ADDIS ABABA UNIVERSITY

COLLEGE OF TECHNOLOGY AND BUILT ENVIRONMENT

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

WATER SUPPLY AND ENVIRONMENTAL ENGINEERING STREAM

Investigation of Urban Drainage in Addis Ababa, the case of Addis Ketema Sub city

By:

ABDURAHIMAN NESIRU

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

Addis Ababa University, Ethiopia

Dec, 2025

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Advisor Name: Prof. Geremew Sahilu

Addis Ababa University, Ethiopia

Dec, 2025

By: Abdurahiman Nesiru Email- Abdurahimannesiru@gmail.com

DECLARATION

I, the undersigned, declare that the thesis presented in this document is my own original work and has not been given a degree in any other university, and that all sources of material used for the thesis has been duly acknowledged, following the scientific guidelines of the institute.

Name: Abdurahiman Nesiru

Signature:

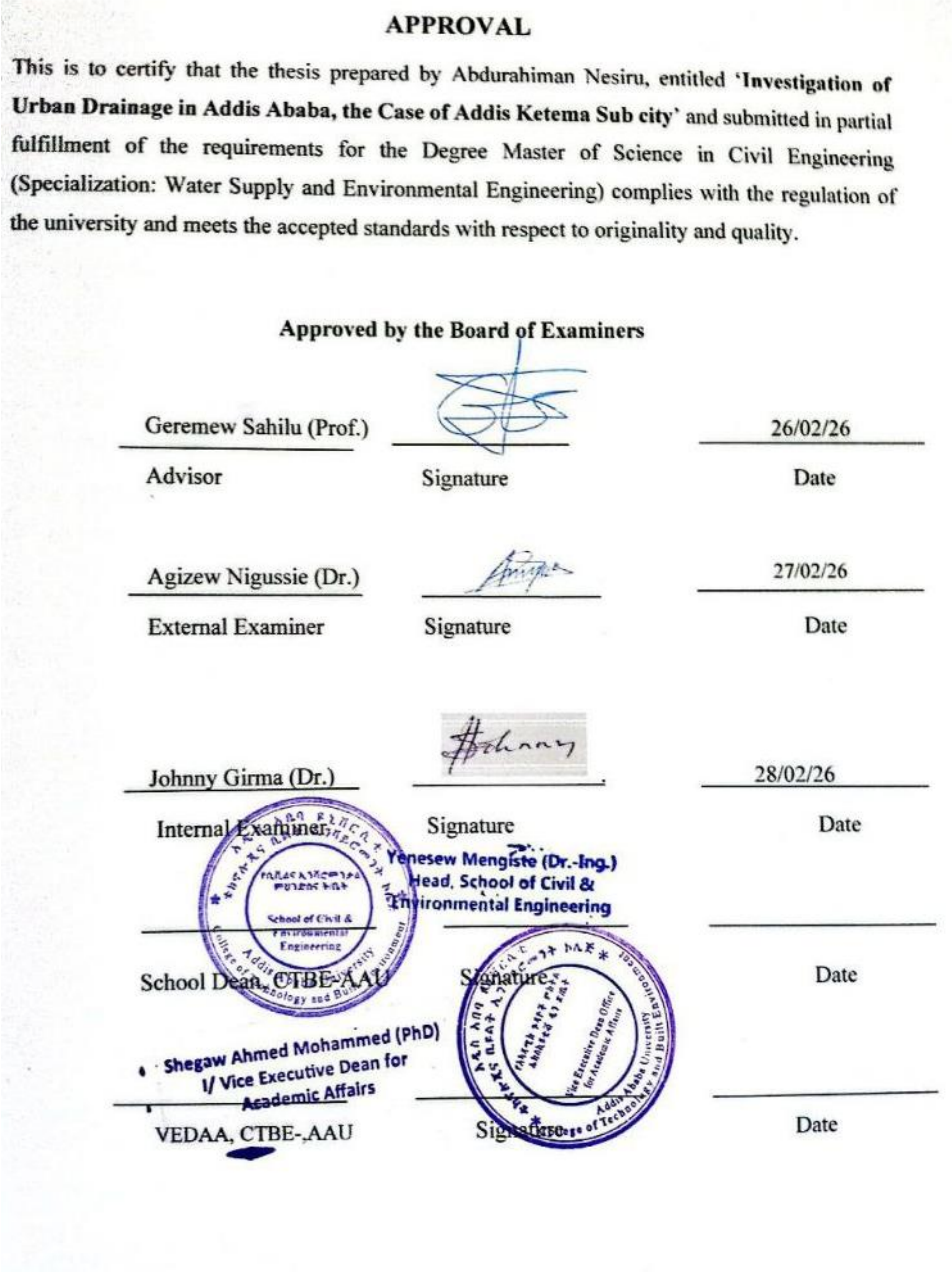
Date:26/02/2026

This dissertation has been submitted for examination with my approval as the student's Advisor

Name: Prof. Geremew Sahilu

Signature:

Date: 26/02/2026



APPROVAL

This is to certify that the thesis prepared by Abdurahiman Nesiru, entitled '**Investigation of Urban Drainage in Addis Ababa, the Case of Addis Ketema Sub city**' and submitted in partial fulfillment of the requirements for the Degree Master of Science in Civil Engineering (Specialization: Water Supply and Environmental Engineering) complies with the regulation of the university and meets the accepted standards with respect to originality and quality.

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Geremew Sahilu (Prof.)		26/02/26
Advisor	Signature	Date
Agizew Nigussie (Dr.)		27/02/26
External Examiner	Signature	Date
Johnny Girma (Dr.)		28/02/26
Internal Examiner	Signature	Date
Yenesew Mengiste (Dr.-Ing.) Head, School of Civil & Environmental Engineering		
School Dean, CTBE-AAU	Signature	Date
Shegaw Ahmed Mohammed (PhD) I/ Vice Executive Dean for Academic Affairs		
VEDAA, CTBE-,AAU	Signature	Date

ACKNOWLEDGMENT

First and foremost, thanks to the Allah for allowing me to be alive and blessing s throughout my research work to complete the research successfully. Next, I would like to express my deep and sincere gratitude to my research supervisor, Prof. Geremew Sahilu for the continuous support of my MSc thesis study, for his patience, motivation, and immense knowledge. He has thought me the methodology to carry out the research. It was a great privilege and honor to work and study under his guidance. I am extremely grateful for what he has offered me. Finally, my thanks go to all people who have supported me to complete the research work directly or indirectly.

My special thanks also go to my mother, she always inspire me to focus on my thesis work. She also covers my household as well as social works which is very difficult for her as she spend the whole day at work.

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

ACRONYMS

GDEM	Global Digital Elevation Model
GIS	Geographical Information System
GPS	Global Positioning System
LID	Low Impact Development
NAM	National Metrological Agency
NASA	National Aeronautics and Space Administration
NSE	Nash-Sutcliffe Efficiency
SWMM	Strom Water Management Model

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ABSTRACT

Urban areas often face storm water management issues because the impermeable surfaces of roads, buildings and pavements do not allow for infiltration and lead to increased volumes of runoff and flooding. Cities in Ethiopia experience intense seasonal rainfall such as Addis Ababa. Addis Ketema sub city us particularly prone to flooding during the rain moths. This research investigates the significant causes of flooding and assesses peak runoff for multiple sub-catchments in various return periods. By applying storm water management model, it also assesses the existing drainage system performance under varied runoff conditions.

Hydrological study, filed survey and questionnaire were used to gather the required data. The study utilized software and L-moment methods to choose an appropriate probability distribution that would help determine runoff with a return period of 2, 5, 10, and 25 years. Further analysis consisted of IDF curves preparation, geospatial mapping with Google earth pro and hydrologic model. The study indicates that the primary causes of flooding are sediment build-up, lack of proper maintenance of drainage facilities, and widely spaced or blocked inlets as well as under sized drainage pipes. .Runoff during 2, 5, 10, and 25 years return periods was majorly traced from sub catchments such as Awtobis Tera, ZZZ Foreign Employments, Addis Ketema Young Center and other vulnerable points. These areas produce a significant amount of runoff at junctions and conduits of drainage systems. Simulation storm water management models should incorporate low impact development interventions including green roofs, permeable pavements and bio-retention cells as per the study recommendation based on the result. The updated model outputs show that low impact development measures reduce peak discharge and flooding, which supported by comparative hydrographs and tabulated results. The hydrologic and hydraulic components of the model were calibrated and validated using observed rainfall and flow measurements. Performance metric show strong reliability, including Nash-Sutcliffe value of0.74 during calibration and 0.86 during validation and coefficient of determination (R^2) values of 0.55 and 0.94 respectively. Overall, these results confirm that the model provides dependable predictions for storm water behavior within the study area.

Key words: Urban flood management, urban drainage, Storm water model management, Low impact development, Return period

1 INTRODUCTION

1.1 Background

Flood is high-water stage in natural or artificial channel which water overflows onto typically dry land. Natural flooding that occurs in the flood tide plains of gutters owing to temporal and spatial variations in rushes and runoff in the catchment area. Flooding from the urban drainage system due to the effect of inhibition of water inflow(Tucci, 2018).

Urbanization increases the extent of impervious surfaces such as compacted soil and rock, which alters hydrological response times and consequently elevates flood risk(Feng et al., 2021). When requires flows and moves toward stream channels contributing to short term surface runoff and potentially leading to soil erosion and flooding (Dingman-P, 2015). Moreover, with the development of urban areas, drainage overflow often occurs with inadequate regard drainage capacity (Ayda et al., 2024). Flooding can be exacerbated, damage to drainage infrastructure can occur due to reduced infiltration, increased runoff, and shorted runoff concentration times. The negative effects of flooding on urban infrastructure can be alleviated using flood event modelling and formulating appropriate action plans.(Muschalla & Ostrowski, 2002).

There are multiple approaches to flood modeling. A popular approach, involving model construction for a single rainfall event, is the event based approach. The process of using the model to determine the behavior of the flood and its consequences takes place next(Filipova et al., 2019). Runoff models reproduce the transformation of rainfall into runoff within a catchment. Mainly, they are important for forecasting runoff quantity and timing which is an essential aspect of water resource management and flood prediction (Hsu et al., 2009). Many models are empirical, using statistical relationships between a rainfall input and a runoff output. According to Kumar et al.,(2023), this technique is often used in flood forecasting. As worldwide urbanization levels are on the rise which often result in the severe consequences one the hydrological regime. Therefore, more attention needs to storm water management (Feng et al., 2021).

Due to climate change and rapid urban expansion, many cities in Ethiopia, in recent years, have frequently suffered floods. Moreover, unplanned and unsuitable urban storm water management practices have aggravated the magnitude and frequency of floods (Jemberie & Melesse, 2021). Urban flooding is increasingly becoming a concern due to impervious surfaces and poor drainage systems in a rapidly developing area (Ahiablame et al., 2012). Low-impact development (LID) techniques have emerged as a suitable and decentralized storm water management solution to these problems. LID interventions like bio-retention cells, green roofs, and permeable pavements help to restore natural hydrologic processes through increased infiltration, reduction in runoff volume, and improved water quality. According to Bibi & Kara, (2023), their implementation reduces peak flows and alleviates local flooding. Incorporating Low Impact Development (LID) into urban drainage modeling, especially with a tool such as the Storm Water Management Model (SWMM), allows a thorough performance evaluation under different spatial and climatic conditions (Jemberie & Melesse, 2021).

This study examines the application of selected LID measures in critical urban catchments and evaluates their effectiveness in reducing flooding using SWMM simulations. Addis Ketema sub city, one of the flood prone areas in Addis Ababa, is selected as the case study area. Accordingly, the aim of this research is to identify the cases and magnitude of the flooding and to propose appropriate mitigation measures for the area.

1.2 Statement of problem

Insufficient urban storm water drainage is one of the major concerns frequently raised by the residents in many Ethiopian towns, and the problem is becoming more serous with rapid rate of urbanization. Road drainage system consists of the pavements and water-handling structures such as surface drains, shoulders, ditches, and culverts. These components must be properly designed, constructed, and maintained to ensure effective drainage.

In the absence of drainage, the pavement is likely to fail. An inadequately designed drainage system can lead to storm water re-entering the roadway or obstructing its drainage. When there’s too much water left on the pavement as traffic loading continues, it causes the pavement to develop potholes, cracks, etc. This lowers the quality and usefulness of the road network and increases travel time, affecting economic activity negatively.

In Addis Ababa, several newly constructed roads quickly deteriorated after their inauguration. The gravel pavements have failed on account of climatic conditions, insufficient drainage, and stipulations and material quality of the bituminous construction. Typical defects in pavement include cracks, ruts, potholes, and depressions. A road safety and pavement performance problem exists due to poor drainage. Traffic hazards and damage that shorten the service life of pavements occur because of such distresses. Engineers must first try to understand the causes of drainage-related pavement problems before implementing any maintenance or rehabilitation measures.

Due to rapid urbanization, the Addis Ketema Sub city is highly built up which in turn increases storm water runoff during rain seasons. Surfaces that do not absorb moisture such as corrugated iron roofs and concrete paving generate large volumes of storm water. Flooding often occurs due to the area's inadequate drainage channels that make storm water and wastewater inundate and become stagnant water. Moisture level of pavement layer gets increased, which weakens the structural strength and accelerates pavement damage during and after rainfall. Rainwater drainage issues causes traffic jams, erosion, environmental damage, and disruption of daily activities.

One of the most densely populated and economically significant sub-cities of Addis Ababa is Addis Ketema. Given its serious drainage problems, this study seeks to examine the causes of flooding and subsequent failures of drainage system in Addis Ababa, especially in Addis Ketema Sub City.

The following picture shows the extent of the problem

Figure1-1: The images on the right and left show that the drainage system is surcharged and, and flowing over the surface

Figure 1-2: improper conveyance of open drainage flow in the left and right images

Figure 1-3: The image on the right and left show solid waste accumulation in open drainage system

1.3 Objectives

1.3.1 General Objective

The overall objective of the study is to investigate the causes and effects of urban flooding within Addis Ketema sub city, Addis Ababa.

1.3.2 Specific Objectives

- To find out the major causes for urban flooding in Addis Ketema sub city.
- To assess the maximum runoff for specific return periods in the sub-catchment area.
- To determine peak discharges at junction nodes along selected conduits for multiple return periods through hydraulic modeling
- To suggest enhancements to the maintenance of drainage systems and flood mitigation practices.

1.4 Research Questions

The following research questions were developed in alignment with the specific objectives of the study;

1. What are the dominant factors contributing to urban flooding in Addis Ketema sub city?
2. How much peak runoff is generated from the sub catchment under different hydrologic return periods?
3. What are the peak discharges at the junction nodes along selected conduits for the selected return periods, as determined through hydraulic modeling?
4. What improvements can be made in the maintenance and management of the drainage system to reduce the flood risks in Addis Ketema sub city?

2 LITERATURE REVIEW

2.1 Flood

A flood occurs when water inflow exceeds the capacity of natural or artificial channels along a stream, leading to overtopping of dikes banks. The magnitude of a flood can be described by parameters such as flood discharge, flood elevation, and flood volume which are essential in the hydrological design of various inflow control structures(Chow et al., 1988). Floods are considered among the most hazardous natural events and their frequency and potential impacts are expected to increase in the future due to the factors such as population growth, urbanization, land subsidence and the effects of climate change. The characteristics and occurrence of floods are influenced by rainfall patterns, the physical and hydrological properties of the drainage basin and the management of land and water resources within the catchment(Qari et al., 2014). While flooding is the natural process that occurs that periodically and provides important ecological benefits, human settlements within floodplains and development practices that constrain river flow often result in floods being perceived primarily as destructive events (Alfieri et al., 2016).

Global warming and climate change have intensified the frequency of rainfall events and urban flooding, making them serious challenges in many countries worldwide(Jemberie & Melesse, 2021; Miller & Hutchins, 2017). Recent studies also confirm a probability of more extreme rainfall events occurring in the future as a result of climate change(Alexander, 2016).

In Ethiopia, many urban areas have increasingly experienced recurring flooding events in recent years, often resulting in widespread social and economic impacts. Flood events can generally be categorized into three major types based on their primary causes, urban flash floods triggered by intense rainfall, riverine floods caused by the overflow of nearby rivers into adjacent areas and valley floods generated by seasonal streams (Hassan et al., 2023).

2.1.1 Urban flooding

Urban areas in the central region are frequently affected by two orders of floods, which are fluvial flooding and urban flooding (Yu & Coulthard, 2015). On the one hand, fluvial floods are related to heavy rain in the stream catchment and rising stream water situations, which affects lower neighboring riverbanks and reduces urban sewer systems drainage capacity. On the other hand, urban flooding results from overflow generation from extreme rainfall in an urban area. In some circumstance, the most severe flooding in this region caused by combining these two flooding types which require critical understanding for developing adaptation measures(Le et al., 2024). Urban flooding is one of the complex problems that numerous cities are dealing with to ensure suitable development. Upgrading the drainage structure system is a standard and extensively habituated result to determine the threat of flooding in numerous areas (Paule-Mercado et al., 2017). More drainage systems are built grounded underground sewer systems and aim to release the immense quantity of storm water out of the system as expeditiously as possible (Le et al., 2024).

2.1.2 Causes of urban flooding

Natural hydrological processes have always had an influence on the movement of water over land, plus extreme rain events would have caused flooding naturally. Human activities have been causing negative effects due to land surface modification. As a result, flood activity has been getting worse. In earlier times, due to lack of technical knowledge, people relied on trial and error to improve drainage systems. Many early urban drainage structures have been developed but were inefficient in controlling storm water runoff and avoiding flooding on a frequent basis. Around the early 20th century, urban drainage was treated as an important part of public infrastructure, leading to a more systematic and scientific method in it. This period saw engineers developing more reliable hydraulic principles and standardized design procedures.

Significant progress was observed during the second half of the twentieth century, when regulatory frameworks, engineering standards and advanced computational tools become widely adapted in the United States, Europe and other regions. The introduction of computer aided modeling greatly enhanced the capacity to design, analyze and manage urban drainage systems. Alongside these innovations, increasing attention to environmental quality, public health and flood risk shifted the focus of urban drainage from merely removing water to developing integrated systems that balance engineering, ecological and societal needs (S.K Garg, 2005).

Urban flooding one of the major contributors to global flood related losses and may occur due to pluvial, fluvial or littoral flooding process. Among these flooding types, pluvial flooding triggered by intense rainfall combined with inadequate storm water systems or limited surface infiltration has often received less attention in comparison to riverine and coastal floods. This is neglect partly due to the assumption that pluvial floods are easier to manage and typically causes lower drainage. However, emerging evidence indicates that pluvial flooding can generate substantial cumulative impacts over time. Moreover, the likelihood and severity of such events are increasing as a result of changing hazard conditions, aging urban infrastructure, rapid urbanization and the influence of global climate change. Consequently, urban flooding associated with extreme rainfall has become an important area of research, particularly within the hydrologic and hydraulic modeling community(Eldho et al., 2018).

2.1.3 Impact of urban flooding

Cities function as major social and economic centers, relying on wide range of essential services such as energy supply, water provision, transportation, housing, education and employment. When flooding occurs in urban areas, these interconnected systems face substantial disruption resulting in wider impacts on communities and city functions(Hammond et al., 2015). Flood events cause both direct and indirect damage to building, infrastructure and public facilities, while also affecting human health and livelihoods. They can interrupt commercial activities damage agricultural lands and threaten cultural heritage sites and natural ecosystems(Koc et al., 2019). Severe soil erosion triggered by flood waters may weaken foundations, compromise areas inaccessible. In this context, the implementation of well-designed and adequately maintained urban drainage systems is widely regarded as a long term measures to reduce recurrent flooding in cities such as Accra (Hammond et al., 2015).

Urban flooding is recognized a major contributor to global flood related losses and may result from pluvial, fluvial or coastal processes. Of these, pluvial flooding generated by intense rainfall combined with limited inflation or insufficient storm water drainage has historically received less attention because it was assumed to result in relatively minor impacts. However, accumulating evidence indicates that pluvial flooding contribute significantly long term damage and is becoming more frequent due to increased exposure, aging drainage structure, rapid urbanization and climate induced changes in rainfall intensity (Eldho et al., 2018).

A flood may generally be described as the accumulation or overflow of water on to terrain that is normally dry. Flooding may develop from heavy rainfall occurring over a short duration, rom overflow within established channels such as a streams, rivers or ponding in areas when runoff is unable to drain effectively. Floods are time dependent events and can occur gradually or suddenly with little warning. They are commonly classified according to duration slow onset, rapid onset and flash floods or according to their geographic context including coastal, arroyo, riverine and urban flooding. In cities the predominance of impervious surfaces such as buildings, paved roads and sidewalks reduces natural infiltration. When intense rainfall exceeds the capacity of storm water networks or when these systems are blocked or undersized water accumulates on streets, underpasses and low lying areas leading to urban flooding.

Traditionally, gray structure plays an important part in mitigation urban flooding as a necessary storm water drainage system (Bakhshipour et al., 2019). Presently, the conception of exclusively depending on gray structure for rapid-fire storm water drainage is no longer sufficient to meet the conditions of urban flooding control (Li et al., 2024). These practice aims to maximize the objectification of green spaces into urban planning using a combination of engineered measures similar as a green roofs, bio-retention cells, dry creeks, passable pavements and artificial wetlands intending to meet the impacts conditions of the hydrological water cycle for urban development (Liu et al., 2021).

2.2 Flood modeling

Flood modeling is an analytical technique used to assess and limit the effects of water-related disasters. Using models and simulations, it helps researchers and decision makers predict how water will flow across a landscape during a flood. Flood models work like a virtual experiment, investigating what might happen under different arrangements, so planners learn about the risks even before they occur. Flood modeling primarily aims to study flood processes, develop and predicts their effects including water levels, flow dynamics, inundation and the consequent damages. Essential for flood risk assessment and the development of mitigation and adaptation strategies that enhance community resilience(Kumar et al., 2023), such model output is the result.

Rainfall runoff modeling represents an important branch of hydrological modeling that simulates how precipitation is transformed into surface runoff within a catchment. These models help both the magnitude and timing of runoff, making them fundamental tools in water resources management and flood forecasting efforts (Hsu et al., 2009). Selecting an appropriate rainfall runoff model depends on the purpose of the study, the data available and the level of detail required capturing relevant hydrological process (Papaioannou et al., 2017).

A frequently utilized type of rainfall runoff model is the empirical model, which employs statistical relationships between the rainfall inputs and observable runoff outputs. Even though these models are not based on the physical mechanism of runoff generation, they are widely used because they are data-scarce, easily calibrate, and provide reasonable performance in many operational situations. The use of empirical models in flood forecasting, design of storm water systems and water resource planning is common. The model involves collecting hydrological data, estimating model parameters, running the model with rainfall inputs while adjusting the model as necessary and simulating the model are the various activities involved. According to a study led by Moradkhani & Sorooshian, (2008), information generated from the models helps in decision making regarding flood preparedness and allocation of water resources.

2.2.1 Flood frequency analysis

Flood frequency analysis is a statistical approach used to describe the relationship between the magnitude of a flood and the probability that this magnitude will be met or exceeded in any given year. In this context, the return period (T) also known as the recurrence interval represents the average time expected between floods of a particular size. The return period and the probability expedience are inversely related. Therefore a flood with a high probability of occurring has relatively short return period. Although frequency analysis is most commonly applied to annual peak discharges, it can also be performed using main daily flows or flood volumes over selected durations. The method relies on historical stream flow records which are treated as a random sample drawn from a consistent population of flood events. This assumption allows the observed data to be used as an approximation of the underlying probability distribution governing flood occurrence(Chandra Skakhar prased Ojha, 2008).

Despite its important, flood frequency estimation remains a challenging task due to uncertainties in data quality, climatic variability and the difficulty of selecting an appropriate probability distribution (Samantaray & Sahoo, 2020). Frequency analysis is used to forecast flows of different magnitudes and it's particularly effective in regions with consistently hydrological and climatic behavior. Numerous studies have explored which probability distributions are most suitable for modeling annual maximum events. Commonly applied distributions include the General Extreme Value (GEV), Gambel, Log-normal, Weibull 3parameter and Log-Prearson type III distribution (Papaioannou et al., 2017).

For many dataset, determining whether a particular distribution provides a satisfactory fit requires formal goodness of fit testing. These tests assess how closely the observed flood data align with values expected from a theoretical distribution. The decision to reject or accept the null hypothesis that the data follow the assumed distribution is based on whether the test statistics exceeds a critical value at a chosen significance level. Commonly used goodness of fit tests include the Chai square tests, the Anderson darling test and the Kolmogorov-Smirnov test (Samantaray & Sahoo, 2020).

2.2.2 Flood forecasting

Reliable forecasts support the planning and execution of structural and non-structural flood mitigation strategies guide the operation of multi-purpose reservoir particularly in regulating inflow during high water periods and enable timely evacuation or warning of communities at risk. Modern flood forecasting systems rely heavily on hydrological models in combination with geospatial tools. Hydrological models provide a simplified but scientifically grounded representation of the rainfall runoff processes within a catchment, allowing analysts to simulate how incoming precipitation may translate into stream flows. When integrated with GIS, these models offer improved spatial interpretation of catchment characteristics, land cover and drainage networks thereby enhancing the accuracy of forecasts.

Developing a forecasting model requires careful consideration of several elements, including the type and quality of input, selection and calibration of model parameters, choice of model structure, and the temporal and spatial resolution at which forecast are needed. Each of these components affects the reliability of predictions and must be adapted to the particular hydrological setting and management objectives of the study area.

2.3 Previous studies

Huang et al., (2014) proposed a modeling approach for urban drainage channels based on modern hydrologic and hydraulic principles and numerical simulation with SWMM platform. The drainage system is organized relative to its three fundamental components; conduit, node and catchment. Surface runoff generation is accomplished using a non-linear reservoir representation. The saint-Venant equations compute the flow in the drainage network. Based on their research, it was discovered that land subsidence alters the height and functioning of underground drainage structures, which in turn impacts the convenience of flow and pounding at the manholes. The authors observed that SWMM can adequately simulate these real world conditions and thus provide a sound scientific basis.

Kalwani et al., (2017) proposed a SWMM parameter estimation producer that allows limited data conditions. They segregate the model parameters into two groups: physical and hydrological. By using DEM data with GIS based geometric analysis, the physical parameter is derived, whereas the hydrological parameter is estimated from the land cover proportions derived using remote sensing image classification. Infiltration is modeled using Horton's equation that requires only three parameters as focus is on minor input modeling. The water balance principle, and Manning's equation are used for modeling the runoff generation. In areas where SWMM may break down, rendering it ineffective, their findings provide options.

According to Pathak & Chaudhari, (2015) , flooding is not only geographically distinguishable but also noticeably different between urban and rural areas. Urbanization is known to greatly affect floods. Peak flows as well as flood volumes drastically increase in urban literature due to the high degree of imperviousness. The SWMM model was used to represent the Mithi river sub catchment of Mumbai taking the land use variability into consideration. Analyzed the efficiency of LID practices (rain garden).

The study by Jiang et al., (2015) examined the use of SWMM for urban flood forecasting in Dongguan city, southern china. Following the creation of the model with DEM data, they simulated runoff using drainage network maps and sub catchment characteristics for several design storms. The results showed that no flooding occurs for a 1 year return period storm but significant flooding does occur for 2, 5, 10 and 20 years event. The model, while quite capable of predicting channel flows, has an inherent limitation in not routing overland flows which limits its ability to accurately simulate the overland flooding.

Dey & Kamioka, (2007) studied the flooding in the Tenpaku basin in Nagoya, Japan, during a heavy rain of 580 mm over 24 hours (return period >100-year). Their modeling approach which combines hydrologic and hydraulic components generated simulated flood depths that are consistent with observed values. The agreement showed that their approach is reliable when it comes to complex flooding processes in the city, which are difficult to replicate with 1D and 2D models.

Cumulatively above studies show versatility of SWMM for diagnosing drainage system deficiencies, evaluating hydraulic behavior and supporting decision making in urban flood management(Huang et al., 2014). Studies also show that SWMM can be applied in data limited context using parameter estimation techniques enabled by GIS and remote sensing (Kalwani et al., 2017). The framework also provides an assessment of various flood mitigation strategies such as LID practices capable of significantly reducing peak water levels (Pathak & Chaudhari, 2015).

Urban flood research in Ethiopia is in high demand on recent research. In Addis Ababa,(Beshir & Song, 2021) identified the strong factors influencing flooding are unplanned settlement expansion, poor drainage networks and accumulation of solid waste.. Birhanu et al., (2016) utilized GIS-based flood risk mapping to determine the degree of vulnerability in various areas, noting that rapid urban expansion lacking drainage planning could have consequences. Similarly, Moses et al.,(2021) utilized the SWMM model to assess the runoff dynamics and drainage performance in the Bole sub-city, demonstrating how the model can be assessed for urban infrastructure planning in Ethiopia.

The literature suggests that SWMM offers a complete analytical environment which allows the evaluation of the existing drainage systems, identification of critical problem areas and generation of hydraulic profile and time series diagnostic for nodes and links. The usefulness of this model in the areas of flood forecasting, drainage design and other urban water management applications is enhanced by its ability to interact with other modelling tools(Kalwani et al., 2017).

3 MATERIAL AND METHODOLOGY

3.1 Description of the study area

Ethiopia’s capital and largest city, Addis Ababa is located in the geographic coordinates of about $8^0$50’ -$9^0$50 ‘N latitude and $38^0$38’ -$38^0$54‘E longitude. Addis Ababa, the country’s capital, lies on the central Ethiopian highlands. The city had an estimated population of some 3.44 million people in 2017. The notable highlands surrounding Addis Ababa include mount Yerer to the east, and mount Entoto and mount Wechecha to the west with the elevation of.3, 300m above mean sea level. The city has a mild subtropical highland climate because it is at a high altitude.

The annual precipitation in Addis Ababa is highly seasonal. The northern hemisphere summer months (July to September) are the wettest period while November to February is the driest period. Addis Ababa has eleven sub-cities that administer it. Almost a quarter of Ethiopia’s urban people, the city has grown rapidly, making it one of the fastest-growing metropolitan areas in Africa. A city is the heart of Ethiopia’s economic development, contributing nearly half of the national GDP and working towards its vision of attaining a middle-income climate-resilient country. Addis Ababa is covering area about 527km2 with an average 2600m above mea sea level. The highest point is Mount Entoto at approximately 3041m, and the lowest is the Akaki lows at just over 2050.

The density of the population varies markedly among all sub cities. Located in the north-west of Addis Ababa near the city center, Addis Ketema has one of the highest densities of more than 37,000 people per square klometer. The construction of residential buildings in the Addis Ababa metropolitan region is on the up rise (Wledgerima et al., 2017). Building activities are considerably increasing in this region. Addis Ketema is the study area for the research. Geographically, it lies close to center of Addis Ababa. Addis Ketema is bordered by the Gullele to the north, Arada to east, Lideta to the south and Kolfe keranio.

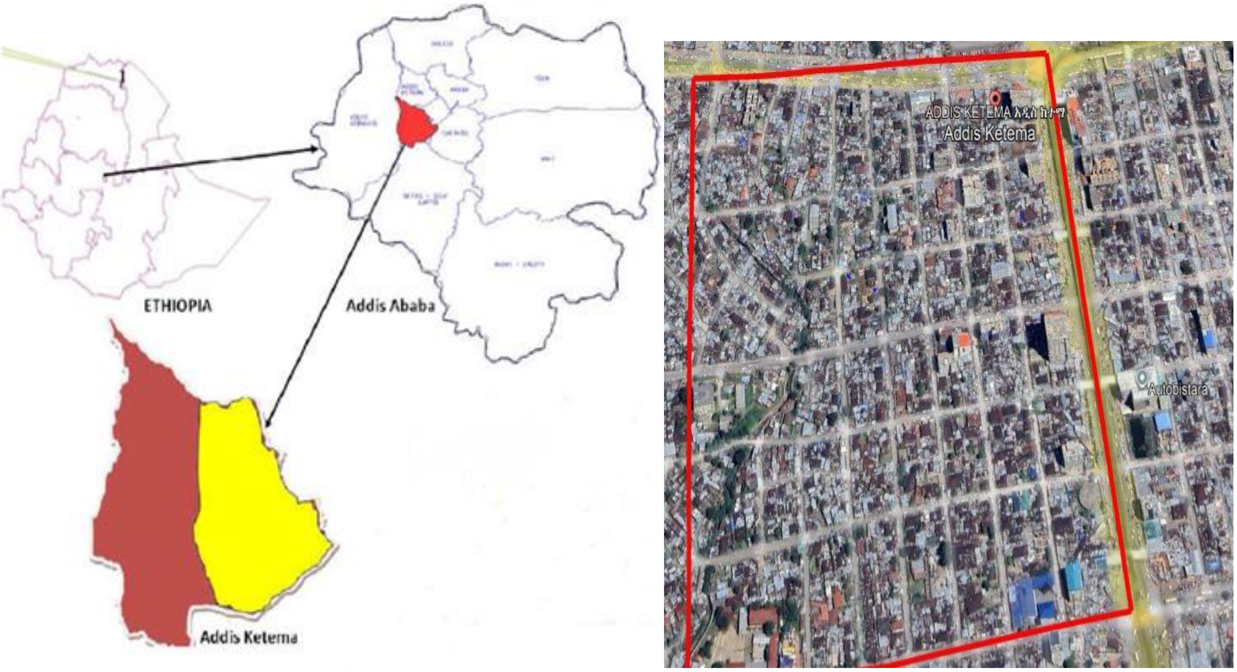


Figure 4-1: Location of the study area

3.1.1 Climate

Addis Ababa   Addis Ababa has “highland subtropical climate” which is mainly due to the high altitude and equatorial proximity. The temperature remains uniform throughout the year, but there are differences in the city as a result of the variations in altitude and wind. June to September is the principal rain season referred to locally as kirmt. It provides the bulk of the annual rainfall. A shorter rainfall period, called Bega, brings extra rain from about mid-February to mid-April. The rest of the months receive less rainfall but with light rains only.

3.2 Data types and source of collection

Through direct observations and surveys conducted in the study area primary data has been obtained. The purpose of these surveys was to assess the physical conditions of the existing drainage lines, including measurement dimensions structural integrity and blockage or malfunction. Field data collection included the use of GPS equipment to take the coordinates and leveling machines to verify ground elevation. To gather information regarding the history, causes and impacts of flooding at the community level, structured questionnaires were also used. The questionnaire was administered on 15 people in Addis Ketema to know its consistency before the full scale administration. Data collectors assisted illiterate respondents with reading the questionnaire which was available in English and Amharic.

The secondary information used in this study consisted mainly of metrological and hydrological records obtained from the National Metrological Agency of Ethiopia (NMA). Rainfall measurements were gathered from several stations distributed across Addis Ababa, including Ayere Tena, Kolfe Keraniyo School, Yekatit23 School, Abyssinia School, Medhaniyalem School, Kality Akaki, Asko, Koebe Techer training college and Entoto. Along with rainfall, additional variables such as stream discharge, evaporation, temperature and relative humidity were incorporated to support the analysis.

Topographic data were prepared using digital elevation method (DEM) which provides a spatial resolution of 30m. To improve the accuracy within the relatively small study area, selected elevation points were cross checked using handheld GPS measurements collected during field visits .land use and Landover information was generated through interpretation of high resolution of Google earth imagery and coordinated by field observation. Based on visible characteristics and ground trothing, the study area was manually classified into categories such as mixed residential neighborhoods, commercial areas, institutional facilities (school and health centers) and various roadway types.

3.3 Questionnaire administration

For collecting quantitative data on flood incidence, duration, perceived causes and damage, a well-structured questionnaire was used. The questionnaire was divided into sections on personal particulars, experience of flooding, mapping parameters and prior research. The responses underwent coding and statistical analysis to confirm their connection to physical flooding data. Out of the total respondents 162 were surveyed; selected using stratified random sampling methods to ensure representation across the different weredas within sub city. Stratification was done on flood prone areas which can be useful for areas where stratification isn't occurring without flooding proficient area.

3.4 Model Setup, Calibration and Validation

3.4.1 Software tool

For this analysis, the Storm Water Management Model (SWMM) was used to conduct the hydrologic and hydraulic analysis. The prepared in Arc GIS spatial configuration of watershed including sub catchments and the drainage network. The model layout setup was done in SWMM using these spatial datasets. Twenty sub-catchments were defined based on the existing land-use pattern, surface slopes, and arrangement of the local drainage system at the project site. The representative physical properties of each sub-catchment have been assigned for realistic simulation of runoff generation and conveyance.

3.4.2 Calibration and Validation

The runoff measurements collected from filed monitoring within the study area was used to perform calibration of SWMM. The observed runoff depths at the main outfall locations were used to calibrate model parameters. Validation expressed using a separate runoff event (not used in calibration) to test whether the model was properly reproducing system behavior for different conditions.

In the calibration process, the percentage imperviousness, manning's roughness coefficient for over land flow, infiltration characteristics, and other sensitivity parameters were continuously changed till simulated response and actual data are in line. The evaluation of model performance was done using well-known statistical measures, including the coefficient of determination (R^2) and Nash-Sutcliffe efficiency (NSE). The indicators were able to assess how well calibrated mode of the hydrologic model simulates actual hydrologic conditions and the prediction accuracy of runoff.

Equations used

$$R^2 = \left(\frac{\sum(O_i - \bar{O})(P_i - \bar{P})}{\sqrt{\sum(O_i - \bar{O})^2 \sum(P_i - \bar{P})^2}} \right)^2$$

Equation3.1

$$NSE = 1 - \frac{\sum(O_i - P_i)^2}{\sum(O_i - \bar{O})^2}$$

Equation3.2

3.5 Rainfall frequency and IDF analysis

To characterize the rainfall behavior of the study area, annual maximum precipitation values were compiled for storm durations of 1 to 24 hours using records from the available rainfall stations. The three probability distributions Gumbel, Log Pearson type III and Log Normal were fitted to the data to find out the best fit for extreme rainfalls at that station.

Each distribution was subject to tests of goodness of fit to check suitability. In particular, the Kolmogorov-Smirnov test, the Chi-Sequa test and the Anderson-Darling test were used to compare how “good” the fit is between the theoretical distribution and the annual maximum. From among the distributions used, the one that gives the best overall fits for each station was chosen and applied to develop intensity duration frequency (IDF) relationships for return periods of 2, 5, 10, 25, 50 and 100 years.

Because the rainfall records were available only at daily resolution, the daily totals were converted to sub daily values using the disaggregation procedure outlined in the ERA (2013) drainage design guideline. This approach enabled the generation of the short duration rainfall intensities required for detailed hydrologic modeling.

Rainfall ratio equation $\mathrm{RRt} = (\frac{t}{24})(\frac{b+24}{b+t})$ Equation 3.3

Intensity calculation $i = \frac{\mathrm{R}}{\mathrm{d}}$ Equation3.4

The resulting IDF curves were then applied in estimating peak runoff using rational method and were also compared with the intensity values provided in the Addis Ababa City Roads Authority (AACRA) design manual to evaluate consistency with existing design standards.

3.5.1 Estimation of surface runoff

Surface runoff in the study area was estimated using the principles of mass balance, which expresses the relationship between incoming rainfall and the various loss components during a storm event;

$$\frac{\eth d}{\eth t} = \mathrm{R} - \mathrm{e} - \mathrm{f} - \mathrm{q} \qquad \text{Equation 3.5}$$

Where d-depth of excess water, R- rainfall rate, e-evaporation rate, f-infiltration rate and q- runoff ate.

This formulation provides a conceptual basis for quantifying the portion of rainfall that becomes direct runoff after accounting evaporation, infiltration and other abstractions. To estimate flow within drainage channels and overland flow paths, the Meaning equation was applied. This equation relates flow rate to the hydraulic radius, channel slope and surface roughness allowing the computation of volumetric discharge under varying flow conditions.

$$\mathrm{Q} = \frac{1.49}{n} A s^{1/2} R^{2/3} \qquad \text{Equation3.6}$$

Horton's infiltration model was used to simulate infiltration losses accounting for decline in infiltration capacity with storm duration. This method was chosen as it naturally embodies the phenomenon where the infiltration rate of soils will usually drop over time. Such methods are useful for urban and pre-urban catchments where the soil surface may become quickly saturated.

3.6 Selection and evaluation of LID infrastructure

An examination of low impact development (LID) practices was carried out to identify feasible and effective measures for reducing urban runoff. In order to ensure feasibility, available space, land use and local soil characteristics were considered. The first step was to map the areas through GIS based studies to implement various LID measures, green roofs, permeable pavements, rain gardens, etc. The suitability criteria were applied with land slopes less than10%, impervious surface coverage greater than 40% and preference for publicly or government owned land for ease of construction and maintenance.

Consultation with local engineers and environmentalists were held for further support on the decision-making process. In terms of practical applicability and maintenance and performance, their feedback helped to determine the various LID strategies. After identifying appropriate sites, the selected LID units were incorporated into the SWMM to evaluate their hydrologic impact, particularly their potential for reducing peak flow rates and total runoff volumes.

3.6.1 Flow routing

The infiltration process within the SWMM simulations was captured using Horton's infiltration model, as it captures the declining infiltration capacity which typically occurs during storm events. The dynamic wave routing technique was preferred to route the runoff through the drainage network since it is useful to model hydraulic behavior (including backwater effects, flow reversals and surcharging conditions) which commonly occurs in urban flooding.

The hydraulic parameters of the watershed model were suited during the trial runs so as to achieve the best fit and observed discharges. Thus, the calibration of the SWMM model ultimately improved the accuracy and dependability of the model. Furthermore, it permitted a realistic analysis of the potential benefits of adopting LID measures across the study area.

3.7 Summary of tools and software

Tools / Software	Purpose
SWMM	Urban storm water simulation
Arc GIS	Sub catchment delineation and spatial analysis
GPS and Leveling machine	Ground truth and verification
Google Earth	Land use mapping and visual interpretation

This methodology ensured that both quantitative and qualitative aspects of flood risk and drainage performance were captured, analyzed and modeled for better urban flood management.

3.8 Conceptual framework

This study follows a structured approach that integrates field based primary data collection, secondary data analysis, hydrological modeling using SWMM and validate with observed data.

The aim is to analyze urban flood risks within the Addis Ketema sub city through the simulation of surface runoff, drainage network efficiency and integration of possible low impact development (LID) technique

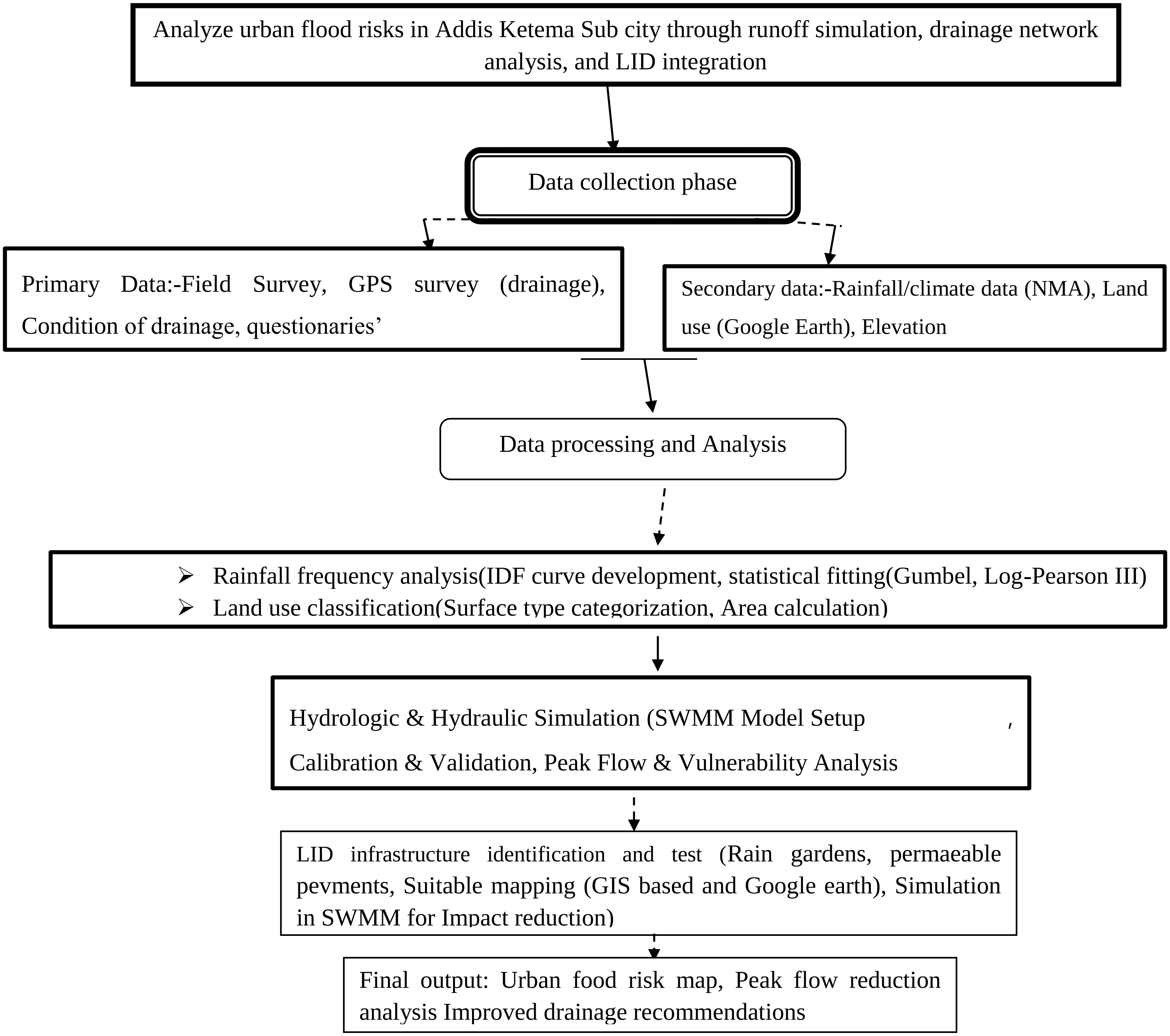


Figure 3-2: Conceptual frame work

By: Abdurahiman Nesiru Email- Abdurahimannesiru@gmail.com

4 RESULT AND DISCUSSION

4.1 Respondent Educational Background

The level of education of respondents during the questionnaire survey is significant for determining their grasp of the questions. Out of the 162 questionnaires analyzed, 50 had an MSc, 80 had a BSc. 24 had a diploma, 4 had completed high school, and 4 were in the lower grades. This level of education result indicates that site personnel were qualified enough to provide credible information by understanding each item on the questionnaire concerning the highway drainage system problem and identifying the key contributing cause.

4.2 Causes of Flooding: Insights, Analysis and Engineering Perspectives

4.2.1 Identifying the Causes of Flooding

The answers the respondents gave indicate several causes of flooding as reflected in the data from the questionnaire. The majority of causes include poor drainage systems (40%), heavy rain fall (30%), and poor planning and land use (15%), obstruction of natural water course (10%). Moreover, five out of one hundred believed that flooding is due to rising sea levels and climate change.

Flooding is multi-faceted, and several causes contribute to the overall flooding problem. The most common cause has insufficient drainage systems and heavy rainfall indicating the flooding risks of urban infrastructure and weather patterns. The results are consistent with engineering studies that call for efficient storm water management systems. The plan for the city and how the land will be used emerged as quite important as well. It suggests that we need a unique combination of improved infrastructure development, better planning and changing environmental conditions.

4.2.2 Analysis of Flooding Causes Based on Questionnaire Data

According to the data from the questionnaire, poor drainage systems was identified as the cause of flooding by 45% of the respondents while heavy rains, an extreme weather event was blamed by 35%. The leftover 20% were due to such reasons as improper waste dumping, construction on floodplains, and deforestation.

The study emphasizes how lack of infrastructure, especially drainage, cannot take more rainwater flows remains a critical matter. As flooding is caused in rural and urban areas due to increase in extreme weather events. This is a time to reflect. Poor waste disposal and building activities in flood-prone areas obstruct the natural flow of water and aggravate the situation. To address such problems, engineering of solutions that enhance drainage capacity and sustainable urban planning must be promoted. As such, a mix of short- and long-term strategies is key to tackling the identified causes.

4.2.3 Factors Contributing to Flooding: Questionnaire Insights

In the study, half of respondents indicated that heavy rains were what causes floods followed by drainage blockage (25%) and poor flood barriers (15%). Only 10% of the respondents cited human activities like – construction, deforestation, and land use as other contributing factors.

Flooding was most frequently due to heavy rainfall, which aligns with the growing severity and frequency of extreme weather patterns produced by climate change. Nonetheless, the large share of responders mentioning clogged drainage systems suggests that it is still a heavy burden. The ineffectiveness of today’s flood defenses shows the potential benefit of investing in better infrastructure. Damage done to nature by human activity like deforestation or urban growth disrupts natural water flow and overall capacity for rain absorption which raises flood risk. Engineering works should focus on upgrading drainage systems and constructing flood defenses, and enforcing a sustainable land use policy.

4.2.4 Engineering Perspectives on Flooding Causes

The questionnaire data reflected that 55% of respondents blamed flooding on poor drain design and maintenance while 25% blamed flooding on rapid urbanization. Another 20% of the responders mentioned riverbank erosion and natural biodiversity absorption of rainfall.

According to engineering, the poor design and maintenance of drainage are basic causes of flooding, most notably in crowded urban areas. People across the world are migrating towards cities from rural areas. Urbanization without infrastructure development will help amplify flood risks. The results highlight how natural systems such as wetlands and floodplains can help by minimizing the impact of floods. Engineering solutions should leverage nature-based solutions in conjunction with traditional infrastructure for resilience. The revelations, therefore, indicate that one should plan ahead and invest for the long haul. They must incorporate nature and man-made systems.

4.2.5 Root Causes of Flooding: A Data-Driven Approach

According to the assessed data questionnaire, the respondents have identified that the main cause of flooding is related to infrastructure development, accounting for 45%. Further, 35% of the respondents have credited environmental factors as a cause of flooding. 20% of the respondents have credited human involvement, such as deforestation and poor land management.

Data shows that flooding in many areas is due to infrastructure failure or inadequacy like the drainage system failing to take excess rain water. Environmental factors – changing rainfall patterns, rising sea levels – are now contributing to flooding, especially in coastal areas. Flood management must comprise alternatives that champions engineering solutions, environmental fixes and civil defense strategies, they stress. A vital solution to avoid flooding could further include the tackling of deforestation or activities being done due to ill-planning of development.

4.2.6 Flooding Mechanisms: Results from Questionnaire Analysis

As per the survey of the study, 60% of the people think floods are caused because of inefficient drainage systems while 25% because of climate change. Fifteen percent reported matters like blocked water bodies and urban sprawl.

The output shows well understanding of drainage systems for flooding control ultimately prevents flooding. Climate change is contributing to more frequent and powerful rain events that overburden existing drainage systems. Blocked streams and urban development compound flooding as a result of decreased water flow. These problems need a combination of solutions such as better drainage systems, flood management methods, and sustainable urban development. To respond to shifting weather patterns, climate adaptation methods must be put into place to reduce problems.

4.3 Urban flood Modeling Performance

One of this study's main aims is to evaluate how effective the urban drainage system is in the Addis Ketema sub-city and find out flooded sites. They used the SWMM (Storm Water Management Model) to simulate rainfall-runoff processes and the hydraulic behavior of drainage networks. The modeling work flow follows a systematic approach, starting with a baseline simulation using uncelebrated parameters, through a sensitivity analysis, then calibration and finally validation.

4.3.1 Initial Simulation Results

The preliminary simulation was performed using parameter values referred from literature and design manuals for the other drainages and was run to check the performance of the drainage system during design storm with a return period of 2, 5, 10, 25, 50 and 100 years.

The simulated rainfall intensity of the SWMM model used IDF curves based on historical rainfall data for the studied area. The surface property of each sub-catchment was considered using land use based imperviousness. To better represent surface runoff behavior, additional morphology attributes like catchment size, average slope, and width were defined. The Horton infiltration model calculated the depression storage and infiltration rates with consideration of initial abstraction and infiltration losses. Additionally, pipe sizes and gradients surveyed in the field, along with Manning's roughness coefficients for the drains as well as the sub-catchments, were implemented to estimate flow resistance and conveyance efficiency.

The model produced several significant output parameters. One of these was surface runoff depth, which is considered the water level produced by rainstorm. The quantity of flow was calculated for rating the drainage systems while conduit velocity has been used to assess hydraulic efficiency as well as scour risk. Locations of surcharge were also identified at which hydraulic capacity of node was exceeded. This helped to identify flooded areas. Subsequently, the model also computed the time to peak as well as the peak flow rates, which are significant for forecasting floods and mitigation measures.

The outcome of the first simulation showed that many nodes in the catchment's middle experienced surcharge conditions in the 25- and 50-year storms as the time of these storms was high. Runoff velocities were in excess of 2.5 m/s in narrower conduits, and runoff volume was greater than designed capacity in many sub-catchments. The outputs demonstrated a need for upgrades to the systems and basis for calibration.

4.4 Sensitivity Analysis

Prior to the calibration, a sensitivity analysis was performed to identify the parameters that had the most impact on model results: runoff depth and flow volume. The sensitivity analysis conducted in this work was on various essential parameters to see the effect on model outcomes like runoff depth and flow volume. Criteria assessed included imperviousness, which determines how much rainfall or runoff (i.e. water body) will be formed, and depression storage which represents the initial losses on account of ponding and storage in micro Topography. Similarly, initial and minimum steady infiltration rates were also measured to study the effect of soil absorption alteration on runoff. The analysis detailed the Manning's roughness coefficients for conduits and sub-catchments, which affects flow rate and resistance. Finally, a look at the width of the sub-catchment was made as it influences the timing and distribution of runoff in the focused area. The sensitivity analysis results led the calibration procedure, determining which parameters had the most effect on the model. Results of analysis revealed that peak runoff and flow depth were largely sensitive to imperviousness, infiltration rate, and Manning's n. The calibration process emphasized the significance of these parameters.

4.5 Capacity Analysis of Existing Drainage System

The drainage network in the SWMM model is designed for the research area. The catchment area is divided into 20 sub-catchments based on the drainage network, slope, and structural blocks. By calculating each sub-catchment in the region and feeding it into SWMM, runoff from that sub-catchment was derived (Fig 4. 1).

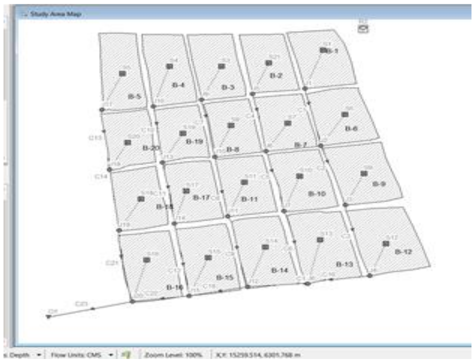


Figure 4-1: Capacity Analysis of Existing Drainage System

The capacity study conducted with the aid of the SWMM model does not covers the entire Addis Ketema sub-city. The study area is selected based on the presence of an existing drainage network, availability of data, and frequency of flooded scenario in Addis Ketema, one of the areas in the sub-city of Addis Ababa. The site selected was split into 20 sub-catchments based on existing drainage, topography slope, structural block, and other features of the site. The site was chosen owing to repeated drainage problems encountered there, which is representative of the common problems associated with urban runoff within the sub-city. Restricting the study to this site make the research more accurate and manageable. Therefore, the insights gained could provide valuable information that could be scaled or adjusted to other parts of Addis Ketema.

4.5.1 Sub-Catchment Peak Runoff

Sub catchments are hydrological units of land in which the topography and drainage system elements direct surface runoff to a single point of discharge. The peak runoff value for the two-year return period is displayed below. The graph shows that the highest peak runoff occurs in sub catchments S1, S6, S7, S8, S13, S15, and S19, while the lowest values are seen in S12, S14, S17, and S18.

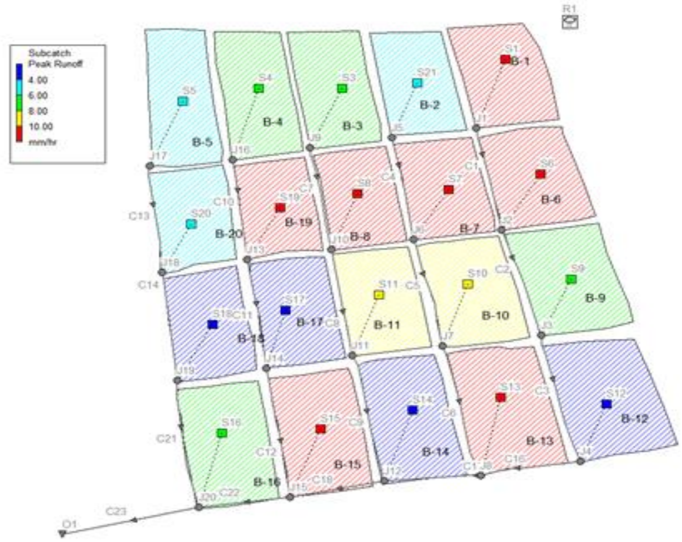


Figure 4-2: Sub-Catchment Pak Runoff

The value of each sub catchment's runoff peak is shown below. The table clearly displays the peak runoff value of each Sub Catchment for return periods of 2, 5, 10, and 25 years. According to the data, the size of peak runoff value for sub-catchments increases as the return duration increases. According to the findings, extending the return period increased the design rainfall intensity. As a result, peak runoff increased dramatically.

Table 4-1: Values of peak runoff for each catchment

Return period	2year	5year	10year	25year
Sub catchment	Peak runoff (CMS)	Peak runoff (CMS)	Peak runoff (CMS)	Peak runoff (CMS)
S1	4.21	5.68	6.62	7.9
S3	2.7	3.64	4.25	5.07
S4	2.33	3.15	3.67	4.38
S5	1.9	2.55	2.96	3.51
S6	4.01	5.42	6.32	7.55
S7	4.67	6.3	7.35	8.78
S8	3.85	5.2	6.06	7.24
S9	2.69	3.62	4.23	5.05
S10	3.44	4.64	5.42	6.47
S11	3.57	4.83	5.63	6.72
S12	1.22	1.59	1.81	2.11

S13	3.75	5.07	5.91	7.06
S14	1.22	1.59	1.81	2.11
S15	3.68	4.97	5.8	6.93
S16	2.46	3.32	3.87	4.62
S17	1.22	1.59	1.81	2.11
S18	1.22	1.59	1.81	2.11
S19	3.6	4.86	5.67	6.77
S20	1.79	2.41	2.81	3.36
S21	1.73	2.34	2.72	3.25

According to the table above, Sub Catchment 7 (S7) discharges the highest peak runoff, whilst Sub Catchment 14 (S14) discharges the lowest runoff when compared to other sub catchments for return periods of 2, 5, 10, and 25 years, respectively.

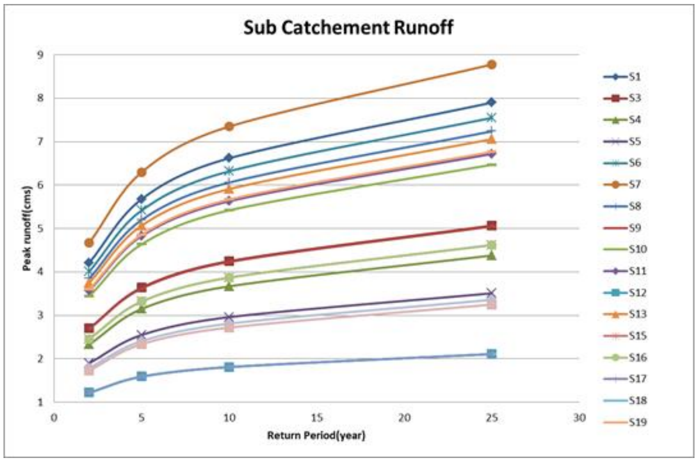


Figure 4-3: Sub-catchment peak runoff vs return period

4.5.2 Node modeling

The modeling results for various return periods reveal that the largest flooding occurs at J1, J2, J6, J7, J10, J11, J13, J15, and J18.Flooding is the accumulation of water in a watercourse. The main source of growing flood magnitude is the influx of water from sub-catchments.

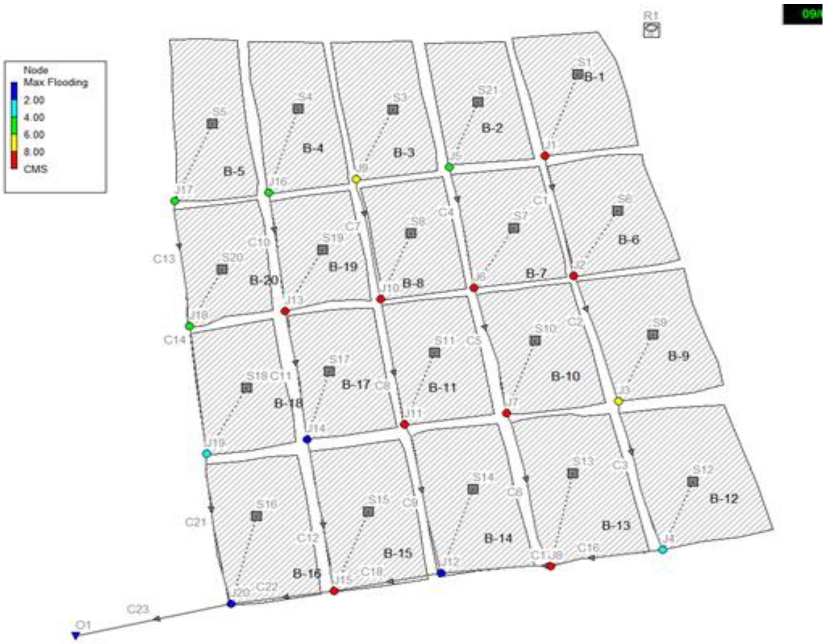


Figure 4-4: Node flooding

The flooding value for each node is listed in the table below. According to the results, nodes 2 (J2) and 6 (J6) have the highest values of flooding when compared to the other values. The primary cause of this result is the greatest peak discharge from sub catchments six (S6) and seven (S6), respectively.

Table 4-2: The flooding values of each node

Node Flooding	2 year CMS	5 year CMS	10 year CMS	25 year CMS
J1	1.502	2.941	3.909	5.162
J2	6.664	8.046	8.975	10.178
J3	1.637	2.553	3.169	3.97
J4	1.218	1.582	1.811	2.106

J6	5.267	7.456	8.921	10.349
J7	4.49	5.674	6.471	7.501
J8	3.344	4.634	5.5	6.62
J10	4.942	6.313	7.203	8.356
J11	5.079	6.312	7.138	8.205
J12	1.299	1.635	1.862	2.157
J13	3.338	4.572	5.403	6.48
J14	4.453	4.827	5.056	5.351
J15	0.65	1.459	2.311	3.414
J16	0.124	0.926	1.463	2.156
J18	1.754	3.004	3.828	4.893
J19	0.425	0.789	1.018	1.313
J20	7.479	8.372	8.94	9.676

4.5.3 Water depth and flow in the tanks

A flood tide occurs when the water surface at a node or connection surpasses the channel's maximum depth. Capacity is one of the factors to consider when calculating the ability of runoff to be transported through conduits in an urban drainage system. The majority of the gouged links and inundated nodes can be attributed to channel depth and grade. According to the simulation findings of a 2, 5, 10, and 25-year return period design storm, maximum flooding has occurred at conduits C1, C4, C7, C10, C13, C14, C11, C8, C21, C17, C18, C22, and C23.

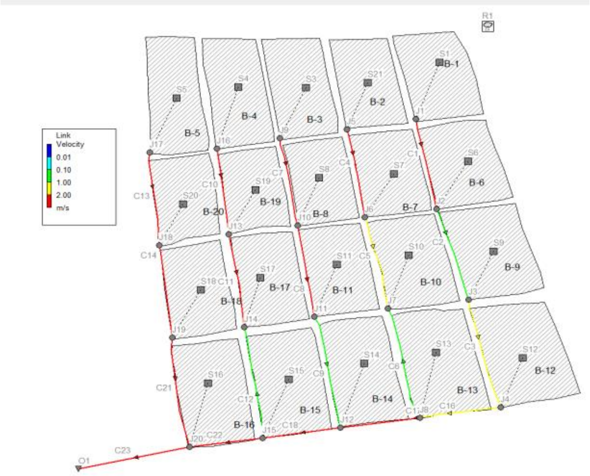


Figure 4-5: Water depth and flow in each link

The water elevation profile plot for Node J1 to Node J2 (C1) reveals that the conduit's height grew from 2457 m to 2457.8 m as the distance from sub-catchments to the outlet increased because the sub-catchment elevation is higher than the conduit's mean water level. The slope, size, and roughness of the conduit affect the inflow in the channels; the water does not flow upward. The elevation profile map from Node J1 to Node J2 (along conduit C1) shows that the conduit's invert elevation has increased from 2457 m to 2457.8 m, which may give the impression of reverse flow. However, this is not a sign that water is moving uphill. Rather, it suggests that the sub-catchments that drain into the conduit are at a higher elevation than the conduit. Gravitational flow therefore continues from a higher hydraulic head to a lower hydraulic head (that is, from sub-catchments to outlet). The actual direction of flow depends on the hydraulic gradient, not on the conduit invert slope. In addition, other conduit features, like slope, size and roughness, control inflow and flow in the conduit. They set the water surface profile, even at a section where the conduit slope seems unfavorable. The slope apparent in the conduit is not an indication of the water flowing uphill, but rather that the system may have isolated reverse slopes or installation issues causing the risk of impeded flow or flooding.

If the conduit invert elevation rises, that indicates installation or design error that is causing a negative slope. However, the hydraulic gradient, or the difference in height between the elevations of the water surfaces, is what determines actual flow direction, not conduit invert elevations.

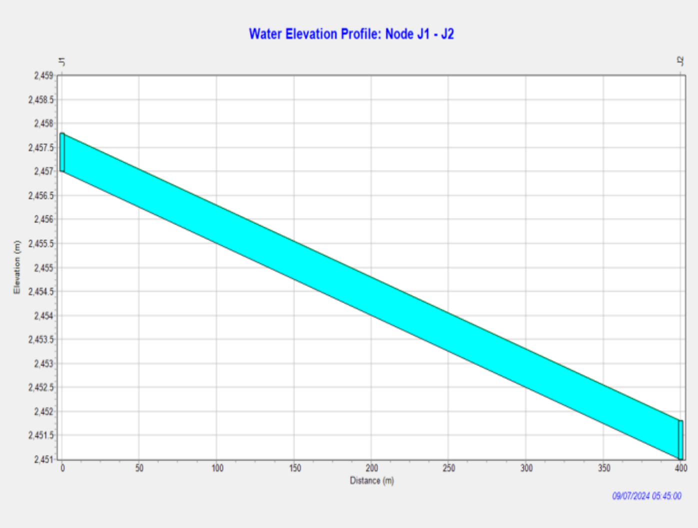


Figure 4-6: Water elevation profile for node-1 and node-12

The shaded area that wraps around the line is possibly indicating water depth or uncertainty, but quite critical, the elevation of the water surface appears to remain above the conduit invert level along the entire length of the length. This condition show that submergence where conduit flowing under full or near-full condition. When submergence happens, it reduces the conduit's ability to convey flow this occurs when the flow regime goes from open channel to pressurized flow. Upstream may be inundated or flooded, particularly during intense flow occurrences.

The elevation profile of water in consecutive conduits C13, C14, and C21 is shown below from J17 to J20.

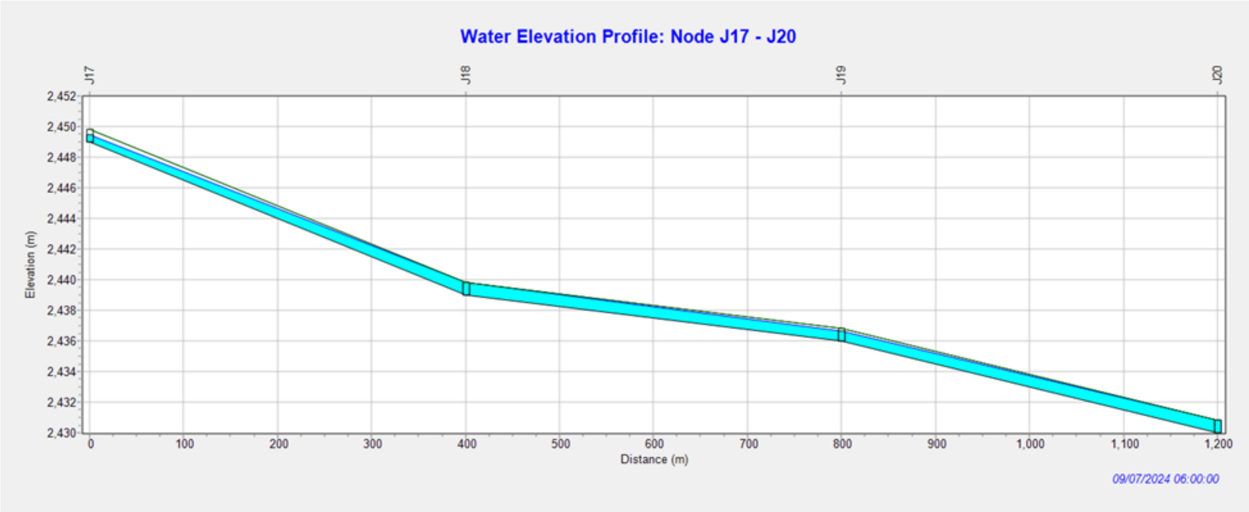


Figure 4-7: Water elevation profile fonode-17 andnode-20

4.6 SWMM Model Calibration

Calibration was accomplished with the use of observed data of a major storm-runoff event of 3-5 hours. Comparative study was made between observed runoff depths at major outfall points with simulated ones. The imperviousness, Manning’s roughness coefficient, infiltration rates etc. were changed iteratively for further improvement.

The calibration of the model produced an R^2 of 0.740 and an NSE of 0.740 which showed a strong agreement between the observed and simulated runoff depth. The model’s performances across the two watersheds were found to be quite satisfactory, with R^2 and NSE reaching values close to one. According to Figure 4.8, a plot of observed runoff depths against simulated values indicates a good fit.

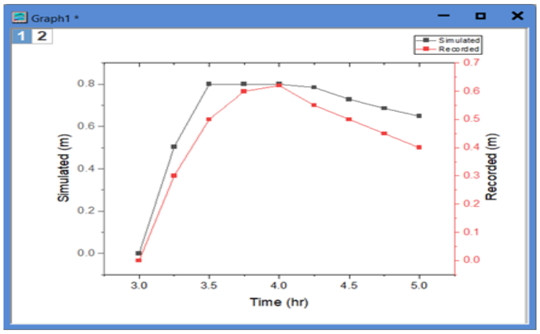


Figure 4-8: SWMM model calibration

4.7 SWMM model validation

The calibration input parameters were used for the validation of the SWMM model. The main purpose of this validation phase was to ascertain whether or not the calibrated parameters are able to simulate runoff for an individual rainfall event that was not used during calibration. The models' performances were acceptable in several statistical measures as indicated by a coefficient of determination (R^2) of 0.94 which is quite close to the ideal of 1. A Nash-Sutcliffe Efficiency (NSE) of 0.86 was recorded, a value within the satisfactory range of 0 to 1. The findings reveal high consistency between simulated runoff and observed runoff data. According to criteria, R^2 and NSE are considered adequate at catchment scale when they are above 0.65. From the findings, it can be stated that the model is well-validated for proper runoff depth and volume calculations. In conclusion, the validated model is suitable for runoff simulation in Addis Ketema sub-city. Presenting the observed and the modeled runoff depths shows that the simulation is trustworthy (Figure 4.5).

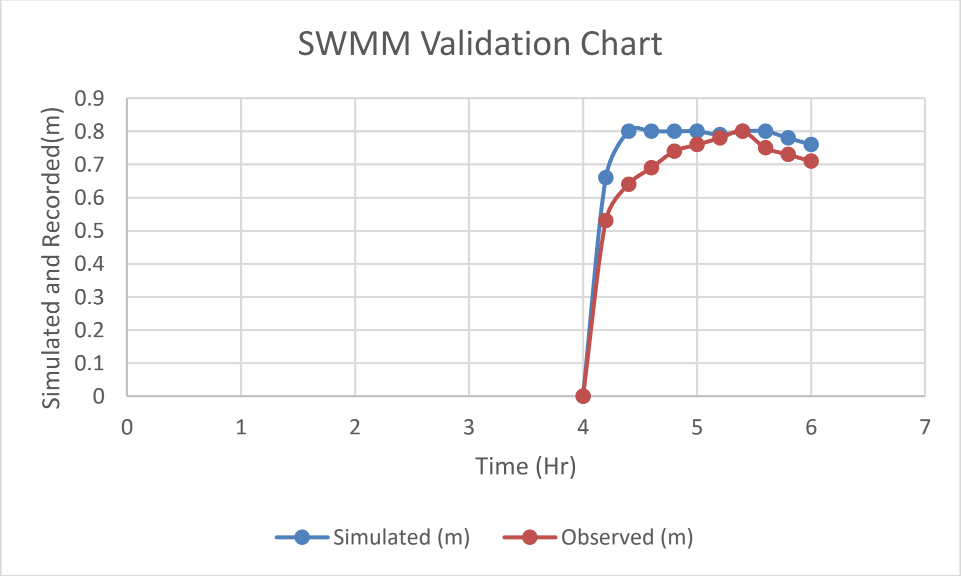


Figure 4-9: SWMM model validation results

In addition, runoff depths measured at outfall on the day of were utilized to validate the model. The SWMM simulation with the default parameter values revealed a rather weak agreement

By: Abdurahiman Nesiru Email- Abdurahimannesiru@gmail.com

between the simulated and observed runoff depth data hydrographs. As a result, parameter adjustments are required to replicate the runoff depth more accurately using calibration.

4.8 Summary of Model Performance

Parameter	Calibration Value	Validation Value
Coefficient of determination(R^2)	0.55	0.94
Nash-Sutcliffe Efficiency(NSE)	0.74	0.86
Flow volume derivation (%)	±12%	±8%
Surcharged Nodes	5nodes	4nodes
Maximum flow velocity(m/s/)	2.8m/s	2.5m/s

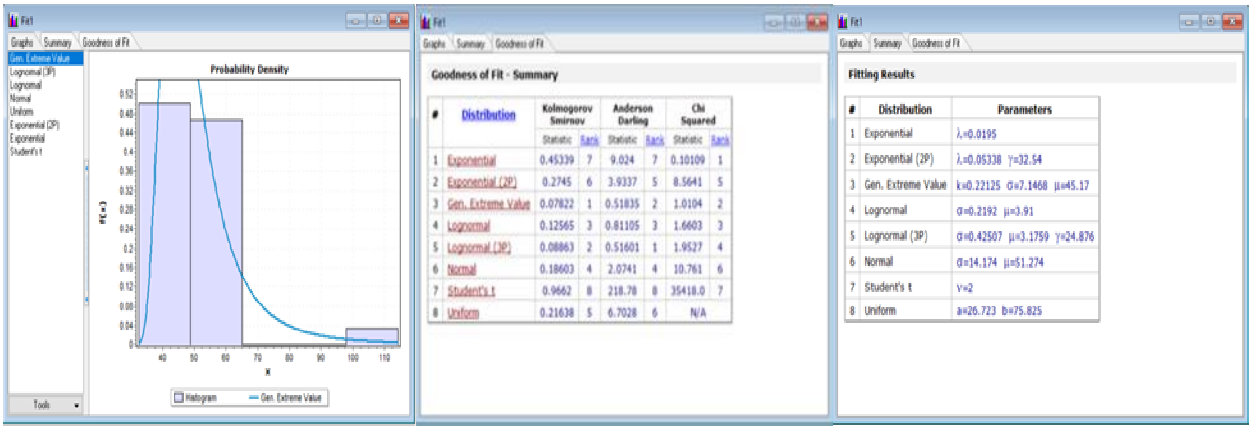
4.9 Frequency Analysis

Rainfall serve for use and control of water in water resources projects in both case the magnitude of the magnitude within its return period is important. Frequency analysis is the method of relating magnitude of extreme event with their frequency of occurrence by probability distribution. Later this distribution used to determine the magnitude for severe hydrologic extreme event such as flood. Probability distribution is useful only for finding the magnitudes of return period below recorded period. For values above recorded period best fit method is used to estimate the value of various return period.

4.10 Best fit Probability Distribution Functions

4.10.1 Easy fit Software

Easy-fit software is built around the calculation of statistical parameters utilizing three (3) test techniques. Kolomgerov-Semirnov, Anderson Darling, and Chi Sequared values for various probability distribution functions are ranked. According to the Goodness of Fit summary, the general extreme value is the best fit probability distribution function. The parameters used to evaluate the magnitude of different return periods are μ=45.17, σ=7.1468, and K=0.22125



Goodness of Fit - Summary

#	Distribution	Kolmogorov Smirnov		Anderson Darling		Chi Squared	
		Statistic	Rank	Statistic	Rank	Statistic	Rank
1	Exponential	0.45339	7	9.024	7	0.10109	1
2	Exponential (2P)	0.2745	6	3.9337	5	8.5641	5
3	Gen. Extreme Value	0.07822	1	0.51835	2	1.0104	2
4	Lognormal	0.12565	3	0.81105	3	1.6603	3
5	Lognormal (3P)	0.08863	2	0.51601	1	1.9527	4
6	Normal	0.18603	4	2.0741	4	10.761	6
7	Student's t	0.9662	8	218.78	8	35418.0	7
8	Uniform	0.21638	5	6.7028	6	N/A	

Fitting Results

#	Distribution	Parameters
1	Exponential	λ=0.0195
2	Exponential (2P)	λ=0.05338 γ=32.54
3	Gen. Extreme Value	k=0.22125 σ=7.1468 μ=45.17
4	Lognormal	σ=0.2192 μ=3.91
5	Lognormal (3P)	σ=0.42507 μ=3.1759 γ=24.876
6	Normal	σ=14.174 μ=51.274
7	Student's t	ν=2
8	Uniform	a=26.723 b=75.825

Figure 4-10: Best fit probability distribution functions

4.10.2 L-Moment

The L-Moment rate representation is utilized for colorful frequency distribution functions to determine the appropriate rainfall frequency distribution (Ayda et al., 2024; Ye et al., 2018). The results demonstrate that from all of the common distribution functions, the created data sample is defined by the values of L-skewness, i.e., -1. 265447943, and L-kurtosis, i.e., 0.553187151) is -0.6 and 0.3661 of general extreme value.

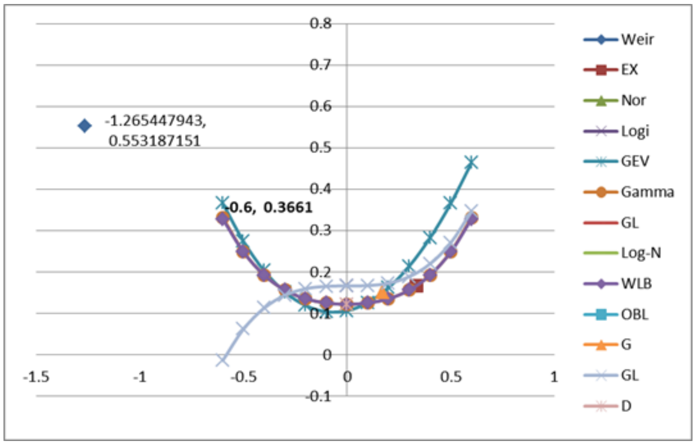


Figure 4-11: L-Moment result

4.10.3 Selection of best fit probability distribution function

From the above two way of finding best fit probability distribution general extreme value is appropriate probability distribution function for the given data.

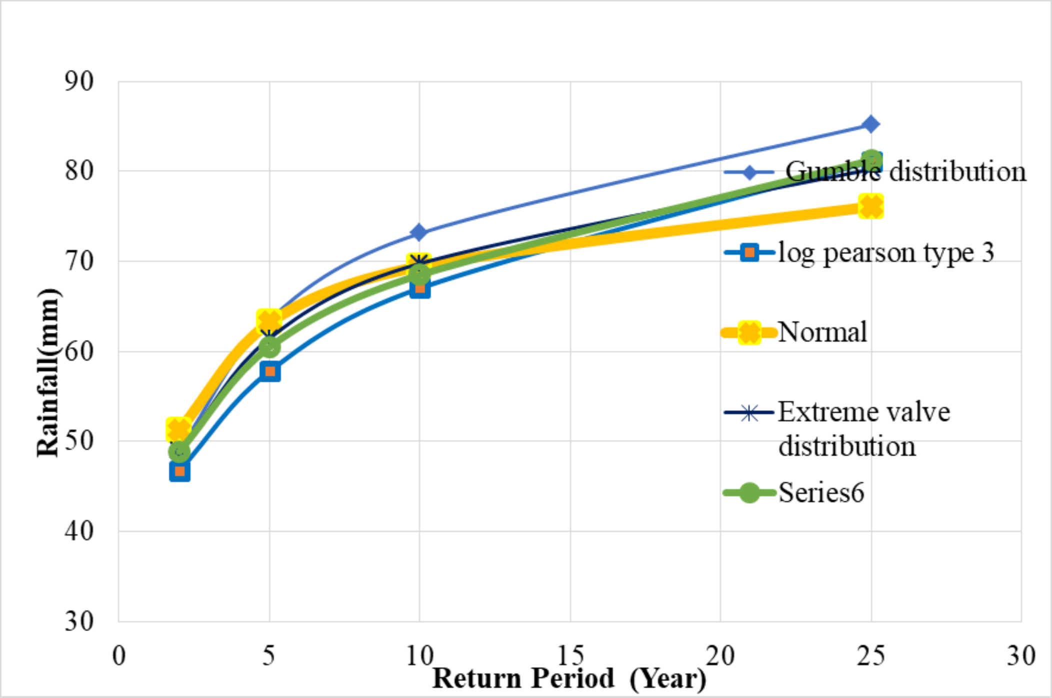


Figure 4-12: Return period vs rainfall

4.11 Forecasting future rainfall magnitude

Based on selected general extreme values for return periods of 2, 5, 10, 25, 50, and 100 years, the magnitude of rainfall is 48.944, 61.48, 69.77, 80.26, 88.04, and 95.7, respectively. The rainfall values for these return periods were estimated using statistical analysis of historical rainfall data, with the generalized extreme value distribution approach driving rainfall intensity for each return period. The graph in Figure 4-13 clearly illustrates that rainfall magnitude rises with longer return periods. This suggests that less frequent storms result in higher rainfall levels. Understanding this relationship is critical for building drainage infrastructure that can endure uncommon but severe rainfall events, hence lowering flood risk.

The hyetograph of the findings demonstrates that rainfall magnitude increases as the value of the return period increases. Similar findings were reported in the Najran and HafrAlbatin regions of Saudi Arabia (KSA) (Elsebaie, 2012).

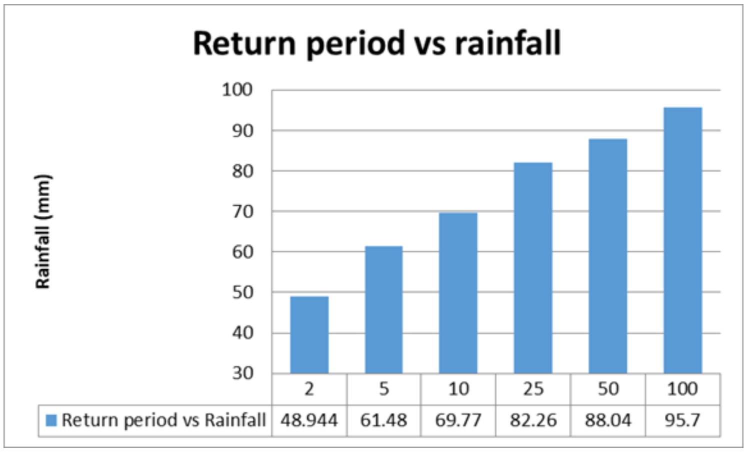


Figure 4-13: Graph of return period vs rainfall

4.12 IDF curve for future rainfall magnitude

The Intensity-Duration-Frequency (IDF) curve is an important tool in hydrological planning since it illustrates how rainfall intensity changes with storm duration and return period. It is used to estimate rainfall intensities for various storm durations, which are then fed into hydrologic and hydraulic models in flood simulation and drainage system design. They are graphical depictions of how much water falls in catchment areas over a certain time period (Bernadette S.Dupont and David L.Allen, 2000). The IDF curve should ideally be provided before simulation results because it contains design rainfall inputs that have a direct impact on simulation outcomes (such as runoff, flow rates, and flooding). Presenting the IDF curves first establishes the rainfall scenarios utilized in your model and allows readers to comprehend the simulation's foundation. The picture below depicts the IDF curve for 5, 10, 15, and 20 hours of estimated future rainfall data for return periods of 2, 5, 10, 25, 50, and 100 years. The graphic demonstrates that the intensity of rainfall is highest for short durations and lowest for long durations (Elsebaie, 2012).

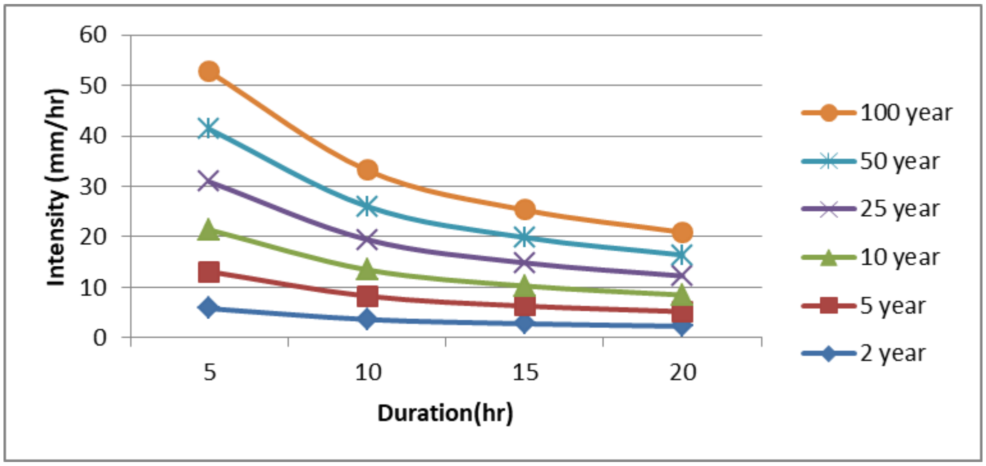


Figure 4-14: IDF curve for future rainfall magnitude

4.13 Effects of various LID scenarios on Rainfall Runoff

All measures designed for low impact development are effective for reducing runoff for small storm. As a result of the incorporation of LID techniques in the SWMM model, there is a significant improvement in storm water management performances of the catchments from study area. The simulated outcomes reveal that bio-retention cells and permeable pavements were most effective at reducing peak discharge rates and total runoff volumes near flood-prone junctions and conduits.

Using LID storage rain barrels (RB) efficiently can reduce peak runoff by 35% and more. The use of LID infiltration techniques like bio-retention cells and infiltration trenches (IT) can reduce the size of peak runoff by 48.5%. The use of LID combination techniques of storage and infiltration can reduce the peak runoff size by 75.5%. Green roofs provided substantial but uniform reductions in rooftop runoff. Particularly in high-density commercial settings where space precludes surface-based LID, it was seen to be of great benefit. The cumulative impact of many LID techniques reduced flow rates in key downstream conduits, reducing the number and severity of flood events in simulations.

5 CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

The study shows that the major cause of urban flooding in the Study Area is inadequate drainage facilities, impervious surfaces, runoff from highlands, and urbanization problems. An analysis done through hydrological modelling indicates that the drain is not capable of taking the peak flow after heavy downpour. This leads to damage to the road, traffic jams and public health issues. Rephrased: To mitigate the impact of floods, the report recommends rehabilitation of drainage system, annual maintenance, and incorporating flood considerations into development plans.

The study successfully integrated field data, community questionnaires and hydrological models, to assess urban floods in Addis Ketema Sub-City. The model effectively simulated surface runoff and drainage performance using SWMM calibrated with observed runoff and tested with independent events. The implementation of LID techniques had significant potential to reduce peak flows, providing feasible flood mitigation measures, depending on land use, soil conditions, and stakeholder involvement.

According to great questionnaire data and field observation, the main consequences of urban flooding of Addis Ketema Sub-City came from inadequate drainage systems, frequently heavy precipitation and poor land use planning. Most community feedback referred to issues related to problems caused by blocked or undersized drains, rain pattern changes due to climate change or unregulated growth in flood-prone areas. As per frequency analysis and rainfall forecasts the value of rainfall is on the rise with an increase in return periods. Thus, it may pose a serious threat to cities with inadequate drainage.

The rainfall-runoff process was simulated by SWMM model as well as hydraulic functioning of drainage network was carried out successfully along with identification of most vulnerable sub-catchments and nodes. The calibration and validation of the model yielded good performance indicators ($R^2 = 0.55$ and 0.94; NSE = 0.74 and 0.86), indicating it can be safely used in flood forecasting as well as infrastructure planning. The study results indicate the necessity of improvements to the drainage systems; nature-based solutions to prevent flooding; and climate-resilient urban planning that enhance the overall resilience of Addis Ketema.

5.2 Recommendations

The paper presents several probable projects and recommendations to improve urban drainage systems in order to reduce flooding in Addis Abeba. Here are the main recommendations drawn from the given contexts.

The study recommends low-impact development strategies that are affordable, environmentally adaptable, and sustainable for urban flood management. This strategy should work in tandem with existing drainage structures to improve storm management.

Future studies ought to examine new research on drainage systems designed and implemented in accordance with the particulars of urban areas. In order to prevent water from accumulating on road surfaces and causing damage, it is essential to ensure that the pavements, shoulders, drains and culverts are properly built and maintained. By using more recent modeling method and analysis, it is recommended to conduct flood frequency analysis at a larger scale. This means software such as SWMM (Storm Water Management Model) to help analyze peak runoff conditions and develop IDF curves that are specific to the site.

Future studies should assess sedimentation problems of drainage systems that silt the drainage and affect drainage performance of the drainage system. It will be a critical task to establish sediment management measures for effective drainage.

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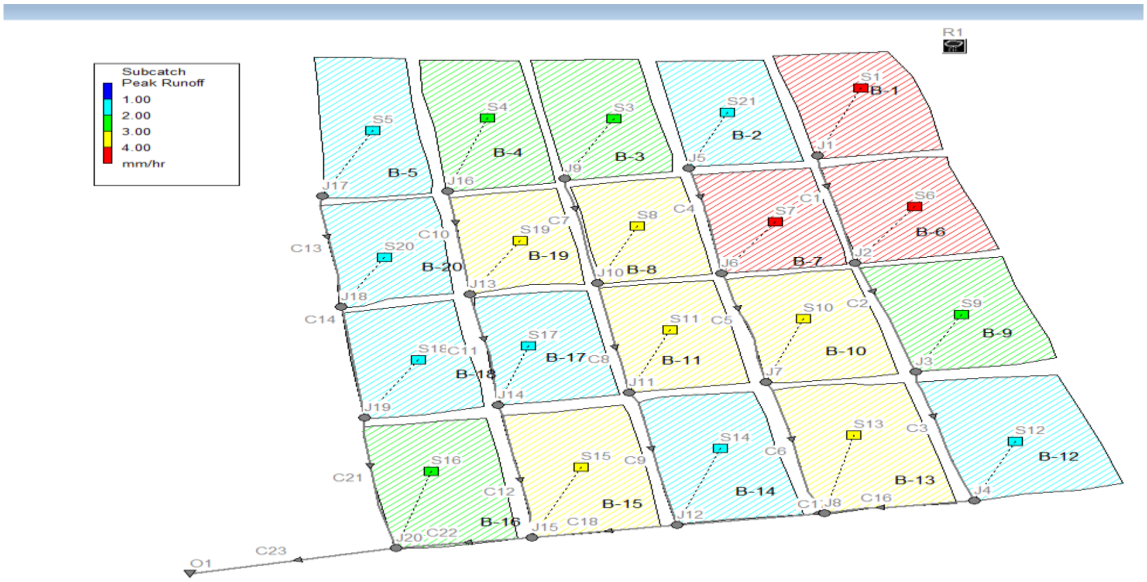
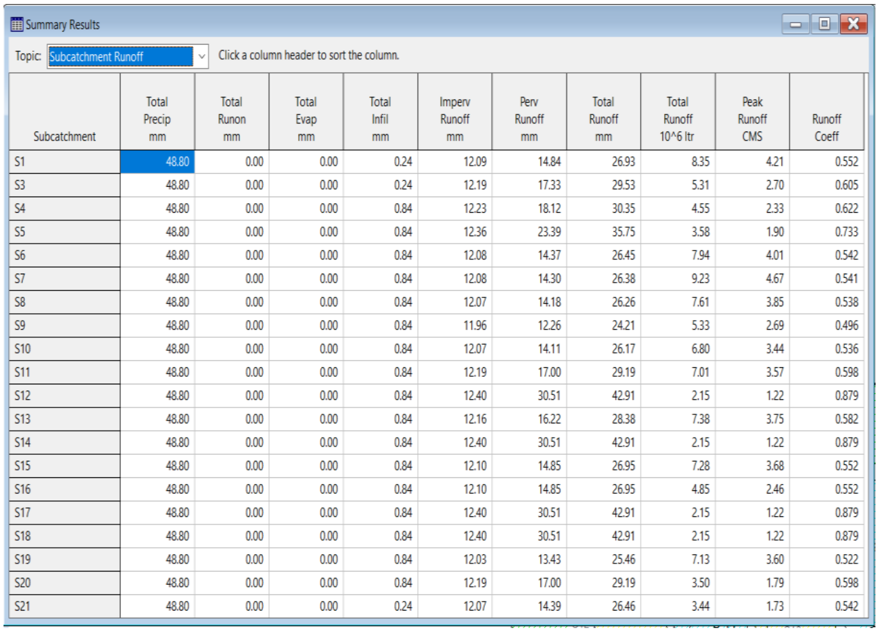
APPENDIX

Appendix A: 2-years return period

1. Peak runoff



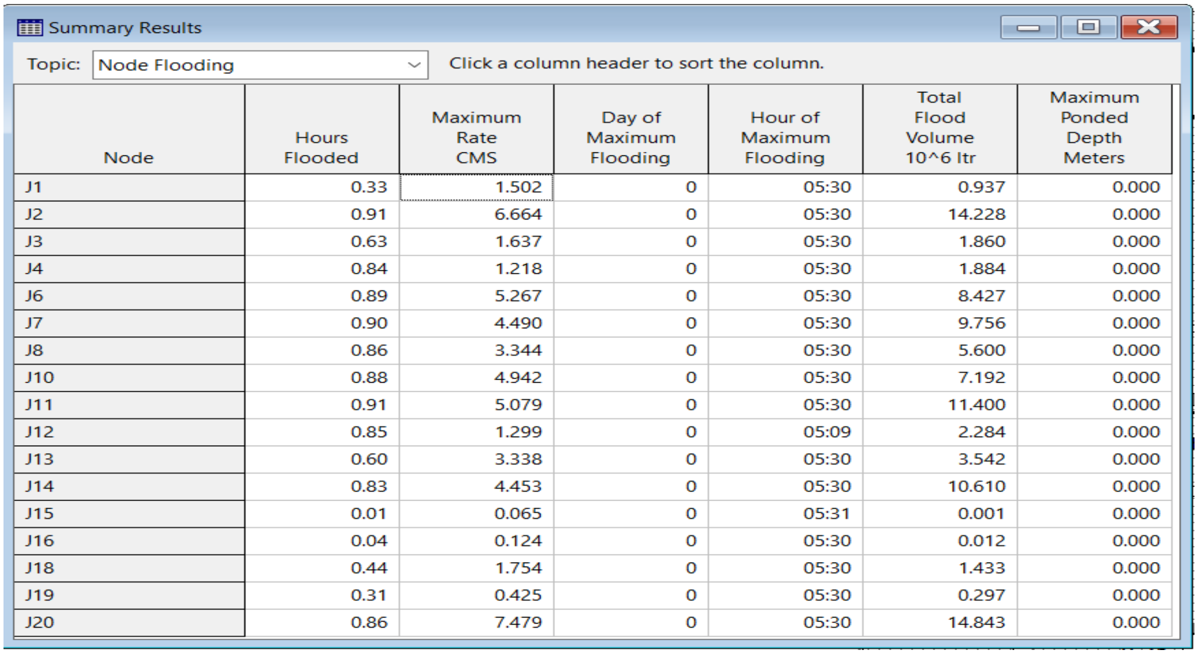
2. Peak runoff value of sub catchments



Click a column header to sort the column.

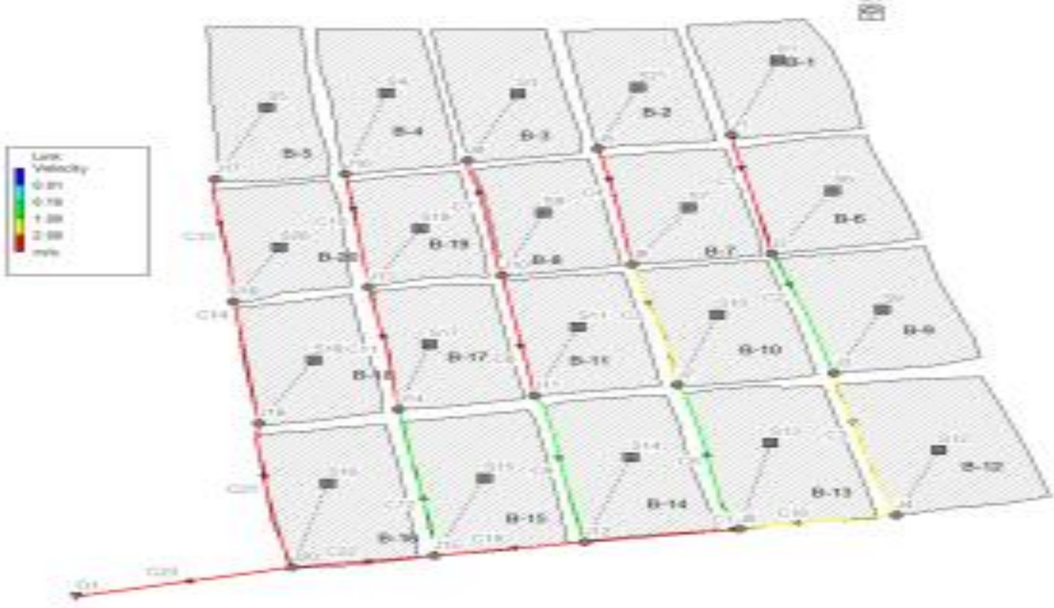
Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Imperv Runoff mm	Perv Runoff mm	Total Runoff mm	Total Runoff 10^6 ltr	Peak Runoff CMS	Runoff Coeff
S1	48.80	0.00	0.00	0.24	12.09	14.84	26.93	8.35	4.21	0.552
S3	48.80	0.00	0.00	0.24	12.19	17.33	29.53	5.31	2.70	0.605
S4	48.80	0.00	0.00	0.84	12.23	18.12	30.35	4.55	2.33	0.622
S5	48.80	0.00	0.00	0.84	12.36	23.39	35.75	3.58	1.90	0.733
S6	48.80	0.00	0.00	0.84	12.08	14.37	26.45	7.94	4.01	0.542
S7	48.80	0.00	0.00	0.84	12.08	14.30	26.38	9.23	4.67	0.541
S8	48.80	0.00	0.00	0.84	12.07	14.18	26.26	7.61	3.85	0.538
S9	48.80	0.00	0.00	0.84	11.96	12.26	24.21	5.33	2.69	0.496
S10	48.80	0.00	0.00	0.84	12.07	14.11	26.17	6.80	3.44	0.536
S11	48.80	0.00	0.00	0.84	12.19	17.00	29.19	7.01	3.57	0.598
S12	48.80	0.00	0.00	0.84	12.40	30.51	42.91	2.15	1.22	0.879
S13	48.80	0.00	0.00	0.84	12.16	16.22	28.38	7.38	3.75	0.582
S14	48.80	0.00	0.00	0.84	12.40	30.51	42.91	2.15	1.22	0.879
S15	48.80	0.00	0.00	0.84	12.10	14.85	26.95	7.28	3.68	0.552
S16	48.80	0.00	0.00	0.84	12.10	14.85	26.95	4.85	2.46	0.552
S17	48.80	0.00	0.00	0.84	12.40	30.51	42.91	2.15	1.22	0.879
S18	48.80	0.00	0.00	0.84	12.40	30.51	42.91	2.15	1.22	0.879
S19	48.80	0.00	0.00	0.84	12.03	13.43	25.46	7.13	3.60	0.522
S20	48.80	0.00	0.00	0.84	12.19	17.00	29.19	3.50	1.79	0.598
S21	48.80	0.00	0.00	0.24	12.07	14.39	26.46	3.44	1.73	0.542

3. Node flooding value



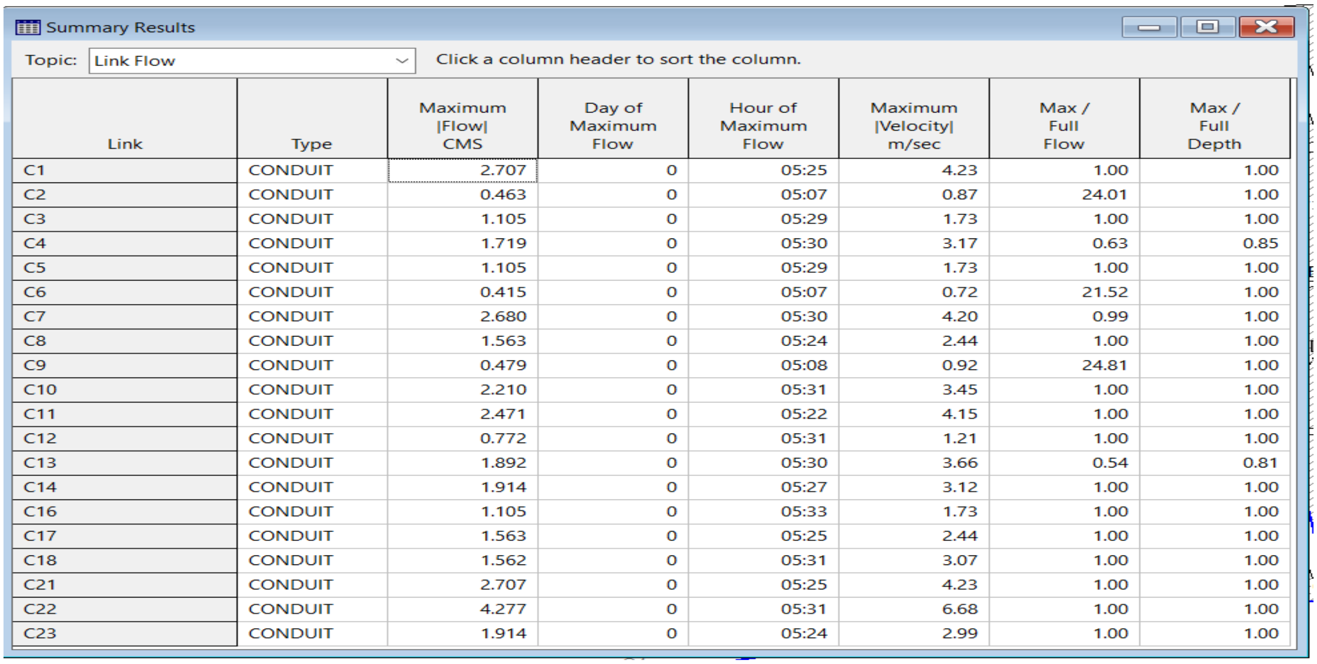
Node	Hours Flooded	Maximum Rate CMS	Day of Maximum Flooding	Hour of Maximum Flooding	Total Flood Volume 10^6 ltr	Maximum Ponded Depth Meters
J1	0.33	1.502	0	05:30	0.937	0.000
J2	0.91	6.664	0	05:30	14.228	0.000
J3	0.63	1.637	0	05:30	1.860	0.000
J4	0.84	1.218	0	05:30	1.884	0.000
J6	0.89	5.267	0	05:30	8.427	0.000
J7	0.90	4.490	0	05:30	9.756	0.000
J8	0.86	3.344	0	05:30	5.600	0.000
J10	0.88	4.942	0	05:30	7.192	0.000
J11	0.91	5.079	0	05:30	11.400	0.000
J12	0.85	1.299	0	05:09	2.284	0.000
J13	0.60	3.338	0	05:30	3.542	0.000
J14	0.83	4.453	0	05:30	10.610	0.000
J15	0.01	0.065	0	05:31	0.001	0.000
J16	0.04	0.124	0	05:30	0.012	0.000
J18	0.44	1.754	0	05:30	1.433	0.000
J19	0.31	0.425	0	05:30	0.297	0.000
J20	0.86	7.479	0	05:30	14.843	0.000

4. Link flow



5. Link flow value

By: Abdurahiman Nesiru Email- Abdurahimannesiru@gmail.com

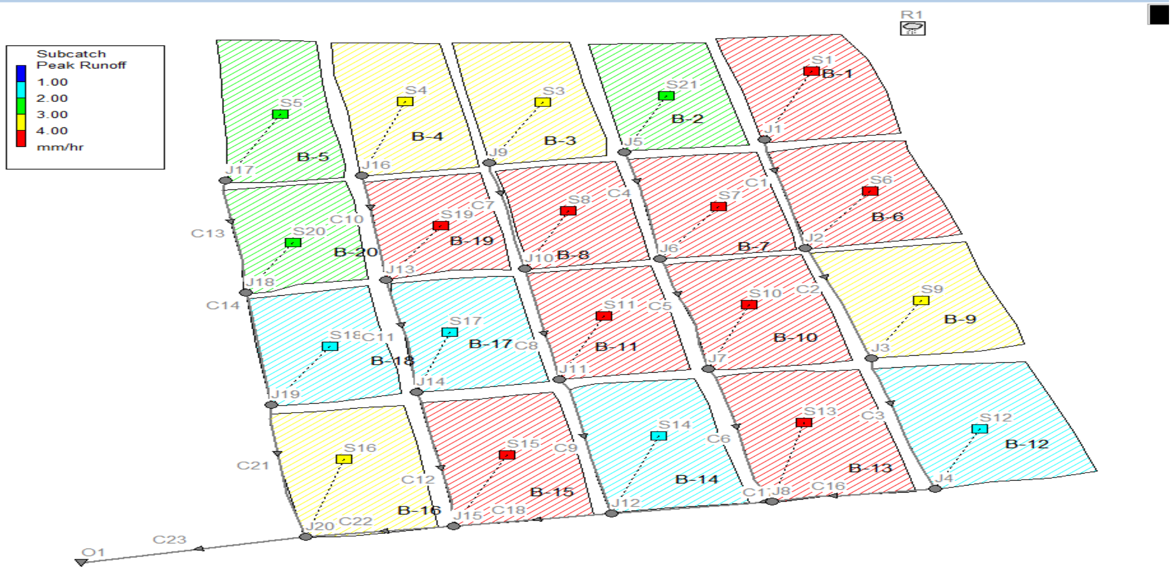


Topic: Link Flow   Click a column header to sort the column.

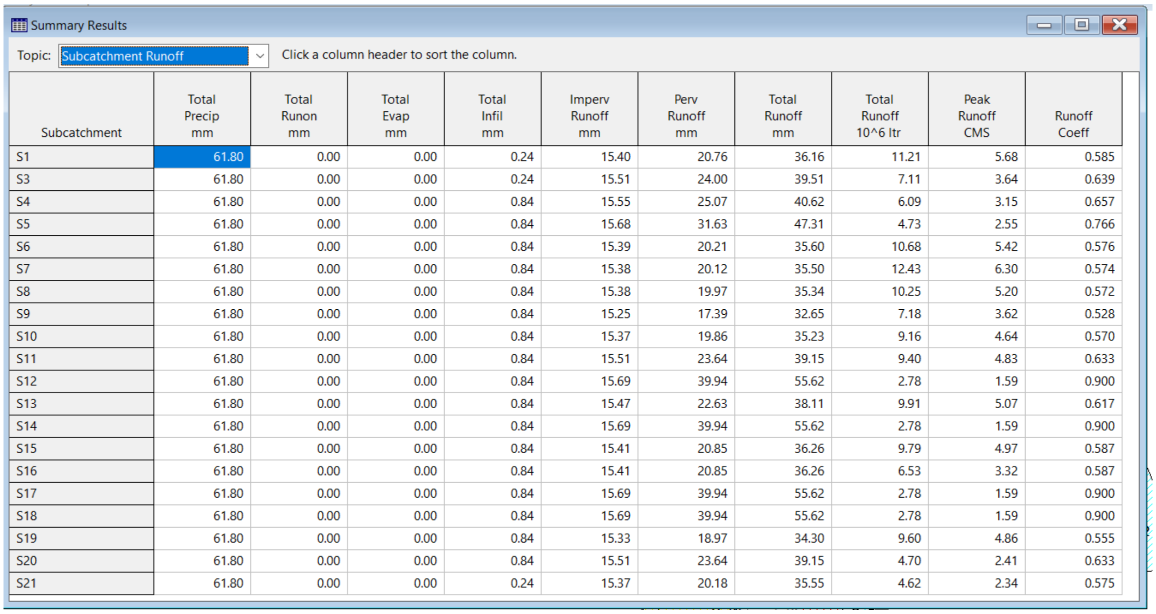
Link	Type	Maximum |Flow| CMS	Day of Maximum Flow	Hour of Maximum Flow	Maximum |Velocity| m/sec	Max / Full Flow	Max / Full Depth
C1	CONDUIT	2.707	0	05:25	4.23	1.00	1.00
C2	CONDUIT	0.463	0	05:07	0.87	24.01	1.00
C3	CONDUIT	1.105	0	05:29	1.73	1.00	1.00
C4	CONDUIT	1.719	0	05:30	3.17	0.63	0.85
C5	CONDUIT	1.105	0	05:29	1.73	1.00	1.00
C6	CONDUIT	0.415	0	05:07	0.72	21.52	1.00
C7	CONDUIT	2.680	0	05:30	4.20	0.99	1.00
C8	CONDUIT	1.563	0	05:24	2.44	1.00	1.00
C9	CONDUIT	0.479	0	05:08	0.92	24.81	1.00
C10	CONDUIT	2.210	0	05:31	3.45	1.00	1.00
C11	CONDUIT	2.471	0	05:22	4.15	1.00	1.00
C12	CONDUIT	0.772	0	05:31	1.21	1.00	1.00
C13	CONDUIT	1.892	0	05:30	3.66	0.54	0.81
C14	CONDUIT	1.914	0	05:27	3.12	1.00	1.00
C16	CONDUIT	1.105	0	05:33	1.73	1.00	1.00
C17	CONDUIT	1.563	0	05:25	2.44	1.00	1.00
C18	CONDUIT	1.562	0	05:31	3.07	1.00	1.00
C21	CONDUIT	2.707	0	05:25	4.23	1.00	1.00
C22	CONDUIT	4.277	0	05:31	6.68	1.00	1.00
C23	CONDUIT	1.914	0	05:24	2.99	1.00	1.00

Appendix B: 5years return period

1. Peak runoff



2. Peak runoff value

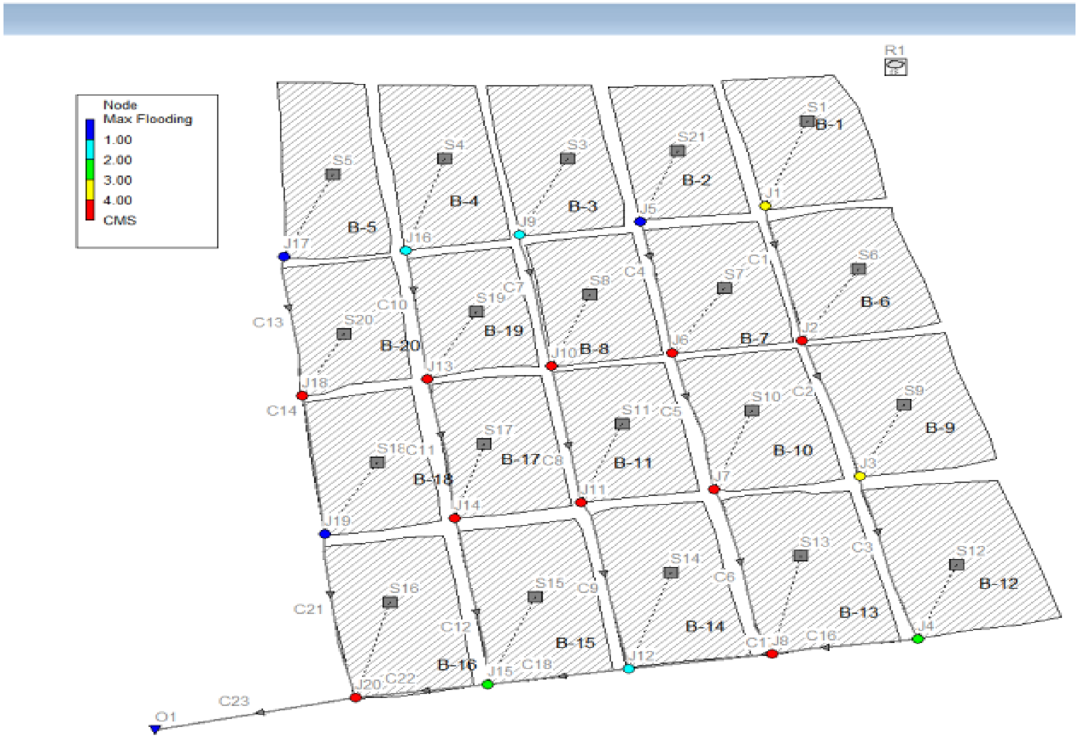


Topic: Subcatchment Runoff

Click a column header to sort the column.

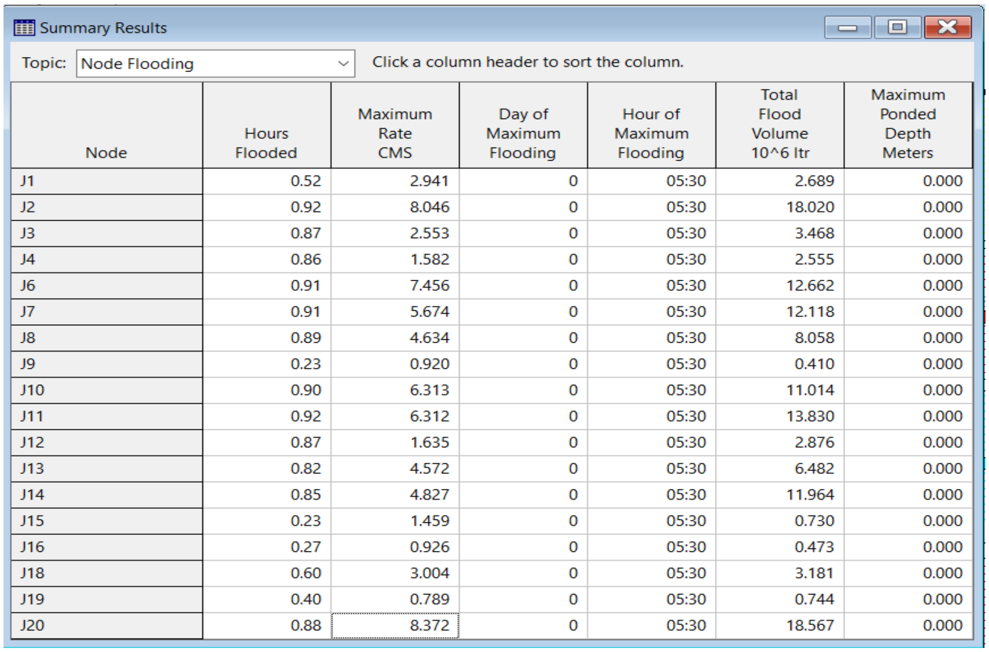
Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Imperv Runoff mm	Perv Runoff mm	Total Runoff mm	Total Runoff 10^6 ltr	Peak Runoff CMS	Runoff Coeff
S1	61.80	0.00	0.00	0.24	15.40	20.76	36.16	11.21	5.68	0.585
S3	61.80	0.00	0.00	0.24	15.51	24.00	39.51	7.11	3.64	0.639
S4	61.80	0.00	0.00	0.84	15.55	25.07	40.62	6.09	3.15	0.657
S5	61.80	0.00	0.00	0.84	15.68	31.63	47.31	4.73	2.55	0.766
S6	61.80	0.00	0.00	0.84	15.39	20.21	35.60	10.68	5.42	0.576
S7	61.80	0.00	0.00	0.84	15.38	20.12	35.50	12.43	6.30	0.574
S8	61.80	0.00	0.00	0.84	15.38	19.97	35.34	10.25	5.20	0.572
S9	61.80	0.00	0.00	0.84	15.25	17.39	32.65	7.18	3.62	0.528
S10	61.80	0.00	0.00	0.84	15.37	19.86	35.23	9.16	4.64	0.570
S11	61.80	0.00	0.00	0.84	15.51	23.64	39.15	9.40	4.83	0.633
S12	61.80	0.00	0.00	0.84	15.69	39.94	55.62	2.78	1.59	0.900
S13	61.80	0.00	0.00	0.84	15.47	22.63	38.11	9.91	5.07	0.617
S14	61.80	0.00	0.00	0.84	15.69	39.94	55.62	2.78	1.59	0.900
S15	61.80	0.00	0.00	0.84	15.41	20.85	36.26	9.79	4.97	0.587
S16	61.80	0.00	0.00	0.84	15.41	20.85	36.26	6.53	3.32	0.587
S17	61.80	0.00	0.00	0.84	15.69	39.94	55.62	2.78	1.59	0.900
S18	61.80	0.00	0.00	0.84	15.69	39.94	55.62	2.78	1.59	0.900
S19	61.80	0.00	0.00	0.84	15.33	18.97	34.30	9.60	4.86	0.555
S20	61.80	0.00	0.00	0.84	15.51	23.64	39.15	4.70	2.41	0.633
S21	61.80	0.00	0.00	0.24	15.37	20.18	35.55	4.62	2.34	0.575

3. Node flooding



By: Abdurahiman Nesiru Email- Abdurahimannesiru@gmail.com

4. Node flooding value

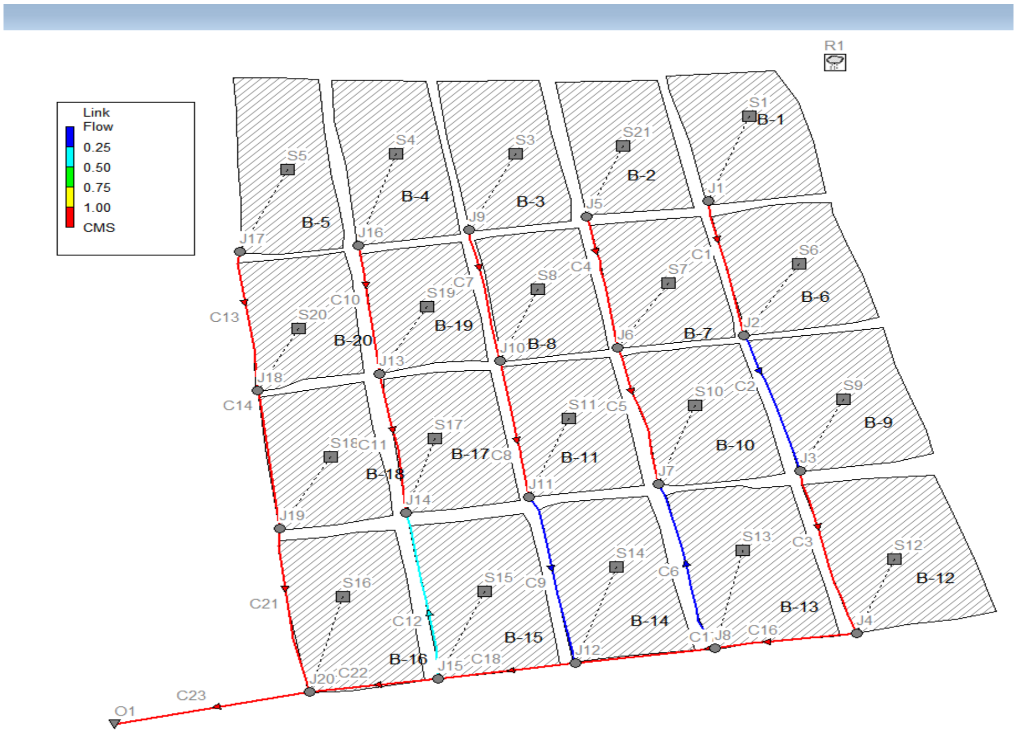


Topic: Node Flooding

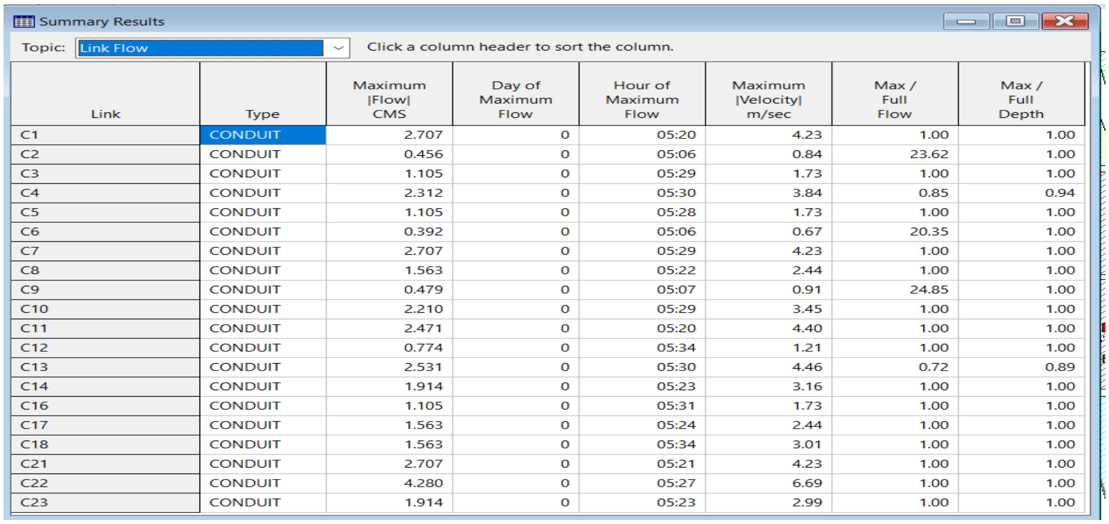
Click a column header to sort the column.

Node	Hours Flooded	Maximum Rate CMS	Day of Maximum Flooding	Hour of Maximum Flooding	Total Flood Volume 10^6 ltr	Maximum Ponded Depth Meters
J1	0.52	2.941	0	05:30	2.689	0.000
J2	0.92	8.046	0	05:30	18.020	0.000
J3	0.87	2.553	0	05:30	3.468	0.000
J4	0.86	1.582	0	05:30	2.555	0.000
J6	0.91	7.456	0	05:30	12.662	0.000
J7	0.91	5.674	0	05:30	12.118	0.000
J8	0.89	4.634	0	05:30	8.058	0.000
J9	0.23	0.920	0	05:30	0.410	0.000
J10	0.90	6.313	0	05:30	11.014	0.000
J11	0.92	6.312	0	05:30	13.830	0.000
J12	0.87	1.635	0	05:30	2.876	0.000
J13	0.82	4.572	0	05:30	6.482	0.000
J14	0.85	4.827	0	05:30	11.964	0.000
J15	0.23	1.459	0	05:30	0.730	0.000
J16	0.27	0.926	0	05:30	0.473	0.000
J18	0.60	3.004	0	05:30	3.181	0.000
J19	0.40	0.789	0	05:30	0.744	0.000
J20	0.88	8.372	0	05:30	18.567	0.000

5. Link flow



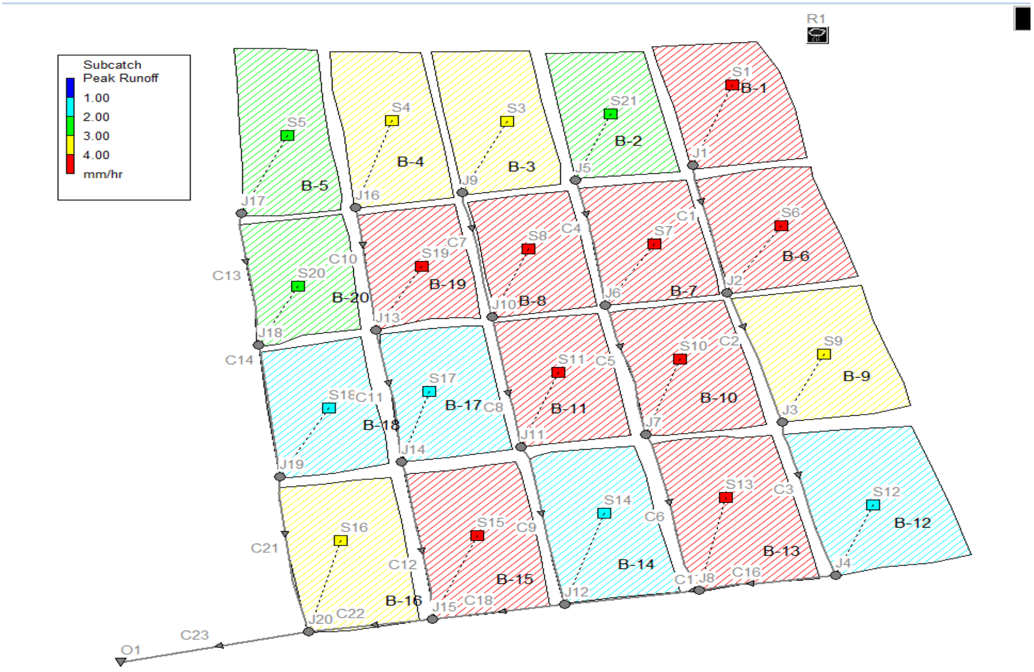
6. Link flow value



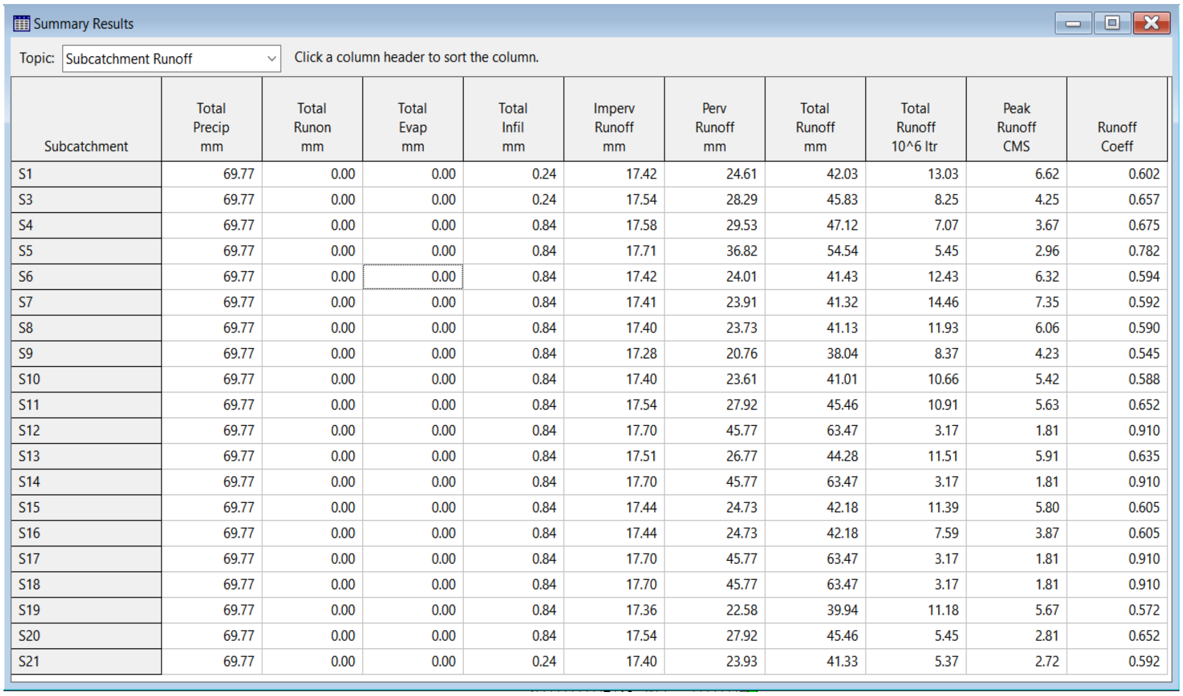
Link	Type	Maximum |Flow| CMS	Day of Maximum Flow	Hour of Maximum Flow	Maximum |Velocity| m/sec	Max / Full Flow	Max / Full Depth
C1	CONDUIT	2.707	0	05:20	4.23	1.00	1.00
C2	CONDUIT	0.456	0	05:06	0.84	23.62	1.00
C3	CONDUIT	1.105	0	05:29	1.73	1.00	1.00
C4	CONDUIT	2.312	0	05:30	3.84	0.85	0.94
C5	CONDUIT	1.105	0	05:28	1.73	1.00	1.00
C6	CONDUIT	0.392	0	05:06	0.67	20.35	1.00
C7	CONDUIT	2.707	0	05:29	4.23	1.00	1.00
C8	CONDUIT	1.563	0	05:22	2.44	1.00	1.00
C9	CONDUIT	0.479	0	05:07	0.91	24.85	1.00
C10	CONDUIT	2.210	0	05:29	3.45	1.00	1.00
C11	CONDUIT	2.471	0	05:20	4.40	1.00	1.00
C12	CONDUIT	0.774	0	05:34	1.21	1.00	1.00
C13	CONDUIT	2.531	0	05:30	4.46	0.72	0.89
C14	CONDUIT	1.914	0	05:23	3.16	1.00	1.00
C16	CONDUIT	1.105	0	05:31	1.73	1.00	1.00
C17	CONDUIT	1.563	0	05:24	2.44	1.00	1.00
C18	CONDUIT	1.563	0	05:34	3.01	1.00	1.00
C21	CONDUIT	2.707	0	05:21	4.23	1.00	1.00
C22	CONDUIT	4.280	0	05:27	6.69	1.00	1.00
C23	CONDUIT	1.914	0	05:23	2.99	1.00	1.00

APPENDEX C: - 10 -years return period

1. Peak runoff



2. Peak runoff value

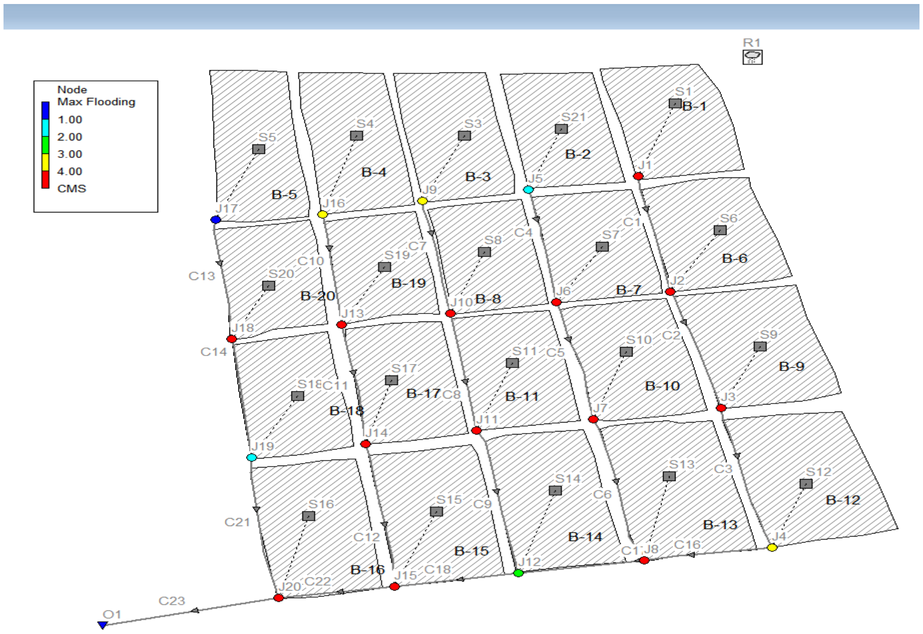


Topic: Subcatchment Runoff

Click a column header to sort the column.

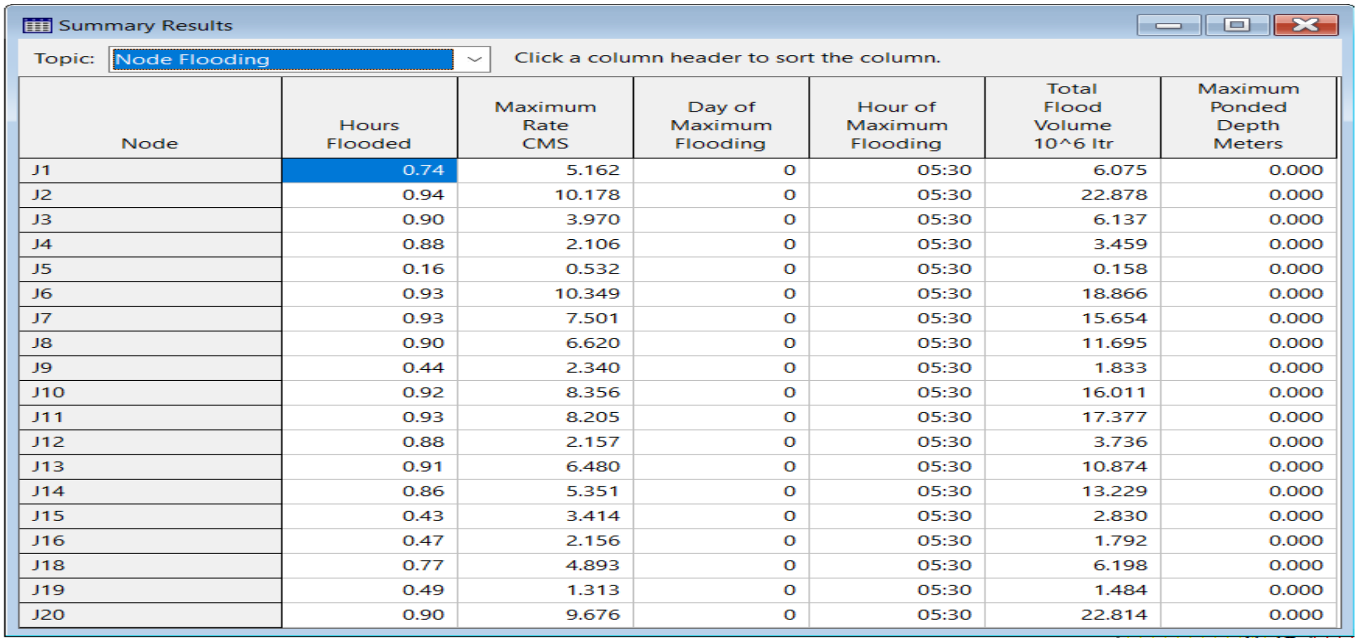
Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Imperv Runoff mm	Perv Runoff mm	Total Runoff mm	Total Runoff 10^6 ltr	Peak Runoff CMS	Runoff Coeff
S1	69.77	0.00	0.00	0.24	17.42	24.61	42.03	13.03	6.62	0.602
S3	69.77	0.00	0.00	0.24	17.54	28.29	45.83	8.25	4.25	0.657
S4	69.77	0.00	0.00	0.84	17.58	29.53	47.12	7.07	3.67	0.675
S5	69.77	0.00	0.00	0.84	17.71	36.82	54.54	5.45	2.96	0.782
S6	69.77	0.00	0.00	0.84	17.42	24.01	41.43	12.43	6.32	0.594
S7	69.77	0.00	0.00	0.84	17.41	23.91	41.32	14.46	7.35	0.592
S8	69.77	0.00	0.00	0.84	17.40	23.73	41.13	11.93	6.06	0.590
S9	69.77	0.00	0.00	0.84	17.28	20.76	38.04	8.37	4.23	0.545
S10	69.77	0.00	0.00	0.84	17.40	23.61	41.01	10.66	5.42	0.588
S11	69.77	0.00	0.00	0.84	17.54	27.92	45.46	10.91	5.63	0.652
S12	69.77	0.00	0.00	0.84	17.70	45.77	63.47	3.17	1.81	0.910
S13	69.77	0.00	0.00	0.84	17.51	26.77	44.28	11.51	5.91	0.635
S14	69.77	0.00	0.00	0.84	17.70	45.77	63.47	3.17	1.81	0.910
S15	69.77	0.00	0.00	0.84	17.44	24.73	42.18	11.39	5.80	0.605
S16	69.77	0.00	0.00	0.84	17.44	24.73	42.18	7.59	3.87	0.605
S17	69.77	0.00	0.00	0.84	17.70	45.77	63.47	3.17	1.81	0.910
S18	69.77	0.00	0.00	0.84	17.70	45.77	63.47	3.17	1.81	0.910
S19	69.77	0.00	0.00	0.84	17.36	22.58	39.94	11.18	5.67	0.572
S20	69.77	0.00	0.00	0.84	17.54	27.92	45.46	5.45	2.81	0.652
S21	69.77	0.00	0.00	0.24	17.40	23.93	41.33	5.37	2.72	0.592

3. Node flooding



By: Abdurahiman Nesiru   Email- Abdurahimannesiru@gmail.com

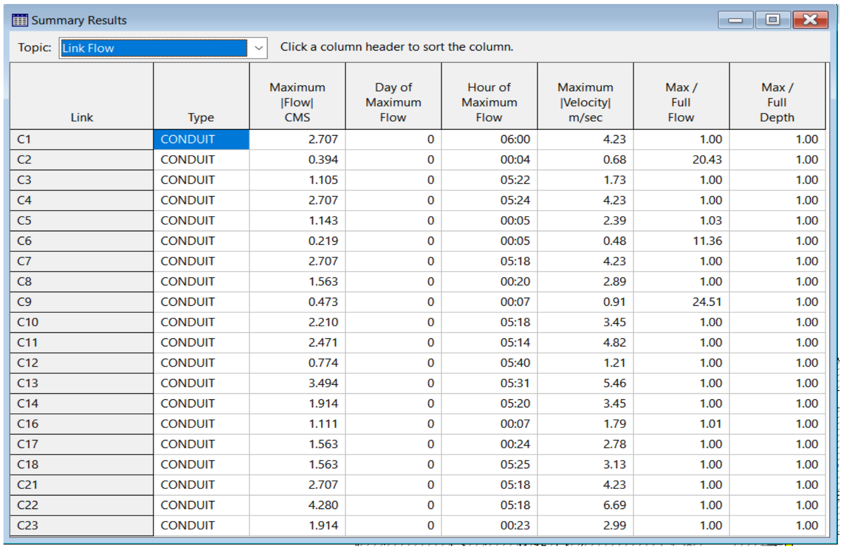
4. Node flooding value



Node	Hours Flooded	Maximum Rate CMS	Day of Maximum Flooding	Hour of Maximum Flooding	Total Flood Volume 10^6 ltr	Maximum Ponded Depth Meters
J1	0.74	5.162	0	05:30	6.075	0.000
J2	0.94	10.178	0	05:30	22.878	0.000
J3	0.90	3.970	0	05:30	6.137	0.000
J4	0.88	2.106	0	05:30	3.459	0.000
J5	0.16	0.532	0	05:30	0.158	0.000
J6	0.93	10.349	0	05:30	18.866	0.000
J7	0.93	7.501	0	05:30	15.654	0.000
J8	0.90	6.620	0	05:30	11.695	0.000
J9	0.44	2.340	0	05:30	1.833	0.000
J10	0.92	8.356	0	05:30	16.011	0.000
J11	0.93	8.205	0	05:30	17.377	0.000
J12	0.88	2.157	0	05:30	3.736	0.000
J13	0.91	6.480	0	05:30	10.874	0.000
J14	0.86	5.351	0	05:30	13.229	0.000
J15	0.43	3.414	0	05:30	2.830	0.000
J16	0.47	2.156	0	05:30	1.792	0.000
J18	0.77	4.893	0	05:30	6.198	0.000
J19	0.49	1.313	0	05:30	1.484	0.000
J20	0.90	9.676	0	05:30	22.814	0.000

5. Link flow

6. Link flow value



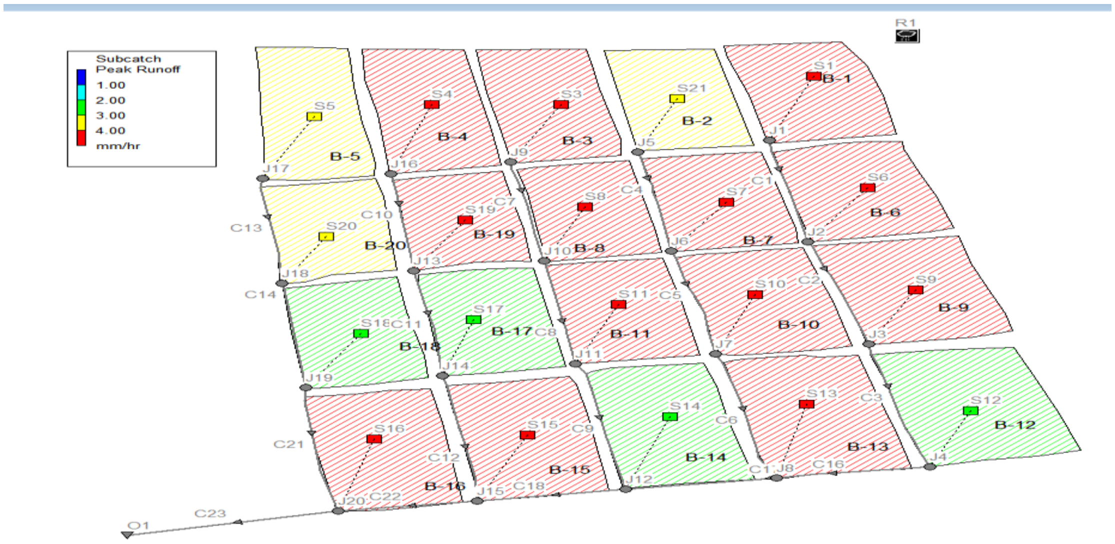
Topic: Link Flow

Click a column header to sort the column.

Link	Type	Maximum |Flow| CMS	Day of Maximum Flow	Hour of Maximum Flow	Maximum |Velocity| m/sec	Max / Full Flow	Max / Full Depth
C1	CONDUIT	2.707	0	06:00	4.23	1.00	1.00
C2	CONDUIT	0.394	0	00:04	0.68	20.43	1.00
C3	CONDUIT	1.105	0	05:22	1.73	1.00	1.00
C4	CONDUIT	2.707	0	05:24	4.23	1.00	1.00
C5	CONDUIT	1.143	0	00:05	2.39	1.03	1.00
C6	CONDUIT	0.219	0	00:05	0.48	11.36	1.00
C7	CONDUIT	2.707	0	05:18	4.23	1.00	1.00
C8	CONDUIT	1.563	0	00:20	2.89	1.00	1.00
C9	CONDUIT	0.473	0	00:07	0.91	24.51	1.00
C10	CONDUIT	2.210	0	05:18	3.45	1.00	1.00
C11	CONDUIT	2.471	0	05:14	4.82	1.00	1.00
C12	CONDUIT	0.774	0	05:40	1.21	1.00	1.00
C13	CONDUIT	3.494	0	05:31	5.46	1.00	1.00
C14	CONDUIT	1.914	0	05:20	3.45	1.00	1.00
C16	CONDUIT	1.111	0	00:07	1.79	1.01	1.00
C17	CONDUIT	1.563	0	00:24	2.78	1.00	1.00
C18	CONDUIT	1.563	0	05:25	3.13	1.00	1.00
C21	CONDUIT	2.707	0	05:18	4.23	1.00	1.00
C22	CONDUIT	4.280	0	05:18	6.69	1.00	1.00
C23	CONDUIT	1.914	0	00:23	2.99	1.00	1.00

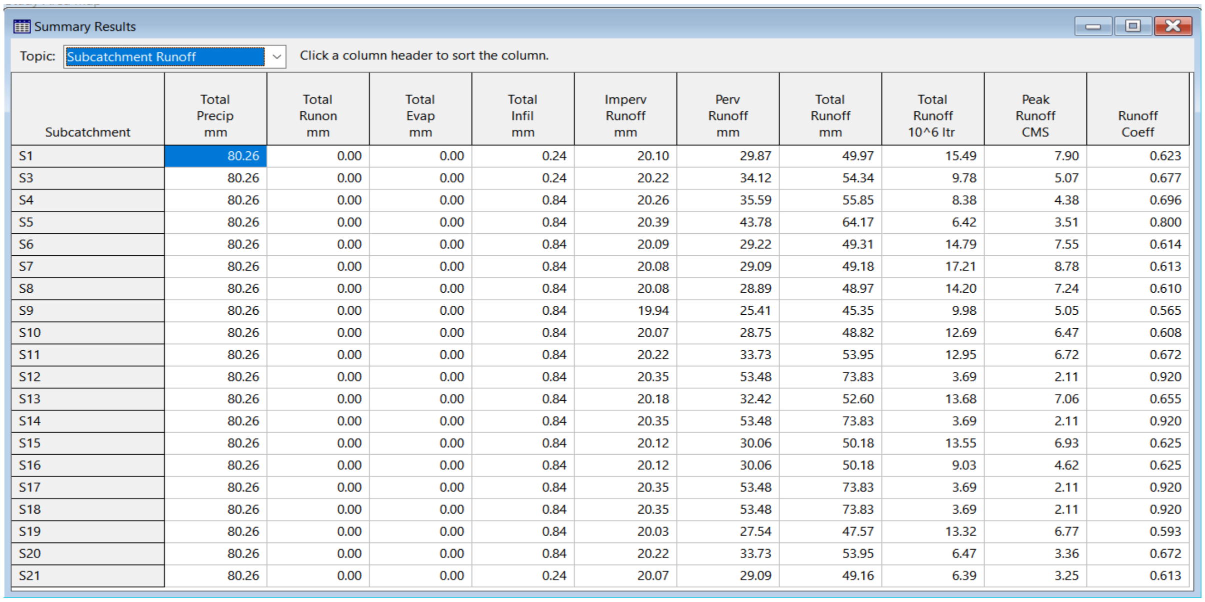
APPENDEX D: - 25 years return period

1. Peak runoff



By: Abdurahiman Nesiru   Email- Abdurahimannesiru@gmail.com

2. Peak runoff value

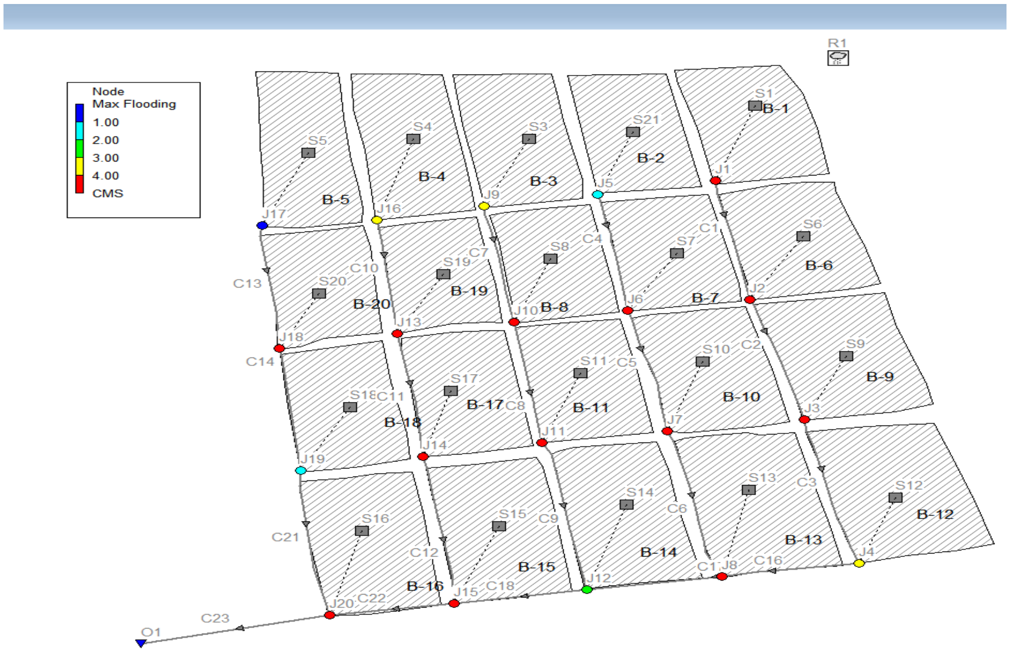


Topic: Subcatchment Runoff

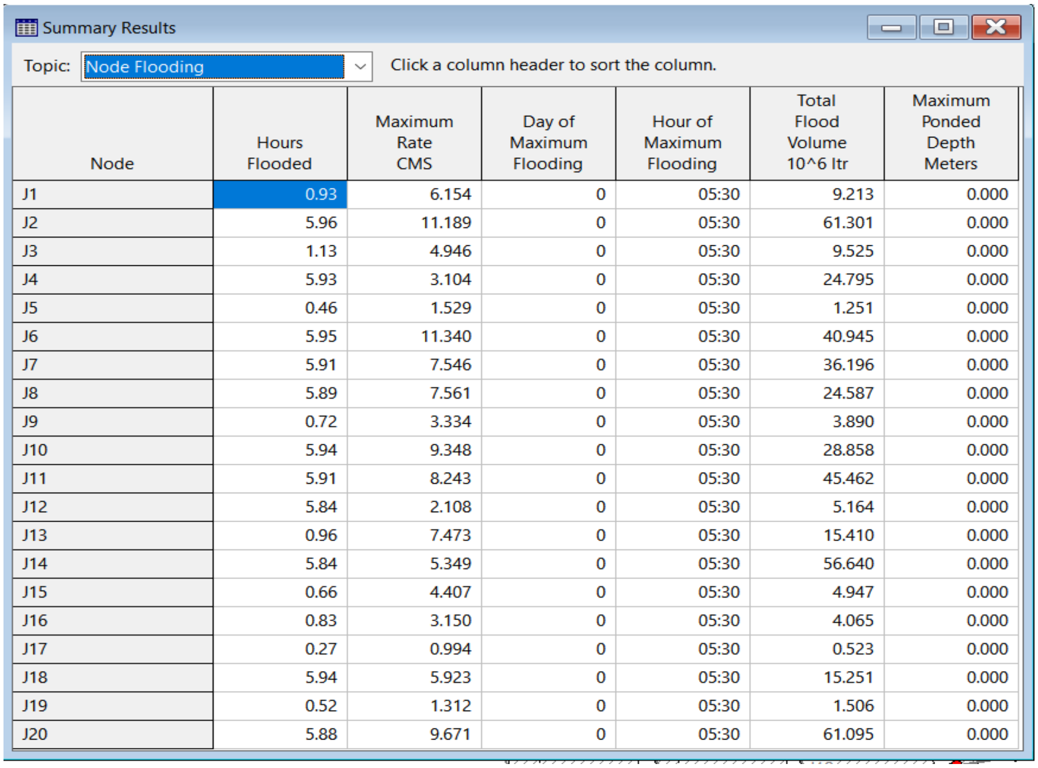
Click a column header to sort the column.

Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Imperv Runoff mm	Perv Runoff mm	Total Runoff mm	Total Runoff 10^6 ltr	Peak Runoff CMS	Runoff Coeff
S1	80.26	0.00	0.00	0.24	20.10	29.87	49.97	15.49	7.90	0.623
S3	80.26	0.00	0.00	0.24	20.22	34.12	54.34	9.78	5.07	0.677
S4	80.26	0.00	0.00	0.84	20.26	35.59	55.85	8.38	4.38	0.696
S5	80.26	0.00	0.00	0.84	20.39	43.78	64.17	6.42	3.51	0.800
S6	80.26	0.00	0.00	0.84	20.09	29.22	49.31	14.79	7.55	0.614
S7	80.26	0.00	0.00	0.84	20.08	29.09	49.18	17.21	8.78	0.613
S8	80.26	0.00	0.00	0.84	20.08	28.89	48.97	14.20	7.24	0.610
S9	80.26	0.00	0.00	0.84	19.94	25.41	45.35	9.98	5.05	0.565
S10	80.26	0.00	0.00	0.84	20.07	28.75	48.82	12.69	6.47	0.608
S11	80.26	0.00	0.00	0.84	20.22	33.73	53.95	12.95	6.72	0.672
S12	80.26	0.00	0.00	0.84	20.35	53.48	73.83	3.69	2.11	0.920
S13	80.26	0.00	0.00	0.84	20.18	32.42	52.60	13.68	7.06	0.655
S14	80.26	0.00	0.00	0.84	20.35	53.48	73.83	3.69	2.11	0.920
S15	80.26	0.00	0.00	0.84	20.12	30.06	50.18	13.55	6.93	0.625
S16	80.26	0.00	0.00	0.84	20.12	30.06	50.18	9.03	4.62	0.625
S17	80.26	0.00	0.00	0.84	20.35	53.48	73.83	3.69	2.11	0.920
S18	80.26	0.00	0.00	0.84	20.35	53.48	73.83	3.69	2.11	0.920
S19	80.26	0.00	0.00	0.84	20.03	27.54	47.57	13.32	6.77	0.593
S20	80.26	0.00	0.00	0.84	20.22	33.73	53.95	6.47	3.36	0.672
S21	80.26	0.00	0.00	0.24	20.07	29.09	49.16	6.39	3.25	0.613

3. Node flooding



4. Node flooding value

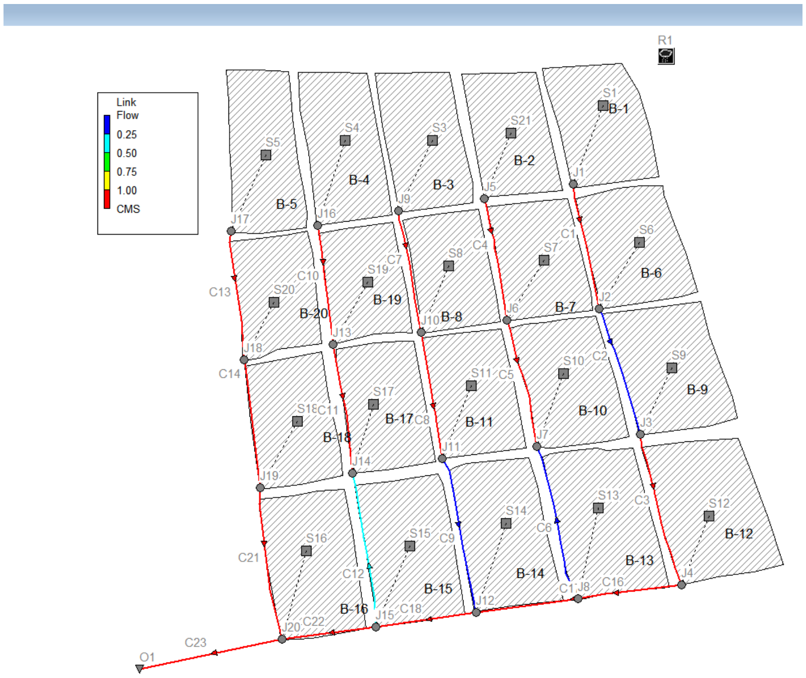


Topic: Node Flooding

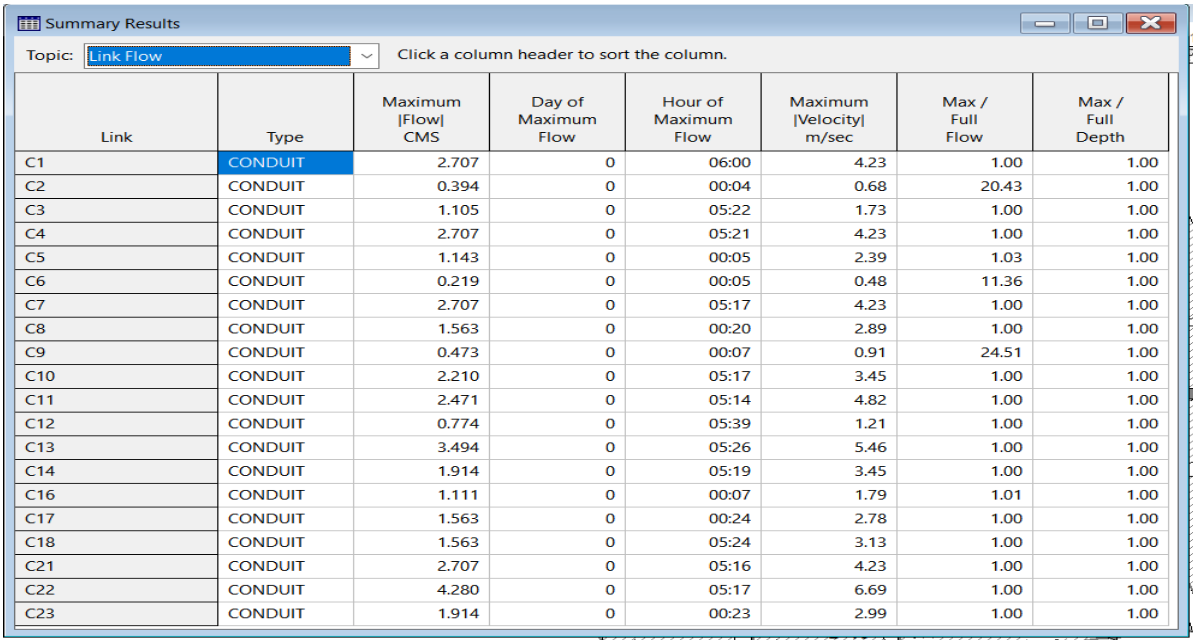
Click a column header to sort the column.

Node	Hours Flooded	Maximum Rate CMS	Day of Maximum Flooding	Hour of Maximum Flooding	Total Flood Volume 10^6 ltr	Maximum Ponded Depth Meters
J1	0.93	6.154	0	05:30	9.213	0.000
J2	5.96	11.189	0	05:30	61.301	0.000
J3	1.13	4.946	0	05:30	9.525	0.000
J4	5.93	3.104	0	05:30	24.795	0.000
J5	0.46	1.529	0	05:30	1.251	0.000
J6	5.95	11.340	0	05:30	40.945	0.000
J7	5.91	7.546	0	05:30	36.196	0.000
J8	5.89	7.561	0	05:30	24.587	0.000
J9	0.72	3.334	0	05:30	3.890	0.000
J10	5.94	9.348	0	05:30	28.858	0.000
J11	5.91	8.243	0	05:30	45.462	0.000
J12	5.84	2.108	0	05:30	5.164	0.000
J13	0.96	7.473	0	05:30	15.410	0.000
J14	5.84	5.349	0	05:30	56.640	0.000
J15	0.66	4.407	0	05:30	4.947	0.000
J16	0.83	3.150	0	05:30	4.065	0.000
J17	0.27	0.994	0	05:30	0.523	0.000
J18	5.94	5.923	0	05:30	15.251	0.000
J19	0.52	1.312	0	05:30	1.506	0.000
J20	5.88	9.671	0	05:30	61.095	0.000

5. Link flow



6. Link flow value



Topic: Link Flow

Click a column header to sort the column.

Link	Type	Maximum |Flow| CMS	Day of Maximum Flow	Hour of Maximum Flow	Maximum |Velocity| m/sec	Max / Full Flow	Max / Full Depth
C1	CONDUIT	2.707	0	06:00	4.23	1.00	1.00
C2	CONDUIT	0.394	0	00:04	0.68	20.43	1.00
C3	CONDUIT	1.105	0	05:22	1.73	1.00	1.00
C4	CONDUIT	2.707	0	05:21	4.23	1.00	1.00
C5	CONDUIT	1.143	0	00:05	2.39	1.03	1.00
C6	CONDUIT	0.219	0	00:05	0.48	11.36	1.00
C7	CONDUIT	2.707	0	05:17	4.23	1.00	1.00
C8	CONDUIT	1.563	0	00:20	2.89	1.00	1.00
C9	CONDUIT	0.473	0	00:07	0.91	24.51	1.00
C10	CONDUIT	2.210	0	05:17	3.45	1.00	1.00
C11	CONDUIT	2.471	0	05:14	4.82	1.00	1.00
C12	CONDUIT	0.774	0	05:39	1.21	1.00	1.00
C13	CONDUIT	3.494	0	05:26	5.46	1.00	1.00
C14	CONDUIT	1.914	0	05:19	3.45	1.00	1.00
C16	CONDUIT	1.111	0	00:07	1.79	1.01	1.00
C17	CONDUIT	1.563	0	00:24	2.78	1.00	1.00
C18	CONDUIT	1.563	0	05:24	3.13	1.00	1.00
C21	CONDUIT	2.707	0	05:16	4.23	1.00	1.00
C22	CONDUIT	4.280	0	05:17	6.69	1.00	1.00
C23	CONDUIT	1.914	0	00:23	2.99	1.00	1.00

Where .s1-Autobis Tera, S6- ZZZ Foreign Employment

S9-Dani Gym,s-12- A.A. revenue authority, mrkato n.2 bra.off, Addis Ketema, s3-Hara Pharmacy, s4-Spine Institute and Care Share Compan, s5-Yekatit 23 School/Mesalemia, s7- ZZZ Foreign Employment lower part , s8- Addis Ketema sub city young center,s10- Abedulkader Tesfaye Zenawi And Friends Loading Unloading Partnership, s11-Lower part of Abedulkader Tesfaye Zenawi And Friends Loading Unloading Partnership, 13- lower part of A.A. revenue authority, mrkato n.2 bra.off, S14- Lentebur Academy Share Company, Addis Ababa, S15-upper part shawel dama school sebategna, Congo St, Addis Ababa, S16-lower part shawel dama school sebategna, Congo St, Addis Ababa, S17-lower part of Kabas Private limited company,S18-Lower Part Spine Institute And Care Share Company, Addis Ababa,S19-Kabas Private limited company,S20 Spine Institute And Care Share Company, Addis Ababa, S21-Shwa robit.