Rainfall of Butajira station

Annual Daily Rainfall Max.	
Year	daily max Rf
1989	23
1990	36.8
1991	52.1
1992	90
1993	43.5
1994	90
1995	90.2
1996	50.2
1997	54
1998	16.5
1999	38
2000	75
2001	46.6
2002	62.1
2003	60
2004	65
2005	47.3
2006	67.2
2007	91.3
2008	48.9
2009	41.8
2010	108.2
2011	58.7
2012	28.7
2013	50

2014	35.5
2015	40.5
2016	50.6
2017	39
2018	28.2
2019	54
2020	54

Appendix 9 : Determined velocity and flow rate for model Validation

Gauged Area	Channel Type	Date of Measured Data	Recorded Depth	Manning Roughness used (n)	Channel Slope (S)	Width (m)	Area (m2)	Perimeter (m)	Hydraulic Radium (m)	Velocity (m/s)	Discharge Q (m3/s)
@ Conduit 7 for Validation	Rectangular	July 20 2022	0.6	0.014	0.004	1	0.6	2.2	0.27	1.90	1.14
		July24,2022	0.7	0.014	0.004	1	0.7	2.4	0.29	1.99	1.39
		July 30 2022	0.75	0.014	0.004	1	0.75	2.5	0.30	2.02	1.52
		Aug 12 2022	0.8	0.014	0.004	1	0.8	2.6	0.31	2.06	1.65
		Aug 222022	0.8	0.014	0.004	1	0.8	2.6	0.31	2.06	1.65

Appendix 10: Correctness function of the model performance

Recoeded average flow depth (m)	Simulated flow rate using SWMM	Calculated (Observed) flow rate using manning equations	qtob -qtavrob	qtsim-qtavrsim	(q1ob -q1avrob)*(q1sim-q1avrsim)	(qtob -qtavrob)^2	(qtsim-qtavrsim)^2	(qtob -qtsim)^2	|qtob -qtsim|
0.6	1.08	1.14	-0.33	-0.29	0.09	0.11	0.08	0.00	0.06
0.7	1.31	1.39	-0.08	-0.06	0.00	0.01	0.00	0.01	0.08
0.75	1.41	1.52	0.05	0.04	0.00	0.00	0.00	0.01	0.11
0.8	1.52	1.65	0.18	0.15	0.03	0.03	0.02	0.02	0.13
0.8	1.52	1.65	0.18	0.15	0.03	0.03	0.02	0.02	0.13
Total	**6.84**	**7.34**			**0.16**	**0.18**	**0.13**	**0.05**	**0.50**
Average	**1.368**	**1.47**							

Coefficient of determination (R2) = $(0.16/0.163)^2$ = 0.98

Nash-Sutcliffe Efficiency (NSE) = 0.87

Relative Error (RE) = 0.19

Appendix 11: Total average flow and total maximum peak flow at each outlet

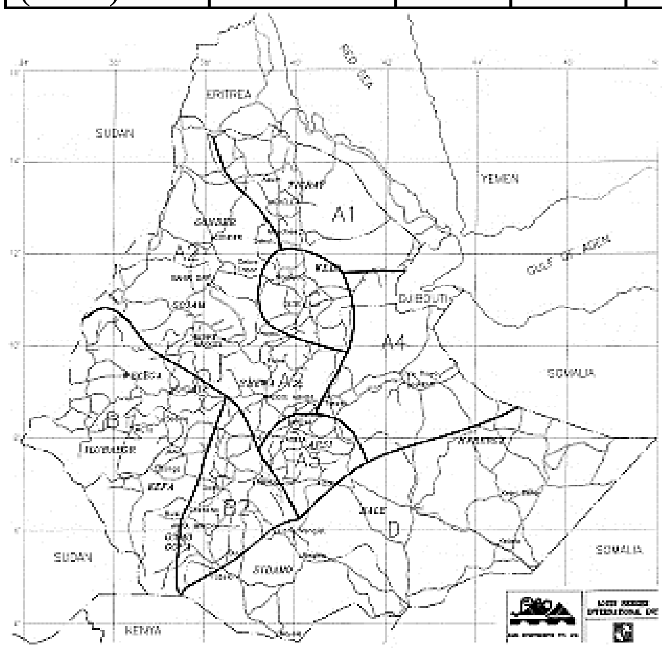
Summary Results

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

Outfall Node	Flow Freq. Pcnt.	Avg. Flow CMS	Max. Flow CMS	Total Volume 10^6 ltr
Out1	82.64	1.904	2.011	33.957

Appendix 12 : SWMM5.1 Results without and with LID

Item	swmm5.1 results without LID (Current condition)	Low Impact Development							
		Infiltration trench		Bio-retention Cell		Permeable pavement		vegetable swales	
		value	%	Value	%	Value	%	Value	%
Peak runoff (m^3/s)	2.011	1.96	4.26	1.82	7.23	1.85	6.25	1.98	0.87
Total volume runoff at outlet one (10^3m^3)	33.95	24.22	24.76	31.95	**26.82**	26.56	**24.51**	31.2	2.02



Figur 1. Rainfall regions (source: ERA drainage manual)