Evaluation of the Existing stormwater drainage System, in the Case of Bishoftu City, Ethiopia

Marga Mosisa Galata

A Thesis Submitted to Department of Water Resources Engineering

College of Civil Engineering and Architecture

Presented in Fulfilment of the Requirements for the Degree of Master of Science in Water Resources Engineering

(Specialization in Water Supply and Environmental Engineering)

Office of Graduate Studies

Adama Science and Technology University

November 2025

Adama, Ethiopia

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Declaration

I hereby declare that this Thesis entitled, "Evaluation of the Existing stormwater drainage System, in the Case of Bishoftu City, Ethiopia" is my original work. That is, it has not been submitted for the purpose of obtaining any academic degree, diploma, or certificate at any other university. All sources of materials utilized in this thesis have been properly acknowledged through suitable citations.

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Recommendation

I, the supervisor of this thesis, hereby confirm that I have examined and revised the thesis titled, "Evaluation of the Existing stormwater drainage System, in the Case of Bishoftu City, Ethiopia", which was prepared under my supervision by Marga Mosisa Galata. This thesis has been submitted in partial fulfillment of the requirements for the Degree of Master of Science in Water Resources Engineering (Specialization in water Supply and Environmental Engineering). Therefore, I recommend the submission of the thesis to the department for further review and defense.

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Approval page for M.SC thesis

I hereby certify that the recommendations and suggestions made by the board of examiners are appropriately incorporated into the final version of the thesis entitled "Evaluation of the Existing stormwater drainage System, in the Case of Bishoftu City, Ethiopia" by Marga Mosisa Galata.

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Major Advisor	Signature	Date

We, the undersigned, members of the Board of Examiners of the thesis open defense by Marga Mosisa Galata, have read and evaluated the thesis entitled, "Evaluation of the Existing stormwater drainage System, in the Case of Bishoftu City, Ethiopia" and examined the candidate during open defense. This is, therefore, to certify that the thesis is accepted for partial fulfillment of the requirements of the Degree of Master of Science in Water Resources Engineering (Specialization in water Supply and Environmental Engineering)

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LIST OF ACRONYMS

AASHTO	American Association of State Highway and Transportation Officials
BMPs	Best Management Practices
BOD	Biochemical Oxygen Demand
CAD	Computer Aided Design
CMS	Cubic Meter per Second
CN	Curve Number
COD	Chemical Oxygen Demand
CSO	combined sewer overflow
DEM	Digital Elevation Model
DGC	Department Graduate Council
EPA	Environmental Protection Agency
ERA	Ethiopian Road Administration
ESRI	Environmental System Research Institute
EWRA	European Water Resources Association
GEV	Generalized Extreme Value
GI	Green Infrastructure
GIS	Geographical information system
GPS	Geographical Positioning System
HGL	Hydraulic Grade Line
IDF	Intensity Duration Frequency
IDW	Inverse Distance Weighting
IGGI	Integrated Green-Gray Infrastructure
IQR	Interquartile range
IWRM	Integrated Water Resources Management
LID	Low Impact Development Low Impact Development
LULC	Land use land cover

MAR	Missing at Random
MCAR	Missing Completely at Random
MNAR	Missing Not at Random
MoTL	Ministry of Transportation and Logistic
NRCS	Natural Resources Conservation Services
OPGS	Office of Postgraduate Studies
SGC	School Graduate Committee
SUDS	Sustainable Urban Drainage System
SWMM	Storm Water Management Model
TDS	Total Dissolved Solids
TR-55	Technical Release 55
WSUD	Water Sensitive Urban Design

ABSTRACT

Urban stormwater management is a key issue in line with Global problems of urbanization and climate change. Effective management of urban stormwater is essential for alleviating the negative effects of urbanization on the natural hydrological cycle and fostering sustainable urban development. The main aim of this study is to evaluate the current stormwater drainage system from both hydraulic and hydrologic points of view. This assessment seeks to investigate the cause of the flood problem in the study area through site investigation and collecting stormwater drainage structures data, land cover data, soil data, and Digital Elevation Model (DEM) to investigate the actual stormwater-related flooding problem in the study area. The methods utilized to accomplish this goal generally, executing literature reviews of existing studies and data, measuring the dimensions of the existing drainage channels, rainfall data analysis, and soil, DEM, and land cover data analysis. Storm Water Management Model (SWMM) 5.2.4 was implemented for the drainage network modelling, with the rational method employed for comparative analysis. The simulated study area consists of 45 sub-catchments with 695 a total area, 69 junctions, 69conduits, 3 outfalls, and 1 rain gauge station. The simulation indicates that the total runoff from the entire area using SWMM is 97.34 m3/sec, while the rational method yields a runoff of 90.87 m 3/sec. and *that 46% of the nodes of the study area are flooded, and 46% of the conduits of the drainage system are surcharged. Accordingly, the overall study result shows that under the current rainfall conditions, the system responded with serious problems and was not able to drain the generated runoff. Therefore, it is essential to understand the hydrologic pattern of the area and its impact on the present functionality of the drainage system in order to develop effective mitigation strategies.*

Keywords: Bishoftu city, Evaluating, Existing, Modelling, SWMM, and Stormwater Drainage System

1.0. INTRODUCTION

1.1.Background

Urban stormwater management is a key issue in line with Global problems of urbanization and climate change, and flooding events are common and widespread natural hazards, frequently resulting in significant socioeconomic losses and environmental consequences (Zhang et al., 2021). In urban settings, the effects can be especially severe, stemming from both the direct destruction caused by the inundation of properties and essential infrastructure and the indirect repercussions, including diminished productivity and lost business opportunities (Cea & Costabile, 2022).

Stormwater drainage networks in urban areas are an essential infrastructure to drain water and control flooding in urban watersheds. The effectiveness of urban stormwater drainage networks is practically linked with the precipitation traits of the urban region of interest, particularly the intensity and amount of precipitation (Kumar et al., 2022). Effective management of urban stormwater is essential for alleviating the negative effects of urbanization on the natural hydrological cycle and fostering sustainable urban development. The benefits of a well-functioning urban stormwater drainage system are diverse, including environmental safeguarding, improved public health and safety, economic advantages, and enhanced urban beauty and resilience (Karamoutsou et al., 2024).

In Ethiopia, the issues of flooding are evident in both urban and rural areas. Because of flooding associated many individuals have been displaced, properties have been destroyed, and human lives have been lost. According to a report by the World Health Organization (2024), over 560 000 individuals in Ethiopia have been impacted by the severe rains and flooding that occurred in April and early May across various districts of the Country. Also, according to Tiwari, (2016.), Ethiopia cities at large, are troubled with storm water leading into floods especially during the rainy season due to inadequate installation of desired infrastructure.

Numerous studies have been conducted in the country regarding flooding issues caused by urban stormwater, yet very few have been carried out in Bishoftu city. Tiwari, (2016) in his study entitled, “urban infrastructure research reviews of Ethiopian cities” identified that urban floods could be managed or mitigated through adequate gray infrastructure provision including proper drainage along with roads (preferably in both sides), sustainable handling of wastewater and storm water through separate handling, and reducing point and non-point water pollution to enhance storm water quality. In addition, among the conducted studies in the city, significant gaps remain in addressing the actual challenges posed by stormwater-related flooding in Bishoftu city. According to a study entitled “Evaluation on the Integration of Road Network and Drainage System for the Occurrence of Flood” carried out by, the current flooding problem of the Bishoftu city only attributed to inadequate provision of stormwater drainage infrastructure with corresponding to the existing roads in the city, and disposal of solid waste into stormwater drainage facilities. Also, another study entitled, “The nature of existing road network in Bishoftu city for the occurrence of flood hazard” by Ameya et al. (2022), poor road network and drainage system integration are the primary causes of the city's flooding.

However, according to the US Dept of Transportation (2018), urban stormwater management presents a complex challenge that necessitates a multifaceted strategy. Although the establishment of sufficient gray infrastructure, which encompasses appropriate drainage systems and roadways, as well as the sustainable management of wastewater and stormwater through separate systems, and the reduction of both point and non-point source water pollution to improve stormwater quality are essential elements, they alone are inadequate for effective urban stormwater management (Manandhar et al., 2023). An effective system must combine these components with green infrastructure, policy initiatives, and community involvement to effectively tackle the evolving dynamics of urban hydrology and the impacts of climate change.

The drawbacks of depending exclusively on gray infrastructure become evident when one considers the rising frequency and severity of extreme weather events. Traditional gray infrastructure, although crucial for transporting large quantities of water, can become overwhelmed during intense rainfall, resulting in flooding, damage to infrastructure, and deterioration of water quality (Lopez-Pajares, 2019).

The primary aim of assessing and modelling the current stormwater drainage systems in Bishoftu city is to analyze the capacity and performance of the existing drainage infrastructure, recognize its shortcomings in managing stormwater runoff, and suggest viable solutions for enhancement and sustainable management to alleviate flooding and associated environmental and infrastructural harm. This aim generally entails a comprehensive approach, combining hydrological and hydraulic modelling techniques.

1.2.Statement of the problem

As a result of the city`s flat topography which is surrounded by hills and slops, exposure to flood and runoff water is very common. Every year during the rainy season flood overflows the drainage structures and causes extensive destruction to public facilities and individual property in many parts of the city. According to the infrastructure department, Kebele 02, and 01, which are part of the study area are among the flooding affected areas of the city

In the study area, even though loss of human life and serious injuries has not been reported, the destruction of homes and property is common during rainfall season. According to the city municipal office, hundreds of houses are flooded every year in various areas of the city including in the designated study area causing serious financial and psychological problems to the affected families.

Past and recent flooding in Bishoftu city had resulted in financial and economic costs for households. For instance, the families who have suffered damages to their properties or house buildings, lost foodstuff and crops could incur significant financial costs to rehabilitate the damages and to replace lost property.

Most of the hills are located on the southern side of the city, these high ground formations of Sofa, Baru, Bubissa, Dhibaayu, and Wulabbo recharge flood that rush down towards the study area and causes damage to property. The plain area found in Kebele 02 is one of the flood-prone areas. In addition, the dawn stream plain areas of Cheleleka are also one susceptible to flooding.

During rainfall along the main asphalt road, the road segment found between Lafto Health Station and Sunshine is flooded by stormwater runoff due to absence of appropriate longitudinal stormwater drainage at this site; consequently, it is difficult to cross the main asphalt road. In addition to the absence of the longitudinal stormwater drainage facility, the stormwater runoff comes from the relatively high elevation of the south direction to the centre or the place where the main asphalt road found with relatively higher velocity than the velocity by which it drains from the site. As a result, the stormwater runoff accumulated and inundated the road surface at this site consequently, the road pavement is found damaged, and seen when frequently maintained. At some place in the study area the existing drainage system is frequently inadequate, poorly maintained, or entirely absent in some regions, resulting in the buildup of surface water.

Therefore, in this research it has been recognized that there is a significant lack of a thorough and current evaluation of the capacity and performance of the existing stormwater drainage system in the designated study area. However, without a comprehensive understanding of the system's present condition, including its hydraulic features, bottlenecks, and failure points, effective planning and intervention are greatly hindered. So, this study focuses on the performance evaluation of the capacity of the existing stormwater drainage facilities in the designated study area

1.3. Research objectives

1.3.1. General objective

The general objective of this study is to investigate the existing stormwater drainage system in Bishoftu city

1.3.2. Specific objectives

- To assess the hydraulic performance of existing drainage systems of the city;
- To assess factors associated with stormwater drainage management systems; and
- To identify and map the flooding affected drainage areas of the city.

1.4. Research questions

- What is the hydraulic performance of existing drainage systems of the city?
- What are the major challenges in managing the drainage system in the study area? and
- Which drainage area of the city is flooding affected?

1.5. Scope of the study

Given the vast catchment area of Bishoftu city, it is not feasible to assess the entire city from both financial and temporal perspectives to resolve the existing stormwater drainage challenges. As a result, several critical flood-prone regions have been pinpointed through discussions with local officials and on-site evaluations. The specified study area is situated within the previous limits of the city, particularly before the addition of Dukem sub-city. This region stretches along the primary asphalt road from Addis Ababa via Gelan and Dukem, covering the southern section of the city adjacent to the roadway from Kurkura Health centre to Ziquala Square. From an administrative standpoint, this study area is part of the Dhaka Bora Woreda within Dhibayu Sub-city.

However, since this thesis is to fulfil academic requirements, the time available for the research is insufficient to cover all areas of Bishoftu city. Additionally, because of the absence of funding for the study, the researcher cannot personally bear the costs. In assessing and modelling the stormwater drainage system, it is crucial to evaluate the quality of stormwater runoff to determine its effect on the quality of receiving water bodies. However, due to budget constraints, water quality parameters also could not be included in the research. Consequently, this study is confined to evaluating and modelling the capacity of the current stormwater drainage system in the designated area.

1.6. Limitation of the study

The evaluation of the performance of the current urban stormwater drainage system in Bishoftu city encountered various limitations. Mainly, these limitations were related to data availability and quality, and the inherent complexities associated with urban hydrological systems. A notable limitation was the precision and completeness of the input data. SWMM needs comprehensive information regarding topography, land use, soil properties, rainfall statistics, and the physical attributes of the drainage network, such as channel dimensions, lengths, slopes, invert elevations, and manhole placement.

Another significant limitation was from the simplifications and assumptions that are intrinsic to the SWMM model itself. Although SWMM is a robust tool, it depends on specific hydrological and hydraulic equations that simplify real-world processes. Additionally, it employs empirical equations for infiltration and runoff routing, which may not accurately depict the intricate interactions among soil, vegetation, and urban surfaces. Successful calibration requires observed flow and water level data from the existing drainage system, which entirely lacking in Bishoftu city. The performance of the existing drainage system is assessed based on present conditions; however, future urbanization and shifts in rainfall patterns due to climate change could substantially affect the system's efficiency, rendering the current evaluation less applicable over time

1.7. Significance of the study

One of the primary functions of stormwater drainage systems is to mitigate flood risks. Performance evaluation allows the identification of weaknesses in the existing systems, such as bottlenecks or inadequate capacity during extreme weather events. By using models to assess these vulnerabilities, the city administration can implement improvements to reduce the likelihood of flooding, thereby protecting lives and property.

The result of this study or performance evaluation through modelling allows engineers to design more efficient drainage systems by identifying optimal configurations and materials. By simulating different design alternatives, stakeholders can compare their effectiveness in managing stormwater, thus leading to cost-effective solutions that meet regulatory requirements while minimizing environmental impacts.

In the city since the built-up area is exponentially increasing, evaluating the performance of drainage systems helps identify potential flooding hotspots and assess the adequacy of existing infrastructure. This proactive approach enables the municipality of the city to implement necessary upgrades or maintenance before severe weather events occur, thereby reducing economic losses and enhancing public safety

As climate change continues to alter precipitation patterns, evaluating the resilience of Urban Drainage systems becomes increasingly important. Performance assessments help identify vulnerabilities within existing infrastructure and inform adaptive management strategies that enhance system robustness against future climate scenarios. For instance, incorporating climate projections into modelling efforts allows for better planning and investment in sustainable drainage solutions.

Scientific evaluations provide a foundation for policymakers when creating regulations related to stormwater management. By presenting empirical data on system performance, researchers can advocate for necessary changes in legislation aimed at improving infrastructure resilience and environmental protection standards.

CHAPTER 2: LITERATURE REVIEW

2.1. Hydrologic cycle and urbanization

Urbanization interferes with and disturbs the pattern of natural hydrologic cycle. In natural land area, water from rainfall in naturally balanced way changed to infiltration, runoff, evaporation, evapotranspiration, interception, and depression storage.

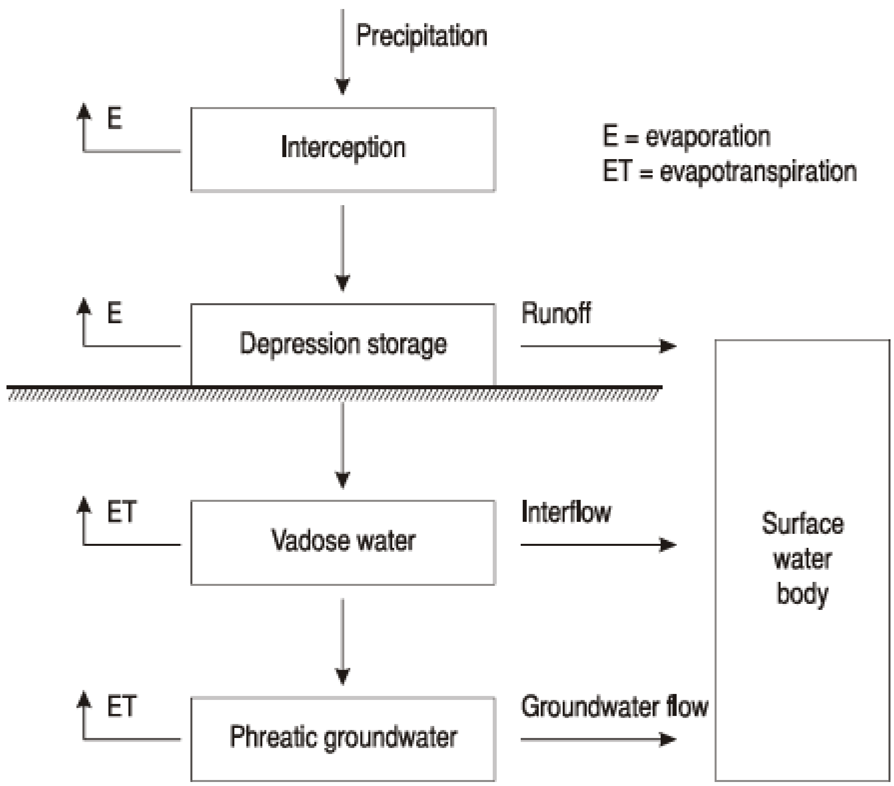

Figure 2. 1: Natural hydrologic Cycle

However, Marsalek et al. (2006) studied that urbanization contributes to changes in the radiation flux and the amount of precipitation, evaporation and evapotranspiration, infiltration into soils, and consequently causes changes in the hydrological cycle. Mohammad Karamouz, (2019), also describes that the impacts of large urban areas on local microclimate have long been recognized and occurred due to changes in the energy regime such as air circulation patterns caused by buildings, transformation of land surfaces and land use planning, water transfer, waste generation, and air quality variations.

According to Butler et al. (2018), urban drainage replaces one part of the natural water cycle and, as with any artificial system that takes the place of a natural one, the full effects must be understood. Mohammad Karamouz, (2019), elaborated that the impacts of large urban areas on local microclimate have long been recognized and occurred due to changes in the energy regime such as air circulation patterns caused by buildings, transformation of land surfaces and land use planning, water transfer, waste generation, and air quality variations.

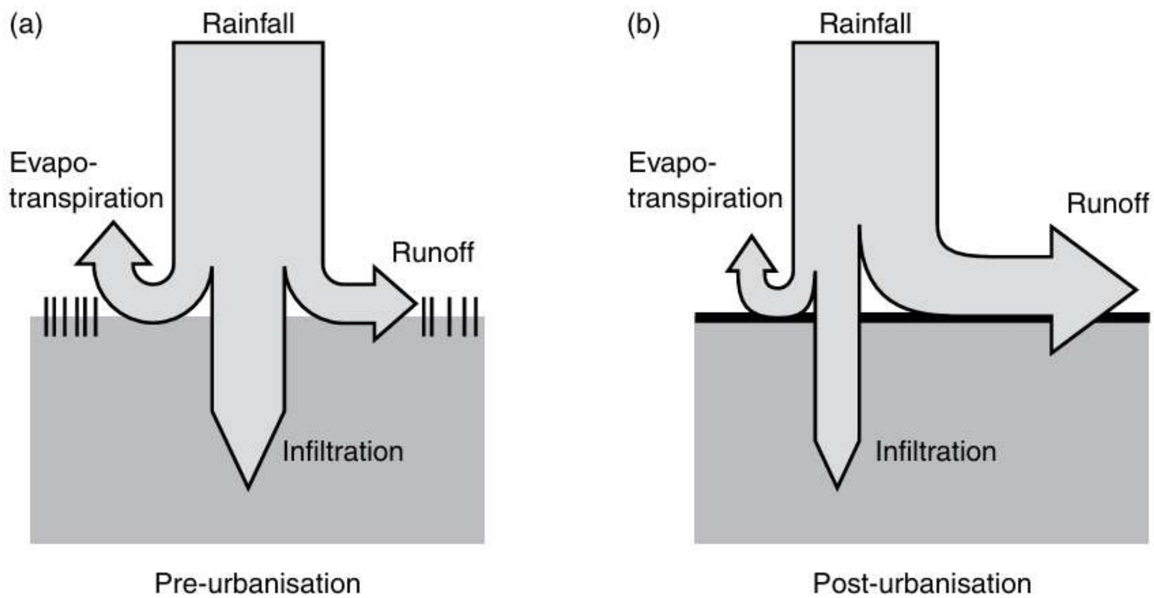

Figure 2. 2: Effect of urbanization on rate of rainfall

On another hand, it is estimated that more than 55% of the Global population lives in cities and that 394 of the world's cities have a population that exceeds 1 million inhabitants and it is anticipated that 83% of the developed world and 53% of the developing world will live in urban areas by 2030 (McGrane, 2016). Furthermore Slootweg et al. (2024), studied that transformation of green into gray surfaces due to urban expansion increases urban vulnerability to health and climate related threats such as flooding and urban heat, and consequences of the urban transformation extend far beyond city boundaries, emphasizing the need for sustainable urban development practices

Also Barranco-Mejía et al. (2024), also elaborated that urbanization increases the change in soils to increasingly impervious surfaces, which alters the urban hydrological cycle, causing an increase in the volume of surface runoff and peak flow. The process of urbanization, along with the development of traditional drainage systems and culverts, primarily aims to swiftly eliminate stormwater from urban regions. The effects of urban flooding are complex, involving direct damage to infrastructure, indirect damages due to disruptions, and long-term social consequences (Zhang et al., 2021).

2.2. Urban Stormwater Drainage Management

In various studies, it has been ensured that rapidly expansion of urban areas has adverse environmental and social consequences. Therefore in order to tackle the urban expansion and climate change caused flooding problem, adoption of appropriate sustainable urban stormwater management is the last alternative.

Seidu et al. (2025) stated that gray infrastructure refers to conventional built environment systems such as culverts and pipework used in stormwater management. Also, Chen et al. (2024), described that conventional stormwater measures prioritize flood mitigation and shield cities from grey infrastructure such as storm sewers, and pipes

However, rather than be dependent on only gray infrastructure for urban stormwater drainage, Karamoutsou et al. (2024), stated that sustainable stormwater management practices are essential for overcoming various environmental challenges and promoting community sustainability and resilience. According to Vinet & Zhedanov. (2011), in order to meet the drainage requirements of sustainable urban development, urban stormwater management has become increasingly sophisticated, multi-dimensional, and integrated in nature.

Teshome & Devi. (2020) state that SC, LID, and SuDS (BMP) is management technique mainly for surface water using natural drainage in conjunction with technologies and policies. BMPs used to reduce or prevent stormwater negative effects through structural, vegetative or managerial practices. They are sustainable urban water systems that have multiple benefits, e.g. stormwater pollution management, frequent flood mitigation, waterway health protection, microclimate improvement and enhancing the amenity of urban landscapes (Gogu et al., 2019)

Krebs et al. (2016) described that LID is a holistic strategy in land planning and engineering design that addresses stormwater runoff within the framework of green infrastructure, and with the objective of emulating natural hydrologic processes on developed sites. It prioritizes conservation and the utilization of on-site natural elements to safeguard water quality by infiltrating, filtering, and storing, evaporating, and retaining runoff near its origin. Kawathe & Thorvat. (2020) also stated that LID aims to preserve or rehabilitate the hydrologic and ecological functions of a watershed by managing water on-site.

2.3. Sustainable Urban Drainage System (SUDS)

A sustainable urban stormwater drainage management necessitates managing the stormwater not to cause deterioration of the quality of the receiving waterbody and to protect the downstream flooding hazards. It also plays a crucial role in replacing or recharging groundwater for potential water supply through enhancing infiltration. Guo et al. (2022), studied that urban stormwater management has historically depended on "gray infrastructure", and that the system encompasses elements like gutters, drains, pipes, and retention basins. Although this method is efficient for quick water removal, it frequently results in heightened runoff volumes, flooding downstream, and the contamination of natural water bodies due to insufficient natural filtration. Also, according to Martínez et al. (2021), as urban areas continue to grow and climate change results in more frequent and severe rainfall events, the current gray infrastructure systems frequently become overwhelmed , and this situation not only leads to environmental pollution but also requires expensive maintenance and upgrades to outdated systems.

As better urban drainage alternative than gray infrastructure Ummah. (2019) states that SUDS are very effective in reducing the potential of downstream flooding by retaining runoff from extreme events and removing pollutants, particularly those that remain attached to particulates through absorption and precipitation. Adding to this, Khan. (2024) described that to effectively adapt urban drainage systems to climate change; cities must adopt a range of innovative approaches that integrate traditional infrastructure with sustainable practices. Green infrastructure solutions, such as green roofs, permeable pavements and rain gardens, help managing stormwater at its source.

Also according to Sun et al. (2024), advanced hydrological models and decision-support tools are essential for the efficient planning and execution of IGGI. In addition according to Lemes de Oliveira et al. (2025) the most efficient and sustainable method for managing urban stormwater entails the integration of both green and gray infrastructure. This hybrid strategy capitalizes on the advantages of both systems: gray infrastructure offers essential conveyance and storage capabilities for substantial water volumes, whereas GI addresses smaller, more frequent rainfall occurrences at the source, alleviating pressure on the traditional system and bolstering overall resilience.

2.4. Modeling Stormwater Drainage

According to Majeed & Chinnamma. (2021), SWMM provides an integrated environment for editing study area input data and running hydrologic, hydraulic, and water quality simulations which can be viewed in various formats, and Rossman. (2022), further described that the essence of stormwater drainage modelling consists of simulating hydrologic, hydraulic, and water quality processes to forecast runoff behavior and guide management decisions. Akan & Houghtalen. (2013), also described that detention basins are small water impoundments with a capacity of about 10 acre-ft. or less. Retention basins are usually larger, and they release stored water at a slower rate, mostly through controlled outlets. Infiltration basins allow stored water to percolate into the ground. Dry wells are small trenches excavated in porous soil and backfilled with rock. They also allow stored stormwater to percolate into the ground

2.5.Barriers to Sustainable Stormwater Management in Developing Countries

According Vasconcelos et al. (2021), sustainable urban stormwater management (SUSM) has been increasingly adopted around the world and proved its effectiveness in enhancing sustainability and quality of life in cities. However, Jallé et al., (2021) states that in the developing countries, that the urban stormwater management is generally inadequate or non-existent. Where they exist, stormwater collection systems serve only the most central or wealthiest areas. Due to a chronic lack of care and maintenance, they are in extremely poor condition. In outlying neighborhoods, water runs along natural ravines, but these are not able to evacuate all surface runoff during heavy rain. In developing countries, the climatic and socioeconomic conditions bring difficulties to the use of solutions adopted in temperate areas. Goldenfum et al. (2021) also expressed that urban drainage master plan for the cities of developing countries is limited by problems such as the lack of appropriate data and the uncontrolled urban expansion. Okour, Y., & Shaweesh, H. (2024) also reviewed that green infrastructure implementation barriers are generally categorized into institutional, regulatory, technical, physical, economic, financial, and community barriers. Further described that Institutional and regulatory barriers relate to the government’s role in GI implementation, and one of the main barriers is that urban planning policies may not prioritize GI in development plans. Technical and physical barriers represent limitations in implementing GI due to site restrictions such as drainage or existing land use. Economic or financial barriers include limited funds and financial resources for green infrastructure projects on the other hand, community barriers relate to limited public perception and knowledge of GI, lack of community support and participation, and the public's resistance to change. Also, Aurkhede & Dane. (2024) , studied that due to their economic status, cities in developing countries lack adequate and modern drainage infrastructure.

CHAPTER 3: MATERIAL AND METHODS

3.1. Description of the study area

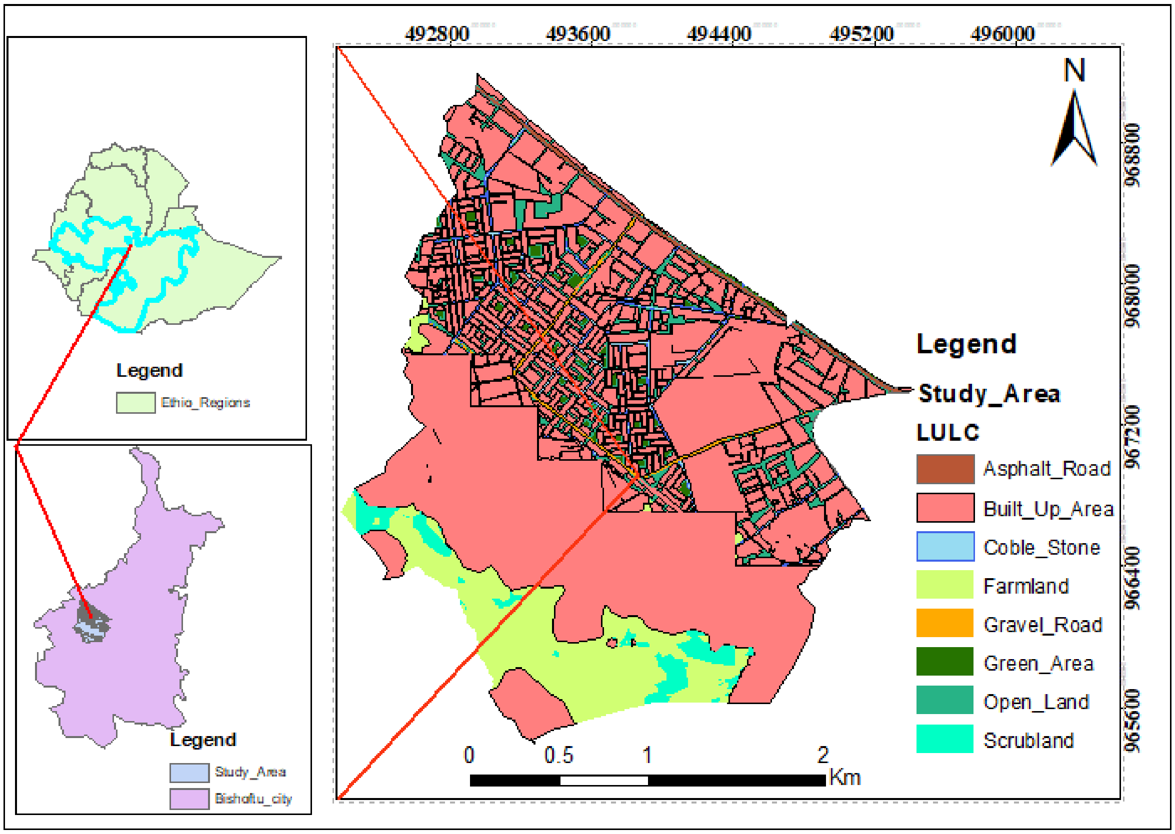

Figure 3. 1: Location Map of the Study Area along with LULC

3.1.1. Land use and cover

In percent, the total study area sub-catchments consist of 0.43% asphalt roads, 75.88% buildings, 2.01% cobblestone, 10.82% farmland, 0.67% gravel roads, 0.62% green areas, 7.64% open land, and 1.93% scrubland.

The composition exhibits a distinct set of features, predominantly influenced by impervious surfaces. With 75.88% of the area occupied by buildings and 0.43% by asphalt roads, the overwhelming majority of the watershed is enveloped in impervious materials, resulting in considerable changes to its hydrological characteristics. This substantial level of imperviousness indicates that the study area is urbanized watershed, notwithstanding the existence of certain natural and agricultural zones. The remaining land uses, which include cobblestone (2.01%), farmland (10.82%), gravel road (0.67%), green area (0.62%), open land

(7.64%), and scrubland (1.93%), will affect particular elements but are predominantly eclipsed by the constructed environment. The main feature of this watershed is its altered hydrology. The impervious surfaces hinder the absorption of rainfall into the soil, resulting in a greater volume and speed of surface runoff. This leads to more pronounced hydrographs, marked by swift increases in streamflow during rainfall events and faster recession times.

Table 3. 1: The study area land use land cover

S. No	Landover	Area_ha	Coverage in (%)
1	Asphalt Road	3.00	0.43
2	Built_Up_Area	527.46	75.88
3	Coble_Stone	13.97	2.01
4	Farmland	75.20	10.82
5	Gravel Road	4.66	0.67
6	Green Area	4.31	0.62
7	Open Land	53.11	7.64
8	Scrubland	13.42	1.93
		695.13	100.00

3.1.2. Soil types of the city and the study area

Soil types demonstrate different infiltration capacities, leading to varying amounts of runoff generated from the same rainfall events within the same area, influenced by the presence of various soil types. Therefore, comprehending the soil types in a specific region is essential for accurately predicting the runoff produced by rainfall in that area. According to information from the Ministry of Agriculture, Bishoftu city comprises three distinct soil types: chromic Luvisols, Eutric Vertisols, and lithic Leptosols. As shown on the figure 4.5 Eutric Vertisols is the most common soil type in the city; consequently, the total land area designated for this study is primarily characterized by Eutric Vertisols.

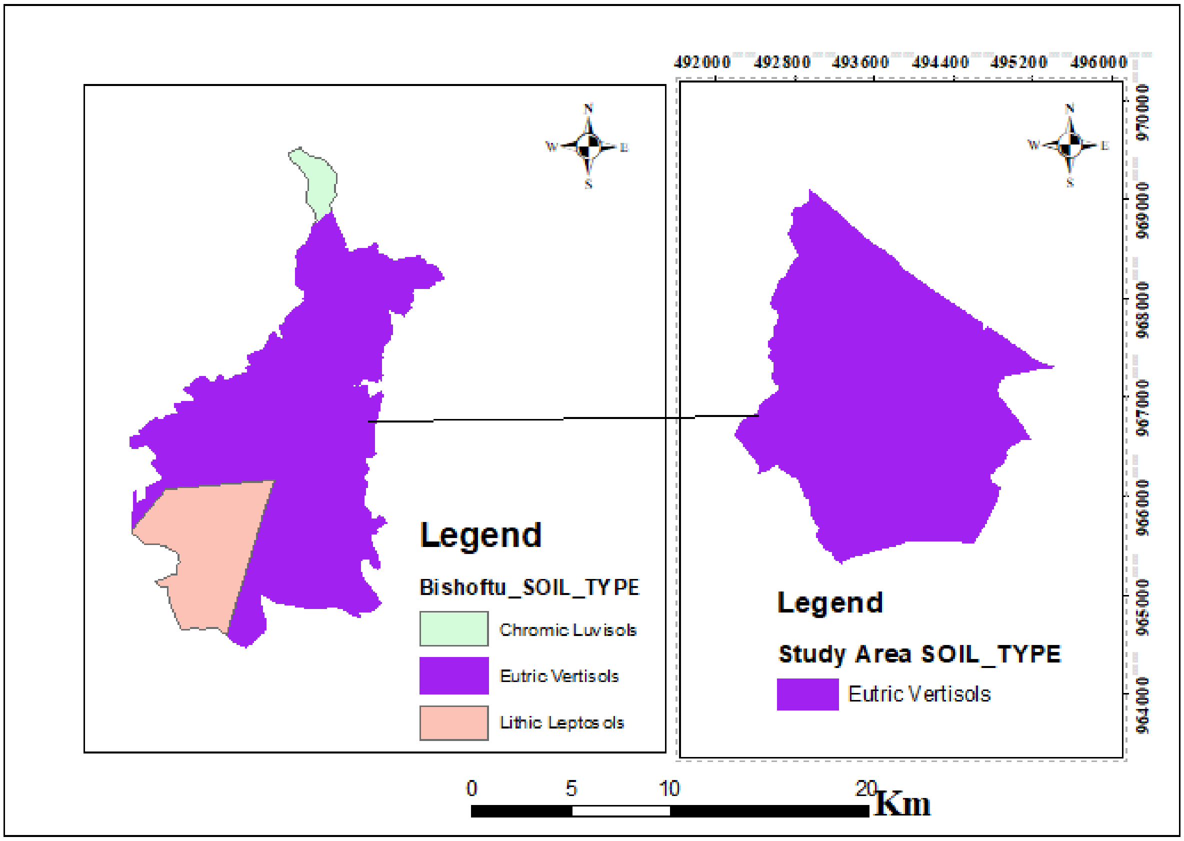

Figure 3. 2: Major soil types of the city and the designated study area

According to ERA, (2013), Eutric Vertisols are categorized as Group C soils within the hydrologic classification framework. This categorization is determined by their physical properties, especially their texture and drainage abilities. Group C soils exhibit a moderate infiltration rate when saturated, resulting in considerable water retention due to their high clay content, particularly the prevalence of smectite minerals found in Vertisols.

Typically, Eutric Vertisols have a significant clay content, often surpassing 30%, which increases their ability to expand and contract significantly in response to changes in moisture levels. This characteristic affects the movement of water throughout the soil profile. Eutric Vertisols, characterized by their significant clay content and structure, frequently demonstrate inadequate drainage properties, particularly when saturated. This condition can result in surface ponding and heightened runoff during periods of heavy rainfall.

3.1.3. Slope of the study area

The gradient of the land in the study area ranges from zero to 10 percent elevation. This terrain is regarded as extremely flat, making it challenging to establish an adequate slope that facilitates the easy movement of water. However, in the design and construction of stormwater drainage facilities, the consideration of the land area's slope is crucial.

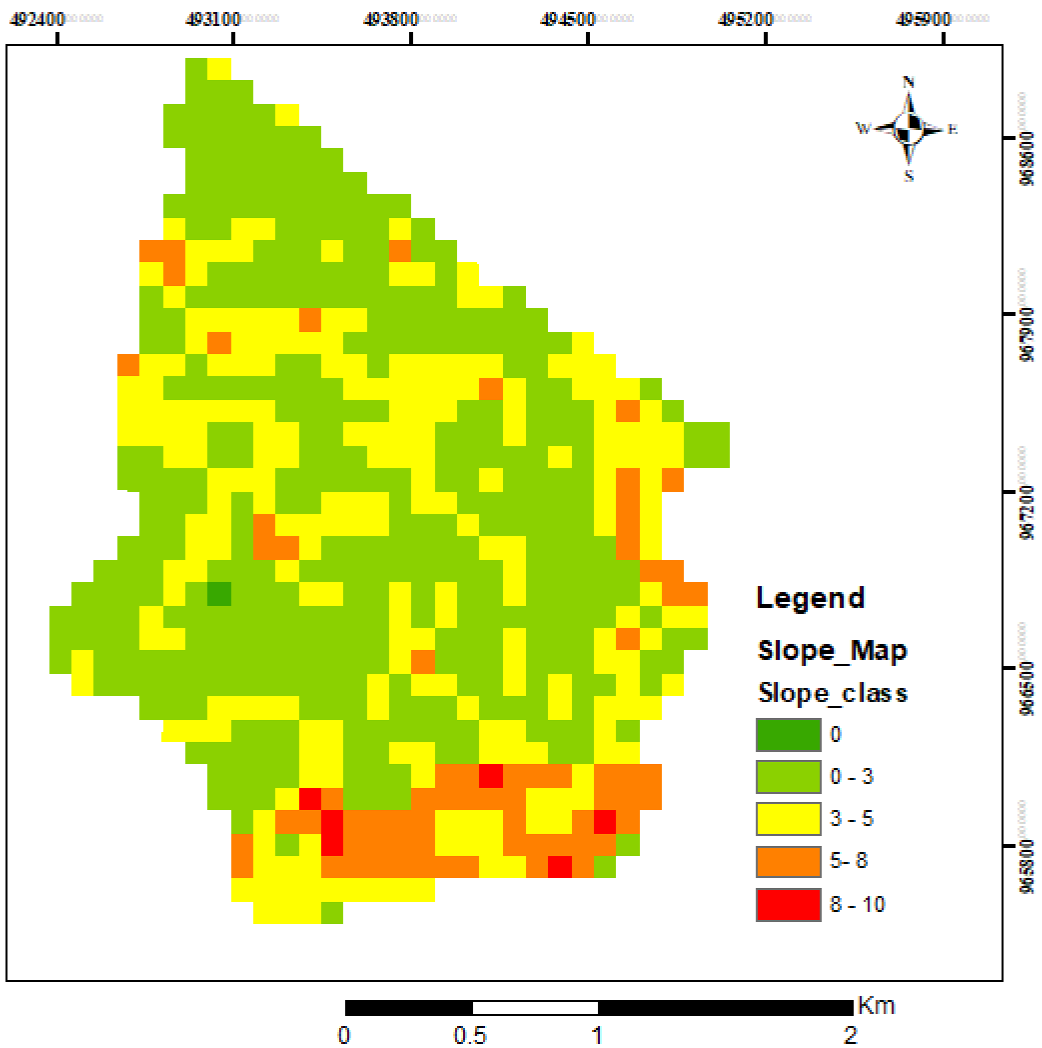

Figure 3. 3: Slope of the designated study area

A steep slope facilitates the movement of water across the contour, whereas a very gentle slope impedes water movement and promotes ponding. Consequently, a very gentle slope results in increased excavation costs to create an adequate slope for water flow. Similarly, the majority of the land in the study area is flat with a gentle slope. Over 75 percent of the land in the study area has a slope of less than 10%. Consequently, in a flat terrain, it is challenging to create sufficient slope for water movement.

3.1.4. Elevation map of the study area

Elevation plays a crucial role in the analysis of stormwater drainage, providing essential information required for the efficient planning, design, and management of drainage systems.

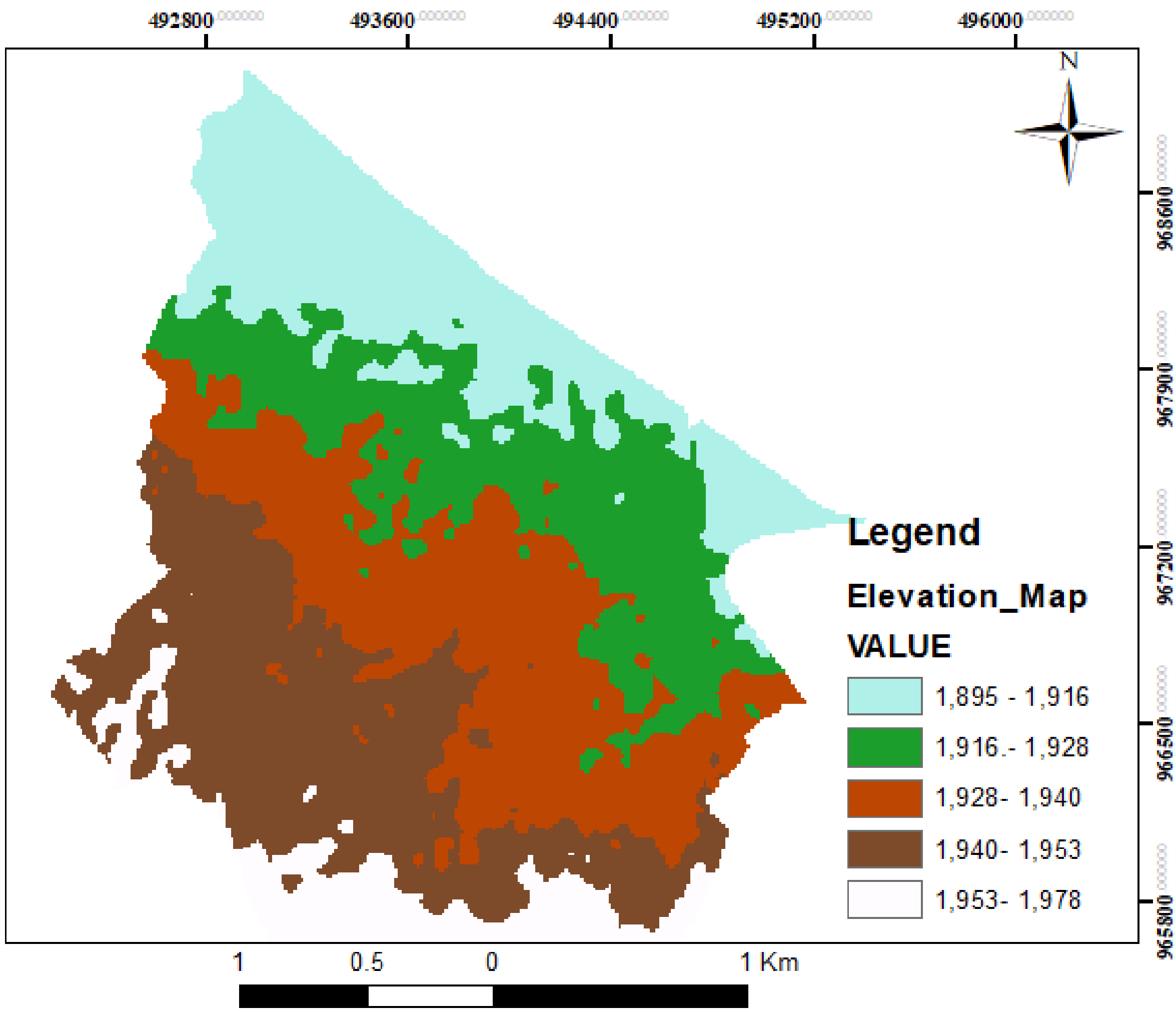

Figure 3. 4: Elevation Map of the Study Area

In the context of Bishoftu, a topographic map would prove to be highly advantageous for various reasons. Firstly, it offers detailed elevation data, which is vital for understanding the terrain and the movement of water across the landscape.

This elevation data is crucial for evaluating the direction and slope of surface runoff. By analyzing the contours, areas prone to flooding can be identified. Furthermore, the elevation of the specific area designated for this study falls within this range. As illustrated in the figure 3.4 the maximum and minimum elevations of the study area are 1,979 and 1,895 m above sea level, respectively.

3.2. Materials

Material utilized during the study are handy GPS, Mattock, Shovel, tape meter, notebook, and pen.

3.3. Data Types and Sources

For this research rainfall data, Drainage Data, land use land cover data, Soil data, and a DEM have been intensively utilized. These data were gathered from multiple sources. In this regard, rainfall data spanning 30 years were obtained from the National Meteorological Institute, while the drainage line data in shapefiles format was sourced from the municipal office of the city. Additionally, soil data for the study area was extracted from Ethio-soil shapefiles acquired from the Ministry of Agriculture. Furthermore, LULC data were collected from the city land administration and ESRI 2024.

3.4. Methodology

3.4.1. Research framework

The assessment of the effectiveness of urban stormwater drainage systems in Bishoftu city has been comprehended through a structured framework that incorporates diverse evaluation and modelling techniques. This framework consists of several critical components. In order to fulfil the objectives of the study, comprehensive data gathering was performed for an accurate performance evaluation. This procedure involved the acquisition of historical rainfall data, land use information, soil characteristics, and details regarding the existing drainage systems. Advanced technologies, including Geographical Information Systems (GIS), have been extensively employed to analyses spatial data related to the drainage systems in the area under investigation.

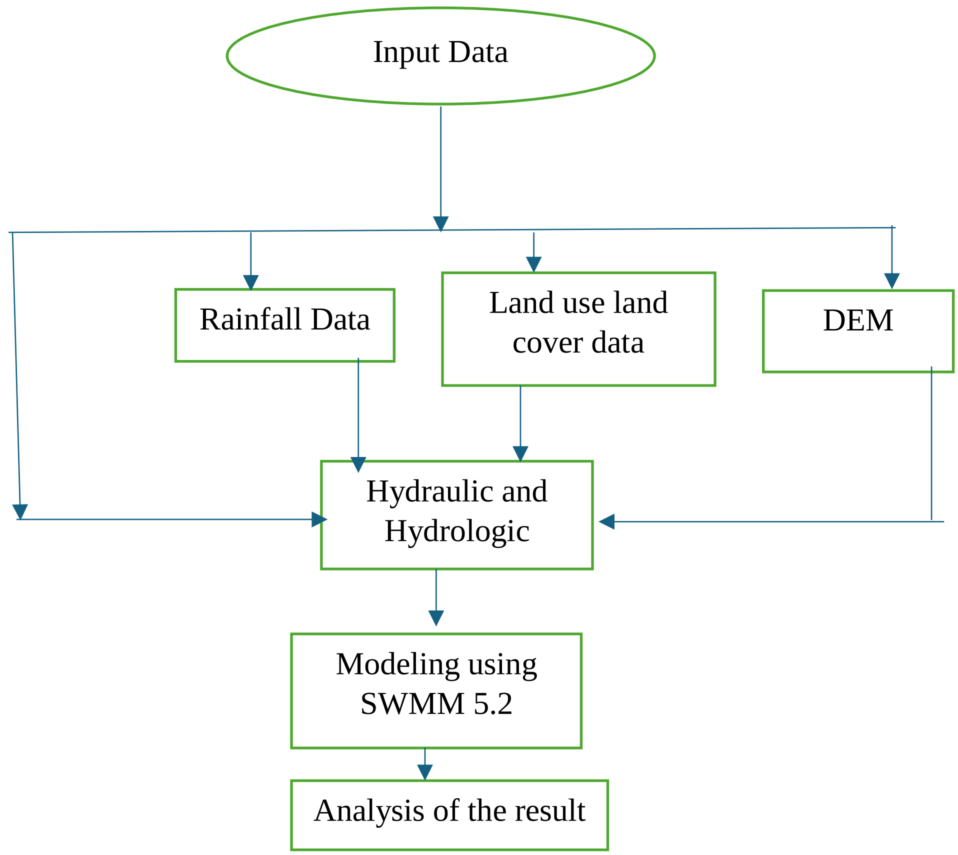

Figure 3. 5: Research Framework

The examination of the assessment and modelling of the current stormwater drainage system in Bishoftu city has been conducted by reviewing relevant literature, previous studies, and stormwater management guidelines. The primary focus of the stormwater drainage assessment and modelling has been on the hydrologic and hydraulic characteristics of the existing drainage infrastructure. To enhance the study, a comprehensive review of existing literature has been undertaken throughout the research process, and both relevant primary and secondary data have been gathered, analysed, and presented.

3.4.2. Literature review

A thorough literature review entails analyzing prior research on the hydrology of Bishoftu, urban development strategies, climate information, and current engineering documentation concerning the drainage system. This process aids in comprehending the historical background and pinpointing areas where data is lacking.

3.4.3. Measurement of drainage channel dimensions

Precise measurement of the dimensions (width, depth, and slope) of current drainage channels is crucial for hydraulic modelling. These measurements, along with material properties and roughness coefficients, facilitate the calculation of flow capacity through hydraulic equations such as Manning's equation. To identify their discharge capacity, the drainage ditches have been measured in overall the study area:

$Q = A\ V$ ____ 3.1

Where

Q = volumetric flow rate (m^3/s)

A= Area of channel

V= average flow velocity across the area

$V = \frac{1}{n}(R)^{2/3} * (S)^{0.5}$

Where:

n = Manning roughness coefficient (dimensionless)

R = hydraulic radius (A / P, where P is the wetted perimeter) (m)

S = slope of the energy line (m/m)

This equation is essential for assessing whether the existing channels can sufficiently handle the anticipated stormwater runoff.

Table 3. 2: Sample measured drainage channel with its location

S/No	X	Y	Elevation	IE	ED	Distance		
1	492720.888	968084.596	1926	1925.40	0.60		TW	0.95
2	492730.816	968085.788	1926	1925.158	0.84	10.00	BW	0.80
3	492740.745	968086.980	1926	1924.915	1.08	10.00	Depth	0.6

4	492750.674	968088.172	1925	1924.673	0.33	10.00	slope	0.024
5	492760.603	968089.361	1925	1924.431	0.57	10.00	n	0.013
6	492770.533	968090.540	1925	1924.188	0.81	10.00		
7	492780.463	968091.720	1924	1923.946	0.05	10.00		
8	492790.393	968092.899	1924	1923.703	0.30	10.00		
9	492800.327	968094.044	1924	1923.461	0.54	10.00		
10	492810.248	968095.301	1925	1923.219	1.78	10.00		
11	492820.163	968096.600	1925	1922.976	2.02	10.00		
12	492830.078	968097.898	1926	1922.734	3.27	10.00		
13	492839.994	968099.196	1927	1922.492	4.51	10.00		
14	492849.920	968100.403	1927	1922.249	4.75	10.00		
15	492859.863	968101.474	1927	1922.007	4.99	10.00		
16	492869.805	968102.544	1926	1921.765	4.24	10.00		
17	492879.764	968103.435	1925	1921.522	3.48	10.00		
18	492889.706	968104.502	1925	1921.28	3.72	10.00		
19	492899.634	968105.701	1925	1921.038	3.96	10.00		
20	492909.561	968106.899	1923	1920.795	2.20	10.00		
21	492919.489	968108.097	1921	1920.553	0.45	10.00		
22	492929.417	968109.296	1920	1920.311	-0.31	10.00		
23	492939.345	968110.494	1921	1920.068	0.93	10.00		
24	492949.273	968111.692	1921	1919.826	1.17	10.00		
25	492959.201	968112.891	1921	1919.584	1.42	10.00		
26	492969.129	968114.089	1921	1919.341	1.66	10.00		
27	492979.057	968115.287	1922	1919.099	2.90	10.00		
28	492988.985	968116.486	1922	1918.856	3.14	10.00		
29	492998.913	968117.684	1921	1918.614	2.39	10.00		
30	493008.841	968118.882	1920	1918.372	1.63	10.00		

31	493018.769	968120.081	1919	1918.129	0.87	10.00	
32	493028.697	968121.279	1918	1917.887	0.11	10.00	
33	493038.625	968122.478	1918	1917.645	0.36	10.00	
34	493048.647	968123.687	1918	1917.4	0.60	10.10	
						330.09	

3.5. Data analysis and modeling

3.5.1. Stormwater drainage data analysis

As previously stated, regarding the available stormwater drainage data, only drainage lines have been acquired from the municipal office of the city. Consequently, since the length and elevation of both the inlet and outlet of the drainage line are crucial for estimating the slope of each drainage line, the length of each drainage line has been estimated using ArcMap. The steps taken to estimate the length of the drainage line are as follows: first, the drainage line data was opened in ArcMap; then, by utilizing the editing tool, the coordinates were calculated for each drainage line. Finally, by unitizing the obtained (x, y) coordinates, the total length of the drainage line was computed using the formula distance = $((x_2 - x_1)^2 + (y_2 - y_1)^2)^{0.5}$

Once the length of the drainage line is established, it is necessary to determine the elevations of both the inlet and outlet in order to calculate the slope of the drainage line. Consequently, to acquire the elevations at the inlet and outlet, the drainage line has been overlaid on a 12.5 resolution DEM. Subsequently, by utilizing the spatial analyst tool in ArcMap, the elevations of the inlet and outlet have been extracted from the DEM.

3.5.2. GIS analysis

- Development of a geodatabase encompassing all gathered spatial data (including drainage network, land use, and topography);
- Delineation of watersheds and identification of sub-catchments utilizing DEMs; and
- Computation of hydrological parameters (such as catchment area, slope, and imperviousness) for every sub-catchment.

3.5.3. Topographic map and DEM

DEM was employed to delineate watershed boundaries, ascertain flow paths, compute slopes, and extract elevation data of the drainage line from this DEM.

3.5.4. Delineation of sub-catchment

Terminologies such as watershed, catchment, and basin all refer to a land area that drains to a single outlet. Consequently, to delineate all the sub-catchments, the United States Environmental Protection Agency's storm water management and modelling (EPA SWMM) was utilized. After identifying flood-prone areas based on the existing flood issues, the next step was to delineate each sub-catchment within these flood-prone regions.

Prior to commencing the delineation, the natural watershed was defined using a DEM at a resolution of 12.5 metres with ArcMap 10.8 to ascertain the flow direction. Following the delineation of the natural watershed or sub-catchment, the drainage line data was overlaid onto the delineated natural sub-catchment in ArcMap to determine which land areas drain into which drainage systems. Subsequently, after overlaying the natural watershed boundary with the existing stormwater drainage lines, the data was converted to Keyhole Markup Language (KML) format in ArcMap and exported to Google Earth Pro. It was then downloaded in JPG format, which is compatible with EPA SWMM. Finally, by loading this data into EPA SWMM, the delineation of the sub-catchments was carried out.

3.5.5. Determination of sub-catchment longest path

The interaction to next paint (inp) format is the structure employed by the SWMM to outline a stormwater network along with its associated parameters for simulation. This text-based file, typically bearing an inp extension, contains all the vital information necessary for SWMM to perform a simulation, which includes details about the physical layout of the drainage system, hydrological and hydraulic properties, as well as time series data for inputs such as precipitation.

Then the inp EPA SWMM format was imported into QGIS, and subsequently exported as a shapefiles in QGIS by utilizing the "Generate SWMM input files from layers" plugin. This plugin also enabled the importation of SWMM input files into QGIS. It enabled the specification of sub-catchments in an INP file into QGIS layers, which can then be exported as shapefiles.

Afterwards, the shapefiles of each sub-catchment are re-imported into ArcMap, and a DEM is extracted for each sub-catchment. The extracted DEM for each sub-catchment is utilized to determine the highest and lowest elevations of the sub-catchments. The distance between the highest and lowest elevations of each sub-catchment is then calculated using the formula distance = $((x_2 - x_1)^2 + (y_2 - y_1)^2)^{0.5}$ that represents the longest path of the sub-catchment.

By obtaining the distance through these procedures, the slope is calculated by subtracting the lowest elevation from the highest elevation and then dividing by the distance between the two points. Subsequently, the time of travel for the catchment is calculated using the NRCS Velocity Method Tt = $\frac{0.007*(n*L)^{0.8}}{P_2^{0.5}*S^{0.4}}$ ____3.2

3.5.6. Analysis and processing of LULC map

Due to the absence of the city master plan, pdf format of the previous land cover map of the city downloaded from the Internet. The land cover data of part of the city, which was demarcated in the city boundary but not included into the city master plan, were downloaded from ESRI Website, which was released in 2024. Then, after processing, the data has been together in ArcMap and land cover prepared. Then, based the prepared land cover, runoff coefficient for sub-catchments were determined

3.5.7. Hydrological data analysis

The collected rainfall data contains numerous missing values; consequently, XLSTAT 2018 has been utilized to address these gaps. When evaluating XLSTAT, Inverse Distance Weighting (IDW), and the Arithmetic Mean for filling in missing rainfall data, it is essential to comprehend their foundational methodologies, advantages, and limitations. Each technique presents a unique method for spatial interpolation or basic estimation, which renders their appropriateness contingent upon the particular, attributes of the rainfall data and the level of accuracy sought.

To address the gaps in rainfall data, the selection between XLSTAT (which provides multiple interpolation techniques), IDW, and the Arithmetic Mean is significantly influenced by the spatial attributes of the rainfall, the concentration of the current rain gauges, and the required degree of precision and intricacy.

XLSTAT serves as an extensive statistical software add-in for Microsoft excel, providing a diverse array of data analysis tools, which encompass several spatial interpolation methods. Therefore in this rainfall data analysis XLSTAT 2018 selected from its accuracy point of view (Wangwongchai et al., 2023).

Prior to filling in the missing data, the raw data was organized on a monthly basis in excel. For instance, to complete the missing rainfall data for the month of January, data from all January months over a span of 30 years was compiled. Subsequently, the chosen data type in XLSTAT was Quantitative, and the estimation method employed was multiple imputation. After verifying the variables, the process was executed (Addinsof, 2022).

Steps followed to fill Missing Rainfall Data in XLSTAT 2018

- Rainfall data has been organized in an Excel spreadsheet with each column representing variables such as date, Station ID, and each row representing observation.
- XLSTAT has been launched, and to look for missing data XLSTAT menu has been navigated;
- Data range that contains missing value has been specified; and
- Multiple Imputation method has been selected (Chinasho et al., 2021).

3.5.7.1. Check the quality of data

Outlier identification

Interquartile range (IQR) method

To identify outliers among various existing methods, the interquartile range has been chosen. This method is especially effective for data that is not normally distributed. Furthermore, it is the simplest approach among the available techniques for calculating outliers. The steps taken to determine the lower and upper boundaries are as follows:

- The daily maximum rainfall data has been sorted in ascending order;
- The First Quartile (Q1), which represents the median of the lower half of the dataset, has been computed;
- The Third Quartile (Q3), which denotes the median of the upper half of the dataset, has been computed; and
- The Interquartile Range (IQR), defined as the difference between the third and first quartiles, has been calculated (IQR=Q3-Q1).

Boundaries for outliers has been defined through:

Lower boundary = $Q_1 - 1.5IQR$ ____3.3

Upper boundary = $Q_3 + 1 \cdot 5 * IQR$ ____3.4

Therefore, any data point falling outside these boundaries is considered an outlier

3.5.7.2. Frequency analysis

The aim of conducting a frequency analysis on a set of observed values of a hydrologic variable is to forecast future values associated with various return periods of interest. To accomplish this, it is essential to identify the probability distribution that best fits the available hydrologic data through statistical methods. Once a suitable probability distribution that accurately reflects the data series is established, we can intelligently interpolate and extrapolate from the observed data values. Frequency factors and specialized probability graph papers are valuable tools in this process. (Akan & Hough Talen, 2013).

3.5.7.3. Frequency factor

Frequency factor was determined S

In hydrology, many theoretical distributions lack closed-form analytical expressions for their cumulative density functions.

$$K_T = \frac{-\sqrt{6}}{\pi}\left(0.5772 + \ln[\ln\left(\frac{T}{T-1}\right)\right) \qquad 3.5$$

$$x_T = m + K_T * S \qquad 3.6$$

Where m and s represent the sample mean and standard deviation, respectively; Rainfall Depth associated with a particular return period ($X_{T)}$ T; and K_T is the frequency factor applicable to that return period.

Gumbel theory of distribution

The Gumbel distribution approach was chosen for conducting the flood probability analysis. This distribution is the most commonly utilized for intensity duration frequency (IDF) analysis due to its effectiveness in modelling extreme values. Its methodology is straightforward and focuses solely on extreme events, such as maximum values or peak rainfall. The Gumbel method computes return intervals of 2, 5, 10, 25, 50, and 100 years for each duration period, necessitating a series of calculations. Frequency precipitation point ($P_{T)}$ (mm) for each duration with a specified return period T (in year) is given by the following equation.

$$P_T = P_{ave} + KtS \qquad 3.7$$

P_{ave} represents the mean value of the highest precipitation recorded for a particular duration.

$P_{ave} = \frac{1}{n}\sum_{i=1}^{n} P_i$ ---3.8

In this context, P_i represents the specific extreme value of rainfall, while n denotes the total number of events or years documented. The standard deviation is calculated using the subsequent formula:

Where K is Gumbel frequency factor given by:

3.5.7.4. Statistical parameters

The majority of theoretical probability distributions are defined by statistical parameters that describe the characteristics of a population, including the mean, standard deviation, and skewness. Since we lack access to all values within the complete population, we are unable to ascertain these parameters with precision. Nevertheless, it is possible to estimate these statistical parameters using data obtained from a sample.

Consider a sample consisting of N observed values of a random variable, denoted as xi, where i ranges from one to N. In the context of an annual maximum streamflow series, xi represents the highest streamflow recorded in the itch year. The sample estimate of the mean (m) is then calculated as follows:

$m1 = \frac{1}{N}\sum_{1=1}^{N} x_i$---3.9

In straightforward terms, m represents the mean of all the values observed within the sample. Variance serves as an indicator of the data's variability. The standard deviation, which is the square root of the variance, provides a measure of dispersion. The sample estimate of the standard deviation is denoted as (s) is

$s = \left[\frac{1}{N-1}\sum_{i=1}^{N}(x_i - m)^2\right]^{\frac{1}{2}}$ ____3.10

Skewness, often referred to as skew, quantifies the asymmetry of a probability distribution in relation to its mean. The skew coefficient can be estimated using the available data.

$G = \frac{N\sum_{i=1}^{N}(X_i - m)^3}{(N-1)(N-2)S^3}$ ____3.11

Log Pearson Type III

The Log Pearson Type III probability model is employed to determine rainfall intensity across various durations and return periods, thereby creating historical IDF curves for each station. This model utilizes the logarithms of the observed values. The mean and standard deviation are calculated based on the logarithmically transformed data. Similarly, to the Gumbel method, the frequency of precipitation is derived using the Log Pearson Type III approach. The simplified formula for this distribution is presented as follows:

$$P* = \log(Pi) \qquad 3.12$$

$$Log\, x_T = m_1 + K_T S \qquad 3.13$$

The value of the frequency factor depends on the probability distribution being considered. An explicit analytical expression for K_T is available only for the Gumbel distribution:

3.5.7.5. Testing goodness of fit

The chi-square test is a statistical method used to assess how well a set of data aligns with a specified probability distribution. This procedure involves partitioning the entire spectrum of potential values of the hydrologic variable into k class intervals. We then evaluate the actual count of data points within these intervals against the anticipated count based on the probability distribution under examination. The selection of the number of class intervals, k, is made to ensure that each interval contains an expected minimum of three data points. The boundaries of these class intervals are established to maintain an equal expected count of data points across all intervals.

3.5.7.6. Rainfall intensity

Rainfall intensity (I) refers to the average rate of rainfall measured in millimeters per hour, corresponding to the duration equal to the time of concentration for a specified return period. After selecting a return period for design purposes and calculating the time of concentration for the catchment area, the rainfall intensity can be obtained from Rainfall-intensity-Duration curves. In Ethiopia's drainage regions, it is possible to calculate the rainfall intensity for any specified duration by utilizing the 24-hour rainfall depth, which is referred to as the IDF relationship.

$$R_{Rt} = \frac{t}{24} * \frac{(b+24)^n}{24(b+t)^n} \qquad 3.14$$

Where, R_{Rt} = Rainfall Depth ratio R_t:R_{24}, R_t = rainfall depth in a given duration "t"

R_{24} = 24 hr. rainfall depth, and b and n = coefficients b=0.3 and n= (0.78-1.09)

3.5.7.7. Runoff coefficient

The runoff coefficient (C) is the aspect of the rational method that is least amenable to accurate measurement, necessitating the designer's discernment and comprehension. A standard coefficient reflects the cumulative influence of various parameters within a drainage basin. The subsequent discussion will examine the impacts of soil classifications, land utilization, and average land gradient.

3.5.7.8. IDF Curve

The rainfall characteristics of a location can be fully understood if the intensities, durations, and frequencies of the storms that occur there are identified, and during a significant rainfall event takes place, its intensity and duration are typically recorded through meteorological observations. This data can then be utilized to ascertain the frequencies of different rainfall events. Frequency information for storms of varying durations can be illustrated using IDF curves. An IDF curve graphically represents the average rainfall intensity, where the rainfall depth is averaged over the specified duration.

$$\frac{\text{Rainfall Depth}}{\text{Duration}} = \text{Average rainfall intensity in that duration} \qquad 3.15$$

The rainfall measurements collected from the gauging station represent depths over a 24-hour period. However, the design and analysis of drainage systems necessitate understanding the relationship between rainfall intensity and duration for shorter periods. Due to the lack of available rainfall data for these shorter durations, it is essential to derive appropriate IDF relationships. The Ethiopian Road Authority (ERA) Drainage Design Manual, published in 2013, provides a specific equation for converting 24-hour rainfall data into estimates for shorter durations.

The techniques utilized to create the IDF curve for shorter duration events, based on the aforementioned equations, are outlined below.

- The trend line equation derived from the Log Pearson-III distribution method of frequency analysis is expressed as y = -37.76ln(x) + 205.92. In this equation, y represents the 24-hour rainfall depth (R24) corresponding to a specific return period x. The values of R24 are computed for return periods of 2, 5, 10, 25, 50, and 100 years. Rearranging the above equation gives

$$R_t = \frac{t(b+24)^n}{24(b+t)^n} * R_{24} \qquad 3.16$$

Substituting Intensity (mm/hr.), $I_t = \frac{R_t}{t}$, in the above equation

$$I_t = \frac{R_{24}(b+24)^n}{24(b+t)^n} \qquad 3.17$$

By applying b = 0.3 and n = 0.92 as recommended by the ERA manual, data is compiled for rainfall durations of 10, 20, 30, up to 180 minutes. The compiled data is then represented graphically for each return period. Consequently, the IDF curve is generated utilizing the reduction formula.

3.5.7.9. The rational formula

This approach is extensively utilized for determining peak discharge from smaller catchments (areas less than 0.5 km2), as advised by the ERA drainage modelling manual. Even though areas of some of the sub-catchments are greater than 50 ha or 0.5km2, in this thesis, the peak discharge from 42 (45) sub-catchment areas is calculated using the rational method. The fundamental concept of the rational method is that when a rainfall of intensity (I) begins instantaneously and persists indefinitely, the rate of runoff will rise until the time of concentration tc, at which point the entire watershed contributes to the flow at the outlet. The inflow rate for the system is represented by the product of rainfall intensity I and watershed area A, denoted as intensity * Area (IA), while the ratio of peak discharge Q to this inflow rate defines the coefficient of runoff C. The discharge is represented in the rational formula in the following manner:

$$Q = 0.00278CIA \qquad 3.18$$

Where, Q = peak discharge

I = Rainfall intensity (mm/hr.),

A = Catchment area (hectares); and

C= Runoff coefficient

The formula is based on the subsequent assumptions:

- ❖ The maximum likelihood of occurrence (return period) corresponds to the intensity of rainfall;
- ❖ The runoff coefficient C remains constant throughout the duration of the rainstorm; and
- ❖ The time of concentration is being approached.

Runoff coefficient: C Value

The runoff coefficient, also known as the C value, is a dimensionless empirical constant that indicates the percentage of rainfall that transforms into runoff (Rossman, 2022). It is understood to fluctuate based on time and the intensity of rainfall. The C value is variable and is influenced by the permeability of different surfaces. Areas characterized by low infiltration capacity, such as impervious surfaces, urban regions, and steep gradients, exhibit a higher C value in comparison to permeable surfaces like forests, cultivated lands, flat terrains, and pervious surfaces. In other words, impermeable surfaces generate more runoff than their permeable counterparts do

The elevated C value indicates a reduced infiltration capacity of the surfaces, thereby heightening the risk of urban flash or surface flooding.

$$C_{weighted} = \frac{\sum A_{i*} C_i}{A_T} \quad \text{\_\_\_\_\_\_\_\_\_\_} 3.19$$

Where:

C_i = Runoff coefficient for a given hydrological soil group

A_i = Area under each hydrologic soil gropu

A_T = Total catchment area under consideration

Table 3. 3: Runoff Coefficient C Value (Gong et al..., 2023)

Types of surfaces	Runoff coefficient
Impervious surface, (rooftop, concrete/asphalt, mountain)	0.85-0.95

Urban centre dense inhabited area	0.7-0.9
Apartments or townhouse	0.6-0.8
Detached/family house area	0.5-0.7
Gravel/ unpaved road	0.5-0.8
Lawns, cultivated land, parks, cemeteries	0.3-0.5
Industrial area	0.3-0.9

Time of concentration

The time of concentration, also referred to as travel time, is defined as the duration from the onset of a rainfall event until excess water exits the catchment at the furthest downstream outlet. Despite its significance, determining the time of concentration can often be quite challenging. There are various methodologies available for calculating the time of concentration; however, the most prevalent approach, and the one utilized in this study, is the NRCS Velocity Method. In this method, the time of concentration is calculated as the sum of the respective travel times (Tt) for the different flow types: sheet flow, shallow concentrated flow, and open channel flow (Gburek, 1998)

i. sheet flow

Sheet flow takes place when water moves over flat surfaces, usually at the onset of stream development. Research indicates that the maximum extent of sheet flow is restricted to between 100 and 130 metres. As per the NRCS Velocity Method, the duration of travel for sheet flow is determined as

$$\bar{T}_t = \frac{0.007 * (n * L)^{0.8}}{P_2^{0.5} * S^{0.4}} \quad 3.20$$

Where: T_t = travel time, hr.; n = Manning's roughness coefficient; L = flow length, m; P_2 = 2-year, 24-hour rainfall, mm; S = slope of hydraulic grade line (land slope), m/m (United States Department of Agriculture, 1986)

ii. Shallow concentrated flow

After a distance of up to 100 metres, sheet flow typically transitions into shallow concentrated flow. The average velocity of this flow can be calculated using the following formula, where the average velocity depends on the slope of the watercourse and the type of channel (ERA, 2013)

For unpaved $v = 4 \cdot 9178(S)^{0.5}$ ____ 3.21

For paved $v = 6.1961 * (S)^{0.5}$ ____ 3.22

Subsequently, the velocity is divided by the flow distance according to the equation provided below in order to calculate the total overland flow time.

$$T_{tvavel} = \frac{L}{60 * V} \qquad 3.24$$

Where:

Tt= travel time of concentration (minutes)

L = flow length, m

V=Average velocity over the surface (m/s)

S = slope of hydraulic grade line (watercourse slope), m/m

iii. Channel flow

When shallow concentrated flow enters a storm drainage system (i.e., pipes, culverts, ditches, and canals). The travel time during open channel flow is determined using the average velocity calculation based on Manning's equation.

$$\mathbf{V = \frac{1}{n}(R)\frac{2}{3}(S)^{0 \cdot 5}} \qquad \mathbf{3.25}$$

Where V represents the average velocity measured in metres per second, R denotes the hydraulic radius in metres, S indicates the slope of the hydraulic grade line expressed in m/m, and n refers to Manning's roughness coefficient. The time of concentration is calculated as the total of Tt values across the different consecutive flow segments:

$Tc = Tt_1+Tt_2+Ttn$ ____ 3.26

Where:

Tc = time of concentration, hr.,

n = number of flow segments

3.6. Modeling rainfall using EPA SWMM

3.6.1. EPA SWMM

SWMM is a dynamic model for simulating rainfall-runoff, applicable for both single events and long-term (continuous) runoff quantity and quality assessments, primarily in urban settings. The runoff aspect of SWMM functions based on a series of sub-catchment areas that collect precipitation, subsequently generating runoff and pollutant loads. The routing segment of SWMM facilitates the movement of this runoff through a network of pipes, channels, storage and treatment devices, pumps, and regulators. SWMM monitors the quantity and quality of runoff produced within each sub-catchment, as well as the flow rate, flow depth, and water quality in each pipe and channel throughout a simulation period that consists of multiple time steps. Even in small catchments, the runoff and the resulting model predictions (along with prototype measurements) can be extremely sensitive to spatial variations in rainfall. For example, thunderstorms (which produce convective rainfall) can be very localized, leading to significantly different readings from nearby gauges.

To ensure modelling accuracy, or more precisely, to achieve a successful calibration of SWMM, it is crucial that rain gauges are positioned both within and adjacent to the catchment.

Model setup procedure

- Establish the coordinates for the area map/image;
- Illustrate the network representation and detail the sub-catchments;
- Modify the properties of the components that constitute the system;
- Explain the operational procedures of the system;
- Choose a range of analysis options;
- Execute the simulation for rainfall/runoff and flow routing.

3.5.2. Model parameterization

SWMM necessitates three key pieces of information for modelling runoff quantity:

1. Physical characteristics of the catchment,
2. Rainfall data, and

3. Infiltration rates.

The physical catchment data includes total catchment area (A), the percentage of impervious area (percentage Imp), flow width (W), average slope (So), surface depression storage, and surface roughness. Most of this data was obtained from the master plan of Bishoftu city. Additionally, a field survey was conducted to verify the surface and subsurface drainage patterns, ensuring accurate discretization of the sub-catchment areas. The area-weighted percentage of imperviousness was calculated by adding the impervious area of each sub-catchment and dividing this total by the overall catchment area.

First, the DEM of each sub-catchment extracted, then the highest and lowest elevation of each sub-catchment known from the extracted DEM. Following the distance between the highest and lowest elevation which is the longest flow path of the sub-catchment has been calculated using distance = ((X2-X1)^2+(Y2-Y1)^2)^0.5. Then slope each sub-catchment was calculated by dividing the difference of the highest and lowest elevation by the longest flow path. Flow width is regarded as one of the least tangible parameters in SWMM. It is defined as the 'characteristic width of the overland flow path for sheet flow runoff' (Rossman, 2022). According to (Rossman, 2022), the width parameter can be determined by dividing the sub-catchment area by the length of the longest overland flow path within that area. The impervious depression storage (Dimp) was estimated based on existing literature. In the context of SWMM, the potential for runoff to begin immediately following rainfall is taken into account by permitting a percentage of the impervious area to have zero depression storage. The default value for zero depression storage is set at 25%. The values for surface roughness were taken from the SWMM User's manual (Rossman, 2022).

The model necessitates specific infiltration parameters, which include suction head (ψ_s), conductivity (Ks), and maximum moisture deficit (θ_{max}). Due to the impracticality of acquiring such detailed information via field samples, the infiltration parameters utilized were derived from the SWMM manual (Rossman, 2022) according to the soil types, specifically silty clay.

For every sub-catchment recorded in the model, it is essential to have an associated rain gauge to provide data regarding wet weather, sourced from either time series inputs or *.DAT data files. For the four outfall points, the invert elevations were established at the height of the incoming pipe ends. Since all outfalls are situated below the water level of the sub-catchments, a fixed boundary condition was implemented, reflecting the appropriate water elevation at the average water level of the outfall points.

3.6.3. Governing equation

SWMM represents a sub-catchment as a rectangular area characterized by a consistent slope S and a width W that directs water to a singular outlet channel, as illustrated in the figure below. The overland flow is produced by simulating the sub-catchment as a nonlinear reservoir

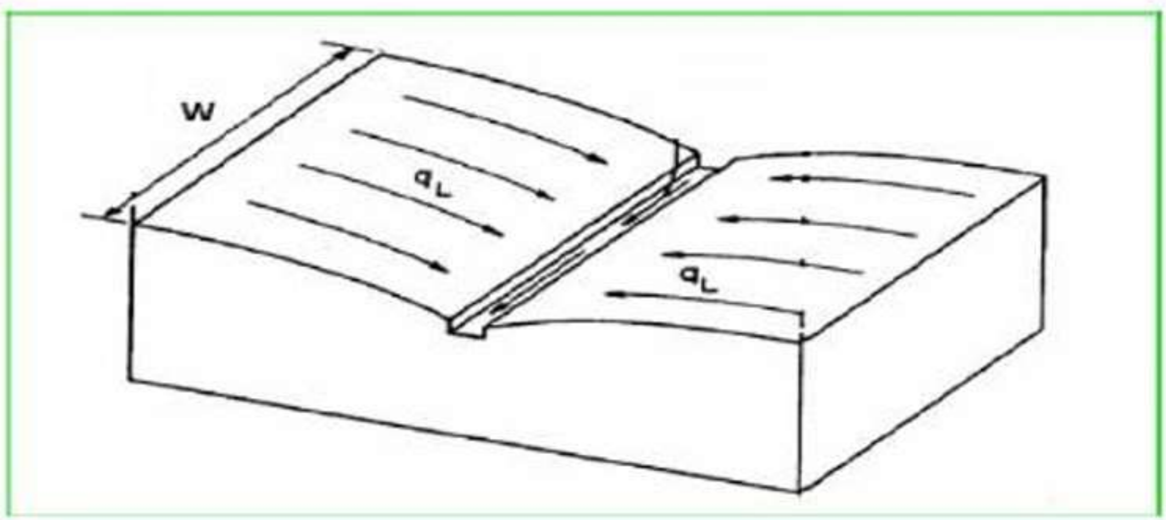

Figure 3. 4: Depiction of Sub-Catchment

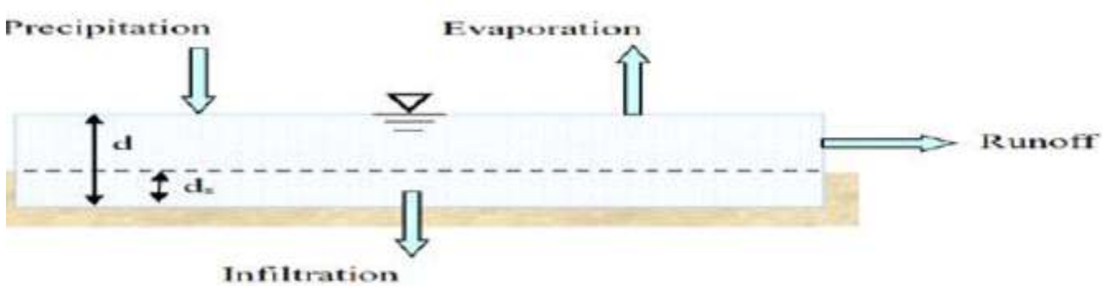

Figure 3. 5: Nonlinear of reservoir model

In this depiction, the sub-catchment receives inflow from precipitation (including rainfall and snowmelt) while also experiencing losses due to evaporation and infiltration. The net surplus accumulates on the surface of the sub-catchment to a depth d. Water that is ponded above the depression storage depth ds may result in runoff outflow q. Depression storage takes into account initial rainfall abstractions such as surface ponding, interception by flat roofs and vegetation, as well as surface wetting.

From the principle of conservation of mass, the net change in depth d per unit of time t is essentially the difference between the inflow and outflow rates across the sub-catchment:

$$\frac{\partial d}{\partial t} = i - e - f - q$$

Where: i = rate of rainfall plus snowmelt (ft/s); e = surface evaporation rate (ft/s); f = infiltration rate (ft/s); q = runoff rate (ft/s). It is important to note that the fluxes i, e, f, and q are represented as flow rates per unit area (cfs/ft2 = ft/s). The necessary rain input data for Urban Drainage applications is contingent upon the specific nature of the engineering task. Rain data is typically quantified as intensity (mm/h) or depth (mm), and it is associated with a statistical concept known as frequency. Frequency is generally depicted as the return period, which indicates the likelihood that a rainfall event of a certain intensity will be equaled or exceeded in any given year (Haestad Methods & S. Rocky Durrans, 2003)

3.6.4. Runoff evaluation

As stated by Haestad Methods & S. Rocky Durrans (2003) runoff refers to the volume of water from rainfall that is not absorbed by interception, evapotranspiration, or infiltration, ultimately flowing into water bodies or stormwater collection systems after traversing the surface. Therefore, the quantity and characteristics of runoff are influenced not only by the rainfall pattern but also by the properties of the catchment.

In order to simulate runoff generation, it is necessary to establish certain parameters within the programme to delineate the characteristics of the sub-catchments. The catchment area has been segmented into 45 smaller sub-catchments, all of which are connected to a single node.

The impermeability was determined for each sub-catchment based on the proportion of various surfaces, as illustrated in the equation below.

$$\omega = \frac{\omega_1 A_1 + \omega_2 A_2 +_{-\mp} \omega_n A_n}{A_1 + A_2 \pm - - - \mp A_n}$$

Where:

ω = the imperviousness of the hole sub-catchment, = the imperviousness of each surface type, A1 = the area of each surface

CHAPTER 4: RESULTS AND DISCUSSION

4.1.PRainfall Pattern of the Study Area

The rainfall recorded in Bishoftu city from 1991 to 2020 shows that the highest annual total was 1162.79 mm, which occurred in 2016. Conversely, the lowest rainfall during this period was 589.80 mm, recorded in 1995. The average or mean rainfall, along with the median, and standard deviation for this period, are 867.94 mm, 895.32 mm, and 143.38 mm, respectively.

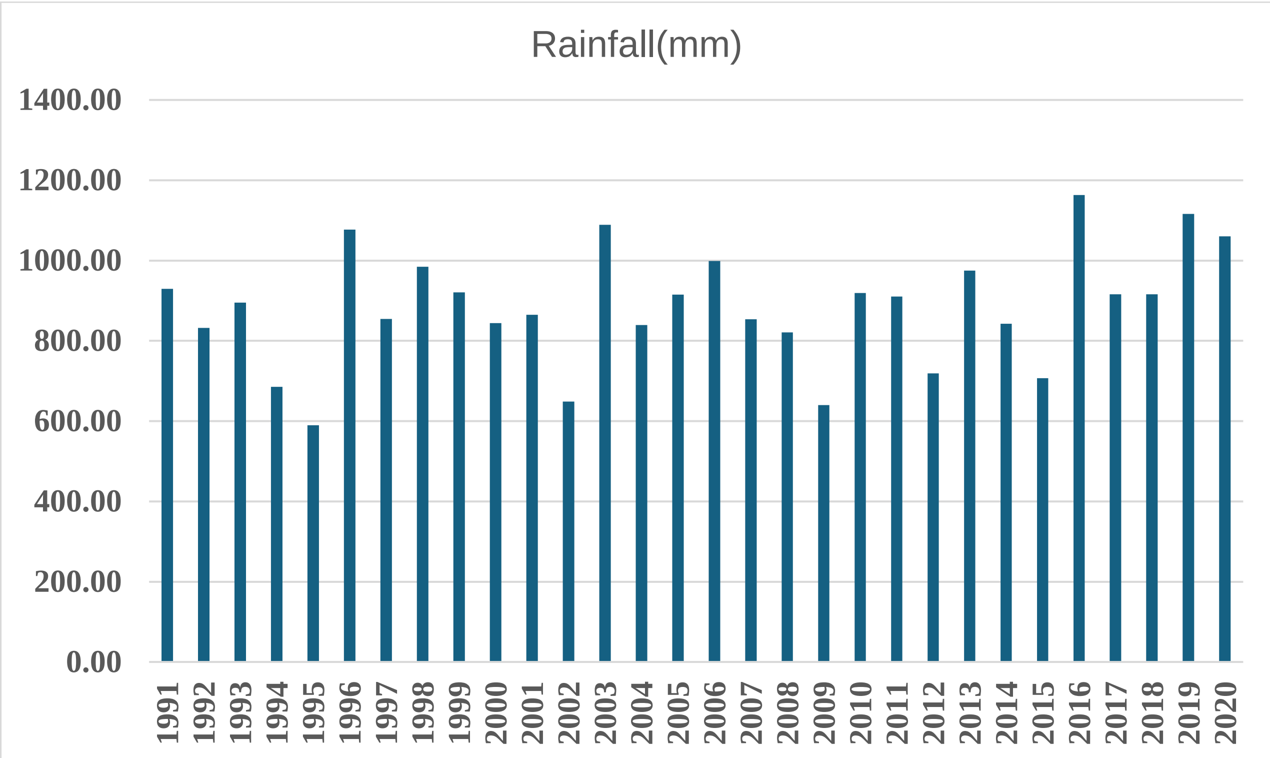

Figure 4. 1: Graph of Bishoftu city annual Rainfall from 1991 to 2020

The maximum annual rainfall recorded was 1162.79 mm in 2016, whereas the minimum was 589.80 mm in 1995. This demonstrates a considerable variation in annual precipitation throughout the three decades. The average (mean) rainfall during this period was 867.94 mm, with a median of 895.32 mm. The fact that the median is slightly higher than the mean indicates a minor negative skew in the data, suggesting that there are more years with rainfall totals that either are close to or exceed the mean compared to those that fall below it or that the years with lower rainfall are less extreme than those with higher rainfall.

4.1.1. Daily maxim rainfall data with outliers

Based on the implemented outlier identification, the maximum daily rainfall exhibit two upper boundary while lower boundary outlier has not been sensed as illustrated in the preceding table 4.2, the lower and upper limits are 16.99 and 71.08, respectively. A lower boundary outlier refers to a data point that is considerably lower than the other data points within a distribution. In the context of daily annual maximum rainfall, a lower boundary outlier indicates an exceptionally low maximum daily rainfall value for a specific year when compared to the other years in the 30-year span.

If there is no lower boundary outlier present in this dataset, it indicates that all the annual maximum daily rainfall values, including the lowest, fall within the anticipated range of variability for this 30-year span. There was no a single year in which the maximum daily rainfall was significantly lower than the maximums recorded in other years.

In the course of analyzing the daily annual maximum rainfall, two upper boundary outliers were identified and the two data points markedly surpass the overall trend and distribution of the remaining annual maximum rainfall values, positioned well above the upper quartile and generally exceeding a specified statistical threshold. These outliers signify extraordinarily rare and intense rainfall occurrences within the thirty-year observation period. The existence of two upper boundary outliers in the daily maximum rainfall dataset has considerable consequences for stormwater management, primarily suggesting that the current infrastructure and design standards might be insufficient for accommodating future extreme rainfall occurrences. Traditional stormwater infrastructure is often designed based on historical rainfall data, typically using return period analysis (e.g., 10-year, 25-year, 100-year storms). If these two outliers represent events more severe than the design storm for which the infrastructure was built, it means the design storm itself might be underestimated. This could lead to frequent exceedance of design capacity, resulting in increased flooding and damage.

Table 4. 1: Annual daily maximum rainfall (mm)

Years	Annual Daily Max (mm)	Years	Annual Daily Max (mm)
1991	32.98	2006	74.4
1992	26.28	2007	38.1
1993	40.09	2008	45.9
1994	34.6	2009	31.5

1995	32.4	2010	44.6
1996	62	2011	50.7
1997	57	2012	61
1998	70	2013	51.00
1999	78	2014	32.05
2000	47	2015	32.90
2001	39.8	2016	46.57
2002	44.7	2017	38.80
2003	45.6	2018	48.00
2004	46.2	2019	50.19
2005	37	2020	46.89

Table 4. 2: A tabular analysis of daily peak rainfall after the Outlier test

Station	Debrezeit
Maximum	78.00
Minimum	26.28
Mean	46.55
Std	13.12
Median	45.75
Skewness	0.812
Q1	37.28
Q3	50.80
IQR	13.52
upper boundary	71.08
Lower boundary	16.99

4.1.2. Rainfall frequency

After addressing any missing data, the maximum daily rainfall was extracted from each thirty-year period spanning from 1991 to 2020. This maximum daily data was then disaggregated into intervals of 60, 120, 180, 240, 300, 360, 420, 480, 540, 600, 55, 60, 660, 720, 780, 840, 900, 960, 1020, 1080, 1140, 1200, 1260, 1320, 1380 and 1440 minutes.

Table 4. 3: Yearly extreme series frequency analysis

Return Periods	**Extreme Rainfall depth (mm)**	
	Gumbel	Log Pearson Type *III*
2	41.21	46.24
5	50.32	56.52
10	56.35	63.30
25	63.97	71.89

50	69.62	78.26
100	75.24	84.59

Table 4. 4: Goodness of Log Pearson Type III and Gumbel method

Distribution	Kolmogorov Smirnov		*Anderson Darling*		*Chi Squared*	
	statics	rank	statics	rank	statics	rank
Log Pearson Type II	0.20221	1	1.3749	1	4.6416	1
Gumbel	0.21544	2	1.6024	2	5.317	2

Table 4. 5: Log Pearson daily heaviest rainfall analysis for Bishoftu city

Return period	Exceedance probability	Skewness Coefficient	Frequency factor	Standard deviation, Std	Y_T	X_T (mm)
2	0.5	0.812	-0.152	10.3	0.367	41.34
5	0.2		0.874	10.3	1.5	51.905
10	0.1		1.553	10.3	2.25	58.898
25	0.04		2.412	10.3	3.199	67.746
50	0.02		3.048	10.3	3.902	74.297
100	0.01		3.681	10.3	4.6	80.817

4.1.3. Development of IDF curve

Following the removal of outliers from the upper limits, the annual daily maximum was derived from the collected rainfall data. Subsequently, the daily maximum for each year was categorized into intervals of 60, 120, 180, 240, 300, 360, 420, 480, 540, 600, 55, 60, 660, 720, 780, 840, 900, 960, 1020, 1080, 1140, 1200, 1260, 1320, 1380 and 1440 minutes. After this breakdown, the mean and standard deviation were calculated for each time interval, and the frequency factor was established using the Gumbel distribution method along with Log Pearson Type III.

Subsequently, the rainfall depth for each selected return period was determined, incorporating the calculated mean, standard deviation, and frequency factor. To derive the intensity, the rainfall depth was divided by the duration of the rainfall, resulting in the construction of an IDF curve, as shown table and below fig 11 with duration plotted on the x-axis and rainfall intensity on the y-axis.

Table 4. Six: Twenty-four-hours rainfall one hour interval rainfall intensity

Duration (min)	Rainfall intensity in mm/hr. for 2,5,10,25, 50, and 100 years return periods					
	2yrs	5yrs	10yrs	25yrs	50yrs	100yrs
	mm/hr.	mm/hr.	mm/hr.	mm/hr.	mm/hr.	mm/hr.
60	46.24	56.52	63.30	71.89	78.26	84.59
120	24.49	29.94	33.53	38.08	41.46	44.81
180	16.88	20.63	23.11	26.25	28.57	30.88
240	12.96	15.84	17.74	20.15	21.94	23.71
300	10.56	12.90	14.45	16.41	17.87	19.31
360	8.93	10.91	12.22	13.88	15.11	16.33
420	7.75	9.47	10.61	12.05	13.11	14.17
480	6.85	8.38	9.38	10.66	11.60	12.54
540	6.15	7.52	8.42	9.56	10.41	11.25
600	5.58	6.82	7.64	8.68	9.45	10.21
660	5.11	6.25	7.00	7.95	8.66	9.35
720	4.72	5.77	6.46	7.34	7.99	8.64
780	4.39	5.36	6.00	6.82	7.42	8.02
840	4.10	5.01	5.61	6.37	6.93	7.49
900	3.84	4.70	5.26	5.98	6.51	7.03
960	3.62	4.43	4.96	5.63	6.13	6.63
1020	3.43	4.19	4.69	5.33	5.80	6.27
1080	3.25	3.97	4.45	5.05	5.50	5.95
1140	3.09	3.78	4.24	4.81	5.24	5.66
1200	2.95	3.61	4.04	4.59	4.99	5.40
1260	2.82	3.45	3.86	4.39	4.78	5.16
1320	2.70	3.30	3.70	4.20	4.58	4.95
1380	2.59	3.17	3.55	4.03	4.39	4.75
1440	2.50	3.05	3.42	3.88	4.22	4.56

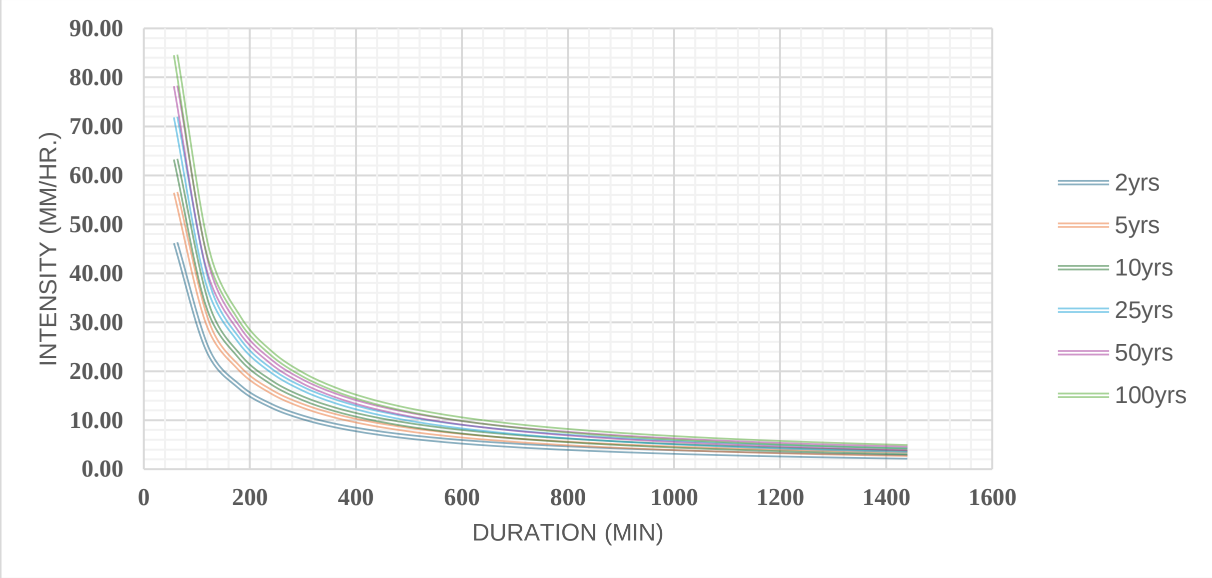

Figure 4. 2: Bishoftu City IDF Curve

The IDF curve, computed for the designated study area of Bishoftu city, holds significance for this research; consequently, using 24-hour rainfall data at one-hour intervals, the IDF curve for the study area has been established. For this analysis, 10 years of 24-hour rainfall intensity have been utilized

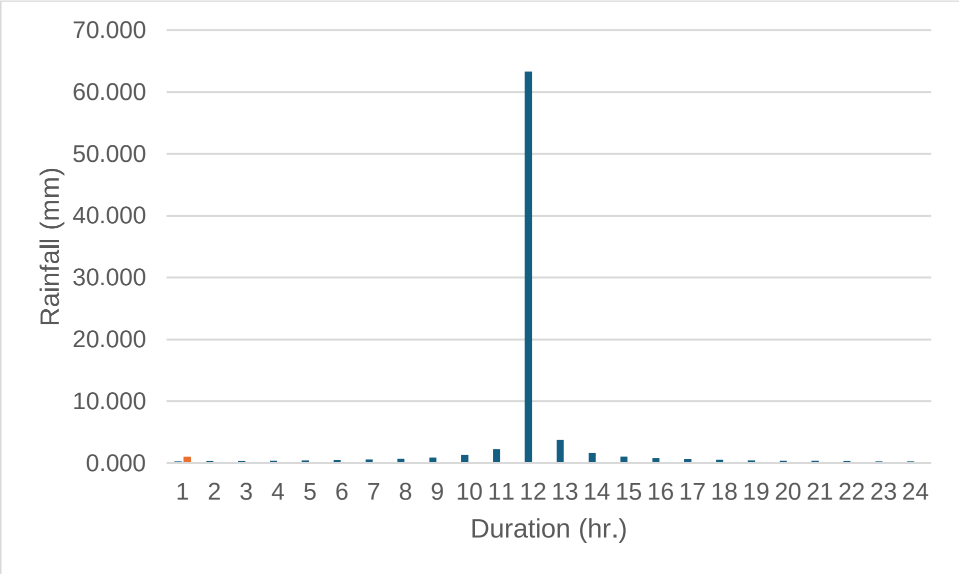

Figure 4. 3: Synthetic Design storm

4.2. Modeling the Discharge and the Capacity Drainage Network

In response to the identified issue of stormwater flooding, a thorough evaluation and modelling have been carried out in the study area. Urban stormwater drainage systems are designed based on various criteria to improve their efficiency in safely directing urban runoff into waterways, while preventing siltation or scouring.

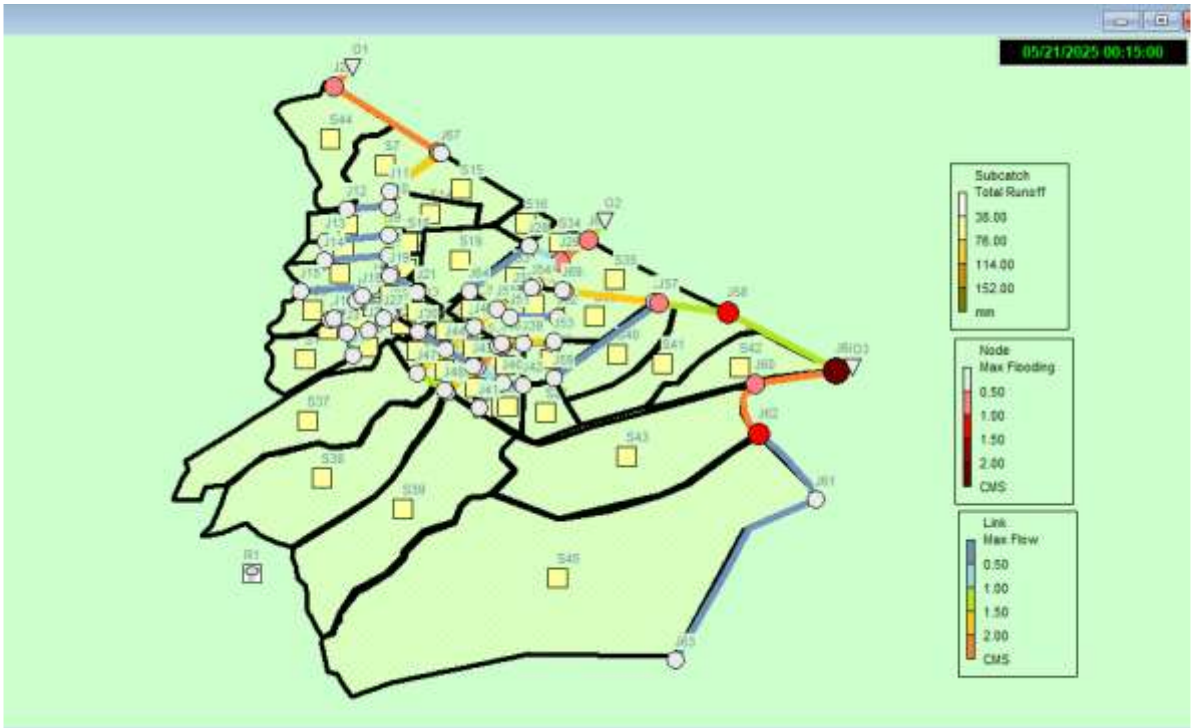

Figure 4. 4: Study area Drainage Network

However, flooding on asphalt surfaces, walkways, and within residential neighborhoods has emerged as a major concern in the city. As a result, modelling has been performed to analyses the capacity and functionality of these drainage systems. As depicted in the map above, the study area is divided into 45 sub-catchments, with water flow originating from building, pavements, agricultural fields, open spaces, and green areas. The designated study area consists of 45 sub-catchments, 69 Junctions, 69 conduits, 3 outfalls and 1 rain gauge station

The area of the overall sub-catchments is 695.13hectares mostly characterized by impervious surfaces. Of this total area, 150.37 hectares, representing 21.63%, drain to outfall 1; 110.53 hectares, or 15.90%, drain to outfall 2; and 434.23 hectares, accounting for 62.46%, drain to outfall three. The overall length of the simulated conduit that drains all sub-catchments is 14.93

Kilometers. The US EPA SWMM 5.2.4 model simulation result in the study area indicates that the total runoff from the entire subctahments is 97.34 m3/sec,

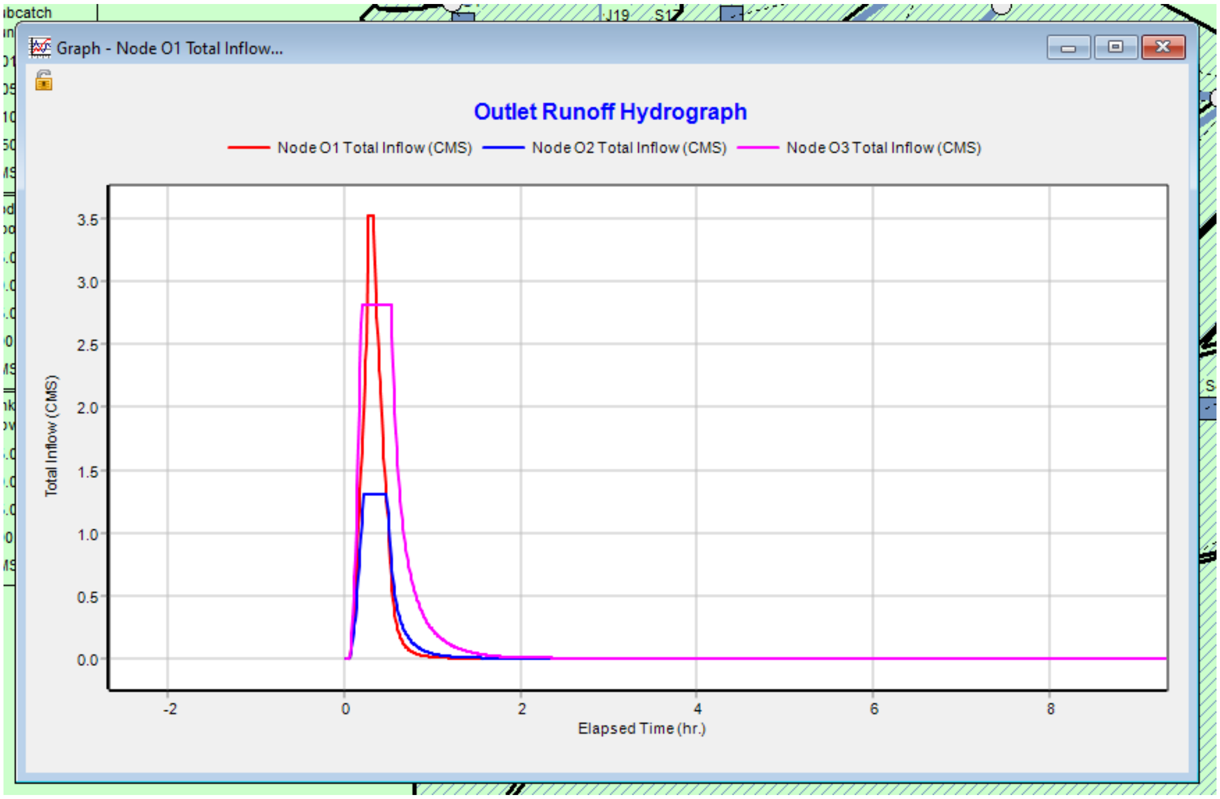

Figure 4. 5: Outfall Runoff Hydrograph

The simulation indicates that among the 69 conduits, 32exhibited surcharged results. Consequently, it is determined that approximately 46% of the stormwater drainage system in the study area has exceeded its capacity to effectively drain the area.

As illustrated in figure 4.10, the surcharged J62 begins beneath the Burka Bishoftu Mekane Jesus Church.

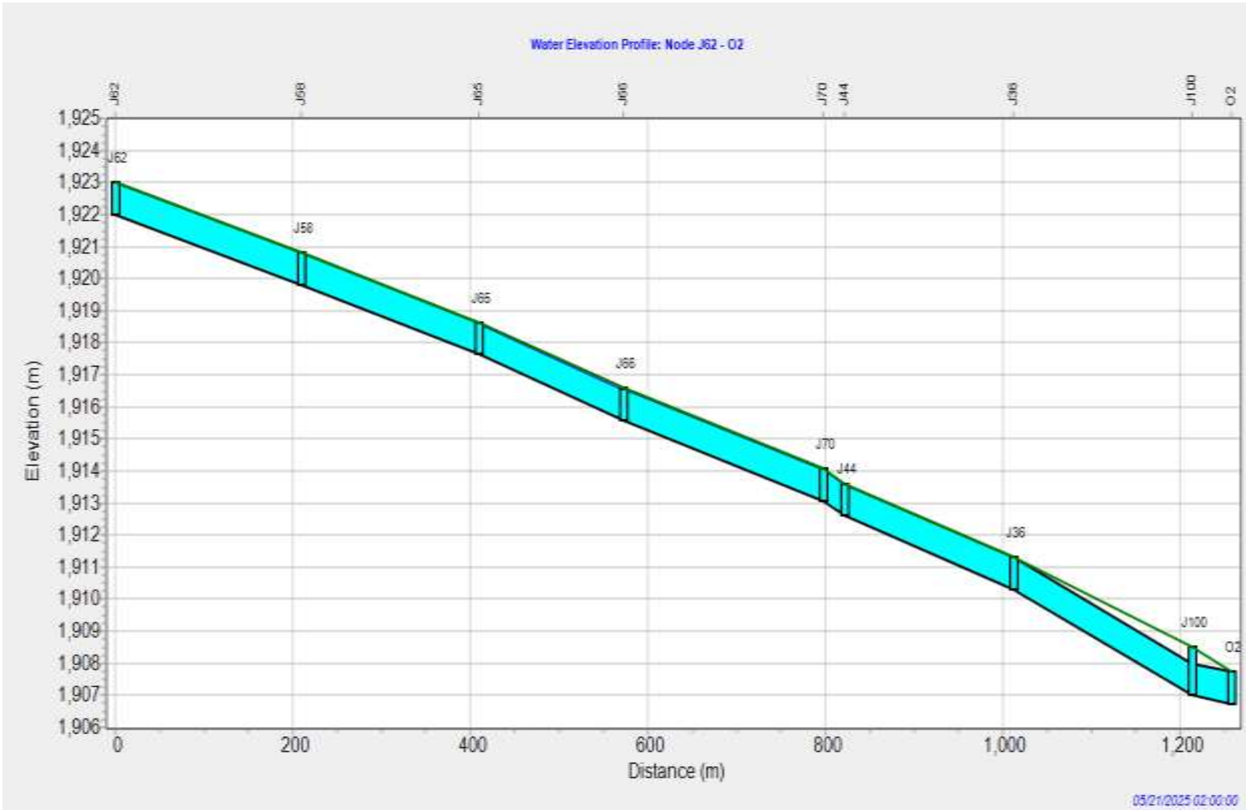

Figure 4. 6: Sample water Profile from around Mekane Jesus Church

All junctions modelled are in a rectangular, and it is assumed that there are no energy losses in the manholes. Additionally, the model incorporates boundary conditions to represent different types of water loads, such as infiltration or fixed water levels. Precipitation is integrated into the model by linking each sub-catchment to the rainfall time series. The overall performance of the network is influenced by infiltration rates and the average production of water flow.

Figure 4.6 illustrates the hydrographs corresponding to each outlet for the design storm (10-year rainfall), where each hydrograph depicts the cumulative inflow at each outlet. A rainfall duration of 6 hours is taken into account for the simulation.

4.3. Rational Formula and SWMM Comparison

In the course of comparison, the simulated peak flow demonstrates very good correlations with the values derived from the rational formula in the catchments. The quality of the fit was further evaluated by graphing the simulated peak flow against the calculated values, as illustrated in in below table and figure 4.14 below.

Table 4. 6: Rational and SWMM Result

Sub-Catchment	Area (ha)	C	I (mm/hr.)_10yr	Rational formula Q (CMS)	SWMM Q (CMS)	Difference
S1	14.6	0.85	63.304	2.184	2.06	-0.124
S2	1.29168	0.77	63.304	0.175	0.44	0.265
S3	9.94	0.77	63.304	1.347	1.83	0.483
S4	10.1	0.75	63.304	1.333	2.46	1.127
S5	6.66	0.75	63.304	0.879	1.79	0.911
S6	7.23	0.75	63.304	0.954	2.1	1.146
S7	14.1	0.81	63.304	2.010	2.56	0.550
S8	5.03	0.81	63.304	0.717	1.05	0.333
S9	3.1	0.76	63.304	0.415	0.95	0.535
S10	0.77	0.77	63.304	0.104	0.28	0.176
S11	0.8	0.77	63.304	0.108	0.28	0.172
S12	3.3	0.74	63.304	0.430	0.94	0.510
S13	3.33	0.76	63.304	0.445	1.15	0.705
S14	9.16	0.76	63.304	1.225	1.6	0.375
S15	12.31	0.82	63.304	1.776	1.75	-0.026
S16	3.84	0.81	63.304	0.547	1.02	0.473
S17	2.44	0.71	63.304	0.305	0.61	0.305
S18	2.8	0.73	63.304	0.360	1	0.640
S19	16.76	0.73	63.304	2.153	2.68	0.527
S20	5.26	0.56	63.304	0.518	1.15	0.632
S21	3.78	0.71	63.304	0.472	0.97	0.498
S22	3.84	0.72	63.304	0.487	1.16	0.673
S23	3.36	0.72	63.304	0.426	1.12	0.694
S24	3.27	0.75	63.304	0.432	1.18	0.748
S25	3.46	0.74	63.304	0.451	0.97	0.519
S26	4.37	0.71	63.304	0.546	1.17	0.624
S27	8.18	0.73	63.304	1.051	2.12	1.069
S28	6.33	0.73	63.304	0.813	1.52	0.707
S29	3.12	0.76	63.304	0.417	0.97	0.553
S30	6.64	0.55	63.304	0.643	2.05	1.407
S31	8.27	0.74	63.304	1.077	1.59	0.513
S32	4.44	0.77	63.304	0.602	1.24	0.638
S33	10.24	0.74	63.304	1.334	3.19	1.856
S34	3.74	0.75	63.304	0.494	1.23	0.736
S35	15.93	0.81	63.304	2.271	4.86	2.589
S36	0.29	0.58	63.304	0.030	0.1	0.070
S37	39.67	0.79	63.304	5.515	2.66	-2.855
S38	40.85	0.69	63.304	4.960	4.75	-0.210

S39	55.45	0.7	63.304	6.831	9.36	2.529
S40	15	0.82	63.304	2.165	2.06	-0.105
S41	25.25	0.79	63.304	3.510	2.8	-0.710
S42	20.98	0.81	63.304	2.991	2.63	-0.361
S43	65.24	0.84	63.304	9.644	6	-3.644
S44	22.813	0.81	63.304	3.252	2.79	-0.462
S45	187.8	0.68	63.304	22.474	11.15	-11.324
Sum	695.13			**90.87**	**97.34**	6.467

The values of both simulated and calculated runoff exhibited a correlation, coefficient 0.886477 deemed acceptable. A correlation coefficient of 0.886477 indicates a very strong positive linear relationship between the results of the SWMM and the rational formula. This shows the great relation between runoff result generated by SWMM and rational formula. The results of the comparison suggested that the model's structure and parameters aligned with the runoff production pattern, confirming that the model in question was appropriate for simulating storm runoff in the study area.

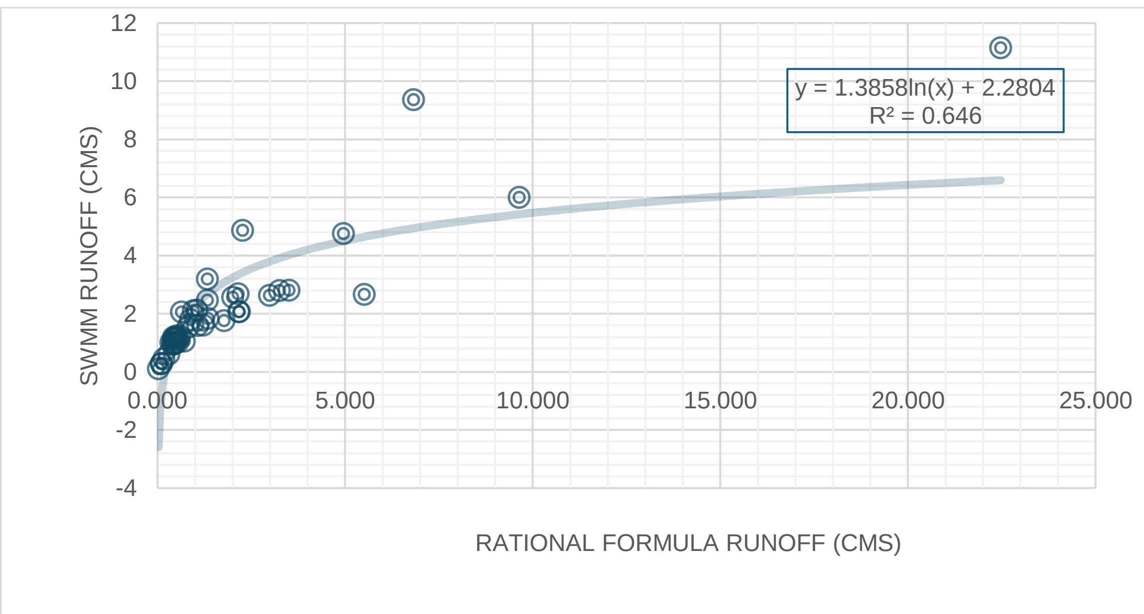

Figure 4. 7: Correlation indicating Value

In the process of comparing models, the alignment between calculated and simulated values can be assessed through graphical methods. This is achieved by comparing the calculated and simulated peak discharge, as well as the rising and falling limbs.

The highest values derived from the model study and rational methods are presented in table 4.7 above. These values have been illustrated in graphical format for the purpose of comparison and are displayed in figure 4.9 below.

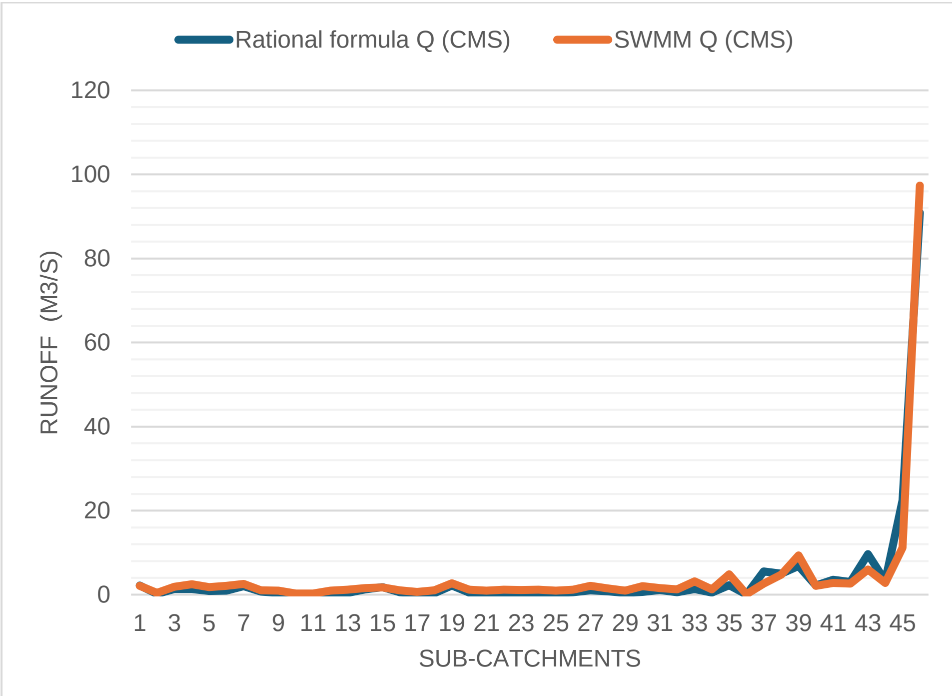

Figure 4. 8: Graph indicating Runoff rational formula and SWMM

The US EPA SWMM 5.2.4 model was utilized for an urban catchment in Bishoftu city, with the rational method employed for comparative analysis. Calculations indicate that the total runoff from the entire subctahments according to SWMM is 97.34 m^3/sec, while the rational method yields a runoff of 90.87 m^3/sec.

SWMM is a dynamic, physically based on model that simulates rainfall-runoff and pollutant transport in urban areas over a range of timescales. SWMM explicitly models various hydrological processes such as interception, depression storage, infiltration (using methods like Horton, Green-Ampt, or Curve Number), and surface routing (using kinematic wave or dynamic wave equations). It can also account for spatial variability in land use, soil types, and topography by dividing the catchment into subctahments, each with its own characteristics. This detailed representation of losses and routing mechanisms means that SWMM typically produces a more realistic and often higher peak runoff, as it accounts for the attenuation and delay of flow that the rational formula largely ignores. For instance, the rational formula calculates peak flow using the equation Qp = CIA. This equation inherently assumes that the entire area contributes to runoff at the peak intensity and that losses are constant and represented solely by C. SWMM, on the other hand, simulates the time-varying infiltration and storage, which can significantly reduce the effective rainfall contributing to runoff and spread out the hydrograph, thus lowering the peak

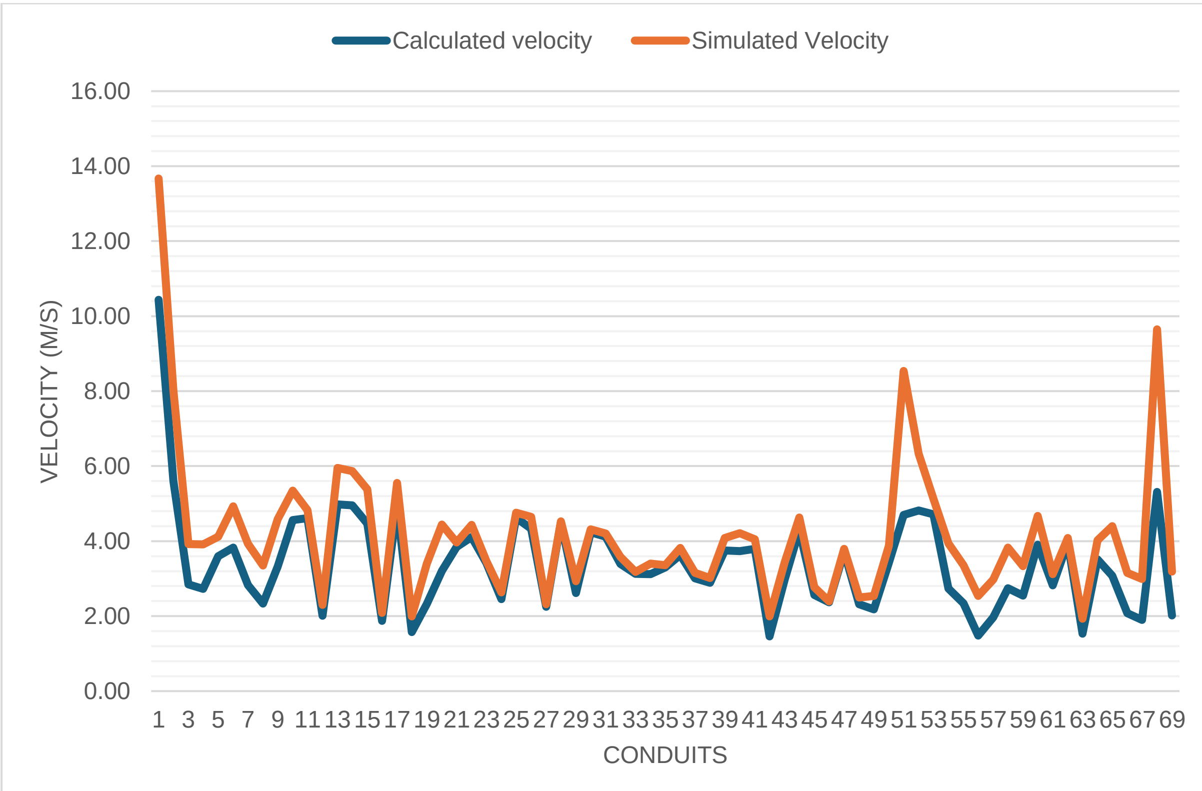

Figure 4. 9: Comparison of simulated and calculated velocities

However, correlation coefficient of 0.91 between the velocity calculated using the Manning equation and the velocity simulated by SWMM signifies a robust positive linear relationship between these two variables. This implies that as the velocity derived from the Manning equation rises, the velocity predicted by SWMM also generally increases, and conversely. The strength of this correlation is deemed strong due to the value being relatively near to 1.0, which denotes a perfect positive linear correlation.

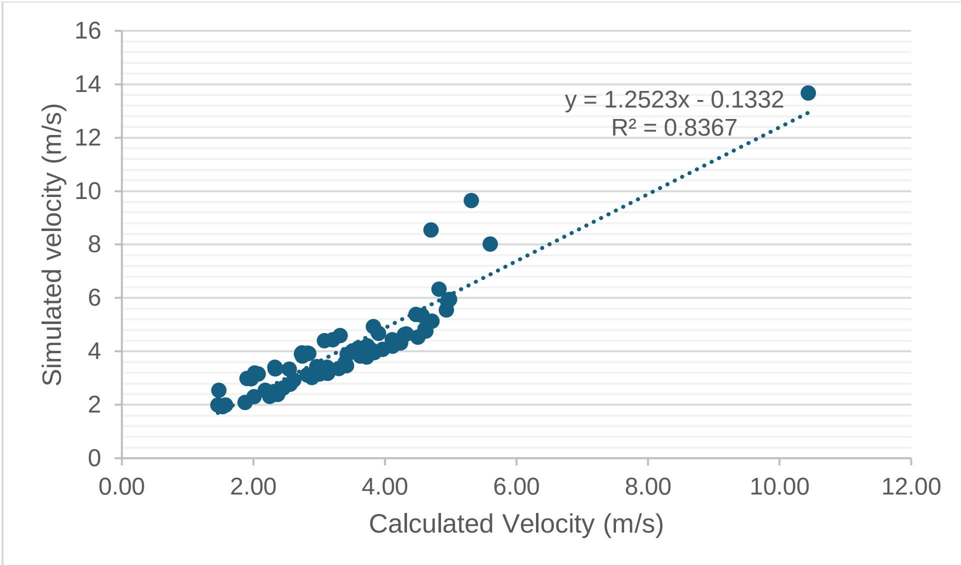

Figure 4. 10: Correlation indicating value

4.4. Challenges in Stormwater Drainage System Management

During the evaluation, it was noted that certain drainage systems were constructed without adequate slope, leading to the observation of backflow and standing water in the Dinkitu area, particularly around the real estate and above it, specifically near Shawel's supermarket and Birhanu Aweke's Grinding mill. In this vicinity, despite the horizontally laid drainage ditches not being filled with solid waste, backflow and standing water were still observed in these particular drainage ditches. Furthermore, at Getshet, from the main asphalt road heading south along the cobblestone road near Jafar shop, backflow and standing water in the horizontal drainage ditches within the residential area were also noted.

Furthermore, in certain residential zones where necessary, drainage ditches were not installed. As a result, the residential area located below Jafar shop, which spans 53 hectares, lacks any drainage ditch. Additionally, along the primary asphalt road extending from Inova through Getshet and Lemlem to the military camp, approximately 1.5 kilometers in length, there is an absence of longitudinal drainage. Consequently, stormwater from the upper catchment area, through the residential drainage systems, floods the main asphalt road at this location.

In nearly every study area, the majority of drainage ditches that were designed with sufficient capacity are currently facing overflow or surcharge because of obstruction caused sediment accumulation. This problem is especially noticeable in the Ziqual, Lemlem, Dinkitu, and adjacent of Inova regions.

Therefore, based on the evaluation carried out in the study area, the difficulties associated with managing the stormwater drainage system in the city are attributed to insufficient provision of stormwater drainage facilities, poor stormwater drainage facilities management, unsuitable design and construction of the existing drainage system.

4.5. Flooding Vulnerable Area

Bishoftu city is flanked by two mountains to the north and south. As a result, at the border of the former Dukem town, water from both the south and north converges into the lowland located at the centre of the western part of Bishoftu city. After collecting at this point, the floodwater flows eastward towards the Mojo River and Cheleka Lake, moving at a slower pace compared to the floodwaters that come from the northern and southern highland areas.

The elevation difference between the western and eastern sides of the city is slight at the centre, particularly when contrasted with the elevation changes between the northern and southern parts of the city and its centre. Consequently, stormwater surges in rapidly from the north and south, gathering at the centre where the city borders the former Dukem town. Due to the minimal elevation difference from west to

To the east, the stormwater subsequently moves at a reduced speed towards Chelelека Lake and Mojo River. Consequently, due to the significant disparity between the speed at which the stormwater arrives at the site and the rate at which it flows away after rainfall, the stormwater remains pooled at this location for an extended period. Thus, the Inova, Getshet, and Dambi regions have been recognized as the most susceptible to flooding within the city

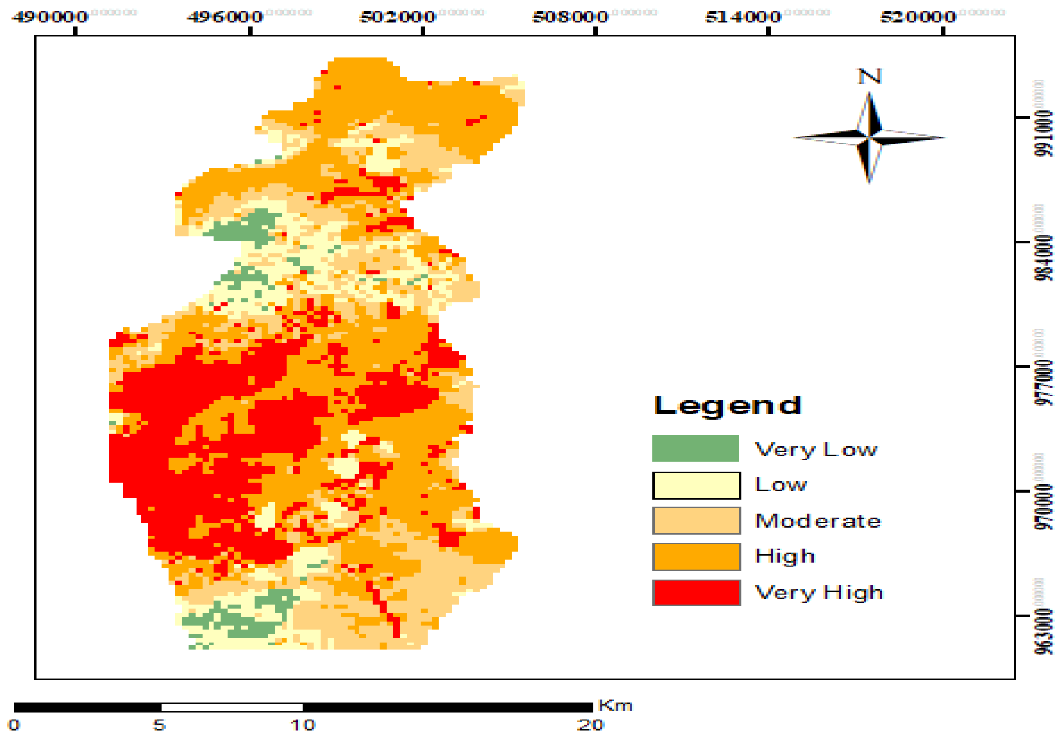

Figure 4. 11: Map of Most Flood Vulnerable Area

4.6. Introduction of LID

The percentages of peak runoff reduction achieved through LID scenarios in SWMM were calculated and analysed to assess their effectiveness in reducing peak runoff (Table 4.12). The runoff reduction percentages for vegetative Bioretention, Rain Barrel from approximately 6.52% to 44.34% across all sub-catchments, with significantly higher reduction percentages (observed in sub-catchments S34, S5, and S32, depending on LULC. Conversely, the lower runoff reduction percentages of 6.52%, 7.20, and 7.51% were recorded in sub-catchments S38, S35 and S1, respectively, due to their designation as highly built up area. In summary, LID measures can effectively reduce peak runoff to a certain extent.

Table 4. 7: Comparison of runoff generated before and after lid introduction

Catchment	Area (ha)	SWMM Q (CMS) before LID	After LID	Difference
S1	14.6	2.06	1.545	0.515
S2	1.29168	0.44	0.33	0.11
S3	9.94	1.83	1.3725	0.4575
S4	10.1	2.46	1.845	0.615
S5	6.66	1.79	1.3425	0.4475

S6	7.23	2.1	1.575	0.525
S7	14.1	2.56	1.92	0.64
S8	5.03	1.05	0.7875	0.2625
S9	3.1	0.95	0.7125	0.2375
S10	0.77	0.28	0.21	0.07
S11	0.8	0.28	0.21	0.07
S12	3.3	0.94	0.658	0.282
S13	3.33	1.15	0.805	0.345
S14	9.16	1.6	1.12	0.48
S15	12.31	1.75	1.225	0.525
S16	3.84	1.02	0.714	0.306
S17	2.44	0.61	0.427	0.183
S18	2.8	1	0.7	0.3
S19	16.76	2.68	1.876	0.804
S20	5.26	1.15	0.805	0.345
S21	3.78	0.97	0.679	0.291
S22	3.84	1.16	0.812	0.348
S23	3.36	1.12	0.784	0.336
S24	3.27	1.18	0.826	0.354
S25	3.46	0.97	0.679	0.291
S26	4.37	1.17	0.936	0.234
S27	8.18	2.12	1.696	0.424
S28	6.33	1.52	1.216	0.304
S29	3.12	0.97	0.776	0.194
S30	6.64	2.05	1.64	0.41
S31	8.27	1.59	1.272	0.318
S32	4.44	1.24	0.992	0.248
S33	10.24	3.19	2.552	0.638
S34	3.74	1.23	0.984	0.246
S35	15.93	4.86	3.888	0.972
S36	0.29	0.1	0.08	0.02
S37	39.67	2.66	2.128	0.532
S38	40.85	4.75	3.8	0.95
S39	55.45	9.36	7.956	1.404
S40	15	2.06	1.751	0.309
S41	25.25	2.8	2.38	0.42
S42	20.98	2.63	2.2355	0.3945
S43	65.24	6	5.1	0.9
S44	22.813	2.79	2.3715	0.4185
S45	187.8	11.15	9.4775	1.6725
		97.3	**77.1915**	**20.148**

CHAPTER 5: CONCLUSION AND RECOMMENDATION

5.1. Conclusion

In the study area, it has been understood that the drainage problem is a cause of flooding on public and private properties. To investigate the cause of the problem, modelling of the drainage system in the flood-prone areas and site investigation was done through analyzing hydrologic and hydraulic patterns of the study area

The hydrologic performance of the drainage systems was simulated using SWMM and rational formula. The simulation result shows that the total runoff or discharge from the overall sub-catchments is 97.34 m3/sec, whereas the calculated value is 90.87 m3/sec. SWMM result is relatively high excess runoff values as compared to the rational formula.

Even though the SWMM result, slightly greater than the rational formula result, under the current rainfall conditions, the system responded with serious problems and was not able to drain the generated runoff. Out of 69 conduits in the simulated network, 32 conduits were found their capacity is highly exceeded, and the systems were vulnerable for flooding and surcharges atop the nodes were considerable in most drainage systems.

Therefore, this drainage problem is because of:

- Inadequate structure provision, which is the hydraulic capacity of the drainage structures, is less than the design discharge;
- Absence of drainage facilities where required; and
- Siltation and blockage the drainage structures with sand and waste material

5.2. Recommendation

The drainage problem is seen as a significant challenge for the newly being expanded built up area on the upstream of the study area. When plan to expand the urban area, it is recommended that the urban expansion should be accompanied with drainage adequate drainage facilities taking into consideration the amount of runoff from the current land use, and the potential land use given the substantial investments made in various building projects, it is essential to prioritize drainage-related concerns. The recommendations outlined below have been derived from this study.

1. Design of the structures;
    - All consideration, such as appropriate design method which depends on the catchment area, variability of climate, future settlement of people, expansion of urbanization and other factors shall be taken into account during the detail design of the drainage facilities so as the structures capacity shall accommodate the design flood;
    - In case if the problem occurs and the city administration shall to take action to keep the serviceability of the road, the rehabilitation needs to be supplemented by the detail design to alleviate the problem permanently with low cost;
    - Integrated urban stormwater drainage or LID should be introduced
    - From south direction mountains Since the velocity of runoff is high across the contour, it is need to provide drop structure to decrease flow velocity;
2. Continuous Monitoring of the drainage facilities;
    - Stormwater drainage systems must be established in locations where such facilities are completely or partially lacking; for instance, between Getshet and Sunshine, there is no internal drainage ditch present;
    - Additionally, from Inova to Abay Hotel along the primary asphalt road, there is an absence of longitudinal stormwater drainage; thus, it is essential to implement longitudinal drainage in this area;
    - Continuous monitoring of the drainage facilities is required to take timely action where unexpected problem encounter that may create risk on the people, road and surrounding environment; and
    - Periodically, cleaning of the drainage facilities is also required to prevent of clogging of the drainage system.

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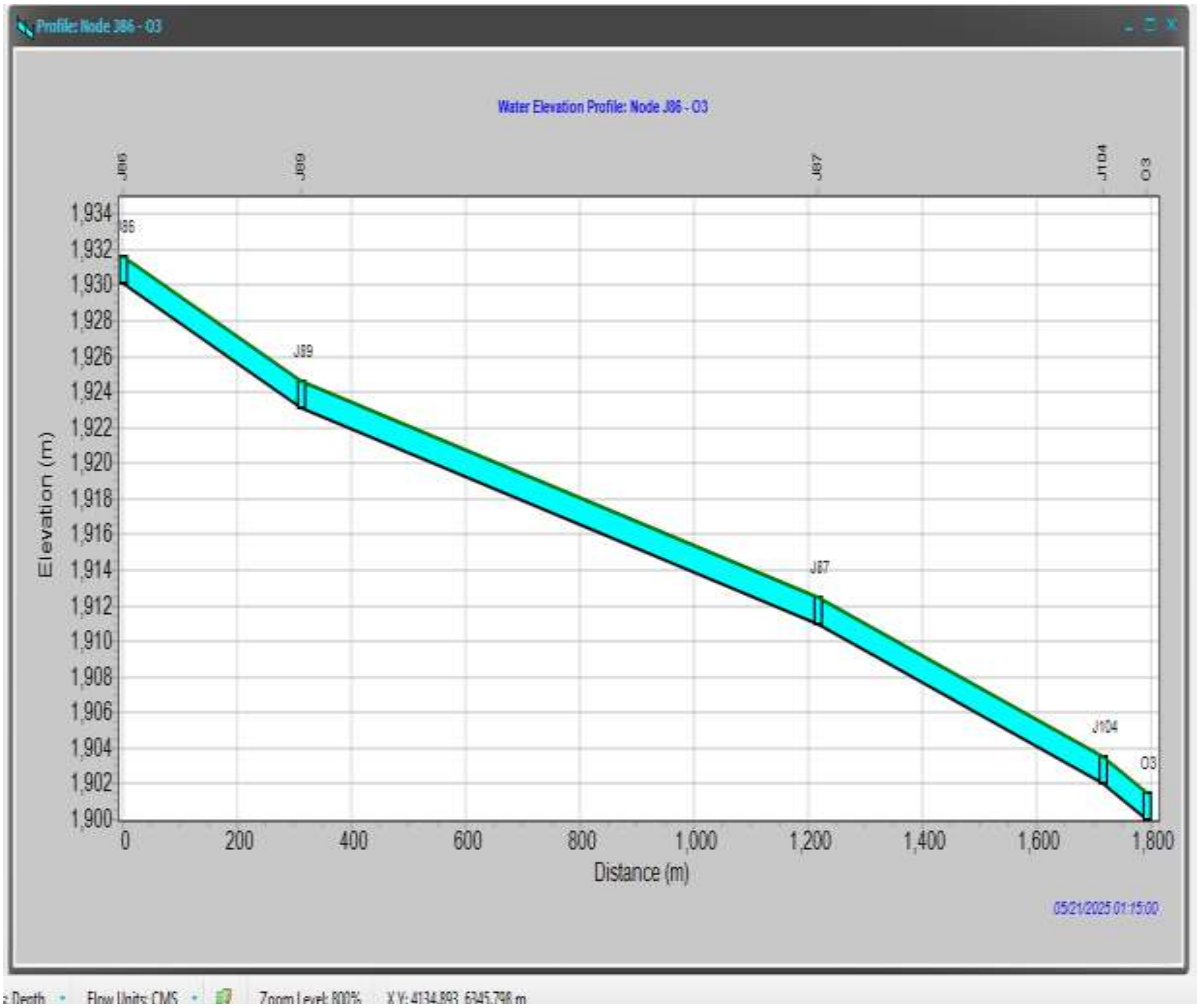
Appendix

Annex A: Water Profile Plot from Above Military Camp to Outlet 3



Annex B: Runoff Coefficient for Various Land use type

Table 5-6: Recommended Runoff Coefficient C for Various Land Uses

Description of Area	Runoff Coefficients
Business: Downtown areas	0.70-0.95
Neighborhood areas	0.50-0.70
Residential: Single-family areas	0.30-0.50
Residential: Multi units, detached	0.40-0.60
Residential: Multi units, attached	0.60-0.75
Suburban	0.25-0.40
Residential (0.5 hectare lots or more)	0.30-0.45
Apartment dwelling areas	0.50-0.70
Industrial: Light areas	0.50-0.80
Industrial: Heavy areas	0.60-0.90
Parks, cemeteries	0.10-0.25
Playgrounds	0.20-0.40
Railroad yard areas	0.20-0.40
Unimproved areas	0.10-0.30

(Source: Hydrology, Federal Highway Administration, HEC No. 19, 1984)

Table 5-7: Coefficients for Composite Runoff Analysis

Surface	Runoff Coefficients
Street : Asphalt	0.70-0.95
Concrete	0.80-0.95
Drives and walks	0.75-0.85
Roofs	0.75-0.95

(Source: Hydrology, Federal Highway Administration, HEC No. 19, 1984)

Annex C: Sample drainage line data

S. No	X	Y	Z	Invert Level	Excavation Depth	Distance		
1	493 435.1468	967 250.7380	1931	1930.2	0.80		Depth	0.800
2	493 441.3971	967 258.5441	1931	1930.1	0.92	10.0	B	0.6
3	493 447.6473	967 266.3501	1931	1930.0	1.03	10.0	T	0.6
4	493 453.8976	967 274.1561	1931	1929.9	1.15	10.0	distance	172.15
5	493 460.1479	967 281.9622	1930	1929.7	0.26	10.0	fall	2
6	493 466.3982	967 289.7682	1930	1929.6	0.38	10.0	SLOPE	0.012
7	493 472.6484	967 297.5742	1929	1929.5	-0.50	10.0		
8	493 478.8987	967 305.3802	1927	1929.4	-2.39	10.0		
9	493 485.1490	967 313.1863	1928	1929.3	-1.27	10.0		
10	493 491.3993	967 320.9923	1925	1929.2	-4.15	10.0		
11	493 497.6495	967 328.7983	1926	1929.0	-3.04	10.0		
12	493 503.8998	967 336.6043	1926	1928.9	-2.92	10.0		
13	493 510.1501	967 344.4104	1927	1928.8	-1.81	10.0		
14	493 516.4004	967 352.2164	1927	1928.7	-1.69	10.0		
15	493 522.6506	967 360.0224	1927	1928.6	-1.57	10.0		
16	493 528.9009	967 367.8285	1928	1928.5	-0.46	10.0		
17	493 535.1512	967 375.6345	1929	1928.3	0.66	10.0		
18	493 541.4015	967 383.4405	1929	1928.2	0.77	10.0		
19	493 542.7467	967 385.1206	1929	1928.2	0.80	2.2		
						172.15		

$$Distance = ((x_2 - x_1)^2 + (y_2 - y_1)^2)^{0.5}$$

Annex D: All sub-catchment land use land cover and Runoff Coefficient

sub-catchment	Land use type	Area(ha)	Runoff coefficient, C	Total area (m^2)	A*C (ha)	Weighted C
S1	Built_Up_Area	14.60	0.85	14.60	12.41	0.85
S2	Built_Up_Area	1.29	0.85	0.87	1.29	0.77
	Coble_Stone	0.16	0.44			
	Gravel Road	0.06	0.6			
	Green Area	0.01	0.375			
	Open land	0.15	0.56			
S3	Built_Up_Area	8.90	0.85	11.36	8.72	0.77
	Coble_Stone	0.52	0.44			
	Farmland	0.79	0.35			
	Gravel Road	0.20	0.6			
	Green Area	0.03	0.375			
	Open land	0.93	0.56			
S4	Built_Up_Area	6.92	0.85	9.99	7.48	0.75
	Coble_Stone	0.61	0.44			
	Farmland	0.00	0.35			
	Gravel Road	0.00	0.6			
	Green Area	0.27	0.375			
	Open land	2.20	0.56			
S5	Built_Up_Area	4.64	0.85	6.66	4.96	0.75
	Coble_Stone	0.87	0.44			
	Green Area	0.05	0.375			
	Open land	1.10	0.56			
S6	Built_Up_Area	5.16	0.85	7.15	5.35	0.75
	Coble_Stone	0.82	0.44			
	Green Area	0.27	0.375			
	Open land	0.90	0.56			
S7	Asphalt Road	0.62	0.95	14.22	11.47	0.81
	Built_Up_Area	11.54	0.85			
	Coble_Stone	0.69	0.44			
	Open land	1.37	0.56			
S8	Built_Up_Area	5.64	0.85	6.58	5.33	0.81
	Coble_Stone	0.16	0.44			

	Gravel Road	0.13	0.6			
	Open land	0.65	0.56			
S9	Built_Up_Area	2.28	0.85	3.23	2.45	0.76
	Coble_Stone	0.15	0.44			
	Gravel Road	0.03	0.6			
	Green Area	0.00	0.375			
	Open land	0.76	0.56			
S10	Built_Up_Area	0.55	0.85	0.76	0.58	0.77
	Coble_Stone	0.04	0.44			
	Open land	0.17	0.56			
S11	Built_Up_Area	0.63	0.85	0.81	0.62	0.77
	Coble_Stone	0.15	0.44			
	Gravel Road	0.02	0.6			
S12	Built_Up_Area	2.24	0.85	3.30	2.43	0.74
	Coble_Stone	0.35	0.44			
	Green Area	0.12	0.375			
	Open land	0.58	0.56			
S13	Built_Up_Area	2.49	0.85	3.36	2.57	0.76
	Coble_Stone	0.17	0.44			
	Gravel Road	0.02	0.6			
	Green Area	0.09	0.375			
	Open land	0.60	0.56			
S14	Built_Up_Area	6.59	0.85	9.47	7.22	0.76
	Open land	2.89	0.56			
S15	Asphalt Road	1.07	0.95	12.39	10.16	0.82
	Built_Up_Area	9.80	0.85			
	Coble_Stone	0.00	0.44			
	Open land	1.52	0.56			
S16	Asphalt Road	0.09	0.95	3.83	3.10	0.81
	Built_Up_Area	3.29	0.85			
	Coble_Stone	0.20	0.44			
	Open land	0.25	0.56			
S17	Built_Up_Area	1.44	0.85	2.42	1.72	0.71
	Coble_Stone	0.22	0.44			
	Green Area	0.17	0.375			
	Open land	0.59	0.56			
S18	Built_Up_Area	1.80	0.85	2.82	2.06	0.73

	Coble_Stone	0.29	0.44			
	Green Area	0.09	0.375			
	Open land	0.63	0.56			
S20	Built up area	**3.46**	**0.85**	5.26	3.77 0.56	
	Coble_Stone	0.16	0.44			
	Green Area	0.86	0.375			
	Open land	0.78	0.56			
S19	Built_Up_Area	11.12	0.85	16.69	12.19	0.73
	Coble_Stone	1.15	0.44			
	Green Area	0.98	0.375			
	Open land	3.45	0.56			
S21	Built_Up_Area	3.13	0.85	5.46	3.88	0.71
	Coble_Stone	0.25	0.44			
	Gravel Road	0.44	0.6			
	Green Area	0.29	0.375			
	Open land	1.35	0.56			
S22	Built_Up_Area	2.26	0.85	3.77	2.72	0.72
	Coble_Stone	0.49	0.44			
	Gravel Road	0.19	0.6			
	Green Area	0.12	0.375			
	Open land	0.72	0.6			
S23	Built_Up_Area	2.50	0.85	3.90	2.81	0.72
	Coble_Stone	0.43	0.44			
	Gravel Road	0.07	0.6			
	Green Area	0.19	0.375			
	Open Land	0.70	0.56			
S24	Built_Up_Area	2.71	0.85	3.94	2.96	0.75
	Coble_Stone	0.25	0.44			
	Gravel_Road	0.35	0.6			
	Green_Area	0.02	0.375			
	Open_Land	0.61	0.56			
S25	Built_Up_Area	2.16	0.85	3.25	2.41	0.74
	Coble_Stone	0.18	0.44			
	Gravel_Road	0.19	0.6			
	Green_Area	0.12	0.375			
	Open_Land	0.60	0.56			
S26	Built_Up_Area	2.11	0.85	3.44	2.44	0.71

	Coble_Stone	0.48	0.44			
	Green Area	0.17	0.375			
	Open Land	0.68	0.56			
S27	Built_Up_Area	2.92	0.85	4.39	3.21	0.73
	Coble_Stone	0.42	0.44			
	Green Area	0.16	0.375			
	Open Land	0.90	0.56			
S28	Built_Up_Area	5.22	0.85	8.13	5.93	0.73
	Coble_Stone	0.65	0.44			
	Gravel Road	0.05	0.6			
	Green Area	0.40	0.375			
	Open Land	1.81	0.56			
S29	Built_Up_Area	4.70	0.85	6.33	4.81	0.76
	Coble_Stone	0.41	0.44			
	Green Area	0.14	0.375			
	Open Land	1.08	0.56			
S30	Built_Up_Area	2.05	0.56	3.10	1.70	0.55
	Coble_Stone	0.18	0.44			
	Gravel Road	0.19	0.6			
	Green Area	0.12	0.375			
	Open Land	0.55	0.56			
S31	Built_Up_Area	4.58	0.85	6.64	4.91	0.74
	Coble_Stone	0.69	0.44			
	Green Area	0.20	0.375			
	Open Land	1.17	0.56			
S32	Built_Up_Area	6.13	0.85	8.28	6.38	0.77
	Coble_Stone	0.19	0.44			
	Gravel Road	0.50	0.6			
	Green Area	0.09	0.375			
	Open Land	1.36	0.56			
S33	Built_Up_Area	3.07	0.85	4.51	3.34	0.74
	Coble_Stone	0.69	0.44			
	Open Land	0.75	0.56			
S34	Built_Up_Area	7.34	0.85	10.25	7.69	0.75
	Coble_Stone	0.89	0.44			
	Green Area	0.14	0.375			
	Open Land	1.89	0.56			
S35	Asphalt Road	0.20	0.95	3.72	3.01	0.81
	Built_Up_Area	2.93	0.85			

	Gravel Road	0.21	0.6			
	Green Area	0.05	0.375			
	Open Land	0.34	0.56			
S36	Built_Up_Area	0.05	0.85	0.28	0.16	0.58
	Gravel Road	0.07	0.6			
	Green Area	0.07	0.375			
	Open Land	0.10	0.56			
S37	Built_Up_Area	35.06	0.85	39.67	31.34	0.79
	Coble_Stone	0.09	0.44			
	Farmland	3.02	0.35			
	Gravel Road	0.13	0.6			
	Open Land	0.27	0.56			
	Scrubland	1.10	0.28			
S38	Built_Up_Area	27.67	0.85	40.85	28.19	0.69
	Coble_Stone	0.03	0.44			
	Farmland	10.52	0.35			
	Open Land	0.26	0.56			
	Scrubland	2.38	0.28			
S39	Built_Up_Area	38.39	0.85	55.45	38.82	0.7
	Coble_Stone	0.29	0.44			
	Farmland	15.28	0.35			
	Open Land	0.68	0.56			
	Scrubland	0.81	0.28			
S40	Built_Up_Area	13.82	0.85	15.00	12.30	0.82
	Coble_Stone	0.31	0.44			
	Green Area	0.06	0.375			
	Open Land	0.81	0.56			
S41	Built_Up_Area	21.73	0.85	25.25	19.95	0.79
	Coble_Stone	0.18	0.44			
	Gravel Road	0.40	0.6			
	Green Area	0.04	375			
	Open Land	1.05	0.56			
	Scrubland	1.86	0.28			
S42	Asphalt Road	0.84	0.85	20.98	16.99	0.81
	Built_Up_Area	17.12	0.85			
	Gravel Road	0.57	0.6			
	Green Area	0.16	0.375			
	Open Land	2.28	0.56			
S43	Asphalt Road	0.10	0.85	55.45	47.20	0.84

	Built_Up_Area	55.06	0.85			
	Coble_Stone	0.72	0.44			
S44	Asphalt Road	0.98	0.85	26.09	21.13	0.81
	Built_Up_Area	22.06	0.85			
	Farmland	0.78	0.35			
	Gravel Road	0.21	0.6			
	Open Land	1.70	0.56			
	Scrubland	0.35	0.28			
S45	Asphalt Road	0.65	0.85	187.81	127.71	0.68
	Built_Up_Area	121.71	0.85			
	Coble_Stone	1.20	0.44			
	Farmland	52.46	0.35			
	Open Land	2.85	0.56			
	Scrubland	8.94	0.28			

Annex E: Assessment of the current Drainage System

S/No. 1	Flow direction		length	slope	Roughness coefficient (n)	area	perimeter	runoff	velocity
	From	To	(m)	(m/m)		m^2	m	m^3	m/s
1	J1	J2	426.35	0.021	0.013	0.9	3.6	3.98	8.44
2	J2	J3	247.25	0.023	0.013	1.05	3.8	5.15	4.90
3	J3	J4	146.16	0.009	0.013	1.35	3.9	4.97	3.68
4	J4	J5	46.17	0.009	0.013	1.425	3.95	5.30	3.72
5	J5	J6	189.51	0.0103	0.013	1.47	3.98	5.92	4.03
6	J6	J7	13.23	0.0103	0.013	0.98	2.94	4.51	4.60
7	J7	J8	441	0.0149	0.013	1.5	4	9.10	6.07
8	J8	J9	339.83	0.022	0.013	1.50	4	23.68	5.92
9	J9	J10	119.36	0.025	0.013	0.9	3.6	3.24	4.84
10	J10	J12	292.06	0.020	0.013	0.7	2.7	3.10	4.43
11	J11	J12	394	0.013	0.013	1.2	3.2	5.53	4.61
12	J12	J13	118	0.017	0.013	1.2	3.8	5.51	4.59
13	J13	J15	100	0.02	0.013	1.2	3.8	6.05	5.04
14	J14	J15	361	0.011	0.013	1.35	3.9	5.27	3.94
15	J15	J17	113.88	0.007	0.013	1.2	3.8	3.70	3.08
16	J16	J17	353	0.0048	0.013	0.9	3.6	1.90	2.11
17	J17	J19	163.18	0.024	0.013	1.43	3.95	23.67	5.99
17	J18	J19	236	0.0038	0.013	0.92	2.92	2.01	2.19
18	J19	J20	85.88	0.033	0.013	1.44	3.96	10.22	7.10
19	J21	J22	110.17	0.020	0.013	0.9	3.6	3.88	4.31
20	J23	J4	179.47	0.017	0.013	0.8	2.8	3.44	4.30
21	J22	J24	47.49	0.008	0.013	1.35	3.9	4.58	3.39
22	J24	J25	186.77	0.034	0.013	0.9	3.6	5.09	5.65
23	J26	J5	168.76	0.018	0.013	0.9	3.6	3.70	4.11
25	J27	J28	10.28	0.014	0.013	1.35	3.9	5.98	4.43
26	J28	J6	168.61	0.023	0.013	1.38	2.92	8.05	5.83
27	J29	J31	754	0.011	0.013	1.35	3.9	5.41	4.01
28	J30	J31	200	0.012	0.013	1.35	3.9	5.67	4.20
29	J31	J32	544	0.009	0.013	1.8	4.2	7.59	4.21
30	J32	J33	9.86	0.014	0.013	1.8	4.2	8.18	4.54
31	J33	J34	365.34	0.016	0.013	2.1	4.4	12.50	5.95
32	J34	O1	152.52	0.016	0.013	2.1	4.4	12.63	6.01